Upgrading the Structural Capacity of a Steel Frame using Externally Bonded CFRP

Master’s Thesis in the International Master’s Program Structural Engineering

RICHARD BERGGREN
JOHN LENNBY

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2008
Master’s Thesis 2008:12
Upgrading the Structural Capacity of a Steel Frame using Externally Bonded CFRP

Master’s Thesis in the International Master’s Program Structural Engineering

RICHARD BERGGREN

JOHN LENNBY

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2008
Upgrading the Structural Capacity of a Steel Frame using Externally Bonded CFRP

Master’s Thesis in the International Master’s Program Structural Engineering
RICHARD BERGGREN & JOHN LENNBY

© RICHARD BERGGREN & JOHN LENNBY, 2008

Master’s Thesis 2008:
Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Illustrations of the analyzed frame and the stress-strain relationship for available CFRP laminates and ordinary steel.

Chalmers Repro Service / Department of Civil and Environmental Engineering
Göteborg, Sweden 2008
Upgrading the Structural Capacity of a Steel Frame using Externally Bonded CFRP
Master’s Thesis in the International Master’s Program Structural Engineering
RICHARD BERGGREN & JOHN LENNBY
Department of Civil and Environmental Engineering
Division of Structural Engineering
Chalmers University of Technology

ABSTRACT

Externally bonded Carbon Fiber Reinforced Polymer (CFRP) laminates can be an excellent alternative to reinforce degraded elements. The use of this relatively new technique in a proper way with suitable laminates and adhesive for the structure in matter can result in a considerable upgrade of the structural capacity.

Benefits of CFRP laminates are their superior mechanical and physical properties. They are also easy to handle and apply, which minimizes the time that the structure has to be out of use. This master thesis investigates how much the collapse load for an A-shaped steel frame can be increased, using externally bonded CFRP. The frame is serving as a pipe support at a nuclear power plant. A hold-up in the production in this kind of facility leads to severe economical consequences, which is why a quick and easy measure to strengthen is of great importance.

A final strengthening scheme of CFRP laminates, that increases the collapse load the most, was searched for. The collapse load is defined according to the American design code ASME III – Rules for Construction of Nuclear Power Plants Components. The frame with bonded CFRP was modeled in the finite element programs I-DEAS and ABAQUS, using continuum (solid) plane stress elements.

In the field of civil engineering, CFRP laminates are generally used for strengthening members in bending or tension. For this reason, this study had its original focus on cases where the laminates are bonded to the tension flanges. However, the analysis of the unreinforced frame showed that failure was a result of immense shear forces in the web. As a consequence to this, these web areas needed to be reinforced by adding steel plates. The use of steel plates was not part of the original objective, but due to the unexpected behavior of the frame, this measure was considered to be essential in order to justify the use of CFRP in further reinforcing measures. The results obtained from the analysis of the web reinforced frame confirmed that yielding started in the flanges, which motivated a strengthening scheme with CFRP.

Applying High-Strength laminates to this web reinforced frame increased the collapse load at the most with 24 %, from 3.52 MN for the unreinforced frame, to 4.63 MN. However, as this thesis was written there was neither any information about laminates subjected to compression, nor limits for allowable stresses in the adhesive layer. This is two essential factors that need to be verified before the success of this strengthening scheme can be confirmed.

Key words: CFRP, laminates, steel frame, FE analysis, capacity, collapse load, ASME III, shear force
Uppgradering av bärförmågan hos en stålram förstärkt med limmade kolfiberlaminat
Examensarbete inom Civilingenjörsprogrammet Väg- och vattenbyggnad
RICHARD BERGGREN & JOHN LENNBY
Institutionen för bygg- och miljöteknik
Avdelningen för konstruktionssteknik
Stål- och träbyggnad
Chalmers tekniska högskola

SAMMANFATTNING

Pållimmade kolfiberlaminat kan vara ett utmärkt alternativt till att förstärka degraderade element i en konstruktion. Används denna relativt nya teknik på rätt sätt, med rätt typ av laminat och lim för konstruktionen ifråga, kan resultatet bli en avsevärd uppgradering av bärförmågan.


En slutgiltig konstellation av laminat och lim, som höjer kollapslasten maximalt eftersöktes. Denna kollapslast var definierad enligt den amerikanska designnormen ASME III - Rules for Construction of Nuclear Power Plants Components. Den förstärkta ramen modellerades med ”plane stress” element i de finita element programmen I-DEAS och ABAQUS.

Inom byggnadskonstruktion används kolfiberlaminat normalt för att stärka element utsatta för böjande dragspänningar. Av denna anledning fokuserar denna studie på fall där laminaten är fästa på dragflänsarna. Dock visade analysen av den oförstärkta ramen att brott inträffar på grund av extremt höga tvärkraftar i livplåtarerna. Detta gör att dessa livplåtar måste förstärkas med extra stål. Förstärkning med hjälp av extra stål var inte en del av den ursprungliga förstärkningsmetoden, men på grund av ramens oväntade beteende ansågs detta vara avgörande för att rättfärda användandet av kolfiberlaminat i ytterligare förstärkningsåtgärder. Resultaten från analysen av den livförstärkta ramen bekräftade att det nu istället var stålet i flänsarna som började flyta, vilket motiverade en förstärkning med kolfiberlaminat.


Nyckelord: kolfiberlaminat, stålram, FE-analys, kapacitet, kollapslast, ASME III, tvärkraft
## Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>I</td>
</tr>
<tr>
<td>SAMMANFATTNING</td>
<td>II</td>
</tr>
<tr>
<td>CONTENTS</td>
<td>1</td>
</tr>
<tr>
<td>PREFACE</td>
<td>3</td>
</tr>
</tbody>
</table>

1 **INTRODUCTION**
   1.1 Background 4
   1.2 Problem description 4
   1.3 Aim and scope 5
   1.4 Method 5
   1.5 Limitations and assumptions 6

2 **LITERATURE REVIEW**
   2.1 Fiber Reinforced Polymers 7
   2.2 Structure and contents of CFRP laminates 7
   2.3 Adhesives 11
      2.3.1 Influence of temperature 11
      2.3.2 Influence of curing time 13
   2.4 Behavior of steel-CFRP joint and possible failure modes 14
   2.5 Strengthened steel beams 15
   2.6 Installation procedure of a CFRP strengthening system 17
   2.7 Case study 18

3 **MODELING PROCEDURE** 20
   3.1 Criterion of collapse load 20
   3.2 Choice of laminate 21
   3.3 Material properties 23
      3.3.1 CFRP-laminate 23
      3.3.2 Adhesive 23
      3.3.3 Properties of the frame 24

4 **PRELIMINARY ANALYSIS** 27
   4.1 Unreinforced frame 27
   4.2 Assessment of the strengthening effect in the elastic phase 30
   4.3 Conclusions from the preliminary analysis 31

5 **ANALYSIS OF THE UNREINFORCED FRAME** 32
   5.1 Verifying the model 32

**CHALMERS, Civil and Environmental Engineering, Master’s Thesis 2008:12**
5.2 Unreinforced frame
  5.2.1 Conclusions

6 NEW DESIGN CONSIDERATIONS
  6.1 Web reinforcement
  6.2 Flange reinforcement
  6.3 Evaluation

7 ANALYSIS OF CFRP STRENGTHENED FRAME
  7.1 Strengthening scheme A
    7.1.1 Strengthening scheme A, UHM laminates
    7.1.2 Strengthening scheme A, HS laminates
  7.2 Strengthening scheme B
    7.2.1 Strengthening scheme B, HS laminates
  7.3 Evaluation
  7.4 Strengthening scheme C
    7.4.1 Strengthening scheme C, HS laminates
    7.4.2 Strengthening scheme C, combination of HS & UHM
  7.5 Evaluation
  7.6 Results

8 EVALUATION OF THE FINAL STRENGTHENING SCHEME
  8.1 Laminates subjected to compression
  8.2 Interfacial stresses in the adhesive layer
    8.2.1 Interfacial stresses at the cross bracing strut
    8.2.2 Interfacial stresses at area of load application
    8.2.3 Interfacial stresses at support
  8.3 Evaluation
  8.4 Results

9 SUMMARY AND CONCLUSIONS

10 REFERENCES

Preface

This Master’s thesis was carried out at Epsilon Hightech AB, Sweden, and at the Division of Structural Engineering, Department of Structural Engineering, Steel Structures, Chalmers University of Technology, Sweden. The working period for this project lasted from September 2007 to March 2008.

The frame support studied in this thesis exists, and is as this is written, still in use. It is situated at unit 3 at Ringhals nuclear power plant, 60 kilometers south of Göteborg, Sweden.

We would like to express our utmost gratitude to Dr. Mohammad Al-Emrani, the supervisor and examiner of this project. This is especially for his valuable advices, commitment and support, but also for his friendliness, openness and great sense of humor.

Special thanks to the supervisors at Epsilon Hightech, Per Löfqvist, Manager Nuclear Engineering and M.Sc. Arshad Abosh for their help, support and supervision during this project. Per Löfqvist together with his colleagues at Ringhals came up with the idea for this study, with out them this thesis could never have been accomplished.

We also like to extend our thanks to the PhD students Reza Haghani Dogaheh and Dag Linghoff for helping us with the modeling procedure. Their ideas and guidance with the software ABAQUS and I-DEAS were really appreciated.

Our gratitude goes to our opponent members Henrik Andersson and Dmitry Vysochinskiy for their help and feedback throughout the project.

Last but not least, best regards and gratitude to our families.

Göteborg, March 2008

Richard Berggren

John Lennby
1 Introduction

1.1 Background

Measures to strength and repair existing steel structures are a big concern for designers in order to extend the lifetime of structural elements or to meet upgrading requirements. As a substitute for replacing degraded elements or adding new ones to a structure, the use of externally bonded Carbon Fiber Reinforced Polymer (CFRP) laminates is a powerful but yet a relatively new technique in civil engineering. The success of this technique is to a great extent dependent on the adhesive layer, which is used to bond the laminates to a substrate.

One benefit of CFRP is the outstanding mechanical and physical properties in comparison to other conventional construction materials. It is distinguished by its low weight to tensile strength ratio as well as its high stiffness. Traditionally strengthening and repair of steel structures usually involves either welding or bolting of additional steel plates to the structure. The shortcoming of this method is decreased fatigue life, in terms of cracks and high stress concentrations in joints, as well as an increased self-weight. Furthermore, welding is labor expensive and not always possible for old steel qualities. All these problems could be eliminated by adhesive-bonded laminates.

A strengthening scheme using externally bonded CFRP is particularly motivated where access is limited and disruption costs are high. This due to the easy handling and application of the laminates, which minimize the time a structure cannot be utilized for its intended purpose. An example of where a hold-up in the production leads to severe economical consequences is at nuclear power plants. In countries possessing nuclear power, these facilities are often a main producer of the total electrical power consumption. A fast and easy procedure to strengthen and repair components in the plants is therefore of great importance.

Since this strengthening technique is relatively new, the research within this field is limited. Up to now most of the research has been concentrated on structures with a simple geometry, for instance beams or bridges with one span where the CFRP is applied at the tension flange. Here the short term strengthening effect for axial and flexural tension is well documented, but when a more complex structure is introduced more uncertainties will arise. There is for example poor knowledge in how to make a proper selection of CFRP and adhesives to best suit the intended application.

1.2 Problem description

This thesis work will investigate the possibility to strengthen a steel frame with the use of externally bonded CFRP laminates. The frame, shaped as an A, is serving as a support for a steam pipe at a nuclear power plant. This frame will carry a case in which the steam pipe is situated. The case is fastened to the frame by four pairs of U-rods. If a break in the steam bore occurs, the pipe will hit the case and the force is transmitted via the U-rods to the frame. The rods are allowed to plastify and by this, the energy of the pipe is absorbed.
New norms in the nuclear industry demand an increased design flow and design temperature in the steam pipes. As a consequence to this, the pipe support needs to be designed for a higher impact load in case of a pipe break. The capacity of the frame is restricted to the deflection. An increased load results in higher deflection, which explains the need for strengthening.

1.3 Aim and scope

The aim of this study is to analyze the global behavior of the strengthened frame support in case of a pipe break. Suitable characteristics of the CFRP laminates are searched for and a final constellation of these, which in an adequate way results in the largest increase of capacity, will be presented. The capacity of the energy absorbing U-rods is beyond the scope of this thesis work.

From this study, the following questions are to be answered:

- How much can the failure load be increased by strengthening with CFRP?
- Where and to which extent should the reinforcement be applied?
- Which characteristics of the CFRP laminates are desirable for this strengthening scheme?
- What is the location and magnitude of the interfacial stresses in the adhesive layer? Will these stresses be critical for the load-carrying capacity of the system?

1.4 Method

An unreinforced model of the support is first created in order to determine the present capacity. This model is subsequently refined by applying different reinforcement configurations of CFRP laminates. Reinforcing the frame will result in a new stress profile, which will also influence the way it deforms. The stress profile will to a large extent depend on the characteristics of the CFRP and the adhesive. Since there is a large variety in these characteristics, literature studies has been carried out in order to gain understanding of the behavior of the components. With this knowledge, features of the components found most suitable for upgrading the frame are used in the analysis of the strengthening scheme.

Results for analysis are derived, using the Finite Element Method. The model is created in the commercial FE software I-DEAS version 9 and ABAQUS 6.5.1. The latter software also performs the calculations, based on both linear and non-linear material behavior.
1.5 Limitations and assumptions

The support is designed according to the American design code ASME III – Rules for Construction of Nuclear Power Plants Components. This code does not comprise the use of CFRP, for this reason, the capacity of the reinforced frame will be determined based on the criterions applied for steel structures.

In a loaded structure, strengthened with externally bonded CFRP, interfacial stresses will arise in the adhesive layer. This may cause debonding or interlaminar failure in the laminate. At the time this thesis is written, there are no regulations or guidelines for the design of adhesively bonded CFRP laminates on steel members. Consequently, it is not well established when failure will occur, or of what mode. This thesis work will assume that no failure will occur in the adhesive layer. However, the magnitude of the interfacial stresses can be obtained. These stresses will be compared to the maximum tensile strength of the adhesive and a rough estimation of how critical these stresses are can be established. If the interfacial stresses are high, measures to cope with this may be necessary. This is however not a part of the study.

In order to decide how much the structural capacity of the frame has been upgraded, collapse loads for both the unstrengthened and the strengthened frames will be estimated and later compared. According to the regulations stated in ASME III, these collapse loads should be obtained from displacement history curves in the point with the largest displacement. This will be done graphically and by hand calculations and could therefore give rise to a small margin for errors. This margin is however so small that it is considered not to play a significant part for the results.

This thesis work has its focus on strengthening due to upgrading requirements of a structure. All materials are assumed to be ideal with no geometrical imperfections, corrosion, cracks, air bubbles etc. No long-term effects are taken into consideration. The influence of creep and fatigue will therefore be excluded.

The frame is situated in indoor climate. Hence there is no exposure to extreme temperatures, to which the adhesive in particular is sensitive.

The strengthening scheme assumes a regular installation technique with prefabricated laminates. To date, there are several methods known in how to optimize the high strength of the CFRP. For example pre-stressing of the laminates, mechanical anchorages, tapered ends of the laminates, etc. However, these techniques are not discussed in this study.
2 Literature Review

The literature study for this thesis work has been focused on previous research made on the behavior and characteristics of CFRP-Metallic composites, and the materials involved in a strengthening scheme.

2.1 Fiber Reinforced Polymers

A Fiber Reinforced Polymer (FRP) is a composite material comprising a polymer matrix reinforced with fibers. FRP products were introduced for engineers in the 1950’s, preliminary as a way to strengthen concrete structures. Still under technical development, little was known about these very costly products. During the next two decades, the quality improved significantly and automated manufacturing methods enabled the material costs to decrease. In the 1980’s, the use of these products for external reinforcement of concrete bridge structures started [9]. Since then, they have successfully and increasingly been used for strengthening and repair of existing concrete, masonry and lately also timber and steel structures.

Commonly used fibers in FRP laminates are E-glass, carbon or aramid. The properties of these fibers can have a great variation. Nevertheless, they are all, in varying extent, excellent alternatives for the repair and retrofit of structures due to their high tensile strength and high Young’s modulus.

In this study, laminates reinforced with carbon fibers, CFRP, will be used. This is motivated since carbon fibers are available with a higher strength and a higher Young’s modulus than both E-glass fibers and aramid fibers. This is considered to be the most essential features for the laminates when upgrading the capacity of the frame.

2.2 Structure and contents of CFRP laminates

Strengthening with CFRP is a method that could be used on almost any metallic structure. Bridges, metal-framed buildings, pipes, tanks and vessels are all examples of structures that have been strengthened in this manner [4]. In general, the member’s flexural, shear, or axial strength is increased or better mobilized by the external application of high tensile strength material. Carbon Fiber Reinforced Polymer consists of carbon fibers, resin, fillers and additives.

Carbon fibers have a typically diameter of 8 μm and they can be either continuous or discontinuous. Examples of this are illustrated in Figure 2.1. The mechanical properties of the CFRP are highly dependent on the magnitude and orientation of these fibers. To date, carbon fibers are produced using the following three types of raw materials or precursors: polyacrylonitrile (PAN), pitch and rayon (C₆H₁₀O₅)ₙ. PAN and rayon fibers are made out of organic precursor fibers and pitch fibers are produced from gas growth [5] (pitch is a by-product of petroleum refining or coal coking). [11]
Figure 2.1 Illustration of composites reinforced by (a) continuous and aligned fibers, (b) discontinuous and aligned fibers, (c) discontinuous and randomly oriented fibers [13]

Carbon fibers are classified in different grades; High-Strength (HS), High-Modulus (HM) and Ultra-High-Modulus (UHM). The properties of the different reinforcing fibers are given in Table 2.1. An increase in stiffness is usually related with a reduction in strength and strain to failure of the fiber. UHM fibers can be very brittle, which calls for extra caution in design.

Table 2.1 Properties of the carbon fibers [2]

<table>
<thead>
<tr>
<th>Carbon Fiber</th>
<th>High-Strength (HS)*</th>
<th>High-Modulus (HM)*</th>
<th>Ultra-High-Modulus (UHM)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity (GPa)</td>
<td>230–240</td>
<td>295–390</td>
<td>440–640</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>4300–4900</td>
<td>2740–5940</td>
<td>2600–4020</td>
</tr>
<tr>
<td>Strain to failure</td>
<td>1.9–2.1</td>
<td>0.7–1.9</td>
<td>0.4–0.8</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>1800</td>
<td>1730–1810</td>
<td>1910–2120</td>
</tr>
<tr>
<td>Coefficient of thermal expansion (parallel to fiber), (10$^{-6}$/°C)</td>
<td>1.9–2.1</td>
<td>0.7–1.9</td>
<td>0.4–0.8</td>
</tr>
</tbody>
</table>

* PAN precursor  
**Pitch precursor
Carbon fibers are built up by sheets of hexagonally arranged carbon atoms. The atoms are kept together through strong covalent bonding, which explains the fibers superb in-plane strength and stiffness. On the other hand, weak van der Waals forces act between the sheets, giving the fiber its soft and brittle characteristics. These poor chemical bonds result in low modulus and tensile strength perpendicular to the fibers. Hence CFRP is an anisotropic material [5].

Long continuous fibers, oriented in a single direction are often used in high performance composites. This generates the great strength and stiffness parallel to the fibers in a laminate. In order to reduce the weakness in the transverse direction of these composites, layers with different fiber orientation can be added into the laminates, see Figure 2.2.

![Image of laminate with fiber layers oriented in longitudinal direction only, and fiber layers oriented in both longitudinal and transverse direction respectively [14]](image)

**Figure 2.2** Laminate with fiber layers oriented in longitudinal direction only, and fiber layers oriented in both longitudinal and transverse direction respectively [14]

Resins are constituents that provide high compressive strength and bind the fibers into a firm matrix, see Figure 2.3. They are divided into two classes, thermoplastics and thermosets. A thermoplastic resin does not cure permanently when it is exposed to elevated temperatures. This behavior makes it undesirable for structural application. On the other hand, a thermosetting resin will cure permanently even when it is heated, which makes it very attractive for structural applications. The most frequent resins used in composite industry today are epoxies, unsaturated polyesters, phenolics and polyurethanes [8]. Properties for these thermoset resins are given in Table 2.2.
Fillers are added to the resin matrix not only to improve its mechanical and chemical properties, but also because it is less expensive than resins. Using simply resins to fill up voids in the composite matrix would not be cost effective. Furthermore, adding one or several appropriate fillers could also generate an increased fire and chemical resistance, and low shrinkage. There are three major sorts of fillers; calcium carbonate, kaolin and alumina trihydrate.

The final constituent, additives, is used for a better material property, workability and aesthetics for the composite system. Although these materials are generally used in relatively low quantity by weight compared to the other constituents, they perform critical functions [9].

**Fillers** are added to the resin matrix not only to improve its mechanical and chemical properties, but also because it is less expensive than resins. Using simply resins to fill up voids in the composite matrix would not be cost effective. Furthermore, adding one or several appropriate fillers could also generate an increased fire and chemical resistance, and low shrinkage. There are three major sorts of fillers; calcium carbonate, kaolin and alumina trihydrate.

The final constituent, **additives**, is used for a better material property, workability and aesthetics for the composite system. Although these materials are generally used in relatively low quantity by weight compared to the other constituents, they perform critical functions [9].
2.3 Adhesives

When strengthening an existing structure with CFRP laminates, a structural adhesive has to be applied in order to bond the components. In civil engineering the adhesive is usually applied as a paste. Film adhesives are also possible but require a more careful surface preparation since the film does not fill any imperfections in the substrate. If the strengthening scheme is formed in-situ, the matrix resin will also bond the adherents.

The same type of resins that form the matrix can be used in the adhesive paste. Typical properties of different resins are showed in the table 2.2. Most commonly used are the two component epoxies, consisting of resin and hardener. One of its most attractive characteristics compared to many other civil engineering polymers is its toughness. Epoxies can provide good adhesion to many substrates and experience moderate shrinkage during polymerization [2].

2.3.1 Influence of temperature

The mechanical properties of a polymer and consequently of both the composite matrix and the adhesive, are temperature-dependent. Figure 2.4 illustrates the strength development for a two-part epoxy at various temperatures. As the figure shows, thermal exposure may be an advantage. The explanation to this is that up to a certain temperature, the FRP composite and the adhesive may post-cure. Generally, the time required for an adhesive to cure halves for each 10°C rise in the temperature of the epoxy resin. Conversely, the curing stops altogether below about 5 °C.

![Figure 2.4 Effect of formulation and cure temperature on the development of flexural strength of a two-part epoxy adhesive [2]](image)

Thermosetting polymers for use within civil engineering are glassy in nature. As the temperature increases, the polymer first approaches its glass transition temperature (Tg), and begins to soften. Figure 2.5 shows this phenomenon for an ambient-cure
epoxy. Furthermore, at elevated temperatures, all polymers will decompose. The \( T_g \) of a polymer is dependent upon the detailed chemical structure.

Figure 2.5  
Variation in epoxy adhesive shear strength with temperature [2]

Polymers harden and become increasingly brittle with decreasing temperature. Their fracture toughness and critical energy release rate are significantly reduced as a result. These are critical properties for adhesion strength, but is not usually a problem until the temperature drops to around -20°C, and then only if the polymer is exposed to this temperature for a long time. If an FRP strengthening scheme is expected to operate at low temperatures for long durations, the material properties must be determined from tests carried out at a representative temperature. This due to that each polymer has unique behavior at low temperatures [2].

Figure 2.6 shows the variation in mechanical properties of an ambient cure two-part epoxy adhesive with temperature. It also illustrates the average shear stress in the test specimen at failure, the peak stress, plus the shear fracture toughness of the adhesive.

Figure 2.6  
An indication of the variation in the properties of a two-part ambient cure epoxy with temperature [2]
2.3.2 Influence of curing time

As well as the influences of temperature, different adhesives will be influenced by curing time in varying degrees. An indication of the variation in characteristics with curing time can be seen in Figure 2.7. Tests performed on an ambient epoxy adhesive carried out in room temperature, around 22°C, shows that the ultimate tensile strength was increasing with time and the final value of about 30 MPa was reached after 3 days. Young’s modulus was also increasing and after 5 days of curing a value of approximately 12.8 GPa was attained. The ultimate strain is decreasing with time up to 5 days [1].

![Figure 2.7](image-url)

Figure 2.7 Variation of (a) ultimate tensile strength, (b) Young’s modulus and (c) ultimate strain with curing time for an ambient epoxy [1]
2.4 Behavior of steel-CFRP joint and possible failure modes

The quality of a strengthening scheme using CFRP is in large extent depending of the performance in the steel-CFRP joint and the effectiveness of the adhesive. When a strengthened element is loaded, the two adherents will carry a portion of the load according to their relative stiffness. Stiffer laminates will attract a larger portion of the load. A difference in stiffness will subsequently result in interfacial shear stresses to counter the difference in deformation. These shear stresses have been showed to act within the first 100 mm from the end of the bond line, as most of the load is transferred from the steel substrate to the CFRP laminate within this length [1]. Due to the eccentricity in the load transfer, normal stresses (or peeling stresses) perpendicular to the plane are also acting at the ends of the laminates, see Figure 2.8 (a). The shear and peeling stresses are prone to cause debonding failure or interlaminar failure at the ends of the laminate. Debonding failures can arise either in the interface of steel-adhesive or CFRP–adhesive. In a similar manner interlaminar failure can occur in either the adhesive layer or in the CFRP laminate.

If loading is allowed to increase, shear stresses will also arise in areas where the steel is yielding. Additional load cannot be carried by the steel anymore and has to be transferred to the CFRP laminate. As the steel is yielding, large differences in strain will occur since the laminate is still linear elastic. These strain differences have to be accommodated by the adhesive, which might crack or debond from the adherents. If the adhesive is sufficiently flexible and strong enough to cope with the interfacial shear stresses, the final failure mode will be rupture of the CFRP laminate when its ultimate strength is reached. The interfacial stresses before and after yielding of the steel beam is illustrated in Figure 2.8 (b).

![Figure 2.8 Schematic illustration of the principal load effects in a CFRP retrofitted steel beam [1]](image-url)
2.5 Strengthened steel beams

Tests on steel beams with a double symmetric cross section has shown that in the ultimate limit state, an increase in bending strength up to about 18% is possible when CFRP laminates are bonded to the outer side of the tension flange [1]. This amount of increased capacity is obtained when laminates with high strength and a Young’s modulus equivalent to steel is used. Table 2.3 shows how much the collapse load for four HEA180-beams reinforced with CFRP laminates in different constellations have been increased. The biggest increase (18%) was obtained when HS-laminates was attached on both sides of the tension flange.

Using CFRP laminates with Young’s modulus close to steel also results in an overall ductile behavior, if a sufficiently good bond is assured. Stiffer laminates are suitable in order to delay yielding in the steel member. These laminates has on the other hand lower tensile strength and due to their stiffness, high concentrations of shear stresses will appear at the ends of the laminates, which may result in sudden and premature interlaminar shear failure. If the top flange of a beam is prevented from yielding in compression, studies have shown that an increase in strength up to 75% is possible when strengthening with CFRP [3].

Table 2.3 Difference in ultimate load and the corresponding deflection between beams strengthened with CFRP laminates and a reference beam [1]

<table>
<thead>
<tr>
<th>Beam</th>
<th>CFRP laminate</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f_t [MPa]</td>
<td>E [GPa]</td>
<td>Area [mm²]</td>
<td>ΔP (load) [%]</td>
<td>Δd (deflection) [%]</td>
<td></td>
</tr>
<tr>
<td>Reference</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3100</td>
<td>165</td>
<td>288</td>
<td>2</td>
<td>-56</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3300</td>
<td>200</td>
<td>192</td>
<td>17</td>
<td>-34</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3300</td>
<td>200</td>
<td>288</td>
<td>18</td>
<td>-12</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1500</td>
<td>450</td>
<td>170</td>
<td>2</td>
<td>-56</td>
<td></td>
</tr>
</tbody>
</table>

Beams strengthened with CFRP also show a remarkable improvement in fatigue life. Compared to an unreinforced beam, experimental investigations concluded that strengthened beams experienced between 2.6 to 3.4 times longer fatigue life for stress ranges of 345 to 207 MPa respectively [7]. Bonded CFRP does not eliminate crack growth but it substantially slows it down. Figure 2.9 shows the crack propagation after a crack is initiated for an unretrofitted and retrofitted steel beam respectively.
Figure 2.9  Change in crack length for (a) unretrofitted beams and (b) retrofitted beams during fatigue tests [7]
2.6 Installation procedure of a CFRP strengthening system

Strengthening a metallic structure by adding externally bonded CFRP, is a method that involves a few, yet important working steps. The first step is to make sure that the adhesive bond properly and stay durable. In order to achieve this, surface treatment and accurate bonding is essential. Inappropriate surface preparation can lead to premature debonding. To avoid this, the surface can be either sandblasted or treated with a method called the SACO method. Both methods regulate the roughness and unevenness of the surface, making it well susceptible for the adhesive layer. However, the aim of the SACO method is not only to clean (SAndblasting) the surface, but also to give it a sooth COating. The particles used in the SACO method have a certain primer coating that is transferred to the steel surface when it is impacted. The result is a clean and sooth primer coating, ready for the adhesive to be attached.

If not the SACO but the ordinary sandblasting method is used, it is necessary to optimize the adherence between the steel and the adhesive even more. This is done by applying a primer in the traditional way, using a paintbrush. This will also prevent any corrosion of the substrate. Making sure that the CFRP laminate is dirt free, cleaning with acetone before bonding is well recommended. Some producers have protected the surface with a special protection layer (e.g. peel ply), which shall be removed before application. Hence, these surfaces do not need to be cleaned [1].

The next step is to prepare the adhesive. In the case where a two-component epoxy adhesive is used, it is especially important to follow the instructions for the mixing of the two parts, to achieve a successful result. These instructions together with the so-called potlife time should be provided by the supplier. The potlife starts when the two components of the epoxy are mixed together [10]. After the adhesive is properly prepared and the primer has cured, the adhesive can be applied. The best way to reach a high homogeneity of the adhesive layer without any air pockets is to apply the adhesive on one side (preferentially on the CFRP side) and in a triangular shape (a triangular cross-section).

The final step is to assemble the CFRP and the steel member. Once the adhesive has been applied it is important to place the CFRP laminate during the potlife time of the adhesive. This time is affected by both the temperature and the amount of the mix. Since this step also has to be carried out with great care to avoid any contamination or air pockets, the CFRP should be attached in one working step. Any detaching or replacing should be avoided [1]. Figure 2.10 shows the different working steps of the installation procedure of a CFRP strengthening system onto an existing steel girder bridge.
Figure 2.10 Different working steps of the installation procedure of a CFRP strengthening system. Figure (a) shows cleaning of the metal surface, (b) application of the primer, (c) cleaning of the CFRP laminates, (d) preparation of the adhesive, (e) application of the adhesive and finally figure (f) the assembling of the parts [1]

2.7 Case study

In general, there is very little information concerning field experiences of strengthening metallic structures with CFRP laminates and its achieved affects. One example of where CFRP laminates has been used is on a 45.7 m long three span continuous I-beam bridge. The two edge spans with the length of about 13.9 m, and the centre span with the length of 18 m, is supported by six beams of different dimensions, see Figure 2.11. The main goal of this strengthening was to increase the capacity of overstressed beams by attaching CFRP laminates to the tension flanges.

The CFRP laminates used, consisted of continuous unidirectional carbon fibers. Performed calculations had shown that the overstressed beams could be properly strengthened by the use of CFRP laminates bonded to the bottom (tension) flange of the beams. The laminates were applied in the areas of positive moment of all three spans, on both interior and exterior beams. For each span a different amount of laminates were attached. This was done to investigate the effect of different constellations, to compare the response of the different scheme used in different conditions, to investigate the ease of construction of multiple layers, and also to evaluate the durability of the installation. In order to investigate the performance and in-service durability under detrimental environmental conditions, one beam had half of the laminates installed on the bottom of the tension flange and half applied on the top of the bottom flange.
Unfortunately, the results of this strengthening procedure are not reported yet. Therefore, the conclusions on performance and behavior of this bridge cannot be presented. [1]

Figure 2.11 Side view of the strengthened bridge (a). Figure (b) shows a CFRP laminate attached on the top of the tension flange [1]
3 Modeling procedure

In the early stage of the analysis, an overall behavior of the frame is searched for as well as the most critical load case with respect to the deflection of the frame. For this purpose, beam elements are chosen to model the frame in the software ABAQUS. The linear response from CFRP strengthening was also evaluated in this simplified model.

Further investigations of the strengthening scheme require a more accurate model. A new model is created in the software I-DEAS, using continuum plane stress elements where the out-of-plane stress, $\sigma_{33}$, is assumed to be zero. This model is analyzed in the non-linear response by successively increasing the load. The collapse load can be determined by the displacement history in a certain node. Apart from determining the current collapse load, an analysis of the unstrengthened frame is supposed to detect spots where yielding first occur and consequently where the strengthening should be focused.

The model is subsequently refined by applying different reinforcement configurations of CFRP laminates in critical regions and an evaluation of how much the collapse load has increased is carried out. When an efficient strengthening configuration is found, the interfacial stresses are also assessed. It is however not possible to predict if these stresses will cause failure in the CFRP-steel joint. This study will establish if the stresses are reasonably low or if they are critical for the load bearing capacity of the strengthened frame and if extra measures are needed to be taken into concern.

3.1 Criterion of collapse load

To meet the requirements of the current American design code for components in nuclear power plants, ASME III, it is essential to evaluate both the linear and non-linear response of the loaded frame. The goal of this study is to investigate how much the load, causing the frame to collapse, can be increased by CFRP strengthening. In ASME III, appendix II, part II-1430 (b), the criterion of collapse load is stated as:

“The least square fit (regression) line as determined from the data in the linear elastic range is drawn on each plot considered. The angle that the regression line makes with the ordinate is called $\theta$. A second straight line, hereafter called the collapse limit line, is drawn through the intersection of the regression line with the abscissa so that it makes an angle $\Phi = \tan^{-1}(2\tan \theta)$ with the ordinate.” (See Figure 3.1)

“The test collapse load is determined from the maximum principal strain or deflection value at the first data point for which there are three successive data points that lie outside of the collapse limit line. This first data point is called the collapse load point. The test collapse load is taken as the load on the collapse limit line which has the maximum principal strain or deflection of the collapse load point.”

This criterion is decisive for the capacity of the frame in any arbitrary location point. It should be noted that local plasticization with high strain concentrations is not considered as a failure of the whole frame. In this study, the collapse load will therefore be decided by looking at the displacement history in the point with the largest displacement.
3.2 Choice of laminate

The aim of this strengthening scheme is to increase the collapse load as much as possible. But, since the collapse load is to a large extent depending on the global deflection, it is not obvious how the largest increase in capacity is achieved. Earlier research has showed that the largest increase of ultimate load is attained from laminates with high tensile strength and Young’s modulus similar to steel. On the other hand, the laminates with high Young’s modulus are preferable in order to minimize the elastic deflection. With this in mind, two different approaches can be distinguished in order to find the best strengthening effect of the frame.

One approach is to use UHM laminates. This scheme will decrease the deflection since yielding in the steel members is delayed. If a large amount (thick laminates) is used, the frame will also be stiffer and the elastic deflection will decrease. These laminates have however two weaknesses compared to laminates with lower stiffness. Laminates with high Young’s modulus have lower ultimate strain at failure. Furthermore, these laminates will put higher demands on the adhesive layer since the interfacial stresses increases and could be critical for the whole system.
The second approach is to use HS laminates with a Young’s modulus equivalent to steel. In the elastic phase, this scheme has very little effect. However, when the steel starts to yield, the high tensile strength in the laminates can be utilized and can cope with higher tensile stresses than the UHM laminates. The largest increase of the collapse load would be gained if the frame is allowed large plasticization and by that utilize the full strength of the HS laminates. But since it is the angle between the elastic and the plastic displacement that determines the collapse load, high strength laminates might not come to their full use if yielding is restricted.

Figure 3.2 shows an illustrative graph of the stress-strain behavior of steel and CFRP in tension. It can be seen that more ductile CFRP laminates have higher strength and consequently lower strength in the stiffer laminates.

Both the UHM- and the HS laminates that will be evaluated have unidirectional fibers aligned in the longitudinal direction in order to best utilize the essential characteristics. An adequate placement and the length required to achieve good strengthening effect can be decided from simulations in ABAQUS. In civil engineering, CFRP laminates are mainly used to strengthen members in bending or in tension. It is expected that bending moments will be critical for the frame and that the reinforcement can be applied on the tension flange where yielding starts, as this is where the qualities of CFRP are most useful. Each laminate (HS and UHM) are available on today’s market. Properties of these are presented in Table 3.1, chapter 3.3.1.

Figure 3.2  Stress-strain graph for steel and CFRP with various characteristics.
3.3 Material properties

The material models included in this analysis are idealized for simplicity in the modeling. No geometrical or other imperfections are considered. The material data for the laminate and the adhesive were obtained from material manufacturers.

3.3.1 CFRP-laminate

The laminates used in the analysis are pultruded CFRP laminates, designed for externally bonding on steel, concrete, timber and masonry structures. The cross-sectional dimension is set to 100 x 6 mm². The number of strips and their lengths are regulated according to available space in order to provide a sufficient strengthening effect.

Table 3.1 Properties of the CFRP-laminate

<table>
<thead>
<tr>
<th>Properties of CFRP-laminate</th>
<th>High-Strength (HS)</th>
<th>Ultra-High-Modulus (UHM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (mean value in GPa)</td>
<td>210</td>
<td>600</td>
</tr>
<tr>
<td>Tensile strength (mean value in GPa)</td>
<td>3.2</td>
<td>1.1</td>
</tr>
<tr>
<td>Compression strength</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

3.3.2 Adhesive

The adhesive used for the analysis is a 2-component solvent free epoxy resin. Since factors like the effect of temperature and curing time are not taken into consideration, the most important property for the adhesive is its stiffness and strength. Therefore, a strong adhesive with a high tensile strength was chosen. The thickness of the adhesive layer is set to 2 mm. Other essential properties can be seen in Table 3.2.
Table 3.2 Properties of the adhesive

<table>
<thead>
<tr>
<th>Properties of adhesive</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
</tr>
<tr>
<td>(mean value in GPa)</td>
</tr>
<tr>
<td>Tensile strength</td>
</tr>
<tr>
<td>(mean value in MPa)*</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
</tr>
</tbody>
</table>

* 14 days curing time at +15°C

3.3.3 Properties of the frame

The approximately 6 meters high steel frame consists of I-beams and rectangular hollow core sections, forming the shape of an A, see Figure 3.3. To prevent local buckling, web stiffeners are positioned where the U-rods are fixed to the frame, since this is where the force from a possible pipe break is transmitted.

Figure 3.3 Illustration of the frame with the steam pipe in the middle, and the geometry of the five belonging cross-sections
In the bottom of the frame, 500 mm high vertical stiffeners are welded to the columns in all directions. These stiffeners are considered stiff enough to prevent any deflection in this region. In the FE model, boundary conditions are therefore applied at the level of the stiffeners’ top ends, with all degrees of freedom locked, see Figure 3.4.

![Applied boundary condition](image)

**Figure 3.4** (a) illustrates the support seen from the front plus the location of the boundary condition. Figure (b) illustrates the support seen from above.

All steel in the frame has the quality of S275JR. A stress-strain relationship for this steel, obtained from tensile tests, is illustrated in Figure 3.5. In order to describe the behavior of the steel, the values in Table 3.3 are used in the modeling procedure. Other properties that are of significance for this steel can be seen in Table 3.4.

![Stress-strain relations](image)

**Figure 3.5** Stress-strain relations for the steel in the frame
Table 3.3  Stress and strain values for the steel in the frame

<table>
<thead>
<tr>
<th>Stresses [MPa]</th>
<th>Total strain [%]</th>
<th>Plastic strain [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>279.3906</td>
<td>0.001399</td>
<td>0</td>
</tr>
<tr>
<td>303.6072</td>
<td>0.025375</td>
<td>0.023930</td>
</tr>
<tr>
<td>381.0785</td>
<td>0.059683</td>
<td>0.057868</td>
</tr>
<tr>
<td>392.0400</td>
<td>0.076961</td>
<td>0.075094</td>
</tr>
</tbody>
</table>

Table 3.4  Properties of the steel in the frame

<table>
<thead>
<tr>
<th>Properties of steel</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
<td>210</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>392</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>
4 Preliminary analysis

In a preliminary analysis of the linear response of the frame, a simplified model was created using beam elements. The main advantage of beam elements is that they are geometrically simple and have few degrees of freedom. This means that the use of beam elements is computational time saving and also easy to manage in the modeling procedure.

Beam theory is a one-dimensional approximation of a three-dimensional continuum and can be used when the length is significantly greater than the other two dimensions. In ABAQUS a beam element is a one-dimensional line element that has a stiffness associated with the deformation of the beam's axis. The elements assume that plane sections perpendicular to the axis of the beam remain plane during deformation [12].

The beam elements in the frame are modeled according to the cross section profiles showed in Figure 3.3. Boundary conditions are applied in the bottom of the frame. Translation in x- and y-direction is locked as well as rotation around the z-axis.

4.1 Unreinforced frame

Depending on how the pipe breaks, three different load cases can be distinguished, see Figure 4.1. The load effect is simulated as static concentrated point loads in the rods that hold the pipe in place. Since the rods are assumed to plastify, no impulse effect is needed to take into concern. A static load is therefore regarded as the most proper way to simulate a pipe break.

![Figure 4.1 Load case when pipe breaks (a) downwards, (b) upwards and (c) horizontal.](image-url)
A reference load of 1.5 MN in each rod is chosen to determine the case that creates the largest displacement for any point in the frame. Furthermore, a gravity load of $9.81 \text{ Nm}^2/\text{kg}^2$ is applied on the whole structure. The resulting displacement from the three different load cases can be seen in Figure 4.2.

![Displacement plots](image)

**Figure 4.2 Displacement for load cases (a), (b) and (c)**

The analysis proved load case (c), when the pipe breaks in a horizontal direction, to result in the largest displacement of the frame. In this case, the top beam is displaced almost 10 mm horizontally. The displacement for load case (a) is concentrated to the top beam, with a maximum value of 2.7 mm in vertical direction. Load case (b) results in a vertical displacement of 1.7 mm in the middle beam.

The CFRP reinforcement is assumed to be applied in areas with high flexural tension. Therefore, the moment distribution from the three different load cases is studied (see Figure 4.3). These plots can be a good guidance for where to place the laminates for an efficient strengthening effect.
Figure 4.3  Moment distribution in the frame for load case (a), (b) and (c)
4.2 **Assessment of the strengthening effect in the elastic phase**

At this early stage of the analysis where a simple beam model is used, the CFRP strengthening effect was evaluated in the elastic phase by increasing the area of the flanges, equivalent to the stiffness that the laminates provide.

A (200 x 1.7 mm) UHM laminate with a Young’s modulus of 600 GPa was simulated. This laminate is equivalent to an increase of 2 mm of the tension flanges. Depending on the load case, the laminates were applied in different configurations based on the moment distributions shown in Figure 4.3. The different locations (1), (2), (3) and (4), are seen in Figure 4.4.

![Figure 4.4 Locations of applied CFRP](image)

Four different reinforcement configurations was analyzed

A1: Reinforcement at location (1) for load case (a)

B3: Reinforcement at location (3) for load case (b)

C2: Reinforcement at location (2) for load case (c)

C24: Reinforcement at location (2) and (4) for load case (c)

The largest strengthening effect came for configuration C24 where the displacement was reduced with 1.2 %. For the other cases a reduction between 0.4 % and 0.8 % was obtained. The displacement from the different configurations is shown in Figure 4.5.
4.3 Conclusions from the preliminary analysis

In the linear response, the displacement is directly proportional to the stiffness of the frame. Since the cross sections in the frame are relatively large, the effect from the added UHM laminates is rather modest. The strengthening effect can nonetheless still be improved by applying a larger proportion of CFRP to the cross sections, and thereby increase the stiffness of the frame. From this analysis, it can be concluded that a much larger amount of UHM laminates is needed in order to achieve any considerable strengthening effect in the elastic phase. It should also be noted that the high tensile strength of the laminates is not utilized until the steel starts to yield.

This analysis showed that a pipe break in horizontal direction would cause the largest deflection of the frame. It also indicated that the largest increase of the capacity of the frame was achieved at this load case. Further studies in a refined model will therefore be focused on load case (c) and reinforcement in areas where large bending moments arise from this load. The analysis showed that the largest deflection is located to the node in the upper right corner. Evaluation of the collapse load of the frame will therefore be related to this node, which from now on will be referenced as node N1 (see Figure 5.1).
Analysis of the unreinforced frame

This analysis is focused on load case (c), where the forces resulting from the pipe break acts horizontally. The current capacity of the frame is obtained as well as how much this capacity can be increased by CFRP strengthening.

For this analysis a model is created in the software I-DEAS, using continuum (solid) plane stress elements. The continuum family of stress/displacement elements is the most comprehensive one, available in ABAQUS. Continuum elements can be used to build models of nearly any shape since they may be connected to other elements on any of their faces. These elements are preferable to use in order to capture the interfacial stresses in the CFRP and adhesive layers. For computational matters, the stress through the depth of the frame is assumed to be zero. This assumption is considered to have a very small influence of the global behavior and will therefore not be of importance for the capacity of the frame.

The components in the model are built up based on the cross-sections showed in Figure 3.3. The quadratic hollow core columns are modeled as I-profiles. Except from a slightly reduced web thickness in order to obtain a moment of inertia, $I_z$, that is equivalent to the original shape of the boxes, this I-profile has the same dimensions as the boxes.

As in the simplified beam model, the load is applied as two static concentrated point loads. The boundary conditions are also the same, fully fixed at the height of the vertical stiffeners in the bottom. The steel is modeled with non-linear behavior and with the properties given in chapter 3.3.3.

Verifying the model

A quality check of the model was conducted by comparing results from the simplified beam model. Looking at node N1, see Figure 5.1, the beam model showed a horizontal displacement of 9.3 mm at a load level of 1.5 MN (no plasticity has yet occurred at this load magnitude). In the same node and at the same load level, the plane stress model showed a displacement of 9.2 mm. This indicates a good agreement of the two models.
The stresses in the flanges were also compared for the two models. From the beam model, a bending moment of 970 kN/m was obtained in the column, 150 mm from the bottom. According to Navier’s equation (5.1) this moment results in a stress magnitude of 146 MPa in the outer fiber of the flange. The stress in y-direction $\sigma_{22}$ from the plane stress model proved to be 140 MPa. This also confirms that the model is correct. It should be noted that no correction of the inclination of the columns has been done.

$$\sigma = \left( \frac{M}{I} \right) * z$$  \hspace{1cm} (5.1)

Where:

- $M$ = 970 kN/m
- $I_z$ = 1590*10$^6$ mm$^4$
- $z$ = 225 mm

## 5.2 Unreinforced frame

The displacement history for node N1 is plotted in Figure 5.2. In this plot, the regression line is drawn so that it follows the elastic response with an angle $\theta$, to the $x$-axes. The collapse limit line is subsequently drawn so that it makes an angle $\Phi = \tan^{-1} (2\tan \theta)$ with the $x$-axes. The collapse load is determined from the intersection between the collapse limit line and the displacement curve.
Figure 5.2  Determination of collapse load for the unreinforced frame

The analysis shows that the current capacity of the frame corresponds to a force of 3.52 MN in each rod. This force is causing a deflection of 47 mm in the horizontal direction.

In Figure 5.3, the behavior of the frame is illustrated for three different steps:

(a) $F = 2.6$ MN, $\delta = 16.6$ mm
(b) $F = 2.96$ MN, $\delta = 21.0$ mm
(c) $F = 3.52$ MN, $\delta = 47.3$ mm

Figure 5.3  Behavior and plastic regions at step (a), (b) and (c)
Areas where yielding of the steel has started (principal stresses of 380 MPa or above) are colored red. In step (a), the force is 2.6 MN in each rod. At this load, the first yielding occurs in the frame and is located in the web of the column, between the point of load application and the middle beam. At a load of 2.96 MN, (b), plasticity is fully developed in this area. Plasticity is also starting to grow in the web of the middle beam; at the intersection with the cross bracing struts. In (c) the frame has reached failure at a load of 3.52 MN and yielding of the web has been extended to several locations.

5.2.1 Conclusions

From this analysis it can be concluded that failure of the frame is a result of high shear forces. As the load is increasing, the frame is redistributing its capacity after first yielding has occurred. Yielding is extended to new locations but still, in large extent concentrated to the web. The analysis showed that bending moments was hardly developed in the frame. This can be explained by: the relatively short distance between rigid joints, that the load is applied close to rigid joints and/or that the flanges of the columns are over dimensioned.

This behavior was not expected when the objective of the strengthening scheme was developed. CFRP laminates could be used to strengthen the tension zone that is created in the web due to the action of shear. The area, on which the laminates can be applied, is on the other hand considered to be too small to provide an efficient strengthening effect, because the height of the web is not enough in order for the tensile strength to be fully developed in the laminates. In order to relief the steel frame from the high stresses and postpone yielding, a considerably large amount of CFRP with a high Young’s modulus would be needed. Stiff and thick laminates bonded to the small area that the web provides will also create significant interfacial stresses in the adhesive layer.

Yielding was expected to start somewhere in the flanges of the columns, due to bending. This behavior would have been preferable in order to utilize the characteristics of CFRP, when the length of the laminates is not restricted to the height of the web. Furthermore, the model was created with the condition that reinforcement would be applied on the flanges so that the stresses in the adhesive layer and the laminates could be captured. An accurate evaluation of CFRP strengthening of the web requires a completely new model, where the assumption of plane stress cannot be used. An equivalent stiffness of strengthened areas could be assessed in this model but then with the behavior of steel. This means that the elastic response of the laminate cannot be obtained when the steel starts to yield. In order to analyze the stresses and the behavior for each part in the composite strengthening (steel, adhesive and laminate), measures that are not within the scope of this thesis work are needed.

Yielding in the flanges do occur but is concentrated to small areas in the frame. At the load of 2.96 MN the frame starts to yield in the flange of the column just above the support. Later at a load of 3.1 MN, yielding has also started in the same flange, but now above the intersection with the middle beam. At 3.3 MN the cross bracing struts are beginning to yield. However, it is believed that yielding in these locations will have a minor affect on the capacity of the frame since the web yielding is much more extended and the plastic displacement has almost reached its allowable limit according to ASME.

CHALMERS, Civil and Environmental Engineering, Master’s Thesis 2008:12

35
Looking at Figure 5.2 it can also be concluded that rather limited yielding is allowed to take place in the frame until the criterion for collapse is fulfilled. This may imply that the use of UHM laminates would give the most efficient strengthening effect, since they are preferable in order to delay yielding. These high modulus laminates will on the other hand attract a large amount of the force, which can be critical for both the fibers and the adhesive layer.
6 New design considerations

At this stage, the original idea for this thesis work was to model the reinforcing effect of the frame by applying CFRP laminates to the parts of the tension flanges that have reached yielding. Since this predicted behavior turned out to be incorrect, the reinforcing measures needs to be revised.

It is obvious that a large strengthening effect is obtained by reinforce the areas of the web where yielding first occur. If this is feasible or not using CFRP laminates is highly uncertain, due to the facts discussed in chapter 5.2.1. In order to delay yielding, it is estimated that a great amount of laminates is required. This is due to the great shear forces that are developed in the frame, and also that the area where the laminates can be applied on is very small in relation to the forces acting there. In addition to this, reinforcing the web area using laminates will result in difficulties in the analysis since the laminate, adhesive, and the steel cannot be modeled each by them selves in this plane stress model. An equivalent thickness of steel would therefore have to be used.

From this standpoint two approaches can be chosen for further analysis. One is to strengthen the web in areas where shear stresses are critical. These improvements are probably not feasible with the use of CFRP laminates and steel plates are therefore used. The use of steel plates was not part of the original objective. However, this measure is considered to be so essential in order to upgrade the structural capacity of the frame, that it should be analyzed. Strengthening the web areas will also result in a new behavior of the frame that hopefully will justify the use of CFRP in further reinforcing measures.

If the web area is reinforced enough to withstand yielding, and if the failure of the frame is governed by bending, further reinforcing strengthening schemes using CFRP laminates can be analyzed. The amount and application area for these laminates is still hard to predict, due to the many and close situated rigid joints in the frame that give rise to large local stress concentrations.

The second approach is to accept that yielding starts in the web and instead increase the entire stiffness of the frame as much as possible. This measure will reduce the global deflection and is done by applying a large quantity of UHM laminates.

6.1 Web reinforcement

The unstrengthened model was revised by increasing the web thickness in the red colored areas, showed in Figure 6.1. In the columns, the web thickness was increased from 58 mm to 98 mm. In the middle beam, the web thickness was increased from 30 mm to 54 mm. This increased the original web thickness with almost 100 %, but this large amount of extra steel was found necessary in order to avoid yielding due to shear.
The collapse load for the web reinforced frame has increased from 3.52 MN to 4.20 MN, see Figure 6.2. Even though the frame deflects in another way then before, maximum displacement is still 47 mm.

(a) $F = 3.1 \text{ MN}, \; \delta = 17.5 \text{ mm}$

(b) $F = 3.8 \text{ MN}, \; \delta = 25.7 \text{ mm}$

(c) $F = 4.2 \text{ MN}, \; \delta = 47.0 \text{ mm}$
Yielding starts due to high stress concentrations in the joint between the middle beam and the column at a load of 3.10 MN. Plasticity in this area is not considered to affect the global displacement in any larger extent. However, larger displacement is starting to take place when yielding at the supports is initiated at a load of 3.25 MN. At 3.80 MN the cross bracing struts starts to yield. This is soon followed by yielding in the web, above the strengthened supports, where large shear forces appear.

### 6.2 Flange reinforcement

In this analysis, the effect of reinforcement applied on the flanges only is assessed. Since yielding will start in the web and hardly be developed in the flanges at all, the largest increase in capacity will be achieved when UHM laminates are used to increase the total stiffness of the frame.

At this stage the largest theoretical increase of stiffness was analyzed with a simplified model. Using an equivalent steel area, all flange widths were increased 93 mm which would represent 200 mm wide and 6 mm thick UHM laminates. This amount is the largest reasonable amount of CFRP that can be applied to all of the beams in the frame. If this measure results in a satisfying capacity increase, a more detailed analysis can be performed. It should be noted that the elastic behavior of the laminates does not show in the analysis. This means that the plastic deformation obtained is overestimated since the strain in the laminates is not extended as much as the strain in the steel.

Figure 6.4 shows that the collapse load for the flange reinforced frame has increased to 3.85 MN. The behavior of the flange reinforced frame and plastic regions are plotted in Figure 6.5 for three steps:
(a) $F = 2.6 \text{ MN}, \quad \delta = 13 \text{ mm}$

(b) $F = 2.8 \text{ MN}, \quad \delta = 21 \text{ mm}$

(c) $F = 3.85 \text{ MN}, \quad \delta = 40 \text{ mm}$

Figure 6.4  Collapse load for the flange reinforced frame

Figure 6.5 Behavior and plastic regions at step (a), (b), and (c) for the flange reinforced frame

As expected, this model and the unreinforced model seem to act in a similar way. In both models the first yielding occurs at 2.6 MN and is located in the web of the column, between the lower point of load application and the middle beam. Yielding in the frame is propagating like it did in the unstrengthened model, but the deflection is now more restrained due to the thicker flanges. This increased stiffness, results in a collapse load of 3.85 MN.
6.3 Evaluation

The two analyses showed that a larger increase in capacity of the frame is achieved when the web is reinforced. This measure results in an increase of 16.2 % while the largest possible increase in capacity, without web reinforcement, is only about 8.6 %. This proves the prediction that strengthening of the web is essential in order to obtain a fairly upgrade of the capacity of the frame.

The behavior of the frame is shown in Figure 6.6, where a displacement-load diagram is plotted for the unreinforced-, web reinforced- and flange reinforced model. Judging by the graph, the highest increase in capacity will be achieved when yielding is delayed. On the other hand, the displacement in the web reinforced model increases faster after yielding has started, compared with the flange reinforced model. This indicates that additional improvements on the web reinforced model can be done.

![Figure 6.6 Displacement-Load diagram](image)

It can be concluded that, if no measures to strengthen the web is taken, the largest increase in capacity from a CFRP strengthening scheme is only about 8.6 %. Since the flange reinforced model showed such a poor strengthening effect, this method is not further analyzed. Focus is instead turned to the web reinforced model and how CFRP laminates best can be used to further increase the capacity.
7 Analysis of CFRP strengthened frame

In these analyses the web reinforced model (from chapter 6.1) is refined by modeling CFRP laminates in different configurations. The laminates and the adhesive are modeled as elastic materials until failure occurs. Material properties for these can be found in chapter 3.3. The laminates have a width of 200 mm (2 x 100 mm) and the thickness is 6 mm.

In the earlier models, boundary conditions for the frame were set where the stiffeners at the support ended and all degrees of freedom were locked. This type of boundary condition applies also when the laminates are modeled. The geometry of the real structure allows the laminates to continue below the boundary in the model, see Figure 7.1. This has been taken into account for a better effect from the strengthening scheme and motivates also why the adhesive and laminates are fully fixed at the boundary. Modeling the support in this way will result in local stresses that not exactly comply with a real behavior. This will mainly affect the adhesive layer and is discussed later in the report. The length of the stiffeners is 500 mm, which is long enough to ensure that the load is fully transmitted to the laminates at the point where the boundary condition is applied.

![Figure 7.1 Laminates at support](image)

Except from strengthening at the supports, laminates on strengthened members are cut off 100 mm from rigid joints. This is done to avoid large stress concentrations at end of the laminates.

7.1 Strengthening scheme A

In the web reinforced model, yielding had started in three different places when collapse was reached. These locations are in the middle beam, at the support and in the cross bracing struts. Consequently, in a first attempt to strengthen the frame, these locations were chosen to apply the CFRP laminates on. The strengthening scheme is showed in Figure 7.2. Two analyses are performed with this strengthening scheme; one with UHM laminates and one with HS laminates.
7.1.1 Strengthening scheme A, UHM laminates

The displacement history for node N1 is plotted in Figure 7.3. As the figure shows, two collapse loads are shown in this curve. The reason for this is that failure has occurred in the laminate long before the collapse load corresponding to the deformation based failure criteria. The laminates break at the supports. When this occurs, the stress-displacement relationship for the reinforced frame continues to follow the behavior for the web reinforced frame. The frame is considered to have reached collapse when the web reinforced model has reached collapse.
The UHM laminates used in this configuration have an ultimate tensile strength of 1.1 GPa. This limit is reached at a load of 3.51 MN, which is almost 0.7 MN lower than the collapse load obtained from model with web reinforcement only. The reason for failure at this early stage is that the UHM laminates are very stiff with low ductility and they attract a lot of the stresses. The outcome of this is no improvement in the capacity of the frame. Nevertheless, if the laminate would have had higher tensile strength, Figure 7.3 shows that the collapse load according to ASME would be 4.71 MN (i.e. giving a capacity increase of 33.5 %).

7.1.2 Strengthening scheme A, HS laminates

In this model, High-Strength laminates have been added according to Figure 7.2. The high tensile strength in the laminates is expected to be utilized when the steel starts to yield, and early failure in these ductile laminates can be avoided due to the high tensile strength. The lower stiffness of the laminates will also attract less stresses in the elastic stage.

Figure 7.4 below shows that both the collapse load and the maximum displacement for the HS reinforced frame has increased. The collapse load has increased from 3.55 MN (unreinforced frame) to 4.60 MN, which is an increase of almost 23.5 %. Consequently, the maximum displacement has also increased. The frame is now allowed to deflect up to 52 mm before it is considered to have reached collapse. Nevertheless, the important matter is the magnitude of the collapse load. When collapse of the frame is reached, the tensile stress in the laminates at the support is 1.75 GPa which is well below their ultimate strength of 3.3 GPa.

![Figure 7.4 Collapse load for the frame reinforced with HS laminates according to strengthening scheme A](image-url)
The deflection and the plastic regions of the HS reinforced frame are plotted in Figure 7.5 for the stage of collapse: \( F = 4.60 \text{ MN}, \quad \delta = 51.6 \text{ mm}. \)

![Figure 7.5: Deflection and plastic regions at collapse](image)

As it can be seen in the picture, the behavior of this reinforcement is much alike the behavior of the web reinforced model with no CFRP applied. Yielding at the support starts at 3.25 MN and in the cross bracing struts at 3.8 MN, which is identical for the two models. However, it is obvious that the laminates restrain the deflection when yielding has initiated. This is most noticeable at the support, even though less than 55% of the capacity of the tensile strength of the laminates is utilized.

### 7.2 Strengthening scheme B

The strengthening effect from a new strengthening scheme according to Figure 7.6 (a) is now analyzed. The idea is that the frame can be seen as a horizontally loaded column, see Figure 7.6 (b). The Analyses from strengthening scheme A showed that critical stresses appeared at the support. In a loaded column, the highest bending moment will arise at the fixation. Consequently, the largest reinforcing effect will be when laminates are applied at the supports. But a stiffer behavior further up in the frame will also affect the global displacement. The analysis will hopefully give answer to how much reinforcement that is needed in these locations to upgrade the capacity for the frame.

The earlier analysis with UHM laminates showed that theses laminates attract a too large portion of stresses at the support than they are able to withstand. UHM laminates are therefore not used in this analysis.
Figure 7.6  (a) illustrates where the laminates are attached for strengthening scheme B, and (b) shows a horizontally loaded column which act in a similar way as the frame

7.2.1 Strengthening scheme B, HS laminates

Figure 7.7 below shows the collapse load and the maximum displacement for the HS laminate reinforced frame. The collapse load has increased 22.8 % up to 4.60 MN, and the maximum displacement is now 52 mm.

Figure 7.7  Collapse load for the frame reinforced with HS laminates at the columns according to strengthening scheme B
The deflection and the plastic regions of the HS laminates reinforced frame at the stage of collapse are plotted in Figure 7.8. The collapse load is stated as 4.60 MN, and the maximum deflection is 52 mm. These results are more or less identical to those from reinforcement strengthening scheme A with HS laminates. However, yielding in the cross bracing struts are slightly more extended now and the tensile stresses in the laminates at the support is now 2.5 GPa. This is considerably higher than in the earlier analysis and 76 % of the laminates capacity is now utilized. On the tension flange further up in the column, where the load is applied, tensile stresses of 1.2 GPa are found in the laminates.

![Figure 7.8 The deflection and the plastic regions of the frame reinforced with HS laminates according to strengthening scheme B at the stage of collapse](image)

**7.3 Evaluation**

The result from the three different reinforcement schemes, together with the web reinforced and unreinforced model, are plotted for comparison in Figure 7.9. It can be seen that the stiff UHM laminates would give the highest increase of capacity, provided that the laminate have higher tensile strength.

In the two analyses with HS laminates, the collapse load was increased with almost 23 % in comparison to the unreinforced frame. A large part of this increase is however due to the increased web thickness in critical areas. Compared to the web reinforced model, the increase in collapse load is less than 8.7 % This value is very close to what the simplified flange reinforced model (see chapter 6.2) showed in capacity increase. When no measures to strengthen the web were done this model showed a capacity increase of 8.6 %.
Figure 7.9 shows that any differences in behavior of strengthening scheme A and B with HS laminates are hardly noticeable, which is quite surprising. An explanation to this could be that neither the reinforcement on the cross bracing struts nor the reinforcement further up on the columns have any large influence on the strengthening effect. On the other hand, strengthening scheme A showed lower stresses in the laminate at support. Laminates on the cross bracing struts could be the reason for this. If they attract more of the load, the laminates at the support would be subjected to less stresses. But since there is a lot of capacity left in the laminates at the support, no increase of capacity of the frame is achieved. Judging by these analyses, it is also believed that the laminates on the middle beam in strengthening scheme A has very little influence on the strengthening scheme. Yielding at the joint between the middle beam and the column starts at the same load magnitude and is not restricted by the applied laminates.

The similarity of strengthening scheme A and B also indicates that the increase in stiffness further up in the frame is very small. UHM laminates in these locations could give a more satisfying behavior. But when HS laminates are used, the stresses in the laminates are already rather high.

Figure 7.9  Displacement-Load diagram

In order to further upgrade the capacity of the frame, these analyses leave some interesting alternatives worth investigating. One is to analyze the effect of combining strengthening scheme A and B. It is also believed that a combination of HS and UHM laminates can be effective. HS laminates are in this case used where the largest stresses appear at the support and UHM laminates are applied further up the frame where the stresses are smaller.
7.4 Strengthening scheme C

This placement of the laminates is a combination of strengthening scheme A and B but without laminates on the middle beam. Figure 7.10 shows the strengthened flanges.

![Strengthening scheme C](image)

Figure 7.10 Placement of the laminates in strengthening scheme C

7.4.1 Strengthening scheme C, HS laminates

With this reinforcement, a collapse load of 4.63 MN was achieved. This is an increase of almost 24 % compared to the unreinforced frame. Compared to the web reinforced model, the collapse load has increased with 9.3 %. The tensile stress in the laminate at the support is 2.0 GPa at the time the frame has reached collapse. In the top, where the load is applied, the tensile stress is 1.0 GPa. The load-displacement relationship can be seen in Figure 7.11.
Figure 7.11  Collapse load for the frame reinforced with HS laminates according to strengthening scheme C

Figure 7.12 shows plastic regions in the frame when collapse is reached for strengthening scheme C with HS laminates. As it is seen, yielding in the web is well extended, even though web reinforcement is applied in the worst places.

Figure 7.12  The deflection and the plastic regions of the HS reinforced frame at the stage of collapse
7.4.2 Strengthening scheme C, combination of HS & UHM

In this configuration, HS laminates are applied on the columns from the support up to the intersection with the cross bracing struts. All other flanges are strengthened with UHM laminates as Figure 7.10 shows.

The analysis showed that the stiff laminates at the upper part of the frame attracted too high stresses. The maximum tensile strength of 1.1 GPa is reached at a load of 4.38 MN. Figure 7.13 shows that collapse load of 4.70 MN would have been achieved if the laminates had stayed intact. At this load, tensile stresses of 2.0 GPa are found in the HS laminates at the support.

![Figure 7.13 Collapse load for the frame reinforced with both HS and UHM laminates according to strengthening scheme C](image)

7.5 Evaluation

From strengthening scheme C, a 24 % increase of the collapse load was achieved. This increase is only 30 kN more compared to strengthening scheme A and B, which is a bit remarkable. Especially when comparing the strengthening effect from configurations A and C with the amount of laminate that has been used in each strengthening scheme. In strengthening scheme A, roughly 7 meters of laminates has been used, while in strengthening scheme C, the amount is 19 meters.

The load-displacement relationship from the three different strengthening scheme is plotted in Figure 7.14. As mentioned earlier, strengthening scheme C with UHM laminates in the top attracts too large stresses. However, if these laminates would have stayed intact, the graph shows that stiffer laminates would increase the collapse...
load. On the other hand, the elastic behavior is also improved, which affects the angle that decides the collapse of the frame.

![Displacement-Load diagram](image)

**Figure 7.14 Displacement-Load diagram**

Judging by the graph it can also be concluded that since the collapse load is derived from the elastic angle, the differences in different strengthening schemes does not affect the collapse load very much. The differences are more obvious when plasticization of the frame has extended and appears in new members. It is however believed that this difference would be even bigger if the problem with high shear stresses were not so serious. Figure 7.12 shows plastic regions in the frame when collapse is reached for strengthening scheme C with HS laminates. As it is seen, yielding in the web is well extended, even though web reinforcement is applied in the most stressed places. This behavior is not possible to control by CFRP reinforcement and the displacement of the frame will grow.

The analyses proved strengthening scheme C with High Strength laminates to provide the best strengthening effect when upgrading the structural capacity of the frame. This has much to do with the fact that all strengthening schemes using UHM laminates failed due to rupture of the laminates long before the criteria for collapse load is fulfilled according to the ASME code. When strengthening scheme C with HS laminates has reached failure, there is 40% of the capacity left. This fact gives an indication that laminates with intermediate strength and stiffness is more efficient, or that a reduction of the thickness is possible. This will however not be evaluated in this study.
Strengthening scheme C with High Strength laminates showed the largest increase of capacity and is chosen as the final strengthening scheme. The collapse load was increased from 3.52 MN to 4.60 MN. It should however be noted that a large part of the total increase of 1.08 MN is a result of web reinforcement. Reinforcing the web resulted in a collapse load of 4.20 MN which represents 63 % of the strengthening effect. The remaining 37 % is due to the CFRP strengthening scheme.

7.6 Results

A summary of the results from all of the analyses is presented in Table 7.1 below. As it was mentioned before, the largest strengthening effect is obtained from strengthening scheme C with HS laminates. However, the largest difference of the structural capacity is obtained when the web has been reinforced. Without this measure, strengthening with CFRP would have very little significance.

Table 7.1 Summary of the results

<table>
<thead>
<tr>
<th>Strengthening scheme</th>
<th>Type of laminate</th>
<th>Approximate laminate length [m]</th>
<th>Collapse load (in each rod) [MN]</th>
<th>Increase [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>-</td>
<td>-</td>
<td>3.52</td>
<td>-</td>
</tr>
<tr>
<td>Web Reinforcement</td>
<td>-</td>
<td>-</td>
<td>4.20</td>
<td>16.2</td>
</tr>
<tr>
<td>A</td>
<td>High-Strength</td>
<td>7</td>
<td>4.60</td>
<td>23.5</td>
</tr>
<tr>
<td>A</td>
<td>Ultra-High-Modulus</td>
<td>7</td>
<td>Rupture of laminate</td>
<td>-</td>
</tr>
<tr>
<td>B</td>
<td>High-Strength</td>
<td>14</td>
<td>4.60</td>
<td>23.5</td>
</tr>
<tr>
<td>C</td>
<td>High-Strength</td>
<td>19</td>
<td>4.63</td>
<td>24.0</td>
</tr>
<tr>
<td>C</td>
<td>High-Strength/ Ultra-High-Modulus</td>
<td>(3.6 + 15.4)</td>
<td>Rupture of laminate</td>
<td>-</td>
</tr>
</tbody>
</table>
8 Evaluation of the final strengthening scheme

A more detailed evaluation of strengthening scheme C with HS laminates is now to be conducted. This configuration of CFRP laminates showed the largest increase of structural capacity but a few issues are still to be answered before the success of this strengthening scheme can be assured.

8.1 Laminates subjected to compression

Attaching HS laminates according to strengthening scheme C onto the frame, showed to increase the collapse load the most. Strengthening in this manner is therefore considered to be the ultimate and consequently also to be the final solution for upgrading the structural capacity of the frame. But an important factor that has not been taken into consideration yet is that the laminates are in some locations subjected to compression, see Figure 8.1. Today the manufacturers cannot give any information about how much the laminates can withstand in compression. It is therefore impossible to say whether the HS laminates in strengthening scheme C, or for any other laminate in any configuration, will stay intact or not.

There is however one experimental test made by the Swedish Institute of Composites, Sicomp, that showed that the mean value for allowable strain for laminates is 0.001. This means that the allowable compression stress is 0.1 % of the Young’s modulus (Hooke’s law). Hence, a HS laminate with a Young’s modulus of 210 GPa would tolerate compression stresses up to 210 MPa. More detailed information about the test, such as which type of laminate, thickness of the laminate, fiber direction, fiber amount, that has been used, have not been obtained. Because of this, there are some uncertainties whether this value is valid for the laminates used in this thesis. The allowable limit for the compression stresses is therefore very uncertain and will be used more as a guideline than as a strict toleration level.

Figure 8.1 Locations of where the laminates are subjected to compression
The analysis of strengthening scheme C with HS laminates shows a very high level of compression stresses. In the marked area, down at the support to the right in Figure 8.1, the compression stresses reaches a value of 1.8 GPa. As it was just mentioned, the values obtained from Sicomp’s test are supposed to give an indication whether the laminates will break or not. But it is clear that a compression failure of the laminate is most likely at this stress level. The magnitude of the stresses in the other compression area (to the upper left in Figure 8.1) is 0.55 GPa, which will probably lead to failure of the laminate as well.

Since the compression stresses showed to be of such a great magnitude, it could be of interest to analyze a frame that is reinforced with laminates subjected to tension forces only. Placing HS laminates onto the tension zones in the frame, see Figure 8.2, will result in a collapse load of 4.46 MN. This means that if the laminates in strengthening scheme C breaks, the collapse load will be 4.46 MN. This is since after the failure, the stress-displacement relationship will continue to follow the behavior for the frame reinforced with laminates in tension alone. The load-displacement curves for these strengthening schemes are illustrated in Figure 8.3.

Still, even though the values of the compression stresses in strengthening scheme C can seem to be somewhat large, this thesis will assume that the laminate will not break. This will demand that the manufacturers of the laminates can assist with a laminate that will fulfill this requirement.
As discussed earlier in the report, it is rather difficult to predict when failure in the adhesive layer will occur since there are no well established calculation models for steel strengthened with externally bonded CFRP. It is neither possible to predict of what mode the failure will be. The different types of failure that can occur in the adhesive layer were discussed in chapter 2.4. However, a mean value of the tensile strength is given by the producer. The mean tensile strength of the adhesive is 26.5 MPa and can be used for a rough estimation of where the interfacial stresses are critical.

The interfacial stresses, shear and peeling, have their peaks at the end of the laminates, where the stress from the steel is transmitted. Large shear stresses in the adhesive layer will also appear in locations where the steel flange is yielding. With this in mind, three regions in the frame are identified to analyze the interfacial stresses. These regions are at the support, where the load is applied and on the cross bracing strut in tension. In order to capture the interfacial stresses, a very fine mesh has been used in these regions. Figure 8.4 shows the density of the mesh, where analysis has been performed. In this figure, x marks the starting point for the plotted
interfacial stresses in the following sub chapters. Results are obtained from the middle of the adhesive layer at the point when the frame has reached collapse.

Figure 8.4  Mesh at (a) end laminate, (b) at area of load application

8.2.1  **Interfacial stresses at the cross bracing strut**

Figure 8.5 shows the interfacial stresses in the bottom of the adhesive on the cross bracing strut. In Figure 8.5 (a), the shear stress is plotted. It can be seen that a peak of almost 60 MPa is acting within the first 10 mm, before the stress stabilizes at a magnitude of about 15 MPa. The peeling stress, plotted in Figure 8.5 (b) is showing a similar behavior. In this case the stress is almost 100 MPa and is acting within the first 6 mm. Both these stresses are significantly higher than the mean tensile strength of the adhesive, which is 26.5 MPa.
In the same manner, shear and peeling stresses in the top of the cross bracing strut are plotted in Figure 8.6. The peeling stress (Figure 8.6 (b)) is also here 100 MPa, while the shear stress (Figure 8.6 (a)) now have a magnitude of 130 MPa. This is logical since yielding of the cross bracing strut is starting in the top and is much more extended here, which will cause strain differences between steel and laminate that are accumulated by the adhesive layer.

In Figure 8.7 the shear stress through the whole length of the adhesive layer at the strut is plotted. Except from large stresses at the very ends of the laminates, high shear stress is also acting within 500 mm from the top end. Here, yielding in the steel is well extend which causes large strain differences between the steel and the laminate.
Figure 8.7  *Shear stress through the length of the cross bracing strut.*

It can be concluded that the interfacial stresses are considerably higher than the capacity of the adhesive at the ends, and some kind of mechanical anchorage is needed here. The high stresses in the ends of the adhesive are explained by the fact that forces are transmitted to the laminate here, in a short length, simultaneously as great plasticization has occurred where the laminate ends.

**8.2.2  Interfacial stresses at area of load application**

Figure 8.8 shows the interfacial stresses at the top end of the laminate that is placed on the tension flange where the load is applied. Both the shear stress (Figure 8.8 (a)) and the peeling stress (Figure 8.8 (b)) are exceeding the mean tensile strength of the adhesive.
Figure 8.8  Shear (a) and peeling (b) stress at the top of the tension flange where the load is applied.

The interfacial stresses in the bottom of the same laminate are plotted in Figure 8.9. These results show that the adhesive here is in compression. The magnitude of both the shear and peeling stress are relatively low and is not considered to cause problems for the success of the strengthening scheme.

Figure 8.9  Shear (a) and peeling (b) stress at the bottom of the tension flange where the load is applied.

Yielding has been developed in the flange where the upper load is applied. The shear stress in the adhesive in this region is plotted in Figure 8.10 (a). In Figure 8.10 (b) the shear stress in the region of the lower load application is plotted. Yielding in this region has not started at the time the frame has reached collapse.

Figure 8.10  Shear stress at (a) upper load application and (b) lower load application.

A small peak in shear stress can be seen at the lower load application. The magnitude is however less than 18 MPa and is not considered as a problem. The graph in Figure
8.10 (a) does not show a smooth behavior. An explanation to why the shear stress suddenly drops can not be given. Nonetheless, the analysis shows a large peak in shear stress. The magnitude of the stress is more than 70 MPa and will surely lead to a local rupture of the adhesive. This does not necessarily have to mean that the whole strengthening effect in this area is lost. The high stress in the adhesive layer is due to large strain differences in the laminate and the plasticized steel. The load transfer from the frame to the laminates occurs further away from the plasticized region which means that the capacity of the laminates still can be fully utilized and the strengthening effect is unchanged. The analysis showed shear stresses greater than 26.5 MPa that within a distance of 220 mm below the point of load application and 110 mm above it.

### 8.2.3 Interfacial stresses at support

The interfacial stresses at support are plotted in Figure 8.11. Due to the boundary condition at this point, the shown behavior is not correct but the graphs can give a guidance of the magnitude of the stresses in this region.

The shear stress, plotted in Figure 8.11 (a), shows a magnitude of 190 MPa at the fixation. This is not true since the force is transferred further down, below the stiffeners. The behavior of the shear stress in the adhesive layer here is hard to predict since this stress depends on the relative stiffness between steel and laminate. Because of the stiffeners, the steel has a high stiffness at the bottom, which decreases further up the stiffeners. This means that the load is not transferred to the laminates as quickly as usually. Closer to the end of the stiffeners (and point of boundary), the shear force is believed to reach a value of 120 MPa as a result of the large plasticization that has started here.

In Figure 8.11 (b), the peeling stress at the point of boundary is plotted. Unlike what the graph shows, the peeling stress in this area is not believed to be critical, since no load is transferred here. However, a similar behavior of that the graph shows is possible further down, at the end of the laminate. But the peeling stress is also depending on the behavior of the shear stress, which is not known in this area. This magnitude of peeling stress is considerably larger than the strength of the adhesive.

![Figure 8.11](image)

**Figure 8.11  Shear (a) and peeling (b) stresses at the support**
It can be concluded that extra measures to anchor the laminates most certainly will be needed at the end below the stiffeners. Strain differences right above the stiffeners will cause failure in the adhesive. This will not be critical for the load bearing capacity in the laminates as long as a proper anchorage at the end can be assured, where the load transfer occur.
Summary and conclusions

The aim of this study was to investigate the possibility to upgrade the structural capacity of a steel frame with the use of externally bonded CFRP laminates.

The analysis of the unreinforced frame showed that a force of 3.52 MN is needed in each of the two U-rods to cause failure of the frame according to the American design code ASME III – Rules for Construction of Nuclear Power Plants Components. However, from this analysis another interesting fact was confirmed. Failure proved to be a result of immense shear forces in the web. Plasticity in bending as was assumed in the start of this thesis work did not occur. Hereby, this study reached a clear turning point. A new way of thinking and new considerations had to be made in order to obtain a method for upgrading the frame with CFRP laminates.

Two new approaches were now chosen for further analysis. It was obvious that the web needed to be strengthened in areas where shear stresses were critical. Therefore, one new design approach to investigate is based on the effect from increasing the web thickness that much that yielding would start in the flanges instead. In reality, this measure would be performed by adding steel plates to the web. This measure was believed to be essential in order to upgrade the structural capacity of the frame. Furthermore, this would result in a new behavior of the frame that would justify using CFRP in additional reinforcing measures. The other approach was to allow that yielding started in the web and instead increase the entire stiffness of the frame as much as possible. Applying a large quantity of CFRP would then reduce the global deflection, and consequently increase the magnitude of the collapse load.

Results from the latter approach showed that the largest increase of the capacity from a CFRP strengthening scheme only, was 8.6 %. Since this effect was so poor, this method was not further analyzed and all focus was instead set on the web reinforced model and on how CFRP laminates best could be used to further increase the capacity. The analyses showed that the biggest strengthening effect was obtained when High-Strength laminates was applied to the columns and cross bracing struts. The collapse load increased to 4.63 MN, which is 24 % larger than for the unreinforced frame. This strengthening scheme is therefore stated as the best and final solution to the problem. It should be noted though, that reinforcing the web only increases the collapse load with 16.2 %, and this measure has thereby the largest strengthening effect.

Unfortunately, the analyses leave two rather important questions unanswered. The first is if the laminates subjected to compression will hold. As this thesis is written, the manufacturers can not provide any information of the laminates’ capacity in compression. There is however one technical study made that give an indication that failure will occur, but this question should be studied further before the method can be applied.

The second question is associated with the adhesive layer and whether extra measures to anchor the laminates are necessary or not. Given that there are no regulations concerning the allowable stresses in the adhesive layer bonding CFRP laminates on a steel member, it is not possible to state failure at a specific load level. However, the analyses showed that the interfacial stresses in many locations were well above the
mean tensile strength of the adhesive. Measures to anchor the laminates are believed to be needed on the cross bracing strut and at the support.

The strengthening effect from externally bonded CFRP laminates proved to be rather modest. In addition to this, there are great uncertainties concerning the laminates in compression and the stresses in the adhesive layer that are essential for the success of the strengthening scheme. It is believed that a more efficient upgrade of the frame’s structural capacity would be achieved by strengthening the web in critical location. To further increase the capacity of the frame, elongated and thicker stiffeners at the support is believed to be a better alternative to externally bonded CFRP in this case.

The poor effect from the CFRP reinforcement is above all explained by the geometry of this specific frame. The frame has several and close situated rigid joints where yielding is initiated. Except from at the supports, bending moments are hardly developed in the frame, hereby, the intended benefits with CFRP laminates are not fully utilized.
10 References


Internet


Pictures


Design codes

[16]  ASME III, appendix II, part II-1430 (b), 2001