Faculty of Civil Engineering & Geosciences

Lifetime Enlargement of Steel Bridge Decks

COURSE:	CT 5050 – ADDITIONAL MSc THESIS
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1. PROBLEM DESCRIPTION

1.1 Introduction

In the Netherlands there are many orthotropic steel deck bridges and most of them were built between 1960 and 1980. The design philosophy was much different then and the knowledge concerning the phenomenon of fatigue was not as extensive as it is today. This is why in this kind of bridges fatigue cracks are very commonly observed.

1.2 Orthotropic bridge deck

This type of orthotropic bridge deck is used for fixed as well as for moveable bridges and the configuration depicted in figure 1.1 is very widespread and commonly used. This type of deck is composed of four types of structural components, which are:

- The deck plate
- The longitudinal stiffener, generally closed trough profiles
- The crossbeams, also called floor beams
- The main girder

Besides those structural components a surfacing layer is applied on top of the deck plate for both corrosion protection and skid resistance purposes.

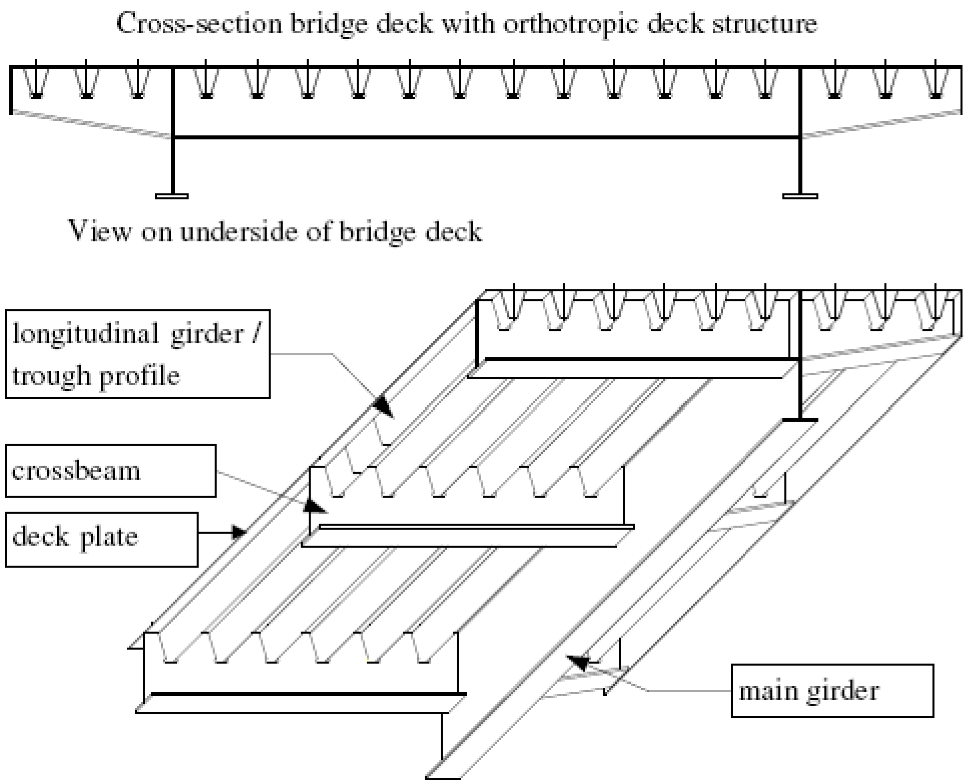

Figure 1.1 Orthotropic bridge structure

The role of the deck is to transfer the traffic loads (axle loads) directly to the longitudinal. A typical fixed bridge deck plate in the Netherlands has a thickness of 10 mm covered with a mastic asphaltic layer with thickness of 50 mm. On the other hand the thickness of a movable bridge deck will be 12 mm and will be covered with an epoxy layer.

In Netherlands the closed trough profile is generally used with a typical distance of 300 mm. The longitudinal troughs and the deck plate are welded together with a longitudinal weld and combined they act as a longitudinal beam system that transmits the loads to the cross beams. The deck also acts as an upper flange for the cross beams which they have a typical spacing of 3 – 5 m. So, the cross beams finally transfer the deck loads the main structure of the bridge.

1.3 Fatigue cracks in orthotropic bridge decks

During the last decades fatigue cracks have appeared in the deck of bridge structures mainly due to the limited knowledge of fatigue phenomena of steel structures at the time of design and also due to the increasing traffic load. Fatigue cracks are considered to be a threat to the structure and that is why repair and renovation techniques have been used to cope with that problem. There are various methods of constructions for orthotropic bridge decks and different type of details connections and therefore there different type of fatigue cracks. More precise they can be divided in to four categories, namely:

- Cracks in the deck plate
- Cracks in the longitudinal weld between deck plate and trough web
- Cracks in the trough splice joint
- Cracks in the connection between trough profile and crossbeam

The first two types of cracks can be observed from the cracks on the asphalt overlay, especially when the crack in the deck plate is severe and has a sufficient width. Also the first two types of cracks can endanger the safety of the bridge deck by reducing its bearing loading capacity. The mechanism of those type of cracks will be reviewed more thoroughly further on.

1.3.1 Fatigue cracks in the deck plate

In the part that the crossbeam crosses the trough girder there is a high concentration of stresses and as a result fatigue cracks are very common at that point. The troughs are continuous through the cross beam, and therefore the cross beams provide support only to the parts of the deck that is located in between the longitudinal troughs. On the other hand the part of the deck inside of the troughs is only supported by the webs of the trough that are welded on it. Therefore the problem arises because of the concentrated loads due to the wheels of heavy vehicles which cause a local deflection of the deck plate. Thus a hogging moment appears to the connection point of the web with the deck plate and high stress concentration arises creating fatigue phenomena.

Another location that these kind of cracks appear is in the area located in between of the cross beams. Nonetheless the mechanism that causes them is actually quite different. In this case the deck plate could be modelled as a multiple span continuous beam over elastic supports and therefore there is a high stress concentration in the area of the mid span in each one of these beams causing crack to appear.

Comparing the two aforementioned mechanisms it is easy to conclude that the stress concentration in the first one is higher than in the second one. Therefore cracks in the part that the crossbeam crosses the trough girder appear more frequently and in fact they are the most frequently observed fatigue cracks.

Both types of cracks can be seen in figure 1.2.

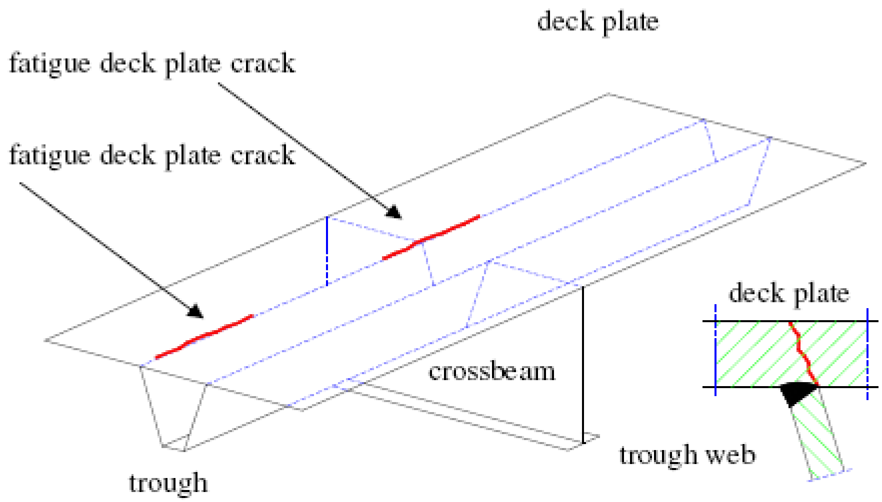

Figure 1.2 Fatigue cracks in the deck plate

Both cracks have similar propagation mechanisms. For the first type the mechanism consist of three stages:

- Stage 1: The crack initiation occurs at the root of the crossbeam and the continuous close trapezoidal stiffeners.
- Stage 2: The crack propagates in a vertical direction from the bottom fibres to the top of the deck.
- Stage 3: After the crack has grown through the deck plate, it then grows in the longitudinal – horizontal direction.

For the second type of crack the difference consists in the fact that this is growing simultaneously in the vertical and in the horizontal direction. In other words, stage 2 and 3 happen at the same time. Both cracks have a semi-elliptical layout. Also various inspections and NDT tests (Schat, 1997-a, 1997-b, 1997-c) have shown that the length of the crack in the bottom of the deck is four times the thickness of the deck larger than the length of the crack in the top of the deck plate. The three stages described, as well as the length of the crack, can be seen in figure 1.3.

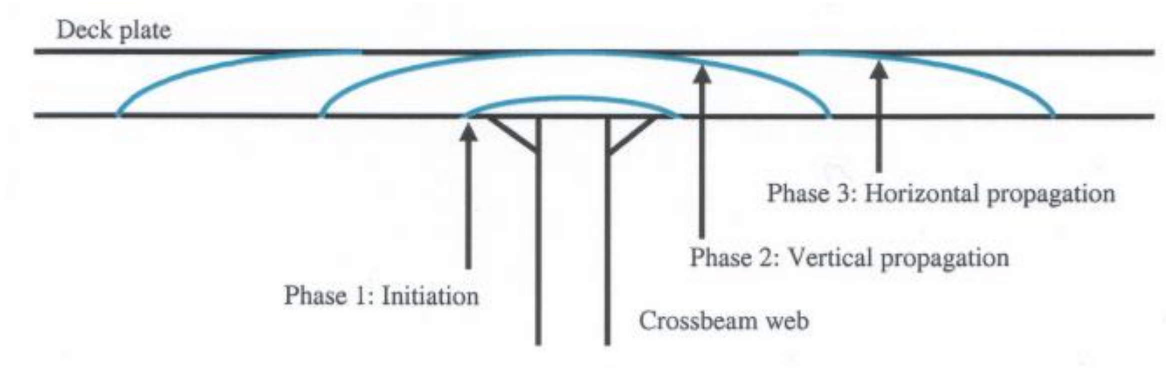

Figure 1.3 Crack propagation stages

Visual examples

Finally some visual examples of these types of cracks are presented in figures 1.4 and 1.5 and will be commented further on.

Figure 1.4 Cracks in the asphalt layer, possible indicating deck plate cracks

The crack depicted in figure 1.4 is a spider's web crack. This type of crack in combination with a relatively good condition of the rest of the asphalt layer could be an indication of fatigue crack of the deck plate. The only way to make sure is the removal of the existing asphaltic surface at the location of the cracks for a closer inspection of the deck.

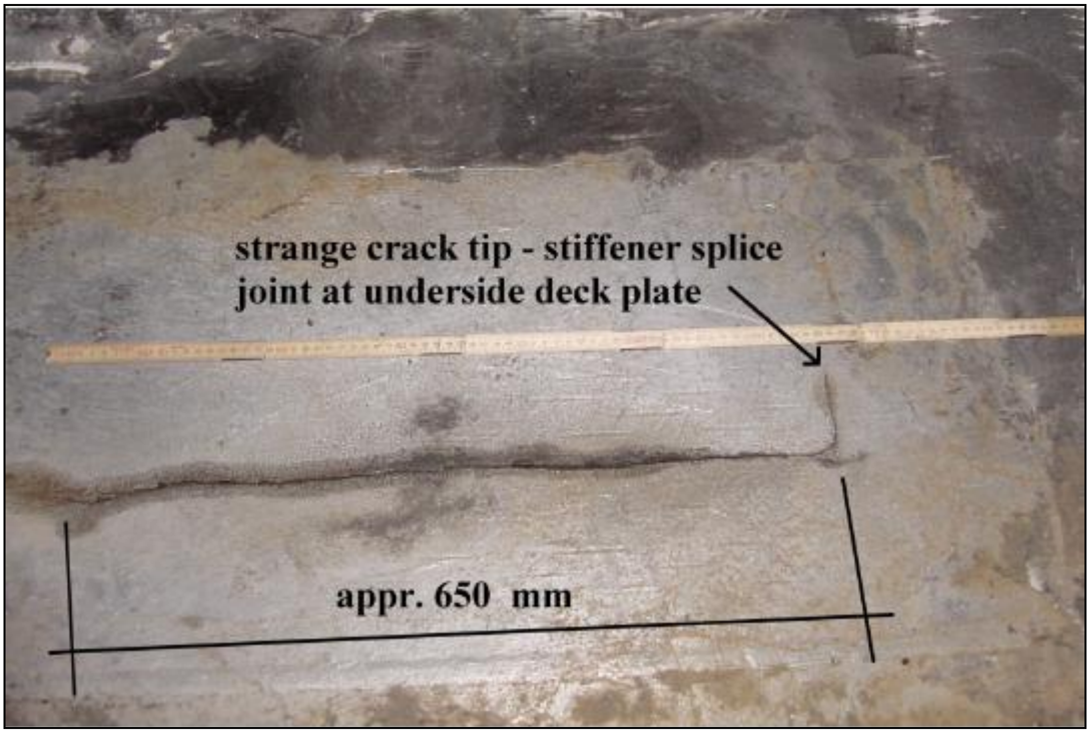

Figure 1.5 Deck crack after the removal of the asphalt layer deck

In figure 1.5 a crack in the deck plate can be seen which has been spotted after the removal of the asphalt layer in the location of a spider's web crack on the web of the Hagestian Bridge on the motorway A27 in the Netherlands. The length of the crack, as it can also been seen in the figure is of 650 mm length.

1.3.2 Fatigue cracks in the longitudinal weld between deck plate and trough web

This type of crack initiates at the root of the longitudinal weld between the trough web and the deck plate. The deck plate acts as beam with multiple supports, which are the troughs. The wheel loads applied on the part of the deck plate in between the trough webs cause a deflection of the deck plate and of the trough webs since they are welded together. That deflection causes bending moment to both the deck and the trough webs and results in high stress concentration in the longitudinal weld. The crack may initiate at any point in the longitudinal direction, except at the intersection of the crossbeam and the continuous closed trapezoidal stiffeners. The explanation for that is due to the limited bending stiffness of the trough profiles, the trough webs act as elastic supports depending on the distance from the cross beam and the point of the intersection has the minimum deformation and thus the minimum bending moment.

This type of cracks can be encountered in moveable and fixed bridges alike. The crack propagates through the weld from the inner to the outer surface of the trough web. After the crack has propagated through the web it then grows longitudinally parallel to the axis of the deck. The crack propagation rate depends on the quality and the size of the weld. Also fatigue tests have shown that the pre-weld gap between the trough web and the deck plate also affects in a negative way the fatigue behaviour. A full penetration weld shows a better fatigue behaviour that a fillet weld of 3 mm.

Visual examples

The longitudinal fatigue crack that is depicted in figure 1.6 has appeared in the deck of the Moerdijk Bridge in Netherlands.

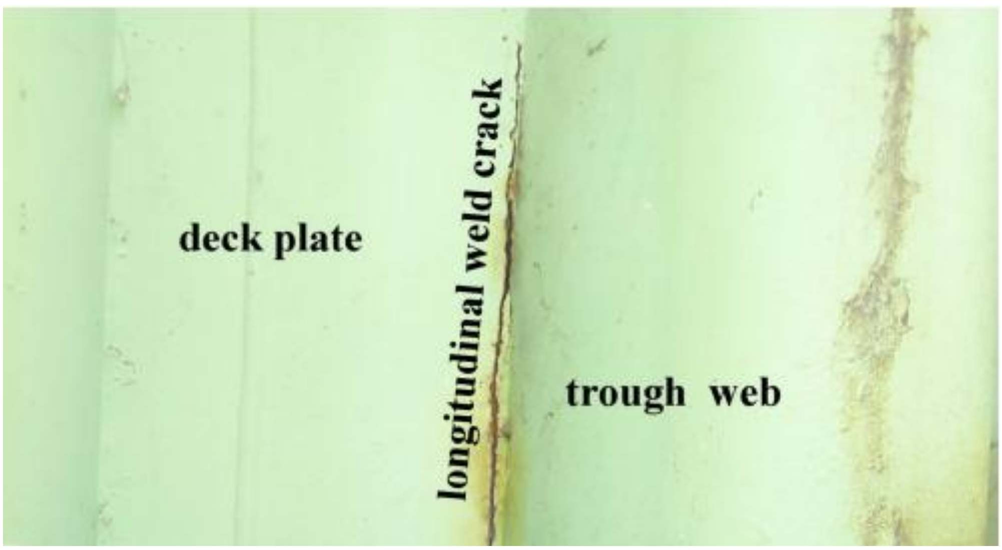

Figure 1.6 Fatigue crack in the longitudinal weld between the deck plate and the trough web

2. EXISTING RENOVATION TECHNIQUES

2.1 Introduction

The method of renovation that is going to be examined in this document is strengthening of the steel deck via a layer of FRP with glass fibres. Therefore it would be very useful to review the existing renovation techniques before examining whether or not the proposed technique is feasible.

2.2 Fatigue crack repair prior to the application of the renovation method

In some occasions it is necessary to repair the cracks in the deck plate before applying the renovation technique. However this is not always the case. This is why that there are two requirements that must be fulfilled if a crack is to remain:

- The fatigue crack is sufficiently small so as the required lifetime is achieved without repair.
- The static load ultimate capacity of the deck plate is sufficient despite of the crack.

However, even if those two requirements are fulfilled the general practice is two repair the cracks before the application of the renovation technique.

2.3 General requirements of renovation techniques

Due to the fact that every bridge is actually has different requirements not all the renovation techniques are suitable for every bridge. For instance there might be differences in the importance of the bridge, the expected service life, the traffic loads etc. Also, usually the more economically favorable solution is to be chosen. But that is also not an easy task since there are many factors that influence the cost of each renovation solution. The most important of those being:

- Service life extension and durability
- Mechanical and technical properties

  For moveable bridge decks, that the addition of extra weight must be matched by extra weight at the counter weight, the hinges must have the capacity to bear this extra weight.
- Execution time
- Traffic disruption

After a thorough investigation of all the factors it might be even the case that the repair and renovation cost will exceed that of constructing a new bridge. In the case that the most economical solution is to renovate the bridge then there are some

2.3 Fixed bridges

In this part the renovation techniques for fixed bridges will be presented as well as their advantages and disadvantages. For this purpose it is assumed that the deck is covered by a mastic asphaltic layer with a thickness of 50 mm. Also in all the figures that will be presented the vertical scale of the deck plate and the elements above it are different than the one of the troughs. The vertical scale of the trough is smaller.

2.3.1 Renovation using high performance concrete (HPC)

This renovation method is suitable for the most sever cracks in the heavy vehicle lane because it is applicable at the cross beams and in the field between the cross beams as well.

The procedure is as follows:

- Removal of the mastic asphalt layer, including the interface layer between the steel plate and the asphalt.
- In the case that cracks have already been formed a decision must be made, based on their severity, of repairing them or not.
- Application of a bonding layer to provide shear connection between the steel deck plate and the concrete layer creating this way composite action
- Pouring of the concrete (HPC) layer.

  The concrete layer should have an approximate thickness of 50 mm and should be reinforced according to the load requirements.
- Application of an epoxy layer on the concrete surface to provide with the required roughness needed for skid resistance. An alternative is shot blasting of the concrete surface.

The configuration of the deck after the renovation is depicted in figure 2.1.

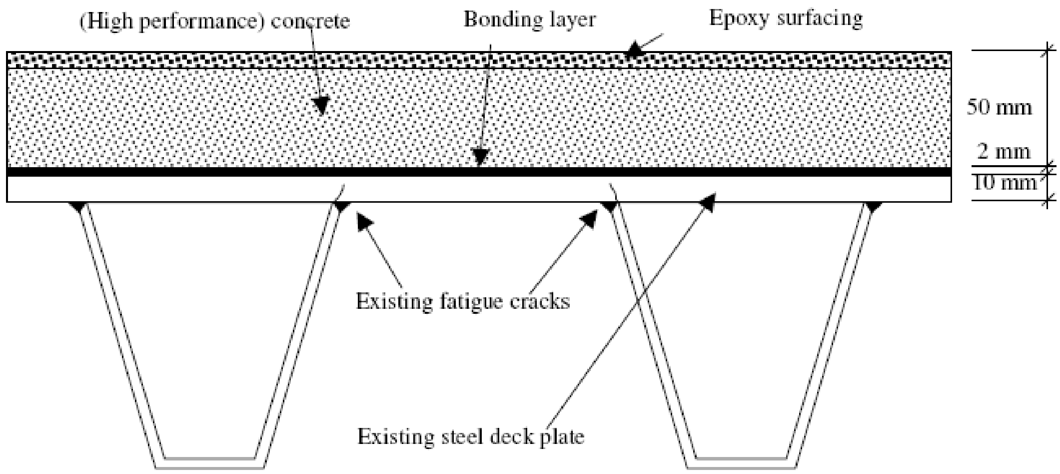

Figure 2.1 Renovation using high performance concrete

Advantages

- ✓ Significant stiffness increase of the deck → Large stress reduction
- ✓ Tolerances and dimensions are not critical
- ✓ Durable solution

Disadvantages

- × Concrete cracking
- × Lack of experience with bonding layers

2.3.2 Renovation using formed steel plate with bolted connection

This renovation method is applicable at the crossbeams as well as in the field between the cross beams and is suitable for the most severe cracks occurring in the heavy vehicle lane. In this method a steel plate is used which is connected to the deck via bolts. However it is formed in such way that there is neither connection nor contact of the steel plate and the deck in the areas of the deck that are located exactly above the troughs as can be seen in figure 2.2

The procedure is as follows:

- Removal of the mastic asphalt layer, including the interface layer between the steel plate and the asphalt.
- In the case that cracks have already been formed a decision must be made, based on their severity, of repairing them or not.
- Application of the steel plate by bolting on the deck plate
- Application of an epoxy layer on the surface of the new steel plate. The purpose of this layer is threefold. It provides the required roughness for skid resistance, corrosion

protection of the steel plate and especially of the steel bolts and also levels the height differences the bolted steel plate.

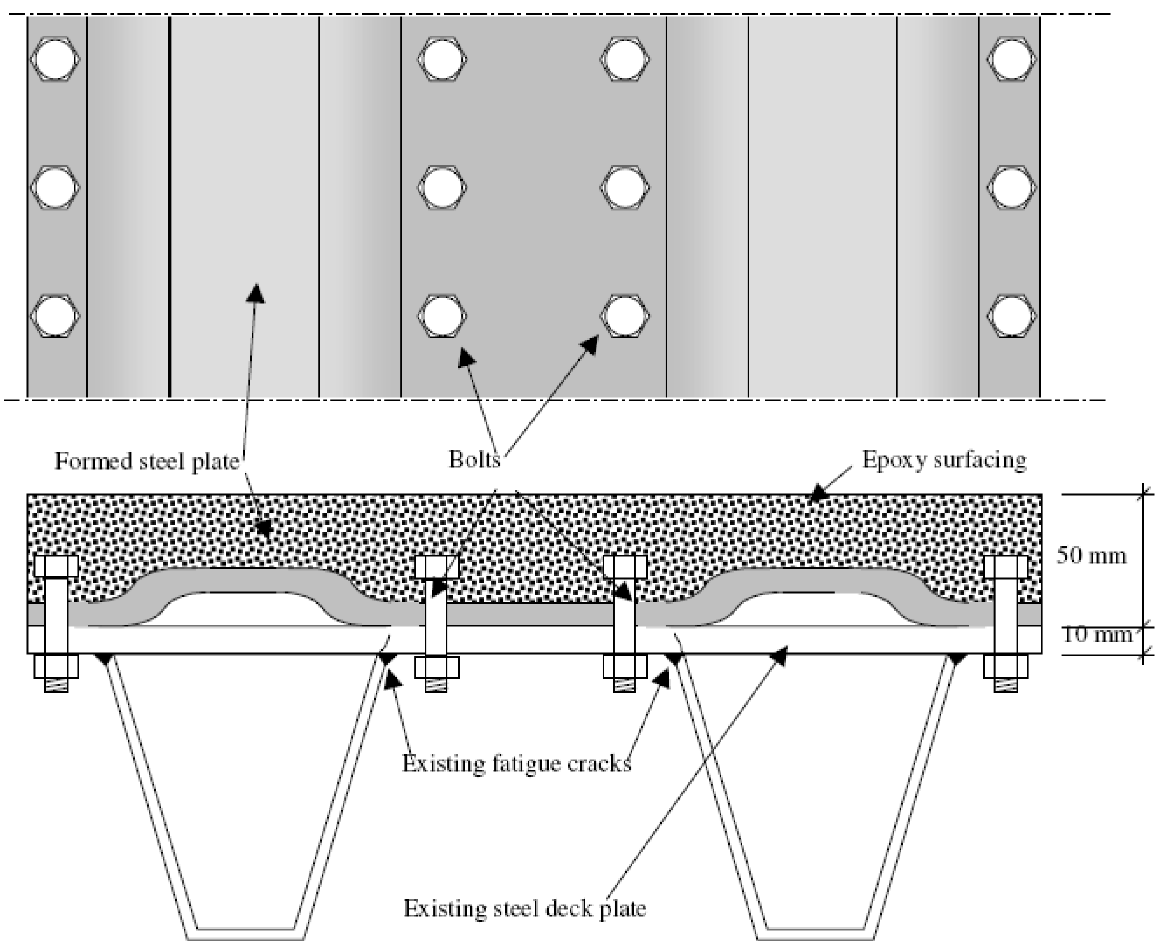

Figure 2.2 Renovation using formed steel plate with bolted connection

Advantages

- ✓ Stiffness increase of the deck → Stress reduction in the deck
- ✓ Applicable at local fatigue sensitive areas

Disadvantages

- × Bolted connections are sensitive to fatigue
- × Intermediate layer is sensitive to corrosion
- × Corrosion protection must be provided for the intermediate layer
- × Difficult to cope with the tolerances

2.3.3 Renovation using formed steel plate with bonded connection

This renovation method is very similar to the one described in 2.3.2 and the only difference is that the steel plate is not bolted to the deck plate but instead they are connected by the use of a bonding agent. The configuration is depicted in figure 2.3.

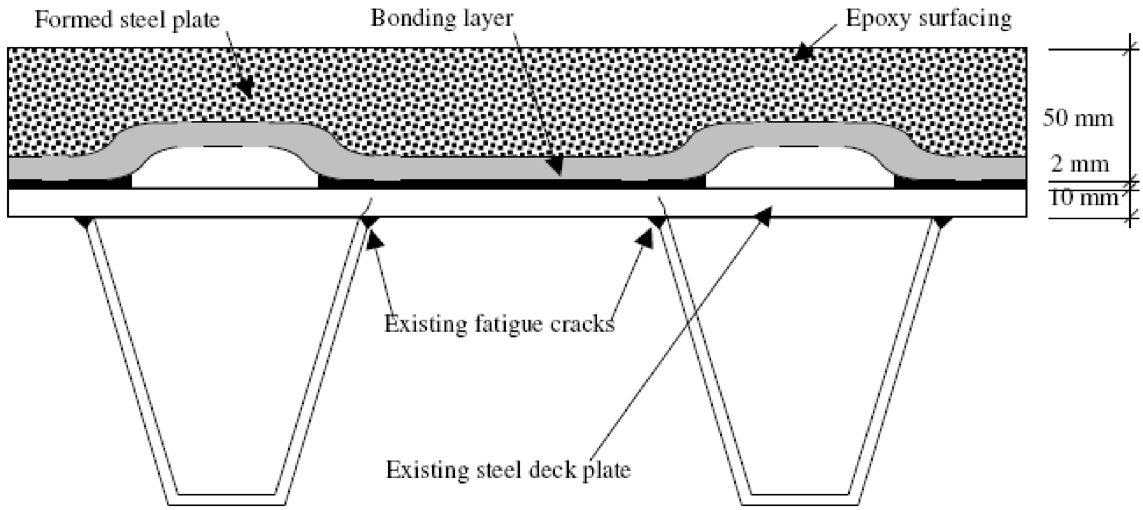

Figure 2.3 Renovation using formed steel plate with bonded connection

Advantages

- ✓ Stiffness increase of the deck → Stress reduction in the deck
- ✓ Applicable at local fatigue sensitive areas

Disadvantages

- × Lack of experience using a bonded connection
- × Critical installation procedure as the pot life of the adhesive is limited
- × Discontinuous bonding layer
- × Difficult to cope with the tolerances

2.3.4 Renovation using steel plate connected with rubber vulcanization

This is renovation technique is applicable in both the crossbeams and in the field located between the crossbeams. However, if chosen as a renovation method it must be applied on both, since it is not a method that can be used for local renovation. It can be used for the repair of the most severe crack caused by the vehicles in the heavy traffic lane.

The application procedure is as follows:

- Removal of the mastic asphalt layer, including the interface layer between the steel plate and the asphalt.

- In the case that cracks have already been formed a decision must be made, based on their severity, of repairing them or not.
- Placement of the new steel plate.
- Vulcanization of a rubber layer between the steel plate and the deck plate to achieve connection.
- Application of an epoxy layer on the top surface of the steel plate.
  The configuration of the deck after the renovation is depicted in figure 2.4.

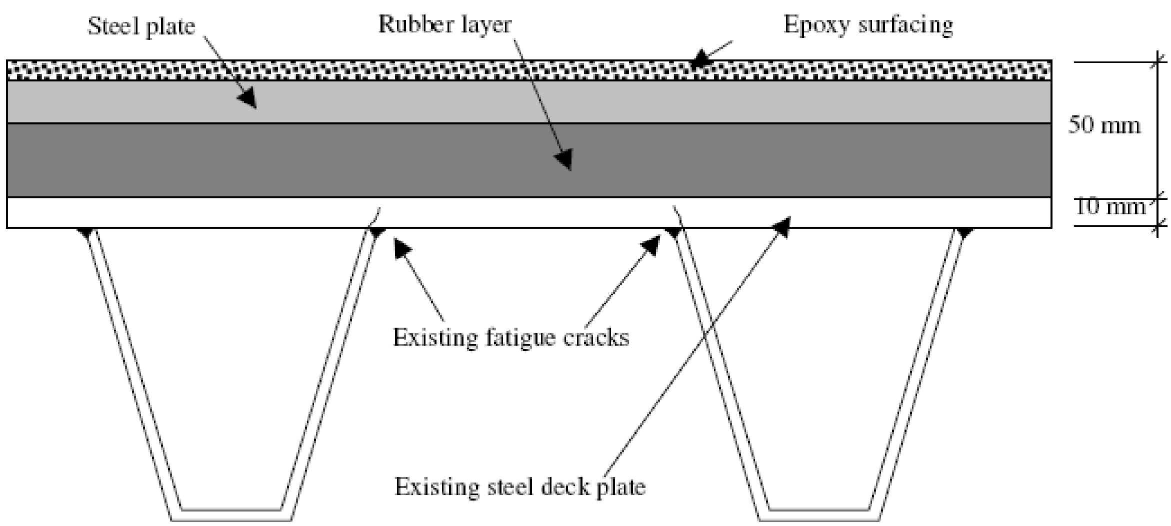

Figure 2.4 Renovation using steel plate connected with rubber vulcanization

Advantages

- ✓ Tolerances and dimensions are not critical

Disadvantages

- × No composite action due to the low stiffness of the rubber matrix
- × Low stress reduction
- × Difficult installation process as the rubber layer is applied after the new steel plate

2.3.5 Renovation using steel plate connected with polyurethane foam

This method is similar to the one presented in 2.3.4 except for the fact that the interface material that will be used is polyurethane instead of rubber. Polyurethane has a larger stiffness comparing to rubber and this is why a composite behaviour can be achieved.

The application procedure is exactly the same except for the fact that the polyurethane is in the form of foam. The final result is depicted in figure 2.5.

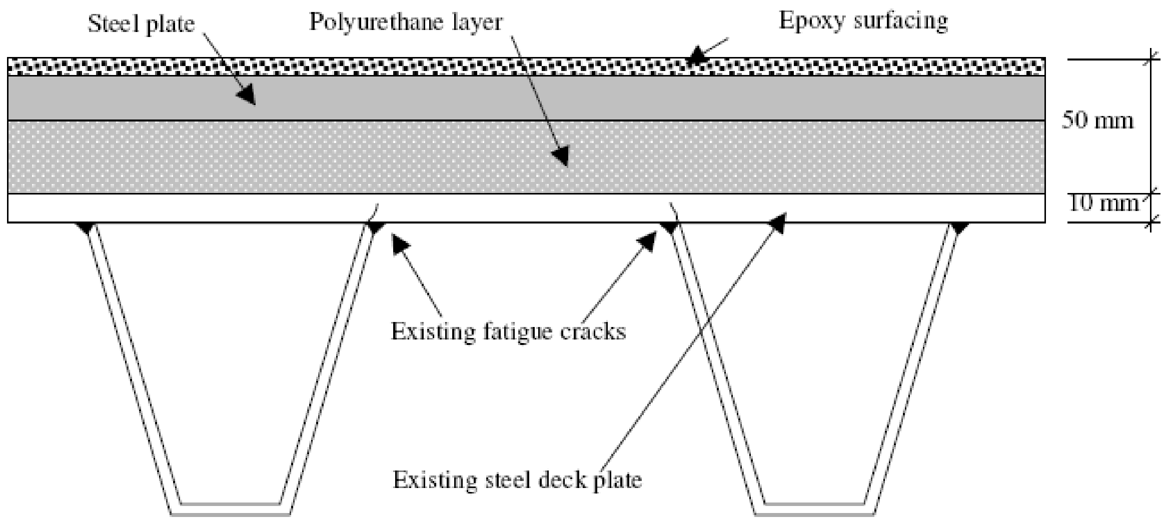

Figure 2.5 Renovation using steel plate connected with polyurethane foam

Advantages

- ✓ Tolerances and dimensions are less critical
- ✓ Complete composite action that increases the stiffness of the deck

Disadvantages

- × Limited fatigue resistance and durability of the polyurethane layer
- × Difficult installation process as the polyurethane layer is applied after the placing of the new steel plate

2.3.6 Renovation using prefabricated sandwich panel

For this renovation method a prefabricated panel will be used. The panel is bonded to the deck plate and via a bonding layer which has a purpose ensuring complete composite action.

The application procedure is as follows:

- Removal of the mastic asphalt layer, including the interface layer between the steel plate and the asphalt.
- In the case that cracks have already been formed a decision must be made, based on their severity, of repairing them or not.
- Application of the adhesive material on the surface of the deck plate.
- Application of the sandwich panel.
- Application of an epoxy layer on the top surface of the sandwich panel.

The configuration of the deck after the renovation is depicted in figure 2.6.

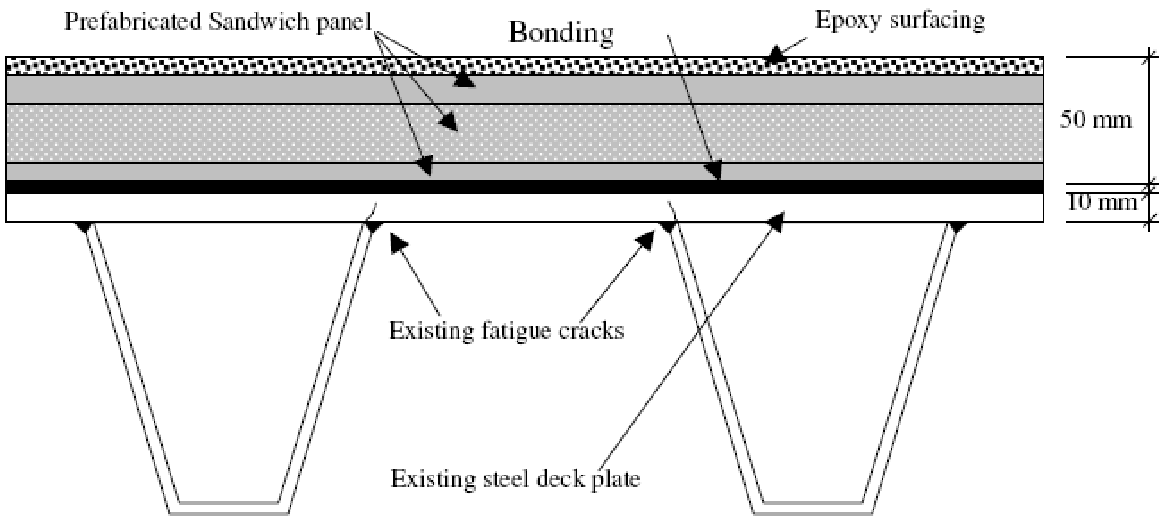

Figure 2.6 Renovation using prefabricated sandwich panel

Advantages

- ✓ Prefabrication of the sandwich panel
- ✓ Complete composite behaviour reduces the stresses in the deck
- ✓ Lighter that the steel alternative

Disadvantages

- ✗ Limited fatigue resistance and durability of the polyurethane layer
- ✗ Lack of experience concerning the use of a bonding layer
- ✗ Prefabricated panels can't be applied if the surface of the deck is uneven
- ✗ Critical installation procedure as the pot life of the adhesive is limited

2.3.7 Renovation using aluminium planks with bolted connection

This is a renovation method that is applicable for both the cross beams and the field located between the cross beams. Therefore it is suitable even for most severe cracks in the heavy vehicle lane. Also it is possible to use this renovation method only locally.

The aluminium planks used usually have a thickness of a approximately 50 mm. However the thickness is reduced for the area that is located above the troughs as it is depicted in figure 2.7. The application technique is the same as the one described for the steel plates in 2.3.2 and the only difference is that instead of steel plates, aluminium planks are used.

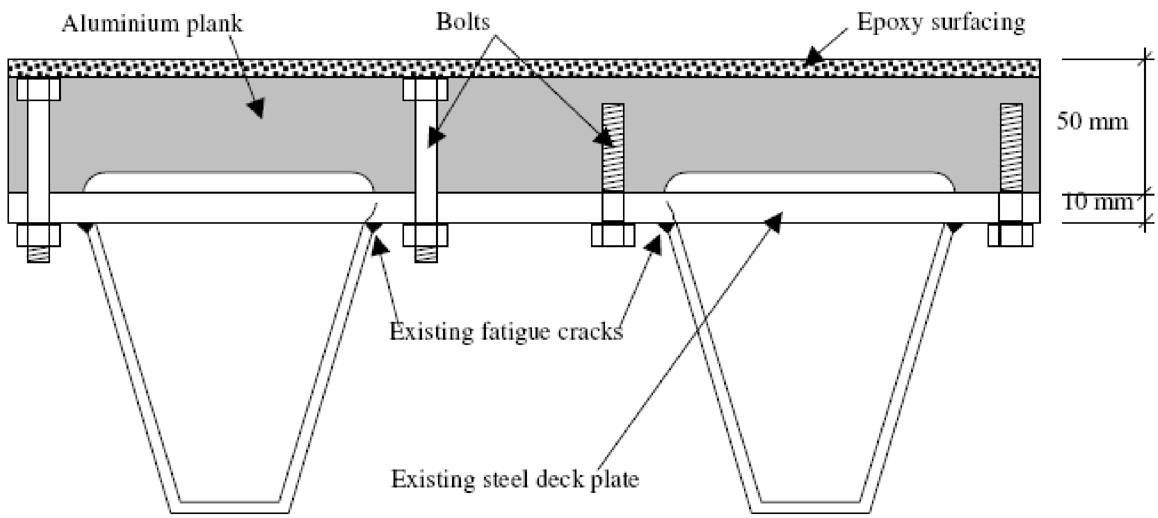

Figure 2.7 Renovation using aluminium planks with bolted connection

Advantages

- ✓ Can applied locally without having to renovate the hole deck
- ✓ Stress reduction to the old structure due to composite behaviour
- ✓ Lighter than the steel alternative

Disadvantages

- × Difference in the thermal coefficient factor between the steel deck and the aluminium plank causes stresses
- × Danger of corrosion due to different electric potential → necessity for corrosion protection of the steel deck
- × Bolted connections sensitive to fatigue

2.3.8 Renovation using aluminium planks with bonded connection

This type of renovation is the same as the one described in 2.3.3, except for the fact that instead of steel plates aluminium planks are used. The configuration of the deck after the application of the renovation technique is depicted in figure 2.8.

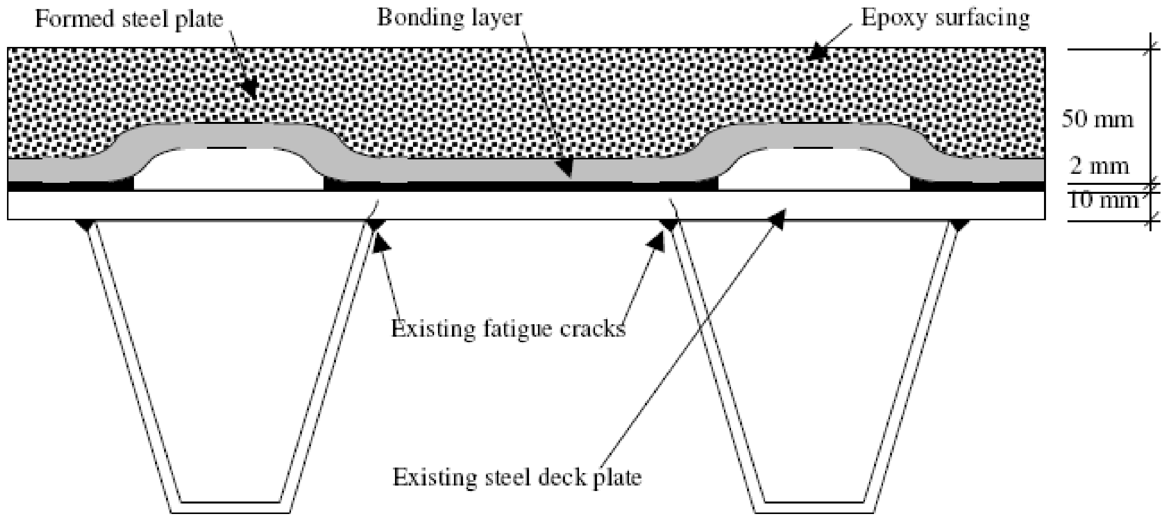

Figure 2.8 Renovation using aluminium planks with bonded connection

Advantages

- ✓ Can applied locally without having to renovate the hole deck
- ✓ Stress reduction to the old structure due to composite behaviour
- ✓ Lighter than the steel alternative

Disadvantages

- ✗ Difference in the thermal coefficient factor between the steel deck and the aluminium plank causes stresses
- ✗ Danger of corrosion due to different electric potential → necessity for corrosion protection of the steel deck
- ✗ Lack of experience with bonding connection
- ✗ Critical installation procedure as the pot life of the adhesive is limited
- ✗ Discontinuous bonding layer

2.3.9 Renovation using extruded aluminium panels with bonded connection

This renovation technique is similar to the one described in 2.3.3 except for the fact that in instead of a solid steel plates a type of sandwich panel is used as the one depicted in figure 2.8. The sandwich panel can either be applied as in shown in figure 2.8 or rotated 90^o in the horizontal direction.

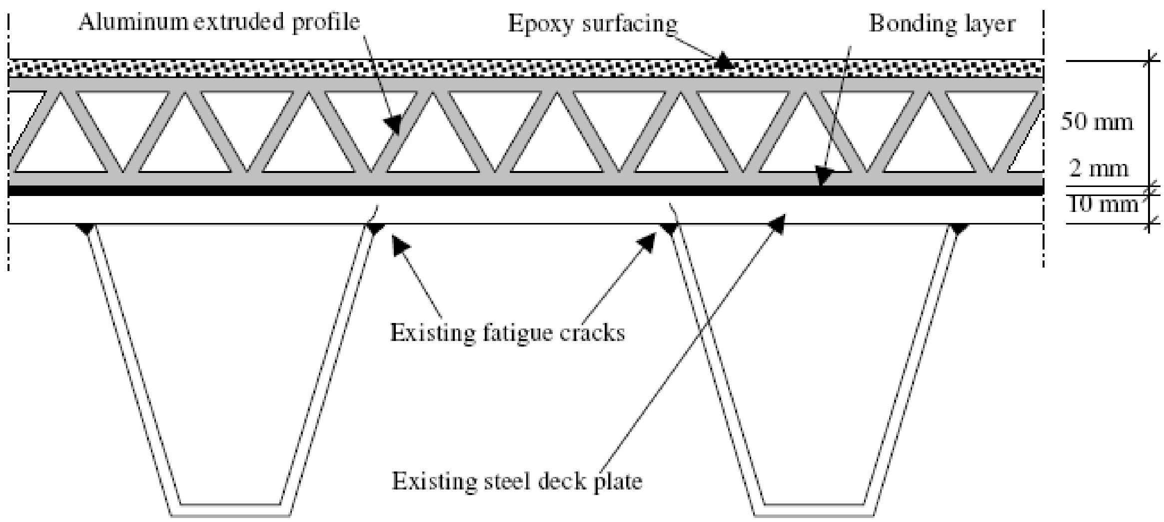

Figure 2.9 Renovation using extruded aluminium panels with bonded connection

Advantages

- ✓ Can applied locally without having to renovate the hole deck
- ✓ Stress reduction to the old structure due to composite behaviour
- ✓ Lighter than the steel and the two previous aluminium alternatives

Disadvantages

- × Difference in the thermal coefficient factor between the steel deck and the aluminium plank causes stresses
- × Danger of corrosion due to different electric potential → necessity for corrosion protection of the steel deck
- × Lack of experience with bonding connection
- × Critical installation procedure as the pot life of the adhesive is limited
- × Discontinuous bonding layer
- × If surface of the deck is uneven then the method is difficult to apply

2.3.10 Renovation using timber planks with bolted connection

This is a renovation method that is applicable for both the cross beams and the field located between the cross beams. Therefore it is suitable even for most severe cracks in the heavy vehicle lane. Also it is possible to use this renovation method only locally.

The application procedure is similar to the one described in 2.3.2 except that the material used is timber in form of planks. The timber planks have usually a thickness of 50 mm. The disadvantage of bolting the timber planks to the deck is that composite action cannot be achieved.

The configuration of the deck after the application method is depicted in figure 2.10

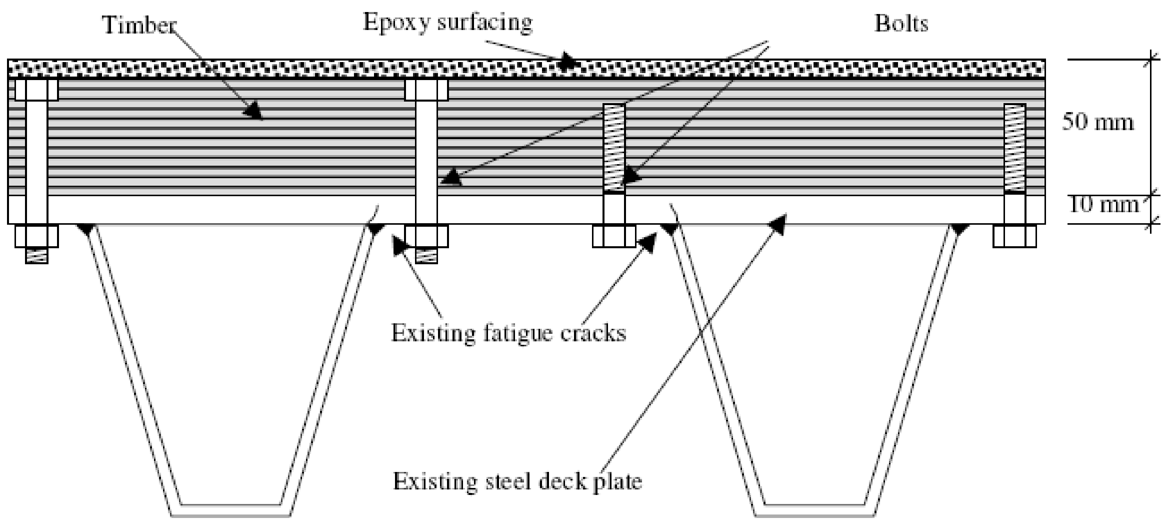

Figure 2.10 Renovation using timber planks with bolting connection

Advantages

- ✓ Can applied locally without having to renovate the hole deck
- ✓ Lighter than the steel and the aluminium alternative

Disadvantages

- ✗ Discontinuities between planks and asphalt
- ✗ Timber has poor material properties compared to other alternatives
- ✗ Bolted connection is sensitive to fatigue
- ✗ Wood planks in combination with the bolted connections require high maintenance
- ✗ No composite action and therefore very small stress reduction
- ✗ Corrosion protection of the steel deck is necessary

2.3.11 Renovation using timber planks with bonded connection

This is a renovation method that is applicable for both the cross beams and the field located between the cross beams. Therefore it is suitable even for most severe cracks in the heavy vehicle lane. Also it is possible to use this renovation method only locally.

The application procedure is similar to the one described in 2.3.3 except that the material used is timber in form of planks. The timber planks have usually a thickness of 50 mm as depicted in figure 2.11.

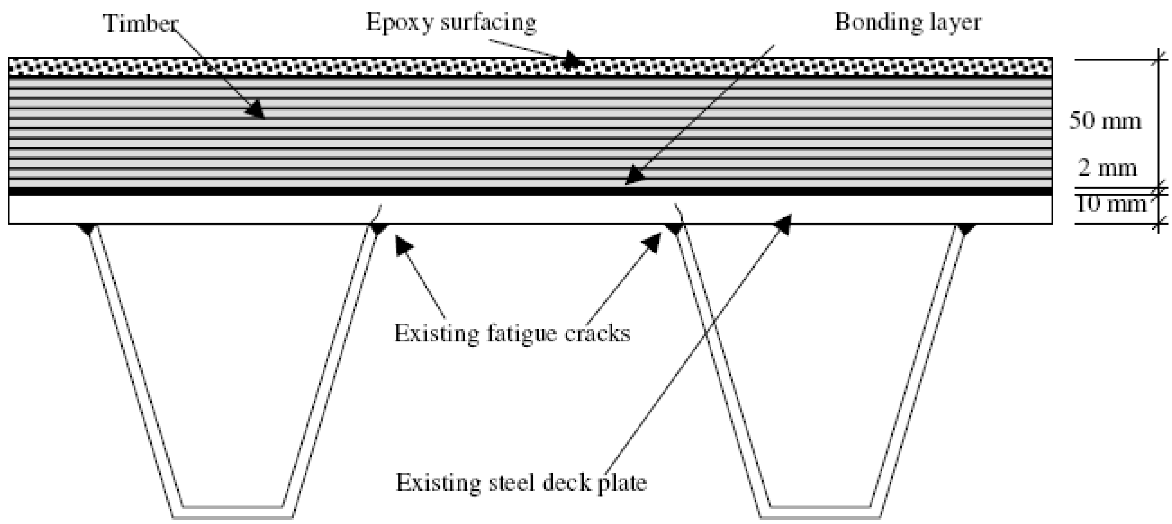

Figure 2.11 Renovation using timber planks with bonded connection

Advantages

- ✓ Can applied locally without having to renovate the hole deck
- ✓ Lighter than the steel and the aluminium alternative
- ✓ Complete composite behaviour limits significantly the stresses in the deck plate

Disadvantages

- × Discontinuities between planks and asphalt
- × Timber has poor material properties compared to other alternatives
- × Lack of experience with bonding connections
- × Critical installation procedure as the pot life of the adhesive is limited

2.4 Movable bridges

In this part the renovation techniques for moveable bridges will be presented as well as their advantages and disadvantages. For this purpose it is assumed that the deck is covered by an epoxy layer with a thickness of 7 mm. Also in all the figures that will be presented the vertical scale of the deck plate and the elements above it are different than the one of the troughs. The vertical scale of the trough is smaller.

2.4.1 Renovation using steel plate with bonded connection

This is a renovation method ideal to be used to decks that are only covered with a thin epoxy layer which is usually the case for moveable bridges. The steel plate is connected to the deck via an adhesive material which provides the shear connection needed to achieve composite behaviour. The application procedure is as follows:

- Removal of the epoxy layer.
- Application of the bonding agent.
- Placing of the steel plate.
- Application of an epoxy layer on the surface of the new steel plate. The purpose of this layer is threefold. It provides the required roughness for skid resistance, corrosion protection of the steel plate and especially of the steel bolts and also levels the height differences the bolted steel plate.

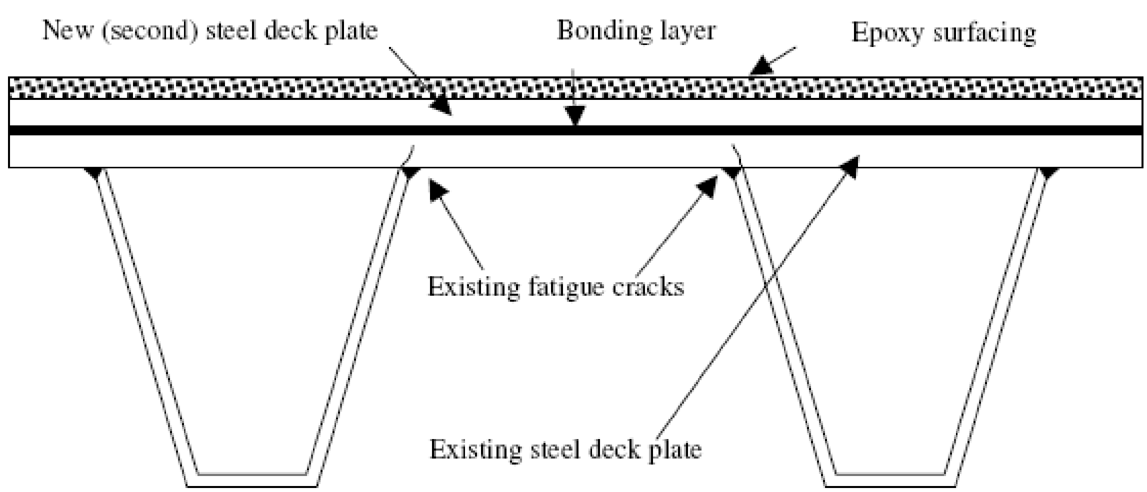

Figure 2.12 Renovation using steel plate with bonded connection

Advantages

- ✓ Small additional height and weight
- ✓ Complete composite behaviour limits significantly the stresses in the deck plate
- ✓ Tolerances and dimensions are not critical

Disadvantages

- × Lack of experience with bonding connections
- × Critical installation procedure as the pot life of the adhesive is limited

2.4.2 Renovation by trough filling

This renovation method is mostly used for moveable bridges but it is also possible to be used in fixed bridges as well. In contradiction to every other method that was reviewed so far, the epoxy layer, for fixed bridges, or the concrete layer, for moveable bridges, does not need to be removed. However, to be in the safe side, the upper surface is usually removed to detect and repair fatigue cracks. For this renovation technique the troughs are filled with a filling material that has to have high stiffness. Commonly polyurethane is used. It is required for a successful renovation for the filling material to be in contact with the bottom side of the deck plate. The deck configuration after the application method is depicted in 2.13.

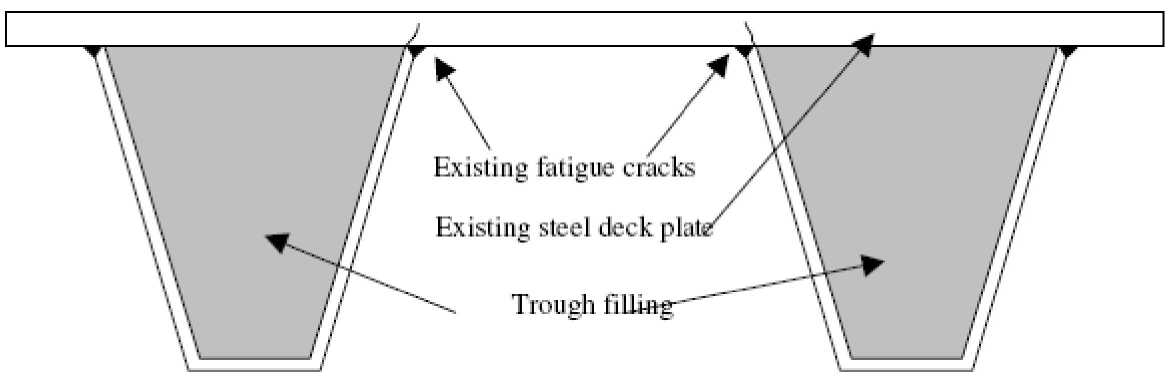

Figure 2.13 Renovation by trough filling

3. FRP COMPOSITES

3.1 Introduction

FRP is an anisotropic non linear material and very difficult to model. Also the type of FRP depends on the type of fibres that will be used as well as the type of epoxy. Therefore in this chapter, a more extensive presentation concerning FRP will take place.

3.2 Materials

The main components that are used for the production of FRP composites are the fibres and the matrix. Nonetheless, when FRP are used for the strengthening of a structural system, a very important role is played by the adhesives used. Further on those three main components will be presented in detail.

3.2.1 Fibres

Fibres have a diameter in the order of 5-25 μm and constitute the primary load carrying elements (parallel to their axis) in a composite material system. Main properties of the fibers are the high tensile strength and the linear elastic behavior to failure can be seen in figure 3.1.

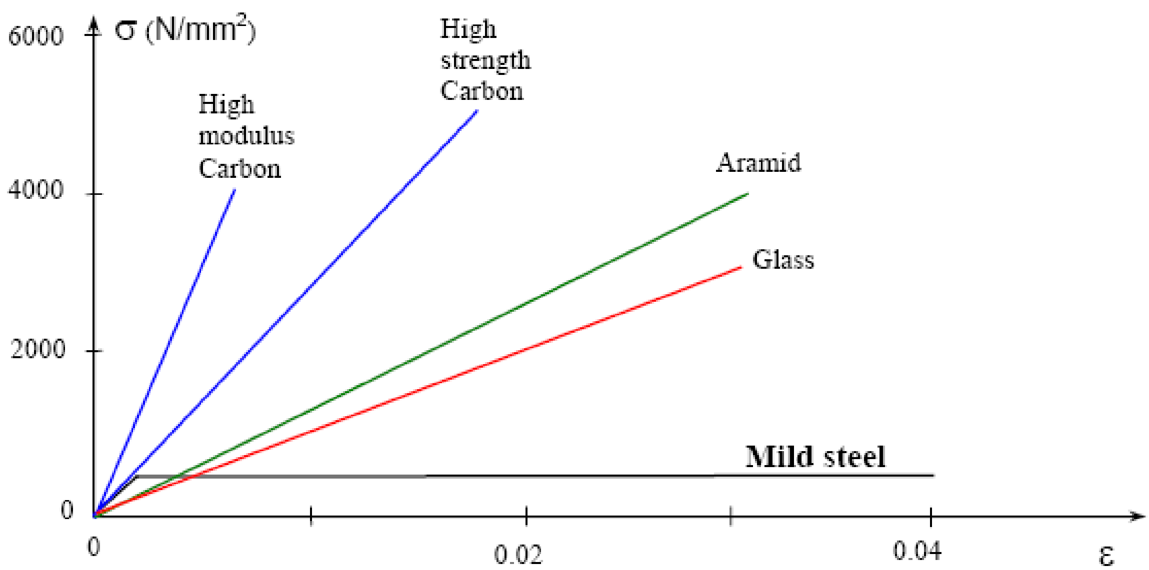

Figure 3.1 Typical unixial tension stress-strain diagrams for different fibres and comparison with steel.

There are three main types of fibres commonly used. Those are carbon, glass and aramid.

Carbon fibres

Carbon fibres are used in structural engineering applications today in FRP strengthening sheets and fabrics, in FRP strengthening strips and in FRP prestressing tendons. Carbon fibre is a solid semi-crystalline organic material consisting on the atomic level of planar two-dimensional arrays of carbon atoms. The two-dimensional sheet-like array is usually known as the graphitic form. Hence, the fibres are also known as graphite fibres. Carbon fibre is produced in grades known as standard modulus, intermediate modulus, high strength and ultrahigh modulus.

Carbon fibres have diameters from about 5 to 10 μm. Carbon fibre has a characteristic charcoal black color. Due to their two-dimensional atomic structure, carbon fibres are considered to be transversely isotropic, having different properties in the longitudinal direction of the atomic array than in the transverse direction. The longitudinal axis of the fibre is parallel to the graphitic planes and gives the fibre its high longitudinal modulus and strength. Approximate properties of common grades of carbon givers are given in table 3.1

Table 3.1 Approximate properties of common grades of carbon fibres

Grade of carbon fibre	Density (g/cm^3)	Tensile modulus (GPa)	Tensile strength (MPa)	Max elongation (%)
Standard	1.7	250	3700	1.2
High strength	1.8	250	4800	1.4
High modulus	1.9	500	3000	0.5
Ultrahigh modulus	2.1	800	2400	0.2

Carbon fibre is produced at high temperatures, 1200^{o}C to 2400^{o}C, from three possible precursor materials: a natural cellulosic rayon textile fibre, a synthetic polyacrilonitrile (PAN) textile fibre or pitch (coal tar). Pitch-based fibres, produced as a by-product of petroleum processing, are generally lower cost than PAN- and rayon-based fibres. As the temperature of the heat treatment increases during production of the carbon fibre, the atomic structure develops more of the sheet-like planar graphitic array, giving the fibre higher longitudinal modulus.

In earlier years carbon fibres have been used primarily with epoxy resins, and suitable sizings for epoxy resin systems are readily available. Nowadays, carbon fibres are being used with vinylester and blended vinylester-polyester resins for FRP profiles and FRP strengthening strips. Sizing for carbon fibres for polyester and vinylester resins are not as common. Care must be taken when specifying a carbon fibre for use with non-epoxy resin system to ensure that the fibre is properly sized for the resin system used.

Glass fibres

Glass fibres are used in a multitude of FRP products for structural engineering, from FRP reinforcing bars for concrete, to FRP strengthening fabrics, to FRP structural profile shapes. Glass is an amorphous inorganic compound of primarily metallic oxides that is produced in fibrous form in a number of standard formulations, constituting from 50 to 70% by weight of the glass. Different grades of glass fibre are identified by letter nomenclature. A borosilicate glass known as E-glass (electrical glass) because of its high electrical resistivity is used to produce the vast majority of glass fibre used in FRP products for structural engineering. A-glass (window glass) and C-glass (corrosion resistant, also know as AR-glass of alkali-resistant glass) are used to produce specialized products for use in structural engineering. S-glass (structural or high-strength glass) is used to produce the high-performance fibres used primarily in the aerospace industry.

The diameter of an individual glass fibre of filament ranges from approximately 3 to 24 μm. For structural engineering the most commonly used fibre diameter is 17 μm. Glass fibres have a distinctive bright white color. Glass is usually considered to be an isotropic material. Approximate properties of commonly used grades of glass fibres are given in table 3.2. Those values are designed as a guide and should not be used in design calculations.

Table 3.2 Approximate properties of common grades of glass fibres

Grade of glass fibre	Density (g/cm^3)	Tensile modulus (GPa)	Tensile strength (MPa)	Max elongation (%)
E	2.57	72.5	3400	2.5
A	2.46	73	2760	2.5
C	2.46	74	2350	2.5
S	2.47	88	4600	3.0

Glass fibres are produced at melt temperatures of about 1400^{o}C. Individual filaments are produced with a surface coating called a sizing that serves to protect the filaments when they are formed in a bundle or a strand. The sizing also contains coupling agents, usually silanes, that are specially formulated to enhance bonding between the glass fibre and the particular polymer resin being used when making a glass-reinforced FRP composite material. Today, most commercially available glass fibres can be obtained with sizings that are compatible with the three major thermosetting resin systems used in structural engineering: epoxy, polyester and vinylester.

Aramid fibres

Aramid fibres were first developed and patented by DuPontin in 1965 under the name Kevlar and today are produced by several manufacturers under various brand names (Kevlar, Twaron, Technora). They consist of aromatic polyamide molecular chains. A combination of their relatively high price, difficulty in processing, high no moisture absorption, low melting temperatures and relatively poor compressive properties have made them less attractive for FRP parts for structural engineering applications. They have a distinctive yellow color. They are the lightest of the high performance fibres having a density of around 1.4 g/cm^3. Depending on the type of aramid fibre, the fibre longitudinal tensile strength ranges from 3400 to 4100 MPa and its longitudinal tensile modulus ranges from 70 to 125 GPa.

3.2.2 Matrix

The matrix for a structural composite material is the polymer ingredient in the non-fibrous part of the FRP material that binds the fibres together and is typically a polymer resin, of thermosetting type or of thermoplastic type, with the first being the most common one. Recent developments have resulted in matrices based on inorganic materials (e.g. cement-based). The function of the matrix is to protect the fibres against abrasion or environmental corrosion, to bind the fibres together and to distribute the load. The matrix has a strong influence on several mechanical properties of the composite, such as the transverse modulus and strength, the shear properties and the properties in compression. Physical and chemical characteristics of the matrix such as melting or curing temperature, viscosity and reactivity with fibres influence the choice of the fabrication process. Hence, proper selection of the matrix material for a composite system requires that all these factors be taken into account. Epoxy resins, polyester, vinylester and phenolics are the most common polymeric matrix materials used with high-performance reinforcing fibres. They are thermosetting polymers with good processibility and good chemical resistance. Epoxies have, in general, better mechanical properties than polyesters and vinylesters, and outstanding durability, whereas polyesters and vinylesters are cheaper. Phenolics have a better behavior at high temperatures.

Recently, polymer-modified cement-based mortars have also become available in some applications. It is expected that these mortars will be used more and more in the near future. Also, recently polyurethane resins have been introduced to the market. A short description of the polymer resins mentioned will follow.

Epoxy resins

Epoxy resins are used in many FRP products for structural engineering applications. Most carbon fibre reinforced procured FRO strips for structural strengthening are made with epoxy resins. In addition, epoxy resin adhesives are used to bond procured FRP strips to concrete (and other materials) in the FRP strengthening process. Epoxy resins are also used extensively in FRP strengthening applications, where the epoxy resin is applied to the dry fibre sheet or fabric in the field and then cured in situ, acting as both the matrix for the FRP composite and as the adhesive to attach the FRP composite to the substrate. When applied to dry fibre sheets or fabrics, the epoxy resins are often referred to saturants. Epoxy resins have also been used to manufacture FRP tendons for prestressing concrete and FRP stay cables for bridges. They are not used extensively to produce larger FRP profiles, due to their higher costs and the difficulty entailed in processing large pultruded FRP parts

Polyester resins

Polyester resin is widely used to make pultruded FRP profiles for use in structural engineering and is also use to make some FRP rebars. They can also be used for strengthening of structures.

Vinylester resins

Developed in the last twenty years, vinylester resins have become attractive polymer resins for FRP products for structural engineering especially due to their good properties, especially their corrosion resistance and their ease of processing. Nowadays vinylester resins are used to make the majority of FRP rebars sold in the world and are also used widely in FRP pultruded profiles. They are generally replacing polyester resins in FRP products in structural engineering, due to their superior environmental durability in alkaline environments.

Phenolic resins

Phenolic resins are the oldest and most widely used thermosetting resins. However, they have only recently been used for FRP products for structural engineering, due to the difficulty of reinforcing them and curing them by condensation polymerization. They have superior fire resistance and they char and release water when burned.

Polyurethane resins

Thermosetting polyurethane resins have recently been introduced into the market as structural resins. The reason for this is that only recently have they been produced in high-density forms that can be used in resin molding and pultrusion operations. They have high toughness and when used with glass fibres produce composites with high transverse tensile and impact strengths.

In the following table (3.3) an approximation of the properties for the thermosetting polymer resins can be found.

Table 3.3 Approximate properties of thermosetting polymer resins

Polymer resins	Density (g/cm^3)	Tensile modulus (GPa)	Tensile strength (MPa)	Max elongation (%)
Polyester	1.2	4.0	65	2.5
Epoxy	1.2	3.0	90	8.0
Vinylester	1.12	3.5	82	6.0
Phenolic	1.24	2.5	40	1.8
Polyurethane	varies	2.9	71	5.9

3.2.3 Composite materials

Advanced composites as strengthening materials consist of a large number of small, continuous, directionalized, non-metallic fibres with advanced characteristics, bundled in the matrix as depicted in figure 3.2. Depending on the type of fibre they are referred to as CFRP (carbon fibre based), GFRP (glass fibre based) or AFRP (aramid fibre based). When different types of fibres are used, the material is called “hybrid”. Typically, the volume fraction of fibres in advanced composites equals about 50-70% for strips and about 25-35% for sheets. Given also that the elastic modulus of fibres is much higher than that of the matrix, it becomes clear that the fibres are the principal stress bearing components, while the matrix transfers stresses among fibres and protects them.

Basic mechanical properties of composites may be estimated if the properties of the constituent materials (fibres, matrix) and their volume fractions are known. Details about the micromechanics of composite materials are not considered here. However, for the simple, yet quite common, case of unidirectional fibres, one may apply the “rule of mixtures” simplification as follows:

For the modulus of elasticity:

$$E_{FRP} \approx E_{fib} V_{fib} + E_m V_m$$

And for the tensile strength:

$$f_{FRP} \approx f_{fib} V_{fib} + f_m V_m$$

where:

E_{FRP} = modulus of elasticity of the fibre-reinforced material in fibre direction
E_{fib} = modulus of elasticity of the fibres
E_m = modulus of elasticity of the matrix
V_{fib} = volume fraction of the fibres
V_m = volume fraction of the matrix = 1-Vfib
f_f = tensile strength of the fibre-reinforced material in fibre direction
f_{fib} = tensile strength of the fibres
f_m = tensile strength of the matrix

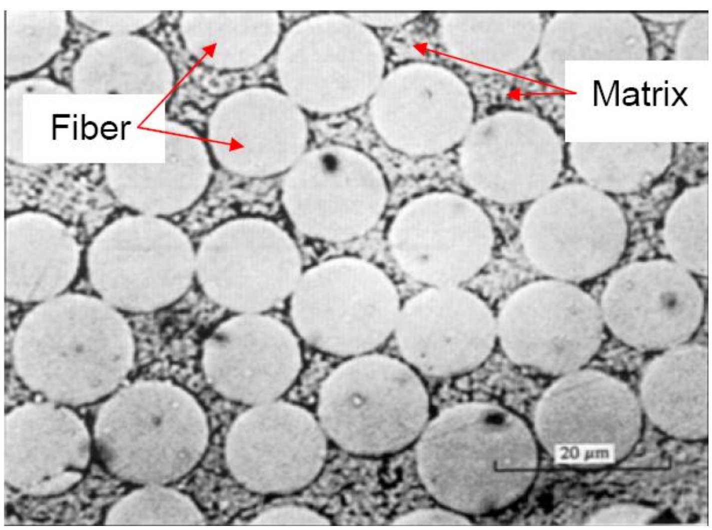

Figure 3.2 Magnified cross section of a composite material with uni-directional fibres

It should be noted that since E_{fib}/E_m >>1 and f_{fib}/f_m >>1, the above equations are approximately valid even if the second terms in the right parts are omitted. In case of prefabricated strips the material properties based on the total crosssectional area can be used in calculations and are usually supplied by the manufacturer. In case of in-situ resin impregnated systems, however, the final composite material thickness and with that the fibre volume fraction is uncertain and may vary. For this reason the properties of the total system (fibres and matrix) and the actual thickness should be provided based on experimental testing. Note that manufacturers sometimes supply the material properties for the bare fibres. In this case a property reduction factor should apply, to be provided by the supplier of the strengthening system.

3.2.4 Adhesives

The purpose of the adhesive is to provide a shear load path between the substrate (concrete or masonry) and the composite material, so that full composite action may develop. The most common type of structural adhesives is epoxy, which is the result of mixing an epoxy resin (polymer) with a hardener. Other types of adhesives may be based on inorganic materials (mainly cement-based). Depending on the application demands, the adhesive may contain fillers, softening inclusions, toughening additives and others.

When using epoxy adhesives there are two different time concepts that need to be taken into consideration. The first is the pot life and the second is the open time. Pot life represents the time one can work with the adhesive after mixing the resin and the hardener before it starts to harden in the mixture vessel. For an epoxy adhesive, it may vary between a few seconds up to several years. Open time is the time that one can have at his/her disposal after the adhesive has been applied to the adherents and before they are joined together.

Another important parameter to consider is the glass transition temperature, T_g. Most synthetic adhesives are based on polymeric materials, and as such they exhibit properties that are characteristic for polymers. Polymers change from relatively hard, elastic, glass-like to relatively rubbery materials at a certain temperature (Fig. 3.3). This temperature level is defined as glass transition temperature, and is different for different polymers.

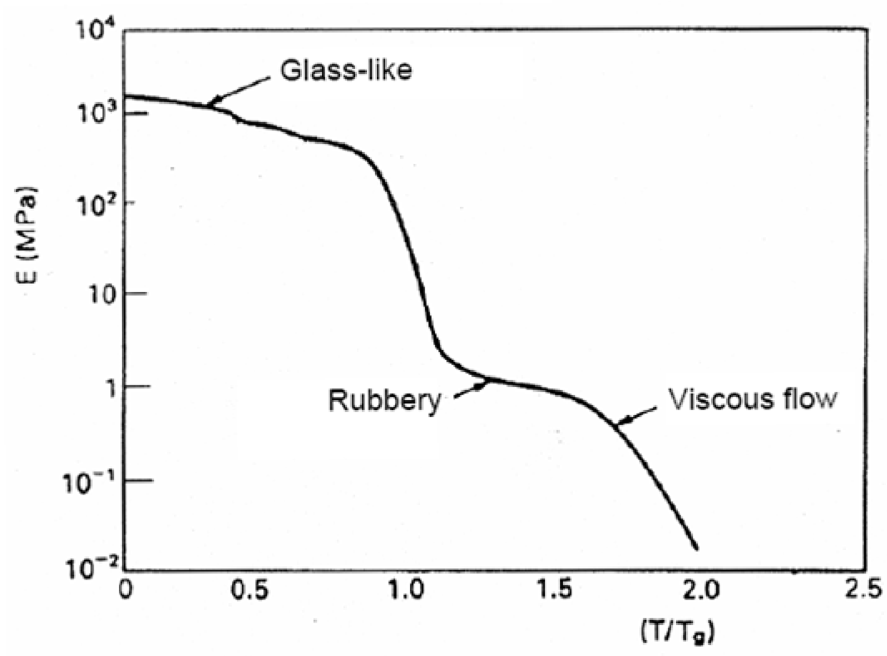

Figure 3.3 Effect of temperature on elastic modulus of polymers

Typical properties for cold cured epoxy adhesives used in civil engineering applications are given in Table 3.4 (*fib* 2001). For the sake of comparison, the same table provides information for concrete and mild steel too.

Table 3.4 Approximate properties of thermosetting polymer resins

Property (at 20^{o}C)	**Epoxy adhesive**	**Concrete**	**Mild steel**
Density (kg/m^3)	1100 – 1700	2350	7800
Elastic modulus (GPa)	0.5 - 20	20 – 50	205
Shear modulus (GPa)	0.2 – 8	8 - 21	80
Poisson's ration	0.3 – 0.4	0.2	0.3
Tensile strength (MPa)	9 – 30	1 – 4	200 – 600
Shear strength (MPa)	10 – 30	2 – 5	200 – 600
Compressive strength (MPa)	55 – 110	25 – 150	200 – 600
Tensile strength at break (%)	0.5 – 5	0.015	25
Approximate fracture energy (Jm^{-2})	200 – 1000	100	10^5 – 10^6
Coefficient of thermal expansion (10^{-6}/ ^{o}C)	25 – 100	11 – 13	10 – 15
Water absorption: 7 days -25 ^{o}C (% w/w)	0.1 - 3	5	0
Glass transition temperature (^{o}C)	50 – 80	-	-

Alternative materials to epoxies may be of the inorganic binder type. These materials are based on cement in combination with other binders (e.g. fly ash, silica fume, metakaolin), additives (e.g. polymers) and fine aggregates. In this case the adhesive also plays the role of the matrix in the composite material, hence it must be designed such that compatibility with the fibres will be maximized. General requirements for inorganic binders are high shear (that is tensile) strength, suitable consistency, low shrinkage and creep and good workability.

3.3 Durability

3.3.1 General

This chapter provides a brief overview of the durability of FRP-based strengthening systems with regard to a number of factors, namely:

- Temperature effects
- Moisture
- Ultraviolet light exposure
- Alkalinity and acidity
- Galvanic corrosion
- Creep, stress rupture, stress corrosion
- Fatigue
- Impact

3.3.2 Temperature effects

High temperatures, in the order of 60-80 °C, cause a dramatic degradation of properties in resins (matrix material in FRPs, adhesive at the FRP-substrate interface). Much higher temperatures, such as those developed during fire, result in complete resin decomposition; hence FRPs during fire cannot carry any stresses. The decomposition of glass, carbon and aramid fibres starts at about 1000 oC, 650 oC and 200 oC, respectively. Experimental results have shown that CFRP jackets suffer substantial strength reduction at temperatures exceeding approximately 260 oC. Hence, an FRP strengthening system without special fire protection measures should be considered as ineffective during (and after) fire. Fire protection may be provided using either standard mortar plastering (with a minimum thickness of at least 40 mm, according to the JSCE 2001 guidelines), or special mortars or gypsum-based boards.

3.3.3 Moisture

FRP materials are, in general, highly resistant to moisture. Occasionally, extremely prolonged exposure to water (either fresh or salt) may cause problems with some fibre/resin combinations. The resin matrix absorbs water, which causes a slight reduction in strength and the glass transition temperature. However, most structural adhesives (high quality epoxy resins) are extremely resistant to moisture (Blaschko et al. 1998). As far as the fibres are concerned, the high susceptibility of aramid to moisture deserves special attention; carbon fibres are practically unaffected, whereas glass fibres have an intermediate behavior.

At this point it is worth pointing out that full jacketing with FRP provides a moisture/vapor/air barrier which increases the longevity of members by protecting them from harsh conditions (e.g. chlorides, chemicals). On the hand, in case of poor concrete conditions, the encapsulation is at risk if the member is exposed to extreme climate cycling and/or excessive moisture. Applications of FRP to a structural member that is at risk of water pooling should not involve fully encapsulating the concrete. Good internal and surface concrete conditions, proper surface preparation, adequate concrete substrate exposure and proper application of an adequate FRP system may substantially reduce this risk.

3.3.4 UV light exposure

UV light affects the chemical bonds in polymers and causes surface discoloration and surface microcracking. Such degradation may affect only the matrix near the surface exposed to UV, as well as some types of fibres, such as aramid (Ahmad and Plecnik 1989); carbon and glass fibres are practically unaffected by UV. Anti-UV protection may be provided by surface coatings or special acrylic or polyurethane – based paints.

3.3.5 Alcalinity and acidity

The performance of the FRP strengthening over time in an alkaline or acidic environment will depend on both matrix and the reinforcing fibre. Carbon fibres are resistant to alkali and acid environment, glass fibres can degrade and aramid displays an intermediate behavior. However, a properly applied resin matrix will isolate and protect the fibres and postpone the deterioration. Nevertheless RC structures located in high alkalinity combined with high moisture or relative humidity environments should be strengthened using carbon fibres.

3.3.6 Galvanic corrosion

The contact of carbon fibres with steel may lead to galvanic corrosion, a problem which is not of concern in the case of glass or aramid fibres.

3.3.7 Creep, stress rupture, stress corrosion

In general, creep strains in composite materials loaded parallel to the fibres are very low. CFRP does not creep, the creep of GFRP is negligible, but that of AFRP cannot be neglected. Hence, the creep behavior of CFRP - or GFRP - plated RC members is governed primarily by the compressive creep of concrete. As AFRP creeps itself, long-term deformations increase considerably in the case of AFRP-strengthened elements. However, it should be born in mind that in (the very common) case when FRP strengthening systems are designed for additional loads (beyond the permanent ones), creep is not of concern.

Another important issue regarding time-effects is the poor behavior of GFRP under sustained loading. Glass fibres exhibit premature tensile rupture under sustained stress, a phenomenon called stress rupture. Hence the tensile strength of GFRP drops to very low values (as low as 20%) when the material carries permanent tension.

Stress occurs when the atmosphere or ambient environment is of a corrosive nature but not sufficiently so that corrosion would occur without the addition of stress. This phenomenon is time, stress level, environment, matrix and fibre related. Failure is deemed to be premature since the FRP fails at a stress level below its ultimate. Carbon fibre are relatively unaffected by stress corrosion at stress levels up to 80% of ultimate. Glass and aramid fibres are susceptible to stress corrosion. The quality of the resin has a significant effect on time to failure and the sustainable stress levels. In general, the following order of fibres and resins gives increasing vulnerability either to stress rupture or to stress corrosion: carbon-epoxy, aramid-vinylester, glass-polyester. We may also state that, in general, given the stress rupture of GFRP and the relatively poor creep behavior of AFRP, it is recommended that when the externally bonded reinforcement is to carry considerable sustained load, composites with carbon fibres should be the designer’s first choice.

3.3.8 Fatigue

In general, the fatigue behavior of unidirectional fibre composites is excellent, especially when carbon fibres are used, in which case the fatigue strength of FRP is even higher than that of the steel rebars (e.g. Kaiser 1989, Deuring 1993, Barnes and Mays 1999).

3.3.9 Impact

The strength of composites under impact loading is highest when aramid fibres are used (hence the use of these materials in bridge columns that may suffer impact loading due to vehicle collision) and lowest in the case of carbon fibres. Glass gives intermediate results.

4. FINITE ELEMENT ANALYSIS

4.1 Introduction

In this chapter a finite element analysis will be performed with the use of ANSYS software, in order to determine a stress reduction factor for the steel stresses. The FEM analysis will be performed on a bare steel plate and a composite plate which is composed by a steel plate reinforced with an FRP plate. The composite plate will be analyzed with the edge fibre of the steel being in tension and afterward in compression. Hence, three models in total were made.

4.2 Specifications

4.2.1 General specifications

As already explained, the reason for the FEM analysis is too evaluate a stress reduction factor for the steel stresses. Also the problem that we have to cope with is fatigue due to cycling loading. This means that the stresses in the steel remain in the elastic area and therefore a linear elastic analysis is sufficient. The analysis will be performed as a four point bending test (fig 4.1). The dimensions of the plates are 400 mm length, 100 mm width and 12 mm thickness for the steel plate and 16 mm for the FRP plate.

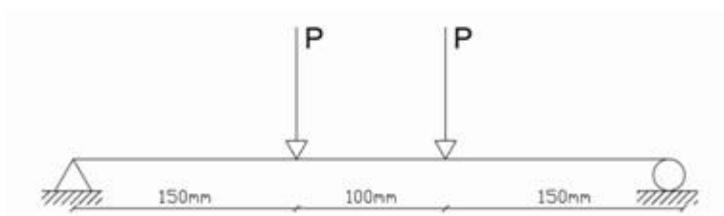

Figure 4.1 Four point bending test configuration

The modulus of elasticity for the steel is 200 GPa and the Poisson ratio 0.3. However the case for the FRP is a little bit more complicated. The problem that has to be solved here is the evaluation of an equivalent modulus of elasticity for the FRP due to the fact that it is not a linear elastic material. A typical value for the Poisson's ratio for this type of material is 0.28 which that value that is going to be used.

4.2.2 Modulus of elasticity for the FRP

The tensile modulus of elasticity will be evaluated by utilizing the corresponding formula from paragraph 3.2.2

$$E_{FRP} \approx E_{fib}V_{fib} + E_m V_m$$

In order to be able to use this formula the volumetric fibre percentage must be known and also the modulus of elasticity for the fibres and the matrix material. For this situation a typical volumetric fibre percentage is chosen equal to 60%. Assuming type A – glass fibres, the modulus of elasticity of the fibres is 73 GPa (par. 3.2.1.) . Finally assuming that we choose epoxy as a matrix material, then the modulus of elasticity of the matrix will be 3 GPa (par. 3.2.2)

Thus:

$$E_{fib} = 73\ GPa$$
$$E_m = 3\ GPa$$
$$V_{fib} = 60\%$$
$$V_m = 40\%$$
$$E_{FRP} \approx E_{fib}V_{fib} + E_m V_m = 73 \cdot 0.60 + 3 \cdot 0.60 = 45\ GPa$$

On the other since the fibres don't contribute when the FRP element is under compression, only the modulus of elasticity of the epoxy is going to be used which is $E_m = 3\ GPa$. Even from this early stage and due to the big difference between the tensile and the compressive modulus of elasticity, it can be expected that the FRP plate will not be very effective in the case in which the edge fibre of the steel will be in tension since the FRP plate will be mainly in compression.

4.3 Meshing

As already mentioned tree models in total were made. For all the models a 3D brick 20-node element has been used. For the steel plate the number of elements used is 40 for the length direction 10 for the width and 3 for the height. For the FRP plate, the number of elements is the same length and widthwise while in the direction of the height, the number of elements used is 4. In figure 4.2 the model used can be seen. In this figure the top plate is the steel one, while the bottom one the FRP. In the composite model 100% composite behavior has been assumed between the two plates.

The nodes located in the bottom corner of the bottom plate with x – coordinate equal to 0 mm, have been restrained in all directions while the ones with x – coordinate equal to 400 mm have been restrained in the Y and Z direction. The axis can be seen in figure 4.2.

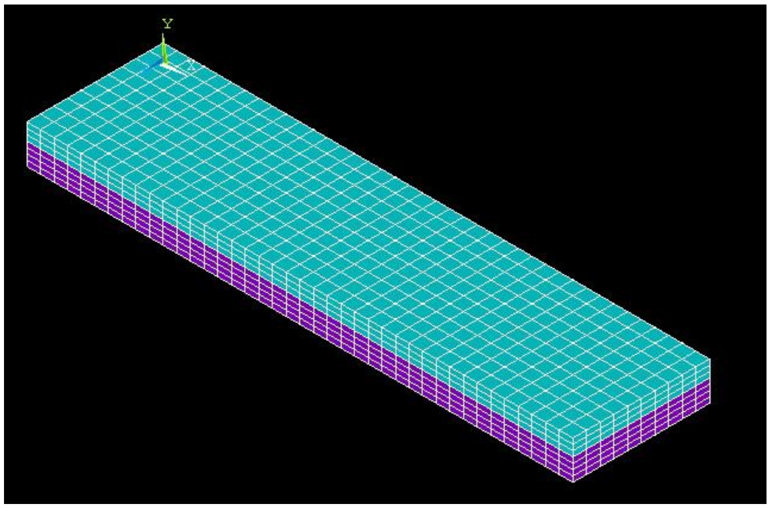

Figure 4.2 FEM model for the composite plate

The loads were applied on the nodes along the width direction with x – coordinate of 150 mm and 250 mm of the top plate. The applied load was 10 kN in total in the form of two forces (fig. 4.1) of 5 kN each.

4.4 Results

The results will be presented separately for each one of the three models. It must be noted that the force that shows up in the diagrams is the total load applied equal to 10 kN. The results that will be presented are the stresses in the steel plate against the load. Also from these stresses the position of the neutral axis will be determined. The FRP stresses are beyond the scope of this document.

Steel plate

The steel plate model is the one in which only the steel plate is analyzed in order to evaluate the stresses without strengthening via the FRP plate. The nodes to which the stresses correspond two, are the ones that are located in the middle of the plate width and lengthwise. The distance between them is 4 mm. So, in the steel plate their position is 0 mm, 4 mm, 8 mm, 12 mm. This model is symmetric and this is why the stress distribution is symmetric (fig 4.3) as well and the neutral axis is located in the mid of the steel plate in the direction of the thickness.

Figure 4.3 Normal stress – Force diagram for the steel plate model

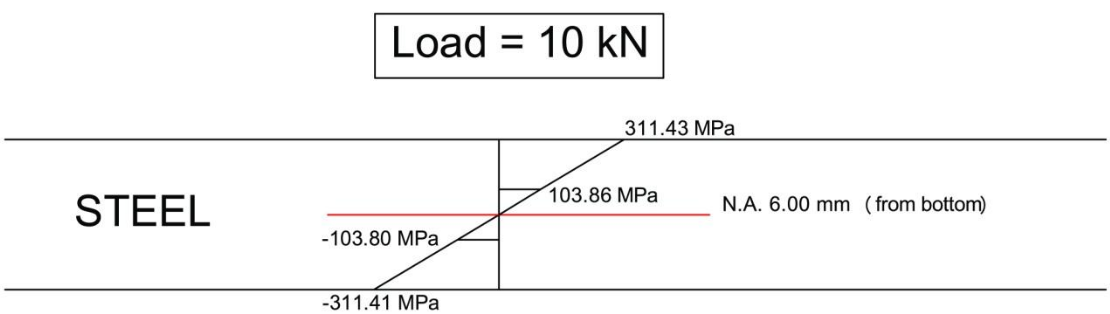

Figure 4.4 Stress distribution and position of the neutral axis for steel plate model

Composite plate 1 (Edge steel fibre in compression)

In this model, as it can be derived from figure 4.5, the FRP is completely in tension. That is why the value for the modulus of elasticity that was used was 45 GPa. The stress results can be seen in figure 4.5. Obviously, since the edge fibre of the steel is in compression, the bottom line in the diagram corresponds to the outer node while the top line to the inner node. To be more specific, the inner node is common for the FRP and the steel plate, and therefore the stress evaluated from Ansys wasn't correct. That is why the stress was evaluated from the strain multiplied with the modulus of elasticity of the steel, according to Hooke's law.

Figure 4.5 Normal stress – Force diagram for the composite plate model 1

The stress distribution for load value of 10 kN as well as the position of the neutral axis in figure 4.6. The neutral axis is located at 9.23 mm from the bottom of the steel plate. That also validates what has been said earlier and that is that the FRP plate is entirely in tension.

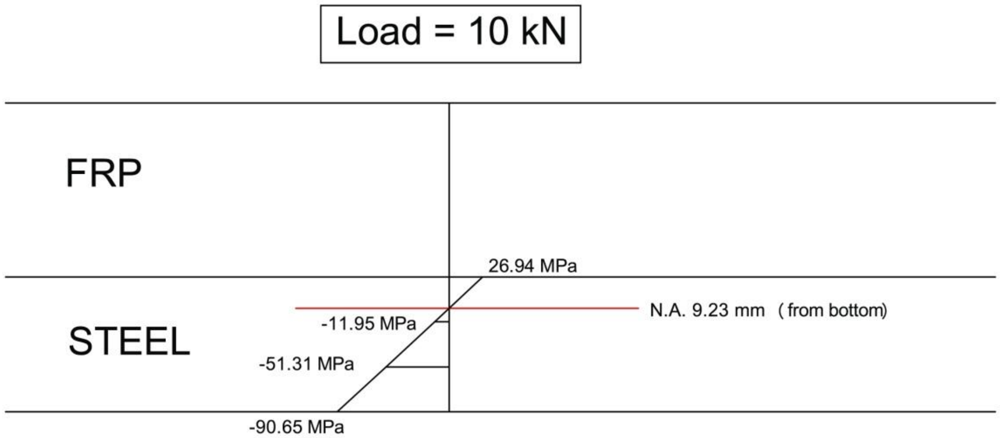

Figure 4.6 Stress distribution and position of the neutral axis for composite plate model 1

Composite plate 2 (Edge steel fibre in tension)

In this model the FRP plate is at its least favorable case, and that is full in compression. This is why the value of 3 GPa was used in the analysis for the FRP plate. Because of the low modulus of elasticity, FRP materials are not usually used in compression. Nonetheless this case is unavoidable when strengthening a steel deck and therefore it must be checked. The stresses along the thickness direction of the steel plate can be seen in figure 4.7. Since the edge fibre of the steel is in tension the line corresponding to the outer node is the upper line. As before the value of the stress for the inner node which is common for the FRP and the steel plate is evaluated from the strains.

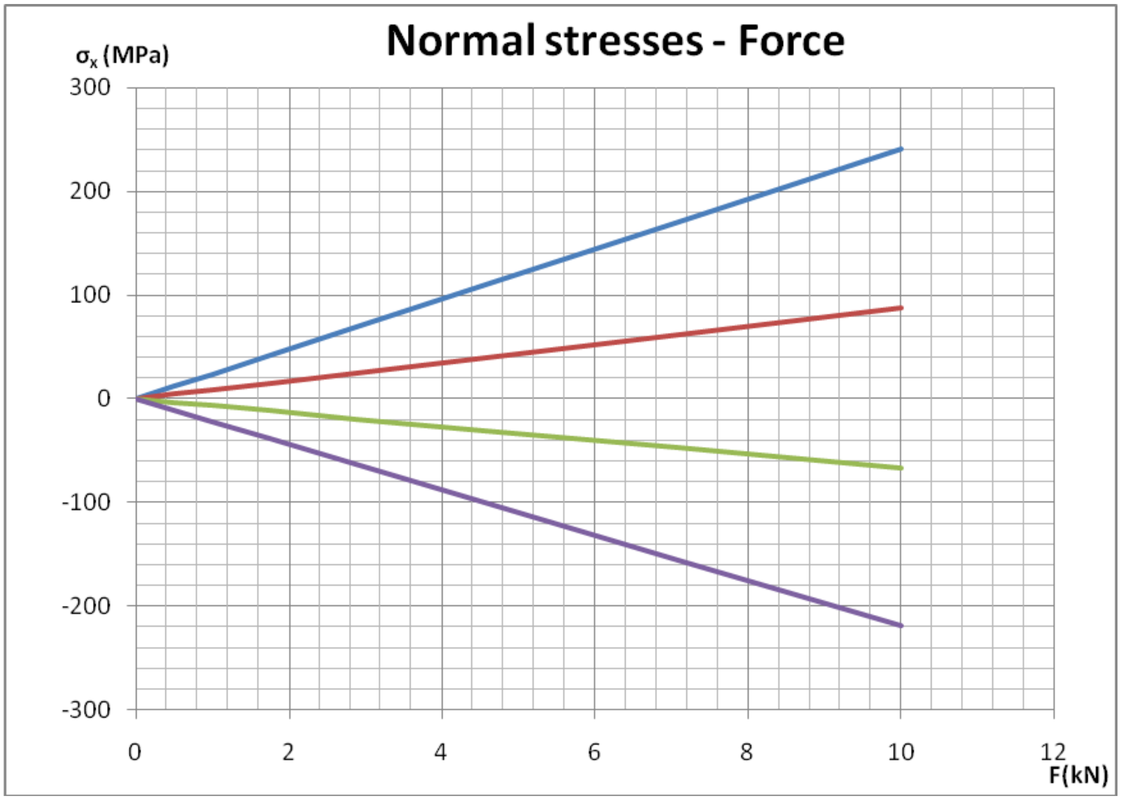

Figure 4.7 Normal stress – Force diagram for the composite plate model 2

From the stress distribution (fig. 4.8) it can be seen that the neutral axis has shifted only 0.27 mm towards the direction of the FRP plate.

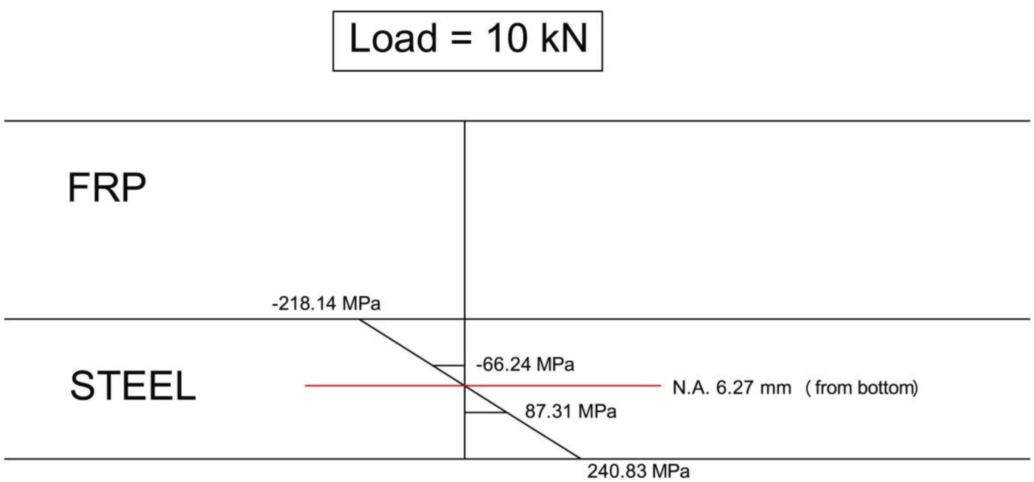

Figure 4.8 Stress distribution and position of the neutral axis for composite plate model 2

4.5 Stress reduction factor (SRF)

The stress reduction factor will help to evaluate how significant is the strengthening of the steel with the FRP plate. Two stress reduction factors must be evaluated, one for when the FRP is in tension and one for when the FRP is in compression.

The reduction factor for the first case is:

$$SRF-case-1=\frac{\text{stress in steel plate (steel plate model)}}{\text{stress in steel plate (composite model 1)}}=\frac{311.41}{90.65}=3.44$$

The reduction for the second case in which the FRP plate is full in compression is:

$$SRF-case-2=\frac{\text{stress in steel plate (steel plate model)}}{\text{stress in steel plate (composite model 2)}}=\frac{311.41}{240.83}=1.29$$

4.6 Conclusions

The stress reduction factor found for the case that the FRP is fully in compression is rather large and equal to 3.44. This could be quite sufficient, but the truth is that it depends on the needs of each individual bridge deck. Also the fact that the FRP is not very effective in compression, is being reflected in the stress reduction factor of the second case, which is 1.29 and 2.66 times less that the stress reduction factor for the first case. That could be a serious disadvantage, but again no decisions can be made with more information. In the end it depends on the needs of the bridge deck and also the cost. Also a very important issue is that a FEM analysis, especially when having to do with an anisotropic material that is so newly developed, can not be fully trusted and therefore experimental validation is needed.

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