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LICENTIATE THESIS

CFRP Strengthening of Concrete Slabs, with and without Openings

Experiment, Analysis, Design and Field Application

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LULEÅ UNIVERSITY OF TECHNOLOGY

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Preface

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Finally, to my family: Aina, Kevin and Jennifer – you have sacrificed a lot and I hope that something good can come out from this, for your sakes, too.

Luleå in November 2005

Ola Enochsson

Abstract

Rehabilitation and strengthening of concrete structures with externally bonded FRP (Fibre Reinforced Polymers) has been a viable technique for at least a decade. The most common way to strengthen a structure is in flexure where the design normally follows traditional concrete design, with exceptions for control of bond and anchorage related to the FRP. The method is also used for strengthening in shear and torsion, as well as for strengthening of columns.

An interesting and useful application is strengthening of slabs or walls without or with openings. In the latter situation, FRP sheets or plates are very suitable; not only because of their strength, but also due to the simplicity in the execution in comparison to traditional steel girders or other lintel systems. Even though many benefits have been shown in the use of FRP strengthening of openings in practical applications, not much research have been presented in the scientific literature.

In this licentiate thesis, the results from laboratory tests on strengthened slabs loaded with a uniformly distributed load are analyzed with analytical and numerical methods. The slabs with openings have been strengthened with CFRP (Carbon Fibre Reinforced Polymers) sheets and are compared to traditionally steel reinforced slabs, both with and without openings. The results from the tests show that slabs with openings can be strengthened with externally bonded CFRP sheets. The performance is, in comparison, even better than for traditionally steel reinforced slabs if bond failure can be avoided. The numerical and analytical evaluations are in good agreement with the experimental results.

The case study presented in chapter 5, shows a practical design application of a courtyard deck strengthened with CFRP using epoxy bonded plates. It also points out the difficulties in retrofitting of existing structures. Since the information was inadequate when the original design was performed, an active design approach was used i.e. the design was changed when the true site conditions was revealed during the reconstruction work.

Notations and Abbreviations

Explanations in the text of notations or abbreviations in direct conjunction to their appearance have preference to what is treated here.

Roman upper case letters

A	Area, [m^2]
A_s	Area reinforcement steel, [m^2]
c	Length to zero shear, [m]
E	Young's modulus, [N/m^2]
F	Sectional force, [N]
F_c	Column support load, [N]
f_{cck}	Concrete compression strength, [Pa]
f_{tck}	Concrete tensile strength, [Pa]
f_{yk}	Steel yield strength, [Pa]
G_f	Fracture energy, [Nm/mm^2]
L	Length, [m]
M_d	Designing bending moment, [Nm]
M_x	Bending moment in x-direction, [Nm]
M_y	Bending moment in y-direction, [Nm]
P	Point load or reaction force, [N]
P_c	Column support load, [N]

Roman lower case letters

m	Distributed designing bending moment, [kNm]
m	Mechanical reinforcement ratio, [-]
m_{av}	Average bending moment, [kNm]
m_{bal}	Balanced mechanical reinforcement ratio, [-]
m_{d}	Designing bending moment, [kNm]
m_{f}	Bending field moment, [kNm]
m_{s}	Balanced support moment, [kNm]

Greek lower case letters

ν	Poissons ratio, [-]
ε_{cu}	Concrete ultimate strain, [-]
ε_{s}	Steel strain, [-]
σ	Stresses, [Pa]

Abbreviations

CFRP	Carbon Fibre Reinforced Polymers
CEN	The Committee European Normalisation
CSHM	Civil Structural Health Monitoring
EN	European Norm
FE(M)	Finite Element (Method)
FRP	Fibre Reinforced Polymers
ISO	International Standardisation Organisation
LVDT	Linear Voltage Displacement Transducer
SM	Strip Method
SLS	Serviceability Limit State
ULS	Ultimate Limit State
YL	Yield Line Theory

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1 INTRODUCTION

1.1 Background

The use of Fibre Reinforced Polymers (FRP) for structural building components is in Sweden a relatively new phenomenon. In the US, Canada, Japan and a number of European countries, FRP materials have been used in building constructions for about three decades as described in works by Karbhari (1998), Busel (1999) and Sims (1999). FRP use in civil structures is particularly frequent in the fields of repair, retrofit and strengthening of existing building constructions made of conventional materials. Conventional materials refer to concrete, Reinforced Concrete (RC), steel and structural wood. Structures such as bridge decks, beams or columns, chimneys, parking decks and water tanks have been treated with excellent results as reported in the publication by Busel (1995).

Floor and wall structures are some of the most commonly existing structural elements in buildings. Nowadays, rebuilding of existing structures has becoming quite common due to structural and/or functional requirements from the clients as well as the end users. The functional requirements entail often that staircases, elevators, escalators, windows, doors and even electrical, heating or ventilation systems, have to be installed. Thus, there exists a great need to introduce sectional openings in floor as well as in wall structures. The structural effect of small openings is often not considered due to the ability of the structure to redistribute stresses. However, for larger openings the static system may be altered when considerable amounts of concrete and reinforcing steel have to be removed. This leads to a decreased ability of the structure to resist the imposed loads and the structure needs therefore to be strengthened.

Today, the use of Carbon Fibre Reinforced Polymers (CFRP) to strengthen existing slabs and walls due to openings is becoming more popular, partly due to ease of installation and partly due to space saving. In these situations, CFRP sheets or plates are applied to the slab or wall before the opening is made, see Figure 1.1.

Figure 1.1 Strengthening with CFRP sheets before making an opening for a ventilation duct in an existing slab due to changed functional requirements. Photo Björn Täljsten (2000).

The required sectional area of CFRP is often calculated by simply converting the area of steel reinforcement, calculated according to existing design codes e.g. the Swedish code, BBK 04 (2004). Even though CFRP is used for strengthening of openings, very few studies on the structural behaviour of slabs with openings have been carried out.

It is clear that more studies on the structural behaviour of CFRP strengthened slabs with or without openings are needed to obtain better understanding of failure mechanisms in order to develop more efficient strengthening methods. This will lead to a safer use of the CFRP strengthening technique and more cost effective design solutions.

1.2 Aim and scope

The scope of the research work presented in this report is limited to the structural behaviour of two-way RC slabs strengthened with CFRP sheets. The aim of the research work is to give the answer to the following research questions:

1. Do the simplified design method of CFRP strengthened RC slabs with cut-out openings used in practice today give a load-capacity equivalent or higher than a RC slab without an opening?
2. In what direction around the opening should a slab be strengthen to obtain the most optimal effect of the CFRP sheets?
3. Is it possible to model the mechanical behaviour of CFRP strengthened RC slabs to better understand the structural system?
4. How can more advanced design tools for the design of CFRP strengthening of existing RC slabs/walls be used in reconstruction of existing structures?

1.3 Method

The research work is divided into the following activities in order to resolve the above research questions, see also Figure 1.2:

- *Literature review*. The aim is get knowledge about the current state of the art in field of strengthening of concrete structures in general, and in CFRP strengthening RC concrete structures in particular.
- *Experimental investigations*. The aim is to perform experiments in laboratory controlled environments in order to resolve the research question 1 and 2.
- *Numerical and analytical analysis*. The aim is to get a deeper knowledge of the mechanical behaviour of the CFRP strengthened slabs (research question 3) in order to explain the obtained result in the experiments.
- *Design case study*. The aim is to put the findings from the literature review, experiments and analysis into design practice in a reconstruction case study (research question 4).

The result from the above activities will then be compiled into conclusion and suggestions for further research activities.

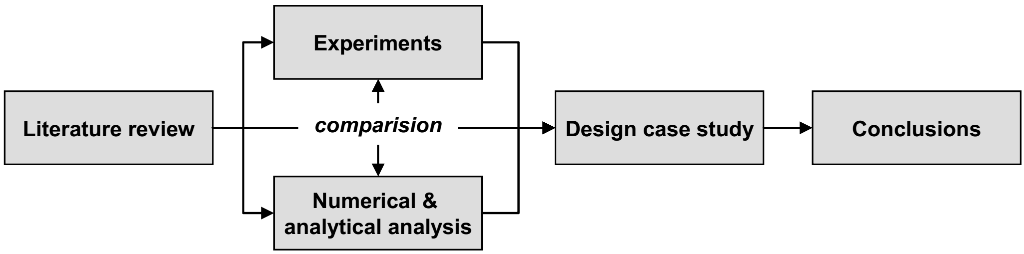


Figure 1.2 Research method.

1.4 Outline

The thesis contains the following chapters:

- Chapter 1: The introduction contains a short background identifying the problem, the aim and scope of the thesis describing the research questions, the method use and outline of the thesis.
- Chapter 2 presents an overview of strengthening of RC structures in general and CFRP strengthening in particular.
- Chapter 3 presents the experimental test setup and results.
- Chapter 4 contains the analysis and comparison of the experimental result with numerical methods used to analyse the experiments and an analytical method used to design the tested slabs.
- Chapter 5 contains the CFRP strengthening design of a reconstruction case study.
- Chapter 6 presents conclusions in relation to the research question and suggestions for future research activities.

2 STRENGTHENING OF CONCRETE STRUCTURES

2.1 Introduction

2.1.1 Strategies for maintenance, repair and upgrading

Buildings and structures are often used in different ways from how they were originally designed. Due to increasing service loads and/or degradation of existing concrete structures, the need for strengthening or retrofitting of aging infrastructure is increasing. Today, a significant portion of our infrastructure is currently either structurally or functionally deficient, Täljsten (2002). Beyond the costs of maintenance, the real consequences for our society are losses in production and overall economy due to functional deficient infrastructure.

It is not always economically viable to replace an existing structure with a new one. The challenge is to develop robust and economical viable techniques for reparation and upgrade that can be used to prolong the life of our existing structures. This challenge also places a great demand on the assessment, the design of the rehabilitation methods as well on the execution of the repair/upgrade procedure.

In the EU-Project: Rehabcon - Strategy for maintenance and rehabilitation in concrete structures, a repair and management system manual for existing concrete structures has recently been developed, Rehabcon manual (2004). The manual classifies suitable technical repair solutions for different types of damages (both cause and type of damage) in the context of an asset management system, see Figure 2.1.

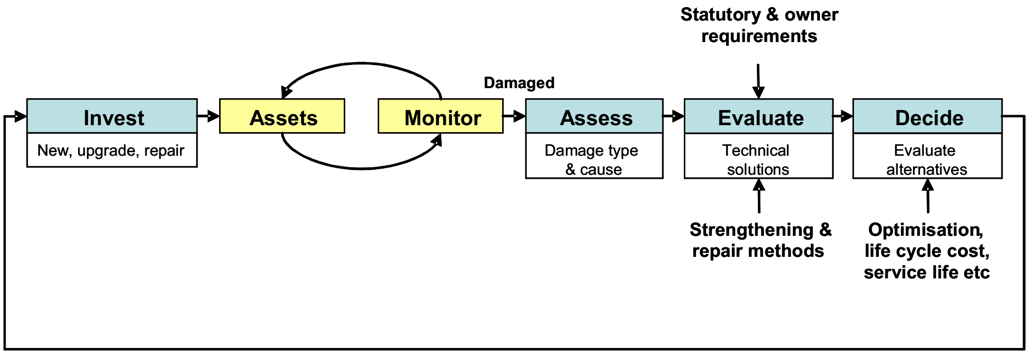


Figure 2.1 *Repair and strengthening methods in the context of an asset management system, adapted after Rehabcon manual (2004).*

Structural assets must be maintained to keep their value, safety and serviceability intact by monitoring the performance. If the inspections reveal that the integrity of the structure do not fulfil the requirements the damage cause and type must be determined in order to take appropriate action of:

- Continue regular maintenance.
- Issue some restriction in use.
- Repair.
- Upgrade.
- Demolish and rebuild.

Appropriate repair and upgrading methods must be evaluated if the structure is to be rehabilitated. The rehabilitation process must optimised from life cycle perspective to support the owners in their decision making process. The same evaluation procedure can be applied in the case of upgrade where the primary cause for action is change of requirements or conditions for the structure and not detected deficiencies.

In this report, maintenance is defined as regularly measures to keep a structure at its original performance level. Repair is defined as measures to lift up a structure to its original performance level which may be needed in cases of accidents or if the maintenance has not been carried our properly. In cases of upgrading the performance level of the structure is increased.

Performance level is referring to function, aesthetics, load carrying capacity, or durability. Here is focused is placed on the load carrying capacity which also often is denoted strengthening.

Strengthening of existing structures shall only be carried out if absolutely necessary, if possible it is preferable to use administrative upgrading where refined calculation methods are used in connection with exact material, loading parameters and partial coefficients to show that the existing structure has a higher load-carrying capacity than what was earlier assumed. This may also be combined with advanced measurement methods, i.e. Civil Structural Health Monitoring (CSHM). However, if it is found that a structure has to be strengthened and that FRP is the solution a strict design methodology shall be followed. The reason for repair or/and strengthening may depends in principle on changed structure, loads or degradation, separately or in combination. This is illustrated in Figure 2.2. Here normally the highest strengthening effect is needed when both the structural system and the load have been altered so the structure is negatively affected. You may also argue that degradation (durability) is another reason for upgrading. However, degradation or aesthetic issues are briefly discussed in the next section of the report.

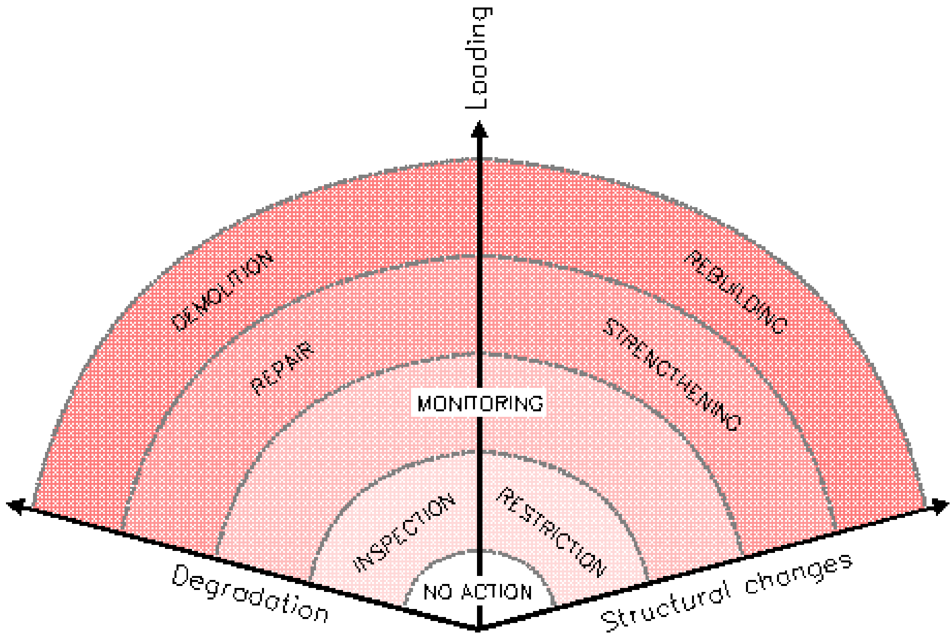


Figure 2.2 The interaction of structural changes, loading or degradation implying the necessary action.

Changed loads, normally increased loads, depends usually on changed loading conditions in existing codes and standards, change of activity, or in extreme cases accidents.

Structural changes are often related to changed activity or changed design of the structure. An obvious example is an introduced opening in an existing slab, or removal of a supporting wall.

2.1.2 European standards

The Committee European Normalisation (CEN) is responsible for developing European Standards (European Norm - EN). EN 1504 is the CEN Standard for materials for the protection and repair of concrete. It contains 10 parts covering products and systems for the protection and repairs of concrete structures from definitions, requirements for different types of repair systems, quality control and evaluation of conformity, see Table 2.1.

Table 2.1 EN 1504: Products and systems for the protection and repair of concrete structures.

CEN standard	**Short description**
EN 1540-1	Scope and definitions.
prEN 1504-2	Surface protection system to increase the durability of concrete structures.
prEN 1504-3	Structural and non-structural repair of concrete structures.
prEN 1504-4	Structural bonding of strengthening materials to existing concrete structures.
prEN 1540-5	Concrete injection of cracks, voids and interstices in concrete structures.
prEN 1504-6	Grouting to anchor reinforcements or to fill external voids.
prEN 1504-7	Reinforcement corrosion protection systems.
prEN 1504-8	Quality control and evaluation of conformity of protection and repair system for concrete structures.
ENV 1504-9	General principles for the use of products and systems protection and repair system for concrete structures.
EN 1504-10	Site application of products and systems and quality control of the repair works.

The parts prENV1504-2 to 7 details the performance test requirements for the repair methods given in ENV 1504-9. Supporting test methods are given in other CEN and ISO standards. These are normally selected and adapted, in order of priority, from existing CEN, ISO, National and other sources of standards. The requirements of products and systems are defined in terms of their typical use. The requirements of the protection and repair measure are not explicitly linked with the environmental or loading conditions but these must be taken into account when specifying the performance testing.

2.1.3 Repair and upgrading methods

The repair and upgrading methods of concrete structures can be classified into:

- Repair and upgrading systems for protection of concrete and reinforcement.
- Structural repair or upgrading systems for existing concrete structures.

Systems for protection include methods like surface coating, filling of cracks to increase the physical resistance and protection of ingress of chemicals, moisture etc. It also includes coating and electrochemical treatments for increasing the resistivity, cathodic protection and control to prevent corrosion attack on the steel reinforcement.

Patch repair is by far the most common technique to structural repair damaged or deteriorated areas in concrete structures. Furthermore, when other remediation techniques are being applied in order to limit the extent of on-going corrosion mechanisms or to prevent their re-occurrence, patch repairs are also used to reinstate the spalled or delaminated areas of concrete.

Increasing demands and changed use of infrastructure often lead to that the structural components of the infrastructure need to be upgraded. This often results in introducing external systems such as:

- Installing extra bonded rebars (steel or FRP) in cut-outs or drilled holes in the concrete.
- Plate bonding of steel, FRP plates or sheets to the surface of the structure to be strengthened.
- Installing of external pre- and post tensioned reinforcement systems.

Figure 2.3 shows a schematic example of how a deteriorated reinforced concrete beam being rehabilitated using patch repair followed by external strengthening using plate bonding. The steps in the rehabilitation process consist of (1) identification of the damage, (2) removal of affected concrete and corrosion products, (3) surface treatment of concrete and reinforcement to improve adherence between repair material and substrate, (4) application of repair material, (5) surface treatment and/or application of bonding agents to improve bond between strengthening plate and the concrete surface, (6) plate bonding and finally (7) creating aesthetic and protective barrier by coating.

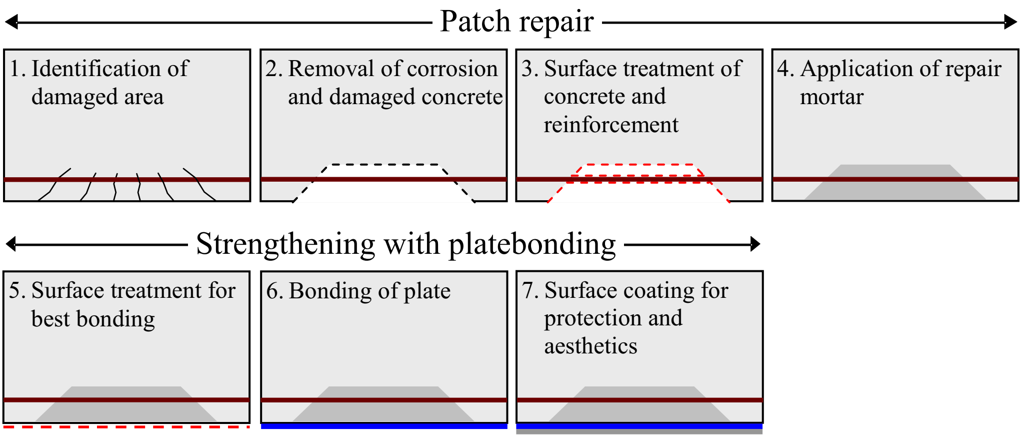


Figure 2.3 Procedure of a patch repair and plate bonding, based on Täljsten (2003).

External strengthening of structural members has been practiced since the mid sixties with steel plates bonded to the tension side of structures, Täljsten (1994). The in situ rehabilitation or upgrading of reinforced concrete members using bonded steel plates is an effective, convenient and economic method of improving structural performance.

However, disadvantages inherent in the use of steel plates such as: handling of the heavy steel plates, corrosion of the interface adhesive steel as well as the need of butt joint systems as a result of limited manufacturing lengths, have stimulated research to find alternative strengthening systems. Meier (1987) and Kaiser (1989) demonstrated that steel plates could be exchanged by Carbon Fibre Reinforced Polymer (CFRP) plates, a lightweight, non-corrosive and no length limited material.

2.2 FRP in repair and strengthening of concrete construction

2.2.1 Introduction

The traditional strengthening methods, such as addition of a girder-column system, or construction of load-bearing walls along the edges, take up useful space and may not be aesthetically convenient. On the other hand, advanced composites as externally bonded reinforcement has been extensively tested as related to its use for strengthening of beams and girders in flexure, shear and even for some extent in torsion, Täljsten (1994, 1997 and 1998), Triantafillou (1998), Täljsten & Elfgren (2000), Neale (2001), Maruyama (2001), Teng et al (2002), Carolin (2003), Carolin et al (2003), Täljsten (2003) and Täljsten et al (2003). This strengthening technique has been successfully used in several repair and strengthening projects in Sweden and elsewhere FIB, Bulletin 14 (2001).

Täljsten (2002) compared the advantage and disadvantages of using FRP in repair and strengthening structural applications, see Table 2.2.

Table 2.2 Potential advantages and disadvantages of using FRP in strengthening applications, after Täljsten (2002).

Advantages	**Disadvantages**
Handling and transportation: FRP reinforcement is very light and easy to handle compared to steel plates. It requires much less transport of material compared to traditional methods such as concrete overlays or shotcrete. ***Durability and maintenance:*** Carbon fibre composites (CFRP) have good durability, long-term fatigue properties, and they do not need to be maintained over time. ***Thin strengthening layers:*** Thin strengthening layers will not change the dimension of the existing structure and can also be combined with other methods such as thin concrete overlays or surface-protecting materials.	***Mechanical damage***: Since the FRP materials themselves are brittle, they can be easily damaged by different type of impacts. Therefore, they should be protected. However, they can easily be repaired. ***Long-term properties***: Experience from long-term applications is lacking. Today there is a concern regarding the adhesive layer. However, experience from steel plate bonding shows that many of the old steel plate bonded structures are still in use with no sign of deteriorated in the adhesive layer. ***Working environment***: Epoxies used for bonding of the CFRP sheets or laminates are unhealthy. Workers must use protective aids to minimize health risks.

Advantages	Disadvantages
Time of construction: FRP strengthening including execution and hardening of bonding agent can often be done under live loads in a short period of time causing less. ***Pre-stressing possibilities:*** New CFRP products on the market can be pre-stressed in combination with bonding. This gives a higher utilisation of the strengthening product, the possibility to reduce existing cracks, and increasing the yield load of the existing reinforcement. ***Design***: The possibility to optimise the FRP materials in the direction most needed is a benefit for design. ***Cost***: The cost of a strengthening work with composites compared to traditional methods is often lower, even though the material costs are higher.	***Temperature and moisture dependent***: The hardening process of thermosetting adhesives is moisture and temperature dependent. ***Lack of experience and conservatism***: It takes time to introduce new methods and materials in the construction sector. Research, education and standardisation are important factors to reach breakthrough in the industry. ***Design***: The lack of experienced design consultants in CFRP strengthening of concrete structures is hindering the exploitation. ***Cost***: The carbon fibre sheets, laminates are much more expensive compared to traditional building materials.

The risk for earthquakes in countries like Japan and western USA has promoted new methods for strengthening of concrete columns. In other countries, such as Sweden, the main needs for strengthening are to adapt the existing structures to higher loads or for change in use.

2.2.2 FRP material and adhesives

FRP materials are sometimes called FRC (Fibre Reinforced Composites) or PMC (Polymer Matrix Composites). FRP has been traditionally used to designate products manufactured by Hand- or Spray-Lay-Up and PMC appears to be the most appropriate denomination for the whole category of materials. However, FRP is the most common term within the civil engineering research community. Composite materials are obtained by combining two or more materials with different mechanical and/or physical properties to obtain a new material better fitted for a specific purpose. FRP is a material where the solid fibres are embedded in a polymeric matrix to reinforce the composite material in specific directions.

The fibres provide the FRP material with its strength and high stiffness. The fibres are filaments with a very small diameter in the order of 10 μm. They may exhibit different mechanical properties in the longitudinal and cross sectional (transverse) directions. For instance, for carbon and aramid fibres the elastic modulus (or the fibre strength) in the longitudinal direction is much higher than the elastic modulus in the transverse direction. Table 2.3 presents typical mechanical properties of some common fibres. Strength and stiffness are given for the longitudinal (strong) direction of the fibres.

Table 2.3 Mechanical properties of some fibres, after Godonou (2002).

Fibre type	**Elastic modulus [GPa]**	**Tensile strength [MPa]**	**Failure strain [%]**
E glass	69 – 72	2400 – 3800	4.5 – 4.9
S-2 glass	86 – 90	4600 – 4800	5.4 – 5.8
Carbon (HS/S)	160 – 250	1400 – 4930	0.8 – 1.9
Carbon (IM)	276 – 317	2300 – 7100	0.8 – 2.2
Aramid (Kevlar 29)	83	2500	–
Aramid (Kevlar 49)	131	3600 – 4100	2.8

Glass fibres are usually used in combination with polyester or vinyl ester matrices in order to obtain lightweight and low cost FRP structural components. Common industrial applications are some automobile, truck and bus components, leisure boats, aircraft interiors, electrical equipment and sporting goods.

Carbon fibres are used for applications where excellent mechanical properties and low weight are the main requirements. Examples are high performance racing vehicles, yatches, spacecrafts, aircraft and sporting goods.

Aramid fibres have excellent toughness and damage tolerance properties. They are very difficult to cut, can absorb moisture and are very expensive. Common applications are impact-prone areas of aircraft, ballistic armor and some sporting goods.

Figure 2.4 shows examples of commercial available glass and carbon fibre products.

a) b)

Figure 2.4 a) Glass fibre woven bi-directional fabric and b) carbon fibre roving.

The term matrix or resin is used to designate the polymer precursor material and/or mixture with various additives or chemically reactive components. Its chemical composition and physical properties fundamentally affect the processing and final properties of the FRP material. Processability, lamina and laminate properties, composite material performance and long-term durability are all dependent on the matrix composition.

Table 2.4 shows some mechanical properties of the matrix material.

Table 2.4 Typical mechanical properties of common resins, after Godonou (2002).

Matrix / resin	**Elastic modulus [GPa]**	**Tensile strength [MPa]**	**Failure strain [%]**
Polyester	3.1 – 4.6	50 – 75	1.0 – 6.5
Vinylester	3.1 – 3.3	70 – 81	3.0 – 8.0
Epoxy	2.6 – 3.8	60 – 85	1.5 – 8.0

The FRP composite can be produced by a number of methods, Godonou (2002), where the most common methods are:

- *Hand layup:* In the hand layup method, the fibres are laid in a male or female mould and the matrix is poured on and spread by means of a roller to facilitate a thorough impregnation.

- *Pultrusion:* In this method fibres are pulled from a creel through a resin bath and then on through a heated die. The cured profile is then automatically cut to the desired length. Pultrusion is mainly used to produce laminates with constant cross-section.
- *Filament winding:* Products manufactured using filament winding are usually hollow, generally circular or oval sectioned components, such as pipes and tanks. Fibre tows are passed through a resin bath before being wound onto a mandrel in a variety of orientations.

Typical properties of commercial pultruded plates are given in Table 2.5. The characteristic design values are compared with the corresponding properties of mild steel.

Table 2.5 Typical properties of pultruded CFRP plates, FIB Bulletin 14 (2001).

Material	Elastic modulus [GPa]	Tensile strength [MPa]	Ultimate tensile strain, [%]
Pultruded laminates:			
- Standard modulus	150	2 700	1.8
- Medium modulus	200	2 200	1.1
- High modulus	300	1 300	0.5
Mild steel	200	400	> 25, yielding 0.2

In cases where the composite is built up in situ using hand layup methods, so-called wrap system, the properties of the pure fibres are often used in the design.

The FRP composite is normally attached to the structural component using an epoxy adhesive. Epoxy is a group of polymers with different chemical, thermal and mechanical properties. The mixing of an epoxy resin with a hardener results in an epoxy adhesive. The properties of epoxy adhesives are mainly dependent on the hardener used. The hardening rate is strongly dependent on the ambient temperature. The reaction is slow in moderate or cold temperatures and faster in warm temperatures. For this reason commercially sold epoxy systems contain additives such as flexibilizers, extenders, dilutents and fillers in different amounts to meet the specific demands of the application.

The success of getting a solid bond between the FRP material and the structural component to be strengthened depends on a number of factors from preparation, mixing, application temperatures, curing temperatures, surface preparation, thermal expansion, creep properties, abrasion and chemical resistance. The execution process is of tremendous importance as it is essential to understand where and when the strengthening materials can and should be used. If the work is not carried out in a careful way - the final strengthening result could be severely affected. Therefore it is of outmost importance that the strengthening systems must not be divided in separate parts, where the FRP materials comes from one supplier and the adhesive from another, unless the systems has been carefully investigated and tested together, Täljsten (1998).

2.2.3 FRP strengthening systems

Different possibilities of strengthening concrete structures are shown in Figure 2.5. FRP strengthening is suitable for concrete beams, walls, slabs and columns, but can also be used to strengthen cut-out openings in slabs or walls.

The possibility of designing the FRP material and adapt the manufacturing process for specific strengthening application has lead to a variety of FRP strengthening system, where the most of them fall into two categories:

- Sheet system using hand layup.
- Plates designed for different types of strengthening applications.

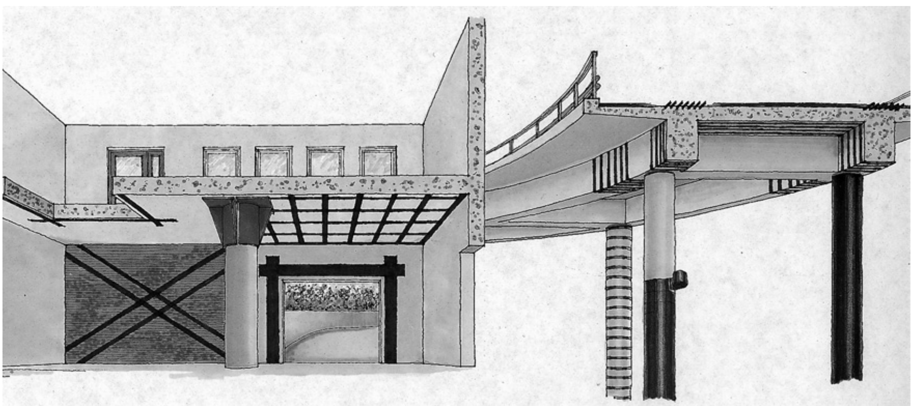


Figure 2.5 Examples of FRP strengthening of concrete structures, from Täljsten (1998).

Sheets systems are usually based on dry unidirectional fabrics, but bi-directional weaves are also used. Sheet system can be used for most strengthening applications but is especially useful in seismic retrofitting and the strengthening of curved structures such as wrapping of columns and silos, see Figure 2.6. The fabric can also be wrapped around beams or columns loaded in compression or torsion to give a confining pressure acting on the structure. These types of systems are also very suitable in cases where openings need to be strengthening in walls or slabs. A typical sheet system consists of an epoxy primer, putty, dry or pre-impregnated fibre and an adhesive system.

The strengthening process for sheet systems is a little bit more time demanding than for the plate system. First, the concrete surface is pre-treated. A primer is then applied and in cases of large unevenness, putty is used to level out these irregularities. The next step is to apply a thin layer of low viscosity epoxy adhesive to the concrete surface and then roll the carbon fibre sheet out over this surface. The fibres are stretched, and a roller is used to press out possible air voids, then a new layer of adhesive is applied. This process can be repeated up to as much as 10 - 15 layers depending on the strengthening system used, Täljsten (1998).

Figure 2.6 Strengthening of a concrete silo with composite wrap system.

The first applications with CFRP plate system were carried out in Switzerland during the beginning of the 1990s, Meier et al (1992), where a concrete bridge was strengthened due to an accident that broke the pre-stressing cables. Since then a large number of objects have been strengthened worldwide. A plate system consists of a flat pultruded profile with a typical size of 1.2×100 mm. The plate can be obtained in different grades and cross-sections. Theoretically, the length of a plate can be unlimited but in practise, the length is limited to 20 meters. Other components are concrete primer and adhesive. The function of the primer is to enhance the bond for the adhesive to the concrete. The adhesive used is a high viscosity filled paste such as epoxy adhesive. A typical bond layer thickness is 1-2 mm. Figure 2.7 shows the strengthening of a concrete wall inside a box girder bridge with CFRP plates.

Figure 2.7 Strengthening of the Gröndal and Alvik bridges, Carolin (2004).

Plates are most suitable for flat surfaces such as beams, walls and slabs. After the concrete has been pre-treated, the adhesive layer is placed on to the plate and in some cases also to the concrete surface. The two adherents are then mounted together and a light pressure is applied on the plate. Thereafter the system is allowed to harden.

A special type of FRP strengthening system is the NSMR (Near Surface Mounted Reinforcement). NSMR systems are used in cases where the strengthening system needs to be protected, for example in the case of possible impact. NSMR systems are also suitable to use if the concrete surface is very uneven. Most NSMR systems consist of circular or rectangular pultruded rods that are bonded in slots in the concrete cover of a structure. It is important to control the thickness of the concrete cover before this method is chosen; a typical depth of at least 25 mm is normally needed. The pre-treatment for this method consists of sawing slots in the concrete cover. The rods are then bonded in these slots with an epoxy adhesive or a high quality cement grout. Figure 2.8, shows a typical strengthening application with NSMR.

Figure 2.8 Strengthening of a bridge joint with BPE® NSMR system.

Promising result of applying pre-stress to NSMR systems have been demonstrated by Nordin (2001).

2.3 Design guidelines

In general, the design for strengthening of concrete structures is of utmost importance. Not only does one have to consider the performance of the existing structure, but also the function of the newly strengthened one. To be able to strengthen structures in an optimal way and to use the FRP materials most effectively proper design guidelines are needed.

Lack of guidelines will not only reduce the use of FRP for strengthening but also risk that the strengthening materials are used incorrectly without understanding of how the FRP material and the structure work together. Accordingly, it is of utmost importance to develop and compile design guidelines and codes for FRP Plate Bonding in general. Today there exist several design guidelines for example in Canada, Neale (2001); Japan, Maruyama (2001) and Sweden, Täljsten (2004) and the FIB Bulletin 14, FIB Bulletin 14 (2001), to mention a few. However, in none of the above mentioned guidelines recommendations for strengthening of opening do exist. In this chapter a general discussion regarding the design for strengthening and rehabilitation of concrete structure, and strengthening with CFRP in particular will take part.

2.4 Design and strengthening methodology

2.4.1 General

All strengthening objects are different, even though similarities exist. We have also to keep in mind that often it is considerably more complicated to strengthen an existing structure compared to building a new one – you are deadlocked with the existing conditions and can not chose the most optimum design methodology. Strengthening is the process of adding capacity to a member or a structure. You may also divide between passive and active design in which the latter is defined so that the strengthening must immediately participate in stress sharing. Active design may also mean that the original design is altered due to changed conditions when the project has already started. In FRP strengthening both active and passive design is used, where the passive design is the most common.

For strengthening of concrete structures the general design methodology can be divided into several steps:

- Evaluation of existing conditions.
- Strengthening analysis.
- Strengthening strategy and method.
- Strengthening procedure, control and follow up

2.4.2 Evaluation of existing conditions

Evaluation of the strengthening need and background to the project is most essential. This phase in the methodology govern the rest of the project. It is extremely important to clarify the condition of the structure, existing documentation, future needs and if possible, the history of the structure, e.g. has it been repaired or strengthened earlier.

2.4.3 Strengthening analysis

The strengthening analysis gives the information for what capacity the existing structure has with regard to shear, torsion, flexure, stability etc. In the strengthening analysis analytical as well as FE-analysis may be used. Also, probabilistic design may be a useful tool.

The disposition of the strengthening analysis is highly dependent on the amount of available information and type of project (repair, reconstruction or upgrading). If for example the information about the existing structure is inadequate an active design and analysis approach can be needed, i.e. the design must be changed during the repair or reconstruction work as more information become available. Furthermore, in projects where constructions are structurally changed and exposed to higher loads, the analysis must be divided in several steps to identify surplus and deficit capacities both before and after the structural and loading conditions have changed.

The design is also governed by codes and practice. In normal design practice moment and shear capacities of a construction are checked in the ultimate limit state, (ULS), whereas deflections and crack widths in concrete construction are treated in the service limit state, (SLS). However, in the design of strengthening systems special consideration must be paid to the redistribution of stresses due to e.g. changes in structural stiffness that can weaken and damage other part of the structure which has not been strengthened. Therefore, it is of outmost important to analyse the whole structure (in ULS and SLS), not only those part that will be strengthened.

2.4.4 Strengthening strategy and method

The strengthening strategy is related to passive or active systems. This also is also dependent on codes and standards, for example the size of existing cracks or permissible deflection. In rehabilitation projects is it often difficult to obtain all information about the structure to be rehabilitated.

Components of the structure may be hidden, not all material data is known, and earlier repair activities have not been reported and so on. The strengthening method chosen depends on the strengthening analysis and strategy. If a structure needs strengthening for flexure and increased loads together with increased stiffness and decreased crack widths, then a pre-stressing system shall be used. Other factors that govern the choice of strengthening method may be weather conditions, accessibility to the component or structure that needs to be strengthened. Also, the need to keep the activity going during strengthening determines the strengthening method.

2.4.5 Strengthening procedure, control and follow up

The success of a strengthening system is highly dependent on the quality of strengthening work. The expected performance and the life-span of a strengthening can be seriously be affected by poor workmanship. It is especially important to provide clear and unambiguous work instructions to avoid detrimental effects on, e.g. the quality of bond between the strengthening material and the parent structure.

Control checklist and following up the strengthening work is also essential to be able to guarantee the intended function of the strengthening system. Unfortunately, this is unusual. However, a complete methodology shall contain a plan for following up and control. In its simplest form this could mean checklists and more advanced systems like use of monitoring.

2.5 Strengthening analyses of slabs

2.5.1 Short historical background

Classical analytical methods are based on the theory of elasticity, see Timoshenko & Woinowsky-Krieger (1959). Concrete slabs have a capacity to redistribute high moment concentrations by cracking and by local yielding of the reinforcement. This is taken advantage of in the yield line theory; see e.g. Johansen (1943), Jones & Wood (1967) and Nielsen (1984). The yield line theory gives an upper bound to the load carrying capacity and may over estimate it, if a too simple or optimistic yield line pattern is assumed in the design. A lower bound to the capacity can be found with the strip method, see e.g. Hillerborg (1996). In Sweden, a standard method has been developed for slab design by Arne Hillerborg (1990). The method is originally based on the theory of elasticity, but to get a more economical design of steel reinforcement the method has been modified to take into account the yield line theory.

The method gives the maximum moment m as a simple formula $m = \alpha q b^2$, where q is the distributed load and α is a tabulated coefficient depending on the boundary conditions at the support and the ratio between the length a and the width b of the slab.

2.5.2 Swedish design methods

In Sweden, a floor structure, in case of any opening, is designed in two different ways depending on the size of the opening in relation to the geometry of the slab. Entry openings for electrical or pipe installations are normally not defined as an opening.

In slabs, subjected to a uniformly distributed load, a sectional opening with a length of maximally 1/3 of the shortest slab span is defined as small in BBK 04, otherwise it is defined as a large opening. In the latter case, the edges of the large opening are considered as free edges i.e. the moments acting in the same direction as the edge are redistributed to be more concentrated closer to the opening. In the former case, the slab is first designed as a slab without an opening i.e. the moments and shear forces are calculated as the opening does not exist. The moment and shear forces that would pass each half of the edge of the opening is added to existing moment and shear forces, along the closest edge of the opening within a band, that is maximum 3 times wider than the slab thickness. The reinforcement is given at least the same length, as it would have had if the opening had not existed.

In Figure 2.9, two arrangements of additional reinforcement due to a small opening are illustrated, one according to BBK 04 and one according to a configuration tested in this study.

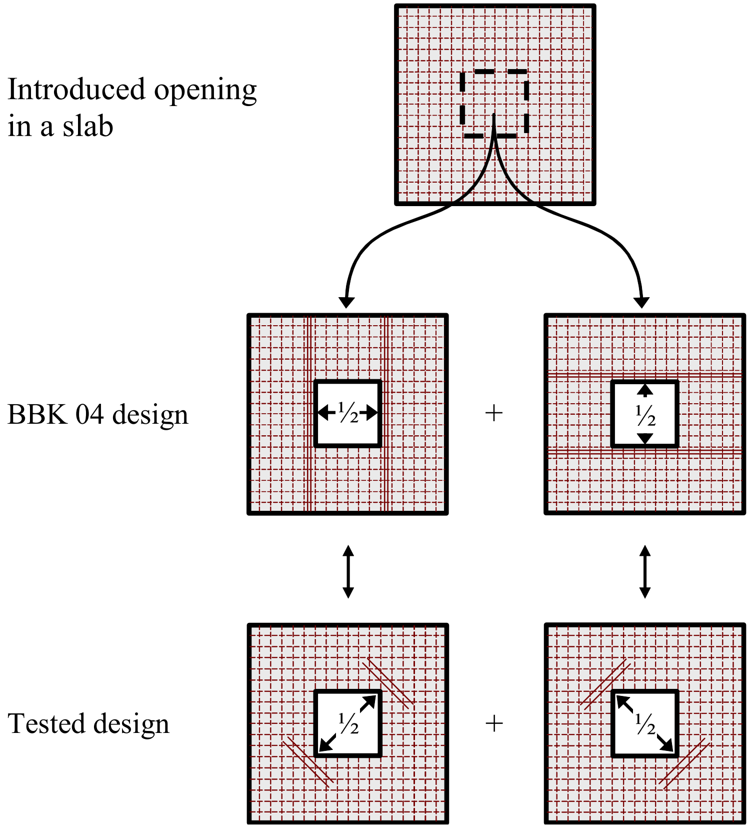


Figure 2.9 Corresponding methods to reinforce a slab due to an introduced small opening according to BBK 04 (2004) and a configuration tested in this study.

2.5.3 Recent research

The flexural behaviour of CFRP strengthened one-way slabs with cut-outs, subjected to point loads have been studied by Vasques & Karbhari (2003). The purpose was to investigate the effectiveness of externally bonded FRP sheets at strengthening of slabs with cut-outs. The failure mechanism and post-debonding response were also studied. The outcome of the study was that externally bonded FRP sheets can be used to restore the original load carrying capacity of slabs weakened by cut-outs. In addition, they observed a more desirable crack pattern for the CFRP strengthened slabs than for the non-strengthened slabs. However, the used method to decide the anchorage length for the FRP, seems not to be appropriate in areas of high curvature. Therefore, the final failure was more or less always initiated by peeling followed by debonding of the FRP sheets.

In another study by Mosallam & Mosalam (2003), the flexural behaviour of FRP strengthened two-way slabs without openings subjected to uniformly distributed loads was investigated. Both carbon/epoxy and E-glass/epoxy composite systems were used in this study. The study shows that both FRP systems can successfully be used to repair or upgrade the structural capacity of both two-way reinforced and un-reinforced concrete slabs. A significant increase in the load carrying capacity for the CFRP strengthened slabs was observed (approximately five times that of the as-built slabs). However, the loading system with high-pressure water bags limited the maximum deflection during the test due to the bag's thickness and bedding ability.

Tan and Zhao (2004) tested six one-way RC slabs with openings strengthened with externally bonded CFRP systems and subjected to concentrated line loads. The CFRP systems consisted of CFRP sheets in all cases except one, in which CFRP plates where applied. The results were compared to those of a solid slab without opening and a slab with a non-strengthened opening. They concluded that the CFRP system proved to be effective in enhancing the load-carrying capacity and stiffness of RC slabs with an opening, provided that premature failure due to CFRP debonding is excluded. In five of the tested CFRP strengthened slabs debonding of the CFRP sheets was part of the ultimate failure. One of the specimens failed in shear. They also compared the result with an analytical model based on a modified yield line theory. The contribution from the CFRP sheets was based on when end crack induced debonding occurs.

To conclude, the problem with debonding in the above investigations shows that the potential strengthening effect of the fibre reinforcement has not been achieved since tensile failure in the CFRP sheets did not occur. Also, testing of one-way slabs in bending subjected to concentrated loads will introduce much higher curvature in a slab compared to a two-way slabs supported on all edges and subjected to a uniform distributed load. Therefore, at Luleå University of Technology, research regarding CFRP strengthening of two-way slabs using distributed loads and supported on all edges has been carried out. The started in 2002 as a pilot test in order to investigate the possibilities to strengthen slabs with CFRP sheets, Ericsson & Larsson (2003). The research was very promising and more detailed studies have been carried out since then, Enochsson et al (2004) and Rusinowski (2005). The major findings from these experiments will be presented in the next chapter.

3 EXPERIMENTAL STUDY

3.1 Introduction

The experimental program at Luleå University of Technology is consisting of two-way RC slabs loaded to failure using a uniformly distributed load by a special test setup. The objective with the tests was to compare the results between different slab configurations:

- Without an opening (homogeneous slab).
- With a cast opening, strengthened with conventional additional steel reinforcement.
- With sawn-up openings;
  a. without additional strengthening (weakened), and
  b. strengthened with CFRP sheets, designed to reach the same load carrying capacity as traditionally steel reinforced slabs without openings.

The slabs are quadratic with a side length of 2.6 m and a thickness of 100 mm. Two different sizes of openings are used, 0.85×0.85 m and 1.2×1.2 m, see drawings in Appendix A. All slabs where designed according to the Swedish standard method for a quadratic freely supported homogenous slab loaded with a distributed load shortly presented in section 2.5.1, except for the slabs with a cast opening. For the slabs with a cast opening, additional reinforcement according to Swedish design practice (BBK 04) is needed around the opening. The needed amount of reinforcement is equal to the amount in a reinforced homogeneous slab that would pass through the opening, see also Figure 3.7a.

3.2 Test setup

To provide a uniformly distributed load on the slab, a new unique test rig is developed, see Figure 3.1. The load is applied using a system of airbags, embedded by an exterior and interior structure, see Figure 3.2.

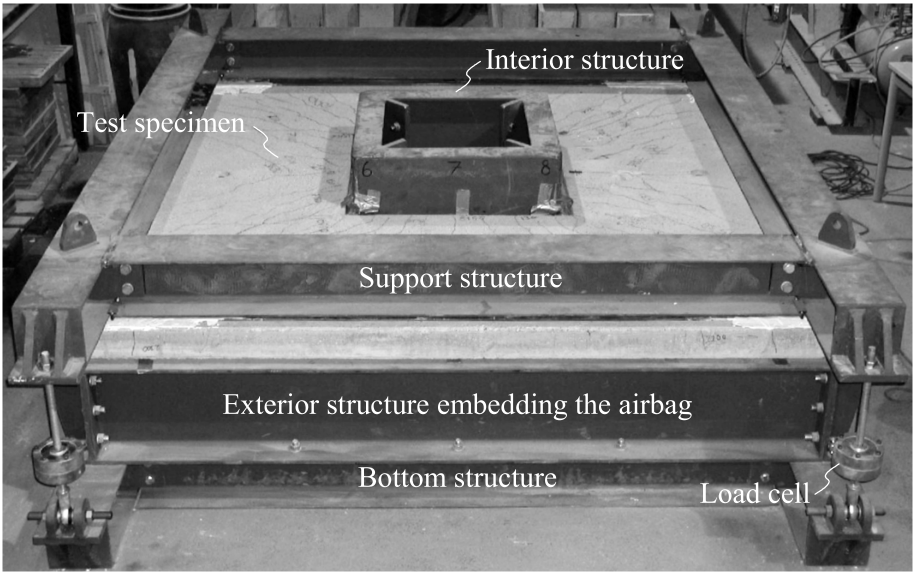


Figure 3.1 Test setup with slab placed on the bottom structure with airbags. Line support structure placed on top of the slab. The support structure is connected to the bottom structure through a load cell in each corner.

The use of airbags to create a distributed load is well tested and established at Luleå University of Technology. The method has been used for a long time to test the load carrying capacity of roof sheeting profiles of thin sheet plates.

The slab specimens are simply supported along their four edges. The loading area is 2.4×2.4 m i.e. somewhat smaller than the total area of the slabs. For the slabs with openings, the loading area is decreased in proportion to the area of the opening. A principal sketch of the test setup is shown in Figure 3.3.

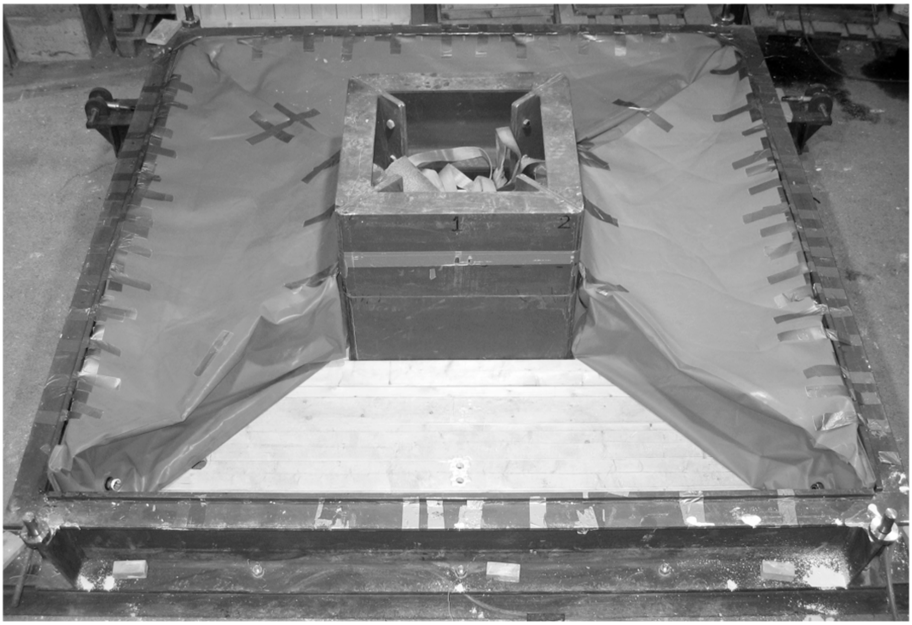


Figure 3.2 *System of airbags inside the embedding structure. An ordinary air compressor is used to fill up one of the airbags. The air can then circulate freely in the system through valves connecting the airbags (one of the airbags is removed in the figure).*

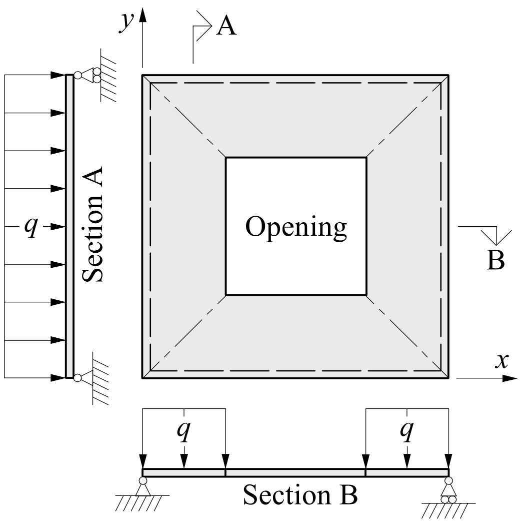


Figure 3.3 *Principal sketch of the test setup.*

The distributed load is calculated from the reaction forces measured by four load cells i.e. one in each corner. Since the slab is loaded “upside-down”, springs are mounted in each corner to eliminate the self-weight of the support structure. Both deflections and strains are measured according to a system of location lines defined over the slab surface. Figure 3.4 shows the location of the measuring points at defined lines for different slab configurations.

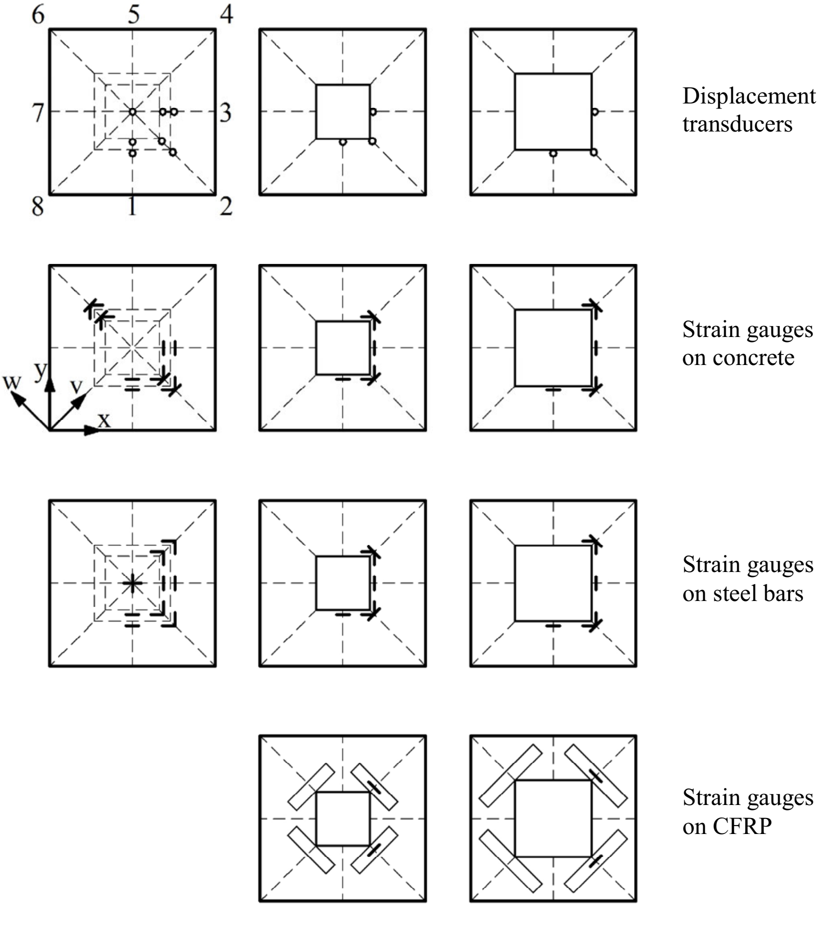


Figure 3.4 Instrumentation of the slabs depending on size of openings (no, small or large). The numbers designate the location lines and the letter the direction of action in the slab’s plane.

The deflections are measured with Linear Voltage Displacement Transducers (LVDTs). The strains are measured with Strain Gauges (SGs); on the concrete using 50 mm glued SGs, on the steel reinforcement using 10 mm welded SGs, and on the CFRP sheets using 10 mm glued SGs, see Figure 3.5.

A typical setup of the instrumentation for a slab with an opening is shown in Figure 3.6.

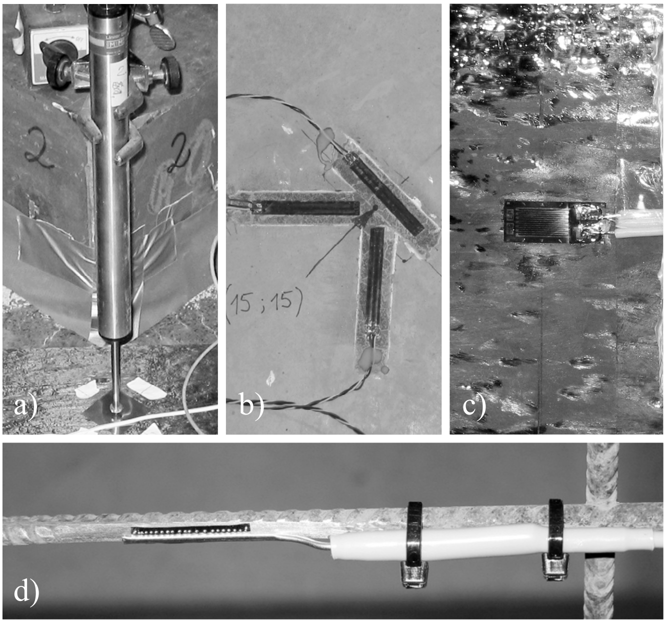


Figure 3.5 Measurement gauges: a) LVDT, b) glued 50 mm SG for the concrete, c) glued 10 mm SG for the CFRP sheets and d) welded 10 mm SG for the steel reinforcement.

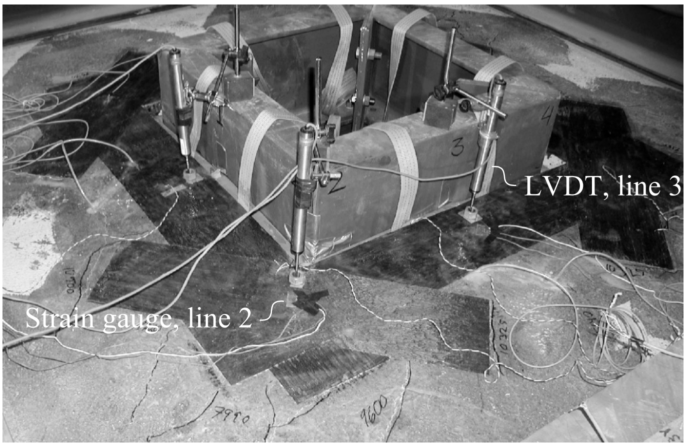


Figure 3.6 A typical setup of the instrumentation for testing a slab with an opening, Ericsson & Larsson (2003).

3.3 Specimen

A total number of 11 slabs (2600×2600×100 mm) were manufactured and cast in four batches, with a designed 28 days characteristic compressive strength of f_{cck} = 40 MPa. Nine cubes (150×150×150 mm) were cast for each batch to measure the compressive strength at 28 days, and the compressive and splitting strength at the time of testing. The concrete surfaces of the slabs to be strengthened with CFRP sheets were sandblasted and cleaned properly with compressed air before bonding.

All slabs are reinforced with welded steel fabric, Nps 50 Ø 5 - s 150, using a concrete cover of 20 mm. Two reinforcement bars with the same nominal characteristic yield strength f_{yk} = 510 MPa, are added in the slabs with cast openings placed in 45-degrees angle as shown in Figure 3.7a. A sample set of three individual steel bars from the welded fabric have been tested to evaluate the tensile strength at the 0.2 %-limit $f_{0.2}$, as is normal for cold worked steel.

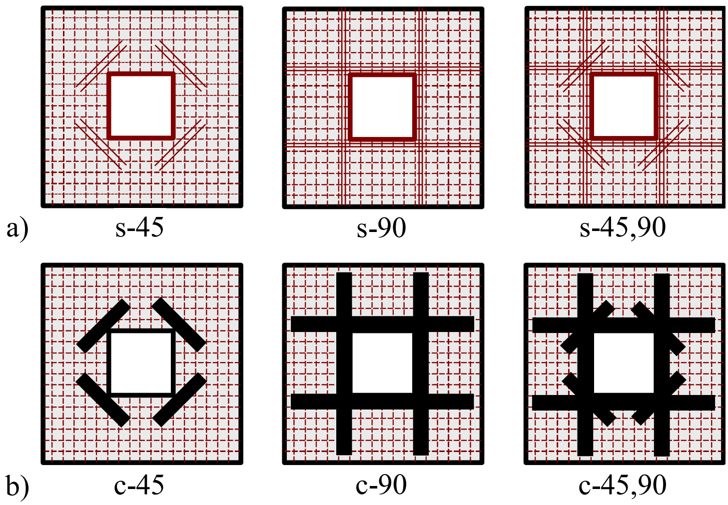


Figure 3.7 Three different arrangements to strengthen a slab with a) additional steel reinforcement due to a cast opening, or b) CFRP due to a sawn-up opening.

The homogeneous reference slab and the slabs with cast openings are designed according to the Swedish concrete code, BBK 04, for a uniformly distributed load of 15 kN/m^2 were the amount of steel reinforcement is calculated using the standard method, see e.g. Hillerborg (1990). The other test specimens have the same distribution of steel reinforcement as the homogeneous slab.

The amount of CFRP for the CFRP strengthened specimen is calculated from the required steel reinforcement according to the simplified method described in section 4.1. Minimum amount of steel reinforcement for cracking due to shrinkage and temperature changes is omitted in the design. The complete test program is given in Table 3.1 and Figure 3.8.

Table 3.1 *Designation and description of the specimens. H = homogeneous slab, S = small opening (0.85×0.85 m), L = large opening (1.20×1.20 m), w = weakened, s = steel reinforced, c = CFRP strengthened, 45 = strengthening applied in corners with 45°, 90 = strengthening applied in 2-directions orthogonally along the opening, and 45, 90 = strengthen with both former configurations of strengthening.*

Designation	Description
H	*Homogeneous slab:* Reference slab traditionally reinforced.
Sw	*Slab weakened by a sawn-up small opening (0.85×0.85 m)*
Ss-90	*Slab with a steel strengthened small opening:* Cast with a small opening traditionally steel reinforced along the opening
Ss-45	*Slab with a small opening strengthened in corners with steel reinforcement:* Cast with a small opening, steel reinforced in corners with 45°
Sc-90	*Slab with a CFRP strengthened small opening:* Sawn-up small opening strengthened with CFRP sheets along the opening
Sc-45	*Slab with a CFRP strengthened small opening:* Sawn-up small opening strengthened with CFRP sheets in corners with 45°
Sc-45, 90	*Slab with a CFRP strengthened small opening:* Sawn-up small opening strengthened with CFRP sheets along the opening and in corners with 45°
Lw	*Slab weakened by a sawn-up large opening (1.20×1.20 m)*
Ls-45	*Slab with a large opening strengthened in corners with steel reinforcement:* Cast with a large opening, steel reinforced in corners with 45°
Lc-90	*Slab with a CFRP strengthened large opening:* Sawn-up small opening strengthened with CFRP sheets along the opening
Lc-45	*Slab with a CFRP strengthened large opening:* Sawn-up large opening strengthened with CFRP sheets at the opening's corners with 45°
Lc-45, 90	*Slab with a CFRP strengthened large opening:* Sawn-up small opening strengthened with CFRP sheets along the opening and in corners with 45°

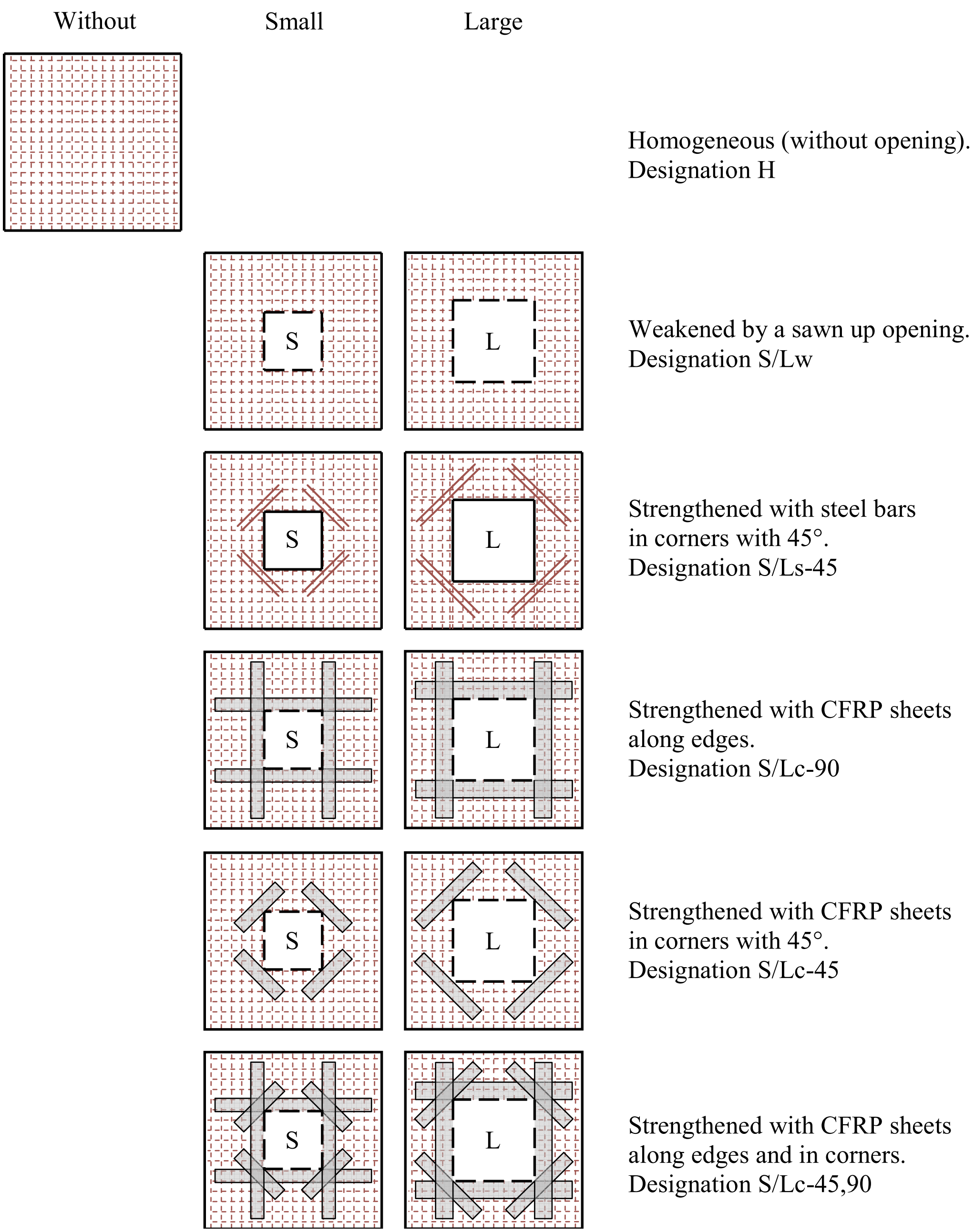


Figure 3.8 Experimental program. An opening drawn with solid lines is cast, and with dashed lines is sawn up.

Square openings of two different sizes (0.85×0.85 and 1.2×1.2 m) were sawn-up in centre of each slab using a mobile concrete wet saw. In order to avoid initiation of cracks in the corners during the tests, a hole (Ø 70) was first drilled out at each corner as a guide prior to sawing. The sizes of the openings were chosen to be slightly larger than the limit defined in BBK 04 for a small opening ($1/3 \cdot 2.4 = 0.8$ m < 0.85 and 1.2 m) in order to investigate the effect of different sizes of the openings. This is explained more thoroughly in next section.

The concrete splitting and compressive strengths, shown in Table 3.2, are the mean values of three test cubes. The tensile strength was evaluated from the splitting strength according to BBK 04 (0.8 of the splitting strength). In addition, torque tests were conducted on sawn-out slab parts from every batch that included any specimen to be strengthened with CFRP sheets, to evaluate the surface shear strength of the concrete.

Table 3.2 Average concrete strengths from splitting and compressive tests of three cubes and surface shear strength from six torque tests.

Slab	Cast batch	Date for casting	Date for cube test	Splitting strength [kN]	Tensile strength [MPa]	Compressive strength [MPa]	Shear strength [MPa]
H, Sw	1	22/09/03	20/10/04	3.95	3.16	46.5	–
S/Ls-45	2	06/10/03	15/06/04	3.90*	3.12*	55.3*	6.2
S/Lc-90, S/Lc-45	3	14/10/03	02/07/04	4.70*	3.76*	56.3*	5.4
Lw	4	24/10/03	21/11/03	–	–	50.6	7.2
S/Lc-45,90	5	04/11/03	14/12/04	4.54*	3.63*	59.0*	8.0

The slabs with a sawn-up opening are strengthened using Sto FRP Sheet C, with two different weights: 200 g/m^2 and 300 g/m^2. Table 3.3 gives the nominal material properties of the CFRP sheets, and the length and width of the applied CFRP are shown in Table 3.4. The material properties of the used primer and adhesive are shown in Table 3.5.

Table 3.3 Nominal material properties of CFRP sheet.

Product	Sheet thickness [mm]	Young's modulus [GPa]	Tensile strength [MPa]	Rupture strain [%]
300S*	0.17	228	3600	1.5
200S*	0.11	228	3600	1.5

* number denotes the mass per cross sectional area [g/m^2] and S qualifies high strength.

Table 3.4 Location, width and length of the applied CFRP sheets.

Slab	Length of opening [m]	Type of Sheet	Location in relation to edges of opening	Width [mm]	Length [m]
Sc-45	0.85	200S	In corner (45º)	195	0.85
Sc-90	0.85	300S	Along (0/90º)	185	2.30
Sc-90,45	0.85	200S	In corner (45º)	195	0.85
		300S	Along (0/90º)	185	2.30
Lc-45	1.20	200S	In corner (45º)	195	1.20
Lc-90	1.20	300S	Along (0/90º)	240	2.30
Lc-90,45	1.20	200S	In corner (45º)	195	1.20
		300S	Along (0/90º)	240	2.30

Table 3.5 Nominal material properties of primer and adhesive.

Product	Adhesiveness to concrete [MPa]	Young's modulus [GPa]	Tensile strength [MPa]	Shear strength [MPa]
Primer StoBPE 50 Super (A+B)	17	–	–	–
Adhesive StoBPE Lim 417 (A+B)	–	2	50	17.6

A typical test setup with the CFRP sheets applied along the opening is shown in Figure 3.9

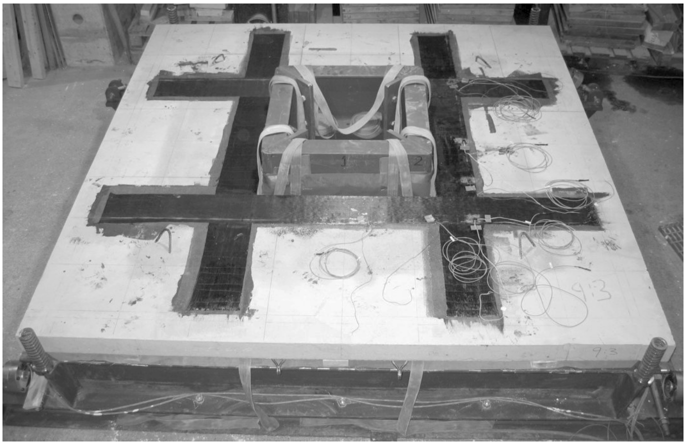


Figure 3.9 Slab strengthened with CFRP sheets applied along a sawn-up small opening.

3.4 Test procedure

The airbag was carefully arranged to give a well-distributed uniform load during the whole test. A thin protective layer of polyester fabric was placed between the airbag and the concrete surface to protect the airbag at slab failure. Special attention to the contact surfaces to achieve an even distribution of the reaction forces along the supports was given. An air compressor was used to fill the airbag with air at an approximate loading rate of 40 N/sec (417 Pa/min for the homogeneous reference slab). The load, displacement and strains were continuously measured and recorded by a computerized acquisition system until failure occurred. Both the crack propagation and the crack distribution were observed and registered throughout the test.

3.5 Experimental result

In Figure 3.10 and Figure 3.11, the load-deflection relationships until failure i.e. collapse for all experimentally tested slabs are shown.

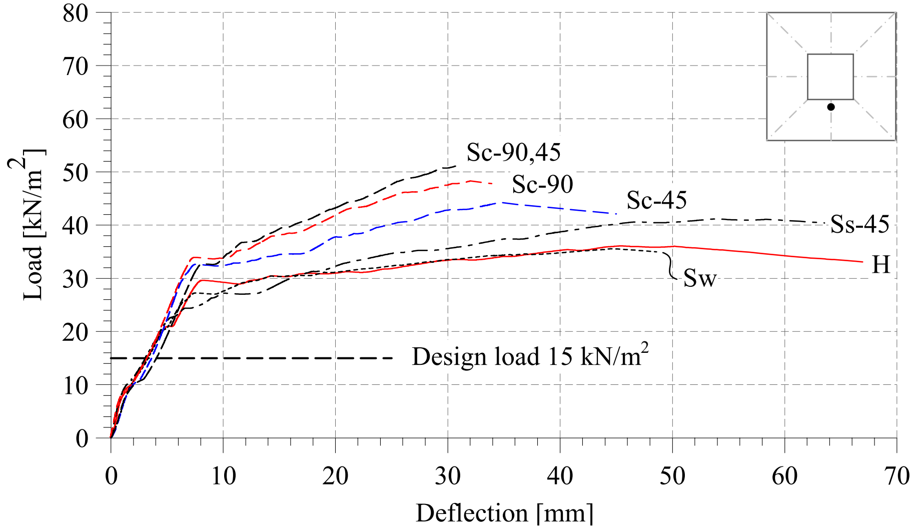


Figure 3.10 *Load-deflection relationships at the midpoint close to the opening, for slabs with a small opening. The load capacities of the tested slabs are compared with the design load of 15 kN/m²* *for a slab without an opening (H).*

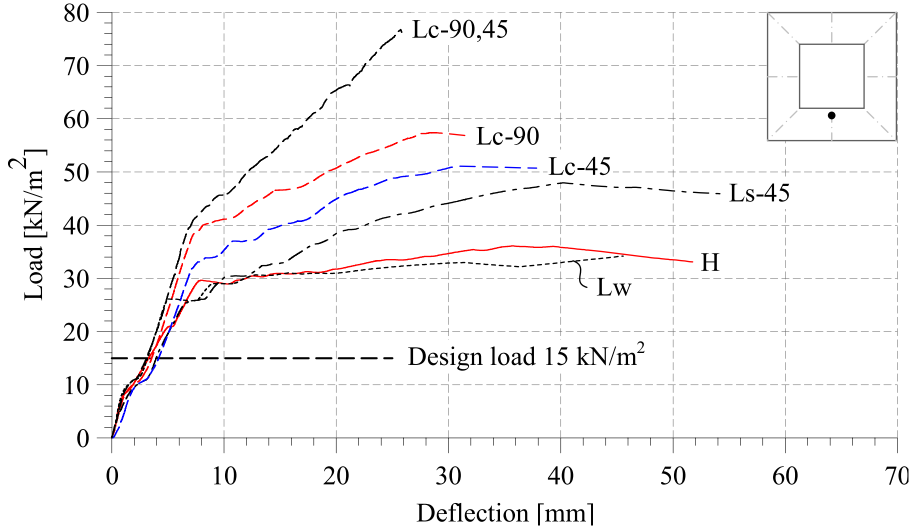


Figure 3.11 *Load-deflection relationships at the midpoint close to the opening, for slabs with a large opening. The load capacities of the tested slabs are compared with the design load of 15 kN/m²* *for a slab without an opening (H).*

The general response for all tests is similar. First, the contact between the support and the slab is developed until a load of approximately 8 kN/m^2 where a small bump can be noticed. The bump is believed to be due to adjustment of the bolted joints in the test rig during the initial loading phase. The first crack in the slabs is observed between 20 – 30 kN/m^2. For the homogeneous slab, the cracking starts in the middle of the slab, in contrast to the slabs with openings where the first crack is initiated in 45° in the vicinity of one corners of the opening.

The observed load levels for the first observed crack are much higher than the design load. This is probably due the membrane forces that develop in slabs with relatively high value of the ratio between the thickness and span.

As the load increases, more cracks initiates and continues to grow. For the CFRP strengthened slabs, the cracks were narrower and more widely spread in comparison to the cracks in the steel reinforced slabs.

All strengthened slabs with openings show, independently of the size of the opening, a considerably higher load carrying capacity than the homogeneous slab (H). Especially the slabs strengthened with CFRP (denoted Sc and Lc). The openings decrease the total available loading area. Thus, a uniform load expressed as N/m^2 gives a lower total load on a slab with an opening. A similar result is seen for the slabs strengthened with additional steel reinforcement in the corners (Ss-45 and Ls-45).

The slab strengthened with additional steel reinforcement due to a sawn-up large opening (Ls-45) show higher load carrying capacity and lower deflection at failure than the homogeneous slab (H). A probable reason for this is the larger lever arm for the reinforcement placed in the corners.

The slabs weakened by a sawn-up opening (Sw and Lw) show surprisingly, a similar load-deflection behaviour as the homogeneous slab, with the exception that the deflection at failure is lower (more brittle).

All the slabs with openings strengthened with CFRP shows similar load-deflection behaviour up to failure. However, the failure mode differs in comparison with the traditional steel reinforced slabs. The steel reinforced slabs show a more ductile response compared to the CFRP strengthened slabs. The mode of failure is also different. The dominating failure for the steel reinforced slabs is governed by the plastic behaviour of the steel reinforcement leading to large deformation before ultimate limit is reached.

The failure for the CFRP strengthened occurred when the CFRP fibres suddenly ruptured. However, the rupture is preceded by steel yielding and comparatively large deflections. The load carrying capacity at failure is considerable larger for the CFRP strengthened slabs.

The different load-deflection relationship for the CFRP strengthened slabs needs to be explained in more detail. S/Lc-45 are strengthened with CFRP sheets only in the corners at 45°. S/Lc-90 are strengthened with CFRP sheets along the edges of the opening. Finally, S/Lc-45, 90 are strengthened both in the corners and along the edges, see also Table 3.1 and Figure 3.8. Even though all CFRP strengthened slabs failed by fibre rupture, the load carrying capacity and the deflections at failure varies considerable between the different slabs. The most probable reason for this is the configurations of the fibre sheets. The highest load carrying capacity is observed for S/Lc-45, 90 where the crack propagation is hindered by three layers of CFRP sheets. This is illustrated in Figure 3.12.

The difference between the carrying capacity between S/Lc-45 and the S/Lc-90 is smaller although the S/Lc-90 configuration has somewhat higher capacity. The explanation is that the S/L-45 configuration has only one layer of CFRP sheet in the corners of the opening whereas the longitudinal sheets were overlapping in the corners of the opening. Furthermore, the slabs with only one CFRP sheets in 45° showed a tendency of bond failure, see Figure 3.12b.

There is also a noticeable difference of the load carrying capacity between the CFRP strengthened small and large opening where the latter slab configurations carried 20 - 50 % higher loads. Firstly, the specimens with the larger opening have a 15 % smaller loading area compared with the Sc specimens. Secondly, the behaviour of the specimens with a large opening is closer to a system of beams than a slab thus distributing the cracks along the side of the opening better compared to the specimens with smaller openings.

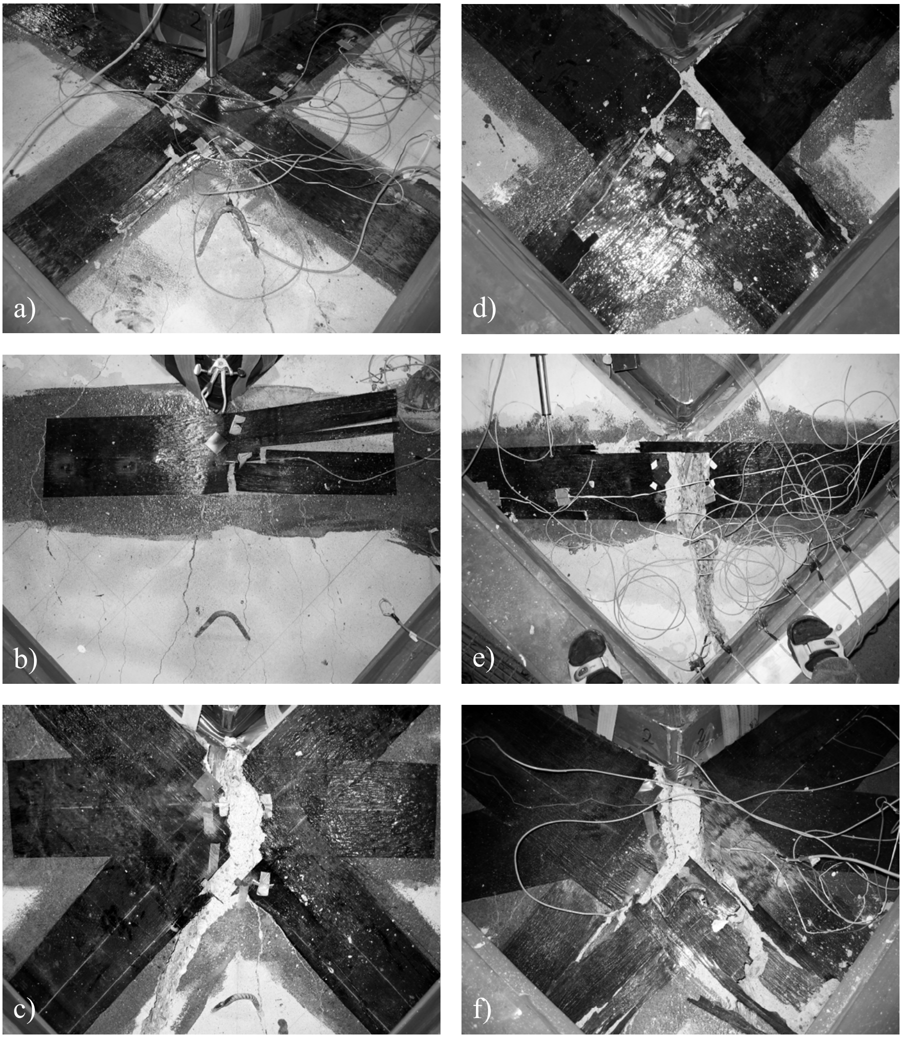


Figure 3.12 *Crack patterns in a corner of a slab at failure for different configurations of CFRP sheets, a) along a small opening (Sc-90), b) in corners of a small opening (Sc-45), c) along and in the corners of a small opening (Sc-45, 90), d) along a large opening (Lc-90), e) in corners of a large opening (Lc-45) and f) along and in the corners of a large opening (Lc-45, 90). The propagation of cracks is governed by the configuration of CFRP sheets.*

4 ANALYSIS

4.1 Introduction

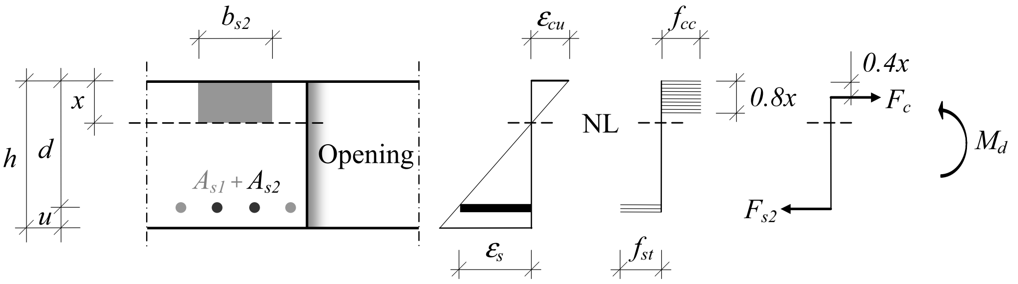
An accurate analysis of a CFRP reinforced cannot be made using theories based on limit state such as the yield line theory. The ultimate failure is based on the total load required to reach a yield mechanism for the slab. That implies that the slab must be able to redistribute the internal forces when the yield limit is reach locally in the slab.

CFRP strengthening behaves linear-elastic up to failure. Therefore, the ultimate failure for CFRP strengthened slab will depend on the strength of CFRP sheet in the most strained area, as has been showed in chapter 3, (if no debonding occurs). If the analysis and design is made according to the theory of elasticity we only have to prove that the capacity requirement is met locally in a slab strengthened with CFRP. In this chapter, a simplified method of estimating the necessary amount of CFRP in a linear elastic design of RC slabs is first presented. Then a numerical study of the experiments conducted in chapter 3 will be presented. Finally, the analytical design target and the numerical results will be compared with the experimental outcome.

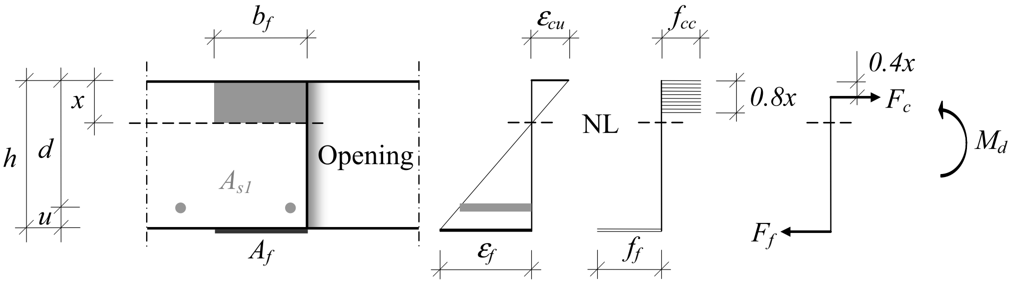
4.2 Simplified analytical method

A simplified method to estimate the necessary amount of CFRP for a structure is to calculate the required steel reinforcement designed with traditional methods and convert it to CFRP. The calculation is made by accounting for the difference in the stiffness between the cross sectional area of the CFRP sheet and the steel.

A more theoretically correct method, especially for slabs with openings, is to account also for the differences in the lever arms and the effective widths of the compression zone, see Figure 4.1.



a) Cross section of a RC slab manufactured with additional reinforcement bars A_{s2} *near an opening.*



b) Cross section of a RC slab strengthened with CFRP A_f *near an opening.*

Figure 4.1 Relationships between strain, stresses and internal forces in a cross section of a RC slab near an opening strengthened with a) additional steel reinforcement or b) CFRP.

The strengthening effect should be equal between the cross section with the additional steel reinforcement and the CFRP strengthened cross section. Hence, the moment capacity must also be equal

$$M_f = M_s \tag{4.1}$$

where M_f and M_s is the contribution to the moment capacity for the slab along the opening from CFRP and additional steel reinforcement, respectively. To calculate the necessary sectional area of the CFRP sheet A_f, equation 4.1 is expressed as the product of the sectional force F and the inner lever arm

$$F_{s2}\left(d-x\right)=F_f\left(h-x\right) \tag{4.2}$$

where the subscript $s2$ is the part of the steel reinforcement bars that will be replaced with the CFRP reinforcement (subscript f). Expressing forces as products of stresses and sectional areas yield

$$\sigma_{s2}A_{s2}\left(d-x\right)=\sigma_f A_f\left(h-x\right) \tag{4.3}$$

Applying Hooke's law gives the sectional area of the fibre as

$$A_f=\frac{E_{s2}\varepsilon_s\left(d-x\right)}{E_f\varepsilon_f\left(h-x\right)}A_{s2} \tag{4.4}$$

The relation between the strain in the steel and CFRP can be evaluated using Bernoulli's hypothesis in Figure 4.1, as

$$\varepsilon_f=\frac{h-x}{d-x}\varepsilon_s \tag{4.5}$$

If perfect bond between the concrete and the reinforcement is assumed, the expression for the sectional area of CFRP becomes only dependent on the level arms and the elastic modulus of the steel and the CFRP reinforcement

$$A_f=\frac{E_{s2}}{E_f}\left(\frac{h-u-x}{h-x}\right)^2 A_{s2} \tag{4.6}$$

where $d = h-u$ is the effective height and u is the distance from the bottom tensile side to the centre of gravity of the additional steel bars.

The differences in the effective width of the compression zone b between the CFRP and the substituted additional steel reinforcement can be iteratively determined. In the first step, the width of the contributing compression zone in the CFRP design b_{f0} is set equal to the width of the contributing compression zone belonging to the additional steel bars b_{s2}, and a new value of b_f is calculated. The iteration continues until the two values coincide. However, if $E_f \geq E_{s2}$, $b_f = b_{s2}$ can be used directly in the design since it then is on the safe side.

4.3 Numerical analysis

4.3.1 FE-model

Numerical analysis of the behaviour of two-way concrete slabs includes several nonlinear considerations, i.e. material, boundary conditions and geometry. Various concepts for describing the quasi-brittle mechanisms in reinforced concrete have been introduced in the FEM. These are well known concepts such as discrete crack and smeared crack approaches, or less used models such as inner softening bands, Tano (2001). In this paper, a damaged plasticity model is used for the concrete. The steel reinforcement is modelled as ideal-elastoplastic and the CFRP as linear elastic material until failure. Equally important in the numerical analysis are the boundary conditions. This is especially true in this study since the support conditions for the concrete slab cannot be prescribed. The analysis must include contact interactions to allow separation between the slab and the support.

Numerical analysis of reinforced concrete structures is customarily performed by static implicit FE solvers where the integration scheme is for example full Newton-Raphson. The solution is obtained from equilibrium iterations minimizing the error of the solution. The outcome is a reliable and stable solution. But this solving technique can have convergence problems in models that have a large degree of non-linearity, such as the two-way slabs in this paper. Apart from the constitutive models that are nonlinear, the support conditions must be handled by contact interactions. This adds complexity to the system that the implicit scheme is not able to take care of. An optional solver for these kinds of problems is a FE-program with an explicit time integration scheme. It is normally used for dynamic problems but can also be applied in static problems. Particularly, it is an efficient solver of contact problems.

Numerical simulations by using the FEM with explicit time integration can be very costly in terms of computer time if not certain adjustments are made. This comprises of either decreasing the simulation time or increasing the density of the material. These two remedies perform essentially the same thing; reducing the number of time steps in the global time integration. The number of time steps in the integration is set by the inherent critical time step, which is usually governed by element dimensions, the density and the dilatational wave speed of the material. The critical time step is normally very small, inducing a great number of time steps to be completed. Since the number of time steps is almost directly proportional to the computer time, it is desirable to decrease this number.

Reducing the number of time steps must be done very carefully, otherwise the simulation becomes unstable and the solution is not reliable. Care must also be taken in order to ensure that the inertia effects are kept within acceptable limits. For this model, an increase in the material density by a factor of 100, which means a decrease of computer time by approximately 10, does not have an apparent effect on the response of the model.

The slab is modelled by eight-node brick elements with reduced integration, the reinforcement in the concrete is represented by discrete truss elements, steel support plates are modelled by shell elements with reduced integration, and the CFRP sheets as membrane elements with no stiffness perpendicular to the fibres. The test rig is not a part of the FE model, i.e. the deformations in the test rig are not taken into account. The FE model of the slab strengthened with CFRP along a sawn-up opening is shown in Figure 4.2. Similar meshes are used for the other models.

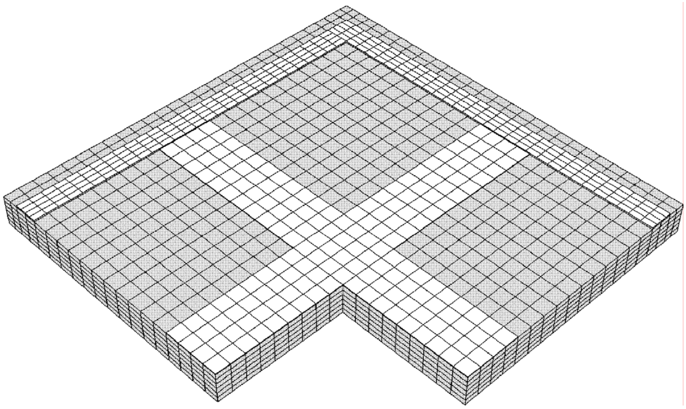


*Figure 4.2 FE model used to analyse the CFRP strengthened slab, Sc-*90.

Quasi-brittle materials e.g. concrete usually experience a sudden drop in the load carrying capacity during cracking and it generally leads to increases in the kinetic energy content of the response. In the explicit solution of the load-displacement response, oscillations will appear due to these inertia effects after the concrete has cracked significantly. Curve smoothing is used to average the outcome of the FE analysis.

4.3.2 Material models

Concrete

Typical behaviour of concrete during the growth of micro-cracks comprises of strain softening, progressive deterioration, volumetric dilatancy, and induced anisotropy. This complex response must be translated into phenomena, which can be described by continuum mechanics. It can be considered as a combination of unrecoverable plastic deformation, degradation of material stiffness, and the initiation, development, and interaction of defects. The plastic deformation and strain softening can be described by using classical plasticity theory. Continuum damage mechanics is used to model the stiffness degradation. Essentially, this means that the cracking of the concrete reduces the stiffness, e.g. the modulus of elasticity. The crack opening and closure is determined by fracture mechanics and plasticity.

The constitutive model used in this paper for the concrete is the damaged plasticity model included in ABAQUS, see Hibbit, Karlsson & Sorensen Inc. (2005). It is based on the work by Lubliner et al (1989) and Lee & Fenves (1998). The evolution of the yield surface is controlled by two hardening variables, one in tension and one in compression. Non-associated flow is assumed where the flow potential g is the Drucker-Prager hyperbolic function. For these functions, a couple of parameters must be defined; the dilation angle ψ and the eccentricity e for the flow potential are set to $\psi = 12°$ and $e = 0.1$. For the yield function, the ratio of initial equibiaxial compressive strength to uniaxial compressive strength σ_{b0}/σ_{c0}, and the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield for any given value of the first stress invariant such that the maximum principal stress is negative K_c the default values in ABAQUS are used, i.e. 1.16 and 2/3, respectively.

The concrete behaviour in tension is linear elastic until cracking is initiated and a strain softening response is assumed in the post failure region. The post failure behaviour is specified in terms of the stress-displacement response in order to minimize mesh sensitivity. It defines the tension softening behaviour and is described here by a bi-linear curve, see Figure 4.3. Similarly, the tensile damage is specified by an assumed linear relationship between the tension damage variable d_t and the crack opening δ. The maximum value of the damage variable d_{t0} is set to 0.9, and the maximum crack opening δ_0 is set to 0.115 mm, see Figure 4.3. The fracture energy G_f for mode I is the area under the softening curve.

The fracture energy is simply set to 100 N/m since the magnitude of G_f will not have any major influence on the outcome due to the embedded steel reinforcement.

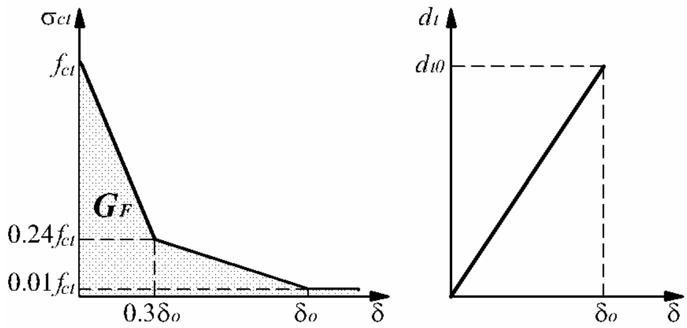


Figure 4.3 *Tension softening modelled by a bi-linear relationship and relationship between the tension damage variable,* d_t *and the crack opening,* δ.

In compression, the concrete behaviour is linear elastic until initial yield stress σ_{c0} is reached. The material enters the plastic regime with a strain hardening before the ultimate compressive stress (strength) σ_{cu}, followed by strain softening. The initial yield stress for the concrete in this paper is assumed to be 60 % of the ultimate stress and the typical plastic strain at ultimate stress is 0.2 %. In this analysis, the nonlinear part of the constitutive model in compression is somewhat unnecessary since the initial compressive yield stress will not be exceeded. The damage evaluation in compression is omitted since crushing does not occur.

Steel

The constitutive model for steel is assumed to be ideal elasto-plastic. Full bond between steel and concrete is assumed and the tension stiffening effect due to reinforcement is not accounted for in the solution. The steel support plates are considered to behave in a linearly elastic manner.

Carbon fibre reinforced polymer, CFRP

The CFRP sheet is considered as linear elastic until failure. The interaction between the concrete slab and the CFRP is modelled without considering debonding.

The dominating failure of the strengthened slabs in the experimental tests is rupture of the CFRP, see e.g. Figure 3.12, i.e. the bond does not have to be explicitly modelled in the numerical analysis.

4.3.3 Boundary conditions

Apparent symmetry in the model has been utilized resulting in a FE-model using only a quarter of the structure. Although, the behaviour after the first crack in the concrete dissolves this symmetry, it has been assumed that a quarter is adequate to give the response of the whole structure with sufficient accuracy.

The supports for the concrete slab cannot be modelled by using simple prescribed boundary conditions. This follows from the uplift of the corners of the slab. The supporting steel plates are included in the model and boundary conditions are prescribed for these. Contact interaction is introduced between the slab and the supports and the formulation is penalty based. The contact interaction is defined by the tangential behaviour, which in this case is set to be rough. This means an infinite coefficient of friction and no slip between the slab and the support plates.

In establishing correct boundary conditions in the numerical analysis, the geometrical imperfections of the specimens and the supporting frame recognised in the experiments needed to be investigated. Uneven concrete surface was smoothed by means of a plaster layer and remaining gaps were shimmed with thin steel plates, see Figure 4.4. However, this solved the problem only partially. The best contact between a specimen and the supporting structure was always reached in the corners.

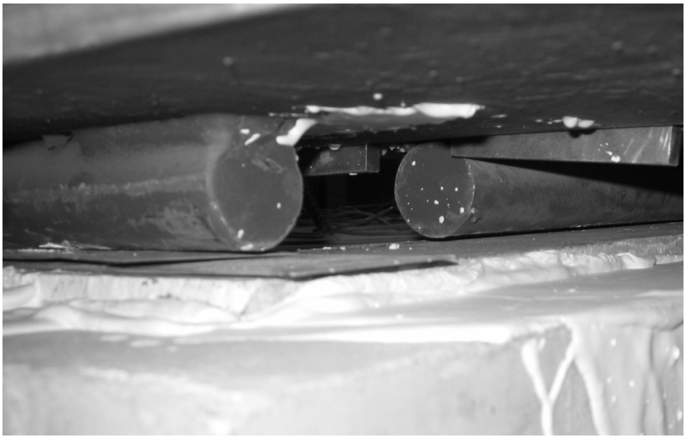


Figure 4.4 Example of support conditions indicating an uneven line support.

Although plaster and shims provided some support along the slab edges, it still allowed deformations. It can be assumed that the specimen was supported stiffly in the corners and elastically along the edges. This can be modelled as a set of discrete springs, proposed by Piotr Rusinowski (2005), or continually by applying simply supported deformable plates along slab edges. The latter method is used here and in Figure 4.5 the experimental deflection curves for a slab strengthened with CFRP along the opening, (Sc-90) is compared with the numerical analyses using stiff and elastic supports.

The analysis with elastic supports shows good agreement with the experiment in the elastic region but introduces dynamic instability in the explicit solution later on. This instability can cause inaccuracies and introduce “numerical” damage in the concrete model. Therefore, only the model with the stiff line supports is used.

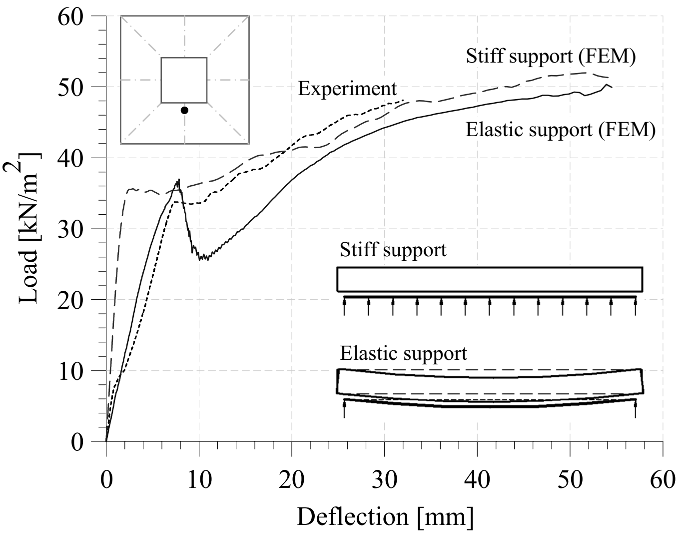


Figure 4.5 *Load-deflection relationships from the experiment and the two analyzed FE models, one with elastic supports and one with stiff supports. The results are compared at the midpoint close to the opening. Slab strengthened with CFRP along a small opening, Sc-*90*. The results for the FE-analyses are truncated at* 55 *mm.*

4.4 Comparison

4.4.1 Experimental versus analytical design loads

In Table 4.1, the load carrying capacity at failure and the crack load of the tested slabs are compared to the analytical design load of the homogeneous slab. The results show that the final load carrying capacity for all slab configurations are from approximately 1.4 to 4.1 times higher than the intended capacity of 15 kN/m^2. Particularly, the strengthening system with CFRP sheets applied along the openings (S/Lc-90) gives a final load carrying capacity that is 1.34 and 1.59 times higher than the final load carrying capacity for the homogeneous slab, respectively. Note that the load carrying capacities for these slabs are 2.22 and 2.83 times higher than the intended capacity, respectively.

Table 4.1 Crack and failure load compared to the analytical design load (15 kN/m²).

Slab	q_{crack} [kN/m²]	$q_{failure}$ [kN/m²]	$\frac{q_{crack}}{q_{design}}$	$\frac{q_{failure}}{q_{design}}$
H	7	36.1	0.5	2.41
Sw	20	35.6	1.3	2.37
Ss-45	18	41.2	1.2	2.75
Sc-45	27	44.2	1.8	2.95
Sc-90	34	48.3	2.2	3.22
Sc-90,45	30	51.1	2.0	3.41
Lw	26	34.2	1.7	2.28
Ls-45	18	48.0	1.2	3.20
Lc-45	32	51.1	2.1	3.41
Lc-90	41	57.4	2.7	3.83
Lc-90,45	41	76.8	2.7	5.12

4.4.2 Experimental versus numerical results

The result from the numerical analysis is compared with the outcome from the experiments in Figure 4.6 - Figure 4.12. The comparison of deflections in Figure 4.6 and the development of steel strains show that the global behaviour of the three analysed slabs in the numeric analysis can reproduce the experimental results. The main difference between the observed behaviour and the numerical analysis is found in the elastic region and at the beginning of the nonlinear behaviour. This difference is believed to be a consequence of the boundary conditions and the models inability to model the crack propagation in a proper way. Isotropic damage models are known to give a more brittle behaviour compared to other softening constitutive models, due to the stiffness' degradation in all directions, Tano (2001). The brittleness is manifested in the sudden drop directly after the onset of the nonlinear behaviour for the three slabs that are not strengthened with CFRP sheets, see Figure 4.6b. This drop is not present in the modelled slab strengthened with CFRP sheets, or in the reality.

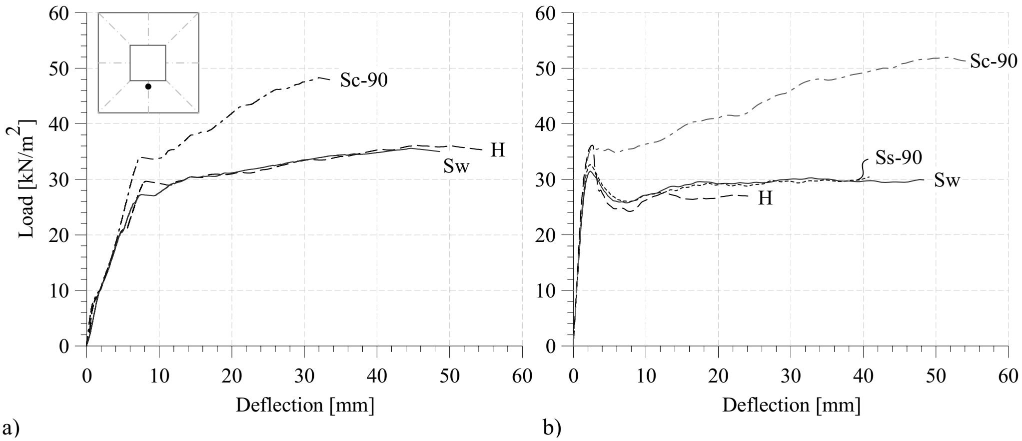


Figure 4.6 *Comparison of load-deflection relationships between results from a) the experiment and b) the FE-analyses at the midpoint close to the opening. Slabs with a small opening. The results for the FE-analyses, Sc-90 and the tested, H are truncated at 55 mm.*

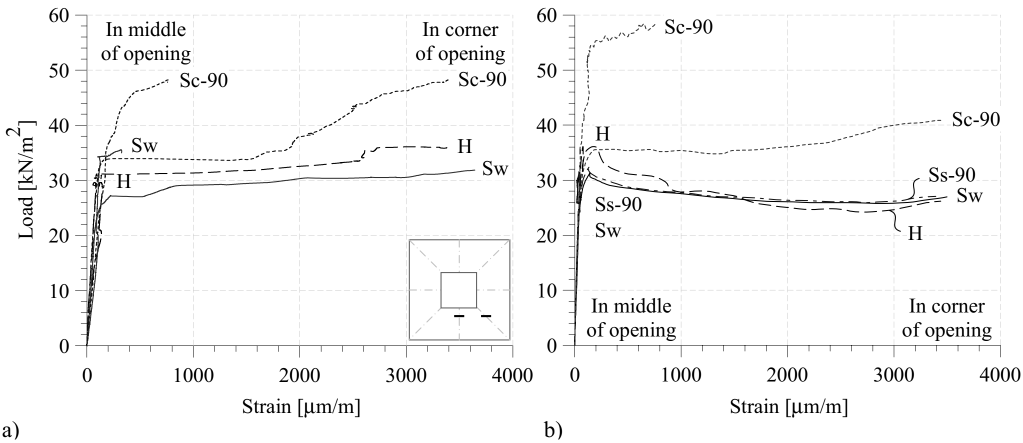
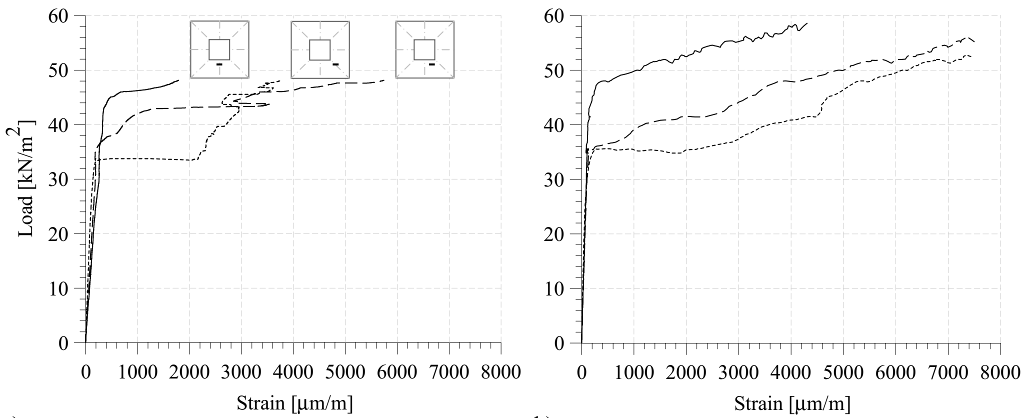


Figure 4.7 *Load-strain relationship in the steel reinforcement along the opening (i.e. in x-direction), for a) the experiment and b) the FE-analysis. The level of strains in the middle and in the corner of a small opening is compared to each other. The final load levels are somewhat to low in figure a, for the tested slabs H and Sw in the corner (the measuring range at these tests were set too low). The results from the FE-analyses, Sc-90, Ss-90, Sw and H, at the location in the corner, are truncated to make the comparison clearer.*

Figure 4.7 and Figure 4.8 shows that the strain development in the CFRP sheet was captured by the numerical model. Still, the failure of the CFRP could not be reproduced in the numerical analysis. The theoretical strain limit of the CFRP sheet was never reached in the FE analysis. This could be an effect of the biaxial state of stress that the CFRP is exposed to reducing the strain limit of the sheet. However, since the strain in the sheet is concentrated to where the cracks are localised it is difficult to measure the ultimate failure strain. The strain gauge must be located exactly over the area where the failure occurs.



a) b)

Figure 4.8 Load-strain relationships in the CFRP from a) the experiment and b) the FE-analysis. The strains are compared along the opening at three different locations; in the middle, at the corner of the opening, and between the middle and the corner of the opening. Slab strengthened with CFRP due to a sawn-up small opening, Sc-90. The results from the FE-analysis are truncated to make the comparison clearer.

Figure 4.9 shows the development of the strain distribution along one of the CFRP sheet. The strain distribution indirectly shows the development of bond shear stress along the sheet. Normally, the rule of thumb states that the bond development length for high strength CFRP reinforcement needs not to be greater than 200 mm, see Täljsten (2004). This is clearly the case in the numerical model and the proof can be found from the fact that the failure is in rupture of the CFRP sheet and not in debonding.

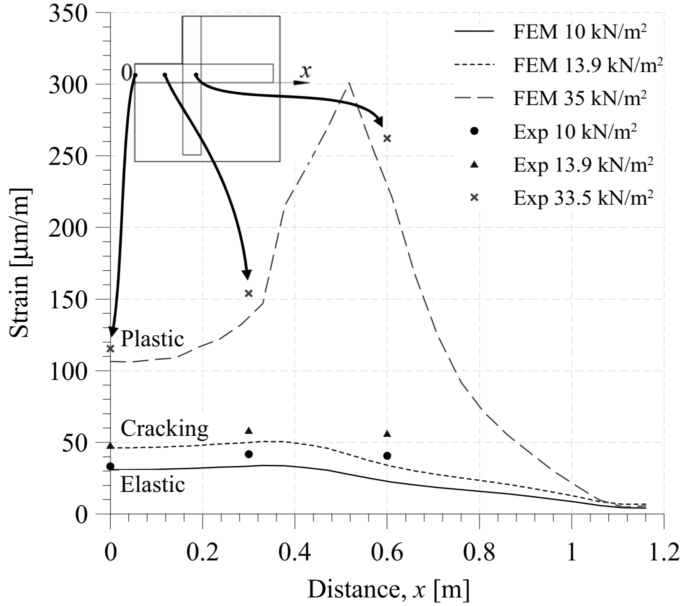


Figure 4.9 *Strain profiles at three selected stages in the CFRP along the opening at a distance* x *from the middle of the opening. The test results (discrete points) are compared to the FE analyses (curves). Slab with a sawn-up small opening strengthened with CFRP, Sc-90. The stages are selected at load levels in the elastic area, just at the cracking and when just reaching the plastic area.*

The strain distribution over the cross section is shown in Figure 4.10 - Figure 4.12. The figures show that the Bernoulli hypothesis is clearly valid and that the height of the compression zone is decreasing when the behaviour becomes non-linear. The strains measured in the tests are larger than the strains from the numerical analysis. This is especially apparent in the non-strengthened slab. This is believed to be a consequence of the difference in crack localisation between the experiments and the numerical analyses. In the experiments, a few localised cracks appear in contrast to the numerical analyses where a “plastic” region of cracks is produced.

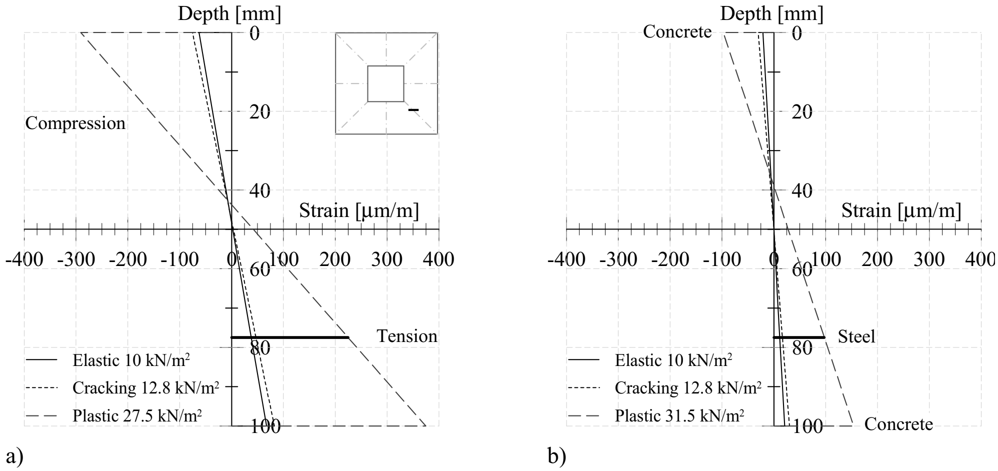


Figure 4.10 *Strain distribution ε_x through the slab thickness in a corner of the opening, for a) the experiment and b) the FE analysis of the slab weakened by a small sawn-up opening, Sw. The strains are measured at three selected stages as discrete values in the concrete on the compression side, in the steel and in the concrete on the tension side, respectively. The experiments show higher strain due to more localized yielding in the steel.*

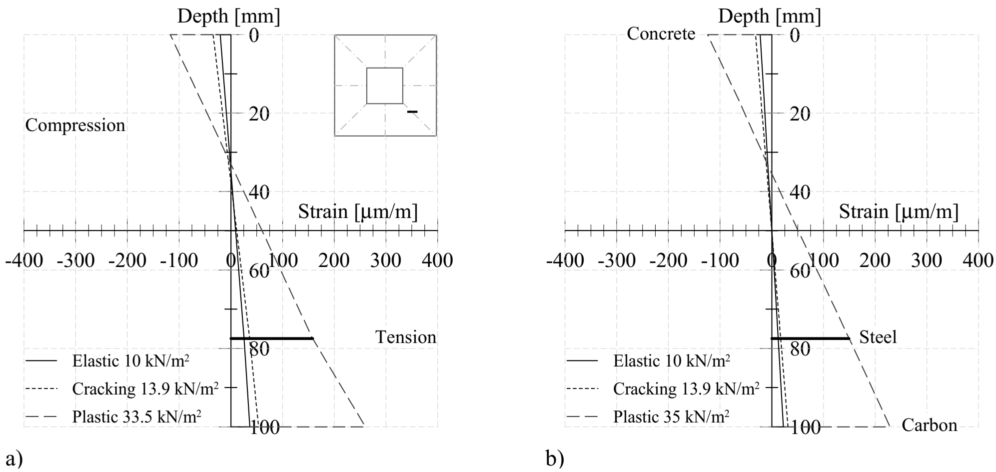


Figure 4.11 *Strain distribution ε_x through the slab thickness in a corner of the opening, for a) the experiment and b) the FE analysis of the slab strengthened with CFRP along a sawn-up small opening, Sc-90. The strains are measured at three selected stages as discrete values in the concrete on the compression side, in the steel and in the CFRP on the tension side, respectively.*

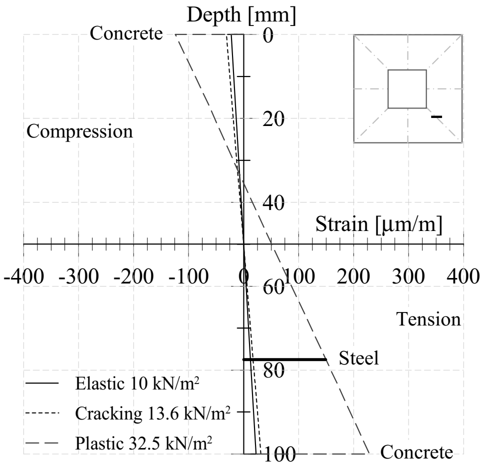
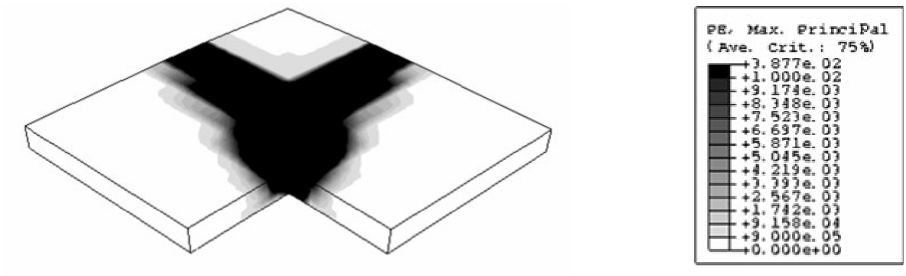
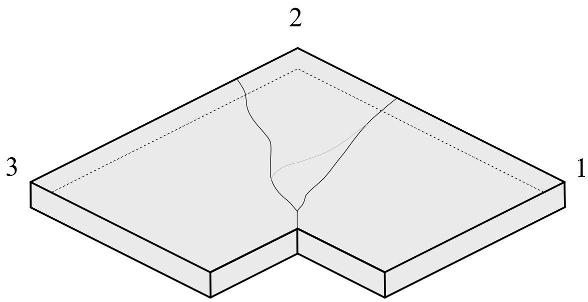


Figure 4.12 Strain distribution ε_x through the slab thickness in a corner of the opening, for the FE-analysis of a slab strengthened with additional reinforcement along a cast small opening , Ss-90. The strains are measured at three selected stages as discrete values in the concrete on the compression side, in the steel and in the concrete on the tension side, respectively.

In the CFRP strengthened slab, the cracks are more evenly distributed resulting in a smoother strain field due to the CFRP. Therefore, a better correspondence is found between the numerical analysis and the experimental result. This is somewhat reflected in the numerical analysis of the strain localisation, see Figure 4.13-Figure 4.15. The figures show the distribution of the plastic strains at the final load step for the three analysed slabs, in comparison to the crack patterns at experiments.



a) Final load step 29.9 kN/m^2, FE analysis



b) Failure load 35.6 kN/m^2, Experiment

Figure 4.13 Comparison between a) the maximum principal strain achieved in the FE analysis and b) the final propagation of cracks in the experiment. Results are shown for a quarter of a slab weakened by a sawn-up small opening, Sw.

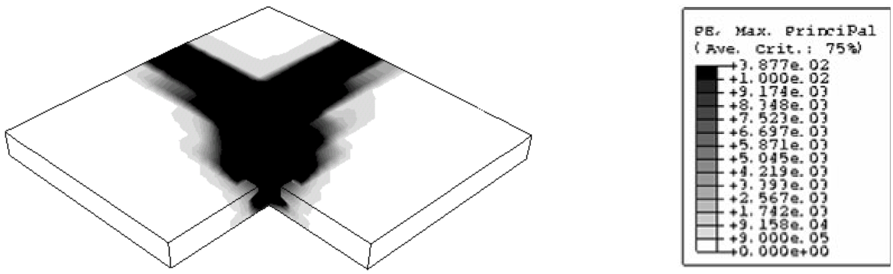
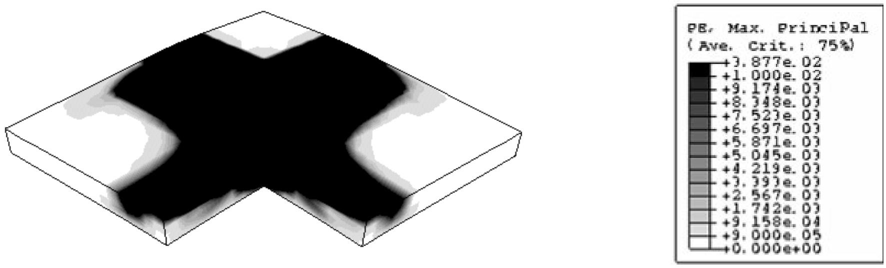
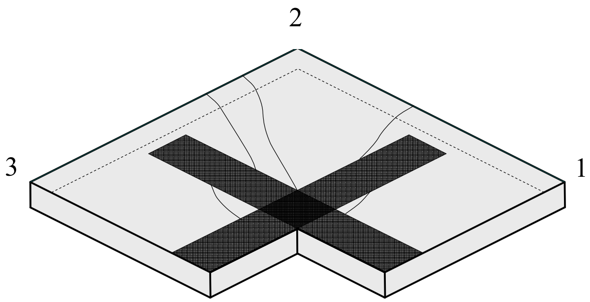


Figure 4.14 Maximum principal strain achieved in the FE-analysis at the final load step 29.7 kN/m^2. Results are shown for a quarter of a slab strengthened with additional steel reinforcements due to a cast small opening, Ss-90.



a) Final load step 58,6 kN/m^2, FE analysis



b) Failure load 48,3 kN/m^2, Experiment

Figure 4.15 Comparison between a) the maximum principal strain achieved in the FE-analysis and b) the final propagation of cracks in the experiment. Results are shown for a quarter of the slab strengthened with CFRP due to a sawn-up small opening, Sc-90.

In Figure 4.16, the slab strengthened with CFRP along the opening is shown at ultimate failure, and Figure 4.17 shows the crack pattern in a corner of the weakened slab and the strengthened slab. The “plastic” region in CFRP strengthened slab is more widely spread compared to the other analysed slabs. This is also in accordance with the observed behaviour in the experiments.

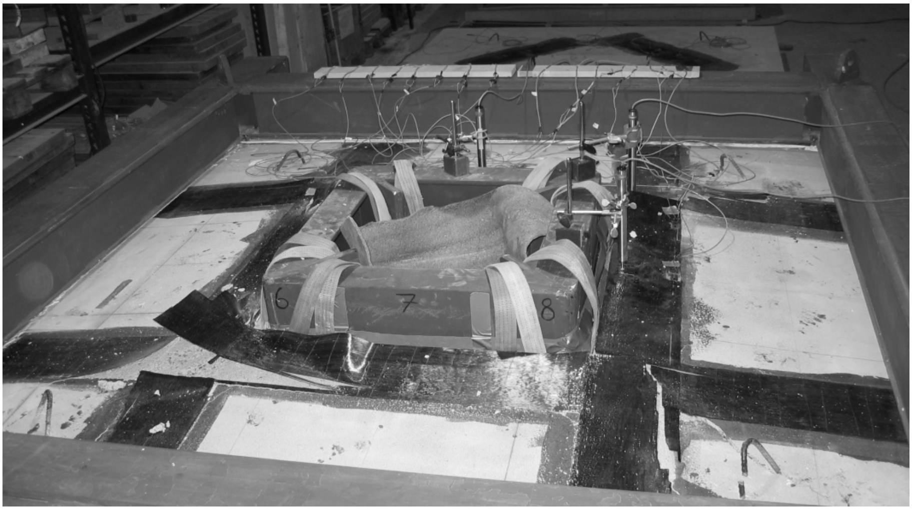


Figure 4.16 The final failure of the slab strengthened with CFRP due to a sawn-up small opening, Sc-90.

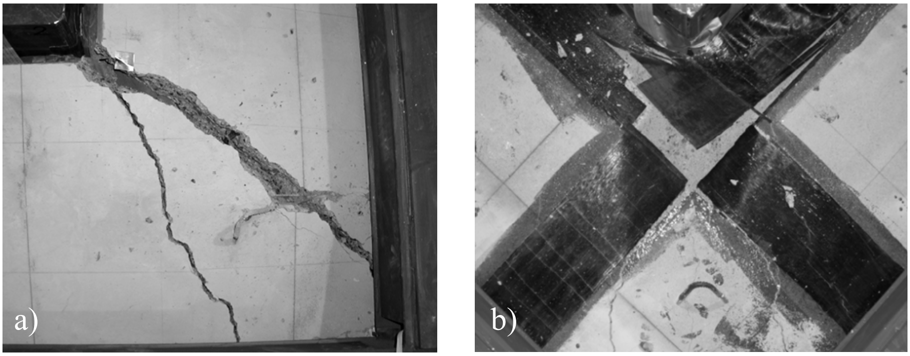


Figure 4.17 Typical crack patterns in corners at failure of a) the slab weakened by a sawn-up small opening, Sw (line 2) and b) the slab strengthened with CFRP due to a sawn-up small opening, Sc-90 (line 6).

5 CASE STUDY – STRENGTHENING OF A COURTYARD DECK

5.1 Background

In this chapter, a case study of a strengthened concrete courtyard deck is presented. The whole procedure from condition assessment through following up to final control is discussed in practical terms. An overview of the courtyard deck, situated in the centre of Vällingby, Stockholm, is shown in Figure 5.1.

Figure 5.1 Part of Courtyard deck at Vällingby Centre in Stockholm, Sweden. The stand is seen to the left. Photo KE-gruppen (2005).

A large reconstruction of the premises underneath the deck was planned, and a part of the interior supporting wall needed therefore to be removed to make place for a turning bay for trucks. Due to the removed wall, the deck had to be strengthened. Several different strengthening methods were discussed but the final method selected was a CFRP strengthening system.

5.2 Design and strengthening

The design of the strengthening system for the courtyard deck is divided into the suggested procedure outlined in chapter 2.4:

- Evaluation, i.e. condition assessment of the courtyard deck and plans for the future. This also includes a preliminary analysis of the need for strengthening and possible requirements on the strengthening system.
- Strengthening analysis; i.e. to determine the existing as well as the needed capacity for the courtyard deck.
- Strengthening method and strategy; what type of strengthening system do we apply, i.e. steel plate bonding, CFRP plate bonding, external prestressing cables? How will the strengthening be applied, do we strengthen before the supporting walls of the courtyard deck are removed or after?
- Strengthening work and follow up and control; Establish work instructions and a control plan for the strengthening work.

5.2.1 Evaluation of existing conditions

Site conditions

In Figure 5.2 the reconstruction plan of the courtyard deck is shown. The deck consist of a 0.6 m thick flat concrete slab supported by reinforced concrete columns, walls and a girder-column system. The concrete deck is made of K40 concrete and reinforced with Ks 60 reinforcement using the nominal diameters of 10, 12 or 16 mm, and a concrete cover of 25 mm in main reinforcing direction. The characteristic values for the concrete and the reinforcement is shown in Table 5.1.

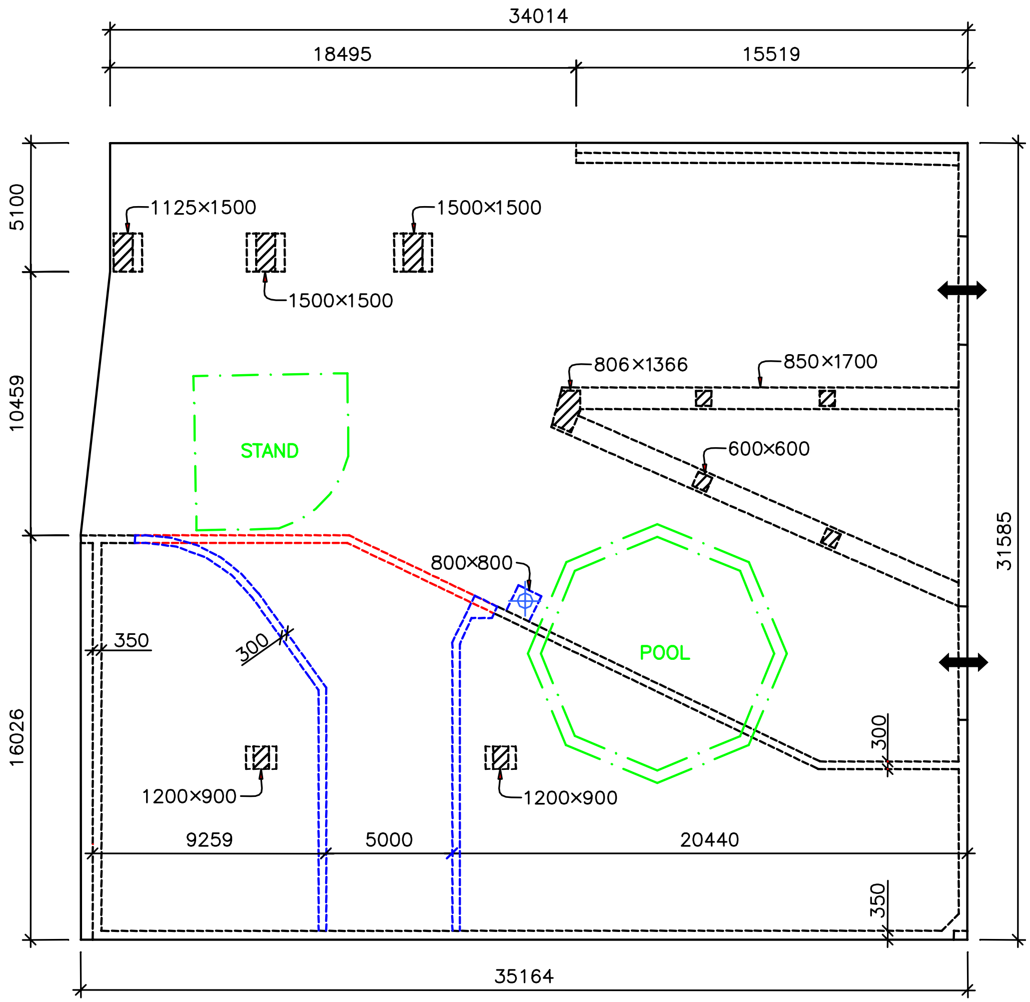


Figure 5.2 *Plan of the courtyard deck. The walls to be removed are drawn with red colour and the new walls to be build with blue colour. The green lines mark an existing stand and a pool on top of the slab that will be kept. The dimensions are in millimeters.*

Table 5.1 Characteristic values of concrete and reinforcement according to the Swedish code BBK 04.

Material	Young's modulus [GPa]	Tensile strength [MPa]	Compressive Strength [MPa]
Concrete quality K40	32,0	1,95	28,5
Steel reinforcement Ks 60, ∅10, 12 & 16	200	600	–

The permanent loads consist of the self weight, the superstructure and a layer of gravel (16 kN/m^2) to protect the concrete surface. It was decided to exchange the gravel for LECA (Light Expanded Clay Aggregate) to reduce the surface load on the slab to 10 kN/m^2. The live load consisted either of an imposed load of 5 kN/m^2 or two traffic loads, in total six point loads with 40 % of an heavy vehicle axle load, 0.4·210/2 = 42 kN. The imposed load was a standard load at time for design of courtyard decks (today 4 kN/m^2). The traffic load is according to the former Swedish code, BBK 04 (2004) and represents a smaller maintenance vehicle with three axles. The loads used in the strengthening analysis are summarised in Table 5.2, and the live loads' placement and extension is shown in Figure 5.3

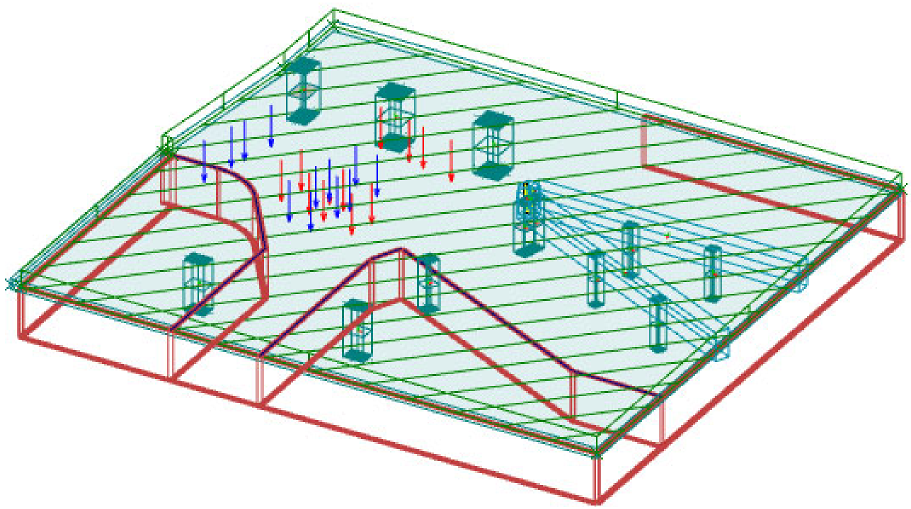


Figure 5.3 Investigated live loads, an imposed load (5 kN/m^2) and a traffic load (6×42 kN). For the traffic load, two positions are investigated (red and blue), each with two vehicles.

Table 5.2 Design loads.

Load case	Permanent load	Variable load
Superstructure	24 kN/m^3	
LECA	10 kN/m^2	
Imposed load		5 kN/m^2
2 × Traffic load		6×42 kN

Preliminary analysis and requirements

The initial evaluation was carried out by traditional hand calculations and by the use of simple software programs. These calculations showed that a lot of reinforcement was missing in the top of the slab, see Figure 5.4. This was mainly due to lack of punching capacity (blue marked areas) and moment capacity in top of the slab i.e. for hogging moments (red marked areas).

The punching capacity was roughly estimated in corners of the interior supporting wall by imaginary columns (1×1 m). The punching force Pc can then be estimated from a simplified equation proposed by Nylander & Kinnunen (1960)

$$P_c = 4c^2 q \qquad 5.1$$

where $4c^2$ is an assumed area of the slab exposed to a uniform load q that will be carried by the column. The required amount of reinforcement can then be estimated from design tables and compared with the existing amount.

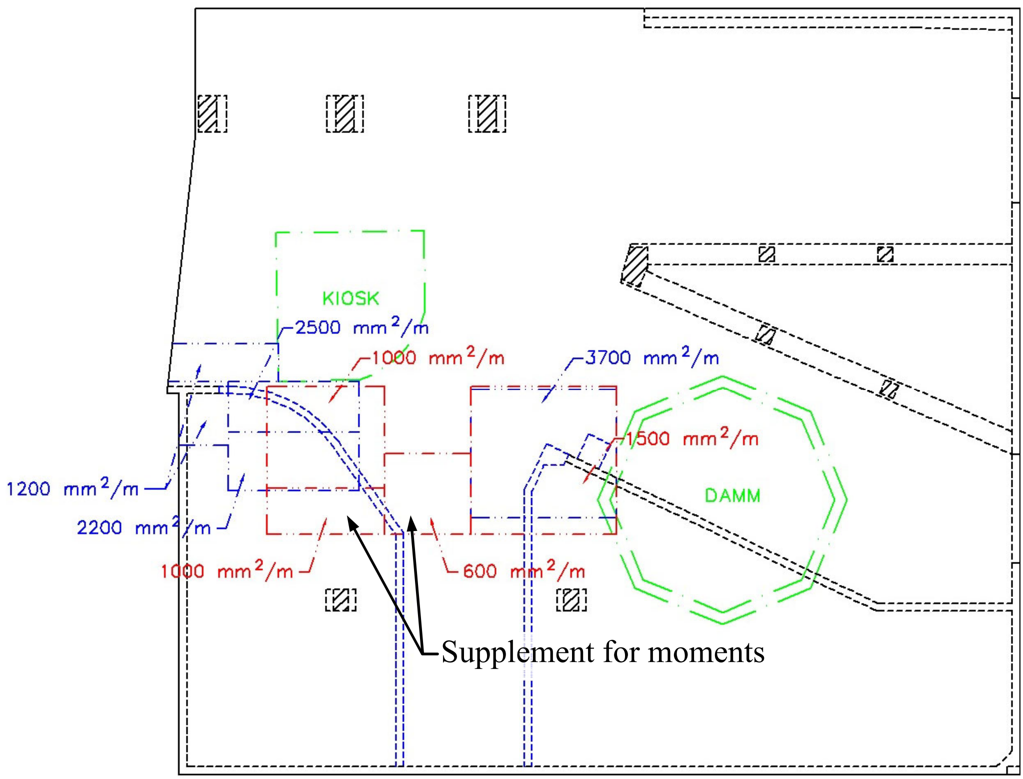


Figure 5.4 Areas with missing amount of top reinforcement in x and y-direction due to hogging moments and punching according to the initial calculation. The amount of missing reinforcement is based on a steel quality of Ks 60 (f_{yk} = 600 MPa).

From a structural point of view, the most advantageous way to strengthen the courtyard deck would be by underpinning the slab with a girder-column system. However, the client required that the strengthening system was not going to disturb the planned activity in the floor beneath the courtyard deck. A long and therefore high supporting beam in place of the removed wall would decrease the clear height and obstruct the intended use of the premises. It was therefore decided to investigate if it was possible to strengthen the slab from the upper side. The selection of strengthening methods was reduced to plate bonding techniques either by steel plates of CFRP plates and/or using NSMR. An other alternative could have been to cast on a concrete layer with additional reinforcement. In this case, extra load had been added to the total dead load.

5.2.2 Strengthening analysis

Introduction

It is often assumed that a structure before reconstruction has the required capacity to resist the existing loads and that any eventual strengthening actions only have to be designed for the additional forces acting on the structure due to reconstruction. The drawback from this kind of simplified assumption is that existing surplus capacities cannot be utilized at the analysis. Furthermore, reconstruction work can also change boundary condition and stiffness that will redistribute internal forces to areas outside the primary strengthening zones. Therefore, to better understand the behaviour of the strengthening, the analysis can be made with Finite Element Methods (FEM). The advantages of using FE-analysis are that:

- Areas of surplus or deficit capacities can be identified for the whole non-strengthened structure.
- The influence of the whole reinforced structure is taken into account i.e. redistribution of forces at an analysis of local phenomena.

It can be necessary to analyse non-linear phenomena such redistribution of internal forces due to cracking to fully take the advantages of existing capacities in RC slabs since the stiffness of the slab is affected by the amount of reinforcement in linear models. However, in reconstruction projects where the boundary conditions are changed, e.g. when supporting walls or columns are removed, the linear FE analysis will give the redistribution of the sectional forces used in the strengthening design.

The final design of a RC structure must often be controlled against local phenomena not covered by the used design method. In the design of RC slabs using plate theory, the punching capacity of the slab, e.g. over columns or under point loads, need to be checked separately. If the punching capacity is to low, the designer needs to install additional reinforcement in the area affected by the punch load.

To conclude it is always better to analyse the whole structure in strengthening designs to identify areas of surplus and deficit capacities. This will most easily be done using linear FE models. Non-linear models can be used to further analyse the behaviour of a cracked RC structure or to take advantage of surplus capacities in special cases.

The final design must always be checked against local failure phenomena not covered by the linear elastic model used in the design.

Analysis of required capacities

FEM-Design, is a multipurpose Finite Element package for design of RC structures, Strusoft AB (2005). The slab module in FEM-Design calculates the amount of required bending reinforcement in the RC slab from the linear elastic distribution of sectional forces according to BBK 04. The final design solution is also checked against punch failure due to high shear stresses in regions around supporting columns or point loads according to BBK 04.

A FE-model of the complete courtyard deck before reconstruction with existing supporting walls, columns and girders was first analysed. The required reinforcement capacities were then compared with the existing amount of reinforcement according to the reinforcement drawings of the courtyard deck.

The amount of existing reinforcement was in many areas considerable greater than needed, whereas the reinforcement in other areas was close to the required amount. Figure 5.5 shows the required top reinforcement in the x-direction (horizontal) compared with the existing. In the second analysis the courtyard deck was modelled with the existing reinforcement before reconstruction and Figure 5.6 shows the missing top reinforcement in the x-direction.

From the result of the initial analyses a modelling strategy was put forward where two additional models was going to be produced. One general model called “after reconstruction” and one “punching” model for the analysis of the shear and punching capacity of two especially exposed areas after reconstruction since the shear/punching capacity was judged to be the decisive factor in the selection of strengthening strategy.

Figure 5.7 shows the “after reconstruction” finite element mesh. More result from the analyses is shown in Appendix B.

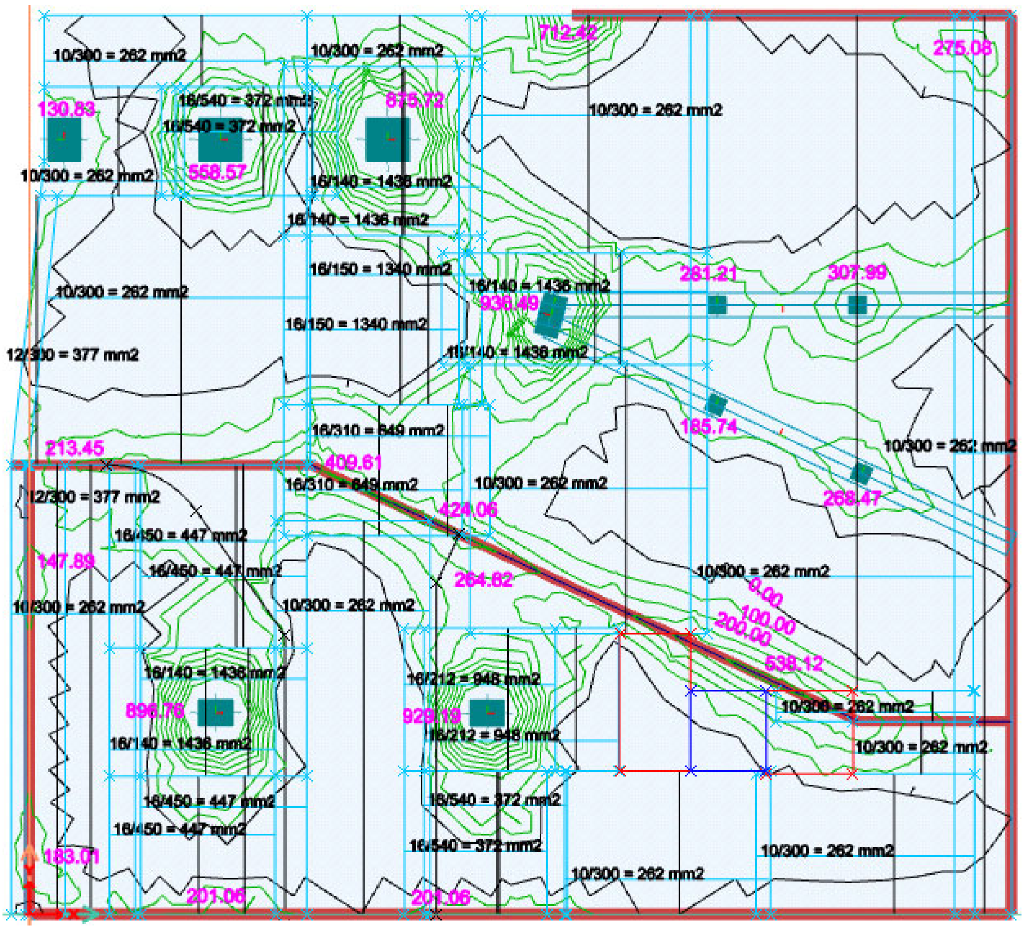


Figure 5.5 Required top reinforcement (magenta) compared to the existing reinforcement in x-direction before the reconstruction [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls*

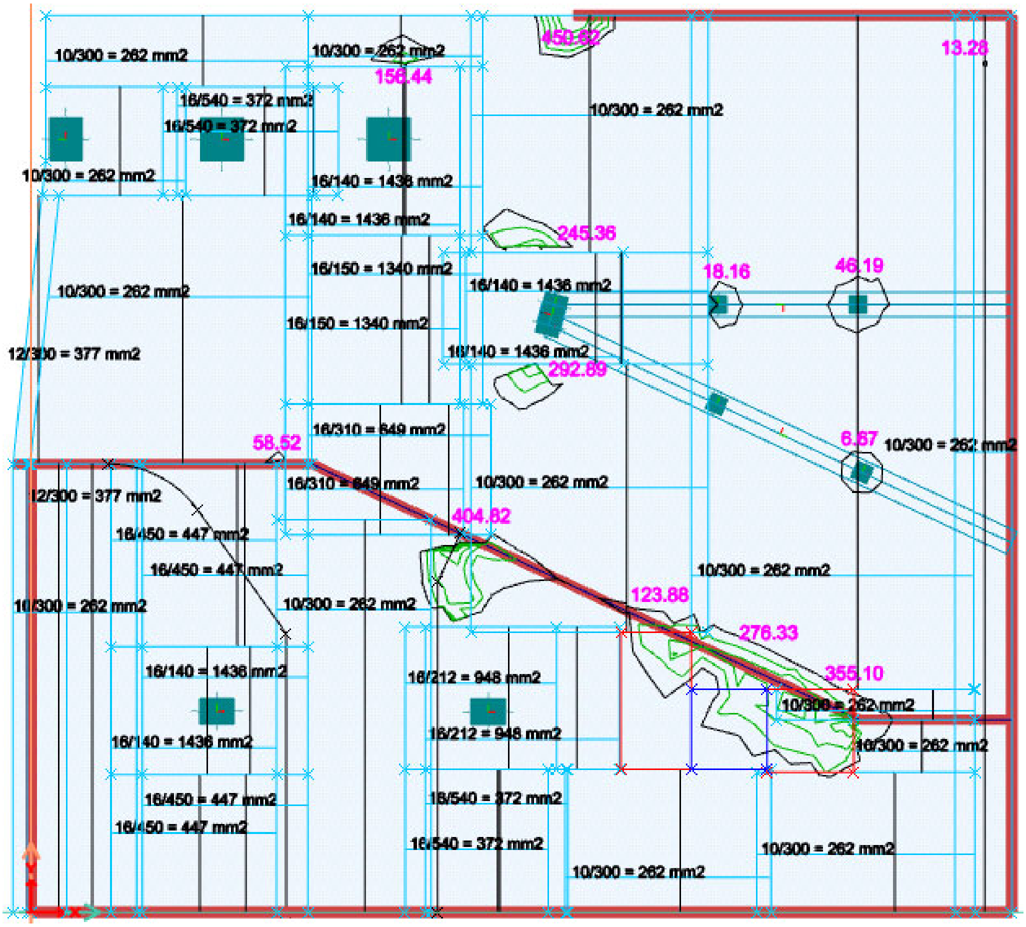


Figure 5.6 *Missing top reinforcement (magenta) compared to the existing reinforcement in x-direction before the reconstruction* [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

Finite element mesh

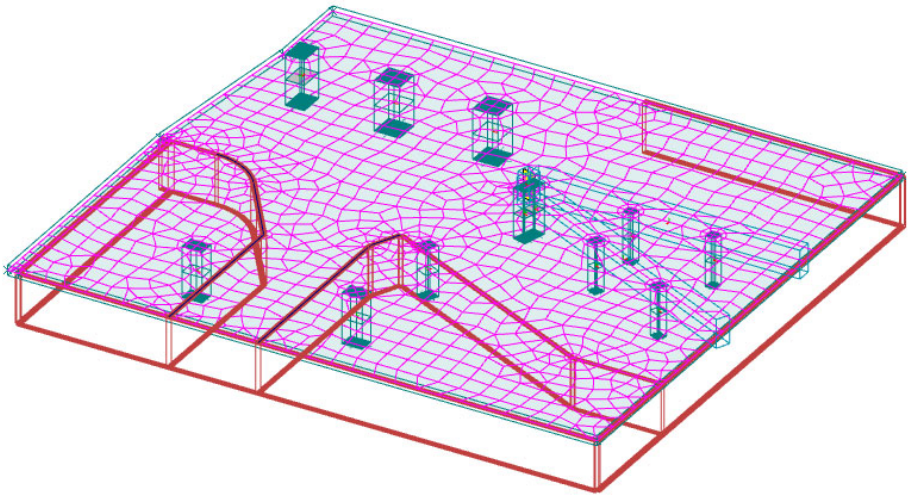


Figure 5.7 The finite element mesh in the model "after reconstruction". The automatic mesh generator in FEM-Design is used.

Shear and punching capacity

The shear and punching capacity is in reality based on the same physical phenomena, but they are controlled with two different design methods. Therefore, two models were needed "after reconstruction" and "punching" to properly identify the required reinforcement capacities.

The first model, "after reconstruction" was used to identify both the needed shear and bending reinforcement capacity in the courtyard deck. The shear capacity is checked according to BBK 04 section 3.7.3.2 where the existing reinforcement is considered. According to BBK 04 section 3.7.3.3 a load near a support can also contribute to the shear capacity. In this case, the contribution was estimated to approximately 25 kN/m which can, (if needed), be subtracted from the missing shear capacity along the supports.

The missing shear capacity is shown in Figure 5.8. The areas that need to be investigated in more detail are marked with circles in the figure. The areas marked with rectangles shows missing capacities at the top edge of the courtyard deck. Since the amount has been missing all the time and the slab demonstrably survived this deficit in shear capacity no further measures are necessary, see also Figure B.5 in Appendix B.

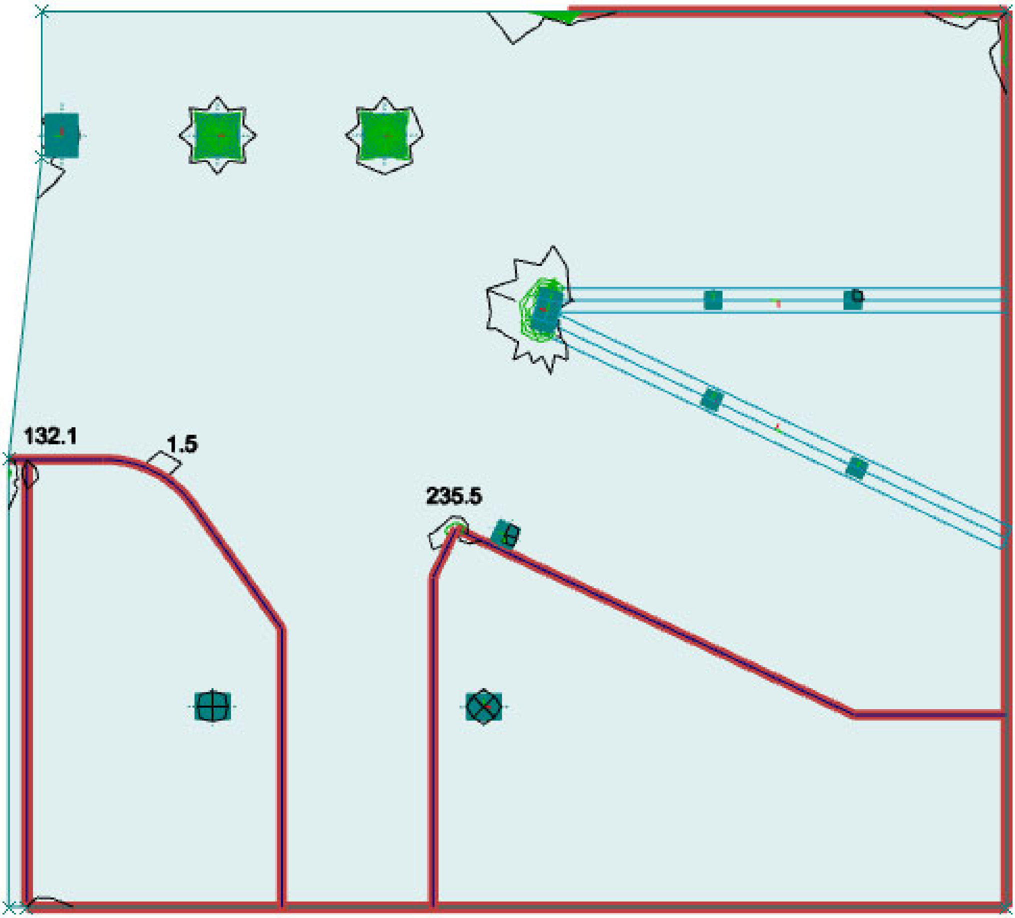


Figure 5.8 *Missing shear capacity* [kN/m] *after the reconstruction when considering the existing steel reinforcement. Compare with Figure B.5 in Appendix B.*

Figure 5.15 shows the "punching" model after reconstruction with defined columns according to BBK 04 section 6.5.4.2. The enlarged figures of the slab show the part of the slab with deficits in shear force capacity. Fictitious columns are replacing part of the wall structure to be able to control the punching capacity instead of the shear capacity.

The punching capacity over the columns is checked according to BBK 04 6.5.4 – 5 and Betonghandbok (1990) section 6.5:34, see also Strusoft AB (2005). The punching capacity is sufficient if $V_u > V_d$, where V_u is the calculated punching capacity of the existing reinforcement and V_d is the required punching capacity.

If $V_u < V_d$ the existing capacity is insufficient and the necessary additional shear reinforcement needs to be calculated. If the punching capacity based on both shear and bending reinforcement, ($V_{us} < V_d$), is insufficient, other strengthening methods for rehabilitation or retrofit have to be used.

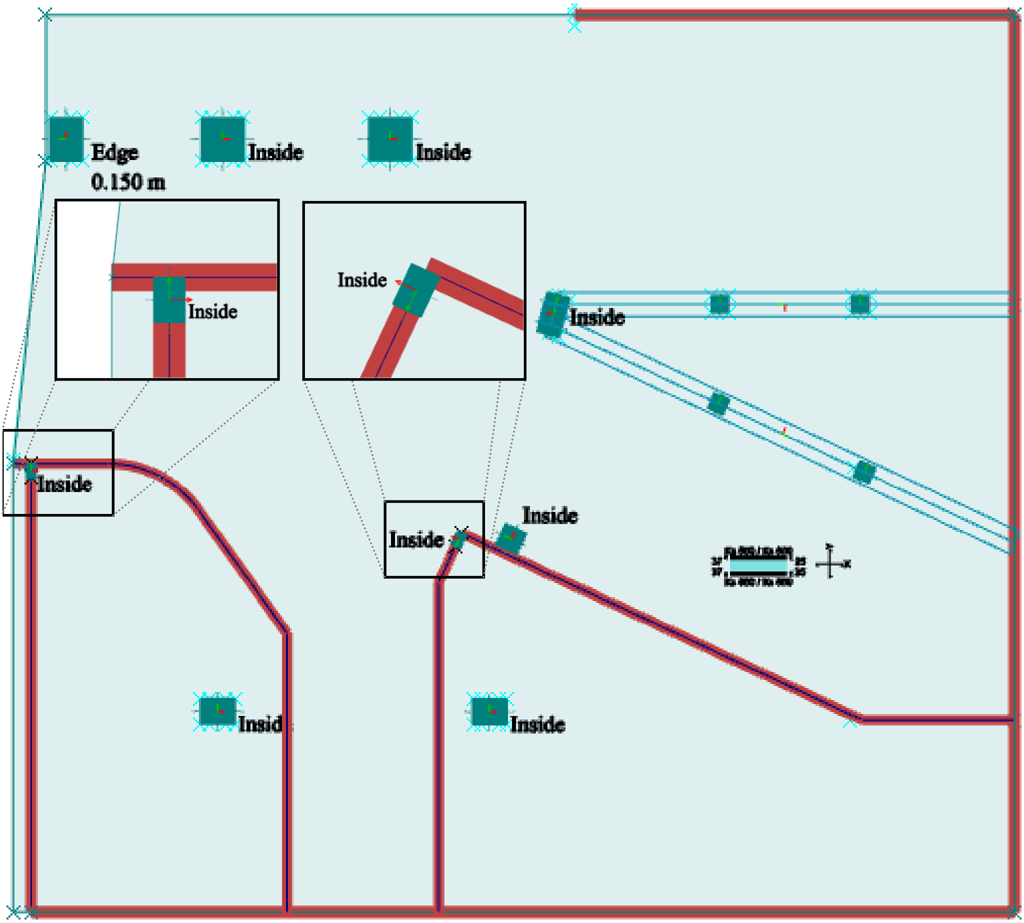


Figure 5.9 Defined columns (edge or insida) for the punching analysis in the model "Punching, after reconstruction". The definition (inside, corner or edge) depends on the critical perimeter according to BBK 94 section 6.5.4.2.

Figure 5.10 shows that the existing reinforcement provide enough punching capacity for the two studied columns. Results for all columns are shown in Figure B.6 in Appendix B.

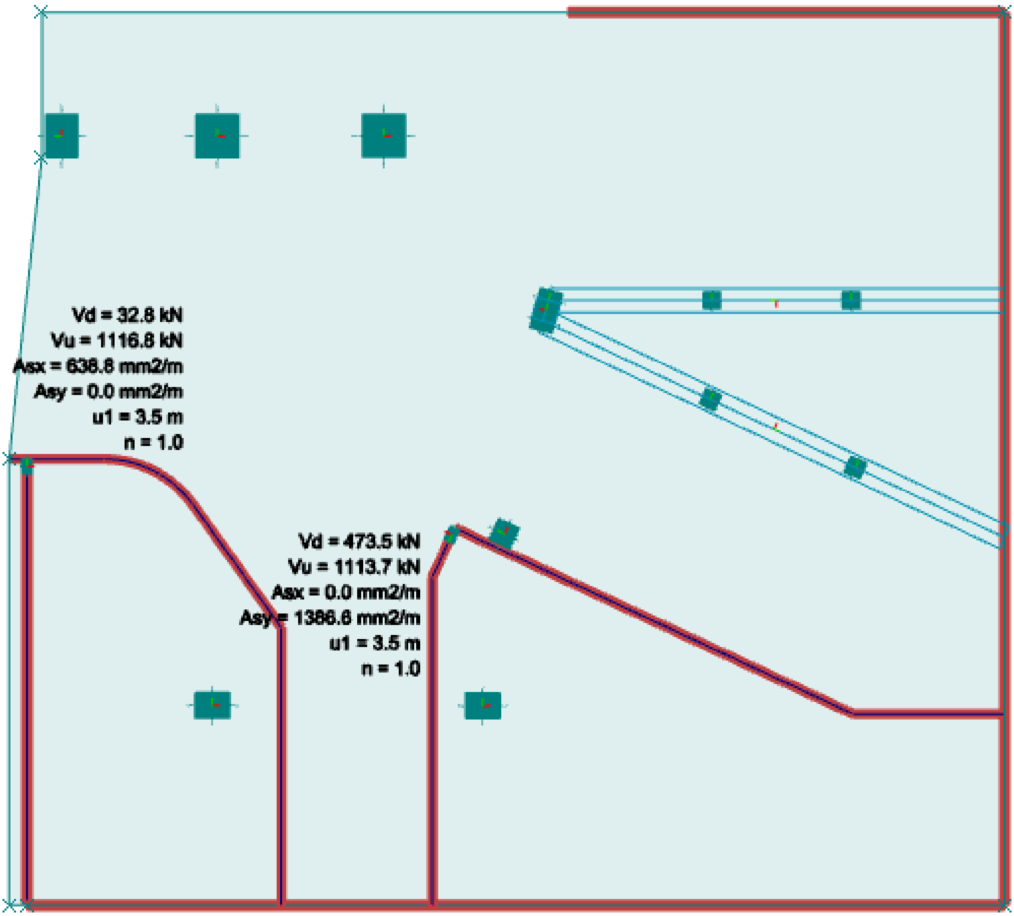


Figure 5.10 *Result from punching control of the two areas with fictive columns replacing corresponding parts of the walls. The bending top reinforcement in x-direction,* A_{sx} *and in y-direction,* A_{sy} *used in the calculation of the capacity is shown together with the result.*

Moment capacity

The first analysis before reconstruction showed that the courtyard deck had areas with surplus and deficit moment capacities. Figure 5.11 and Figure 5.12 shows missing top reinforcement quantities after reconstruction in comparison with existing reinforcement.

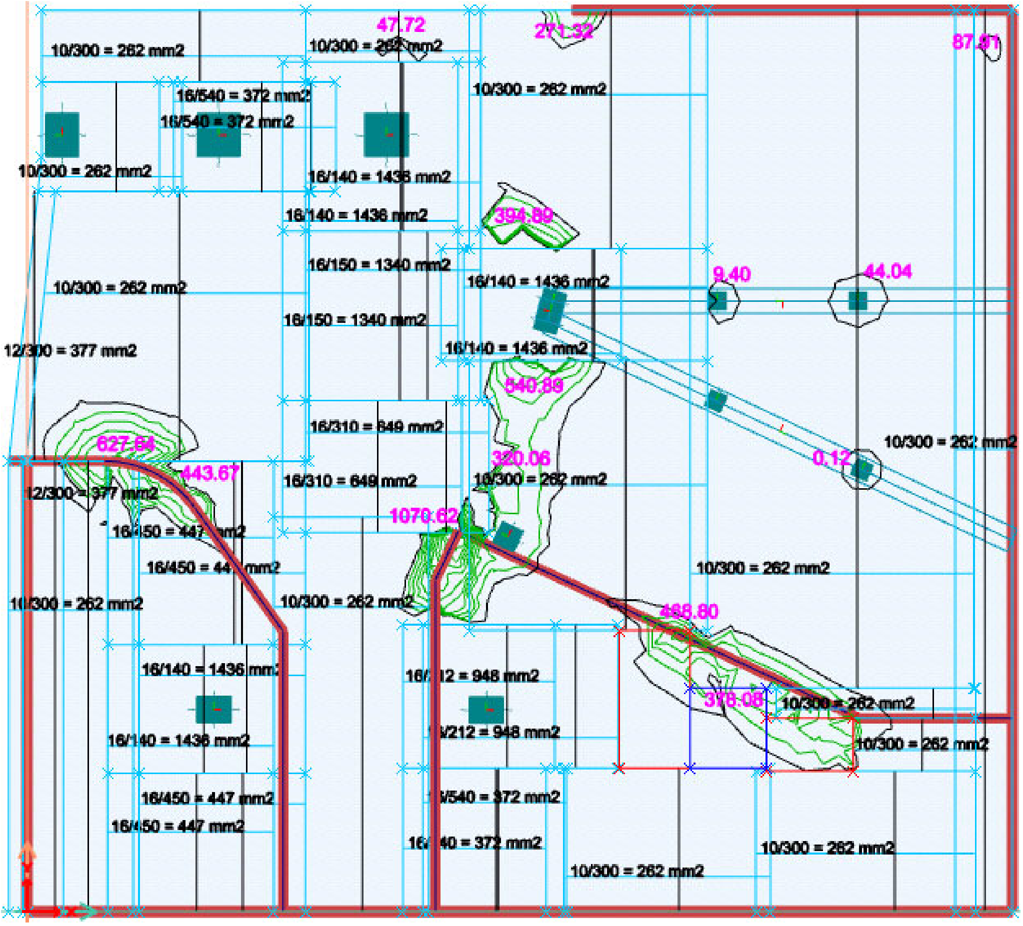


Figure 5.11 Missing top reinforcement (magenta) compared to the existing reinforcement in x-direction after the reconstruction [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

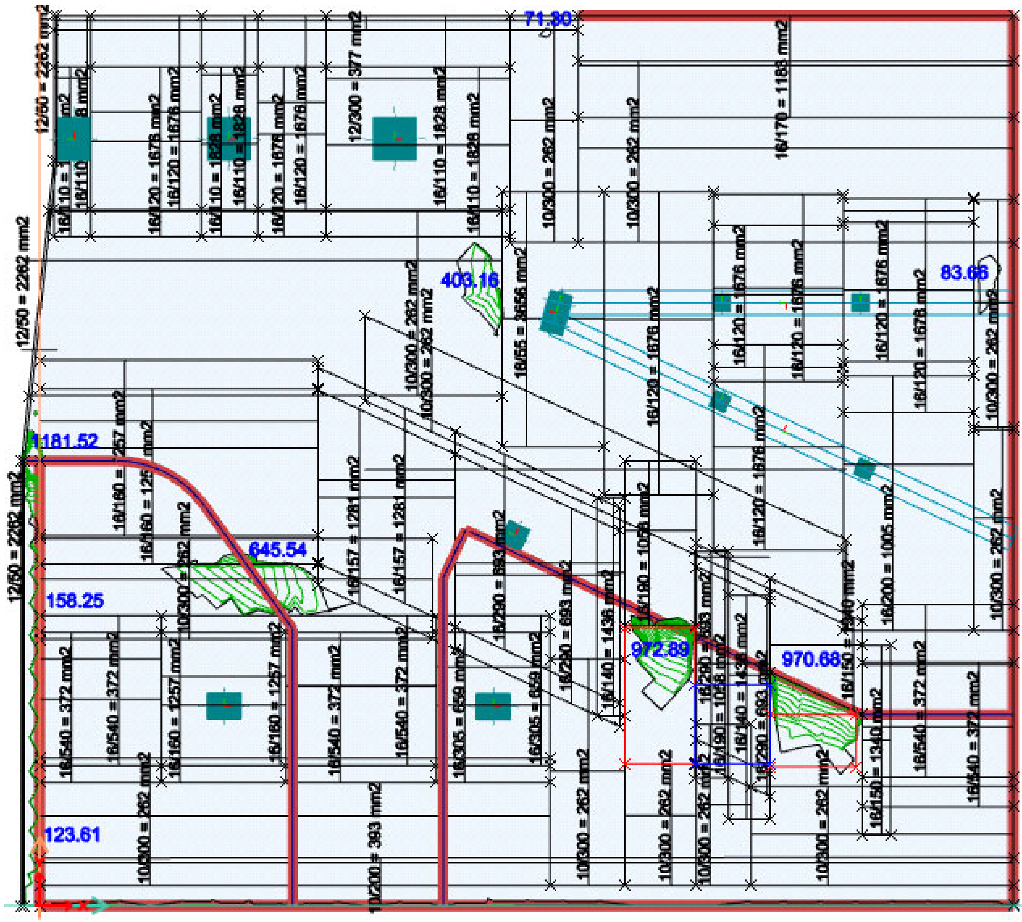


Figure 5.12 Missing top reinforcement (magenta) compared to the existing reinforcement in y-direction after the reconstruction [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

Figure 5.13 and Figure 5.14 shows the areas that needed strengthening together with the necessary amount of reinforcement. The areas were determined from the mean amount of missing reinforcement in these areas. Other areas was left un-strengthened since the mean amount of missing reinforcement was judge to be so small that the moments could be redistributed to adjacent areas where surplus capacities existed. These areas are marked with circles in the figures.

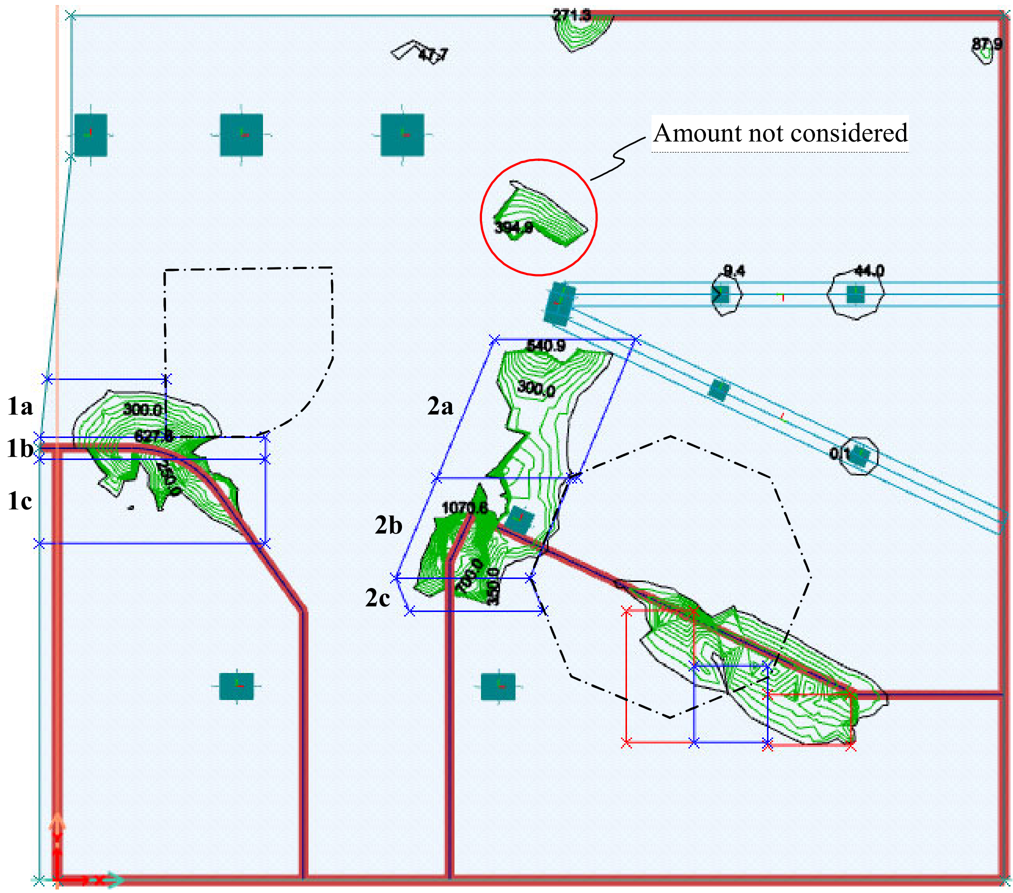


Figure 5.13 Estimated areas required to be strengthened in x-direction (Area 1 – 2 with blue rectangles) compared with contour lines of missing top reinforcement [mm^2/m].

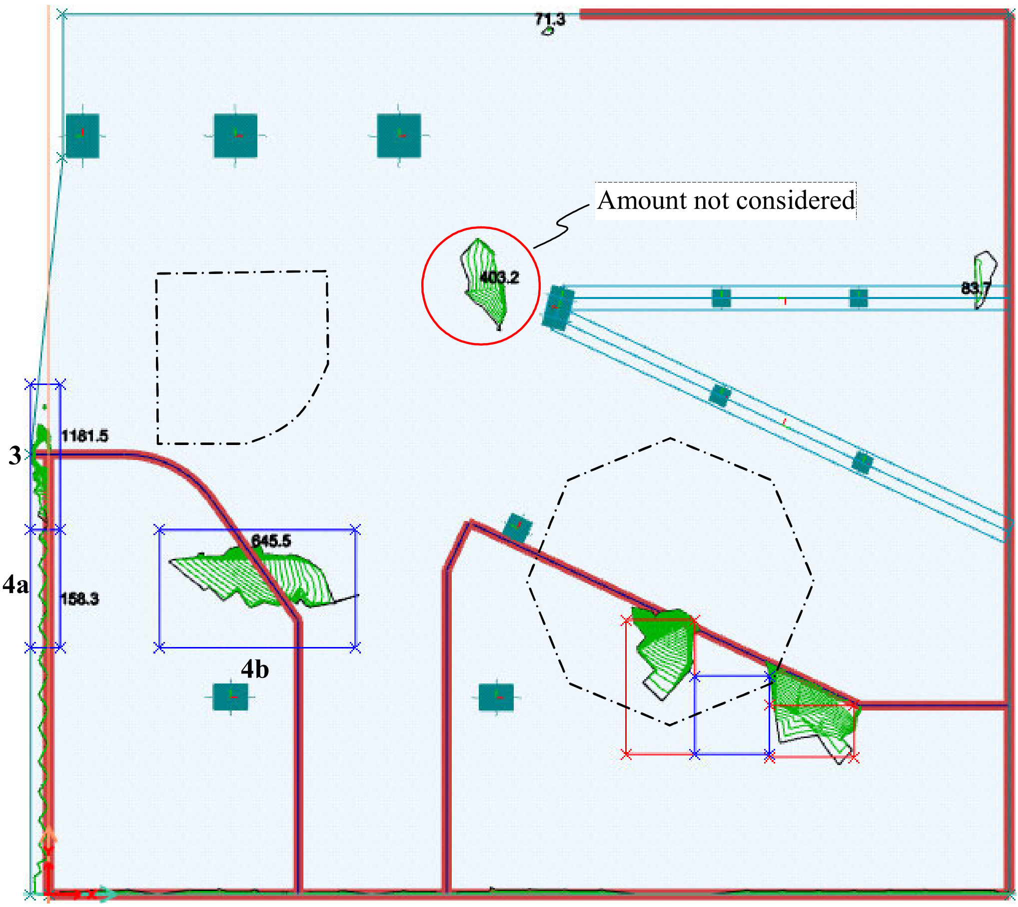


Figure 5.14 Estimated areas required to be strengthened in y-direction (Area 3 - 4 with blue rectangles) compared with contour lines of missing top reinforcement [mm^2/m].

Summary

Table 5.3 shows the result from this strengthening analysis. The total amount of needed reinforcement is listed for each of the strengthened areas shown in Figure 5.13 and Figure 5.14. The reinforcement quantities can then be transformed to equivalent CFRP amounts by using the simplified analytical expressions in section 4.2.

Table 5.3 Required amounts of reinforcement, Ks 60.

Area	Required amount of reinforcement, [mm^2/m]
1a	300
1b	628
1c	250
2a	300
2b	900
2c	350
3	600
3+4a	600
4b	350

5.2.3 Strengthening strategy and method

Selection of method

The main options for the strengthening method was: concrete casting, steel bonded plates or epoxy bonded CFRP strengthening systems. The strengthening method chosen was epoxy bonded CFRP plates and Near Surface Mounted Reinforcement (NSMR). The reason for choosing this strengthening technique is the following:

- Ease of installation.
- Cost effective compared to traditional concrete casting.
- No adding of additional weight.
- Corrosion resistance compared to using steel plates.
- Excellent force transfer from the structure to the plates
- The ability to individually strengthen the slab in two crossing directions, NSMR in the x-direction and CFRP plates in the y-direction, see Figure 5.15

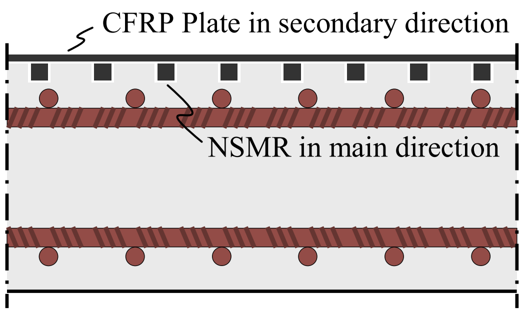


Figure 5.15 Strengthening of a slab in two crossing direction using CFRP plates and NSMR.

CFRP quantities

A CFRP strengthening behaves more or less linear up to failure. Therefore, the ultimate failure for CFRP strengthened slab will depend on the strength of CFRP sheet in the most strained area, as showed in chapter 3, (if debonding is avoided). Therefore, a linear FE-analysis can be used.

It will then be possible to calculate the equivalent amount of bonded strengthening material using the simplified analytical expression in 4.1 from the required amount steel reinforcement calculated by FEM-design. However, this implies that:

- The strengthening material bonded to the surface of the slab will have equal or higher failure load than the selected reinforcement used in the FE-analysis.
- The primary failure must be in the reinforcement and not be governed by failure in the bond. If bond failure is expected, (and can be calculated) the maximum allowable stress in the reinforcement must be adjusted to reflect this behaviour.

The quantities is calculated according to the simplified method described in chapter 4.2. The relation between the total missing reinforcement quantities A_s calculated by FEM design and the required amount of CFRP A_f is:

$$A_f = \frac{E_s}{E_f}\left(\frac{h-u-x}{h-x}\right)^2 A_s \qquad 5.2$$

where $d = h - u$ is the effective height and u is the distance from the tensile side to the centre of gravity of the considered steel bars. The formula takes into account the differences in effective heights and stiffness between the steel and CFRP.

The total quantities and placement of the CFRP plates and the NSMR are presented in Figure 5.16 and Table 5.4.

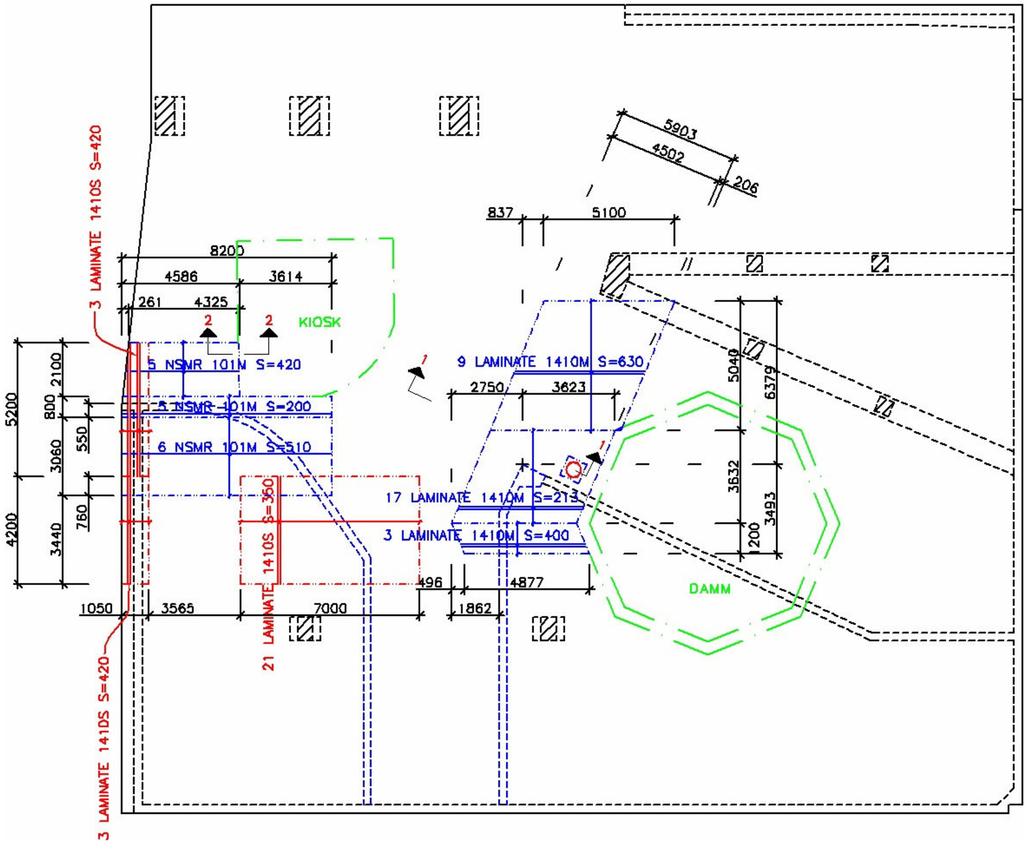


Figure 5.16 Placement of epoxy bonded CFRP plates (red) in y-direction and NSMR + CFRP plates (blue) in x-direction.

Table 5.4 Required amounts, length and distances of CFRP sheets and plates.

Area	Amount CFRP, [mm^2/m]	Type	Pcs.	Distance, s [mm]	Length, [m]
1a	235	NSMR 101M	5	420	≈ 4.6
1b	491	NSMR 101M	5	200	8.20
1c	196	NSMR 101M	6	510	8.20
2a	219	LAMINAT 1410M	9	630	5.10
2b	657	LAMINAT 1410M	17	213	4.88
2c	256	LAMINAT 1410M	3	400	4.88
3	645	LAMINAT 1410S	3	420	5.20
3+4a	645	LAMINAT 1410S	3	420	9.40
4b	391	LAMINAT 1410S	21	350	4.20

Strategy

In this particular case, the aim was to strengthen the slab from the top and to take up the extra negative bending capacity due to the alteration of the wall configuration underneath the slab. The strengthening needed to be carried out before the wall was altered. To facilitate the strengthening work the total strengthening area was divided into three sub-areas. The strategy was then to finish one area before strengthening of the next was started. Since the strengthening work was going to take part from the top of the slab, all materials above the construction concrete have to be removed, i.e. paving-stones, sand, protective concrete etc. Therefore, an active design approach was adopted. In this particular case this meant that the final strengthening procedure was decided when the construction concrete was uncovered and a proper examination of the actual condition of the structure could be examined.

5.2.4 Strengthening procedure, follow up and control

Strengthening procedure

Before strengthening the construction concrete with CFRP plates the surface needs to be pre-treated. Two methods for pre-treatment are recommended, sandblasting or grinding. At time for bonding no debris are allowed on the surface, grease, oil and dust shall be removed. A primer are then applied to the surface, the primer is let harden for approximately 24 hours before bonding.

Adhesive is applied on the CFRP plates and the plates are then mounted onto the pre-treated concrete surface. The plates are given a light pressure to remove possible air inclusions.

The pre-treatment for the rectangular NSMR bars is somewhat different. Here slots are sawed in the construction concrete, in this particular case the size of the slots will be 15×20 mm (width×depth), with a cross sectional area of the bar of 10×10 mm. The slots are cleaned with pressurised water, typically with 150 bar, to remove possible concrete debris. After the surface has dried out an epoxy primer is applied in the slots. An epoxy adhesive is placed in the slots and the cleaned NSMR bar is placed in the adhesive with an adhesive cover of approximately 5 – 7 mm. Excessive adhesive is swept off.

For both systems no water, oil, grease or dust is allowed on the surface at time for bonding. The temperature shall be at least 10 °C and 3 °C above the dew point. The relative humidity, *RH* shall not be above 80 % at time for bonding.

The strengthening system is let harden for approximately 5 days at 20 °C before the full load is applied. The final strengthening result is largely dependent on the execution of the strengthening work. Poor workmanship or mistakes during the construction process may result in inferior strengthening. In addition, insufficient examination of the structure before strengthening may result in incorrect strengthening design.

Control checklist and follow up

The planned following up and control for the project was to use checklists and site controls before, during and after strengthening. The most important factors to control were:

- The quality of the existing construction concrete.
- Surface irregularity.
- Cover to existing steel reinforcement.
- Temperature and humidity at time for applying the primer and the adhesive.
- Placement of the strengthening material in regard to drawings.
- Air enclosure in the adhesive.
- Final strengthening result.

All checkpoints were documented in protocols, and special care was taken of the final placement of the strengthening i.e. the drawing was updated.

5.3 Strengthening work

5.3.1 Background

In this section the actual strengthening work is described, i.e. how the final strengthening scheme become and the difference against preliminary design. In this particular case it was extremely difficult to obtain full information about the structure and concrete surfaces to be bonded, this information was only possible to obtain when all materials above the construction concrete had been removed. The strengthening procedure is divided in three steps; before strengthening, during strengthening and after strengthening.

5.3.2 Area 1

Before strengthening

The strengthening work started at area 1, the area besides the circular pod (pool). The surfacing material and the overlay concrete were removed and the surface was sandblasted. The steps described earlier for plate bonding was followed and a control showed that the surface was even and no debris could be noticed.

During strengthening

First the concrete surface was primed, after approximately 24 hours the CFRP plates were bonded to the concrete surface with an approximate adhesive layer of 2,0 mm. However, only four hours after strengthening it stared to rain, and even though the strengthening area was protected by a tent, water flooded over the strengthening material due to the inclination of the slab and that the strengthening areas had not been properly protected, i.e. not whole over. The flooded CFRP plates are shown in Figure 5.17. A hole was drilled at the lowest point of the slab to let the water drain out. An inspection after that the rain had been removed, this took about 3 days, showed that the bond was very good and that no negative short time effects could be noticed for the strengthening. So far so good.

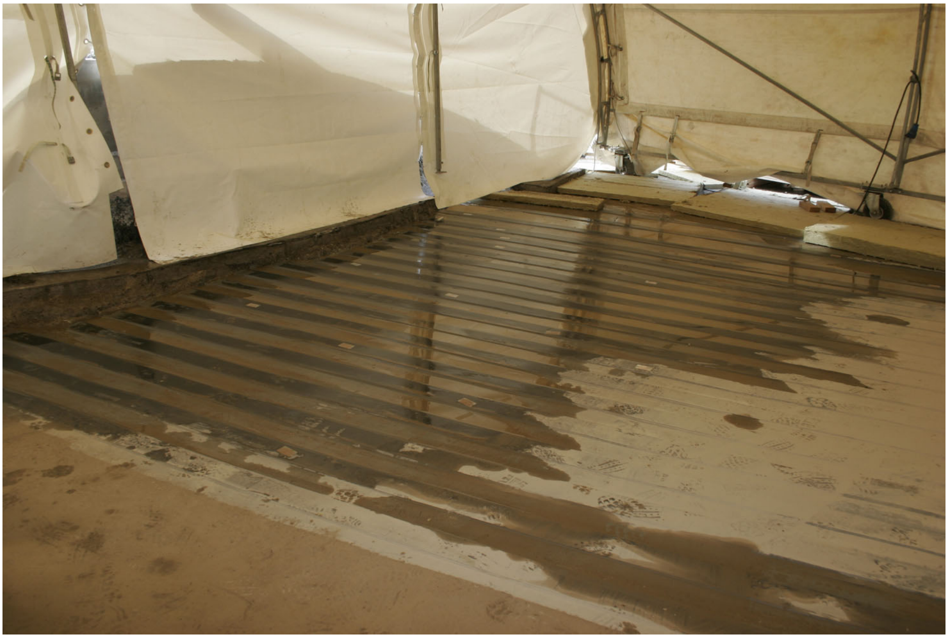


Figure 5.17 Flooded strengthening area.

Nevertheless, by a coincidence it was found out that the bonding had not been carried out on the construction concrete but on a concrete overlay which had not been removed by the contractor. This meant that all CFRP strengthening had to be removed together with the overlay concrete and the work has to start over again. However, this time no mistakes were made. The surface of the construction concrete was more uneven and besides sandblasting, it was decided to grind the largest unevenness.

After strengthening

The result after strengthening is shown in Figure 5.18. It can also be noticed that the end of the CFRP plates have been anchored with CFRP sheets. The result after strengthening was satisfying and the strengthening scheme is directly in correspondence with the design formulation. The control showed no air entrapment in the bond layer and both the temperature and humidity were kept within stipulated limits during strengthening and hardening of the adhesive.

Figure 5.18 Area 1 after strengthening.

5.3.3 Area 2 and 3

Before strengthening

The strengthening areas 2 and 3 were on the opposite part of the courtyard. From the first strengthening area it was learnt that there were two layers of overlay concrete and that both of these needed to be removed before the construction concrete was exposed. It was not an easy task to expose the construction concrete and when this finally was done, the result was not encouraging. In Figure 5.19 the surface after removal of the overlay concrete is shown. It can clearly be noticed that the surface is very uneven.

Figure 5.19 Typical surface condition of area 2 and 3 after removal of the overlay concrete.

Since the intention was to use NSMR bars this should not create to much problem. However, when the slots for the bars started to be sawn another problem was faced – the concrete cover to the steel reinforcement was very thin, see Figure 5.20. At several locations, the rebars were even exposed to the open air with almost no concrete cover. It was therefore decided not to use NSMR bars.

Instead, CFRP plates were also used here. However, before bonding could start extensive pre-treatment of the surface was needed. Firstly, all loose concrete was removed by jack-hammers. The surface was then sand-blasted and finally grinded. The surface was then properly cleaned before the bonding procedure started.

Figure 5.20 Exposed steel rebars.

During strengthening

Firstly, the surface to be bonded was primed. Secondly, the primer was let harden, and since the surface still had to large irregularities, it was decided to use an epoxy putty to level out to prescribed levels. The adhesive was applied to the CFRP plates and bonding and levelling out was carried out wet-in-wet. The bonding procedure become considerably more complicated then for area 1 and the total adhesive + putty layers in some bond areas exceeded 8 mm.

After strengthening

The result after strengthening is shown in Figure 5.21. As can be noticed that the irregularities still remain in between the CFRP plates. This was made by purpose to increase the bond for the overlay concrete that were going to be cast on-top of the strengthening. In Figure 5.21 also CFRP plates used for anchorage can be seen, the ones crossing the plates in the bottom. Consider the complicated surface conditions the final strengthening result for area 2 and 3 was satisfying, even though the result was not as good as for area 1. Also, here the control showed no air entrapment in the bond layer and both the temperature and humidity were kept within stipulated limits during strengthening and hardening of the adhesive.

Figure 5.21 Area 2 and 3 after strengthening.

In this particular case the original design was changed, the NSMR bars were replaced with CFRP plates in area 2.

5.3.4 Comments to the strengthening work

The final result from the strengthening work is shown in figure 5.22. Despite the problems with rain, extra layer of overlay concrete, steel reinforcement in open air, the final result is considered to be satisfying. However, from the project the following conclusions can be drawn:

- It is extremely important to collect all important data for the strengthening work.
- Protection against bad weather is a must.
- Active design is preferable.
- It is often more complicated to strengthen an existing structure than building a new one.

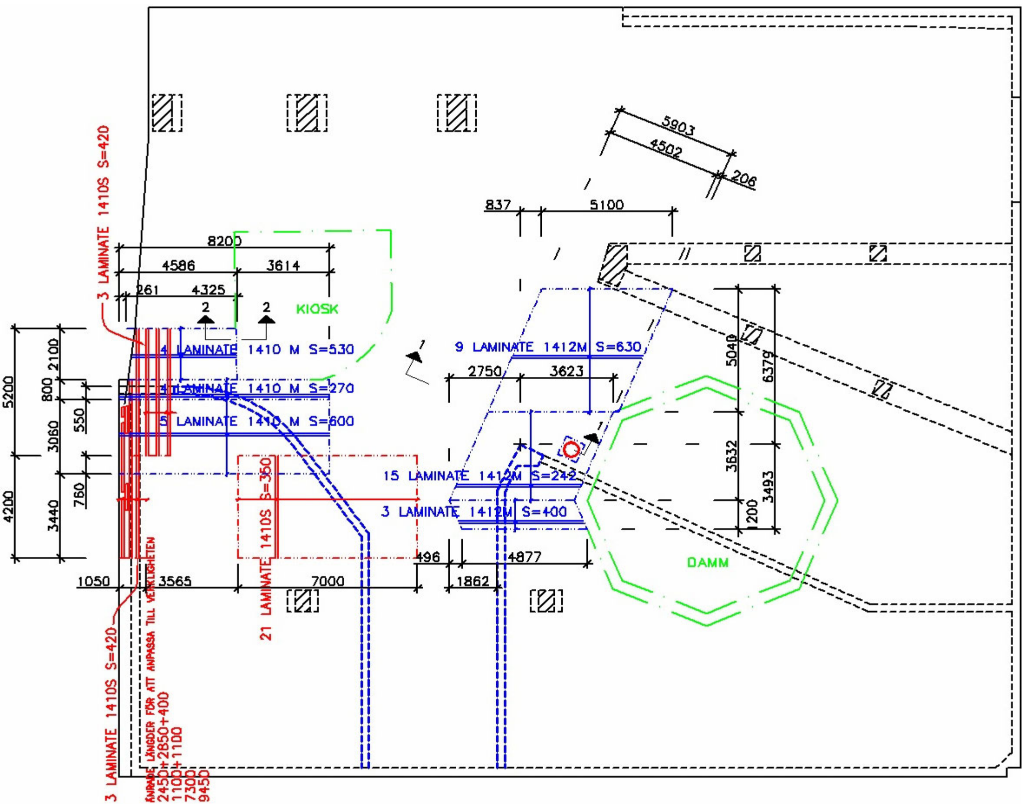


Figure 5.22 Final strengthening result. NSMR in x-direction is replaced with CFRP plates in area 2.

6 CONCLUSION

6.1 Discussion

The CFRP strengthened concrete slab in the experiment shows a stiffer behaviour compared to the reference slabs (H, S/Lw) and especially after the point where yield lines are developed. All strengthened slabs have a higher final load carrying capacity than the homogeneous reference slab.

The test results clearly show that the investigated strengthening system can be used to strengthen existing slabs with made openings, and even that the load carrying capacity can be increased when compared to the homogeneous slab (H). For the CFRP strengthened slabs, the load carrying capacity was increased with 24-125 % in comparison to respective weakened slab (S/Lw), and with 22-110 % in comparison to the homogeneous slab.

The general result of the experimental investigation is that the method to design the required amount of steel reinforcement due to an opening gives a load carrying capacity on the safe side. The slabs with the larger openings have a noticeable higher load carrying capacity and a stiffer load-deflection response than the slabs with the smaller openings. This is in contradiction to the proposed design method. The reason for this can be that the slabs with the large openings behave closer to a system of four beams than a slab.

The FE analyses show a more brittle behaviour compared with experiments. Especially for the non-strengthened slab where a tendency of snap-back behaviour can be noticed when the yield limit is reached in the steel reinforcement. This is probably one of the reasons why the implicit solution strategy failed.

A more correct material model of the reinforcement including the stiffening effect of the surrounding concrete will probably give better result. Also, it is always difficult to compare analytical and experimental strain readings since the level of strain is highly dependent of the crack localisation in quasi-brittle materials like concrete. Apart from these observations, the explicit FE analysis is showing a good agreement with the test and is giving more insight to the strengthening effect of the CFRP sheets. It is also giving more confidence to the future utilization of the non-linear FE analysis in order to evaluate larger slabs with different opening configurations.

The case study of the courtyard deck revealed that linear FE analysis can be a useful tool in the design of CFRP strengthening of structures in reconstruction projects. By using a simplified method for estimating the amount of CFRP from calculated quantities of steel reinforcement a commercial available FEM package for design of RC slabs (FEM-Design) could be used. The estimated CFRP quantities from the initial hand calculation were much higher compared to the outcome from the FE analysis. Furthermore, the FE design tool could easily support the active design approach due to limited information of the site conditions. It is much easier to change adapt an existing FE design calculation to changes than redoing a hand calculation were the designer needs to make a lot of assumption and simplifications before the analysis can be made.

6.2 Conclusion

In Chapter 1.2, aim and scope, four research questions was set-up for this research work.

The answer to the first question is:

1. Yes, the simplified design method of CFRP strengthened RC slabs with cut-out openings used in practice today gives a load-capacity equivalent or higher than a RC slab without an opening. Also, the traditional way to reinforce additionally with steel bars along an opening gives a higher load capacity than a similar slab without an opening.

The answer to the second question is:

2. The optimal direction to place the CFRP sheets around the opening in this study was along the four edges of the opening. This was believed to be an effect of two CFRP sheets crossing over each other in the corners of the opening. However, more studies (experimental and numerical) is needed to find the optimal placements.

The answer to the third question is:

3. Yes, it is possible to model the mechanical behaviour of CFRP strengthened RC slabs using advanced non-linear FE analysis. However, better models of the crack localisation in the concrete are needed to be able to accurately predict the ultimate failure. Also, more studies on the bond behaviour between the CFRP and the underlying concrete is needed to be able to suggest safe design margins and methods to avoid bond failure in CFRP strengthening designs of RC slabs.

The answer to the last question is:

4. Yes, existing advanced design tools, such as FEM design, can be used for the design of CFRP strengthening of existing RC slabs in reconstruction projects. The simplified method for calculating the amount of CFRP was found to be valid and can be used to transform estimated quantities of steel reinforcement in linear elastic structural analysis of RC slabs to equivalent CFRP amounts.

To conclude, the work presented in this paper shows that CFRP sheets can be used to maintain and even increase the original load-capacity of two-way concrete slabs, with or without openings.

Finally, more advanced design methods should lead to a more efficient use of the CFRP sheets in strengthening design, especially if the goal is to achieve a load carrying capacity equal to, or slightly larger than the load action.

6.3 Suggestions for further research

FE analysis can be used to study different configurations of openings in larger slabs. This can also be used to investigate how the design of CFRP strengthening should be made to avoid bond failure. The difference between the experimental and the numerical strains in the CFRP sheet needs also to be investigated more thoroughly since it governs the ultimate failure of the slab. Is the difference a result of inadequate ability to model the crack localisation or do the biaxial strain field lower the theoretical rupture strain in the CFRP sheets? From these types of analyses, better analytical design methods can be derived.

The present study can also be extended to include similar structural elements such as walls. Can CFRP be used to strengthen cut-outs in walls that are also subjected to in plane load in the vertical direction?

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Appendix A EXPERIMENTAL STUDY – DRAWINGS AND RESULTS

A.1 Homogeneous slab, H

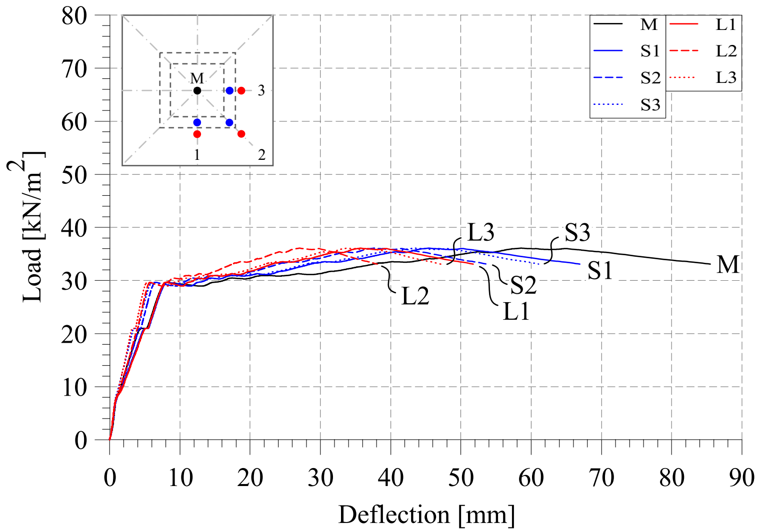


Figure A.1 Load-deflection relationship for a homogeneous slab, H (without an opening) at three discrete points corresponding to a small or a large opening, respectively.

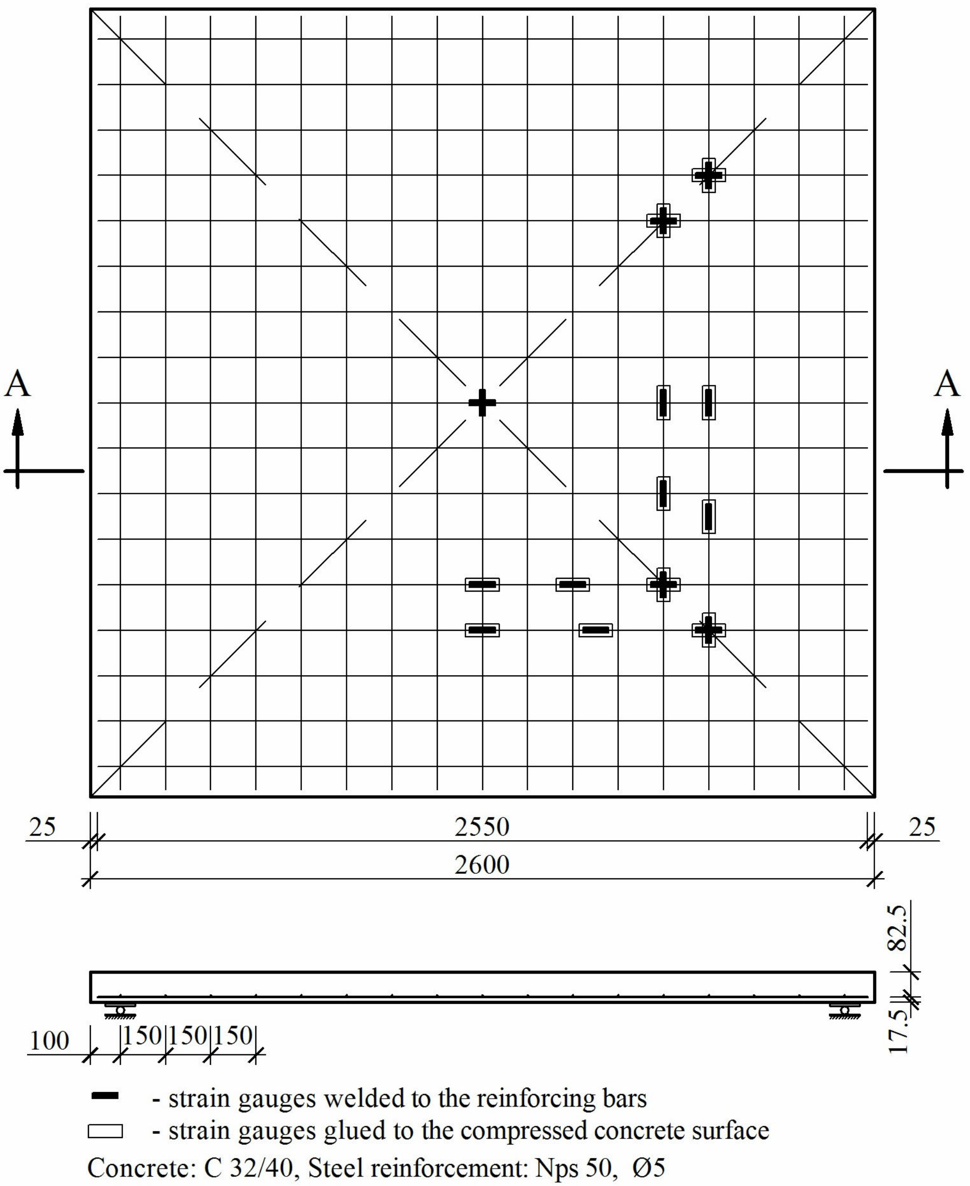


Figure A.2 Design of a homogenous slab, H and location of strain gauges. Drawing scale 1:25.

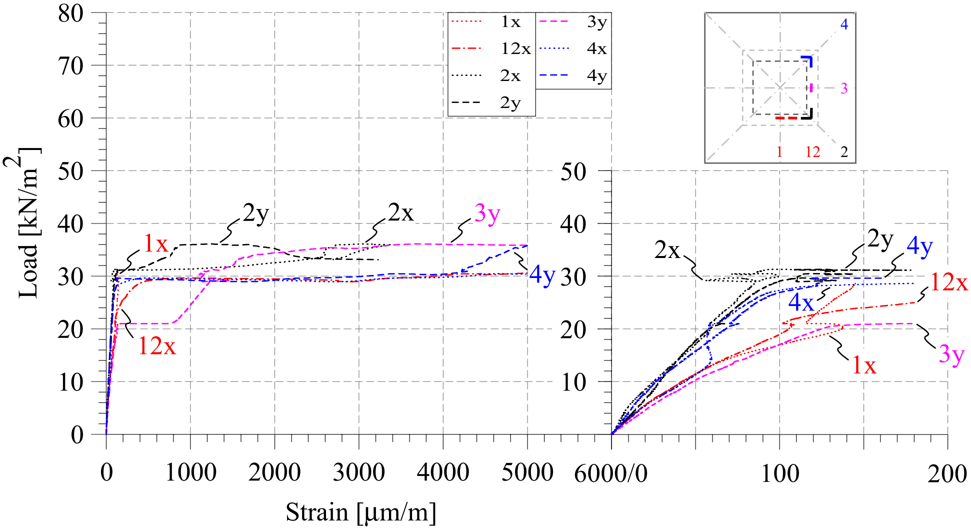


Figure A.3 *Load-strain relationship for the* <u>*steel reinforcement*</u> *in the homogeneous slab, H. The strain is measured at discrete points that correspond to a* <u>*small*</u> *opening. Strains higher than* 5000 µm/*m are truncated. The right graph shows the initial stage up to* 180 µm/m.

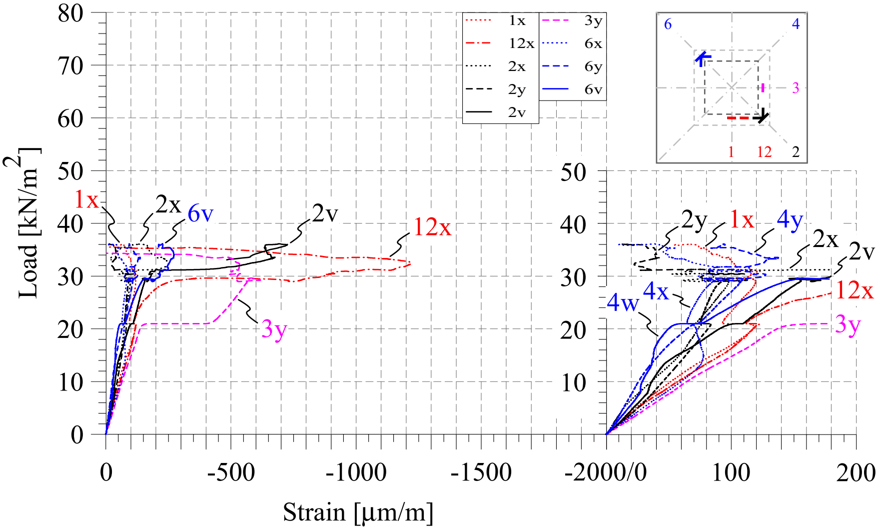


Figure A.4 *Load-strain relationship for the* <u>*concrete*</u> *in the homogeneous slab, H. The strain is measured at discrete points that correspond to a* <u>*small*</u> *opening. The right graph shows the initial stage up to* 180 µm/m.

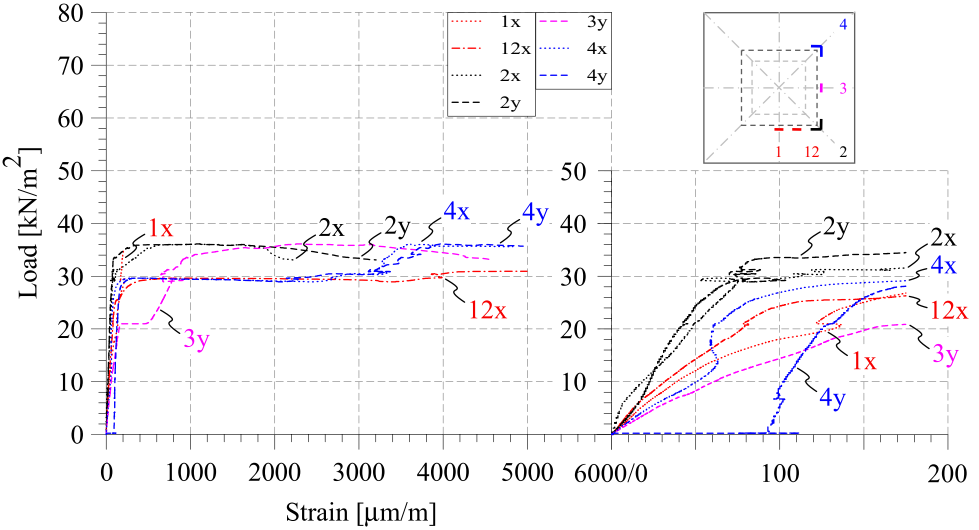


Figure A.5 *Load-strain relationship for the* <u>*steel reinforcement*</u> *in the homogeneous slab, H. The strain is measured at discrete points that correspond to a* <u>*large*</u> *opening. Strains higher than* 5000 µm/m *are truncated. The right graph shows the initial stage up to* 180 µm/m.

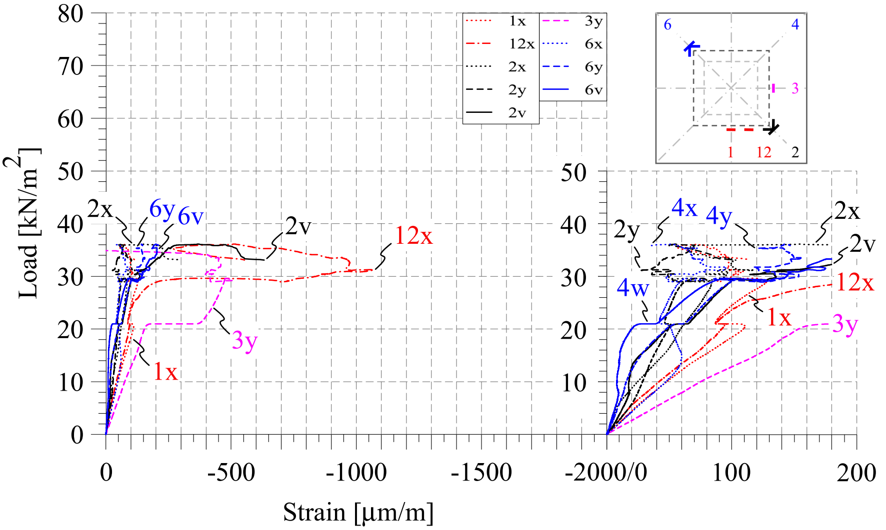


Figure A.6 *Load-strain relationship for the* <u>*concrete*</u> *in the homogenous slab, H. The strain is measured at discrete points that correspond to a* <u>*large*</u> *opening. The right graph shows the initial stage up to* 180 µm/m.

A.2 Slab weakened by a sawn up small opening, Sw

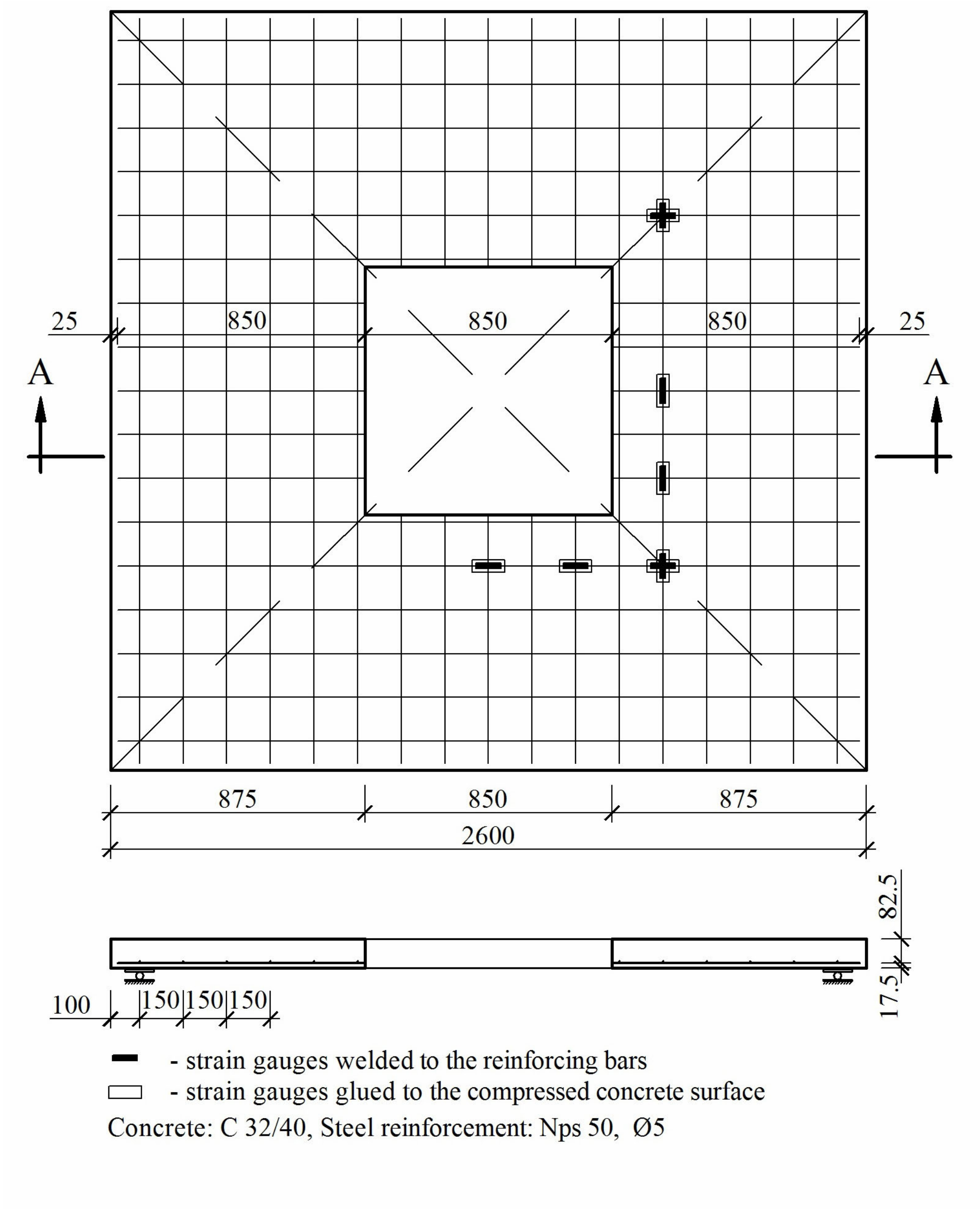


Figure A.7 Design of a slab weakened by a sawn up small opening, Sw and location of strain gauges. Drawing scale 1:25.

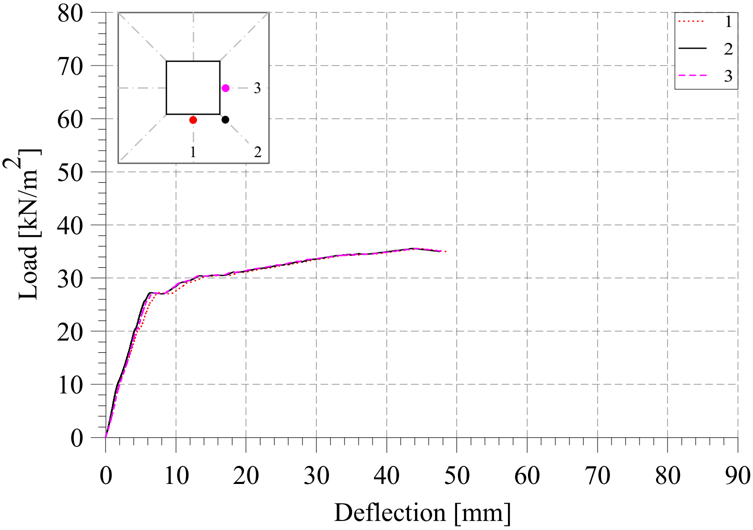


Figure A.8 *Load-deflection relationship for a slab weakened by a sawn up* *__small__* *opening, Sw.*

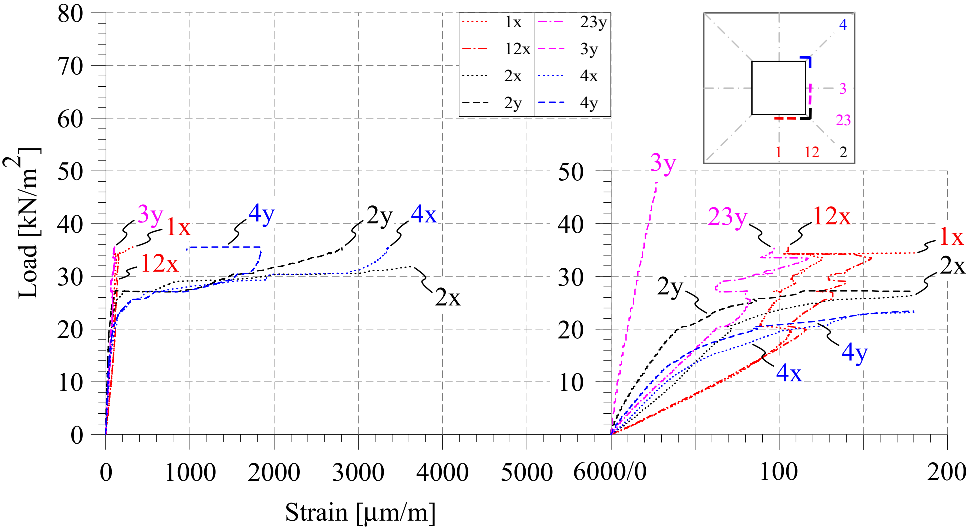


Figure A.9 *Load-strain relationship for the* *__steel reinforcement__* *in a slab weakened by a sawn up* *__small__* *opening, Sw. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

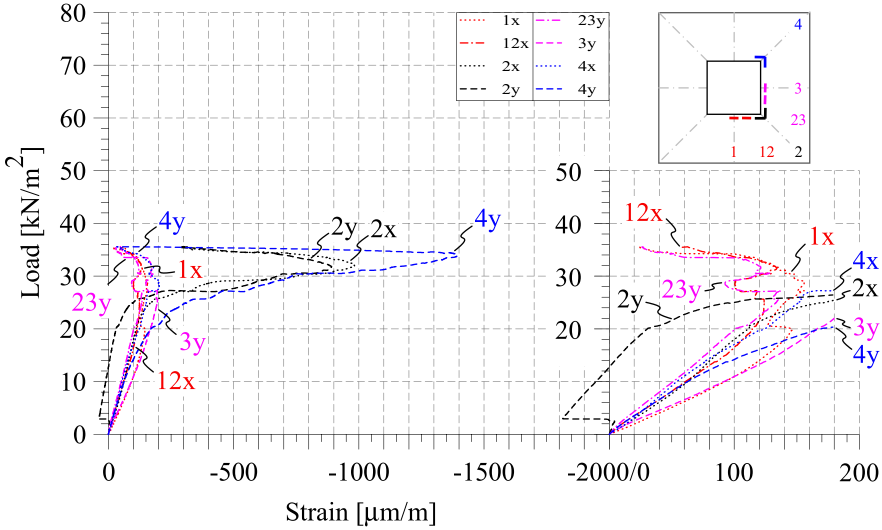


Figure A.10 *Load-strain relationship for the* <u>*concrete*</u> *in a slab weakened by a sawn up* <u>*small*</u> *opening, Sw. The right graph shows the initial stage up to* 180 µm/m*. The theoretical strain at yielding is* 96 µm/m *in tension i.e. at cracking.*

A.3 Slab with a sawn up small opening strengthened in corners with steel reinforcement in 45 degrees, Ss-45

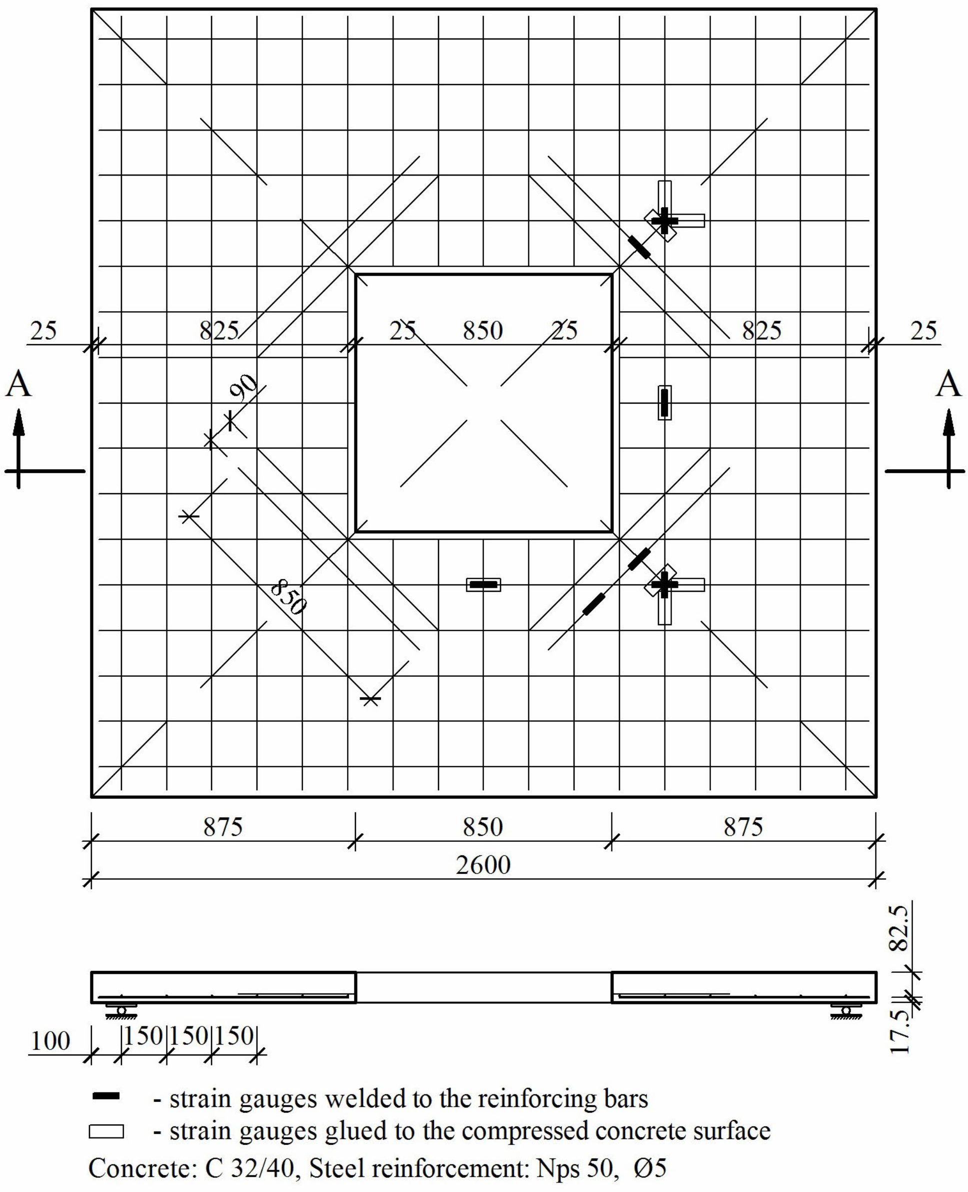


Figure A.11 Design of s lab with a sawn up small opening strengthened in corners with steel reinforcement in 45 degrees, Ss-45 and location of strain gauges. Drawing scale 1:25.

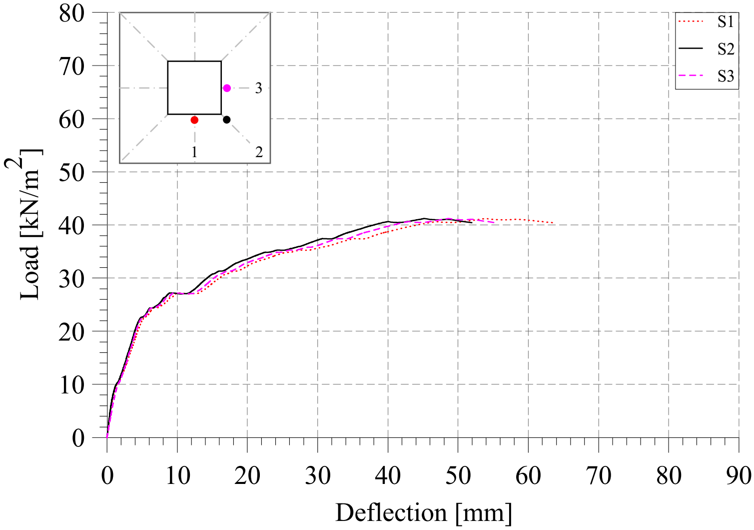


Figure A.12 Load-deflection relationship for a slab with a sawn up <u>*small*</u> *opening strengthened in corners with steel reinforcement in 45 degrees, Ss-45.*

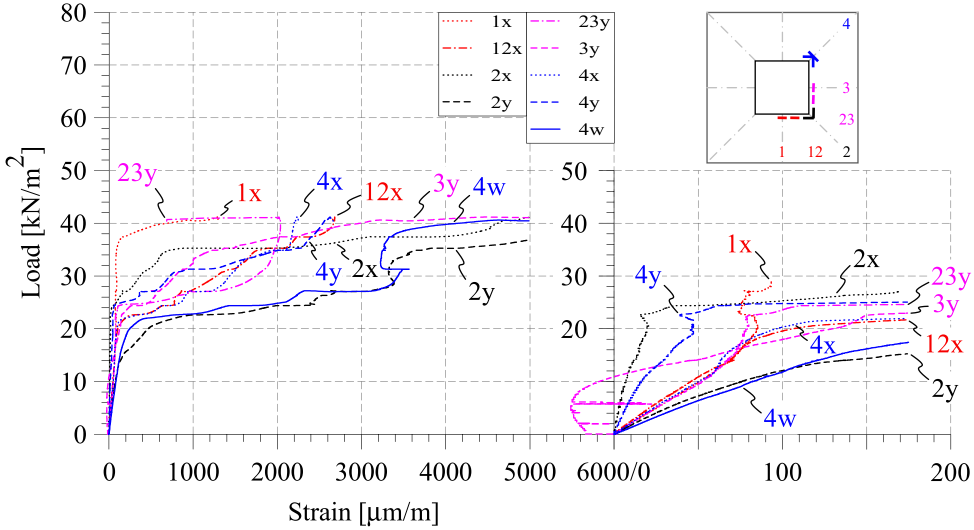


Figure A.13 Load-strain relationship for the <u>*steel reinforcement*</u> *in a slab with sawn up a* <u>*small*</u> *opening strengthened in corners with steel reinforcement in 45 degrees, Ss-45. The right graph shows the initial stage up to* 180 µm/m*. The theoretical strain at yielding is* 2762 µm/m *in tension.*

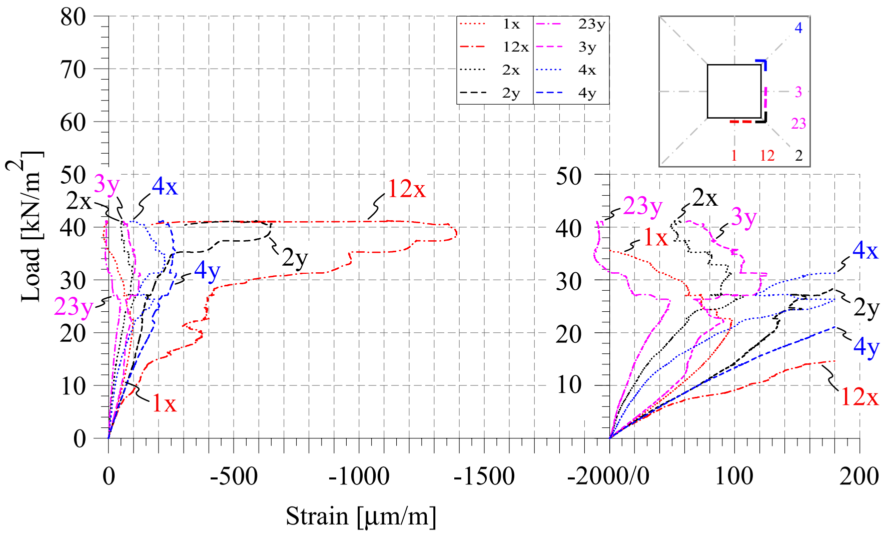


Figure A.14 *Load-strain relationship for the concrete in a slab with a sawn up small opening strengthened in corners with steel reinforcement in 45 degrees, Ss-45. The right graph shows the initial stage up to* 180 µm/m.

A.4 Slab with a sawn up small opening strengthened along the edges of the opening with CFRP, Sc-90

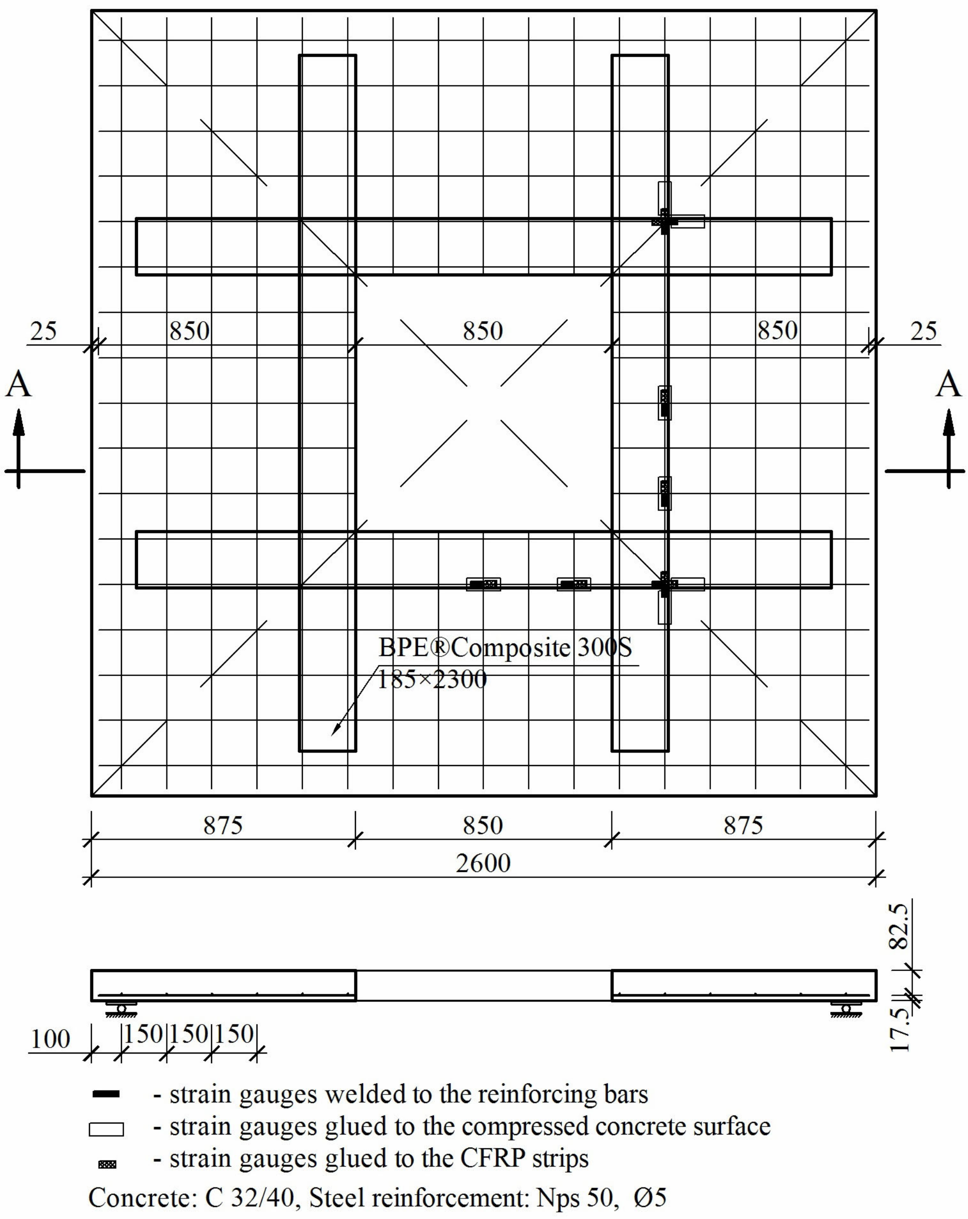


Figure A.15 Design of slab with a sawn up small opening strengthened along the edges of the opening with CFRP, Sc-90 and location of strain gauges. Drawing scale 1:25.

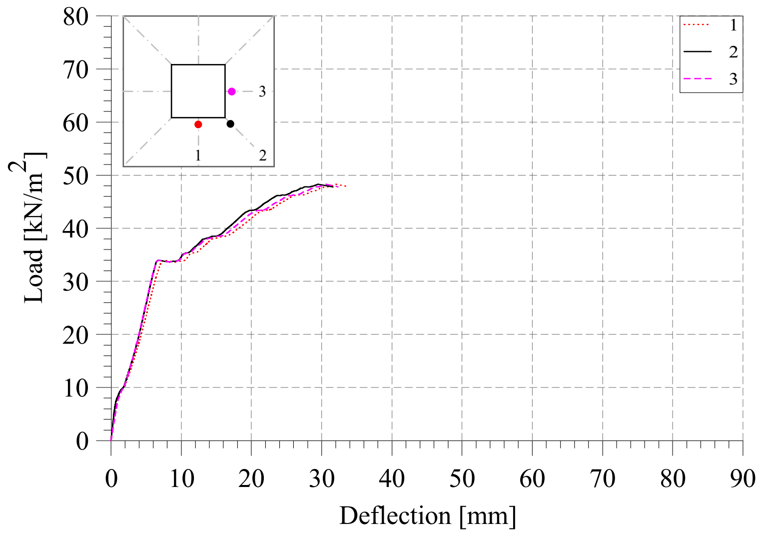


Figure A.16 *Load-deflection relationship for a slab with a sawn up* ***small*** *opening strengthened along the edges of the opening with CFRP, Sc-90.*

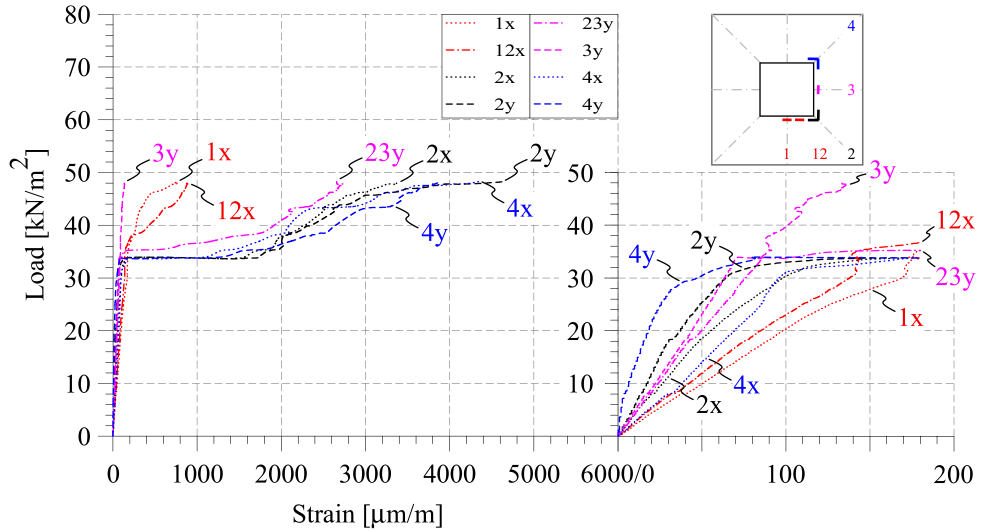


Figure A.17 *Load-strain relationship for the* ***steel reinforcement*** *in a slab with a sawn up* ***small*** *opening strengthened along the edges of the opening with CFRP, Sc-90. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

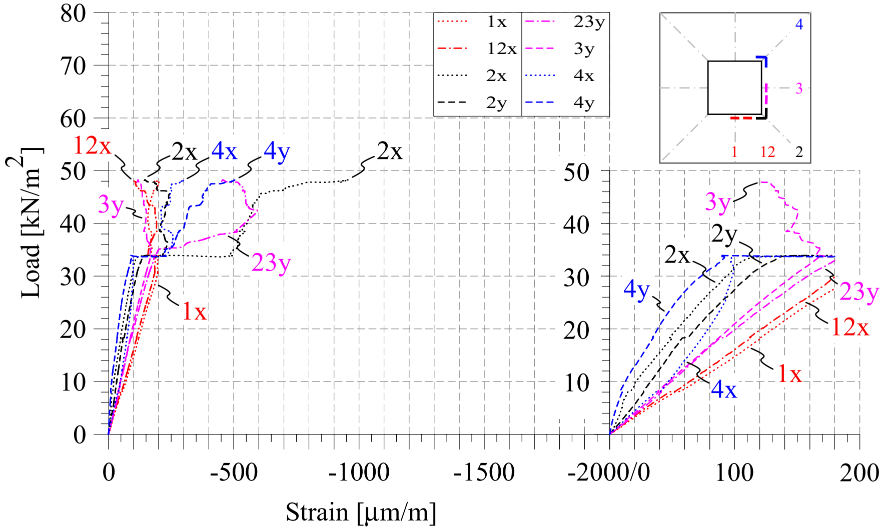


Figure A.18 *Load-strain relationship for the* <u>*concrete*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened along the edges of the opening with CFRP, Sc-90. The right graph shows the initial stage up to* 180 µm/m.

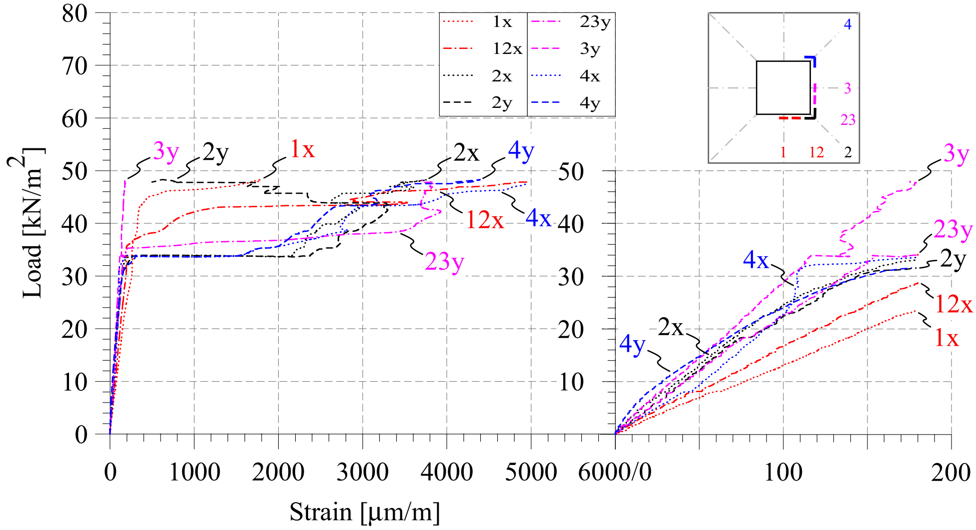


Figure A.19 *Load-strain relationship for the* <u>*CFRP*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened along the edges of the opening with CFRP, Sc-90. Strains higher than* 5000 µm/m *are truncated. The right graph shows the initial stage up to* 180 µm/m.

A.5 Slab with a sawn up small opening strengthened in the corners of the opening with CFRP, Sc-45

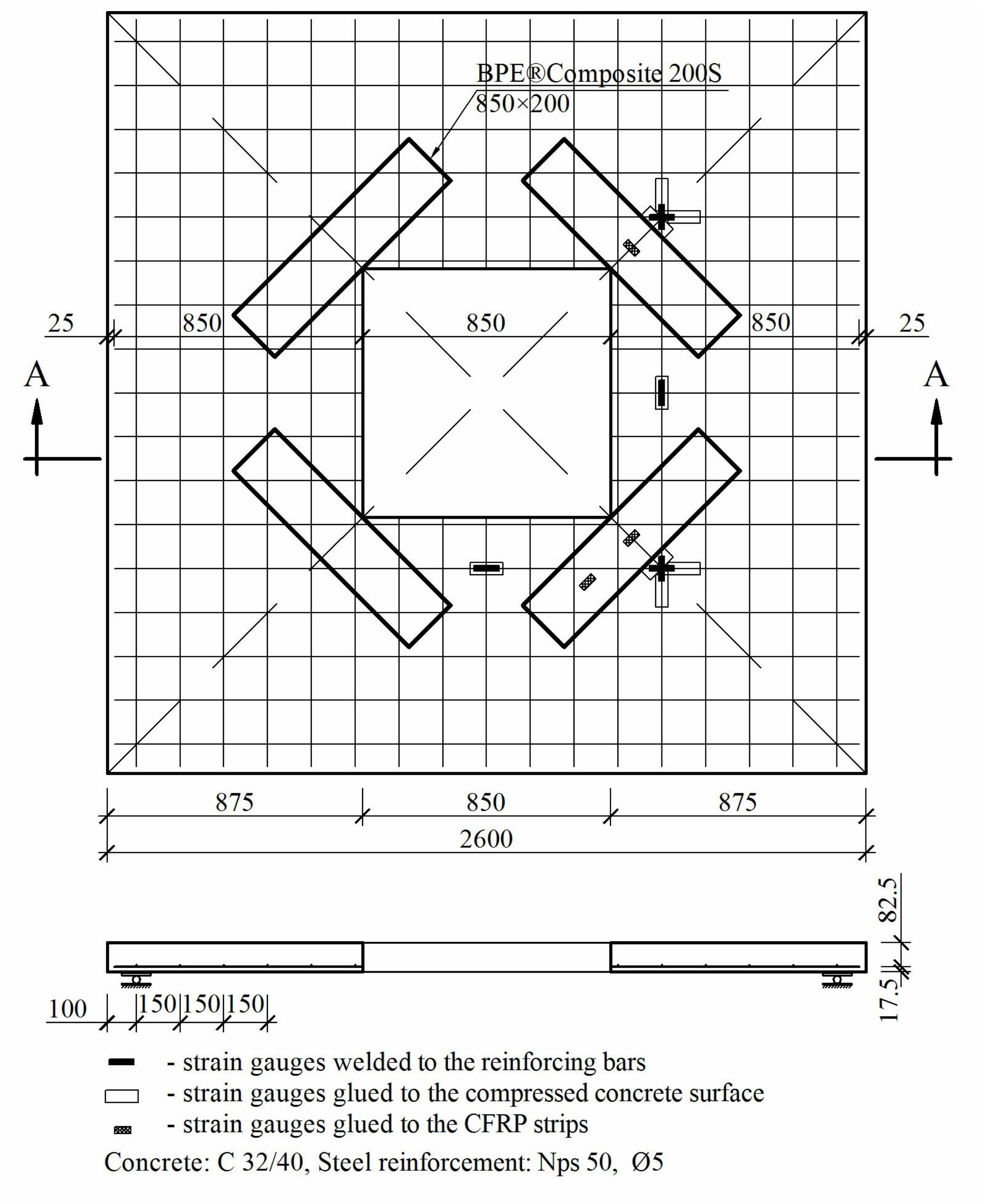


Figure A.20 Design of slab with a sawn up small opening strengthened in the corners of the opening with CFRP, Sc-45 and location of strain gauges. Drawing scale 1:25.

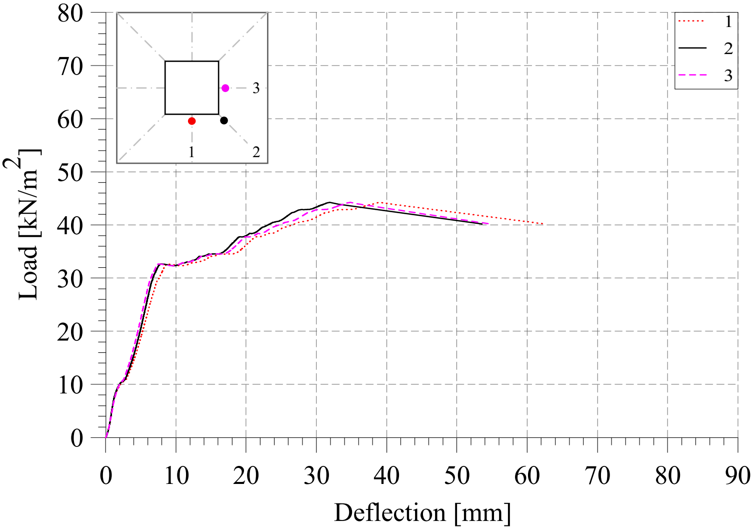


Figure A.21 *Load-deflection relationship for a slab with a sawn up* <u>*small*</u> *opening strengthened in the corners of the opening with CFRP, Sc-45.*

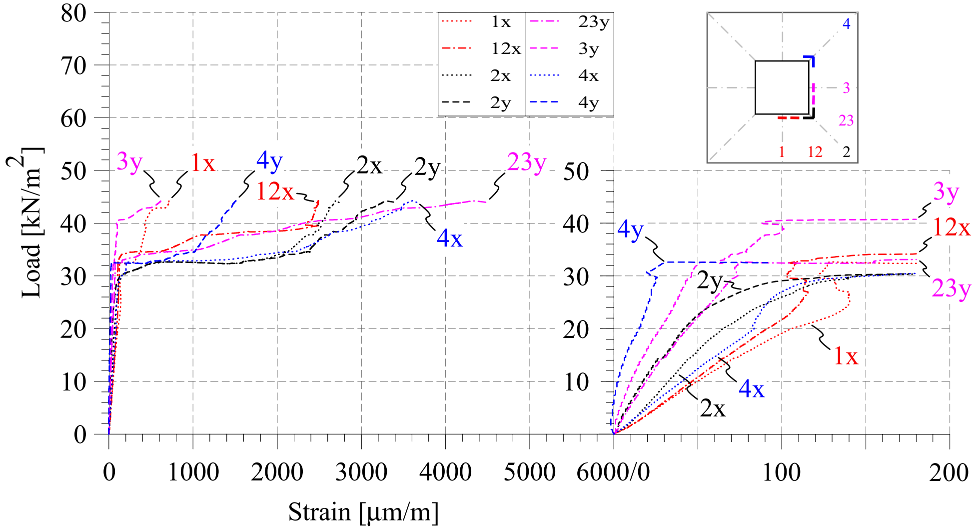


Figure A.22 *Load-strain relationship for the* <u>*steel reinforcement*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened in the corners of the opening with CFRP, Sc-45. The right graph shows the initial stage up to* 180 µm/m*. The theoretical strain at yielding is* 2762 µm/m *in tension.*

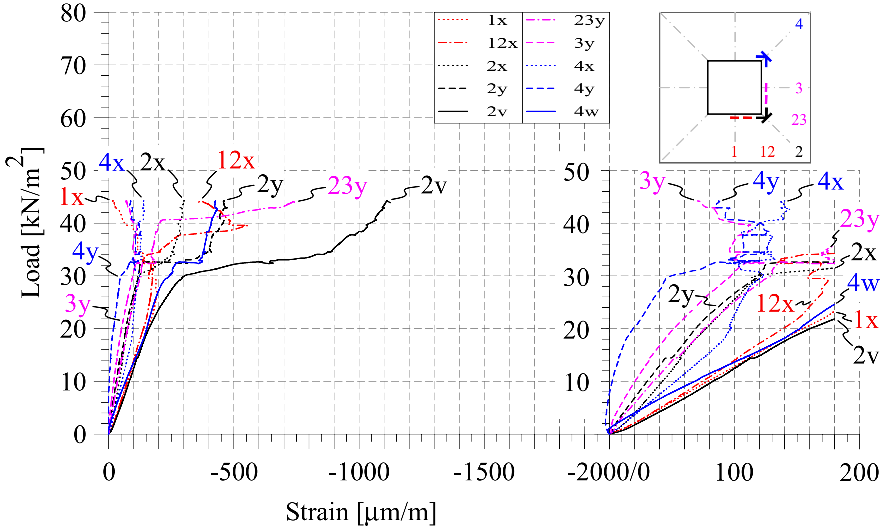


Figure A.23 *Load-strain relationship for the* <u>*concrete*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened in the corners of the opening with CFRP, Sc-45. The right graph shows the initial stage up to* 180 µm/m.

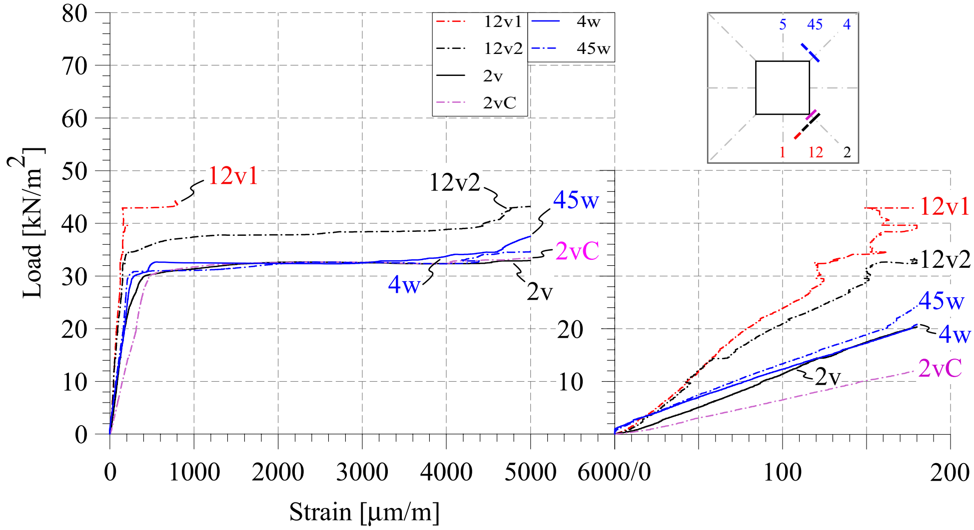


Figure A.24 *Load-strain relationship for the* <u>*CFRP*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened in the corners of the opening with CFRP, Sc-45. Strains higher than* 5000 µm/m *are truncated. The right graph shows the initial stage up to* 180 µm/m.

A.6 Slab with a sawn up small opening strengthened along and in the corners of the opening with CFRP, Sc-45, 90

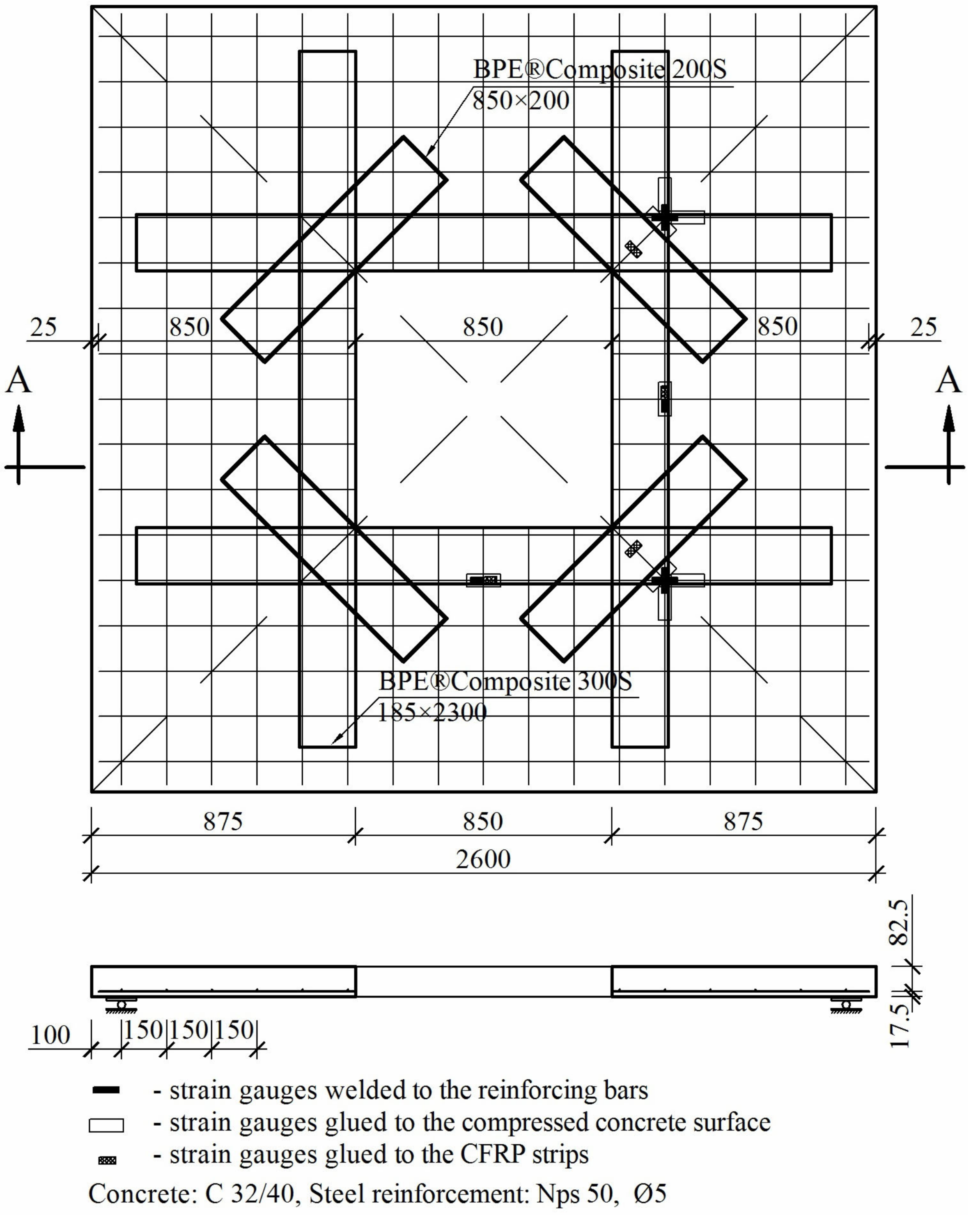


Figure A.25 Design of slab with a sawn up small opening strengthened along and in the corners of the opening with CFRP, Sc-45, 90 and location of strain gauges. Drawing scale 1:25.

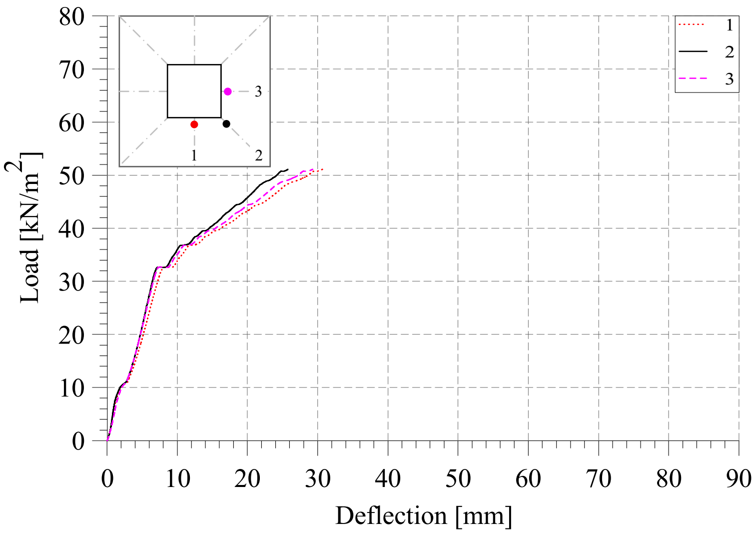


Figure A.26 *Load-deflection relationship for a slab with a sawn up* *__small__* *opening strengthened along and in the corners of the opening with CFRP, Sc-45, 90*

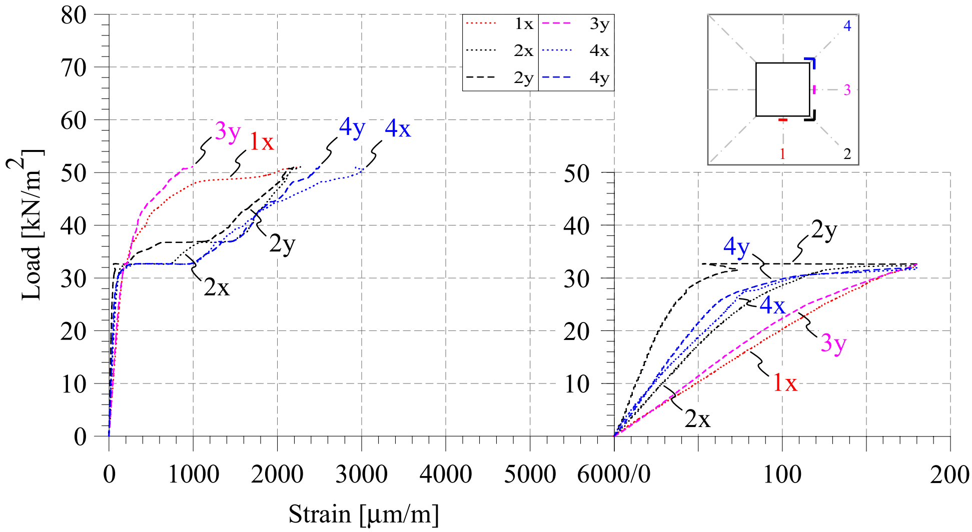


Figure A.27 *Load-strain relationship for the* *__steel reinforcement__* *in a slab with a sawn up* *__small__* *opening strengthened along and in the corners of the opening with CFRP, Sc-45, 90. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

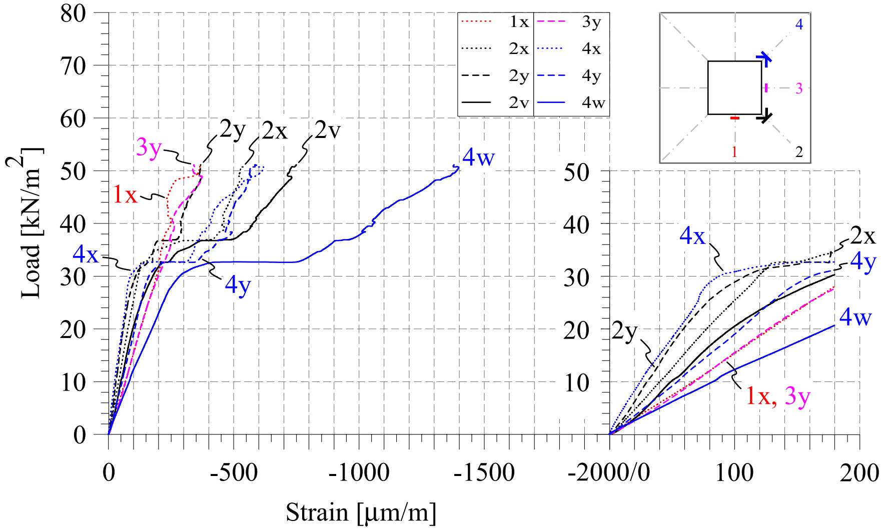


Figure A.28 *Load-strain relationship for the* <u>*concrete*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened along and in the corners of the opening with CFRP, Sc-45, 90. The right graph shows the initial stage up to* 180 µm/m.

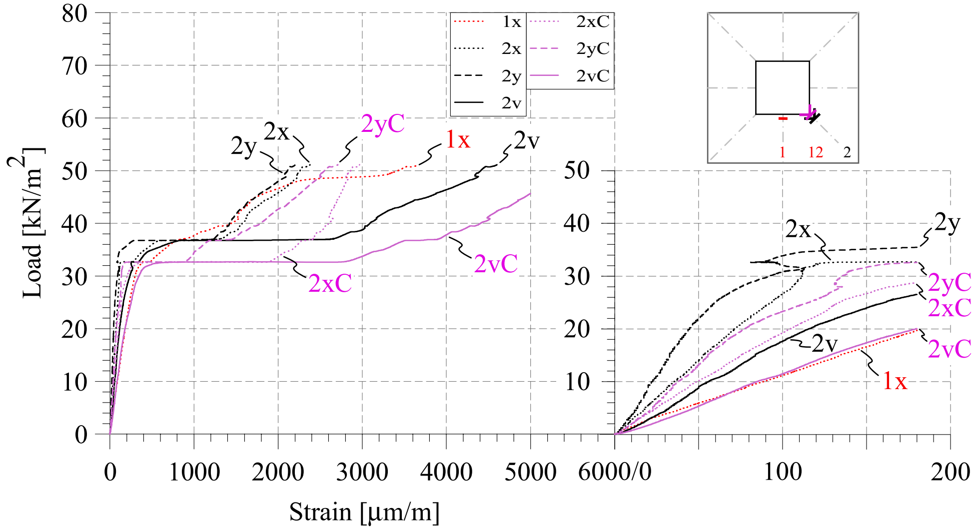


Figure A.29 *Load-strain relationship for the* <u>*CFRP*</u> *in a slab with a sawn up* <u>*small*</u> *opening strengthened along one side and in one corner of the opening with CFRP, Sc-45, 90. The right graph shows the initial stage up to* 180 µm/m.

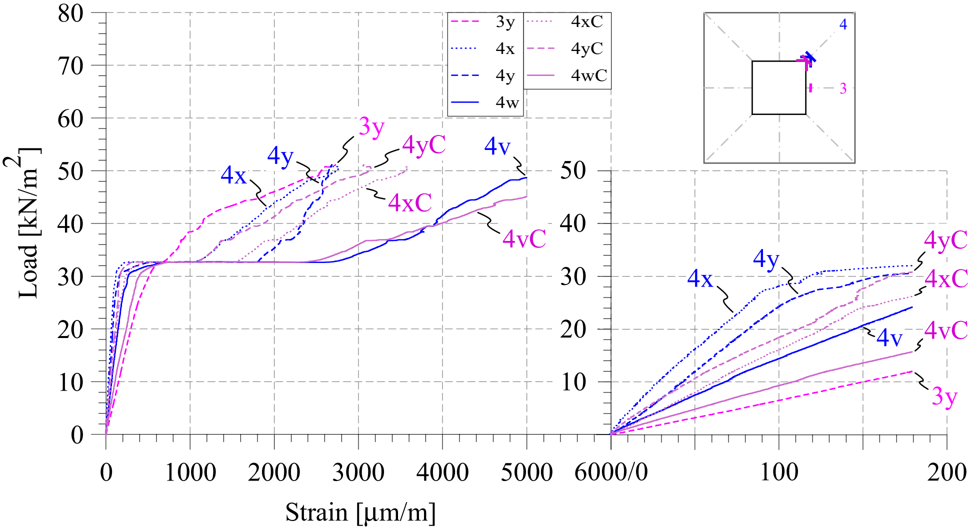


Figure A.30 *Load-strain relationship for the* *<u>CFRP</u>* *in a slab with a sawn up* *<u>small</u>* *opening strengthened along one side and in one corner of the opening with CFRP, Sc-45, 90. The right graph shows the initial stage up to* 180 µm/m.

A.7 Slab weakened by a sawn up large opening, Lw

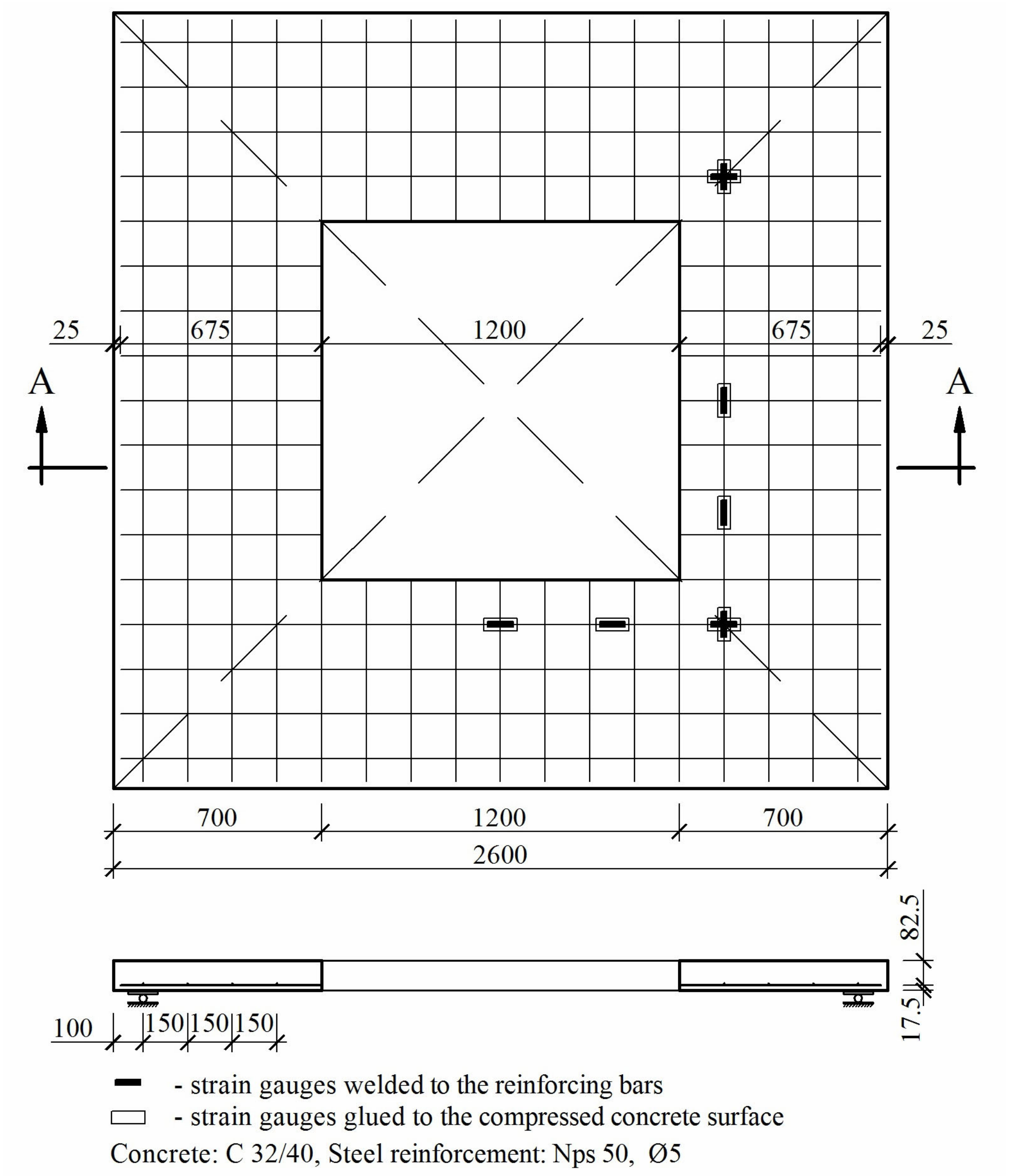


Figure A.31 Design of slab weakened by a sawn up large opening, Lw and location of strain gauges. Drawing scale 1:25.

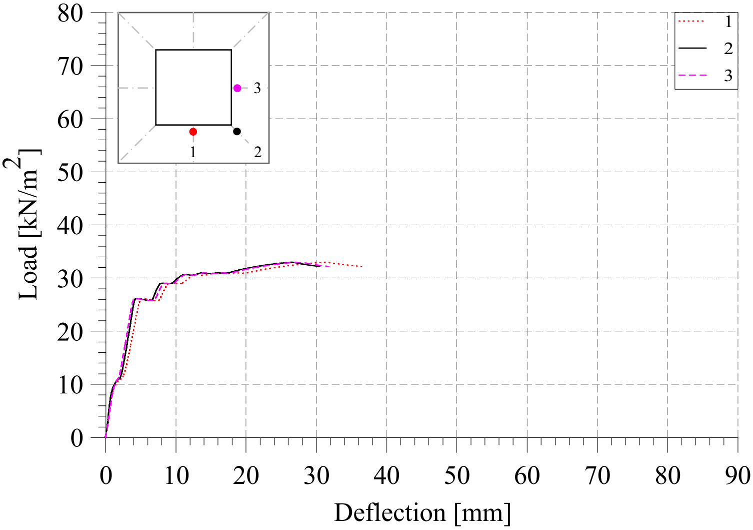


Figure A.32 Load-deflection relationship for a slab weakened by a sawn up large opening, Lw.

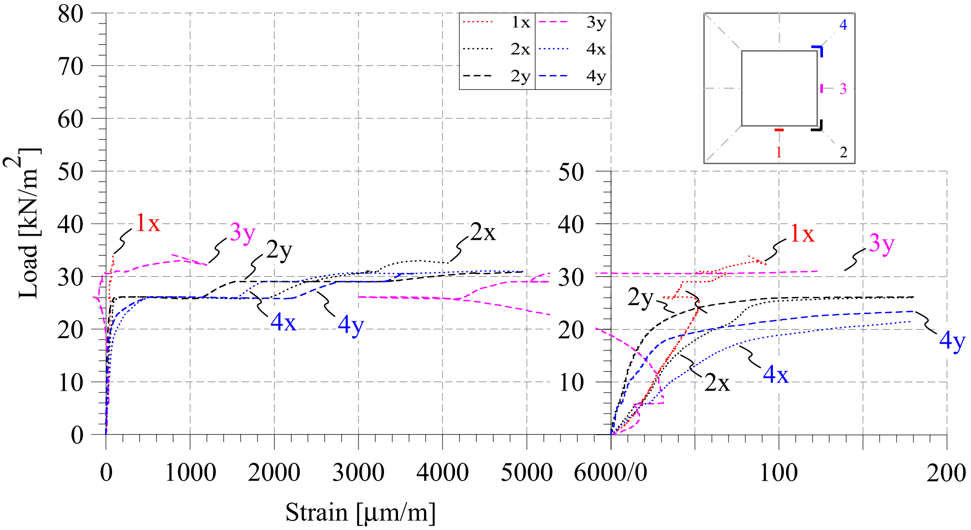


Figure A.33 Load-strain relationship for the steel reinforcement in a slab weakened by a sawn up large opening, Lw. The right graph shows the initial stage up to 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

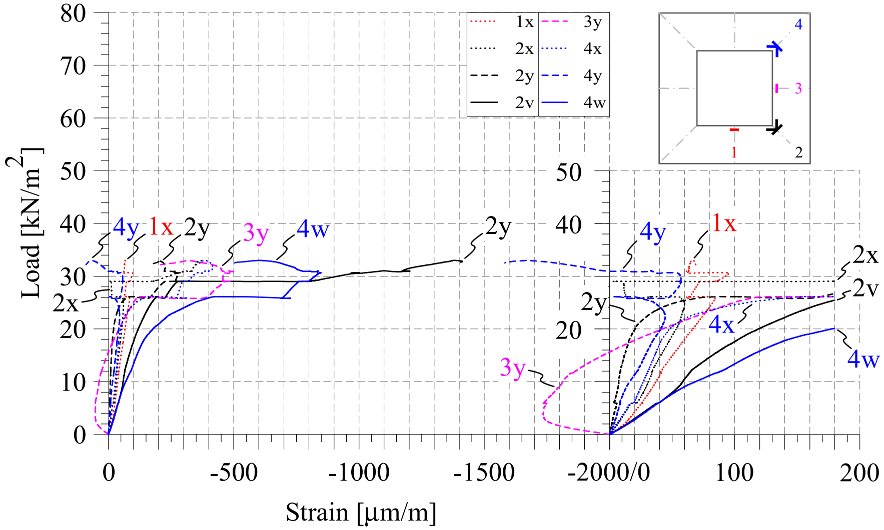
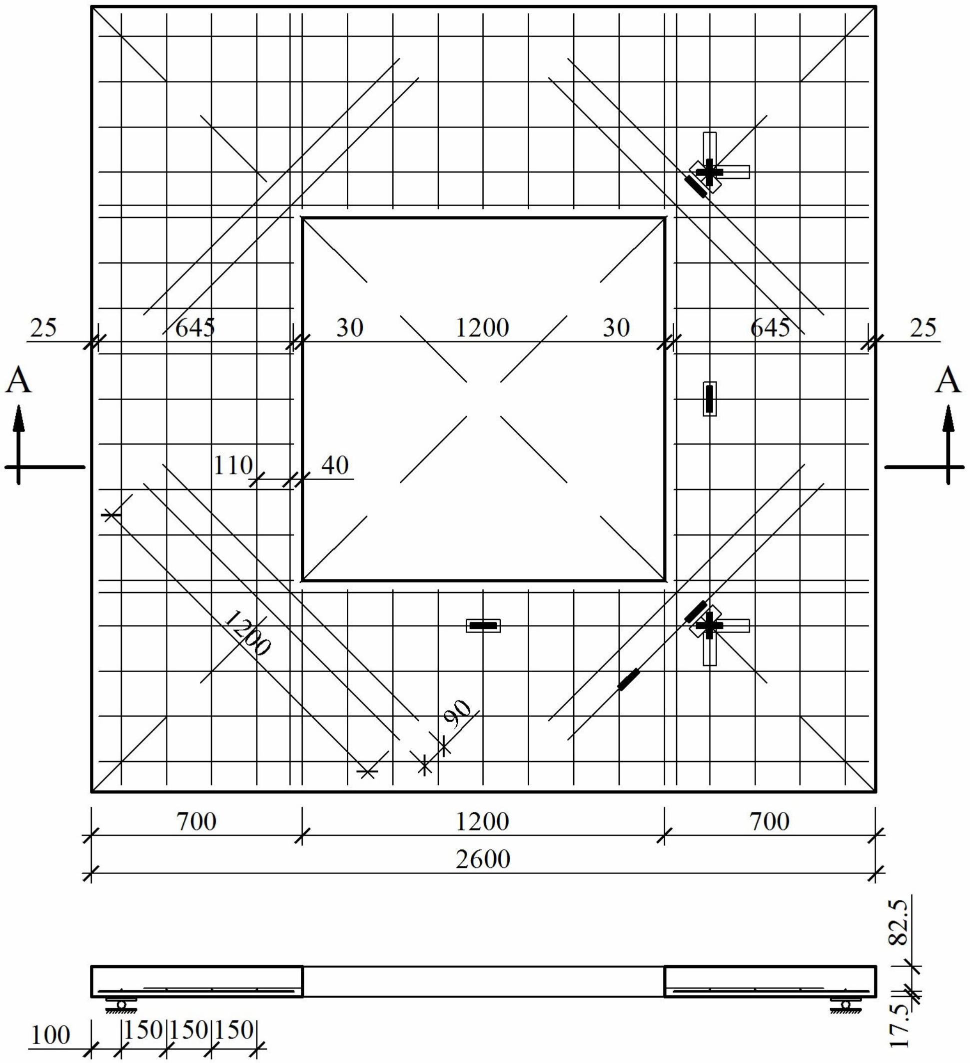


Figure A.34 Load-strain relationship for the <u>concrete</u> in a slab weakened by a sawn up <u>large</u> opening, Lw. The right graph shows the initial stage up to 180 µm/m.

A.8 Slab with a sawn up large opening strengthened in corners with steel reinforcement in 45 degrees, Ls-45



- strain gauges welded to the reinforcing bars

- strain gauges glued to the compressed concrete surface

Concrete: C 32/40, Steel reinforcement: Nps 50, Ø5

Figure A.35 Design of slab with a sawn up large opening strengthened in corners with steel reinforcement in 45 degrees, Ls-45 and location of strain gauges. Drawing scale 1:25.

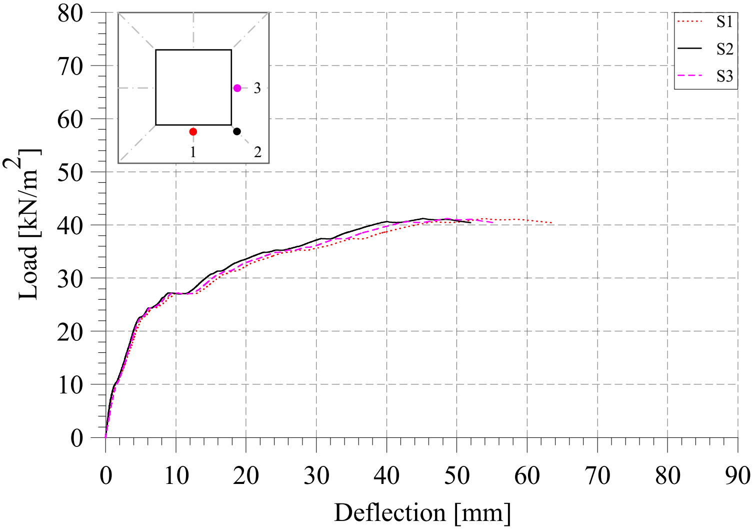


Figure A.36 Load-deflection relationship for a slab with a sawn up <u>*large*</u> *opening strengthened in corners with steel reinforcement in 45 degrees, Ls-45.*

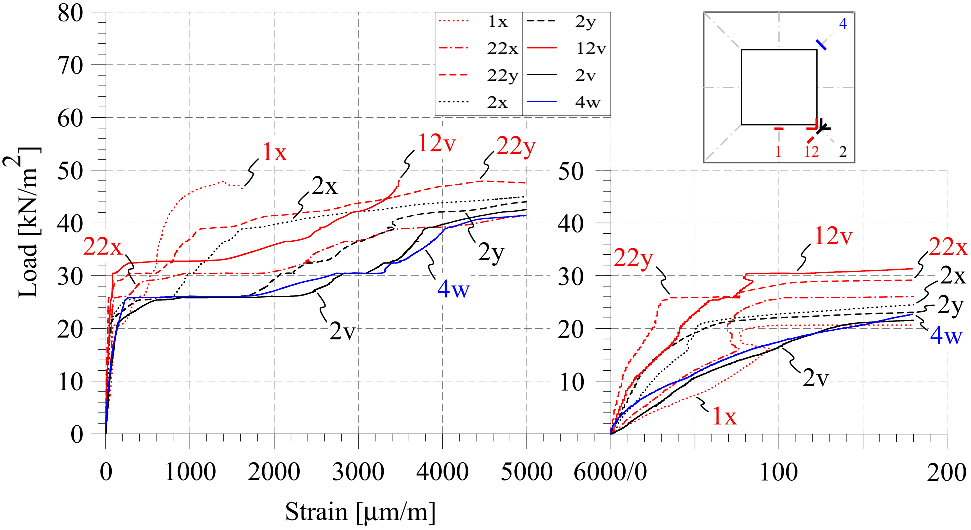


Figure A.37 Load-strain relationship for the <u>*steel reinforcement*</u> *in a slab with sawn up a* <u>*large*</u> *opening strengthened in corners with steel reinforcement in 45 degrees, Ls-45. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

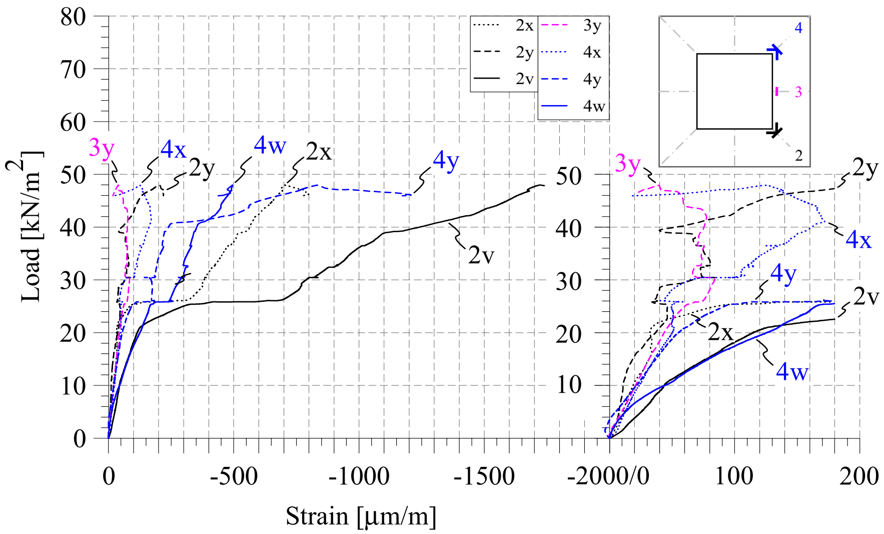


Figure A.38 *Load-strain relationship for the concrete in a slab with a sawn up large opening strengthened in corners with steel reinforcement in 45 degrees, Ls-45. The right graph shows the initial stage up to* 180 µm/m.

A.9 Slab with a sawn up large opening strengthened along the edges of the opening with CFRP, Lc-90

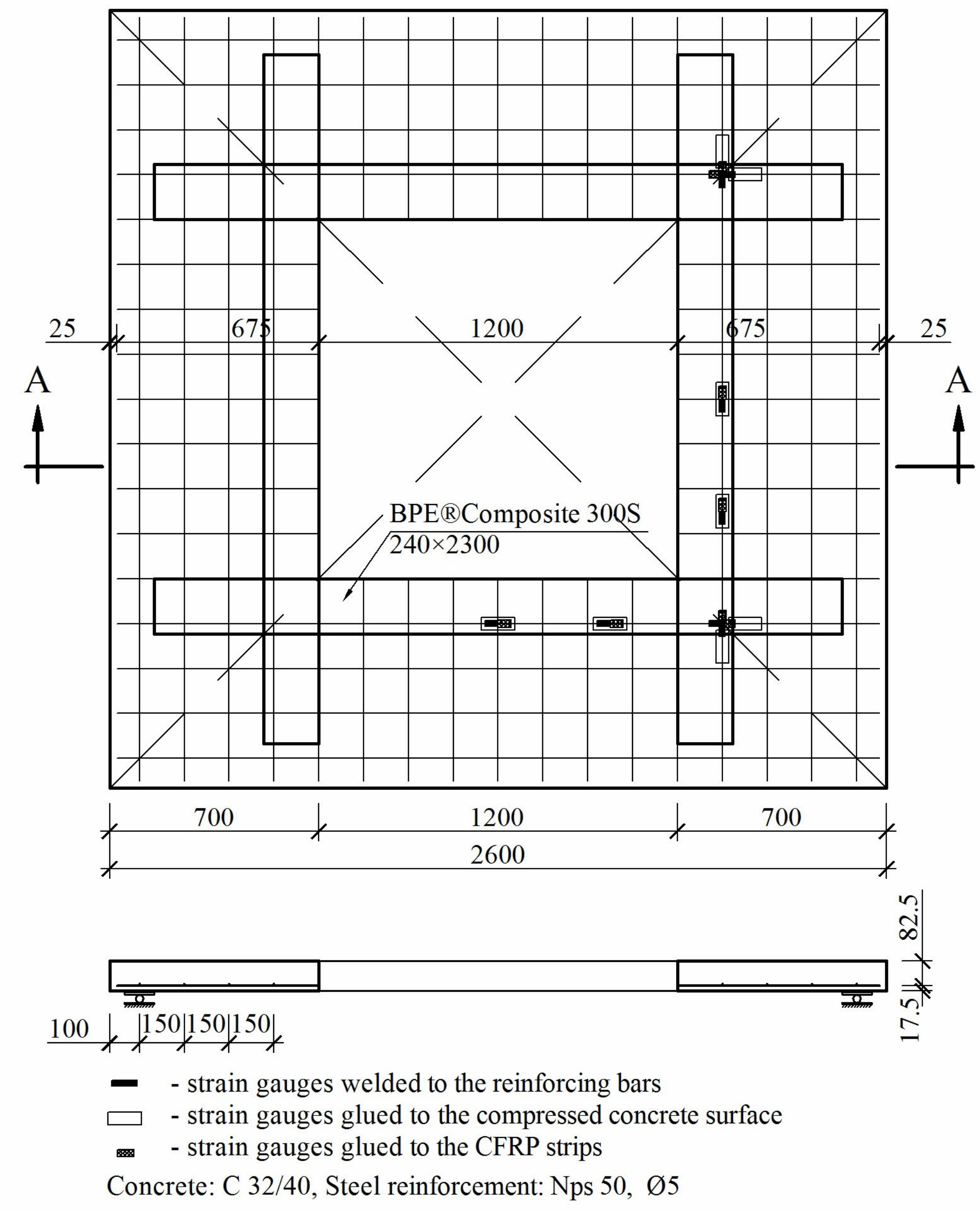


Figure A.39 Design of slab with a sawn up large opening strengthened along the edges of the opening with CFRP, Lc-90 and location of strain gauges. Drawing scale 1:25.

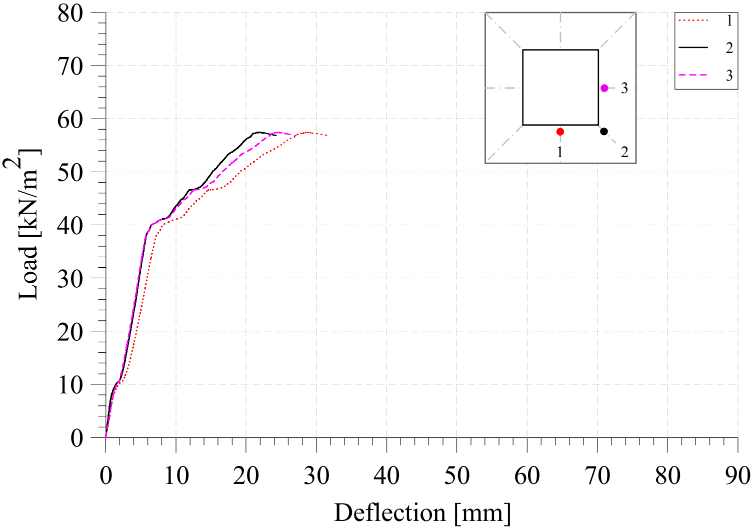


Figure A.40 *Load-deflection relationship for a slab with a sawn up* *large* *opening strengthened along the edges of the opening with CFRP, Lc-90.*

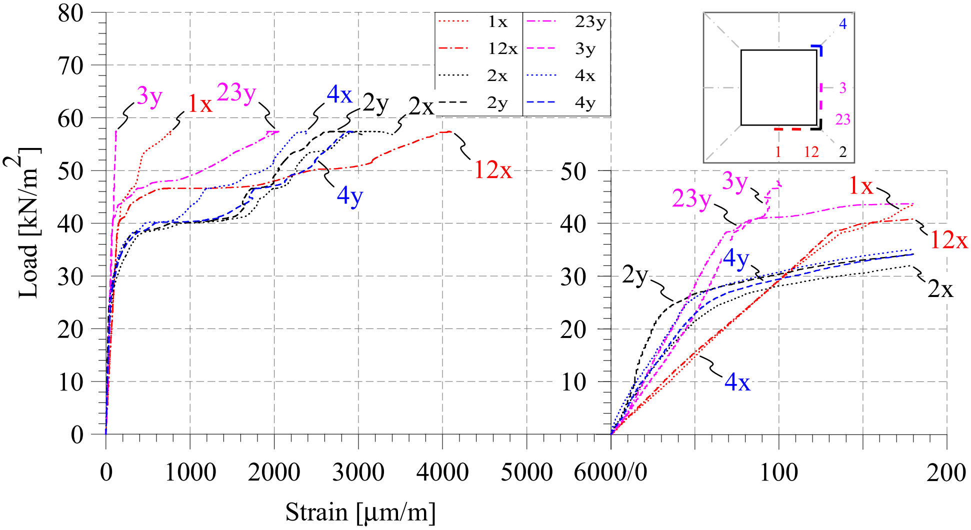


Figure A.41 *Load-strain relationship for the* *steel reinforcement* *in a slab with a sawn up* *large* *opening strengthened along the edges of the opening with CFRP, Lc-90. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

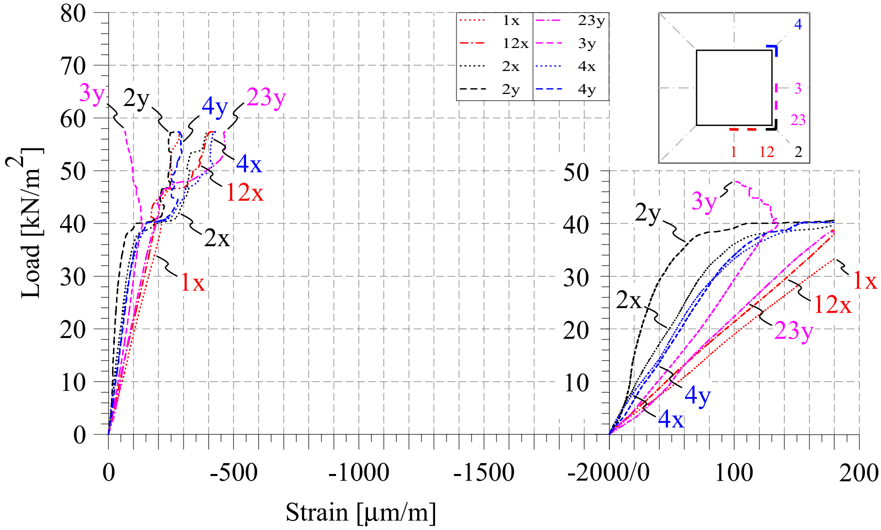


Figure A.42 *Load-strain relationship for the* *<u>concrete</u>* *in a slab with a sawn up* *<u>large</u>* *opening strengthened along the edges of the opening with CFRP, Lc-90. The right graph shows the initial stage up to* 180 µm/m.

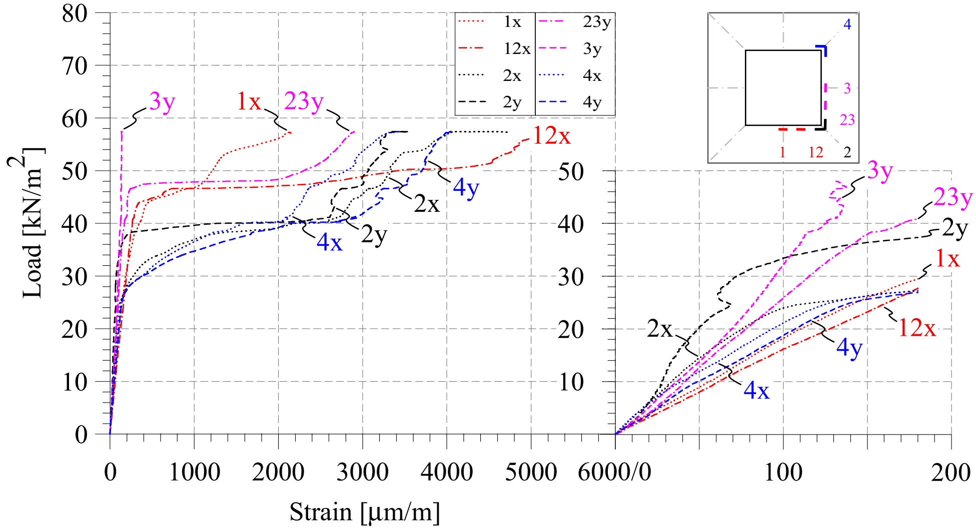


Figure A.43 *Load-strain relationship for the* *<u>CFRP</u>* *in a slab with a sawn up* *<u>large</u>* *opening strengthened along the edges of the opening with CFRP, Lc-90. Strains higher than* 5000 µm/m *are truncated. The right graph shows the initial stage up to* 180 µm/m.

A.10 Slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45

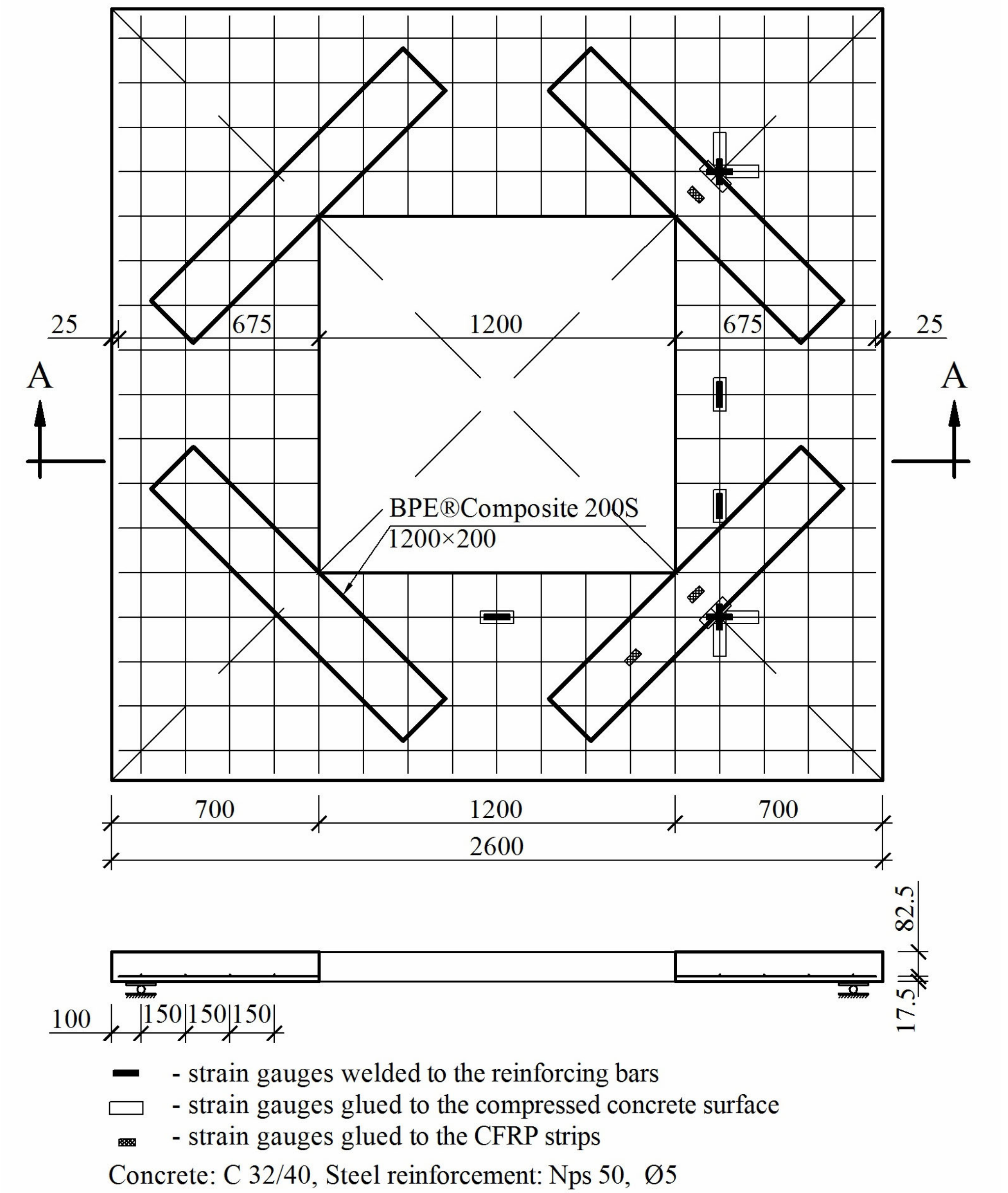


Figure A.44 Design of slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45 and location of strain gauges. Drawing scale 1:25.

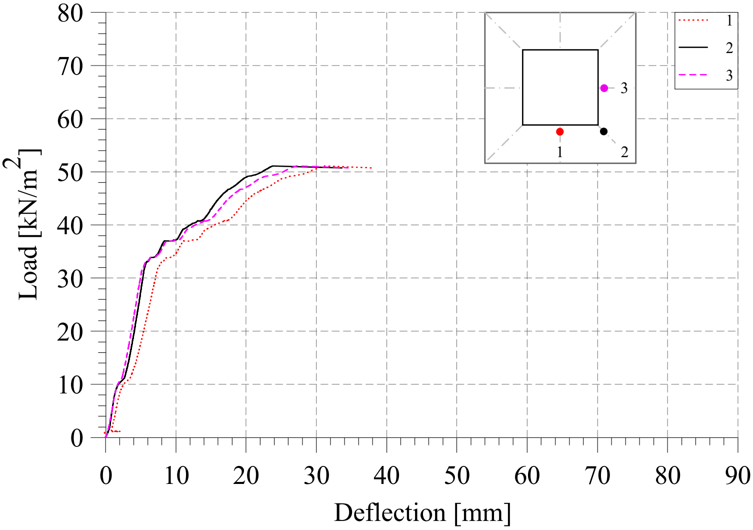


Figure A.45 Load-deflection relationship for a slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45.

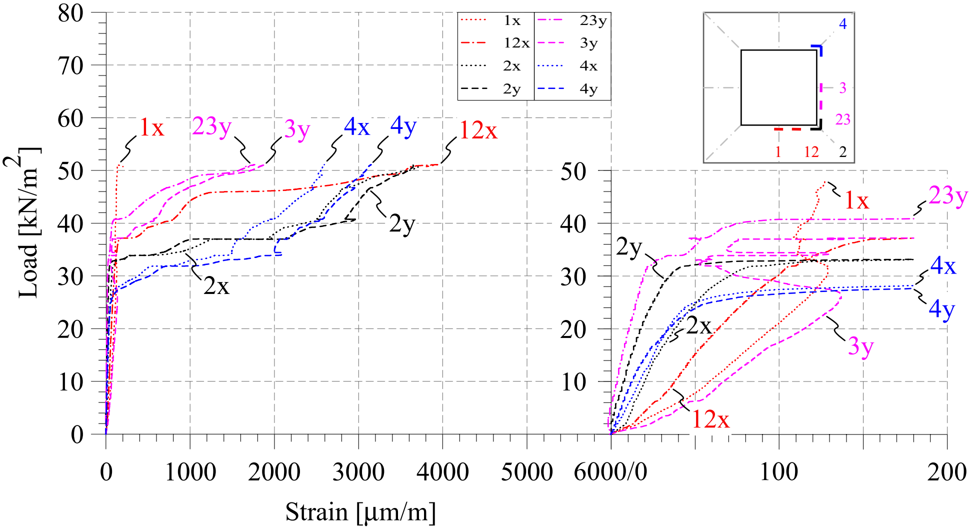


Figure A.46 Load-strain relationship for the steel reinforcement in a slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45. The right graph shows the initial stage up to 180 µm/m*. The theoretical strain at yielding is* 2762 µm/m *in tension.*

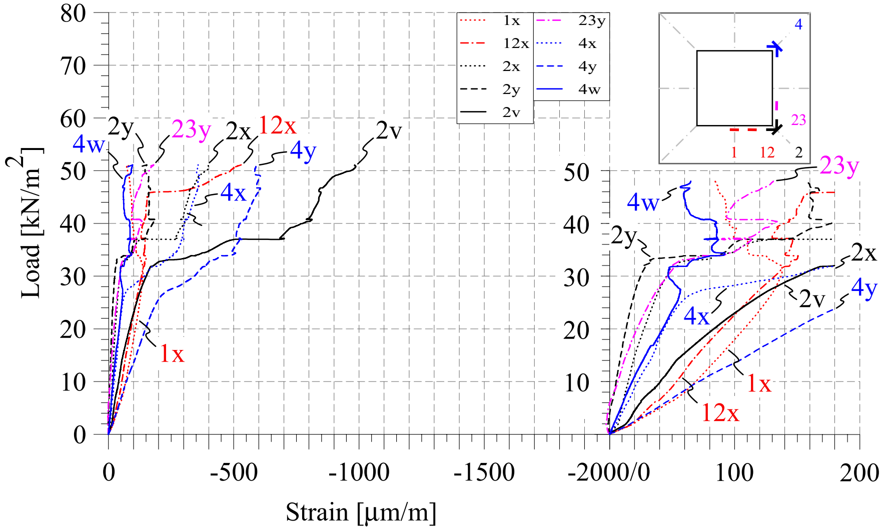


Figure A.47 Load-strain relationship for the concrete in a slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45. The right graph shows the initial stage up to 180 µm/m.

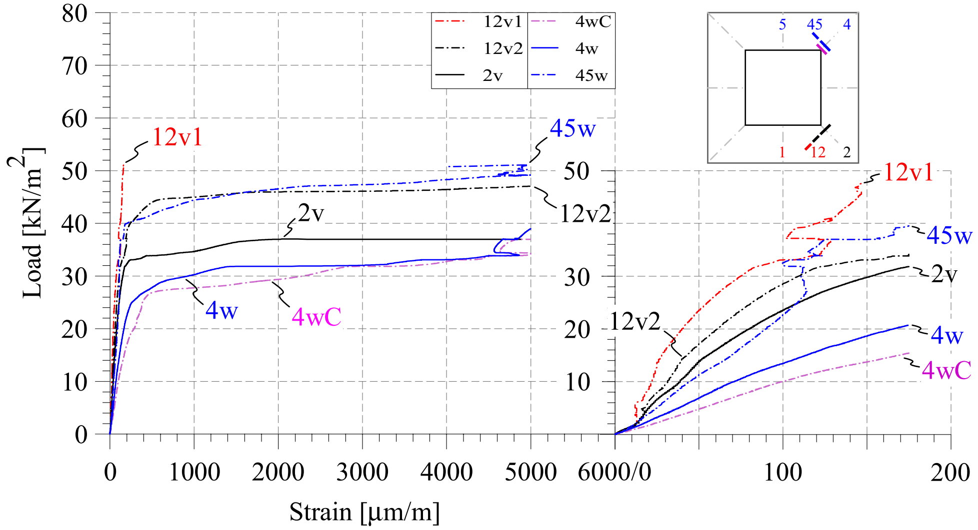


Figure A.48 Load-strain relationship for the CFRP in a slab with a sawn up large opening strengthened in the corners of the opening with CFRP, Lc-45. Strains higher than 5000 µm/m *are truncated. The right graph shows the initial stage up to* 180 µm/m.

A.11 Slab with a sawn up large opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90

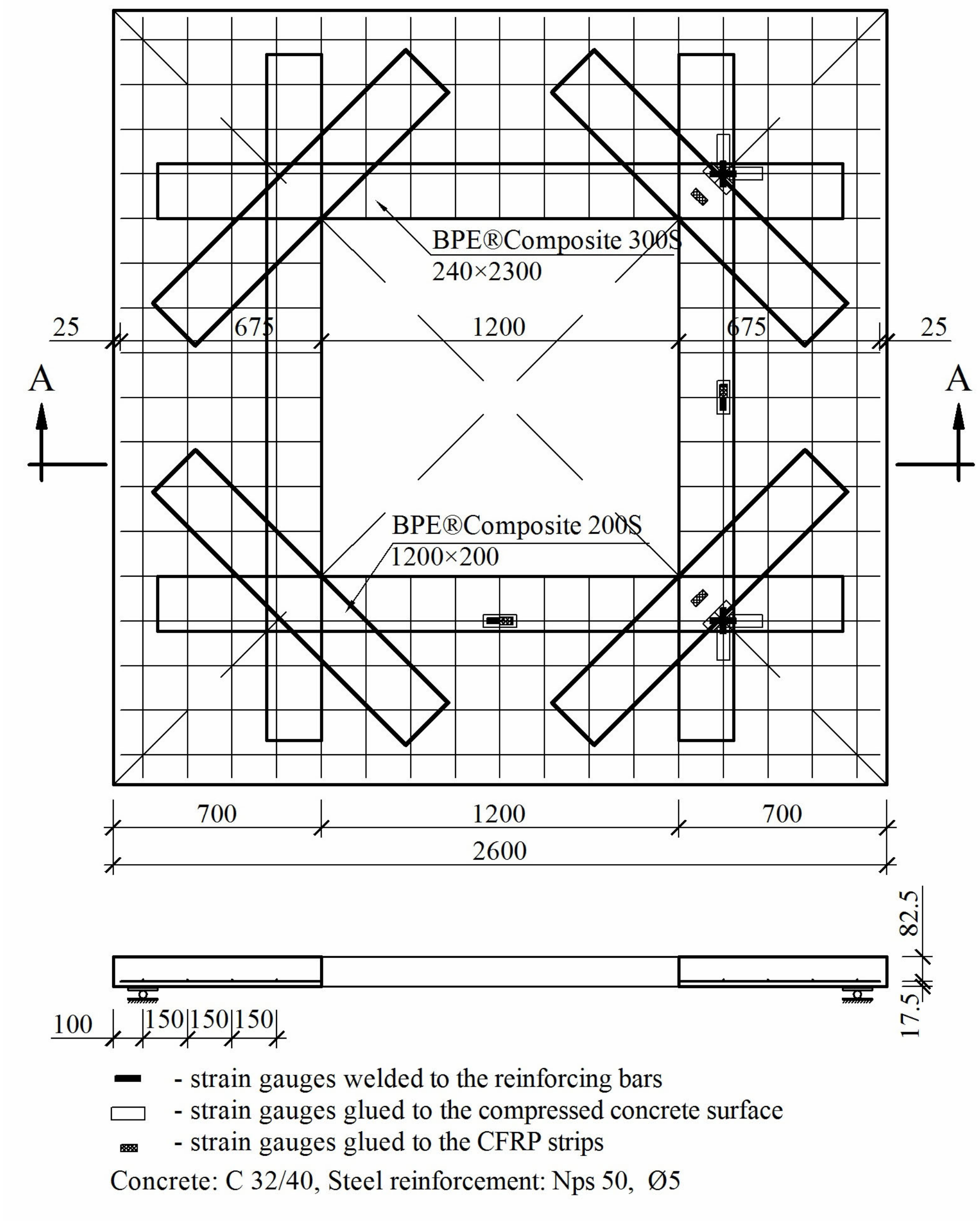


Figure A.49 Design of slab with a sawn up large opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90 and location of strain gauges. Drawing scale 1:25.

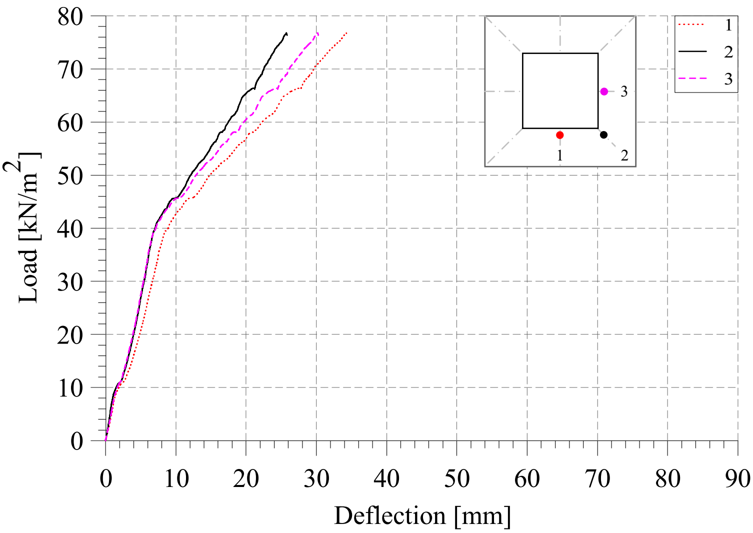


Figure A.50 Load-deflection relationship for a slab with a sawn up *<u>large</u>* *opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90*

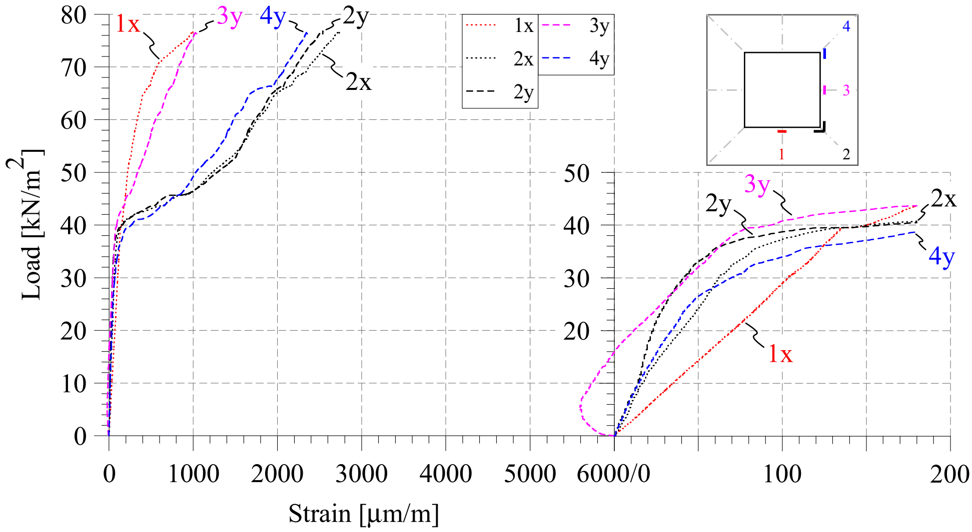


Figure A.51 Load-strain relationship for the *<u>steel reinforcement</u>* *in a slab with a sawn up* *<u>large</u>* *opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90. The right graph shows the initial stage up to* 180 µm/m. *The theoretical strain at yielding is* 2762 µm/m *in tension.*

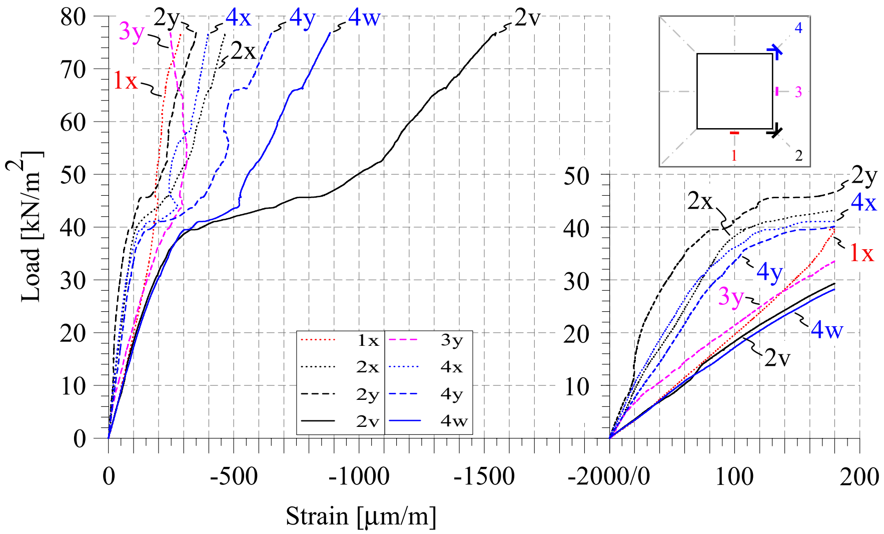


Figure A.52 *Load-strain relationship for the concrete in a slab with a sawn up large opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90. The right graph shows the initial stage up to* 180 µm/m.

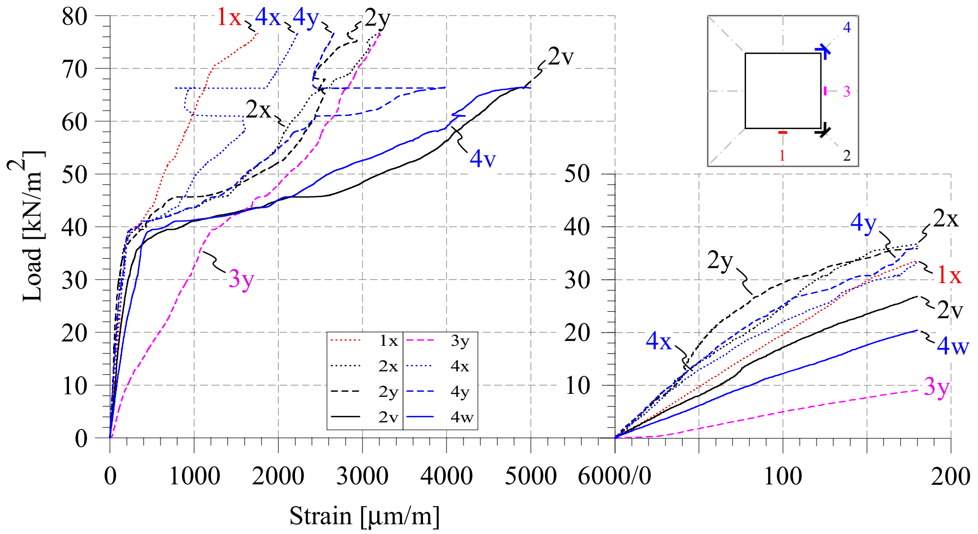


Figure A.53 *Load-strain relationship for the CFRP in a slab with a sawn up large opening strengthened along and in the corners of the opening with CFRP, Lc-45, 90. The right graph shows the initial stage up to 180 µm/m.*

Appendix B CASE STUDY – RESULT FROM STRENGTHENING ANALYSIS

B.1 Result from FE-analysis before reconstruction

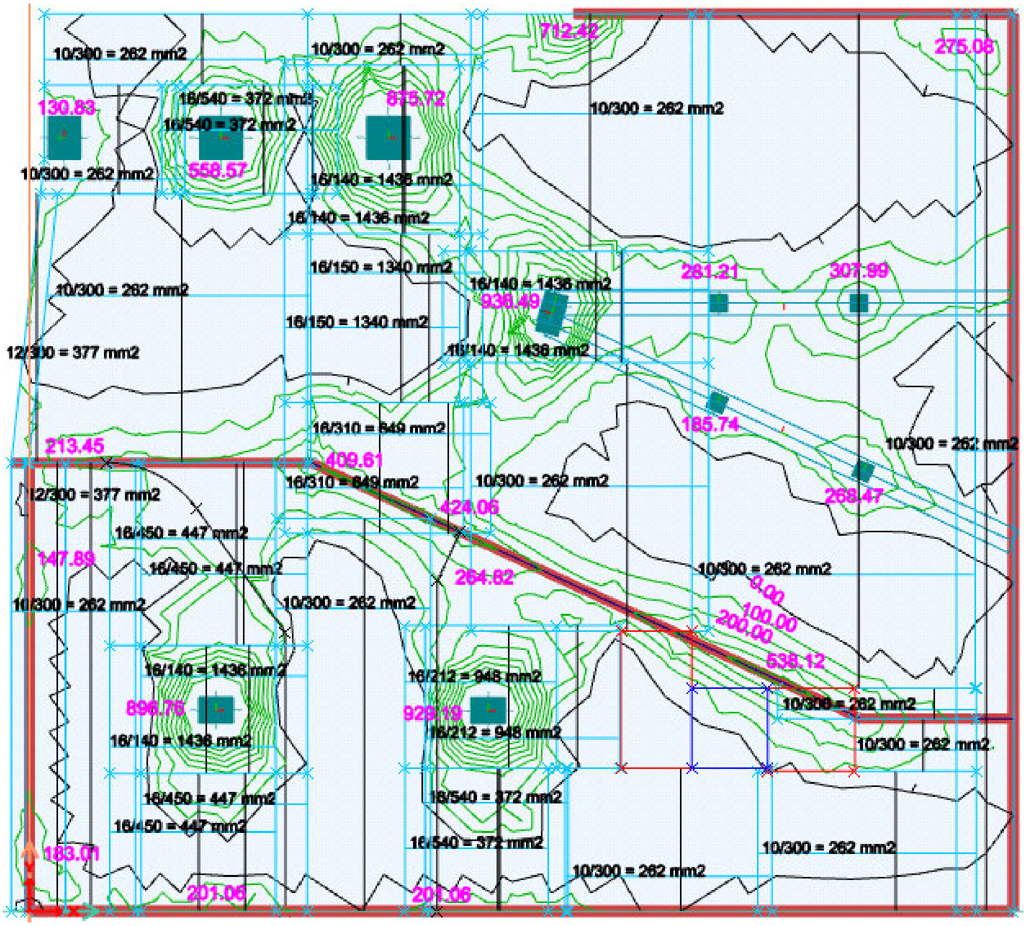


Figure B.1 *Required top reinforcement (magenta) compared to the existing reinforcement in x-direction before reconstruction* [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

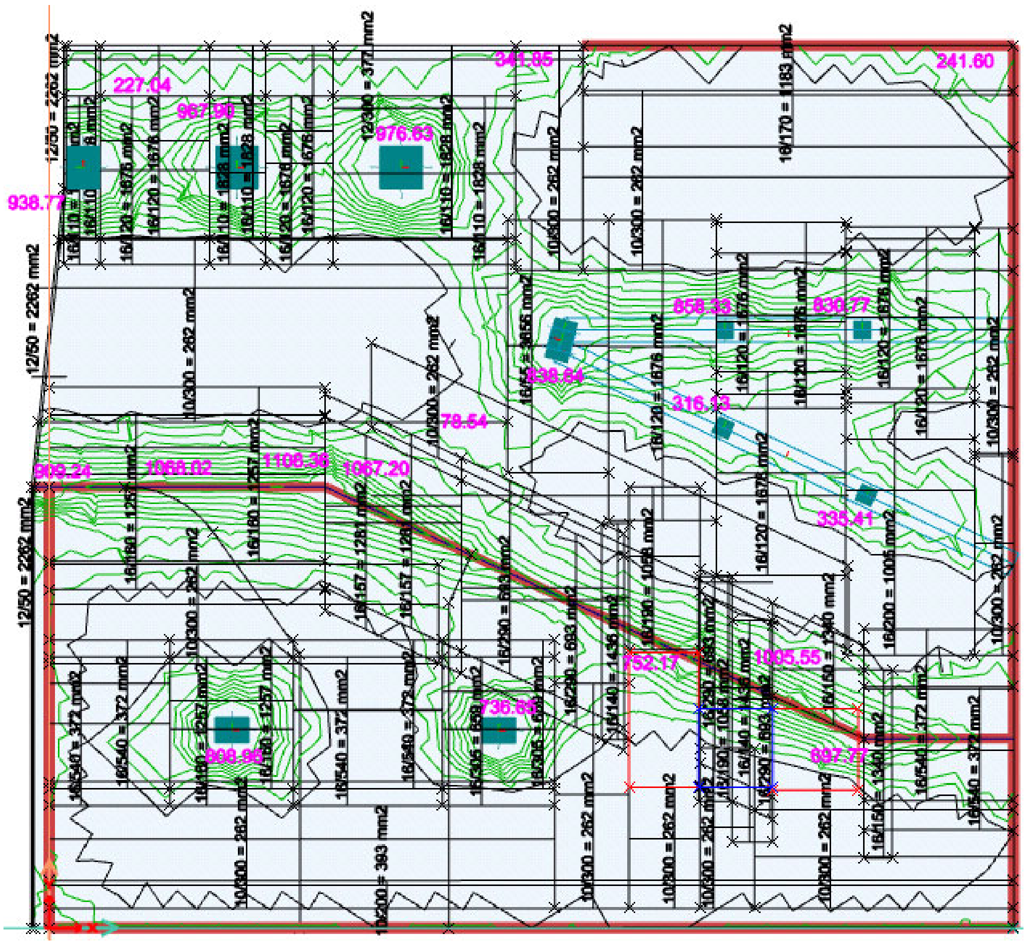


Figure B.2 *Required top reinforcement (magenta) compared to the existing reinforcement in y-direction* [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

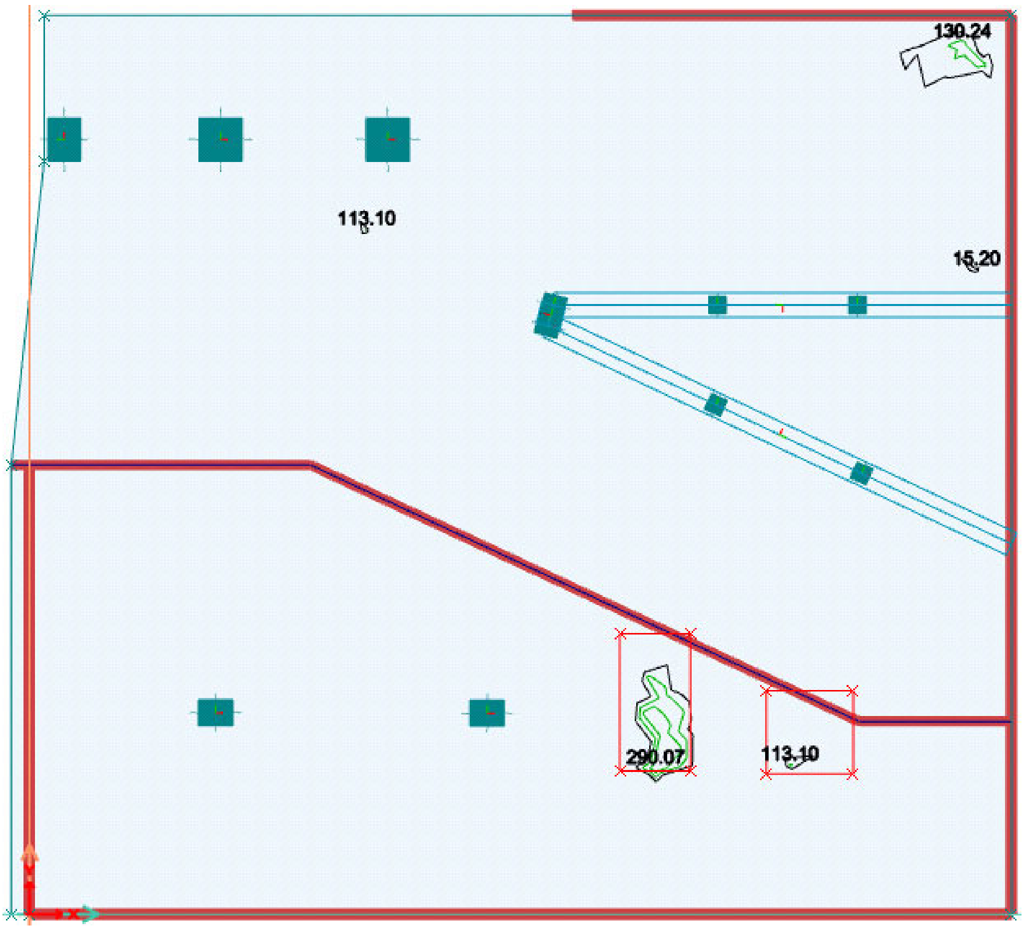


Figure B.3 *Required bottom reinforcement in x-direction* [mm^2/m]. *Red rectangles consist of stiff areas with openings surrounded by cast on walls.*

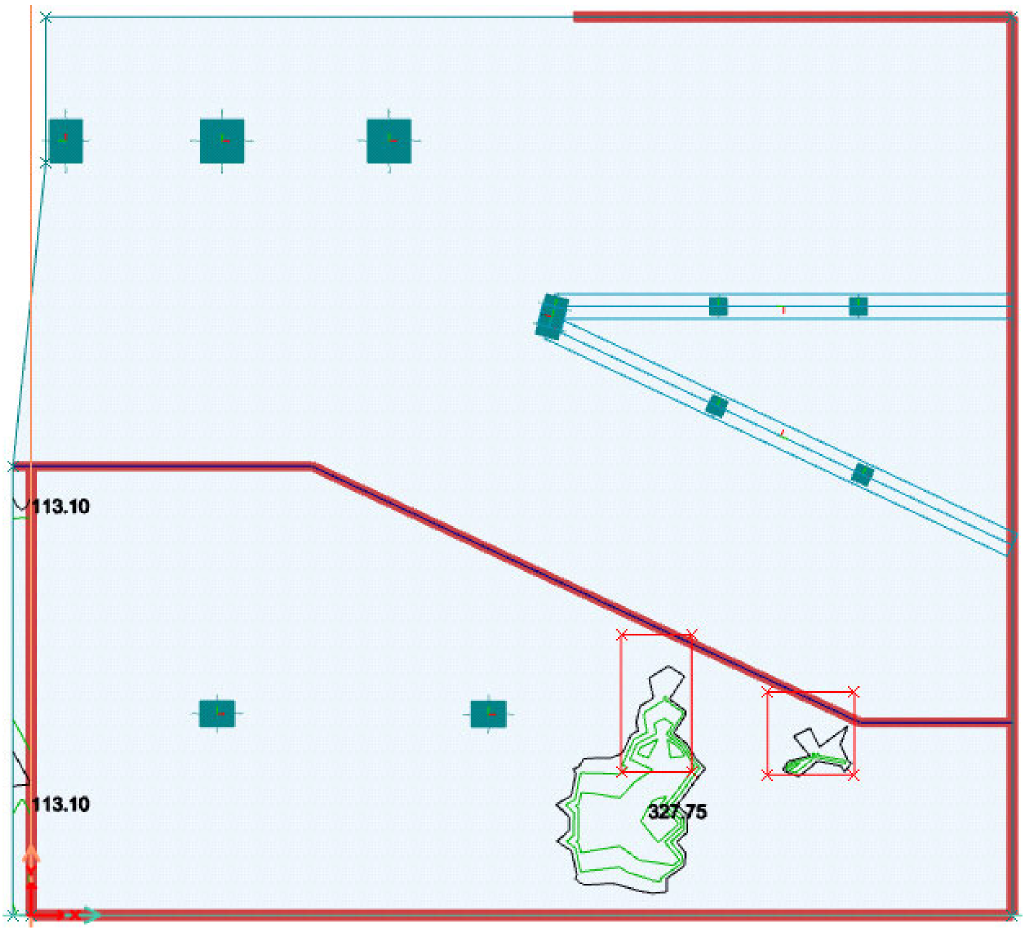


Figure B.4 *Required bottom reinforcement in y-direction* [mm^2/m]. *Red and blue rectangles consist of stiff areas with openings surrounded by cast on walls.*

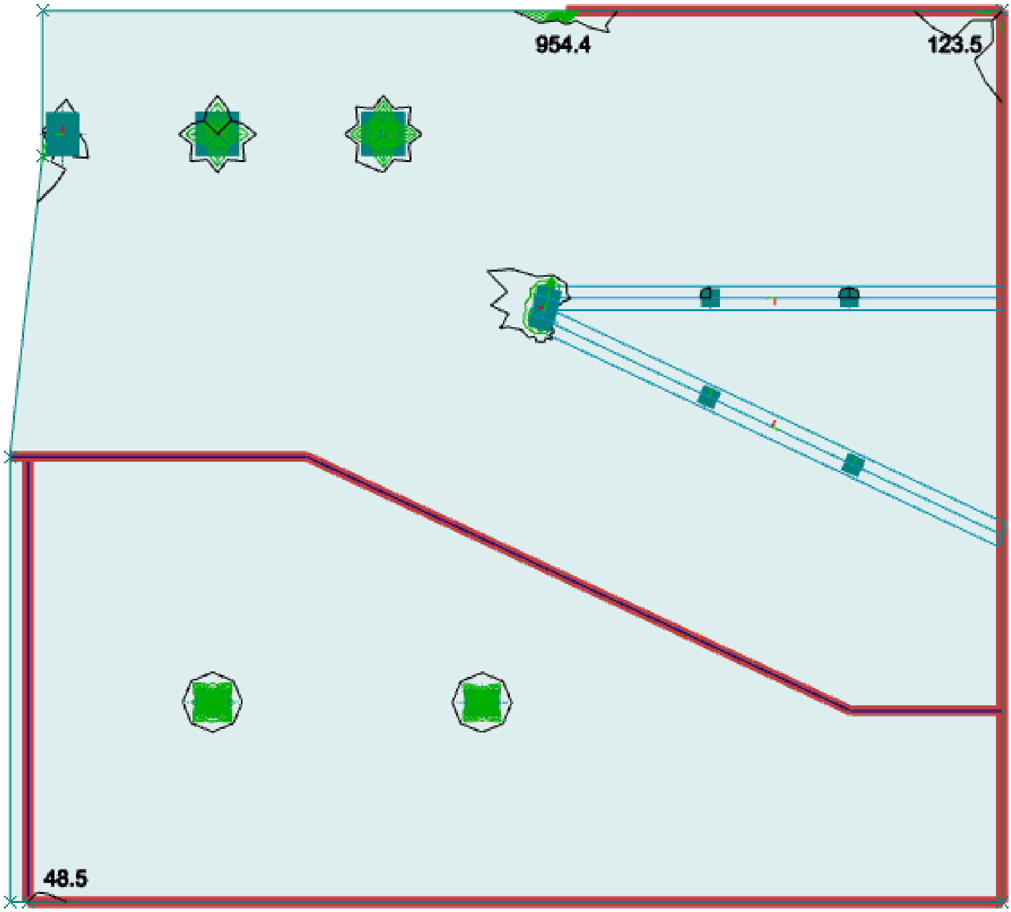


Figure B.5 *Missing shear capacity* [kN/m] when existing reinforcement is considered.

B.2 Result from FE-analysis after reconstruction

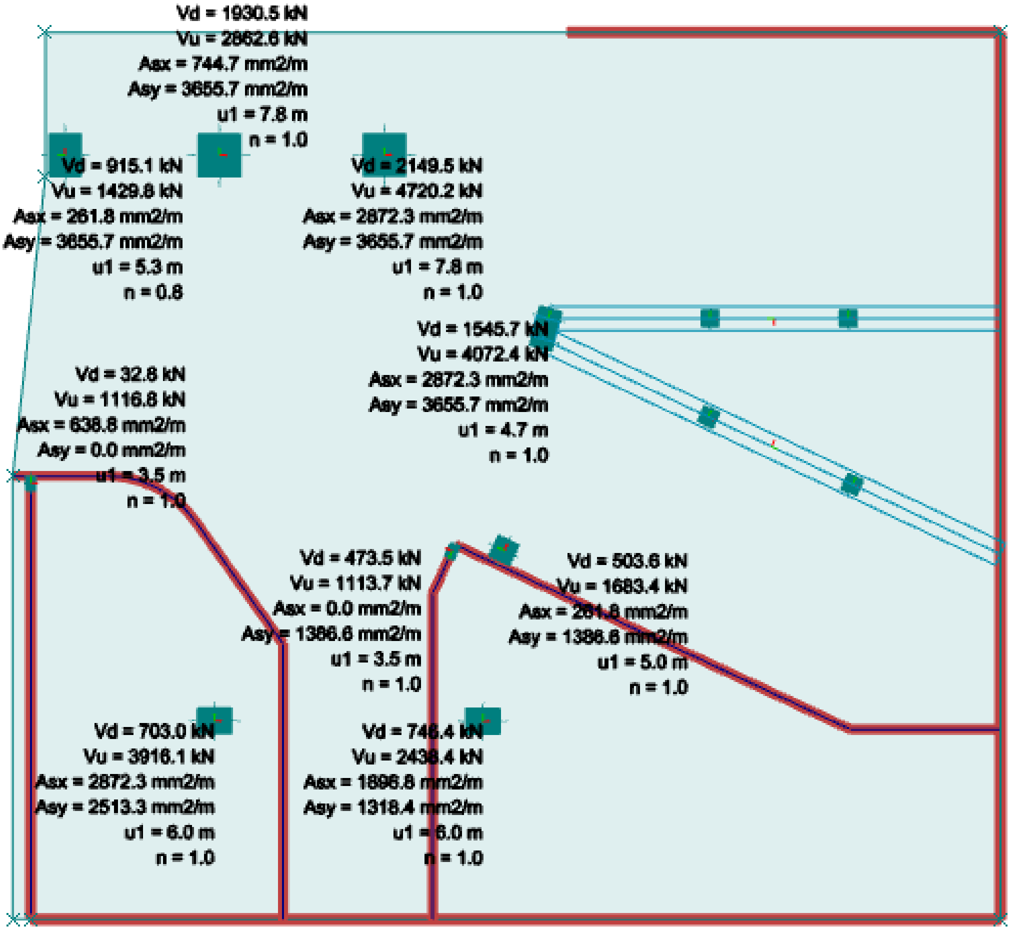


Figure B.6 Punching control and result of all columns and two wall corners (i.e. fiktive columns) in model "Punching, after reconstruction".