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Preface

This master thesis is the final project of my 2-year master program at the faculty of science and technology at the University of Stavanger. The thesis has been completed between the months of March and June in the spring semester of 2022, and has been prepared together with supervisor Guillermo Rojas Orts. The thesis is written in English as a challenge to learn international professional language, easier implementation of online resources, and to better cooperate with my supervisor.

The subject of the thesis was chosen to increase my knowledge of concrete structures that are both prestressed and fiber reinforced, in combination with the required passive reinforcement. The study addresses the dimensioning of a walkway bridge with fiber reinforcement according to the guidelines of the Norwegian Concrete Associations new publication no.38, together with the draft of the new EN1992-1-1 standard for concrete structures. To work with this literature has provided me with valuable insight into the possibilities of fiber-reinforced and prestressed concrete structures. I have gained relevant knowledge that I can bring with me into the working life as a civil engineer.

I would like to thank Guillermo for helping me define the scope of the thesis, as well as following up continually with help and tips, and always be available for advice and guidance.

Bergen, June 2022

Omar Korsnes

Abstract

The advances made in the field of prestressed concrete have widened the freedom in building structures. This allows achieving high loading capacity with the adaption of longer spans and smaller cross-section dimensions. Post-tensioning technique of prestressing is the most adopted method by the industry due to many advantages.

Fiber reinforcement improves the material properties of the concrete through increased general tenacity, reduction of crack widths, and a well-documented increase of shear capacity. (Kanstad, et al., 2020) However, fiber-reinforced concrete is rarely used in load-bearing structures due to lack of experience and guidelines. Its use may increase as the Norwegian Concrete Association's recently published publication no. 38 *''Fiber-reinforced concrete in load-bearing structures''* (NB38) provides guidance in the use of fiber concrete.

Post tensioning reinforcement combined with fiber reinforced high strength concrete is the foundation on which a walkway bridge for this thesis is designed as a study case. It is designed after NB38 combined with the newest draft of EN1992-1-1 with Annex L for fiber reinforcement. The draft of the standard is used for experimental reasons and with interest in looking at future design principles. The thesis examines the potential of designing structural elements with small cross-sections that leads to aesthetically pleasing structures with opportunity to decrease material use and the use of extra reinforcement.

The standards have a required minimum amount of reinforcement for constructions where a collapse can lead to loss of human life or cause large social costs. It is concluded that over half the volume of concrete used for the bridge elements does not need more reinforcement than the minimum, and thus a great deal of reinforcement steel is saved. This leads to significant economic and environmental benefits and points to the advantages of fiber reinforced and prestressed concrete.

The structural design of the bridge is based on similar bridges such as (Lopez, et al., 2015) and (di Prisco, et al., 2022) that both are interesting projects with the implementation of a post-tensioned fiber reinforced concrete. Bridges like these are implementing ultra-high strength concrete, with strengths up to $f_{ck} = 150\ MPa$. However, with the prerequisites of the thesis to be NB38 in combination with the Eurocode draft, the concrete strength has been reduced to an appropriate value of $f_{ck} = 100\ MPa$ to be eligible with formulas from this literature.

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PART I – THEORY

1 Fiber reinforced concrete

Concrete is a material with high compressive strength, while the tensile strength is low. After cracking, an unreinforced concrete has no tensile strength, and therefore reinforcement must be used. The fiber reinforcement works in principle in the same way as traditional reinforcement, where it connects the concrete across the cracks and transmits the tensile stresses. How much stress that can be transmitted will vary with fiber type and dosage amount. (Kanstad, et al., 2020)

The use of fiber as a replacement for ordinary slack reinforcement in concrete structures is very relevant due to increasing requirements for rational design, as well as health, safety and environment-conditions for steel binders and lack of manpower. Fiber reinforcement in cast-in-place concrete structures is today mainly limited to steel fiber reinforcement in floors on ground as well as in shotcrete. It can however also be used in a wide range of construction parts. (Kanstad, et al., 2020)

1.1 What is fiber reinforced concrete?

Fiber reinforced concrete is a concrete that contains a fibrous material which increases its structural integrity. The fibers are uniformly distributed through the concrete and is randomly oriented. There are several different types of fibers, ranging from steel fibers, glass fibers, synthetic fibers and natural fibers. With different types of fibers, the properties both in soft and hardened state of the concrete is affected. Changes in the characteristics are due to varying concretes, fiber materials, geometries, distribution, orientation and density. (Mishra, 2022) Figure 1 illustrates a close-up of a steel fiber reinforced concrete after and before drying. The dried concrete is purposefully broken in half to clearly illustrate the fibers and how they connect the concrete.

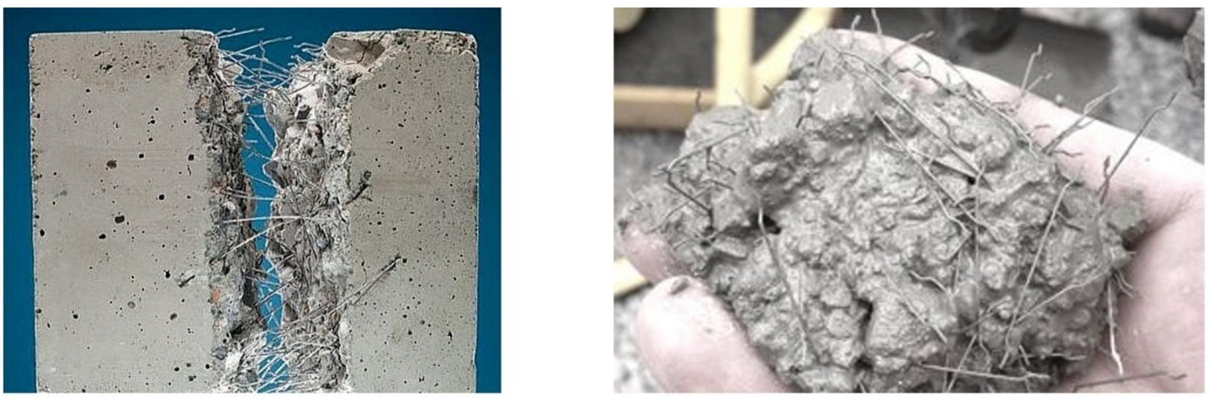

Figure 1 – Steel fiber in concrete (Mahajan, 2022)

Extensive research and development in recent years has provided new insight into problems and opportunities related to the use of fiber as reinforcement in concrete structures. Suitable material types have been developed, there have been performed laboratory and field experiments, as well as theoretical analyzes and practical experiences with non-traditional fiber-reinforced construction elements. For example, fiber reinforcement in combination with self-compacting concrete has in practice been shown to give a load-bearing capacity far higher than corresponding structural elements in ordinary vibrated concrete. (Kanstad, et al., 2020)

Construction parts that are reinforced with fibers is less expensive than hand-tied rebar, while still increasing the tensile strength. The shape, dimension and length of the fibers are important. A thin and short fiber for example, will only be effective the first hours after pouring the concrete and reduces the cracking while the concrete is stills drying and stiffening, but will not increase the concrete tensile strength. (Mishra, 2022) This is due to the size not having enough fastening area in the concrete to be sufficiently reducing cracks after drying.

The technology behind fiber reinforced concrete has reached a level where the solution can be used beyond the more traditional areas of shotcrete and flooring. Practice, experiments, and theory indicate that fiber reinforced structures with and without ordinary bar reinforcement can provide acceptable safety of load-bearing structures if the fiber is distributed and oriented as provided. An important factor regarding fiber reinforced concrete is that elements induced by moment and/or axial force with no apparent traditional reinforcement has significantly poorer strength and ductility against these forces. The guidance therefore presupposes that all construction parts where there is a danger of life in the event of a collapse, or where there is a risk of falling, shall be supplemented with bar reinforcement or prestressing reinforcement that can transmit the tensile forces from bending and axial forces. (Kanstad, et al., 2020)

All structural elements in this thesis are designed with this in mind, always implementing a minimum traditional reinforcement. In some elements it is also strictly needed for load bearing capacities.

1.2 Steel fiber

Steel fiber reinforcement is supplied in different shapes and sizes. Some variations of the different types are illustrated in figure 2. The fibers can for example be straight, bow shaped, toothed, or profiled and come both with and without end hooks. The most common cross section for the steel fibers is the circular cross section. Steel fibers can have different diameters, where it normally varies in a range from 0.2 mm to 1 mm. The length also varies, from 10 mm to 60 mm.

For a different behavior in relation to the concrete, the steel fibers can have different surface types.

The surfaces vary from smooth, rough or surface-treated for increased adhesion or corrosion protection. (Löfgren, 2005)

Steel fibers are most often made of high-strength steel with relatively high tensile strength and modulus of elasticity. The tensile strength varies from 200 to 2600 MPa, and the modulus of elasticity ranges from 195 to 210 GPa. The elongation at break is typically 0.5-5%. (Löfgren, 2005)

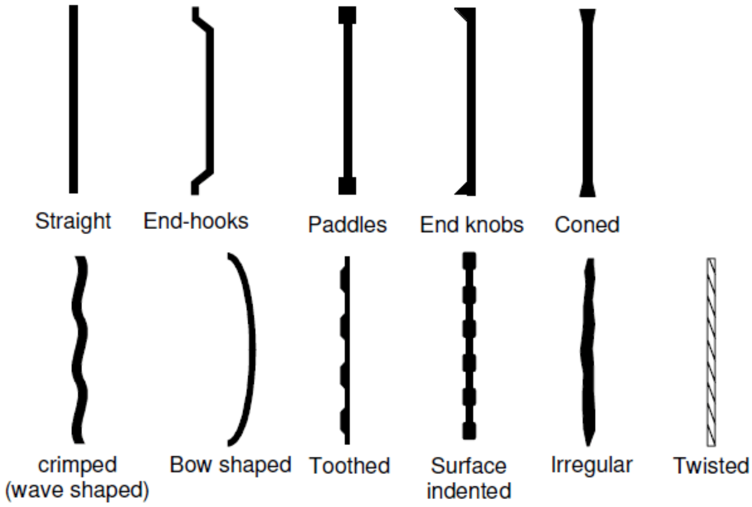

Figure 2 – Different types of steel fiber shapes (Tameemi & Lequesne, 2015)

1.2.1 Compressive strength

Steel fibers does not greatly enhance the static compressive strength of concrete, where the strength can be increased from between 0% to 25%. Structural members that also contains traditional reinforcement, does not have their compressive strength significantly affected by the steel fibers. The steel fibers will however substantially increase the post-cracking ductility, and the energy absorption of the material. (Chanh, 2005)

This is shown graphically in the compressive stress-strain curves in figure 3.

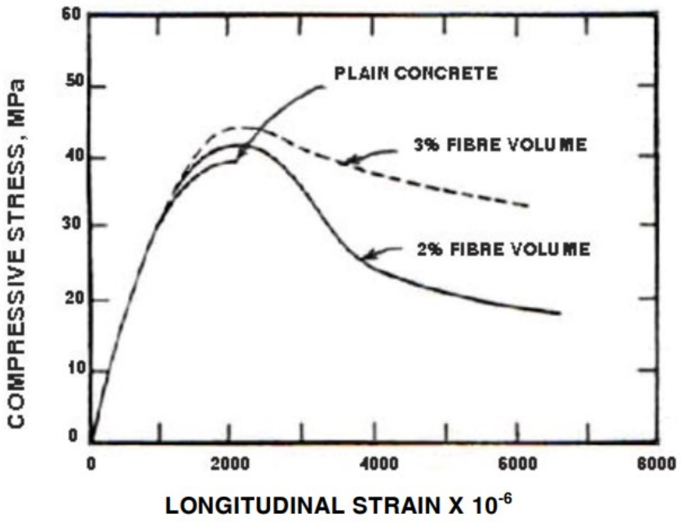

Figure 3 – Stress-strain curves in compression for steel fiber reinforced concrete (Chanh, 2005)

1.2.2 Tensile strength

When fibers are used in the concrete mixture, the alignment of each fiber is significant. When the fibers are aligned in the direction of the tensile stress, they may bring a very large increase in direct tensile strength. However, for more randomly distributed fibers, the increase in strength is less noticeable. This increase can for randomly distributed fibers be as little as 0% and, in some instances, up to perhaps 60%. With steel fibers aligned with the tensile stresses, this value can be as high as 133%. The alignment of the fibers where needed can however not be assured to be the case in normal methods of fiber reinforcing of concrete. (Chanh, 2005)

Investigations have also indicated intermediate values, as shown in figure 4.

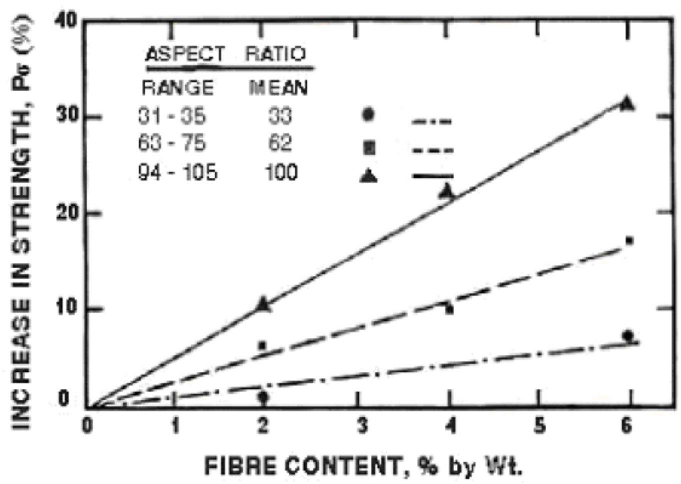

Figure 4 – Influence of fiber content on tensile strength (Chanh, 2005)

1.2.3 Flexural strength

The steel fiber reinforcement has a significant better effect on the flexural strength than either the compressive or the tensile strength. Increases of more than 100% have been reported, where the strength is more sensitive to the type of fiber solution. This means that the increasing flexural strength is very much dependent on the fiber volume and the aspect ratio of the fibers, where a higher aspect ratio leads to greater increase in strength. (Chanh, 2005)

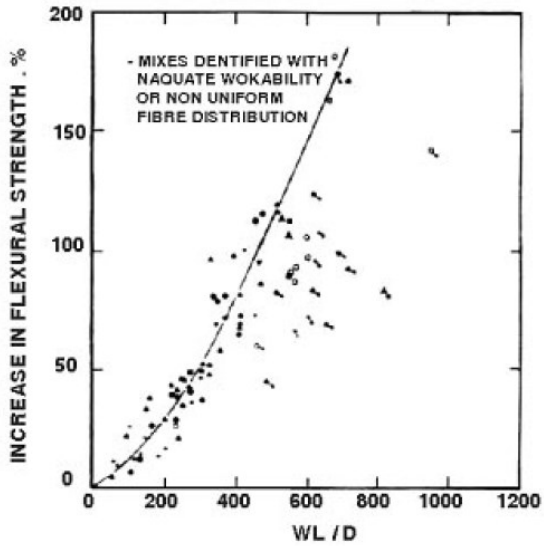

Figure 5 – Effect of aspect ratio and weight of fibers on flexural strength in concrete (Chanh, 2005)

Figure 5 illustrates how the increased values for WL/D causes an increase in the flexural strength of the concrete.

W is the weight percentage of fibers

L/D is the aspect ratio of fibers

With values of WL/D above 600, the characteristics of the mix can be unsatisfactory.

1.3 Practical conditions

A fiber reinforced concrete will have several factors that affects the strength after cracking. There is identified decisive factors for the quality of the end product in the PhD of (Døssland, 2008). These factors are varying form the casting process, the concrete mix, the geometry of the element, and the size of the element as well as obstacles such as rebars.

Experiments have shown that when casting a fiber reinforced concrete, the fibers tend to lie perpendicular to the casting direction, and the vibration of the concrete affects both the orientation and distribution. If the choice of fiber type combined with a poor proportioning of the fiber amount is the case, then the result may be an inhomogeneous concrete with ''fiber balls'', where it is an accumulation of fibers in the same place. It should not be longer breaks during the casting, so that the layer formation and continuity is not disrupted. Fiber reinforced concrete is normally implemented in combination with normal rebars, but these can lead to a fiber accumulation and uneven fiber distribution which should be considered. (Kanstad, et al., 2020)

During casting, the steel fibers tend to orient themselves parallel to the formwork. This provides a two-dimensional orientation for fibers in plates and walls, and will be directly corresponding to the thickness and length of the plate. Parallel fiber orientations will be beneficial in terms of bending moment capacity of thin plate elements. (Døssland, 2008)

In common for these factors is that careful planning is important when making fiber reinforcement concrete structures.

1.4 Fiber reinforcement behavior

Concrete is naturally a brittle material, and application of loads to concrete structures will lead to the formation of cracks. Fiber reinforcement mixes with the concrete solution and forms a composite material between the concrete and the fibers. This creates a continuous reinforcing net in the concrete that can absorb stresses in all directions where cracks may occur. In this way, the load-bearing capacity and robustness of the concrete is increased, and it will have a more ductile behavior. (Löfgren, 2005)

Depending on the shape and strength of the fiber, the following fracture mechanisms may occur during the formation of cracks (Døssland, 2008) (Löfgren, 2005):

- Tension fracture in fibers. Fiber with good adhesion conditions will have a plastic break before tearing.
- Loss of adhesion between fiber and matrix. As the fiber-reinforced concrete is applied to loads, the fibers will absorb more and more tensile stress in critical cracks. Depending on the type of fiber used, loss of adhesion may occur, and the carrier may be torn out.
- Crushing or peeling of the matrix. Large local stresses can occur in the concrete around the fiber if it crosses the crack at an angle or has end hooks.
- Plastic deformation of the fiber ends. For fibers with end hooks or other anchoring, critical tensile stresses can tear out the fiber by plastic deformation of the ends.

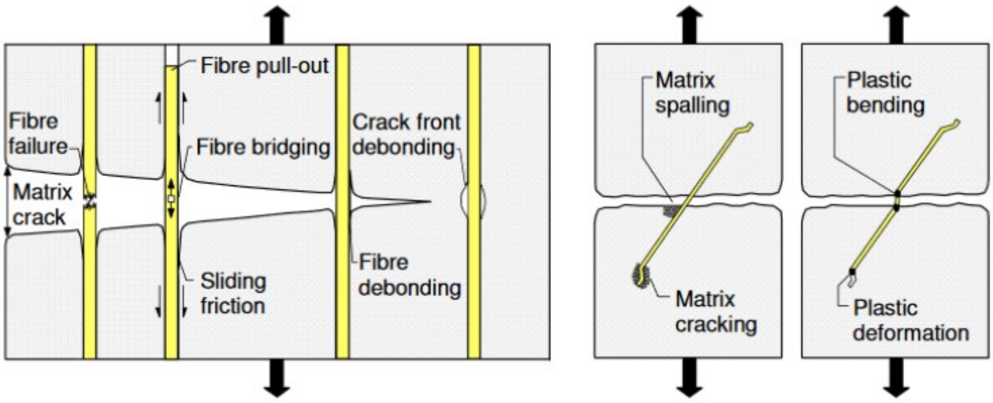

Figure 6 – Fracture mechanics during cracking (Löfgren, 2005)

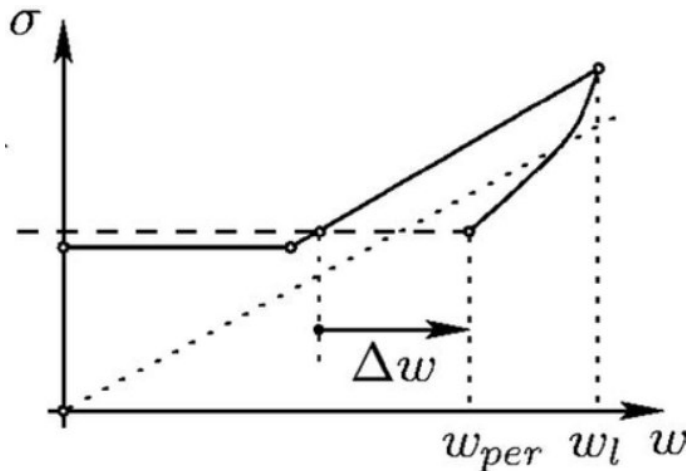

Figure 7 – Stress and crack width diagram showing the increasing of crack widths until failure (Muttoni, 2007)

1.5 Self-compacting concrete

Self-compacting concrete can be defined as a concrete with super-plasticizing additives in a fresh concrete that flows under its own weight and does not need vibration. This leads to good flow properties and consolidation of the concrete. It is used in the construction where it is hard to use vibrators for consolidation of concrete. Vibration of concrete is undesirable when using fiber reinforcement. (Neenu, 2022) (Kanstad, et al., 2020) This type of concrete with good flow properties will lead to reduced working hours and improved working conditions during casting. In general, it will also lead to a good homogeneity of the material and a distribution of the concrete that provides better quality and durability. The surface may also be generally finer after curing. If there is difficult geometry that is casted, this type of concrete will have an advantage. A disadvantage is the increased price of this concrete. (Löfgren, 2005)

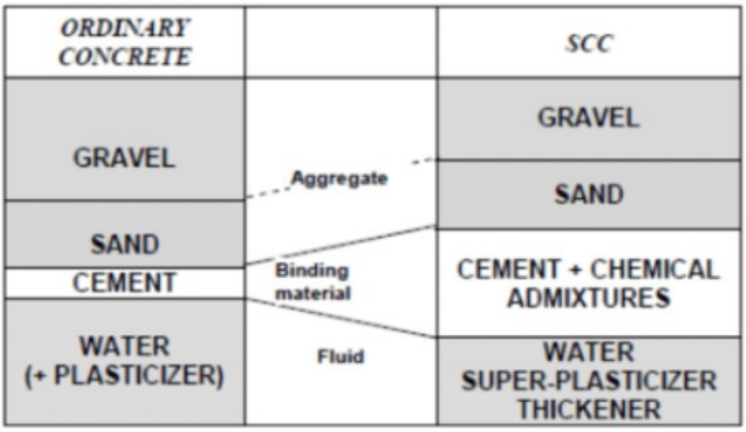

Figure 8 – Different concrete mixtures for ordinary concrete and self-compacting concrete (Neenu, 2022)

2 Post-tensioning

''The engineering-best friend of developers, architects, engineers and contractors – post tensioning enables the construction and refurbishment of concrete structures; improving structural performance while also reducing construction time, costs, materials and environmental impact.'' (Freyysinet, 2017)

Figure 9 – Example of post-tension cables on a construction project (Daily Engineering, 2021)

Figure 9 illustrates a post-tensioning cable layout in casting a large concrete slab. Cables are laid in each direction combined with traditional reinforcement, with different spacing to provide necessary strength where it is needed. The curved layout indicates where there is needed downforce or an uplifting force to counteract external loads, where the uplift is usually needed in spans while downforce is applied over supports in mid span.

2.1 What is post-tensioning?

Post-tensioning is a method of reinforcing concrete where high-strength steel tendons are positioned in ducts or sleeves before the concrete is placed. Once the concrete has gained strength, tension is then applied, pulling the tendons and anchoring them against the outer edges of the concrete, before service loads are applied. (Freyysinet, 2017) Most precast and prestressed concrete is actually pre-tensioned, where the steel is pulled before the concrete is poured. Post-tensioned concrete can thus still be classified as prestressed. (Palmer, 2015)

For prestressed concrete structures as opposed to traditional reinforced structures, concrete with higher strength is used.

This is due to large local anchoring forces as well as the concrete generally being subjected to a greater deal of compressive stresses due to the prestressing reinforcement. Common strength class values are C35-C55, where the chosen concrete of C100 for this thesis is well over the strength requirements.

Figure 10 shows the principle of how one form of post-tensioning works, where the force from the tension-cable counteracts cracking and deflection from external loads on the concrete. The left figure is an exaggeration of the cracking process without post-tensioned cables, showing the advantages of the post-tensioning on the right.

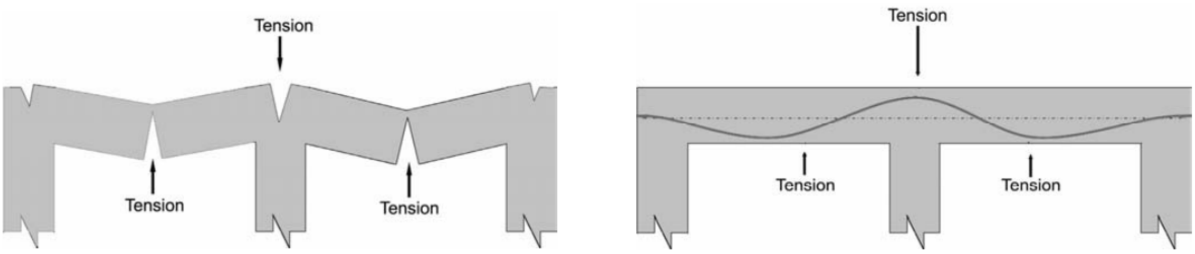

Figure 10 – The principle of post-tensioning in a two-span frame structure (PTI Certification, 2006)

Adding a post-tensioned solution to concrete, combines the reinforcing tension action with the usual advantages of compression in concrete. Additional benefits are obtained when the post-tensioned reinforcement is installed in a curved profile as shown in the right on figure 10. In multiple span constructions, the cables would typically be elevated in a high point over the middle supports, and have a low point in the sagging span. The optimal efficiency is obtained in this way, because the post-tensioned reinforcement is located in the tension zones, the concrete is being compressed, and the post-tensioned reinforcement creates an uplift force in the spans where it is needed the most. (PTI Certification, 2006)

2.2 Applications and advantages

There can be post-tensioning applications in almost all phases of a construction project. In building construction, post-tensioning allows longer spans, thinner slabs, fewer beams and more slender, dramatic elements. This is mainly because the prestressed reinforcement has a beneficial effect on the deflection in serviceability limit state, by counteracting the deformation caused by external loads. (Civil, 2019)

The thinner elements made possible by these solutions also means that less concrete is required. In addition, it leads to a lower overall building height for the same floor-to-floor height. Post-tensioning can thus allow a significant reduction in building weight versus a conventional concrete building with the same number of floors. (Civil, 2019) When reducing the weight of the concrete building, the total load going down in the foundation is greatly reduced, and can be a major advantage in seismic areas.

Another advantage of a post-tensioning solution is that beams and slabs can be continuous. This means that a single beam or slab strip can run continuously from one end of the building to the other. Structurally, this is much more efficient than having a beam that just goes from one column to the next. Post-tensioning is also the favorable system for structures like parking garages. This is because it allows a high degree of flexibility in the column layout, span lengths and ramp configurations.

In areas where there are expansive clays or soils with poor bearing, slabs that are post tensioned can reduce problems with cracking and differential settlement. A slab will shrink as the concrete cures (hardens). In slabs on ground, the friction between the slab and the ground acts to resist this shrinkage and results in what are commonly referred to as shrinkage cracks. By using post-tensioned cables to compress the concrete, the formation of visible shrinkage cracks can be greatly reduced or even eliminated as shown in figure 11. Post-tensioning is used in this way in millions of square feet of warehouse floors, sport courts, housing and specialized paving applications. (PTI Certification, 2006)

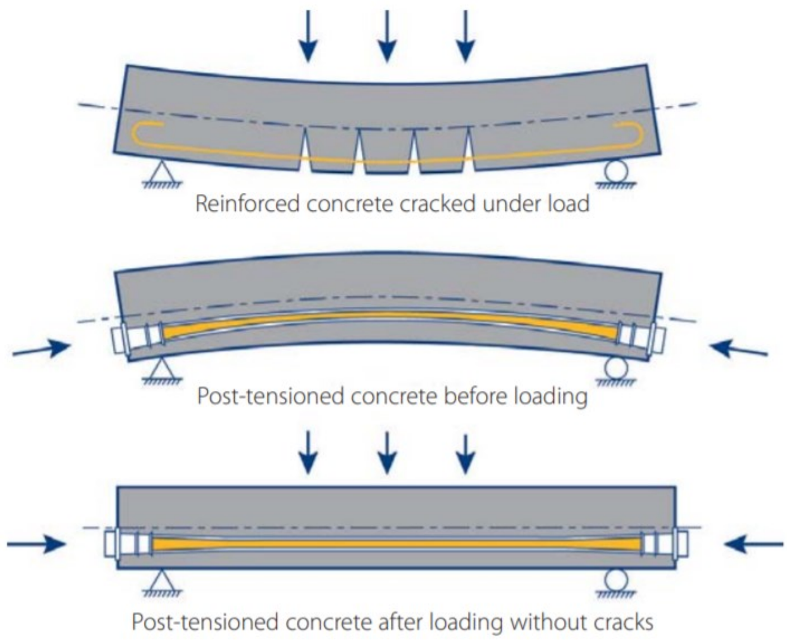

Figure 11 – Example of post-tensioning application (BBR VT INTERNATIONAL LTD, 2022)

The pressure that arises in the concrete that follows from the prestressing reinforcement limits cracking and crack widths, which is favorable in terms of density, durability and flexural stiffness. The compressive stresses will also have a positive effect on the shear capacity, which will increase depending on the compressive stresses in the concrete.

Post-tensioning and prestressed reinforcement are very effective for bridge structures. It allows them to be built with very demanding geometry requirements, including complex curves, variable elevation, and significant grade changes. It also allows longer span in bridges without the use of intermediate supports. This is the case in the bridge design for this thesis. This minimizes the impact on the environment and avoids disruptions to traffic below the bridge on eventual roads or walkways. It allows for slender single-span bridge structures. (Civil, 2019)

2.3 Mechanical properties of cables

In order to achieve the greatest possible prestressing of the structure, i.e. minimize the tensile stresses of the concrete in the serviceability limit state, it is necessary to use steel with high strength. Normally, the tension steel used for prestressed cables have a higher tensile strength than traditional reinforcement. This is to reduce the stress less of long-term effects such as shrinkage and creep in the concrete as well as relaxation in the tensioned prestressed steel. (Sørensen, 2013)

For the tension cables, the characteristic strength the steel has acquired at 0.1% inelastic strain is defined and is usually referred to as the ''0.1% limit'' $f_{p0.1k}$. Figure 12 presents an idealized working diagram for prestressed steel, and is the background for dimensioning prestressed steel in the ultimate limit state.

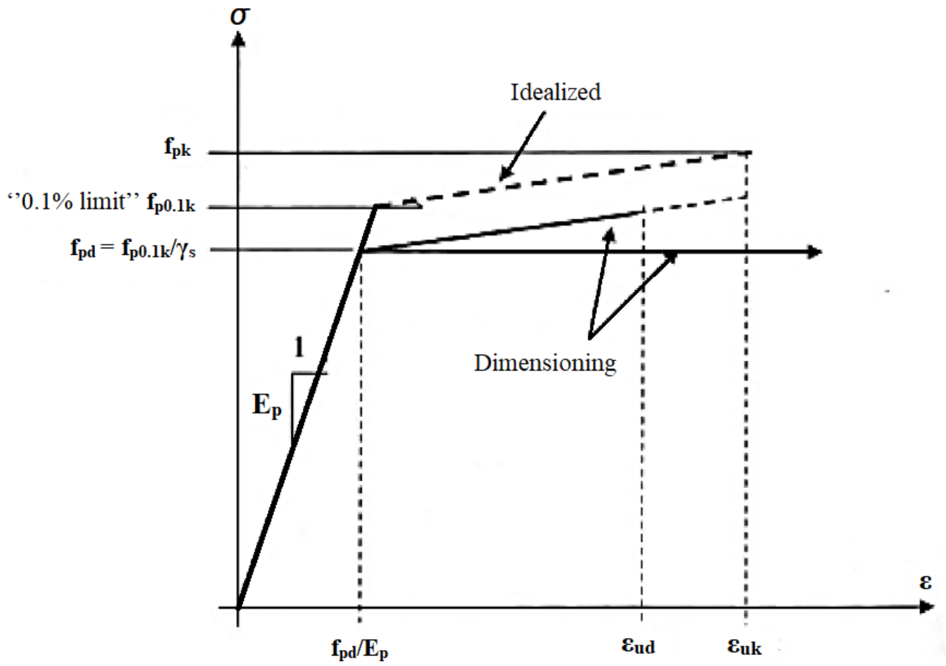

Figure 12 – Stress-strain diagram for post-tensioning cables (Sørensen, 2013)

For post tensioned constructions, the tensioning cables are made in the form of threads with a diameter of 4-5mm. These threads have a characteristic strength between 1500 – 1800 MPa and a modulus of elasticity of 195 GPa. These tensioned threads are then spun together into tensioning cables, which usually consist of seven tensioning threads. The tensioning cables can be normal or compact, as shown in figure 13. The compact cables have a predictably larger steel area for a given diameter, and will therefore be able to absorb greater forces per diameter unit. (Sørensen, 2013)

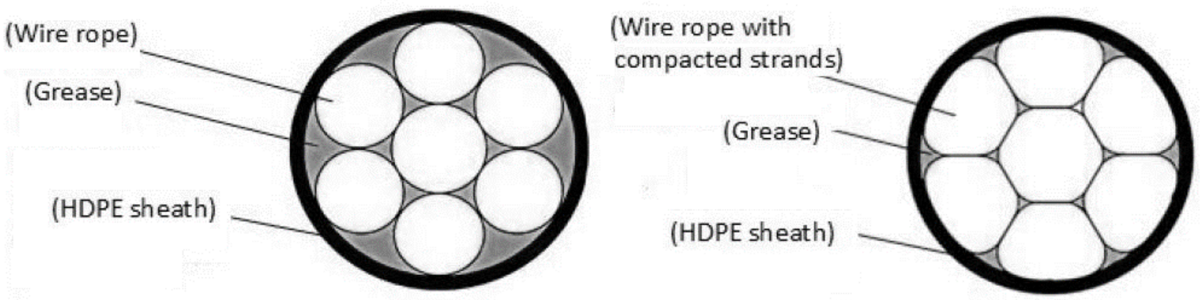

Figure 13 – Illustration of normal and compact tensioning cables (SIA Building & Steel, 2022)

The protective layer of the post-tensioned cables in either ducts or sheaths, provides a corrosion protection. Some cables are unprotected and casted directly in contact with the concrete. A cable in a duct that contains several prestressing steel strands is commonly called a multistrand tendon, and a cable with only a single prestressing steel strand is usually covered in a plastic sheathing and is referred to as a mono-strand tendon.

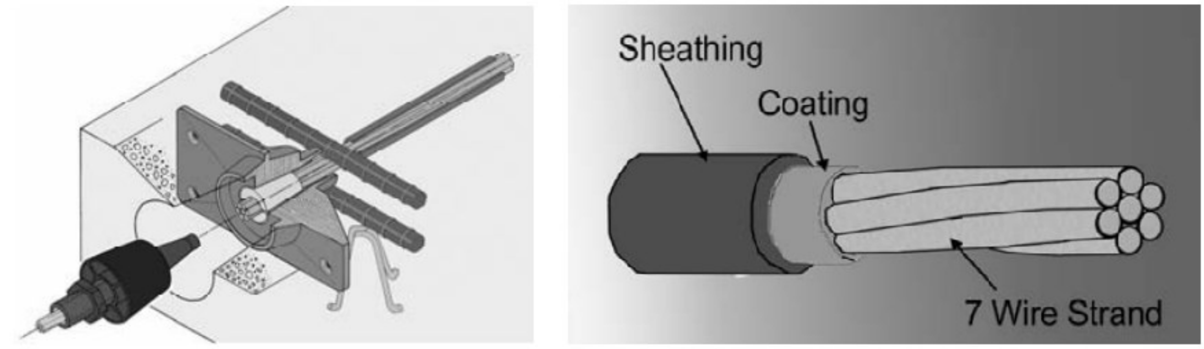

Figure 14 – Illustration of anchorage point and an unbonded cable (PTI Certification, 2006)

2.4 Bonded and unbonded prestressed cables

The sheathing or ducts are normally filled with coating around the prestressed steel that provides another level of corrosion protection as shown in figures 14 and 15. The coating can be a formulated type of grease, or a specially designed type of grout. The difference between grease and grout is the bonding of the steel tendons. When using grease, the steel inside is permanently free to move relative to the sheath or duct around it, and is referred to as an unbonded tendon. When the grout is used, the steel is permanently bonded to the sheathing or duct and is thus a bonded tendon. (PTI Certification, 2006)

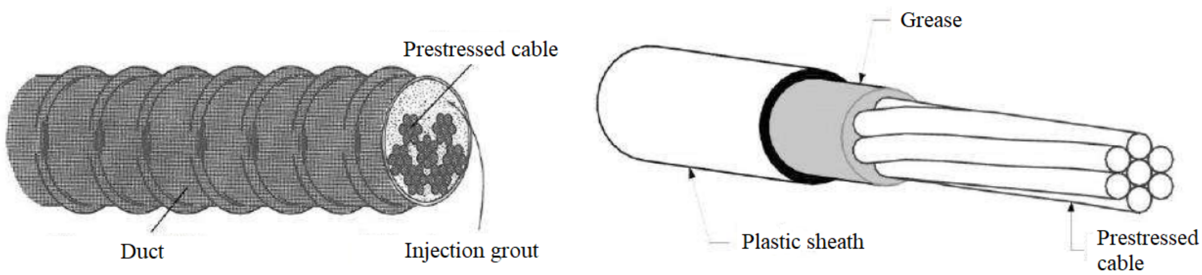

Figure 15 – Example of multiple cables in a duct and a single coated cable (Olsen & Arntsen, 2021)

In general, bonding will provide a greater bending moment capacity, as well as lead to a more favorable crack distribution. In terms of safety, you are also not dependent on the end anchors as you are for the unbonded prestressing reinforcement, as there is adhesion between the reinforcement and the concrete. Unbonded cables on the other hand, is easier and faster to perform as one avoids the injection work, and is therefore a more economical solution. The grease is resistant to fire, and the low friction due to the grease also leads to a less efficient loss of tension in the reinforcement.

In the case of unbonded cables, it is also possible to replace damaged prestressing reinforcement. (Sørensen, 2013)

2.5 Anchoring and anchoring forces

When using the prestressing reinforcement, one is dependent on anchoring to hold the reinforcement in place as it is clamped, and for prestressed structures without injection, it is the anchoring that transmits the compressive stress to the concrete. A distinction is made between active anchoring, passive anchoring, and intermediate anchoring. (Spennteknikk, 2011) Active and passive anchoring must ensure power transmission from the reinforcement to the concrete, and is mounted on the end formwork of the concrete. Passive anchoring retains the prestressing reinforcement in the hardened concrete, while active anchoring makes it possible to prestress the reinforcement by resistance.

Examples of anchoring solutions are shown in figure 16, where the prestressing reinforcement is fixed with a locking wedge at the passive end. This locking wedge is secured in place by means of a spring-loaded end cover. In the active part, the cable is tensioned by a hydraulic jack, and the locking wedge is pressed into the anchor head which ensures the fastening. In cases where elements are cast in several stages, intermediate anchoring can be used in the casting joint to reduce friction loss. (Spennteknikk, 2011) It is also common to have gap tensile reinforcement behind the anchor plate to avoid peeling due to large, concentrated forces.

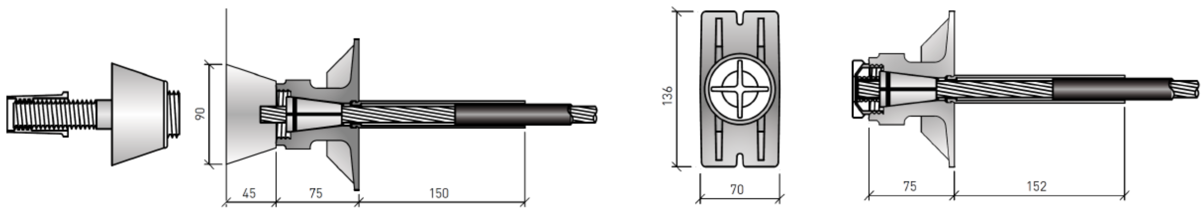

Figure 16 – Anchorage with locking wedge (Spennteknikk, 2011)

While anchoring the prestressed reinforcement the concrete will be subjected to concentrated forces. If the execution of the tensioning is done correctly, there will always be axial compressive forces. Depending on the eccentricity of the anchorage and the angle of the force, vertical forces and bending moments may also occur at the anchorage point. Figure 17 illustrates the end anchoring and the anchoring forces of prestressed concrete, where the left part is the anchorage force, and the right part is the equivalent occurring forces.

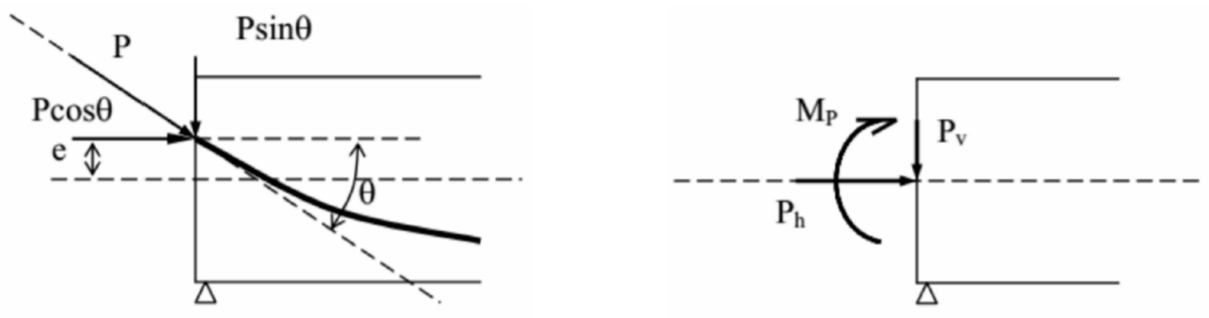

Figure 17 – Anchorage forces simplifications (Sørensen, 2013)

Since the length of the prestressing reinforcement is often large in relation to the height of the concrete cross section, the angle of inclination will often be small and the statically equivalent anchoring forces can be simplified as:

$$P_h = P \cdot \cos\theta \approx P$$

$$P_v = P \cdot \sin\theta \approx P \cdot \theta$$

$$M_p = P \cdot \cos\theta \cdot e \approx P \cdot e$$

2.6 Load balancing

The prestressing reinforcement profile is decisive for the mechanical operation of the concrete, and in cases where the prestressing cables are curved, an evenly distributed transverse force will occur. This force effect is called equivalent forces, and the magnitude depends on the tensile force, the slope and the length of the prestressing reinforcement. Utilizing the equivalent forces to directly counteract self-load and payload is called load balancing. (Sørensen, 2013)

Figure 18 illustrates an arbitrary profile of prestressed concrete layout, and the layout can in general be defined by the following function:

$$y = f(x)$$

The cables can be laid with curvature as described in chapter 2.1. When the angle of curvature is small, the slope of the curve can be defined as:

$$\theta(x) \approx \tan\theta(x) = \frac{dy}{dx}$$

Equilibrium on the y-axis can be defined as approximately:

$$q(x) \cdot dx \approx P \cdot d\theta$$

Now, the equivalent loads in the y-direction can be defined as:

$$q(x) = P \cdot \frac{d\theta}{dx} = P \cdot \frac{d^2y}{dx^2}$$

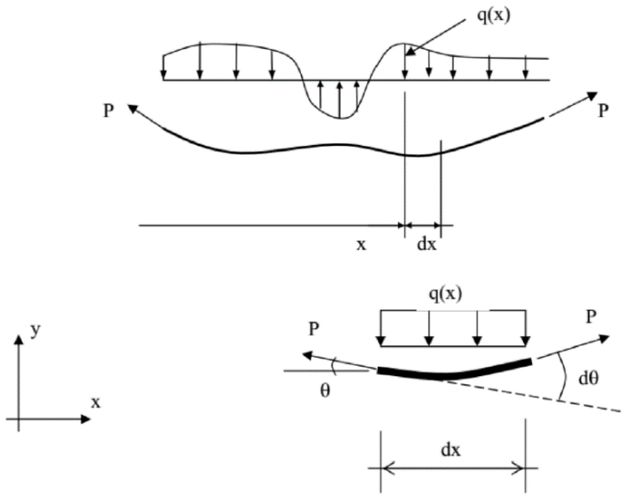

Figure 18 – Arbitrary profile of a prestressed cable (Sørensen, 2013)

The equation for equivalent load in y-direction generally applies to all curved prestressed reinforcement profiles. In cases where the prestressing reinforcement profile is straight, i.e., no curvature, one can show from the equation that no distributed transverse forces arise from the prestressing reinforcement. This is because a linear reinforcement profile $(y = ax + b)$ will give $q(x) = 0$.

For any buckling points, as shown in figure 19, the force situation must be treated separately. A concentrated load will arise from the prestressing reinforcement at such points, which depends on the angular change of the prestressing reinforcement profile as well as the prestressing force.

The concentrated load in the point of buckling is given as:

$$K = P \cdot \sin\theta$$

While small angles give the simplified expression:

$$K = P \cdot \theta$$

Where

P is the prestressed force

θ is the total angle of the change in direction, $\theta_A + \theta_B$, from figure 19

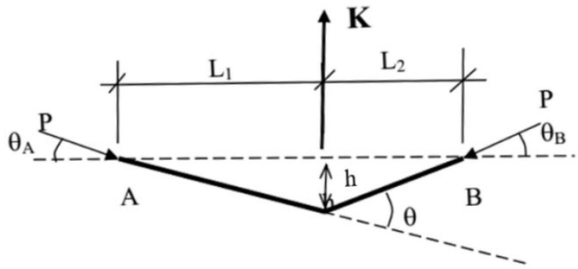

Figure 19 – Forces in buckling points for straight prestressed cables (Sørensen, 2013)

2.7 Hyperstatic moments

For simple structures that are statically determined and prestressed, there is moments from the prestressing tensile forces. These forces can be multiplied with the eccentricity of the prestressed reinforcement with respect to the concrete element, and create forced moments. This can not always be assumed, as for static indeterminate structures where the deformations will be retained by the bearings. (Sørensen, 2013) The hyperstatic moments are given as:

$$M_p = M_0 + M_1$$

Where

M_0 is the primary moment given by the eccentricity multiplied by the prestressed force

M_1 is the coercive moment due to retention forces from bearings

2.8 Loss of prestressed forces

While implementing prestressed reinforcement, there are several reasons for the tension force to not continuously stay at the same level as during jacking of the cables. The forces will be somewhat reduced in relation to the state of tension during jacking. This is called loss of prestressed forces, and it is distinguished between immediate and time-dependent losses. For post-tensioned structures, it is mainly a difference in loss of strain and time-dependent losses that causes a loss of tension force in the cables. (Sørensen, 2013)

2.8.1 Loss of strain

It is several reasons for a loss in strain difference to occur. It can happen when there is little to no adhesion between the prestressed steel and the concrete, and is also a consequence of losses while locking, friction loss and also a loss due to difference in temperature. The prestressed cables are usually anchored by using wedge locks, and during the jacking of the cables, there will be possibilities for a slip before the steel is locked in place at the active end. This is what is defined as losses while locking, and the magnitude of the slip is normally in the order of a few millimeters. (Sørensen, 2013) (Spennteknikk, 2011) This is illustrated in figure 20 where the lack of adhesion between the concrete and the steel creates a slip.

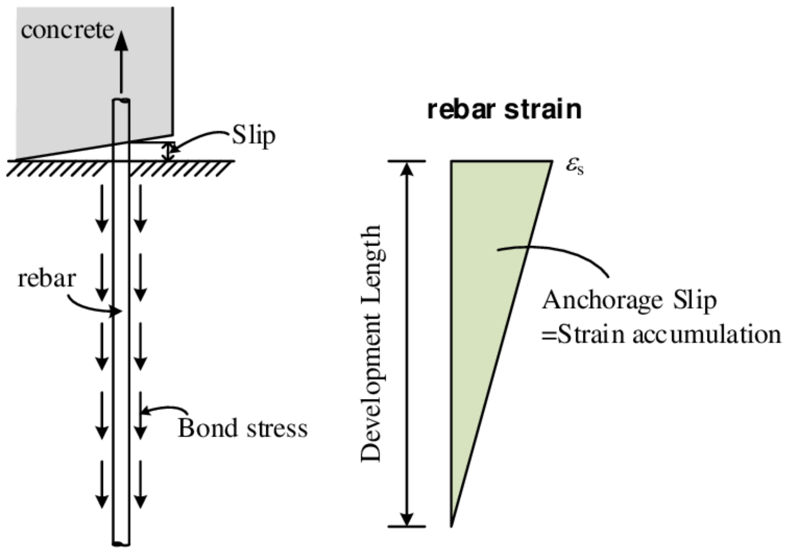

Figure 20 – Illustration of anchorage slip and strain accumulation (W. H. Pan, 2017)

When the prestressed reinforcement layout is with a curvature, a pressure will arise from the prestressed cables towards the duct in which it lays. This leads to frictional forces in the interface between them and contributes to the loss of strain difference between the concrete and the prestressing reinforcement. This is referred to as friction loss and depends on the coefficient of friction between the steel and the duct. This type of friction loss will be small at the active anchoring end and increase during the length of the cable. It should be ensured that the coefficient of friction is as small as possible, and to not have more curvature in the cables than needed. Jacking from both ends with great force and subsequent slacking of the cables will also result in lower friction losses. (Sørensen, 2013)

Reinforcement with significantly lower friction loss can be simple unbonded prestressed cables with grease mass.

2.8.2 Loss due to elastic deformation of concrete

Deformation of the concrete during tensioning leads to a reduction of the prestressed force. When the prestressing cables are tensioned successively, the prestressing of the individual unit will lead to further deformation in the concrete and thus a loss of prestressing force for already prestressed and adjacent cables. The first prestressing reinforcement unit thus suffers the greatest tension force loss, while the last unit suffers zero losses. (Sørensen, 2013)

2.8.3 Time-dependent losses

Both shrinkage and creep are long-term effects that cause the deformation in the concrete to increase beyond the instantaneous deformation during the tensioning. Further deformation of the concrete leads to a reduction in strain in the prestressing steel. Furthermore, prestressing steel subjected to constant strain will for a long time be subjected to a stress drop, called relaxation, which further reduces the tensile force. (Meierhofer, 2018)

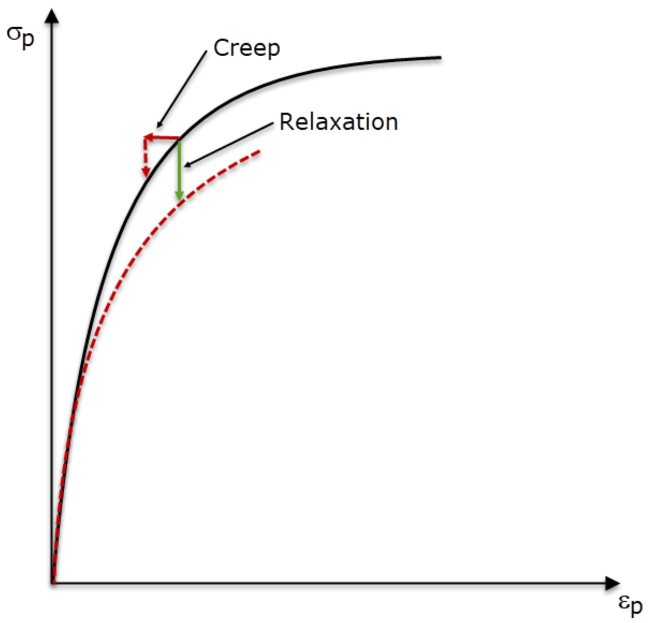

Figure 21 – Illustration of creep and relaxation stress loss in post-tensioned cable (Meierhofer, 2018)

PART II - REGULATIONS AND DESIGN BASIS

3 Materials

The structural design of the footbridge is based on a similar bridge in Alicante made with ultra-high-performance concrete. That bridge is designed by a team of engineers based in Valencia and has a very innovative material solution with the English terminology; *Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC).* (Lopez, et al., 2015) By using this type of structural material together with prestressed reinforcement and traditional reinforcement, they made a long span of 43m possible with very slim dimensions and little material used. The superior properties of this concrete compared to conventional concrete allows for several beneficial factors:

- Reduction of concrete sections and consequent weight reduction of structural elements between 60-75%
- Decrease in the need for lifting equipment in addition to savings in substructures
- Elimination of shear reinforcement and geometric minimums, with considerable savings in the rebar manufacturing process
- Reduction of the minimum coatings and greater slenderness of solutions
- Greater use of prestressing and streamlining of the prefabrication processes

(Lopez, et al., 2015)

The bridge design of this study case is made of several different parts of concrete elements. All elements are made with self-compressing fiber reinforced concrete, and some crucial elements are also reinforced with prestressed reinforcement. All elements have a varying amount of traditional reinforcement, some only needing the minimum, while other elements are subdued to larger amounts of shear and tension and will subsequently need extra reinforcement.

In designing the bridge, the implementation of this concrete is done out of interest to learn and experiment with such a solution. The possibilities of design in the world of civil engineering can be positively affected by becoming more proficient with this composite concrete.

3.1 Concrete

The thesis is based on the *Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC)*. The UHPFRC is an innovative and expensive material with a great compression stress capacity of more than 150 MPa. It has a high ductility, which is conferred by steel fibers, and a high flowability due to powder materials and high range water reducer in its mix. The high prize of this material can be justified by the development of new structural systems which take fully advantage of the material properties. That is only possible by understanding its mechanical behavior and by adapting its design to more efficient cross-sections and details.

Precast and prestressed applications represent the greatest potential for UHPFRC since a high quality control can be developed and prestressing can be made after casting to improve tension capacity. (Angel Lopez, et al., 2012)

With this type of concrete it is a huge range of possibilities in civil engineering that could not be imagined using traditional reinforced or prestressed concrete. It is the first in the world of its kind. Because of this, there has been discussions on the allowance of formulas from the new draft of EN1992-1-1 being used for this type of concrete, as the standard only covers concretes up to $f_{ck} = 100\ MPa$. The concretes strength is therefore kept at this value for the sake of the thesis. Table 1 displays the concrete properties.

Concrete properties	Symbol	Value
Characteristic strength	f_{ck}	100 MPa
Characteristic tensile strength	$f_{ctk0.05}$	3,6 MPa
Middle axial tension strength	f_{ctm}	5,1 MPa
Modulus of elasticity	E_{cm}	45 GPa
Material factor	γ_c	1,5
Load coefficient	α_{cc}	0,85
Dimensioning strength	f_{cd}	56,7 MPa

Table 1 – Concrete properties

3.2 Passive reinforcement

In this thesis it is chosen a traditional B500NC reinforcement to be used. All values are retrieved from EN1992-1-1. Table 2 presents the reinforcement material properties.

Reinforcement properties	Symbol	Value
Characteristic yield strength	f_{yk}	500 MPa
Material factor	γ_s	1,15
Dimensioning strength	f_{yd}	435 Mpa
Modulus of elasticity	E_s	210 GPa

Table 2 – Passive reinforcement properties

3.3 Prestressed reinforcement

The prestressed cables in this thesis are chosen to be Y1860S7 cables to satisfy the structural analysis. All values are retrieved from EN1992-1-1. Table 3 presents the prestressed reinforcement properties.

Reinforcement properties	Symbol	Value
Characteristic tensile strength	f_{pk}	1860 MPa
Characteristic proof strength	$f_{pk0.1k}$	1650 MPa
Modulus of elasticity	E_s	195 GPa
Material factor	γ_s	1,15
Dimensioning strength	f_{pd}	1435 MPa
Max stress	$\sigma_{p,max}$	1485 MPa
Cable diameter	$Ø_p$	16 mm
Cable area	A_p	150 mm^2

Table 3 – Prestressed reinforcement properties

3.4 Fiber reinforcement

The fiber reinforcement chosen for the design of the bridge in this thesis is inspired by (Lopez, et al., 2015), and will have an amount of approximately 78 kg/m^3 fibers in the concrete. Lengths of fibers should be between 40-60mm according to NB38. Table 4 presents the fiber reinforcement properties.

Fiber properties	Symbol	Value
Characteristic residual strength	$f_{R.1k}$	8 MPa
Characteristic residual strength dependent on ductility	$f_{R.3k}$	8 MPa
Fiber material factor	γ_f	1,5
Strength in ULS	f_{Ftuk}	2,96 MPa
Strength in SLS	f_{Ftsk}	3,6 MPa
Dimensioning strength	f_{Ftud}	1,97 MPa
Volume factor (some elements)	K_G	1,2
Dimensioning strength for moment	$f_{Ftud.m}$	2,36 MPa

Table 4 – Fiber reinforcement properties

4 Loads

The design loads are determined from *Eurocode 1: Actions on structures* (EC1) and consists of self-weight of concrete, pedestrian traffic, and wind load, and are verified by the Norwegian Public Road Administrations handbook for bridge design. The combination of these loads and the design criteria is determined from *Eurocode 0: Basis of structural design* (EC0). Design basis for fiber reinforced concrete structures is according to *NB38: Fiber-reinforced concrete in load bearing structures* and *Eurocode 2: Design of concrete structures Annex L.*

4.1 Wind load

The wind load is determined in the 4th part of EC1 (NS-EN 1991-1-4). It is necessary to find the local reference wind speed, determine the roughness factor, and calculate both the basis wind speed and the turbulence intensity. It will then be possible to determine the acting wind load on the structure. The bridge is to be placed in Bergen, and the load is calculated in attachment F.

The peak wind velocity pressure acting on a structure is given by the equation:

$$q_p(z) = \left[1 + 2 \cdot k_p \cdot I_v(z)\right] \cdot 0{,}5 \cdot \rho \cdot v_m(z)^2$$

Where

k_p	is the peak factor which is set to 3,5
ρ	is the density of the air dependent on altitude, temperature and barometric pressure which can be expected in an area with powerful wind gust. Normally set to the value of 1.25 kg/m^3
$I_v(z)$	is the air turbulence intensity dependent on the height
$v_m(z)$	is the local mean wind velocity dependent on the height

4.1.1 Local mean wind velocity

The wind velocity at a certain height z is given by the basis wind speed multiplied by two factors. These factors are the terrain roughness factor and a factor for the terrain shape. The wind velocity is given by the equation:

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$$

Where

$c_r(z)$ is the terrain roughness factor.

$c_0(z)$ is the terrain shape factor given in NA.4.3.3(901.1)

v_b is the basis wind speed

Terrain roughness factor

Equation 4.4 in NS-EN 1991-1-4, 4.3.2 defines the roughness factor as one of the following:

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \quad \text{for} \quad z_{min} \leq z \leq z_{max}$$

$$c_r(z) = c_r(z_{min}) \quad \text{for} \quad z \leq z_{min}$$

Where the different terrain categories are given in table 5 The values for the parameters k_r, z_0 and z_{min} are given in table NA.4.1 in EC1 where z_{max} is set to 200m.

Category nr	Terrain category	z_0 [m]	z_{min} [m]
0	Sea or open coastal area exposed to the open sea	0.003	1
I	Lakes or flat and horizontal are with negligible vegetation and without obstacles	0.01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m	1.0	10

Table 5 – Terrain categories

For the purpose of the thesis, the terrain category **III** will be used.

Basis wind speed

The basis wind speed is given in the Norwegian standard by equation NA.4.1:

$$v_b = c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b,0}$$

Where the different factors can all be conservatively set to the value of 1,0.

Turbulence intensity

Turbulence intensity is defined as the ratio of standard deviation of fluctuating wind velocity to the mean wind speed, and it represents the intensity of wind velocity fluctuation (K.Kimura, 2016). It is given by the following formula from NS-EN 1991-1-4:

$$I_v(z) = \frac{K_I}{c_0(z) \cdot ln\left(\frac{z}{z_0}\right)}$$

Where

K_I is the turbulence factor. Can be set to 1,0 if nothing else is given in NA.4.3.3(901.3.2) or NA.4.3.3(901.4)

4.1.2 Wind actions on bridges

The following figure is displaying how the coordinate system is used in the Eurocode.

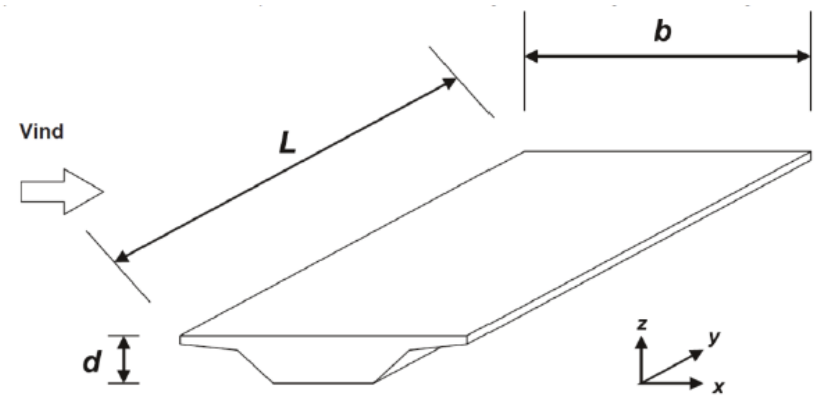

Figure 22 – Directions for wind actions on bridges (Norwegian standard, 2009)

When calculating the total wind force it is needed to divide it into three components; the horizontal force in the x direction ($F_{w,x}$), the horizontal force in the y direction ($F_{w,y}$), and the vertical force in the z-direction ($F_{w,z}$). EC1 instructs how to calculate these:

$$F_{w,x} = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C \cdot d_{tot}$$

$$F_{w,z} = q_p(z) \cdot c_{f,z} \cdot b$$

$$F_{w,y} = 0{,}25 \cdot F_{w,x}$$

Where

C	$C_e \cdot C_{f,x}$
C_e	$\frac{q_p(z)}{q_b} = \frac{q_p(z)}{0{,}5 \cdot \rho \cdot v_b^2}$
d_{tot}	is the depth of the cross section given in EC1 table 8.1
b	is the width of cross section
$c_{f,x}$	is the force factor in the x direction. Given in figure 8.3 in EC1-4
$c_{f,z}$	is the force factor in the z direction. Recommended value is $\pm 0{,}9$

Wind actions happening at the same time as traffic loading is normally neglected when designing for vehicle loads, but is here thought to be acting simultaneously as the pedestrian traffic. The Norwegian Public Roads Administration recommends a lower maximum wind velocity to be acting on a bridge structure at the same time as traffic loading, but this will be conservatively neglected.

4.2 Traffic load

The structure in question is a pedestrian walkway bridge, and the induced loading will accordingly be based on pedestrian traffic. The worst case of this type of loading is a uniform movement of the pedestrians, which will induce a dynamic amplification of the surface load. This load is set according to NS-EN 1991-2 as:

$$q_{t,k} = 5{,}0 \, {}^{kN}/_{m^2}$$

It is the dynamic loading that induces the vibration of the bridge with a known frequency. A common rule of thumb when dimensioning for vibrations from walking, running and the like is that the resonance frequency of the deck should be higher than twice the maximum of the load frequency. For pedestrian traffic, this means a frequency higher than 5.2 Hz.

4.3 Characteristic loads

It is mainly four loads that are included in the calculations: Deadload, live traffic load, wind load and prestressing. For bridges, the snow load is assumed to not act simultaneously as the traffic load and is therefore neglected.

The characteristic value of the deadload is found by using the density of reinforced concrete as indicated in EC1-1-1 table A.1. The values are assumed to be the same for fiber reinforced concrete. The surface load of the deck is then:

$$g_k = t \cdot \rho_c = t \cdot 25 \, {}^{kN}\!/_{m^3}$$

The live traffic load is based on pedestrian traffic. The worst case of this type of loading is a uniform movement of the pedestrians, which will induce a dynamic amplification of the surface load. This load is set according to NS-EN 1991-2 as:

$$q_{t,k} = 5{,}0 \, {}^{kN}\!/_{m^2}$$

The wind load is defined in chapter 4.1 and calculated in attachment F, and will have the characteristic values:

$$F_{w,x} = 1{,}88 \, {}^{kN}\!/_{m}$$

$$F_{w,y} = 0{,}47 \, {}^{kN}\!/_{m}$$

$$F_{w,z} = 1{,}48 \, {}^{kN}\!/_{m}$$

5 Design criteria

5.1 Ultimate limit state (ULS)

In general, the ultimate limit state should be considered according to EC0 for strength, stability, fatigue and geotechnical loading. In this thesis it is only the strength criteria that is controlled, where it is defined in EC0 as; Fractures or excessive deformations in the structure or parts of the structure, including foundations, piles, basement walls and the like, where the firmness of the building materials is important. Consideration of the ultimate limit state is done according to EC0 6.4.3.2(3) equation 6.10a and 6.10b:

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \gamma_p P + \gamma_{Q,1}\psi_{0,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Qi}\psi_{0,i} \cdot Q_{k,i}$$

$$\sum_{j\geq 1} \xi_j\, \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Qi}\psi_{0,i} \cdot Q_{k,i}$$

Where

P	is the prestress force
G	is the deadload
Q	is a variable load
γ	is the partial factor that considers the uncertainties of the loads

- 1,35/1,20 for permanent unfavorable loads
- 1,00 for irreversible unfavorable deformation loads (ex. creep)
- 1,60 for unfavorable wind loads
- 1,2 for unfavorable temperature loads (neglected)
- 1,5 for other unfavorable variable loads

ξ	= 0,89 and is a reduction-factor for the deadload according to EC0 NA.A2.4(B)
ψ_0	is a combination factor for variable effect of live loads. Set to 0,7 for bridges. This is due to the unlikeliness for all loads to act simultaneously.

Where the post-tensioning cables will be designed to counteract parts of this load.

For bridges, the global equilibrium and stability (EQU) is important. This condition is independent of material strength or the capacity of the structure. Exceeding this capacity can lead to instantaneous collapse of the entire structure. This check is important in the construction phase of a bridge.

The designed bridge for this thesis is a single span construction with focus only on the construction methods FRC and post-tensioning, and is assumed to be in sufficient global equilibrium for its entirety.

Load combinations are implemented from the Eurocode as the following:

Loads	Unfavorable combination factor	Favorable combination factor
Dead load	1,20 / 1,35	0,89
Wind load	1,60	1,00
Other variable loads	1,50	1,00
Irreversible unfavorable deformation loads (ex. creep)	1,00	1,00
Prestressing/Post-tensioning	0,90	1,10

Table 6 – Load combinations (Standard Norge, 2016)

Where the unfavorable combination factors are the ones used in the design load combination.

5.2 Serviceability limit state (SLS)

In the serviceability limit state, it must be checked that the construction meets the requirements associated with its use and purpose. According to EC0 6.5.3, it must be demonstrated that a characteristic combination, a permanent combination, and a quasi-permanent combination are satisfied. In NB38, it must be checked that bending moments can be supported by traditional slack or prestressing reinforcement without contribution from the fiber reinforcement. This applies to fiber reinforced structures where a collapse can lead to loss of human life or cause large social costs. In this check, the characteristic load combination is to be used, and all load and material factors are set equal to 1,0.

Characteristic load combination is given in EC0 6.5.3 equation (6.14a) as:

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i\geq 1} \psi_{0,i} \cdot Q_{k,i}$$

5.2.1 Time dependent effects

Concrete that is loaded will, like other materials, be subjected to stretching.

What distinguishes concrete from other materials is that concrete will continue to stretch beyond the instantaneous stretching. This strain is called creep. While shrinkage causes contraction in the concrete and as caused by drying of the concrete, creep can be either contraction or expansion in the concrete. If the concrete experiences tensile stresses, creep will lead to expansion in the concrete, and conversely if the concrete is subjected to compressive stresses. The creep number is a coefficient that takes into account the creep in the concrete.

This is included as time dependent loads in FEM-Design for serviceability limit state, and included in calculations for second order degree biaxial bending and axial compression of slender elements. Creep contribution is done according to EN1992-1-1 7.4.2. Shrinkage as a load is reviewed in chapter 10.1.7.

5.2.2 Crack widths

For the calculation of crack widths in the concrete, there is much to be earned from the contribution of fibers. Fibers works very well at both crack spacing and crack widths in the concrete together with conventional reinforcement, and even moderate amounts of fiber can limit the extent of cracking to a relatively large extent so that the concrete will be virtually free of visible cracks at the serviceability limit state. (Kanstad, et al., 2020)

Crack width calculation

Requirements for crack widths for the various exposure classes given in NS-EN 1992-1-1 Table NA.7.1N applies. In combination with traditional bar reinforcement, the crack width w_k on the concrete surface can be calculated in a similar way as in NS-EN 1992-1-1 with correction for the fibers contribution. In general:

$$w_k = s_{r,max,cal}(\varepsilon_{sm} - \varepsilon_{cm})$$

Where

$s_{r,max,cal}$	is the distance between cracks in a stabilized cracking pattern, or the largest force attack length in the area close to the first generated crack
ε_{sm}	is the middle strain in the traditional reinforcement including losses
ε_{cm}	$= \sigma_{sr}/E_s$ is the middle strain in the concrete between cracks equal to ε_{sm}

The strain difference $\varepsilon_{sm} - \varepsilon_{cm}$ is calculated by:

$$\varepsilon_{sm} - \varepsilon_{cm} = max\left(\frac{\sigma_s - \sigma_{sr}}{E_s}; \quad 0{,}6\frac{\sigma_s}{E_s}\right)$$

Where σ_s is the strain in the longitudinal reinforcement, and σ_{sr} is given by:

$$\sigma_{sr} = k_t \cdot \frac{f_{ctm}}{\rho_{c,ef}} \cdot \left(1 + \frac{E_s}{E_{cm}}\rho_{c,ef}\right)$$

Where

k_t is a factor dependent on the duration of the load, and is set to 0,6 for short term loading and 0,4 for long term

f_{ctm} is the middle uniaxial tensile strength of the concrete (NS-EN 1992-1-1 Tab. 3.1)

E_s is the modulus of elasticity of reinforcement steel set to 210 000 MPa

E_{cm} is the concretes middle modulus of elasticity set to 45 000 MPa in this thesis

$\rho_{c,ef}$ $= {A_s}/{A_{c,eff}}$ and is the ratio of reinforcement in the effective tension area of the concrete

The effective tension area of the concrete, $A_{c,eff}$, is given by:

$$A_{c,eff} = b \cdot h_{c,eff}$$

Where

b is the width of the cross section

$h_{c,eff}$ is the effective height of concrete in tension, and calculated by:

$$h_{c,eff} = \min\left(c + 5 \cdot Ø_s, 10 \cdot Ø_{s,} \ 3{,}5 \cdot c\right)$$

Where $Ø_s$ would be changed with $Ø_p$ for the largest diameter.

While calculating the stresses in the reinforcement, the effect of the fibers should be considered. This can be done by using a lamella method. A simplified solution is chosen for this thesis. (Kanstad, et al., 2020) (Standard, 2021)

The largest distance between cracks is given by:

$$s_{r,max,cal} = \left(2c + 0{,}35\, k_b \cdot \frac{\emptyset}{\rho_{c,ef}}\right) \cdot \left(1 - \frac{f_{Fts,ef}}{f_{ctm}}\right)$$

Where

c is the concrete cover

k_b is a coefficient similar to k_1 in EN1992-1-1 that takes into account the fastening properties to the reinforcement. This value is equal to 0,8 for good fastening and 1,6 for smooth surface bars

Ø diameter of reinforcement bars

$f_{Fts,ef}$ is the uniaxial residual tensile strength for serviceability limit state specified with $k_0 = 1{,}0$ for parts induced by bending moment

Crack width limit

The limit for crack widths are given in the following table from EN1992-1-1 Table 9.2(NDP) where it is verifications for stress and crack width limits for a structural elements durability.

<table>
<tr><th rowspan="3">Exposure Class</th><th colspan="2">Reinforced members; prestressed members with unbonded tendons and with bonded tendons with Protection Levels 2 or 3 according to 5.4.1(3)</th><th colspan="3">Prestressed members with bonded tendons with Protection Level 1 according to 5.4.1(3) and pretensioned elements.</th></tr>
<tr><th colspan="2">combination of actions</th><th colspan="3">combination of actions</th></tr>
<tr><th>quasi-permanent</th><th>characteristic</th><th>quasi-permanent</th><th>frequent</th><th>characteristic</th></tr>
<tr><td>X0, XC1</td><td>–</td><td rowspan="2">–</td><td>–</td><td>$w_{lim,cal} =$ 0,2 mm $\cdot\, k_{surf}$</td><td rowspan="2">–</td></tr>
<tr><td>XC2, XC3, XC4</td><td rowspan="2">$w_{lim,cal} =$ 0,3 mm $\cdot\, k_{surf}$</td><td>Decom-pression [b]</td><td>$w_{lim,cal} =$ 0,2 mm $\cdot\, k_{surf}$</td></tr>
<tr><td>XD1, XD2, XD3
XS1, XS2, XS3</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td><td rowspan="2">–</td><td rowspan="2">Decompression[b]</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td></tr>
<tr><td>XF1, XF3
XF2, XF4</td><td>–</td></tr>
</table>

Table 7 – Verifications, stress and crack width limits for durability (Standard, 2021)

The first row of limits applies to reinforced members and prestressed members with unbonded or bonded tendons. For a quasi-permanent combination of actions in structural elements with exposure class XC2-XC4, XD1-XD3 or XS1-XS3, the following formula is used to calculate the crack width limit:

$$w_{lim,cal} = 0{,}3\ mm \cdot k_{surf}$$

Where the factor k_{surf} considers the difference between an increased crack width at the member surface and the required mean crack width in according to durability performance of the minimum cover. The factor is given by:

$$1{,}0 \leq k_{surf} = \frac{c_{act}}{10\ mm + c_{min,dur}} \leq 1{,}5$$

Where

c_{act}	is the specified actual cover
$c_{min,dur}$	is the minimum cover due to design lifetime

Including prestressed reinforcement

In order to include the prestressed reinforcement in controlling the crack width formation, EN1992-1-1 9.2.2 (5) is applied. The ratio of reinforcement in the effective tension area of the concrete is adjusted to include the prestressed reinforcement. This is done by calculating the adjusted ratio of bond strength considering the different diameters of prestressing and traditional reinforcing steel:

$$\xi_1 = \xi \cdot \frac{\text{Ø}_s}{\text{Ø}_p}$$

Where

ξ	is the ratio of bond strength between traditional and prestressed reinforcement. Suggested values given in table 10.1 from EN1992-1-1 and set to 0,6.
Ø_s	is the diameter of the traditional reinforcement bars
Ø_p	is the diameter of the prestressed reinforcement cables

The adjusted ratio of steel and concrete tension area is then:

$$\rho_{c,ef} = \frac{A_{s,eff} + \xi_1 \cdot n_p \cdot A_p}{A_{c,eff}}$$

Where

$A_{s,eff}$	is the effective traditional reinforcement area
n_p	is the number of prestressed cables in an element
A_p	is the area of a single prestressed cable
$A_{c,eff}$	is given in chapter for crack width calculation

The working tensile stresses are now divided by all working reinforcement, instead of only the traditional reinforcement as done in normal crack width calculation.

6 Prestressed reinforcement

6.1 Prestressed force

The greatest jacking force at the active end during jacking is according to EC:2004 5.10.2.1 given as:

$$P_{max} = A_p \cdot \sigma_{p,max}$$

Or when the jacking force can be measured with an accuracy of $\pm 5\%$ of the final prestressed force, there can be included an extra jacking force. The maximum jacking force could then be increased to:

$$P_{max} = k_3 \cdot A_p \cdot f_{p0,1k}$$

Where

A_p	is the cross-section area of the prestressed cable
$\sigma_{p,max}$	is the largest stress applied to the prestressed cable $= \min\left(k_1 \cdot f_{pk};\quad k_2 \cdot f_{p0,1k}\right)$
f_{pk}	is the characteristic tensile strength of the prestressed cable
$f_{p0,1k}$	is the characteristic ''0,1% limit'' of the prestressed cable

The values for k_1, k_2 and k_3 is found in the national annex, with the respective values of 0.8, 0.9 and 0.95. In general, there are no major differences for procedure with prestressed force between EC2:2020 and EC2:2004.

6.2 Additional stress for unbonded prestressed reinforcement

For post-tensioned constructions with un-injected cables, there is no adhesion between the concrete and the prestressed reinforcement. It should therefore be considered that the deformation of the entire structural part will result in an increase in stress. The additional stress is thus dependent on the length difference in the prestressed cable, and the tension is distributed evenly over the entire length of the cable, not considering the local effects from frictional forces. Given that the cables are in constant tension, the additional stress can according to EC2:2004 NA.5.10.8 be defined as:

$$\Delta\sigma_{p,ULS} = 100\ MPa$$

This is a simplification, and a more accurate value for the additional stress can be found by integrating the concretes strain along the prestressed cable, ε_c, so that the difference in length is given as:

$$\Delta L = \int_L \Delta\varepsilon_c\big(x, e(x)\big)dx$$

Thus, the additional stress is given as:

$$\Delta\sigma_{p,ULS} = \Delta\varepsilon_p E_p = \frac{\Delta L}{L} E_p$$

6.3 Effective strain

When jacking the cables there is an initial strain difference, ε_{p0}, in the prestressed reinforcement that is given as:

$$\varepsilon_{p0} = \frac{\sigma_{p,max}}{E_p}$$

Where

E_p is the modulus of elasticity in the prestressed reinforcement

The initial strain difference will be reduced as a result of the tension force loss. By considering the loss of the reinforcements strain, the effective strain difference can be used for calculation in both serviceability limit state and ultimate limit state. The effective strain difference is given as:

$$\varepsilon'_{p0} = \varepsilon_{p0} - \Delta\varepsilon_{loss}$$

Where $\Delta\varepsilon_{loss}$ is the reduction of the strain due to friction, locking, creep, receding concrete and relaxation.

6.4 Prestressed reinforcement for moment capacity

The prestressing reinforcement can be regarded as an internal resistance or an external load while calculating the moment capacity in ultimate limit state. While computing the tension state of the prestressed reinforcement, it is normal to calculate the jacking force as an external load. After tensioning, the reinforcement will be in the tensile zone and it is regarded as an internal resistance.

Taking into account the deformation of the entire structure during the ultimate limit state, the prestressing force is given as:

$$S_p = A_p(\varepsilon'_{p0} \cdot E_p + \Delta\sigma_{p,ULS}) \cdot \frac{1}{\gamma_s}$$

Where

A_p	is the total area of prestressed reinforcement
ε'_{p0}	is the effective strain difference
E_p	the modulus of elasticity for prestressed strands
$\Delta\sigma_{p,ULS}$	is the additional stress because of deformations

(Kanstad, et al., 2020) (Standard, 2021)

7 Fiber reinforcement

While dimensioning fiber reinforced concrete, the residual tensile strength is an important factor. This is the tensile strength of the cross section after cracking in fiber reinforced concrete. The magnitude of the residual tensile strength depends on the volume proportion of fibers, the type of fibers used, the concrete quality and the orientation of the fibers.

The residual flexural tensile strength of the fiber concrete can be determined by several different standardized tests depending on which regulations are followed. The recommendations according to the Norwegian Concrete Association publication no.38 [NB38], the Annex L of the EN1992-1-1 draft, as well as (Lopez, et al., 2015) and (di Prisco, et al., 2022) are consulted for the assessment of fiber reinforcement.

7.1 Calculating the residual strength

A way of determining the residual bending tensile strength of the fiber concrete, is with the Norwegian 3-point method. This is done by a standardized beam test where the bending moments for four predetermined crack widths, also called CMOD (crack mouth opening displacement), is examined. This is done assuming a linear stress distribution over the cross-sectional height (resistance moment for cross-section of linear elastic material). This does not correspond to the actual stress distribution after cracking, therefore this material parameter is often characterized as a fictitious strength and is not used directly in dimensioning. (Kanstad, et al., 2020)

Figure 23 displays the test setup with the beam dimension and load situation. The beam length can vary by $550mm \leq L \leq 700mm$. The four specified crack widths examined in the standardized test are 0.5mm, 1.5mm, 2.5mm and 3.5mm, where the cracks are measured at the lower edge in the middle of the beam. This will be the area most afflicted by cracks.

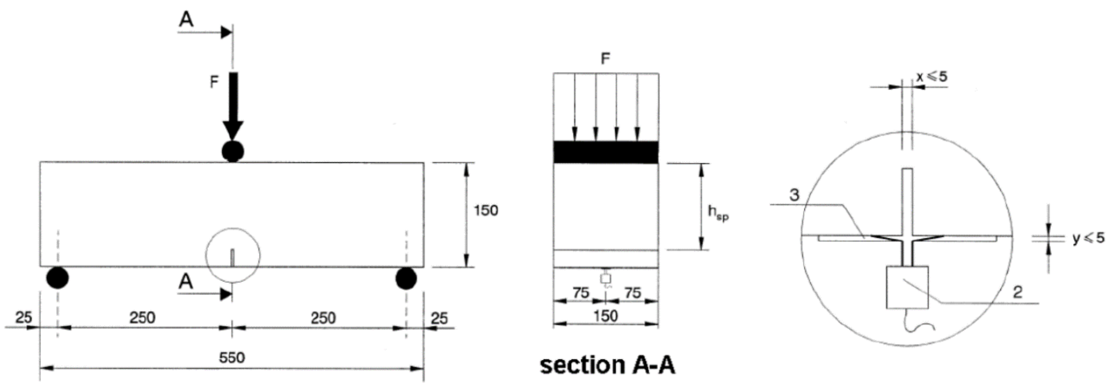

Figure 23 – Test setup of 3-point method (Kanstad, et al., 2020)

Figure 24 illustrates a typical relationship between applied load and crack width at different fiber dosages. This test is described in more detail in NS-EN 14651, and is best suited for concrete with a maximum aggregate size of 34 mm, as well as a fiber length of no more than 60 mm.

By assuming a linear stress distribution over the cross-section height, it is possible to measure the residual strength as:

$$f_{R,i} = \frac{6M_{Ri}}{bh_{sp}^2}$$

Where

M_{Ri} is the moment caused by deflection for the testing of specified crack widths (CMOD) and is equal to $F_{Ri} \cdot \frac{L}{4}$

Where F_{Ri} is the loading during specified crack widths

h_{sp} is the height from the top of the element to the start of the crack. Shown in figure 23

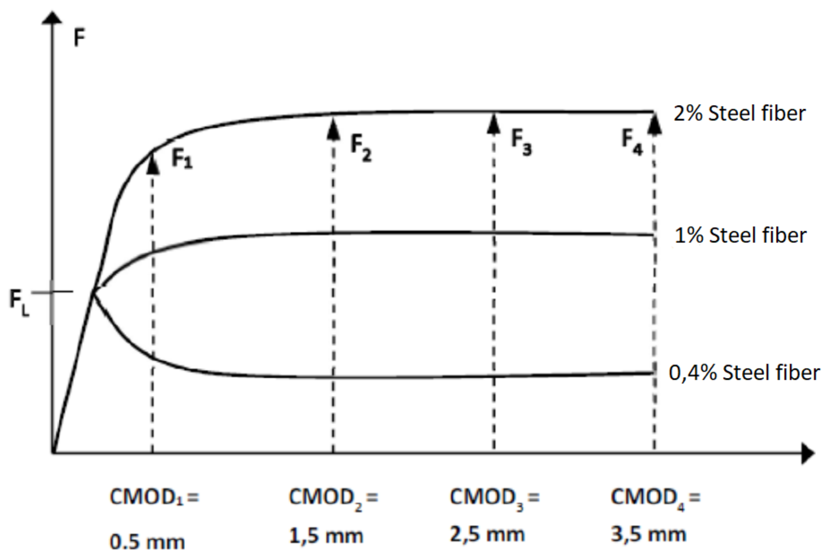

Figure 24 – Force and crack width for different fiber dosages (Kanstad, et al., 2020)

Characteristic values in the 0,05 quantile for residual strength is after recommendations from NB38 given as:

$$f_{Rk,i} = f_{R,i} - k \cdot s$$

Where

s is the standard deviation of the test series

k equal to 1.7 if the test is carried out according to NS-EN1465 and NB38

Residual strength of fiber reinforced concrete could also be determined from the German more detailed method according to DafStb. It is however chosen based on NB38.

7.2 Classification [EN1992-1-1 ANNEX L & NB38]

In the publication NB38 and annex L of the EN1992-1-1 standard, a fiber-reinforced concrete is characterized by a residual strength class and a ductility class. Characteristic residual flexural tensile strength, $f_{R.1k}$, is given for a 0.5mm crack width and indicates the residual strength class of the fiber concrete. The ductility class is determined by residual flexural tensile strength for a 2.5mm crack width, $f_{R.3k}$. Table 8 shows an overview of current residual strength classes together with ductility classes, where the ductility to the fiber concrete increases from class a to e. With this system, a concrete identified as R4.0d will have a residual strength class 4.0 and a ductility class c.

Ductility classes	Characteristic residual flexural strength $f_{R,1k}$									Analytical formulae
	1,0	1,5	2,0	2,5	3,0	4,0	5,0	6,0	8,0	
a	0,5	0,8	1,0	1,3	1,5	2,0	2,5	3,0	4,0	$f_{R,3k} = 0{,}5 f_{R,1k}$
b	0,7	1,1	1,4	1,8	2,1	2,8	3,5	4,2	5,6	$f_{R,3k} = 0{,}7 f_{R,1k}$
c	0,9	1,4	1,8	2,3	2,7	3,6	4,5	5,4	7,2	$f_{R,3k} = 0{,}9 f_{R,1k}$
d	1,1	1,7	2,2	2,8	3,3	4,4	5,5	6,6	8,8	$f_{R,3k} = 1{,}1 f_{R,1k}$
e	1,3	2,0	2,6	3,3	3,9	5,2	6,5	7,8	10,4	$f_{R,3k} = 1{,}3 f_{R,1k}$

Table 8 – Residual strength classes for SFRC (Standard, 2021)

The characteristic residual flexural tensile strength will in general depend on the steel fiber content, and figure 25 shows an example of determining the fiber content required to achieve a specific residual flexural tensile strength. This is done by interpolating between documented test results. The advantage of this classification is that design engineers can operate according to residual strength classes when designing, and in that way describe and communicate more easily with the concrete supplier in that the fiber dosage does not need to be known in advance.

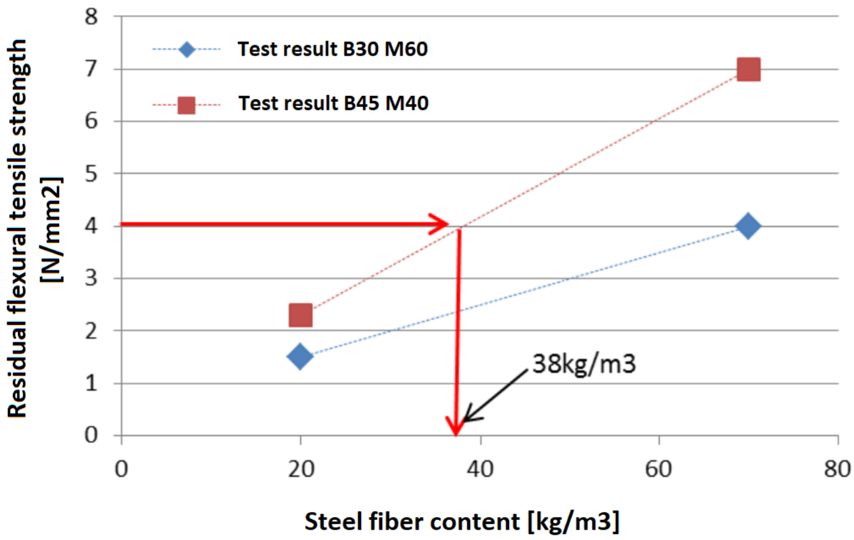

Figure 25 – Residual strength for different concretes and steel fiber contents (Kanstad, et al., 2020)

According to (Lopez, et al., 2015) it was found that to achieve a high enough residual strength of 8 N/mm^2 in the high strength concrete, a steel fiber content of 78 kg/m^3 was needed. In the example in the figure above, it is shown that to achieve a residual strength of 4 N/mm^2, a steel fiber content of 38 kg/m^3 is necessary. (Kanstad, et al., 2020)

NB38 4.1 only allows the use of up to 60% of the average residual strength, $f_{R,1m}$ and $f_{R,3m}$ to not achieve too favorable results in the beam tests. This is due to a smaller predictability in the distribution of strength in the concrete while fiber reinforced. The value of the mean strength is assumed to be known, and the calculated residual flexural tensile strengths are thus:

$$f_{R,1kdes} = \min\left(f_{R,1k},\ \ 0.6 \cdot f_{R,1m}\right)$$

$$f_{R,3kdes} = \min\left(f_{R,3k},\ \ 0.6 \cdot f_{R,3m}\right)$$

When determining the fiber reinforced concretes residual strength for bending, $f_{R,1k}$ and $f_{R,3k}$ is calculated based on the assumption of a linear stress distribution during bending moment. This may not correspond to the real behavior of the cross section, which leads to an inaccuracy in calculations and thus making the residual strength not eligible for direct calculation. While dimensioning in ultimate or serviceability limit state, an ideal plastic stress distribution is used with a conversion factor for both states to conservatively normalize these values. (Kanstad, et al., 2020) (Standard, 2021)

The uniaxial residual tensile strengths are according to NB38 (4-2a) and (4-2b) given as:

Serviceability limit state $\quad f_{Ftsk} = 0.45\ f_{R,1kdes}$

Ultimate limit state $\quad f_{Ftuk} = 0.37\ f_{R,3kdes}$

To take into account the orientation of the fibers in the concrete, there is introduced a factor k_0. When multiplying this factor with the residual strength, you get the effective residual strength.

The factor k_0 is often set equal to 1.0 or 0.5 depending on the construction part. For horizontal construction elements such as flat slabs, the value is set to 1.0, while for vertical elements such as walls and columns, the value is set to 0.5. The effective residual tensile strengths are thus determined from NB38 (4-3a) and (4-3b) as:

$$f_{Fts,ef} = k_0 \cdot f_{Ftsk}$$

$$f_{Ftu,ef} = k_0 \cdot f_{Ftuk}$$

According to NB38 5.8, this factor can be set equal to 1.2 for slim plate elements. This increase in effective residual strength can only be introduced if the following criteria is held:

$$t/l_f < 3.5$$

Where t and l_f are respectively the thickness and the length of the fiber elements.

The dimensioning strength of the fibers in the concrete is then according to NB38 (4-4) and EN1992-1-1 L.5.5.1 (2):

$$f_{Ftud} = \frac{f_{Ftu,ef}}{\gamma_f}$$

Where the material factor for fiber γ_f is the same as for concrete and set to the value of 1.5.

8 Capacities in structural analysis

All capacities in this chapter are calculated according to chapter 5.1: Design criteria for ultimate limit state except chapter 8.3 which is according to the serviceability limit state for long term effects.

8.1 Bending moment and axial force

By using fiber-reinforced concrete (FRC), parts of the concrete cross section can take tensile forces after cracking. The tensile zone can be characterized in a simplified manner by a uniform stress distribution with stress corresponding to the dimensioning residual tensile strength, f_{Ftud}.

Capacity for bending moment and axial force can be determined by assuming that the plane cross section remains plane after deformation, and that the pressure zone of the FRC and the stress and strain of the conventional reinforcement tensile properties are as given in NS-EN 1992-1-1 sections 3.1.7 and 3.2.7. For cross-sections exposed to pure tension, the strains in the FRC must be less than 3/h‰ (cross-sectional height h in meter). Correspondingly, the tensile stresses shall be limited to 3/h‰ in the tensile edge for cross-section subjected to bending. (Kanstad, et al., 2020)

If the axial equilibrium results in the maximum tensile stress exceeding 3/h‰, it is sufficient to set the compressive stress equal to ϵ_{cu3} and the maximum tensile stress equal to 3/h‰.

Alternatively, for example, lamella methods based on the principles given above can be used.

8.2 Moment capacity of fiber and prestressed reinforced elements

The moment capacity shall be determined based on the following principles from NB38:

1. It must be demonstrated that the structural part carries the dimensioning load with cooperation between bar reinforcement and steel fiber reinforcement.
2. The working diagram of the conventional reinforcement is assumed to follow the guidelines in EC2 point 3.2.7.
3. The pressure zone of the concrete shall be characterized by the provisions given in NS-EN 1992-1-1 section 3.1.7.
4. The tensile capacity of the FRC can be included as shown in figure 26, with constant stress above the stretch zone height.

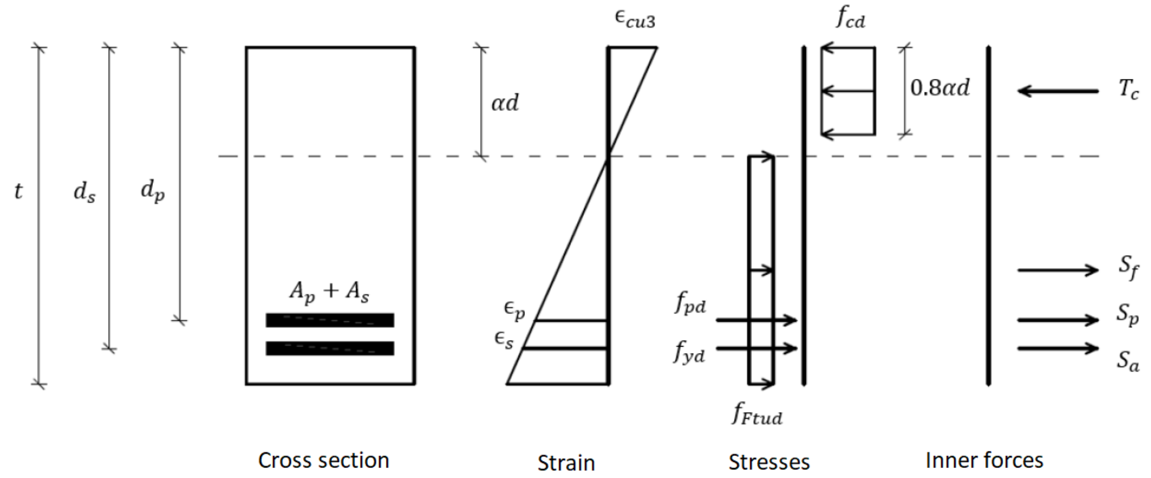

Figure 26 – Stress and strain distribution for a general rectangular cross-section with fiber, passive and prestressed reinforcement (Olsen & Arntsen, 2021)

When calculating the capacity, the pressure zone height shall be determined by means of axial equilibrium:

$$T_c = S_f + S_a + S_p$$

Where

T_c	is the compression resultant force
S_a	is the tension resultant force form the traditional reinforcement
S_p	is the tension resultant force from the prestressed reinforcement

The height of the compression zone is given by the following formula

$$\alpha d = \frac{hbf_{Ftud} + A_s f_{yd} + S_p}{0{,}8bf_{cd} + bf_{Ftud}}$$

The moment capacity can then be determined by balancing the pressure resultant point of attack as:

$$M_{Rd} = S_f(0{,}5h + 0{,}1\alpha d) + S_a(d_s - 0{,}4\alpha d) + S_p(d_p - 0{,}4\alpha d)$$

According to NB38, for constructions with a risk of collapse which can lead to a large social impact and economic expense or loss of human life, the bending moments and axial forces in characteristic combination can be supported by the traditional reinforcement or the prestressing reinforcement without the contribution from fiber reinforcement. For this control, all load and material factors can be set equal to 1,0. This control is in addition to the usual controls in the ultimate limit state.

For the bridge deck and the bottom tensioned chords the moment capacity is considered as ordinary one way slabs. The width of the tension and compression zones are simplified calculated as half the span. Where it is present prestressed reinforcement, the moment capacity is given as:

$$M_{Rd} = M_{Rd,fiber} + M_{Rd,trad} + M_{Rd,pre}$$

Where the contribution from the traditional reinforcement, $M_{Rd,trad}$, is normally determined by the amount of minimum reinforcement. There can in some cases be necessary with more than the minimum amount, like in the support walls. The effect of the fiber reinforcement on the moment capacity is included according to the guidelines from NB38, where the fiber reinforcement has a constant stress distribution in the tension zone.

Because of the elements being statically indeterminate, the dimensioning residual strength of the fiber reinforcement can be multiplied by a factor for volume effect, K_G, according to NB38. The factor is dependent on the area of the tension zone of the cross-section, where the tension zone is dependent on the assumed value of volume effect. In this way, determining the value of K_G turns into an iterative process. The area of the tension zone for one way slabs is very large, and the maximum beneficial volume effect is achieved. The volume effect could then be put as:

$$K_G = 1{,}0 + 0{,}5 \cdot A_{ct} \leq K_{G,max} = 1{,}5$$

Where A_{ct} is the area of the tension zone (in m^2).

8.3 Bi-axial moment combined with axial force

For some elements that are induced by a great compression force while also being twisted in both directions, it is necessary for a comprehensive check on the biaxial capacity. At first there is analyzed if the structural member is a slender compressive chord, where the following criteria must be satisfied:

$$\lambda_n = \frac{\lambda \cdot \sqrt{\frac{n}{1 + 2 \cdot w}}}{A\varphi} \leq \lambda_{lim,simpl} = \frac{10{,}8}{\sqrt{n}}$$

Where

λ is the slenderness ratio for each direction,

$$\lambda = \frac{L_{cr}}{\sqrt{\frac{I}{A}}}$$

n is the axial utilization of the cross section,

$$n = \frac{N_{Ed}}{f_{cd} \cdot A}$$

w is the mechanical reinforcement ratio for each direction,

$$w = \frac{A_{s,eff} \cdot f_{yd}}{f_{cd} \cdot A}$$

$A\varphi$ is a factor for reduced area because of creep, with φ being the creep factor:

$$A\varphi = \frac{1{,}25}{(1 + 0{,}2 \cdot \varphi)}$$

This criterion is not satisfied for the longitudinal top bar of the bridge, nor the lateral trusses on each side. Thus, it is necessary to check for biaxial moment and axial force capacities for both axes.

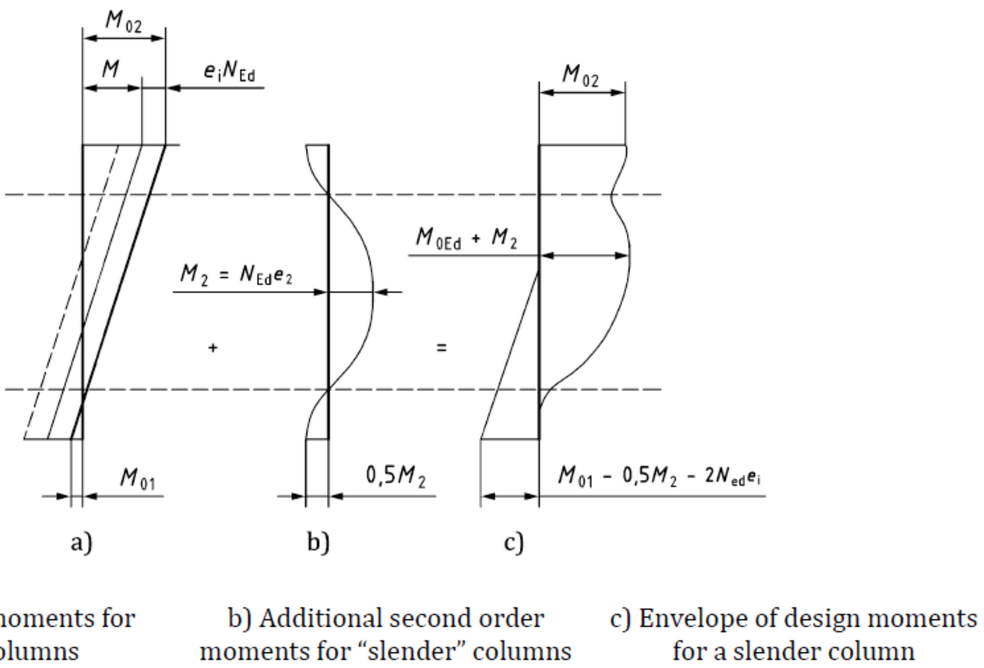

Figure 27 – Second order moments for slender columns (Standard, 2021)

Figure 27 b) shows the second order moments for slender columns that is included in the calculation of these elements. The second order bending moments are found by multiplying the design axial compression with the eccentricities on both axes. The eccentricities are found with the following formula:

$$e_2 = \frac{1}{r} \cdot \frac{L_{cr}^2}{C}$$

Where

r is the nominal curvature for each axis,

$$r = \frac{r_0}{k_r \cdot k_\varphi}$$

r_0 is the base value for curvature,

$$r_0 = \frac{0{,}45 \cdot d}{\varepsilon_{yd}}$$

k_r is a coefficient depending on the axial load

k_φ is a coefficient accounting for creep

L_{cr} is the critical buckling length

C is a factor depending on the total curvature distribution and is equal to π^2

This eccentricity is then multiplied by the dimensioning axial load in serviceability limit state. In both cases of this calculation the eccentricity moment is a great deal larger than the already induced bending moments, pointing to the importance of considering second order moments. The total inflicted moment about each axis is then the bending moment with the added eccentricity moment.

Total bending moment capacity for each axis is defined as:

$$M_{Rd} = m \cdot f_{cd} \cdot A \cdot h$$

Where

m is the moment utilization from the moment and axial force relation curve diagram

Further, the axial utilization later used for computing the combination factor, is given by:

$$u = \frac{N_{Ed}}{A_{s,eff} \cdot f_{yd} + (A - A_{s,eff}) \cdot f_{cd}}$$

Combination factor for combining biaxial moments:

$$a = 1 + \frac{1{,}5 - 1}{0{,}7 - 0{,}1} \cdot u$$

Utilization of bi-axial moment capacity, without including fiber reinforcement:

$$\left(\frac{M_{tot,y}}{M_{Rd,y}}\right)^{a} + \left(\frac{M_{tot,z}}{M_{Rd,z}}\right)^{a}$$

With the calculation of the moment capacity for second order moments being mostly based of the traditional reinforcement needed in the elements, the fiber reinforcements contribution was not needed for satisfying capacities.

The top compressive chord is dependent on its critical buckling length not reaching higher values. This can be done by having sufficient stiffness in the lateral truss and the transverse deck beams together with the deck itself. Figure 28 presents this very case from (Angel Lopez, et al., 2012), where the first buckling mode is the desirable one.

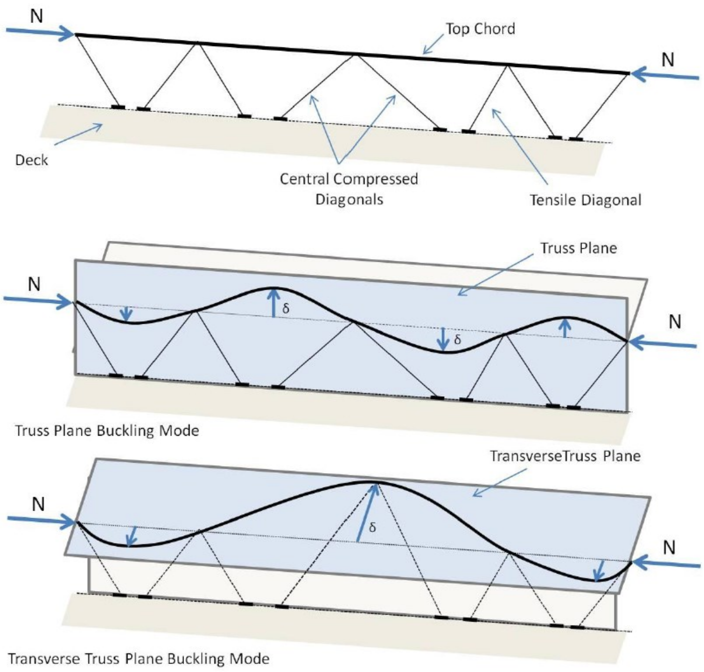

Figure 28 – Transverse truss plane buckling mode (Angel Lopez, et al., 2012)

8.4 Shear force capacity

It has been thoroughly documented in experimental trials that steel fiber provides increased capacity against shear fractures, while it has not been documented that synthetic fibers have a similar effect.

Furthermore, the rules for shear capacity applies to beams, rods, plates, and shells where the ratio of span to cross-sectional height is at least 3 for two-sided constructions and 1.5 for cantilevered parts.

There are numerous methods and models used for calculating shear capacity of fiber reinforced concrete elements. Most of these are based on results from various beam test series with conventional flexural tensile reinforcement in the lower edge of the beam. The main longitudinal traditional reinforcement. (Kanstad, et al., 2020)

The shear force is calculated using stresses, for ease of use. The dimensioning shear stress is given by:

$$\tau_{Ed} = \frac{V_{Ed}}{b_w \cdot z}$$

Where

V_{Ed}	is the dimensioning shear force acting on the member
b_w	is the cross sections minimum web width
z	is the internal moment arm

While calculating the shear force capacity, it is possible to include shear force reinforcement or not. This is in turn the difference between the two shear force capacities; $\tau_{Rdc,F}$ and $\tau_{Rds,F}$, where if $\tau_{Rdc,F}$ is greater than τ_{Ed} then there is no need for shear force reinforcement.

8.4.1 Elements with no dimensioning necessity for shear force reinforcement

The shear capacity is calculated from two factors:

τ_{Rdc}	The concretes shear force capacity, included contribution from longitudinal reinforcement
f_{Ftud}	The fiber reinforced concretes dimensioning residual strength as indicated earlier. Provided that $k_0 = 1,0$.

The concretes shear force capacity is never assumed to be less than the minimum value of:

$$\tau_{Rdc,min} = \frac{10}{\gamma_c}\sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}}$$

Where d_{dg} is a value to take care of any unevenness in the shear fracture zone. This value is set to the diameter of the longitudinal reinforcement + $D_{lower} \leq 40$ for concrete with $f_{ck} < 60\ MPa$.

For concrete with $f_{ck} > 60\ MPa$ the value is set to the diameter + $D_{lower}\left(\frac{60}{f_{ck}}\right)^2$.

The value for D_{lower} is the minimum allowable value of D_{max}, and is here defined as 20.

d is the value of the effective cross-sectional height.

(Kanstad, et al., 2020)

The shear force capacity of the fiber reinforced concrete element is determined as:

$$\tau_{Rd,cF} = \eta \cdot \tau_{Rd,c} + f_{Ftud} > \eta \cdot \tau_{Rd,c,min} + f_{Ftud}$$

Where

$$\tau_{Rd,c} = \frac{0,6}{\gamma_c}\left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{1/3}, \qquad \rho_l = \frac{A_{s,eff}}{b \cdot d}$$

$$\eta = \max\left(\frac{1}{1+0,43 f_{Ftud}{}^{2,85}};\quad 0,4\right)$$

8.4.2 Elements with dimensioning necessity for shear force reinforcement

The shear capacity is calculated from two factors:

$\tau_{Rd,s}$	The capacity of the implemented shear force reinforcement
f_{Ftud}	The fiber reinforced concretes dimensioning residual strength as indicated earlier. Provided that k_0 = 1,0.

The total shear force capacity for shear reinforced fiber concrete is calculated as:

$$\tau_{Rd,sF} = 0,75 \cdot \tau_{Rd,s} + f_{Ftud}$$

Where

$\tau_{Rd,s} = \frac{A_{sv}}{s} \cdot \frac{z}{bd} \cdot f_{yd}$ (Assumed a crack angle of 45°)

The factor of 0,75 is introduced to reduce the contribution of the shear reinforcement. This is due to the fiber reinforced concrete reaching maximum utilization by lesser deformations than a fully strained shear reinforcement. The internal moment arm z can simplified be put to 0,9d. (Kanstad, et al., 2020)

8.4.3 Shear connection

In the design of the bridge, there is longitudinal beams which goes through the whole length of the bridge and connects the transverse smaller beams that carries the bridge walkway deck. These longitudinal beams are cast inside the lateral diagonal truss, in such a way that the downwards forces are transmitted as tension and compression in the truss elements. There is not a significant shear force interface between these elements. However, the transverse beams will need to be cast with shear force reinforcement connecting them to the longitudinal beam and lateral truss. This reinforcement is needed due to the forces coming from the walkway deck that is transmitted throughout the transverse beam pattern. These beams are however spaced close and the forces in each are not great.

The design value of the interface shear force is given as:

$$\tau_{Ed} = \frac{V_{Ed}}{A_i}$$

Where

V_{Ed} is the dimensioning shear force acting on the interface

A_i is the area of the interface. Here, area of transverse deck beam cross section (120x150)

The formula for shear force capacity in the interface can be solved for the necessary ratio of reinforcement in the cross section, which is then given as:

$$\rho_i = \frac{\tau_{Ed} - C_{v1} \cdot \frac{\sqrt{f_{ck}}}{\gamma_c}}{f_{yd}(\mu_v \cdot \sin(\alpha) + \cos(\alpha))}$$

Where

C_{v1} and μ_v are factors dependent on the roughness of the surface (EN1992-1-1 Table 8.1)

The present reinforcement in the transverse beams can be anchored in the perpendicular concrete elements and create a satisfactory shear force reinforcement for the induced shear force in ultimate limit state according to EN1992-1-1 8.2.6.

9 Minimum reinforcement

There are several reasons to lay minimum reinforcement in structures, and according to EC2:2020 12.2 a minimum reinforcement shall be added to:

a) Ensure distributed cracking and to handle forces from restrained deformations where not considered explicitly in the design
b) Ensure sufficient deformation capacity to contribute to structural robustness by allowing alternative load paths
c) Avoid failures due to unpredicted cracking
d) Ensure applicability of design models
e) Ensure constructability

The area of minimum reinforcement can include prestressed and ordinary reinforcement when bonded to the concrete.

9.1 Minimum reinforcement for fiber reinforced structures [NB38]

Depending on the load and type of construction, fiber reinforcement can be used in a combination with traditional reinforcement. This is done to achieve sufficient ductility in a load-bearing structure. The amount of reinforcement must be large enough for it to be a crack-distributing effect (fastening), so that the cracked cross-section is stronger than the non-cracked section. If a solution with only fiber reinforced concrete is used, a high dosage of fibers is required, often higher than the wanted volume percentage of steel fiber. With such high amounts of fiber, it becomes both difficult to cast and expensive, and is thus not commonly the most used solution. NB38 recommends all structures to have a minimum amount of traditional reinforcement, which is a very good solution especially with requirements for small crack widths. The steel fibers take over some of the tension in the traditional reinforcement, which gives less reinforcement strain and thus smaller crack widths. (Kanstad, et al., 2020)

The requirement for minimum reinforcement for slabs is based on the same principle as for beams, only that it applies to both x and y directions. For cross-sections only subjected to bending, the main reinforcement and continuous minimum reinforcement shall correspond to:

$$A_{s,min} \cdot f_{yk} = 0{,}26 \cdot (f_{ctm} - 2{,}15 \cdot f_{Ftu,ef}) \cdot b_t \cdot d > 0{,}13 \cdot f_{ctm} \cdot b_t \cdot d$$

In order to include shear reinforcement in the shear force capacity, this must have a cross-sectional are referred to in the plane of the plate at least equal to [mm^2/ mm^2]:

$$\rho_{w,min} = (0{,}1\sqrt{f_{ck}} - 0{,}3 f_{F_{tu,ef}})/f_{yk}$$

The distance between the reinforcement bars shall not be greater than eight times the cross-sectional thickness and not over 1,2m.

9.2 Minimum reinforcement according to NS-EN1992-1-1:2004

In the Norwegian national annex NA9.2.1.1(1), the smallest area of reinforcement should not be less than:

$$A_{s,min} = max\begin{cases} 0{,}26 \cdot \dfrac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d \\ 0{,}0013 \cdot b_t \cdot d \end{cases}$$

Where b_t is the width of the zone in tension.

9.3 Reinforcement rules for prestressed reinforcement

The rules of minimum reinforcement in the previous points are applied to the prestressed members as well, implementing a distributed traditional reinforcement.

PART III – GEOMETRY AND ANALYSIS

10 Analysis

10.1 FEM-Design

For computing load cases on the construction, FEM-Design is used. FEM-Design is an advanced and intuitive 3D structural design and analysis software. It supports all aspects of structural engineering through 3D modelling, design and finite element analysis of concrete, steel, timber, composite, masonry and foundation structures, according to the Eurocode. The quick and easy nature of FEM-Design makes it ideal for all types of construction tasks from single element design to global stability analysis of large structures and makes it a good practical tool for structural engineers. FEM-Design will also be able to include the post-tensioning cables. (StruSoft, 2019)

10.1.1 Bridge calculation model

The bridge design in this thesis includes a total span of 35m with max height of 2m and lowest height at 1.4m. The total width is 2.62m with a free walkway width of 2.38m. It is composed by several different structural elements. The structure is computed in FEM-Design as shown in the following figure.

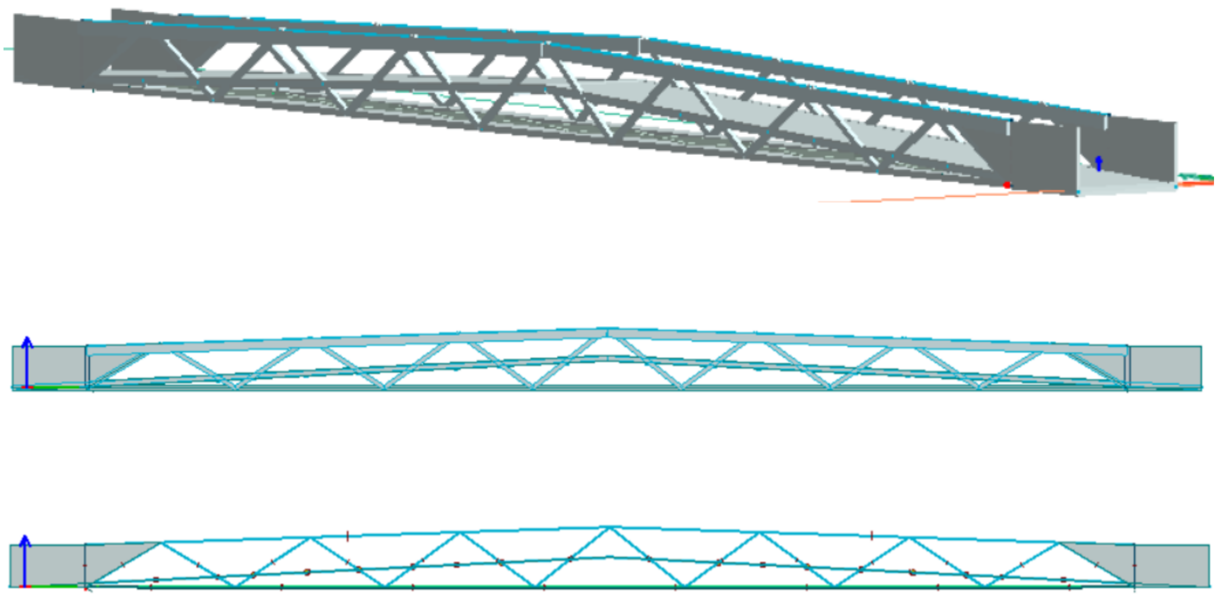

Figure 29 – Computational model from FEM-Design

An important aspect of the structural design is the elevation of the walkway deck. This is done to create horizontal stiffness in the bridge and reduce buckling lengths of the lateral truss as they are connected to the bridge deck. The lateral trusses are made with variable heights, ranging from 2.0m at the middle span down to 1.49m close to the supports. To de-load the lateral trusses close to the support, it is implemented support shear walls that extends diagonally into the bridge span by 4.5m on each side. These form the support system of the bridge, where it is assumed that the placement of the bridge ensures the forces from the supports to be taken by the ground around it.

The walkable bridge deck never goes below a 1.0m distance from the top compressive chord, to always assure a proper height of the handrail. The lateral trusses are connected at their lower part by a horizontal X-shaped truss. Both the lateral and the horizontal trusses have been lightened seeking a balance between the aesthetic component and the efficiency of the material.

The diagonals, both compressed and tensioned, that form the lateral trusses, have a rectangular section of 120x150mm. They are all reinforced with 4xØ20 bars.

The tensioned chords of the bottom part have a height of 150mm and a width of 600mm, each housing a total of 18 post tensioning cables of type Y1860S7. These chords form part of the lower X-shaped truss that connects both lateral trusses. The diagonals in the lower truss all have a section of 100x150mm. The diagonals are shown in green in figure 30, where the bridge is seen from below.

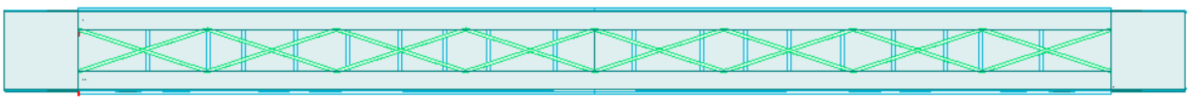

Figure 30 – FEM-Design model seen from below

The deck has a minimum clear width for pedestrian traffic of 2.38m. Along each lateral truss, there is longitudinal beams with a cross-section of 120x150mm that marks the layout of the bridge deck. Connecting these beams, is a series of transverse beams with cross-section 120x150mm. These are located at the intersection of the longitudinal beams with the diagonals of the lateral truss, and in between. The transverse ribs do in no case exceed a distance of 2.3m. These are shown in figure 31, where the bridge is seen from above.

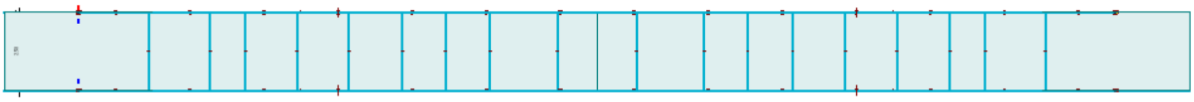

Figure 31 – FEM-Design model seen from above

The structure of longitudinal beams and transverse beams generates frames on which the 2,5 cm thick slab rests on in continuity, which constitutes the walkable surface of the deck. The longitudinal and transverse beams not only supports the slab, but also provides the necessary torsional rigidity to the structural assembly and reduces the buckling length of the lateral diagonals.

10.1.2 Elements

The bridge design is done with a goal of making elements with similar dimensions. This is to ensure an easier building and design process and economic benefits. All elements are designed in their own Mathcad notes attached to the thesis, with load situations retrieved from FEM-Design. No simplified load calculations have been done, and all induced loads are from the program. The main elements for withstanding loads and stabilizing the bridge are the top longitudinal bars and the lateral trusses. These elements are induced by the most axial forces in compression and tension. The support walls are also subdued to a great deal of compression and tension. The bottom strips are the only elements that have prestressed reinforcement, and the moment capacity is calculated using prestressed and traditional reinforcement together with fiber reinforcement.

The bridge is composed of the following elements in table 9 which are all visualized in the chapter 10.1.1 and attached as pictures to each calculation in the annex.

Element	Amount	Length	Cross section (b x h)
Top longitudinal bars	2	35m	(300x300) mm
Lateral truss diagonals	24	~2m	(120x150) mm
Transverse deck beams	18	2,62m	(120x150) mm
Longitudinal deck beams	4	17,5m	(120x150) mm
Bottom horizontal diagonals	16	4,5m	(120x150) mm
Walkway deck slab	2	17,5m	(2620x250) mm
Support walls	4	2,5m	(120x1400) mm
Post-tensioned strip	2	35m	(600x150) mm

Table 9 – All elements cross-sections and lengths

10.1.3 Prestressed reinforcement

The prestressed reinforcement is in the form of 18 strands of Y1860S7 cables in each strip, which are chosen to be injected for better fastening and better crack width properties. The cables are laid inside the chords and work the full length of the span with no sudden curvatures due to for example columns in mid span.

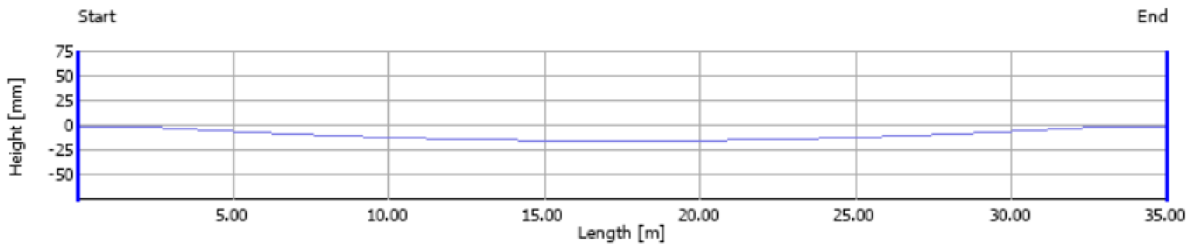

Figure 32 – Automatic post-tensioning cable profile from FEM-Design

This figure illustrates the prestressed reinforcement profile that FEM-Design automatically will use for the cables inside the 35m long element. They will have a very small curvature due to the low height of the elements and be close to straight cables, but as they are in a great number, the produced uplift force is of significant magnitude. This prestressed reinforcement profile is easily implemented on site, and will have no problems of anchorage on either side due to adjoining structures. The eccentricities of the prestressed cables are mainly due to necessary cover and the traditional reinforcement size.

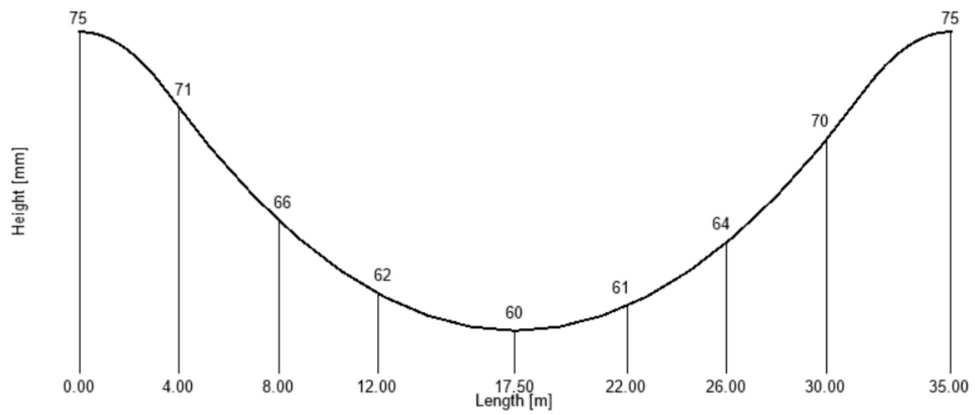

Figure 33 – FEM-Design automatic generated manufacturing drawing for production

10.1.4 Design of geometry

The design drawing for the bridge is made in Revit 2022 and presented in annex K. Revit is a complete BIM tool delivered by Autodesk for the construction industry, used by both architects and engineers. The program has many possibilities with precise 3D modelling, early phase design, visualization, engineering, analysis, fabrication, and construction. (focus, 2022) The design drawing includes the bridge cross section at start and middle of the span, as well as full side profile and a table of concrete volumes.

The drawings are characterized by a lack of time and skill, and will therefore miss an illustration of the reinforcement and a complete level of detail for a design of this type.

10.1.5 Implementation of loads

All loads are manually implemented to FEM-Design, and the following table presents the different load cases active in the analysis.

No.	Name	Type	Duration class (EN 1995 1-1)
1	Dead load	+Struc. dead load	Permanent
2	Wind load	Ordinary	Short-term
3	Pedestrian traffic	Ordinary	Medium-term
4	PTC T0	Post tensioning	Permanent
5	PTC T8	Post tensioning	Permanent
6	Shrink	+Shrinkage	Permanent

Table 10 – Load cases in FEM-Design

The post tensioning, the shrinkage and the self-weight of the materials are the only permanent load cases. Shrinkage is conservatively considered in the FEM analysis, even though the concrete data for such calculations would not be entirely correct due to the special case of ultra-high-performance concrete used in this thesis and the data related to shrinkage not being implemented in FEM-Design. Temperature loads are not included in this analysis.

PTC T0 and PTC T8

The prestressed cables equivalent loads are shown in the following figures, displaying how the forces will work against the induced loads on the bridge along the span. Both long and short term cable effects are considered, with T0 being short term and T8 being long term.

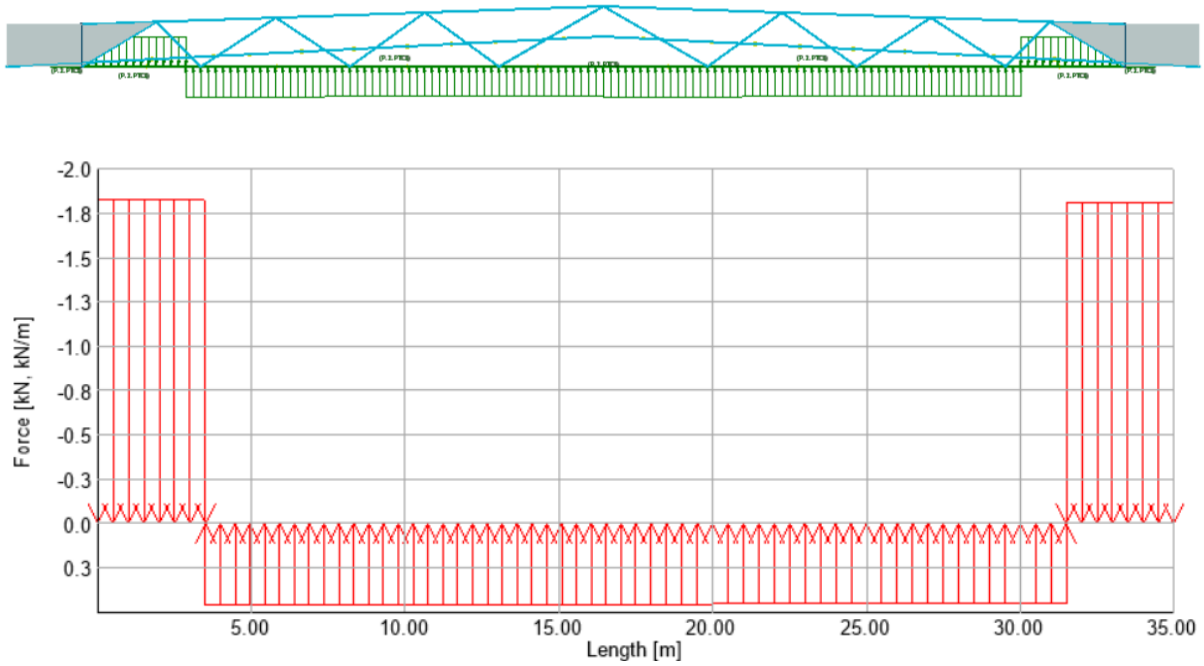

Figure 34 – Equivalent cable force FEM-Design

Pedestrian traffic load

The pedestrian traffic load is elevated to match the shape of the walkable surface of the bridge deck.

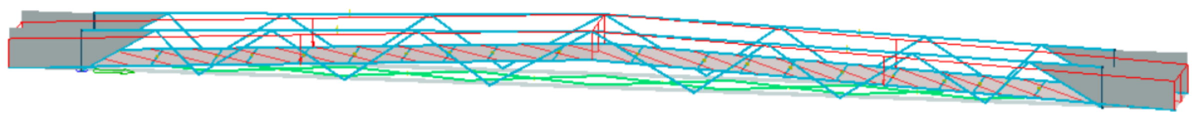

Figure 35 – Illustration of pedestrian traffic load as a uniform area load in FEM-Design

Wind load

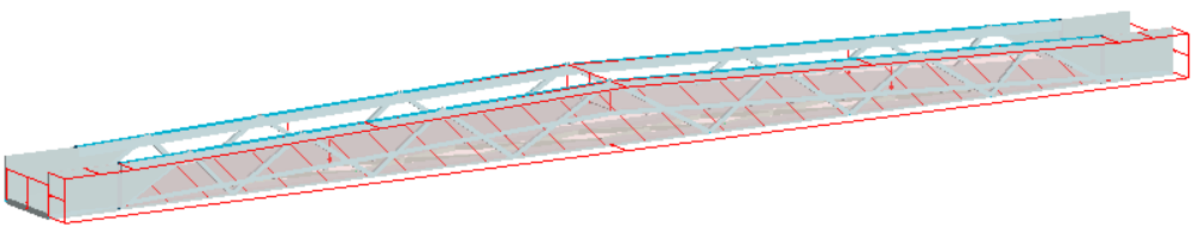

Figure 36 – Illustration of wind load as a uniform distributed load from three directions in FEM-Design

The wind load is a combination of three surface loads, putting pressure along all axes. The bridge being assumed to not be built in a great elevation will significantly reduce the values of the loads.

To ensure the whole load is being applied to the structural elements, the cover function in FEM-Design is used to appropriately distribute the surface loads.

10.1.6 Load combinations

The following table retrieved from FEM-Design presents the two load combination used in this analysis.

No.	Name	Type	Factor	Included load cases
1	ULS	U	1.20	Dead load
			1.60	Wind load
			1.50	Pedestrian traffic
			1.00	Shrink
			0.90	PTC T0
			0.90	PTC T8
2	SLS	Sq	1.00	Dead load
			1.00	Wind load
			1.00	Pedestrian traffic
			1.00	Shrink
			1.00	PTC T8
			1.00	PTC T0

Table 11 – Load combinations in FEM-Design (Ultimate limit state and Serviceability limit state)

Load combination ULS is used to compute all the sagging and hogging moments, together with axial and shear force in all elements. Reaction forces for both supports are also computed with this combination. SLS combination is used to compute the forces necessary for long term eccentricity moments in slender elements and crack formations throughout the structure. The whole bridge is calculated as a fixed one-way span structure, with no significant torsional moments.

10.1.7 Shrinkage in FEM-Design

When including the shrinkage effect in FEM-Design, it must be applied in combination with traditional reinforcement. The program adds the effect as an invisible load, computed from the formulas below.

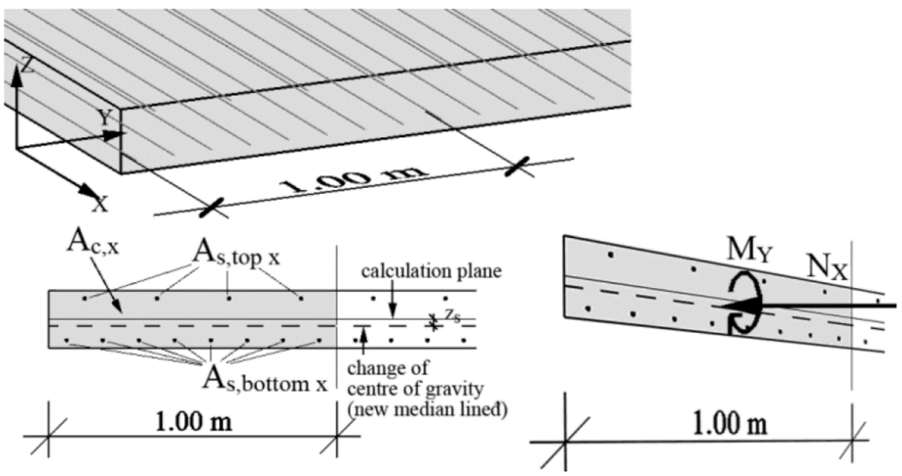

Figure 37 – Shrinkage effect on surface reinforcement bars in one direction (StruSoft, 2018)

Figure 37 illustrates the effect of the shrinkage for a surface reinforcement bars in one direction. Figure shows for reinforcement in x-direction but is also valid in other bar directions. The specific normal force causing the given shrinkage value in the concrete section is given by:

$$N_x = E_c \cdot A_c \cdot \varepsilon_{cs} \; [kN/m]$$

Where

E_c is the modulus of elasticity for the concrete (45 GPa)

A_c is the net area of concrete. Reinforcement area subtracted

ε_{cs} is the strain in the section given in ‰

The position change of the center of gravity considering reinforcement bars are given as:

$$z_s = \frac{n \cdot S_s}{A_c + n \cdot A_s}$$

Where

$n = E_s/E_c$ and E_s is the modulus of elasticity for steel (210 GPa)

S_s is the statical moment of x-direction bars around the y axis

A_s is the area of reinforcement steel

The change in center of gravity will cause an eccentricity induced moment about the y axis because of the normal force N_x, and is given as:

$$M_y = N_x \cdot z_s$$

The specific rotation (curvature) from M_y for a 1 meter wide section is given as:

$$\kappa_y = \frac{M_y}{E_c \cdot I_y}$$

Where

$$I_y = I_{c,y} + n \cdot I_{s,y} - z_s^2(A_c + n \cdot A_s)$$

Here, $I_{c,y}$ and $I_{s,y}$ are the moments of inertia for both the concrete section and the reinforcement steel.

10.1.8 Load results from FEM-Design

The following figures are displaying load results from the structural model in FEM-Design. It is an overview of each type of force, and not in detail for each element due to complexity of diagrams. Diagrams for all elements would be superficial as there are many neglectable load results. All forces from all attached calculations are also not included here due to the abundance of figures this would present. This chapter is for visualization and verification purposes.

Moment

Large peaks at support wall moments due to an increased size of diagrams to show all elements contribution.

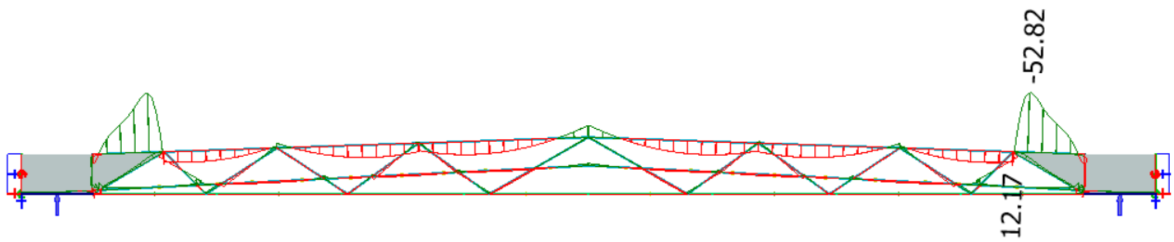

Figure 38 – Moment diagram for bars retrieved from FEM-Design

Axial force

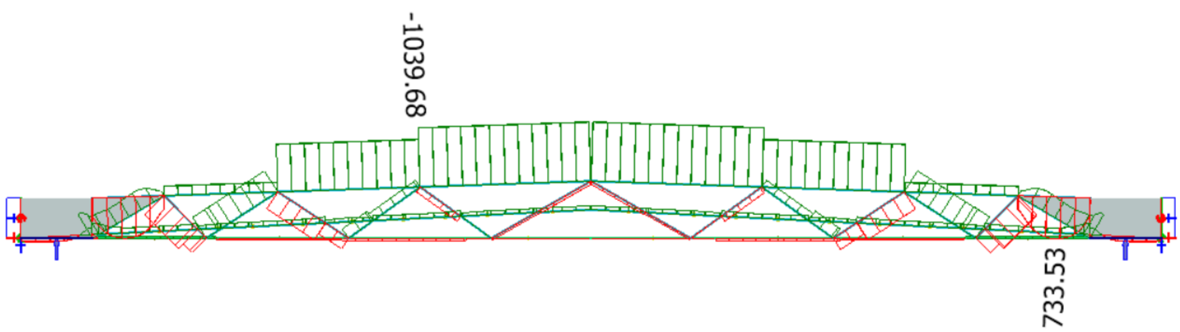

Figure 39 – Axial force diagram for bars retrieved from FEM-Design

Shear force

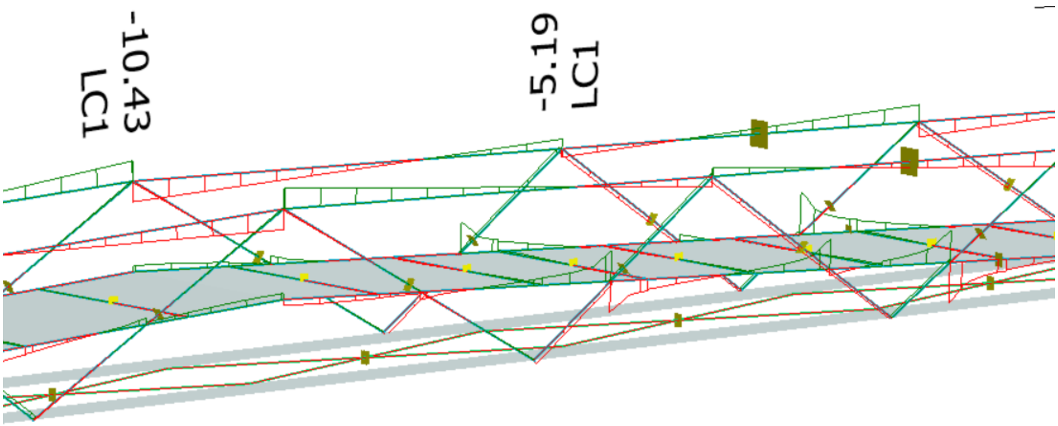

Figure 40 – Shear force diagram for bars retrieved from FEM-Design

11 Reinforcement design

All structural elements are made with the same fiber reinforced concrete, and all structural elements are following the NB38 equation for minimum reinforcement shown in chapter 9.1 of this thesis. For some elements subdued to a great deal of tension, own formulas for minimum reinforcement applies. There is no special cases of punching reinforcement or extra shear reinforcement being needed, except for the interface between the transverse beams along the deck slab and the longitudinal deck beams along the span. In that situation it was confirmed that an extended anchoring of the reinforcement bars in the transverse beams would serve as enough shear reinforcement for the connection forces. All bar elements have the same size stirrups with the same center distance for a simplified construction on site and to ensure the minimum reinforcement necessary to enclose the longitudinal reinforcement bars.

The following table presents the passive reinforcement chosen for all elements:

Element	Bar diameter	Amount (n / spacing)	Stirrup diameter	Amount (n / spacing)
Top longitudinal bars	Ø20	4	Ø8	cc300mm
Lateral truss diagonals	Ø20	4	Ø8	cc300mm
Transverse deck beams	Ø10	2	Ø8	cc300mm
Longitudinal deck beams	Ø10	2	Ø8	cc300mm
Bottom horizontal diagonals	Ø10	2	Ø8	cc300mm
Walkway deck slab	Ø16	cc250mm	-	-
Support walls	Ø24	cc200mm	-	-
Post-tensioned strip	Ø10	2	Ø8	cc300mm

Table 12 – Passive reinforcement in all structural elements

For the walkway deck slab and the support walls, the chosen reinforcement is laid in all directions. This is a simplified way of reassuring the minimum reinforcement in all directions, and a smaller crack width formation. By doing so, all induced forces are also accounted for.

The amount of reinforcement is low in general, with some elements needing further reinforcement. This leads to a high standardization of the construction process with similar reinforcement bars being used in several elements.

11.1 Prestressed reinforcement

For concrete parts with both prestressing and traditional reinforcement, the placement of the reinforcement can be complicated. This is especially true for parts over columns where there is very concentrated reinforcement, which is not the case here. While designing and planning, it is important to keep in mind possible complications during the construction phase and how to avoid it. The selected prestressing reinforcement system as shown in figure 41, leads to a simple design without complications or braiding of reinforcement. The prestressed cables are laid in bundles of three, for easier distribution and spacing. The figure presents the solution in the middle of the bridge span. During installation, the traditional reinforcement is first laid at the bottom of the cross-section, before the concentrated tension ropes are placed above it. After the tensioning cables have been placed, traditional reinforcement can be placed in excess in both directions if needed.

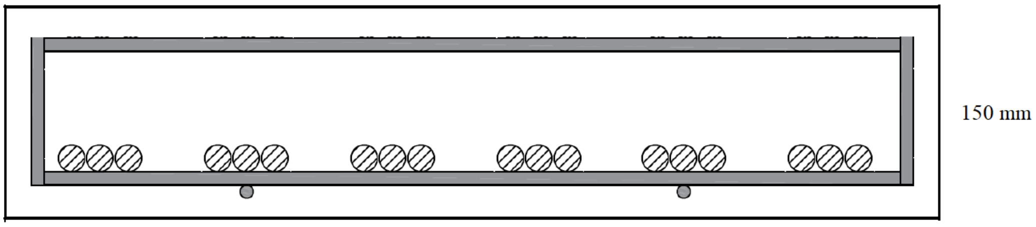

Figure 41 – Prestressed reinforced bottom chord element cross-section

11.1.1 Effective heights

The effective heights of the prestressed cables and traditional reinforcement in the middle span are given by:

$$d_p = t - c - \O_s - \frac{\O_p}{2}$$

$$d_s = t - c - \frac{\O_s}{2}$$

Where

t	is the thickness of the chord
c	is the concrete cover
$\O_s$	is the diameter of the traditional reinforcement bars
$\O_p$	is the diameter of the prestressed cables

These heights are then used to calculate each reinforcements contribution to the total moment capacity.

11.1.2 Anchorage stress distribution

The forces from the prestressed cables are assumed to spread at an angle of around 45 degrees inwards in the flat concrete chord, which is a good approximation of the distribution. (Almeida, 2022) The assumed load distribution for the selected prestressing reinforcement system is presented in figure 42, where the grey striped area is the area without compression.

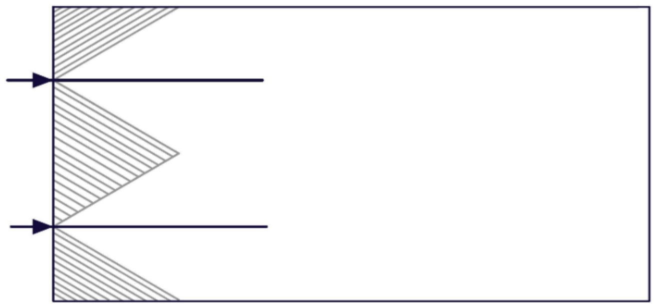

Figure 42 – Anchorage stress distribution, grey areas have no stress

This is only a representation of the effect with two points of force application. In the real post tensioned part there would be six points like this across the 600mm width. Since there is a great number of cables acting on a relatively small element, there will only be very small zones that would not receive compressive stress. The size of the zones will depend on the center distance of the post tensioning cables. It is assumed that the cables are distributed in such a way that the pressure is obtained along the entire edge.

11.1.3 Prestressing force

The clamping force in the prestressed reinforcement will be reduced somewhat in relation to the measured jacking force during the clamping due to different types of clamping force loss. A comprehensive calculation of these losses is omitted in this thesis, and it is instead assumed a total stress loss of about 15%. Based on the prestressed reinforcement properties from table 3, the total prestressing force in each cable is given as:

$$S_p = \frac{A_p}{\gamma_s} \cdot \left(\varepsilon'_{p0} \cdot E_p\right) = \frac{150\ mm^2}{1{,}15} \cdot \left(0{,}85 \cdot 7{,}631 \cdot 10^{-3} \cdot 195000 \frac{N}{mm^2}\right) = 165{,}0\ kN$$

Where all factors are explained in earlier prestressing chapter. For calculation of the moment capacities, an additional stress will be included with a simplified value from EN1992-1-1, $\Delta\sigma_{p,ULS} = 100\ MPa$. The prestressed force in each cable is then:

$$S_p = \frac{150\ mm^2}{1{,}15} \cdot \left(0{,}85 \cdot 7{,}631 \cdot 10^{-3} \cdot 195000 + 100 \frac{N}{mm^2}\right) = 178{,}0\ kN$$

PART IV – RESULTS

12 Capacity ratios

The following table presents the different capacity ratios of all structural elements. The empty fields are due to either neglectable load cases or no load case at all. All calculations are shown in annexes A to J. Crack width utilization is the current crack width divided by the maximum allowed crack width of the element.

Element	Moment		Shear	Axial force		Crack width
	x / z	**y**		**Compression**	**Tension**	
Top longitudinal bars	Biaxial combination: 62 %		2 %	22 %	-	46 %
Lateral truss diagonals	Biaxial combination: 99 %		59 %	27 %	95 %	27 %
Transverse deck beams	-	1 %	2 %	-	7 %	11 %
Longitudinal deck beams	-	3 %	59 %	12 %	51 %	29 %
Bottom horizontal diagonals	-	-	-	7 %	30 %	9 %
Walkway deck slab	9 %	24 %	17 %	7 %	21 %	26 %
Support walls	76 %	1 %	40 %	18 %	97 %	19 %
Post-tensioned strip	5 %	-	84 %	51 %	64 %	76 %

Table 13 – Capacity ratios in percentage for all elements and load types

As seen in the table, the three elements with the highest utilization are the top longitudinal bars, the lateral trusses, and the support walls. This is predictable as they are the main bearing structure of the bridge. The post-tensioned strips will have a significant level of utilization of the tensile strength and shear capacity. These high utilizations would only be possible with the implementation of the high strength fiber reinforced concrete combined with normal traditional reinforcement and prestressed reinforcement. A similar bridge design with such slim dimensions would be impossible to execute with normal concrete. A general high level of utilizations, points to an effective bridge design that uses the full capacity of the different elements, with some elements having the possibility for an even slimmer design, such as the walkway deck. Some elements are induced to lesser loads, and will have a more relevant role to support other structures rather than being fully load bearing and utilized themselves. Elements such as the transverse deck beams does not in itself carry a great deal of loads, and is therefore predictably not highly utilized in the different categories.

These beams are however a significant bearing structure for the walkway deck slab, being the reason for the slabs internal spans to never exceed 2,3m. The walkway deck slab is thus not highly utilized, and both element types are examples of elements that with further work can be designed with the use of less concrete and/or reinforcement.

It is confirmed that with a lower strength $f_{ck} = 100\ MPa$ concrete to correspond to EN1992-1-1, the bridge is still sufficiently designed for the induced loads, crack widths and total deflections according to the Norwegian public road administration handbook for bridge design.

13 Requirement of extra reinforcement

The fiber reinforced concrete has a significant strength on its own, and it is interesting to examine just how big its influence is on the different capacities. The following table illustrates which elements are in need of additional reinforcement on top of the minimum reinforcement according to NB38. As the steel fibers contribute to a great shear force capacity, it is predictable that none of the elements need extra shear reinforcement. The high strength concrete used produces a superior compression force capacity, and because of this, none of the elements need extra reinforcement for the pure axial compression force. However, the first two elements are both affected by a biaxial moment combined with long-term axial force eccentricity. These elements are thus in need of extra reinforcement, as the fiber reinforcement does not have enough strength to support forces of this magnitude. The other elements in need of extra reinforcement are the support walls and the post-tensioned strips. This correlates with the high utilizations of these elements.

Element	Moment	Shear	Axial force	
			Compression	**Tension**
Top longitudinal bars	Yes	No	No	No
Lateral truss diagonals	Yes	No	No	Yes
Transverse deck beams	No	No	No	No
Longitudinal deck beams	No	No	No	No
Bottom horizontal diagonals	No	No	No	Yes
Walkway deck slab	No	No	No	No
Support walls	Yes	No	No	Yes
Post-tensioned strip	No	No	No	Yes

Table 14 – Overview of elements where Yes in red is elements in need of extra reinforcement

What this table illustrates very clearly is the advantages of a fiber reinforced concrete solution.

The economic benefits from not needing extra traditional reinforcement in most of the elements is significant. The high price of reinforcement steel compared to the fiber reinforcement makes it a favorable economic solution.

14 Deflection

According to (Statens Vegvesen, 2015), the limit for maximum deflection of bridges should be:

$$\delta_{max} \leq \frac{L_{span}}{350}$$

For this single span bridge the maximum allowed deflection would be:

$$\delta_{max} \leq \frac{35000\ mm}{350} = 100\ mm$$

Figure 43 shows the deflection of the bridge in millimeters from FEM-Design in serviceability limit state.

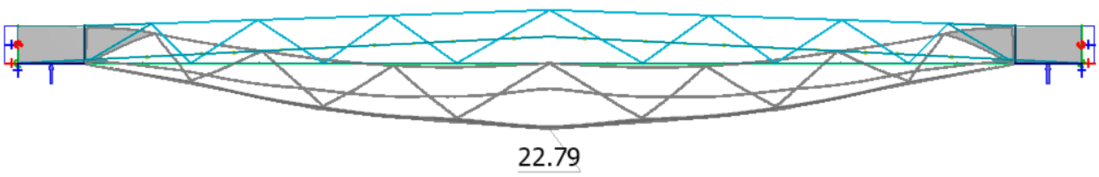

Figure 43 – Illustration of bridge deflection for the whole length retrieved from FEM-Design

The bridge is below the limit for deflections, due to its effective structural system. Combined with the prestressed reinforcement stiffening the structure from the bottom chords, the result is a total deflection of 22.8 mm in serviceability limit state.

Long-span prestressed concrete bridges can suffer from excessive deflection during their service lives. Shrinkage can cause non-uniform strains in the concrete, which is caused by an uneven moisture distribution. This can induce significant additional deflections in the bridge, especially when combined with creep and cracking. Design practices of today will usually overlook these factors, and it is few proposed approaches to consider them as they are complex and computationally expensive.

The results from (Birhane, et al., 2020) shows that the non-uniform shrinkage effect is more significant in medium- to long-span bridges, and that the cracking of concrete will also reduce the stiffness and thereby increase the long-term deflection of the bridge. This is found to be more severe in combination with creep and shrinkage.

Long-term deflections would reasonably agree with the measured data, and be increased. (Birhane, et al., 2020) In this thesis the shrinkage is introduced as an external invisible load in FEM-Design, and defined in chapter 10.1.7.

15 Vibrations

There are multiple criteria for assessment of a structures oscillation and frequency. Normally, it is divided between stresses of structures and human sensitivity. The former point is based on the strength and safety of the structures and can be checked in terms of calculation and dimensioning. With regard to human sensitivity, the usability of the structures is affected. There are criteria for comfort for a constructions condition of use. If these are not met, the building may be impaired. If the fluctuations are noticeably large, questions of safety are often raised by users. Requirements for structural stresses due to dynamic stresses and vibrations are usually specified in the construction standards. Constructive stresses are often related to deformations and stresses. Checking fatigue conditions is an example of this. (Betongelementforeningen, 2005)

Requirements for limiting deflections in construction standards may indirectly include avoiding undesired oscillation sensitivity. Vibration requirements are normally stated in the form of acceptance limits with regard to accelerations and speeds, depending on frequency. The requirements may vary for different types of constructions and their area of use. To avoid unwanted oscillations and vibrations in a structure or in a building, it is assumed that the designer has knowledge of when such phenomena occur. In the case of a walkway bridge mainly oscillated by traffic load by foot, the resonance frequency of the deck should be higher than twice the maximum of the load frequency. For pedestrian traffic, this means a frequency higher than 5.2 Hz. (Betongelementforeningen, 2005)

The following table shows the eigenfrequencies of the bridge structure, retrieved from FEM-Design.

Shape	Frequency	Period	Modal mass	mx'	my'	mz'
[-]	[Hz]	[s]	[t]	[%]	[%]	[%]
1	5.555	0.180	1.000	0.000	29.816	0.000
2	5.761	0.174	1.000	0.000	0.000	51.798
3	7.327	0.136	1.000	0.000	0.000	0.000
4	8.801	0.114	1.000	0.000	0.000	0.000
5	9.333	0.107	1.000	0.000	0.136	0.000

Table 15 – Eigenfrequencies from mode 1-5 retrieved from FEM-Design

The first vibration shape has a frequency of 5.555 Hz and will be over the limit for risk of resonance from pedestrian traffic. The rows for mx’, my’ and mz’ are showing the percentage of total mass that is moving with each vibration shape. This is not assessed here.

For some constructions it may be relevant and necessary to introduce damping measures.

The most important parameters to avoid unwanted oscillations are the frequency ratio between the natural design frequency and the load frequency. The natural frequency of the structure is influenced by the rigidity and mass of the structure. The choice of the constructions span, flexural stiffness and mass is strongly influencing the swing properties, together with external factors. Structural damping is often relatively small, but the possibility of installing damping measures can reduce the oscillations to a satisfactory level. (Betongelementforeningen, 2005)

This type of damping measures will not be needed in this construction as the stiffness of the structure across the span creates a rigid structure with oscillations inducing frequency over the required amount of hertz.

16 Conclusion

In this thesis it has been studied the possibilities of designing a bridge with slim dimensions and a low requirement of reinforcement, with the use of fiber reinforced concrete in combination with prestressed reinforcement. This can be due to economic and aesthetic reasons, as well as for easier and cheaper construction. The structural design of the footbridge is based on a similar bridge with characteristic concrete strength $f_{ck} = 150\ MPa.$ However, the concrete strength has been reduced to a value of $f_{ck} = 100\ MPa.$ The utilizations and required reinforcement of chapters 12 and 13 points to the successful design of the bridge with small concrete dimensions and with the smallest amount of reinforcement necessary. A draft version of the new EN1992-1-1 has been consulted for the design parameters, and when complete will become a more reliable design standard to implement in projects.

A solution of fiber reinforced concrete in combination with prestressed reinforcement is effective and secures the proper strength and durability of the bridge structure. Fiber reinforcement can be a sought-after solution as complicated and time-consuming execution of large amounts of shear reinforcement is avoided. The use of reinforced concrete can thus result in reduced costs related to iron bonding, which is desirable due to expensive labor. The new guidelines in NB38 form the basis for increased experience and further development of regulations for fiber concrete in load-bearing structures. Increased experience, research and demand for fiber concrete with a higher steel content can make the construction solution with post-tensioning and fiber reinforcement highly sought after in the coming years.

Suggestions for further work:

- Dynamic analysis for earthquake dimensioning
- Further design of elements to maximize utilizations and minimize cross-sections
- Bridge design with other solutions and materials to compare
- Curved bridge deck instead of straight deck, analyzed in programs such as SAP2000 or Diana
- Examine the manufacturing and construction process of the bridge
- Perform a real lab test for different composites of fiber and prestressed reinforcement

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PART V – APPENDIX

A – Design of Lateral Truss Element

A.1 Bar geometry and concrete data

Bar length of tension element $L_T := 3.0\ m$

Bar width $b := 120\ mm$

Bar height $h := 150\ mm$

Cover $c := 25\ mm$

Critical buckling length $L_{cr} := 2.0\ m$

Area $A := b \cdot h = 18000\ mm^2$

Some bars design loads are tension and some are compression and buckling. Both element types will be checked here for the same reinforcement situation.

Concrete data

Material factor $\gamma_c := 1.5$

Characteristic compressive strength $f_{ck} := 100\ MPa$

Medium tensile strength $f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$

Dimensioning compressive strength $f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.67\ MPa$

Modulus of elasticity $E_{cm} := 45000\ MPa$

Yield strain for compression $\varepsilon_{cu} := 3.5 \cdot 10^{-3}$

Characteristic tension stength $f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

A.2 Fiber reinforcement [Valid for all structural elements]

Fiber safety factor	$\gamma_f := 1.5$
Characteristic residual flexural tensile strength	$f_{R.1k} := 8\ MPa$
Characteristic residual flexural tensile strength dependent on ductility. Simplified put as same value	$f_{R.3k} := f_{R.1k} = 8\ MPa$
Strength in ULS	$f_{Ftuk} := 0.37 \cdot f_{R.3k} = 2.96\ MPa$
Strength in SLS	$f_{Ftsk} := 0.45 \cdot f_{R.1k} = 3.6\ MPa$
Dimensioning fiber reinforcement strength	$f_{Ftud} := \frac{f_{Ftuk}}{\gamma_f} = 1.973\ MPa$
Assumed factor for volume effect	$k_G := 1.5$
Strength for moment calculations in ULS	$f_{Ftud.m} := k_G \cdot f_{Ftud} = 2.96\ MPa$

A.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.78\ MPa$

Reinforcement bar diameter $Ø_s := 20\ mm$

Strain $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars $d := h - c - \frac{20\ mm}{2} = 115\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 18.319\ mm^2$$

Even with the very low required minimum reinforcement, there will be chosen a Ø20 bar in each corner, to ensure resistance against buckling.

Effective reinforcement area $A_{s.eff} := 4 \cdot 314\ mm^2 = 1256\ mm^2$

Maximum allowed reinforcement area $A_{s.max} := 0.08 \cdot A = 1440\ mm^2$

OK

A.4 Loads (FEM-Design)

Ultimate limit state

Moment in y direction $M_{y.ULS} := 6.3\ kN \cdot m$

Moment in z direction $M_{z.ULS} := 0.3\ kN \cdot m$

Axial force (compression) $N_{Ed.C.ULS} := 426.1\ kN$

Axial force (tension) $N_{Ed.T.ULS} := 472.7\ kN$

Shear force $V_{Ed.ULS} := 90.6\ kN$

Serviceability limit state

Moment in y direction $M_{y.SLS} := 5.2\ kN \cdot m$

Moment in z direction $M_{z.SLS} := 0.1\ kN \cdot m$

Axial force (compression) $N_{Ed.C.SLS} := 391.2\ kN$

Axial force (tension) $N_{Ed.T.SLS} := 351.9\ kN$

Shear force $V_{Ed.SLS} := 85.0\ kN$

Mz for use in second order degree biaxial bending with axial force has a low value here because of focusing on the compressive member with risk of buckling. A higher Mz is for the all over worst element subdued to this moment, but is not the same as for the buckling member. Value used in φ_{effz} will include the correct values for the member in buckling.

A.5 Check if slender compressive chord

Axial force utilization $n := \frac{N_{Ed.C.ULS}}{f_{cd} \cdot A} = 0.418$

Effective reinforcement in one axis $A_{s.eff.y} := \frac{A_{s.eff}}{2} = 628\ mm^2$

$A_{s.eff.z} := A_{s.eff.y} = 628\ mm^2$

Mechanical reinforcement ratio $wy := \frac{A_{s.eff.y} \cdot f_{yd}}{f_{cd} \cdot A} = 0.268$

$wz := wy = 0.268$

Cross section moment of inertia for y axis $Iy := \frac{1}{12} \cdot b \cdot h^3 = \left(3.38 \cdot 10^7\right)\ mm^4$

Cross section moment of inertia for z axis $Iz := \frac{1}{12} \cdot h \cdot b^3 = \left(2.16 \cdot 10^7\right)\ mm^4$

Radius of gyration of the uncracked concrete section (y axis) $i_y := \sqrt{\frac{Iy}{A}} = 43.3\ mm$

Radius of gyration of the uncracked concrete section (y axis) $i_z := \sqrt{\frac{Iz}{A}} = 34.64\ mm$

Slenderness ratio (y axis) $\lambda_y := \frac{L_{cr}}{i_y} = 46.188$

Slenderness ratio (z axis) $\lambda_z := \frac{L_{cr}}{i_z} = 57.735$

Creep coefficient $\varphi := 2.5$

Factor for reduced area because of creep $A\varphi := \dfrac{1.25}{(1+0.2\cdot\varphi)} = 0.833$

Slenderness ratio adjusted for creep, reinforcement strength and utilization

$$\lambda_{ny} := \frac{\lambda_y \cdot \sqrt{\dfrac{n}{1+2\cdot wy}}}{A\varphi} = 28.911$$

$$\lambda_{nz} := \frac{\lambda_z \cdot \sqrt{\dfrac{n}{1+2\cdot wz}}}{A\varphi} = 36.138$$

Limit for slenderness ratio EN1992-1-1 Annex O O.6(1) $\lambda_{lim.simpl} := \dfrac{10.8}{\sqrt[2]{n}} = 16.71$

Values over $\lambda_{lim.simpl} := \dfrac{10.8}{\sqrt{n}}$ => Second order moments must be taken into account. [EN1992-1-1 Annex O O.6(1)]

<u>Check with exentricity on both axes:</u>

Creep ratio for local moment effects $\varphi_{eff.y} := \varphi \cdot \dfrac{M_{y.SLS}}{M_{y.ULS}} = 2.063$

$$\varphi_{eff.z} := \varphi \cdot \frac{M_{z.SLS}}{M_{z.ULS}} = 0.833$$

Factor to calculate the creep coefficient $\beta_y := 0.35 + \dfrac{f_{ck}}{200\ MPa} - \dfrac{\lambda_y}{150} = 0.542$

$$\beta_z := 0.35 + \frac{f_{ck}}{200\ MPa} - \frac{\lambda_z}{150} = 0.465$$

Coefficient accounting for creep

$$k\varphi_y := 1 + \beta_y \cdot \varphi_{eff.y} = 2.119$$

$$k\varphi_z := 1 + \beta_z \cdot \varphi_{eff.z} = 1.388$$

Factor to calculate coefficient for axial load

$$nu_y := 1 + wy = 1.268$$

$$nu_z := 1 + wz = 1.268$$

Value of n at maximum moment resistance

$$nbal := 0.4$$

Coefficient depending on the axial load

$$kr_y := \frac{nu_y - n}{nu_y - nbal} = 0.98 \quad < \ 1.0$$

$$kr_z := kr_y = 0.98$$

Distance from center reinforcement

$$d_y := h - c - \frac{Ø_s}{2} = 115 \ mm$$

$$d_z := b - c - \frac{Ø_s}{2} = 85 \ mm$$

Base value for curvature

$$r0_y := \frac{0.45 \cdot d_y}{\varepsilon_{yd}} = \left(2.5 \cdot 10^4\right) \ mm$$

$$r0_z := \frac{0.45 \cdot d_z}{\varepsilon_{yd}} = \left(1.847 \cdot 10^4\right) \ mm$$

Nominal curvature for each axis

$$r_y := \frac{r0_y}{kr_y \cdot k\varphi_y} = \left(1.204 \cdot 10^4\right) \ mm$$

$$r_z := \frac{r0_z}{kr_z \cdot k\varphi_z} = \left(1.359 \cdot 10^4\right)\ mm$$

Factor depending on the total curvature distribution along the memeber due to first plus second order moments and the non-linear moment-curvature relationship of the cross-sections

$$C := \pi^2 = 9.87$$

Eccentricity for each axis causing second order moments

$$e2_y := \frac{1}{r_y} \cdot \frac{{L_{cr}}^2}{C} = 33.649\ mm$$

$$e2_z := \frac{1}{r_z} \cdot \frac{{L_{cr}}^2}{C} = 29.817\ mm$$

Second order moments for each axis

$$M_{2edy} := e2_y \cdot N_{Ed.C.SLS} = 13.16\ kN \cdot m$$

$$M_{2edz} := e2_z \cdot N_{Ed.C.SLS} = 11.66\ kN \cdot m$$

Total bending moment about each axis:

$$M_{tot.y} := M_{2edy} + M_{y.ULS} = 19.464\ kN \cdot m$$

$$M_{tot.z} := M_{2edz} + M_{z.ULS} = 11.96\ kN \cdot m$$

Bending moment capacity about each axis:

Distance from middle of reinforcement on each side for both axes

$$hy' := d_y - c - -\frac{20\ mm}{2} = 100\ mm$$

$$hz' := d_z - c - \frac{20\ mm}{2} = 50\ mm$$

Ratio of reinforcement distance and height of cross section for each axis

$$h0_y := \frac{hy'}{h} = 0.667$$

$$h0_z := \frac{hz'}{h} = 0.333$$

Adjusted mechanical reinforcement ratio for each axis

$$Wy := \frac{A_{s.eff.y} \cdot f_{yd}}{2 \cdot f_{cd} \cdot A} = 0.134$$

$$Wz := \frac{A_{s.eff.z} \cdot f_{yd}}{2 \cdot f_{cd} \cdot A} = 0.134$$

Moment utilization from moment and axial force relation table

$$my := 0.20$$

$$mz := 0.20$$

There is trouble with finding sufficient mn diagrams to fit this situation. Will use diagram for h'/h = 0.7 as a simplified solution. There is also problems with applying diagrams of this type to a concrete with this high quality. A simplification done in this thesis to be able to get usable results is to use the current available mn diagrams for up to B45 concrete.

A.5.1 Bi-axial moment utilization ratio

Total bending moment capacity for both axes

$$M_{Rd.y} := my \cdot f_{cd} \cdot A \cdot h = 30.6 \ kN \cdot m$$

$$M_{Rd.z} := mz \cdot f_{cd} \cdot A \cdot b = 24.48 \ kN \cdot m$$

<u>Combination of Mz, My and N:</u>

Axial compression utilization of reinforcement and concrete cross section

$$u := \frac{N_{Ed.C.ULS}}{A_{s.eff} \cdot f_{yd} + f_{cd} \cdot A} = 0.272$$

Combination factor for bi-axial moment

$$a := 1 + \frac{1.5 - 1}{0.7 - 0.1} \cdot u = 1.227$$

Utilization of bi-axial moment capacity, without including fiber reinforcement

$$\left(\frac{M_{tot.y}}{M_{Rd.y}}\right)^a + \left(\frac{M_{tot.z}}{M_{Rd.z}}\right)^a = 0.99$$

Simplified contribution of fiber reinforcement moment capacity

$$M_{Rd.fiber} := 0.4 \cdot f_{Ftud} \cdot b \cdot h^2 = 2.13 \ kN \cdot m$$

Utilization of bi-axial moment capacity, with including fiber reinforcement

$$\left(\frac{M_{tot.y}}{M_{Rd.y} + M_{Rd.fiber}}\right)^a + \left(\frac{M_{tot.z}}{M_{Rd.z} + M_{Rd.fiber}}\right)^a = 0.9$$

A.6 Tension force utilization ratio

Dimensioning tension force (half cross section)

$$M_{tot.ySLS} := M_{2edy} + M_{ySLS} = 18.36\ kN \cdot m$$

$$N_{Ed.T} := \frac{M_{tot.ySLS}}{300\ mm} + \frac{N_{Ed.TULS}}{2} = 297.56\ kN$$

Tension force utilization only concrete

$$u_T := \frac{N_{Ed.T}}{\frac{A}{2} \cdot \frac{f_{ctk0.05}}{\gamma_c}} = 13.88$$

Tension force utilization only fiber reinforcement

$$u_T := \frac{N_{Ed.T}}{f_{Ftud} \cdot \frac{A}{2}} = 16.75$$

Tension force utilization only traditional reinforcement

$$u_T := \frac{N_{Ed.T}}{\frac{A_{s.eff}}{2} \cdot f_{yd}} = 1.09$$

Tension force utilization total

$$u_T := \frac{N_{Ed.T}}{\frac{A}{2} \cdot \frac{f_{ctk0.05}}{\gamma_c} + \frac{A_{s.eff}}{2} \cdot f_{yd} + f_{Ftud} \cdot \frac{A}{2}} = 0.95$$

A.7 Control for shear force with fiber reinforcement [NB38]

Distance to middle of reinforcement bar

$$h' := hy' = 100.0\ mm$$

Dimensioning shear force

$$V_{Ed.ULS} = 90.6\ kN$$

Induced shear stress on cross section

$$\tau_{Ed} := \frac{V_{Ed.ULS}}{b \cdot h'} = 7.55\ MPa$$

Aggregate size. 20mm + Reinforcement bar diameter

$$d_{dg} := 20\ mm + Ø_s = 40.0\ mm$$

Minimum concrete shear resistance

$$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.89$$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):

$$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.251$$

Reinforcement ratio

$$\rho_l := \frac{A_{s.eff}}{b \cdot d} = 0.091$$

Shear force resistance without fiber

$$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$$

$$\tau_{Rd.c} := 2.72\ MPa$$

Shear force resistance with fiber contribution

$$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 12.806\ MPa$$

Shear force utilization

$$u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.59$$

A.8 Control for cracking in SLS

As the cross section contains bar reinforcement, there is a requirement for control of crack widths in the service limit state.

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $Ø_s = 20\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min\left(c + 5\ Ø_s, 10\ Ø_s, 3.5\ c\right) = 87.5\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 0.011\ m^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.12$

Largest distance between cracks $s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \dfrac{Ø_s}{\rho_{c.ef}}\right) \cdot \left(1 - \dfrac{f_{Fts.ef}}{f_{ctm}}\right) = 0.029\ m$

Total moment in SLS: $M_{tot.ySLS} := M_{2edy} + M_{ySLS} = 18.364\ kN \cdot m$

Simplified calculation of stress in reinforcement bars:

$$\frac{M_{tot.ySLS}}{300\ mm} = 61.21\ kN$$

$$\frac{61.21\ kN + \dfrac{N_{Ed.TSLS}}{2}}{A_{s.eff.y}} = 377.64\ MPa$$

$\sigma_s := 350\ MPa$ (175 MPa in each bar, < 0,8 fyk [EN1992-1-1 TABLE 9.1])

Simplified reduction of σ_s to account for the fibers contribution.

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term

$k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \frac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 0.002$$

or

$$\varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 0.001$$

$\varepsilon_{dif} := 0.002$

Minimum concrete cover

$c_{min.dur} := 25\ mm$

Reduction factor for crack width limit

$$k_{surf} := \frac{c}{\left(10\ mm + c_{min.dur}\right)} = 0.714$$

Crack width limit

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.214\ mm$

Crack width

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.057\ mm$

Utilization

$$u_w := \frac{w_k}{w_{lim}} = 0.27$$

B – Design of Longitudinal Top Bar

B.1 Bar geometry and concrete data

Bar length	$L := 35\ m$
Bar width	$b := 300\ mm$
Bar height	$h := 300\ mm$
Cover	$c := 25\ mm$
Critical buckling length	$L_{cr} := 5\ m$
Area	$A := b \cdot h = 90000\ mm^2$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \frac{0.85 \cdot f_{ck}}{\gamma_c} = 56.67\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

B.2 Fiber reinforcement

See attachment A.2.

B.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.78\ MPa$

Reinforcement bar diameter $Ø_s := 20\ mm$

Strain $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars (assuming Ø16 reinforcement) $d := h - c - \frac{16\ mm}{2} = 267\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 106.332\ mm^2$$

To meet this requirement there is enough with 2xØ10 traditional reinforcement bars. There will however be chosen a Ø20 bar in each corner, to ensure resistance against buckling.

Effective reinforcement area $A_{s.eff} := 4 \cdot 314\ mm^2 = 1256\ mm^2$

B.4 Loads (FEM-Design)

Ultimate limit state

Moment in y direction (hogging) $M_{y.HOG.ULS} := 8.8\ kN \cdot m$

Moment in y direction (sagging) $M_{y.SAG.ULS} := 13.8\ kN \cdot m$

Moment in z direction (hogging +) $M_{z.HOG1.ULS} := 0.8\ kN \cdot m$

Moment in z direction (hogging -) $M_{z.HOG2.ULS} := 1.2\ kN \cdot m$

Axial force (compression) $N_{Ed.ULS} := 1228.3\ kN$

Shear force $V_{Ed.ULS} := 11.0\ kN$

Serviceability limit state

Moment in y direction (hogging) $M_{y.HOG.SLS} := 7.3\ kN \cdot m$

Moment in y direction (sagging) $M_{y.SAG.SLS} := 10.7\ kN \cdot m$

Moment in z direction (hogging +) $M_{z.HOG1.SLS} := 0.6\ kN \cdot m$

Moment in z direction (hogging -) $M_{z.HOG2.SLS} := 0.0\ kN \cdot m$

Axial force (compression) $N_{Ed.SLS} := 934.7\ kN$

Shear force $V_{Ed.SLS} := 8.9\ kN$

B.5 Verification of stability for axial loads

Axial force utilization $n := \frac{N_{Ed.ULS}}{f_{cd} \cdot A} = 0.241$

Effective reinforcement in one axis $A_{s.eff.y} := \frac{A_{s.eff}}{2} = 628 \ mm^2$

$A_{s.eff.z} := A_{s.eff.y} = 628 \ mm^2$

Mechanical reinforcement ratio $wy := \frac{A_{s.eff.y} \cdot f_{yd}}{f_{cd} \cdot A} = 0.054$

$wz := wy = 0.054$

Cross section moment of inertia $Iy := \frac{1}{12} \cdot b \cdot h^3 = \left(6.75 \cdot 10^8\right) \ mm^4$

Radius of gyration of the uncracked concrete section $i := \sqrt{\frac{Iy}{A}} = 86.603 \ mm$

Slenderness ratio $\lambda_y := \frac{L_{cr}}{i} = 57.735$

Creep coefficient $\varphi := 2.5$

Factor for reduced area because of creep $A\varphi := \frac{1.25}{(1 + 0.2 \cdot \varphi)} = 0.833$

Slenderness ratio adjusted for creep, reinforcement strength and utilization $\lambda_{ny} := \frac{\lambda_y \cdot \sqrt{\frac{n}{1 + 2 \cdot wy}}}{A\varphi} = 32.315$

Value over $\lambda_{lim.simpl} := \dfrac{10.8}{\sqrt{n}}$ => Second order moments must be taken into account. [EN1992-1-1 Annex O O.6(1)]

Check with exentricity on both axes:

Creep ratio for local moment effects $\varphi_{eff.y} := \varphi \cdot \dfrac{M_{y.SAG.SLS}}{M_{y.SAG.ULS}} = 1.938$

$\varphi_{eff.z} := 0$

Factor to calculate the creep coefficient $\beta_y := 0.35 + \dfrac{f_{ck}}{200\ MPa} - \dfrac{\lambda_y}{150} = 0.465$

$\beta_z := \beta_y = 0.465$

Coefficient accounting for creep $k\varphi_y := 1 + \beta_y \cdot \varphi_{eff.y} = 1.902$

$k\varphi_z := 1 + \beta_z \cdot \varphi_{eff.z} = 1$

Factor to calculate coefficient for axial load $nu_y := 1 + wy = 1.054$

$nu_z := 1 + wz = 1.054$

Value of n at maximum moment resistance $nbal := 0.4$

Coefficient depending on the axial load $kr_y := \dfrac{nu_y - n}{nu_y - nbal} = 1.244 \quad > \quad 1.0$

=> $kr_y := 1.0$

Coefficient depending on the axial load $$kr_z := \frac{nu_z - n}{nu_z - nbal} = 1.244 \quad > \quad 1.0$$

=> $kr_z := 1.0$

Base value for curvature $$r0_y := \frac{0.45 \cdot d}{\varepsilon_{yd}} = \left(5.803 \cdot 10^4\right)\ mm$$

$$r0_z := \frac{0.45 \cdot d}{\varepsilon_{yd}} = \left(5.803 \cdot 10^4\right)\ mm$$

Nominal curvature for each axis $$r_y := \frac{r0_y}{kr_y \cdot k\varphi_y} = \left(3.052 \cdot 10^4\right)\ mm$$

$$r_z := \frac{r0_z}{kr_z \cdot k\varphi_z} = \left(5.803 \cdot 10^4\right)\ mm$$

Factor depending on the total curvature distribution along the memeber due to first plus second order moments and the non-linear moment-curvature relationship of the cross-sections $$C := \pi^2 = 9.87$$

Eccentricity for each axis causing second order moments $$e2_y := \frac{1}{r_y} \cdot \frac{{L_{cr}}^2}{C} = 83\ mm$$

$$e2_z := \frac{1}{r_z} \cdot \frac{{L_{cr}}^2}{C} = 43.649\ mm$$

Second order moments for each axis

$$M_{2edy} := e2_y \cdot N_{Ed.SLS} = 77.58\ kN \cdot m$$

$$M_{2edz} := e2_z \cdot N_{Ed.SLS} = 40.8\ kN \cdot m$$

Total bending moment about each axis:

$$M_{tot.y} := M_{2edy} + M_{y.HOG.ULS} = 86.38\ kN \cdot m$$

$$M_{tot.z} := M_{2edz} + M_{z.HOG2.ULS} = 42\ kN \cdot m$$

Bending moment capacity about each axis:

Distance from middle of reinforcement on each side for both axes

$$hy' := d - c - 8\ mm - \frac{20\ mm}{2} = 224\ mm$$

$$hz' := hy' = 224\ mm$$

Ratio of reinforcement distance and height of cross section for each axis

$$h0_y := \frac{hy'}{h} = 0.747$$

$$h0_z := \frac{hz'}{h} = 0.747$$

Adjusted mechanical reinforcement ratio for each axis

$$Wy := \frac{A_{s.eff.y} \cdot f_{yd}}{2 \cdot f_{cd} \cdot A} = 0.027$$

$$Wz := \frac{A_{s.eff.z} \cdot f_{yd}}{2 \cdot f_{cd} \cdot A} = 0.027$$

B.5.1 Bi-axial combination utilization

Moment utilization from moment and axial force relation table

$my := 0.10$

$mz := 0.10$

Total bending moment capacity for both axes

$M_{Rd.y} := my \cdot f_{cd} \cdot A \cdot h = 153 \ kN \cdot m$

$M_{Rd.z} := mz \cdot f_{cd} \cdot A \cdot h = 153 \ kN \cdot m$

Combination of Mz, My and N:

Axial utilization of reinforcement and concrete cross section

$$u := \frac{N_{Ed.ULS}}{A_{s.eff} \cdot f_{yd} + f_{cd} \cdot A} = 0.218$$

Combination factor for bi-axial moment

$$a := 1 + \frac{1.5 - 1}{0.7 - 0.1} \cdot u = 1.181$$

Utilization of bi-axial moment capacity, without including fiber reinforcement

$$\left(\frac{M_{tot.y}}{M_{Rd.y}}\right)^{a} + \left(\frac{M_{tot.z}}{M_{Rd.z}}\right)^{a} = 0.73$$

Simplified fiber reinforcement moment capacity

$$M_{Rd.fiber} := 0.4 \cdot f_{Ftud} \cdot b \cdot h^2 = 21.31 \ kN \cdot m$$

Utilization of bi-axial moment capacity, with including fiber reinforcement

$$\left(\frac{M_{tot.y}}{M_{Rd.y} + M_{Rd.fiber}}\right)^{a} + \left(\frac{M_{tot.z}}{M_{Rd.z} + M_{Rd.fiber}}\right)^{a} = 0.62$$

Sufficient capacity with or without including the simplified moment capacity of the fiber reinforcement.

This compressive chord is dependent on its buckling length not being any longer. This is done by having sufficient stiffness in the lateral truss which the chord rests on. This way the buckling length would be kept at approximately 5m in each direction.

B.6 Control for shear with fiber reinforcement

Distance to middle of reinforcement bar $h' := hy' = 224.0\ mm$

Dimensioning shear force $V_{Ed.ULS} = 11\ kN$

Induced shear stress on cross section $\tau_{Ed} := \frac{V_{Ed.ULS}}{b \cdot h'} = 0.164\ MPa$

Aggregate size. 20mm + Reinforcement bar diameter $d_{dg} := 20\ mm + \text{Ø}_s = 40.0\ mm$

Minimum concrete shear resistance $\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.238$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973): $\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.251$

Reinforcement ratio $\rho_l := \frac{A_{s.eff}}{b \cdot d} = 0.016$

Shear force resistance without fiber $\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$

$\tau_{Rd.c} := 1.15\ MPa$

Shear force resistance with fiber contribution $\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 6.553\ MPa$

Shear force utilization $u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.02$

B.7 Control for cracking in SLS

As the cross section contains bar reinforcement, there is a requirement for control of crack widths in the service limit state.

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $Ø := 20\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min(c + 5\ Ø, 10\ Ø, 3.5\ c) = 87.5\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 0.026\ m^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.048$

Largest distance between cracks $s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \dfrac{Ø}{\rho_{c.ef}}\right) \cdot \left(1 - \dfrac{f_{Fts.ef}}{f_{ctm}}\right) = 0.049\ m$

Total moment in SLS: $M_{tot.y.SLS} := M_{2edy} + M_{y.SAG.SLS} = 88.28\ kN \cdot m$

Simplified calculation of stress in reinforcement bars: $\dfrac{M_{tot.y.SLS}}{300\ mm} = 294.267\ kN$

$$\frac{294.27\ kN}{A_{s.eff.y}} = 468.583\ MPa$$

$\sigma_s := 450\ MPa$ (225 MPa in each bar, < 0,8 fyk [EN1992-1-1 TABLE 9.1])

Simplified reduction of σ_s to account for the fibers contribution.

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term

$k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \frac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 0.002$$

or

$$\varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 0.001$$

$\varepsilon_{dif} := 0.002$

Minimum concrete cover

$c_{min.dur} := 25\ mm$

Reduction factor for crack width limit

$$k_{surf} := \frac{c}{\left(10\ mm + c_{min.dur}\right)} = 0.714$$

Crack width limit

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.214\ mm$

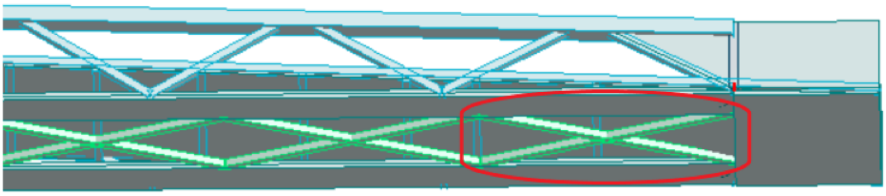
Crack width

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.099\ mm$

Utilization

$$u_w := \frac{w_k}{w_{lim}} = 0.46$$

C – Design of Bottom Horizontal Truss



C.1 Bar geometry and concrete data

Bar length	$L:=4.6\ m$
Bar width	$b:=150\ mm$
Bar height	$h:=100\ mm$
Cover	$c:=25\ mm$
Critical buckling length	$L_{cr}:=\frac{L}{2}=2.3\ m$
Area	$A:=b\cdot h=15000\ mm^2$

Concrete data

Material factor	$\gamma_c:=1.5$
Characteristic compressive strength	$f_{ck}:=100\ MPa$
Medium tensile strength	$f_{ctm}:=1.1\cdot\sqrt[3]{100}\ MPa=5.11\ MPa$
Dimensioning compressive strength	$f_{cd}:=\frac{0.85\cdot f_{ck}}{\gamma_c}=56.67\ MPa$
Modulus of elasticity	$E_{cm}:=45000\ MPa$
Yield strain for compression	$\varepsilon_{cu}:=3.5\cdot10^{-3}$
Characteristic tension stength	$f_{ctk0.05}:=0.7\cdot f_{ctm}=3.57\ MPa$

C.2 Fiber reinforcement

See attachment A.2.

C.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \dfrac{f_{yk}}{\gamma_s} = 434.78\ MPa$

Reinforcement bar diameter $\O_s := 10\ mm$

Strain $\varepsilon_{yd} := \dfrac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars (assuming Ø10 reinforcement) $d := h - c - \dfrac{10\ mm}{2} = 70\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 13.939\ mm^2$$

There is very little minimun reinforcement needed in these elements. For the sake of simplicity there will be implemented the same amount as for similar elements, to ensure an easier construction on site. 2xØ10.

Effective reinforcement area $A_{s.eff} := 2 \cdot 79\ mm^2 = 158\ mm^2$

C.4 Loads (FEM-Design) and slender compressive chord check

Ultimate limit state

Axial force (compression) $N_{Ed.C.ULS} := 31.4\ kN$

Axial force (tension) $N_{Ed.T.ULS} := 39.7\ kN$

Serviceability limit state

Axial force (compression) $N_{Ed.C.SLS} := 27.3\ kN$

Axial force (tension) $N_{Ed.T.SLS} := 29.8\ kN$

Checking if the bar is a slender compressive chord

Axial force utilization $n := \dfrac{N_{Ed.C.ULS}}{f_{cd} \cdot A} = 0.037$

Effective reinforcement in one axis $A_{s.eff.y} := \dfrac{A_{s.eff}}{2} = 79\ mm^2$

$A_{s.eff.z} := A_{s.eff.y} = 79\ mm^2$

Mechanical reinforcement ratio $wy := \dfrac{A_{s.eff.y} \cdot f_{yd}}{f_{cd} \cdot A} = 0.04$

$wz := wy = 0.04$

Cross section moment of inertia $Iy := \dfrac{1}{12} \cdot b \cdot h^3 = \left(1.25 \cdot 10^7\right)\ mm^4$

C.4.1 Slenderness ratio limit

Radius of gyration of the uncracked concrete section

$$i := \sqrt{\frac{Iy}{A}} = 28.868\ mm$$

Slenderness ratio

$$\lambda_y := \frac{L_{cr}}{i} = 79.674$$

Creep coefficient

$$\varphi := 2.5$$

Factor for reduced area because of creep

$$A\varphi := \frac{1.25}{(1 + 0.2 \cdot \varphi)} = 0.833$$

Slenderness ratio adjusted for creep, reinforcement strength and utilization

$$\lambda_{ny} := \frac{\lambda_y \cdot \sqrt{\frac{n}{1 + 2 \cdot wy}}}{A\varphi} = 17.676$$

Slenderness ratio limit for second order analysis

$$\lambda_{lim.simpl} := \frac{10.8}{\sqrt{n}} = 56.191$$

Second order bending moments need not be considered as the slenderness ratio of the element is lower than the limit ratio. Normal buckling will be checked.

C.5 Buckling without 2nd order bending (weak axis)

Recommended critical buckling length factor

$$K := 1.2$$

Critical axial force for buckling

$$N_{cr} := \frac{\pi^2 \cdot E_{cm} \cdot Iy}{K \cdot {L_{cr}}^2} = 874.55\ kN$$

Imperfection factor (buckling curve c, massive profile)

$$\alpha := 0.49$$

Relative slenderness

$$\lambda := \sqrt[2]{\frac{A \cdot f_{cd}}{N_{cr}}} = 0.986$$

Buckling coefficient

$$\phi := 0.5\ \left(1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right) = 1.18$$

Buckling capacity reduction factor

$$\chi := \frac{1}{\phi + \sqrt[2]{\phi^2 - \lambda^2}} = 0.548$$

Axial compression force capacity reduced for buckling

$$N_{b.Rd} := \chi \cdot A \cdot f_{cd} = 465.96\ kN$$

Compression utilization

$$u_N := \frac{N_{Ed.C.ULS}}{N_{b.Rd}} = 0.07$$

Tension utilization

$$u_T := \frac{N_{Ed.T.ULS}}{A_{s.eff} \cdot f_{yd}} = 0.58$$

C.6 Tension force utilization

Tension force utilization only concrete

$$u_T := \frac{N_{Ed.T.ULS}}{A \cdot \frac{f_{ctk0.05}}{\gamma_c}} = 1.11$$

Tension force utilization only fiber reinforcement

$$u_T := \frac{N_{Ed.T.ULS}}{f_{Ftud} \cdot A} = 1.34$$

Tension force utilization only traditional reinforcement

$$u_T := \frac{N_{Ed.T.ULS}}{A_{s.eff} \cdot f_{yd}} = 0.58$$

Tension force utilization total

$$u_T := \frac{N_{Ed.T.ULS}}{A \cdot \frac{f_{ctk0.05}}{\gamma_c} + A_{s.eff} \cdot f_{yd} + f_{Ftud} \cdot A} = 0.3$$

C.7 Control for cracking in SLS

As the cross section contains bar reinforcement, there is a requirement for control of crack widths in the service limit state.

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $\O_s = 10\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min\left(c + 5\ \O_s, 10\ \O_s, 3.5\ c\right) = 75\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 0.011\ m^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.014$

Largest distance between cracks $s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \dfrac{\O_s}{\rho_{c.ef}}\right) \cdot \left(1 - \dfrac{f_{Fts.ef}}{f_{ctm}}\right) = 0.07\ m$

Stress in each reinforcement bar $\sigma_s := \dfrac{\dfrac{N_{Ed.T.SLS}}{2}}{A_{s.eff}} = 94.3\ MPa$

< 0,8 fyk [EN1992-1-1 TABLE 9.1]

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term

$k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \frac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = -2.89 \cdot 10^{-4}$$

or

$$\varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 2.69 \cdot 10^{-4}$$

Minimum concrete cover

$c_{min.dur} := 25\ mm$

Reduction factor for crack width limit

$$k_{surf} := \frac{c}{\left(10\ mm + c_{min.dur}\right)} = 0.714$$

Crack width limit

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.214\ mm$

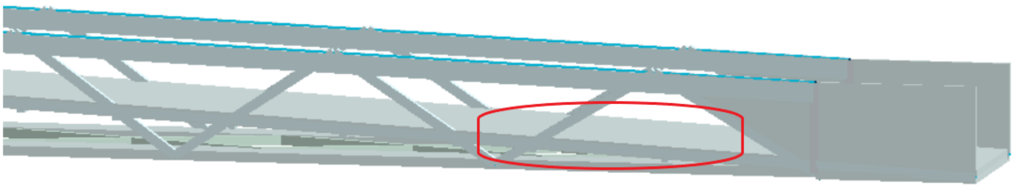
Crack width

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.02\ mm$

Utilization

$$u_w := \frac{w_k}{w_{lim}} = 0.092$$

D – Design of Bridge Walkway Deck



D.1 Slab geometry and concrete data

Slab length (half span)	$L := 17.5\ m$
Slab internal span	$L_i := 2.3\ m$
Slab width	$b := 2500\ mm$
Slab thickness	$t := 250\ mm$
Cover	$c := 25\ mm$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.7\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

D.2 Fiber reinforcement

See attachment A.2.

D.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.8\ MPa$

Reinforcement bar diameter $\text{Ø}_s := 16\ mm$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars $d := t - c - \frac{\text{Ø}_s}{2} = 217.0\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking (per meter):

$$A_{s.min} := \max\left(0.26 \cdot 1000\ mm \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot 1000\ mm \cdot d}{f_{yk}}\right) = 288.07\ mm^2$$

Effective reinforcement area $A_{s.eff} := 5 \cdot 201\ mm^2 = 1005\ mm^2$

Increased reinforcement due to crack requirements. Ø16c250.

D.4 Loads in ULS (FEM-Design)

Ultimate limit state

Moment in x direction (hogging)	$M_{x.HOG.ULS} := 28.1\ kN \cdot m$
Moment in x direction (sagging)	$M_{x.SAG.ULS} := 22.4\ kN \cdot m$
Moment in y direction (hogging)	$M_{y.HOG.ULS} := 75.2\ kN \cdot m$
Moment in y direction (sagging)	$M_{y.SAG.ULS} := 22.9\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.ULS} := 484.9\ kN$
Axial force in x direction (tension)	$N_{x.T.ULS} := 343.2\ kN$
Axial force in y direction (compression)	$N_{y.C.ULS} := 2115.2\ kN$
Axial force in y direction (tension)	$N_{y.T.ULS} := 927.7\ kN$
Shear force in x direction	$V_{x.ULS} := 379.5\ kN$
Shear force in y direction	$V_{y.ULS} := 268.4\ kN$

D.5 Loads in SLS (FEM-Design)

Serviceability limit state

Moment in x direction (hogging)	$M_{x.HOG.SLS} := 25.1\ kN \cdot m$
Moment in x direction (sagging)	$M_{x.SAG.SLS} := 20.6\ kN \cdot m$
Moment in y direction (hogging)	$M_{y.HOG.SLS} := 62.4\ kN \cdot m$
Moment in y direction (sagging)	$M_{y.SAG.SLS} := 16.2\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.SLS} := 583.1\ kN$
Axial force in x direction (tension)	$N_{x.T.SLS} := 266.9\ kN$
Axial force in y direction (compression)	$N_{y.C.SLS} := 2108.6\ kN$
Axial force in y direction (tension)	$N_{y.T.SLS} := 1403.2\ kN$
Shear force in x direction	$V_{x.SLS} := 311.8\ kN$
Shear force in y direction	$V_{y.SLS} := 200.6\ kN$

D.6 Moment capacity for y-axis

Force in effective minimum reinforcement $S_s := f_{yd} \cdot 2.5\ A_{s.eff} = 1092.39\ kN$

Width of tension zone $b_{fy} := 0.5 \cdot L_i = 1.2\ m$

Width of compression zone $b_y := 0.5 \cdot L_i = 1.2\ m$

Heigt of compression zone:

$$\alpha dy_{ys} := \frac{t \cdot b_{fy} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 35\ mm$$

Because of the high concrete and fiber strength, a smaller compression zone develops.

Effective height of traditional reinforcement

$$d_s := t - c - \frac{\O_s}{2} = 217\ mm$$

Tension resultant from fiber reinforcement

$$S_f := \left(t - \alpha dy_{ys}\right) \cdot b_{fy} \cdot f_{Ftud.m} = 731.9\ kN$$

Capacity from fiber reinforcement

$$M_{Rd.f} := S_f \cdot \left(0.5 \cdot t + 0.1 \cdot \alpha dy_{ys}\right) = 94.0\ kN \cdot m$$

Capacity from traditional reinforcement

$$M_{Rd.s} := S_s \cdot \left(d_s - 0.4 \cdot \alpha dy_{ys}\right) = 221.8\ kN \cdot m$$

Total moment capacity

$$M_{Rd.y} := M_{Rd.f} + M_{Rd.s} = 315.8\ kN \cdot m$$

Utilization

$$u_m := \frac{M_{y.HOG.ULS}}{M_{Rd.y}} = 0.24$$

D.7 Moment capacity for x-axis

Force in effective minimum reinforcement $S_s := f_{yd} \cdot 2.3\ A_{s.eff} = 1005\ kN$

Width of tension zone $b_{fx} := 0.5 \cdot b = 1.3\ m$

Width of compression zone $b_x := 0.5 \cdot b = 1.3\ m$

Heigt of compression zone: $$\alpha dx_{xs} := \frac{t \cdot b_{fx} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_x \cdot f_{cd} + b_{fx} \cdot f_{Ftud.m}} = 32\ mm$$

Because of the high concrete strength, a smaller compression zone develops.

Effective height of traditional reinforcement $d_s := t - c - \frac{\O_s}{2} = 217\ mm$

Tension resultant from fiber reinforcement $S_f := \left(t - \alpha dx_{xs}\right) \cdot b_{fx} \cdot f_{Ftud.m} = 806.7\ kN$

Capacity from fiber reinforcement $M_{Rd.f} := S_f \cdot \left(0.5 \cdot t + 0.1 \cdot \alpha dx_{xs}\right) = 103.4\ kN \cdot m$

Capacity from traditional reinforcement $M_{Rd.s} := S_s \cdot \left(d_s - 0.4 \cdot \alpha dx_{xs}\right) = 205.2\ kN \cdot m$

Total moment capacity $M_{Rd.x} := M_{Rd.f} + M_{Rd.s} = 308.6\ kN \cdot m$

Utilization $$u_m := \frac{M_{x.HOG.ULS}}{M_{Rd.x}} = 0.09$$

D.8 Axial force capacity x direction

Axial force cross section area $A_N := b \cdot t = 0.625\ m^2$

Dimensioning tension force $N_{Ed.T} := \dfrac{M_{y.HOG.ULS}}{t - 2\ c - \dfrac{\varnothing_s}{2}} + N_{x.T.ULS} = 734.9\ kN$

Dimensioning compression force $N_{Ed.C} := \dfrac{M_{y.HOG.ULS}}{t - 2\ c - \dfrac{\varnothing_s}{2}} + N_{x.C.ULS} = 876.6\ kN$

Tension resultant from fiber reinforcement (tension force) $S_f := t \cdot b \cdot f_{Ftud} = 1233.3\ kN$

Force in effective minimum reinforcement $S_s := 2 \cdot 2.5\ f_{yd} \cdot A_{s.eff} = 2184.78\ kN$

Reinforcement net with $A_{s.eff}$ per meter on both sides with 2.5m effective length.

Tension capacity of concrete $N_{Rd.cs} := t \cdot b \cdot f_{ctk0.05} = 2233.76\ kN$

Dimensioning tension capacity $N_{Rd.T} := S_f + S_s + N_{Rd.cs} = 5651.88\ kN$

Dimensioning compression capacity (without reinforcement) $N_{Rd.C} := b \cdot t \cdot f_{cd} = 35416.667\ kN$

Utilizations:

Tension utilization $u_T := \dfrac{N_{Ed.T}}{N_{Rd.T}} = 0.13$

Compression utilization $u_C := \dfrac{N_{Ed.C}}{N_{Rd.C}} = 0.02$

D.9 Axial force capacity y direction

Axial force cross section area $A_N := L_i \cdot t = 0.575\ m^2$

Dimensioning tension force $N_{Ed.T} := \dfrac{M_{x.HOG.ULS}}{t - 2\ c - \dfrac{\text{Ø}_s}{2}} + N_{y.T.ULS} = 1074.1\ kN$

Dimensioning compression force $N_{Ed.C} := \dfrac{M_{x.HOG.ULS}}{t - 2\ c - \dfrac{\text{Ø}_s}{2}} + N_{y.C.ULS} = 2261.6\ kN$

Tension resultant from fiber reinforcement (tension force) $S_f := t \cdot L_i \cdot f_{Ftud} = 1134.7\ kN$

Force in effective minimum reinforcement $S_s := 2 \cdot 2.3\ f_{yd} \cdot A_{s.eff} = 2010\ kN$

Reinforcement net with $A_{s.eff}$ per meter on both sides with 2.3m effective length.

Tension capacity of concrete $N_{Rd.cs} := t \cdot L_i \cdot f_{ctk0.05} = 2055.06\ kN$

Dimensioning tension capacity $N_{Rd.T} := S_f + S_s + N_{Rd.cs} = 5199.73\ kN$

Dimensioning compression capacity (without reinforcement) $N_{Rd.C} := L_i \cdot t \cdot f_{cd} = 32583.333\ kN$

Tension utilization $u_T := \dfrac{N_{Ed.T}}{N_{Rd.T}} = 0.21$

Compression utilization $u_C := \dfrac{N_{Ed.C}}{N_{Rd.C}} = 0.07$

D.10 Shear force capacity

Distance to middle of reinforcement bar

$$h' := d - c - \frac{Ø_s}{2} = 184.0\ mm$$

Dimensioning shear force

$$V_{x.ULS} = 379.5\ kN$$

Induced shear stress on cross section

$$\tau_{Ed} := \frac{V_{x.ULS}}{b \cdot h'} = 0.825\ MPa$$

Aggregate size. 20mm + Reinforcement bar diameter

$$d_{dg} := 20\ mm + Ø_s = 36.0\ mm$$

Minimum concrete shear resistance

$$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.3$$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):

$$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.251$$

Reinforcement ratio

$$\rho_l := \frac{2 \cdot A_{s.eff}}{b \cdot d} = 0.004$$

Shear force resistance without fiber

$$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$$

$$\tau_{Rd.c} := 0.75\ MPa$$

Shear force resistance with fiber contribution

$$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 4.96\ MPa$$

Shear force utilization

$$u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.17$$

D.11 Control for cracking in SLS

$c := 25\ mm$ $\quad k_b := 0.8$ (good fastening) $\quad Ø_s = 16\ mm$

$f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min\left(c + 5\ Ø_s, 10\ Ø_s, 3.5\ c\right) = 87.5\ mm$

Effective area of concrete in tension (per meter) $A_{c.eff} := b \cdot h_{c.eff} = 0.22\ m^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{2.5 \cdot A_{s.eff}}{A_{c.eff}} = 0.01$

$$s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \frac{Ø_s}{\rho_{c.ef}}\right) \cdot \left(1 - \frac{f_{Fts.ef}}{f_{ctm}}\right) = 0.13\ m$$

2.5m width makes the effective reinforcement per meter work 2.5 times

Stress in effective reinforcement $\sigma_s := \dfrac{\dfrac{N_{x.T.SLS}}{2} + \dfrac{M_{y.HOG.SLS}}{t}}{2.5 \cdot A_{s.eff}} = 152.46\ MPa$

Reduced stress for fiber contribution $\sigma_s := 150.0\ MPa$

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term $k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \frac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = -1.78 \cdot 10^{-4} \quad \text{or:} \quad \varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 4.29 \cdot 10^{-4}$$

Minimum concrete cover $c_{min.dur} := 25\ mm$

Reduction factor for crack width limit $k_{surf} := \frac{c}{(10\ mm + c_{min.dur})} = 0.714$

Crack width limit $w_{lim} := 0.3\ mm \cdot k_{surf} = 0.214\ mm$

Crack width $w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.056\ mm$

Utilization $u_w := \frac{w_k}{w_{lim}} = 0.26$

E – Design of Longitudinal Bridge Deck Bar

E.1 Bar geometry and concrete data

Bar length	$L := 17.5\ m$
Bar width	$b := 120\ mm$
Bar height	$h := 150\ mm$
Cover	$c := 25\ mm$
Area	$A := b \cdot h = 18000\ mm^2$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.667\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

E.2 Fiber reinforcement

See attachment A.2.

E.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.8\ MPa$

Reinforcement bar diameter $Ø_s := 10\ mm$

Strain $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars (assuming Ø16 reinforcement) $d := h - c - \frac{16\ mm}{2} = 117\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 18.638\ mm^2$$

To meet this requirement there is enough with 2xØ10 traditional reinforcement bars.

Effective reinforcement area $A_{s.eff} := 2 \cdot 79\ mm^2 = 158\ mm^2$

Compression utilization ($N_{Ed.C.ULS} := 132.4\ kN$) $u_C := \frac{N_{Ed.C.ULS}}{A_{s.eff} \cdot f_{yd} + A \cdot f_{cd}} = 0.122$

E.4 Loads (FEM-Design)

Ultimate limit state

Moment in y direction $M_{y.ULS} := 7.6\ kN \cdot m$

Moment in z direction $M_{z.ULS} := 2.5\ kN \cdot m$

Axial force (compression) $N_{Ed.C.ULS} := 132.4\ kN$

Axial force (tension) $N_{Ed.T.ULS} := 20.4\ kN$

Shear force $V_{Ed.ULS} := 41.9\ kN$

Serviceability limit state

Moment in y direction $M_{y.SLS} := 5.8\ kN \cdot m$

Moment in z direction $M_{z.SLS} := 1.9\ kN \cdot m$

Axial force (compression) $N_{Ed.C.SLS} := 100.3\ kN$

Axial force (tension) $N_{Ed.T.SLS} := 14.1\ kN$

Shear force $V_{Ed.SLS} := 31.9\ kN$

E.5 Shear force capacity

Distance to middle of reinforcement bar	$h' := d - c - \frac{\varnothing_s}{2} = 87.0\ mm$
Dimensioning shear force	$V_{Ed.ULS} = 41.9\ kN$
Induced shear stress on cross section	$\tau_{Ed} := \frac{V_{Ed.ULS}}{b \cdot h'} = 4.013\ MPa$
Aggregate size. 20mm + Reinforcement bar diameter	$d_{dg} := 20\ mm + \varnothing_s = 30.0\ mm$
Minimum concrete shear resistance	$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.619$
Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):	$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.251$
Reinforcement ratio	$\rho_l := \frac{A_{s.eff}}{b \cdot d} = 0.011$
Shear force resistance without fiber	$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$
	$\tau_{Rd.c} := 1.21\ MPa$
Shear force resistance with fiber contribution	$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 6.792\ MPa$
Shear force utilization	$u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.59$

E.6 Control for cracking in SLS

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $\O := 10\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min(c + 5\ \O, 10\ \O, 3.5\ c) = 75\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 0.009\ m^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.018$

Largest distance between cracks $s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \dfrac{\O}{\rho_{c.ef}}\right) \cdot \left(1 - \dfrac{f_{Fts.ef}}{f_{ctm}}\right) = 0.062\ m$

Simplified calculation of stress in reinforcement bars: $N_{Ed.T} := \dfrac{M_{y.SLS}}{h} + N_{Ed.T.SLS} = 52.8\ kN$

$$\sigma_s := \frac{N_{Ed.T}}{A_{s.eff}} = 333.966\ MPa$$

Tension utilization $u_T := \dfrac{N_{Ed.T}}{A_{s.eff} \cdot f_{yd} + A \cdot f_{Ftud}} = 0.506$

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term $k_t := 0.4$

Steel modulus of elasticity $E_s := 210000\ MPa$

Concrete modulus of elasticity $E_{cm} := 45000\ MPa$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \dfrac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \dfrac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 9.91 \cdot 10^{-4} \qquad \text{or} \qquad \varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 9.542 \cdot 10^{-4}$$

$\varepsilon_{dif} := 9.91 \cdot 10^{-4}$

Crack width:

$c_{min.dur} := 25\ mm$

$$k_{surf} := \frac{c}{\left(10\ mm + c_{min.dur}\right)} = 0.71$$

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.21\ mm$

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.06\ mm$ < w_{lim} OK

Crack width utilization

$$u_w := \frac{w_k}{w_{lim}} = 0.29$$

E.7 Moment capacity weak axis

Force in effective minimum reinforcement $S_s := f_{yd} \cdot A_{s.eff} = 68.7\ kN$

Width of tension zone $b_{fy} := 0.5 \cdot L = 8.8\ m$

Width of compression zone $b_y := 0.5 \cdot L = 8.8\ m$

Heigt of compression zone: $$\alpha d_{ys} := \frac{h \cdot b_{fy} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 9.4\ mm$$

Cross section in tension, low height of compression zone is predictable. Also because of the heigh concrete strength and thus a smaller compression zone needed.

Effective height of traditional reinforcement $$d_s := h - c - \frac{\varnothing_s}{2} = 120\ mm$$

Tension resultant from fiber reinforcement $S_f := (h - \alpha d_{ys}) \cdot b_{fy} \cdot f_{Ftud.m} = 3642.7\ kN$

Capacity from fiber reinforcement $M_{Rd.f} := S_f \cdot (0.5 \cdot h + 0.1 \cdot \alpha d_{ys}) = 276.6\ kN \cdot m$

Capacity from traditional reinforcement $M_{Rd.s} := S_s \cdot (d_s - 0.4 \cdot \alpha d_{ys}) = 8\ kN \cdot m$

Total moment capacity $M_{Rd} := M_{Rd.f} + M_{Rd.s} = 284.6\ kN \cdot m$

Utilization $$u_m := \frac{M_{y.ULS}}{M_{Rd}} = 0.027$$

Very low utilization. Good capacity.

E.8 Tension force utilization

Tension force utilization only concrete

$$u_T := \frac{N_{Ed.T.ULS}}{A \cdot \frac{f_{ctk0.05}}{\gamma_c}} = 0.476$$

Tension force utilization only fiber reinforcement

$$u_T := \frac{N_{Ed.T.ULS}}{f_{Ftud} \cdot A} = 0.574$$

Tension force utilization only traditional reinforcement

$$u_T := \frac{N_{Ed.T.ULS}}{\frac{A_{s.eff}}{2} \cdot f_{yd}} = 0.594$$

Tension force utilization total

$$u_T := \frac{N_{Ed.T.ULS}}{A \cdot \frac{f_{ctk0.05}}{\gamma_c} + \frac{A_{s.eff}}{2} \cdot f_{yd} + f_{Ftud} \cdot A} = 0.181$$

F – Wind Load

F.1 Bridge geometry

Bridge width	$b := 2.62\ m$
Bridge full length	$L := 35\ m$
Bridge height	$h := 2\ m$
Reference area of bridge along x axis	$A_{ref.x} := h \cdot L = 70\ m^2$

F.2 Calculation

Terrain category III is chosen for this thesis.

Assumed height of bridge over ground	$z := 5\ m$
Maximum wind height	$z_{max} := 200\ m$
Minimum wind height (Table NA.4.1 from EC1)	$z_{min} := 5\ m$
Terrain roughness length (Table NA.4.1 from EC1)	$z_0 := 0.3\ m$
Terrain roughnes length for terrain category II	$z_{0.II} := 0.05\ m$
Terrain roughness factor depending on roughness length	$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.22$
Roughness factor at height z	$c_r := k_r \cdot \ln\left(\frac{z}{z_0}\right) = 0.61$
Terrain shape factor	$c_0 := 1.0$
Peak factor	$k_p := 3.5$

Density of air $\rho := 1.25\ \frac{kg}{m^3}$

Several basis wind speed factors $c_{dir} := 1.0$

$c_{season} := 1.0$

$c_{alt} := 1.0$

$c_{prob} := 1.0$

Reference wind speed at location (theoretical location, set value) $v_{b.0} := 28\ \frac{m}{s}$

Basis wind speed $v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b.0} = 28\ \frac{m}{s}$

Local mean wind velocity $v_m := c_r \cdot c_0 \cdot v_b = 16.97\ \frac{m}{s}$

Turbulence factor $K_I := 1.0$

Turbulence intensity $I_v := \dfrac{K_I}{c_0 \cdot \ln\left(\dfrac{z}{z_0}\right)} = 0.36$

Peak wind velocity pressure $q_p := \left(1 + 2 \cdot k_p \cdot I_v\right) \cdot 0.5 \cdot \rho \cdot {v_m}^2 = 0.63\ \frac{kN}{m^2}$

Exposure factor $C_e := \dfrac{q_p}{0.5 \cdot \rho \cdot {v_b}^2} = 1.28$

F.3 Wind forces

Recommended directional load factor for z direction $c_{f.z} := 0.9$

Recommended directional load factor for x direction $c_{f.x} := 1.5$

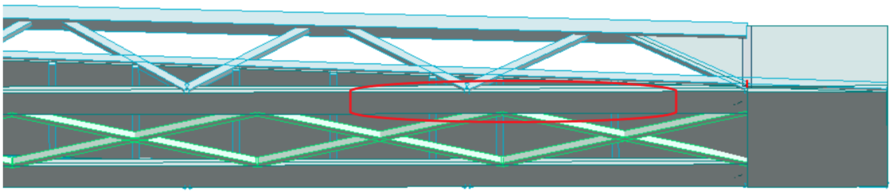
Force factor $C := C_e \cdot c_{f.x} = 1.921$

Force in x direction $F_{w.x} := \frac{1}{2} \cdot \rho \cdot {v_b}^2 \cdot C \cdot h = 1.88 \ \frac{kN}{m}$

Force in y direction $F_{w.y} := 0.25 \cdot F_{w.x} = 0.47 \ \frac{kN}{m}$

Force in z direction $F_{w.z} := q_p \cdot c_{f.z} \cdot b = 1.48 \ \frac{kN}{m}$

G – Design of Post-tensioned Chords



G.1 Chord geometry and concrete data

Chord length	$L := 35\ m$
Chord width	$b := 600\ mm$
Chord thickness	$t := 150\ mm$
Cover	$c := 25\ mm$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.7\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

G.2 Fiber reinforcement

See attachment A.2

G.3 Prestressed reinforcement

Characteristic proof strength	$f_{p0.1k} := 1650\ MPa$
Characteristic tensile strength	$f_{pk} := 1860\ MPa$
Ductility value	$k := \frac{f_{pk}}{f_{p0.1k}} = 1.127$
Characteristic strain at maximum force	$\varepsilon_{uk} := 3.5\%$
Prestressed reinforcement safety factor	$\gamma_s := 1.15$
Modulus of elasticity	$E_p := 195000\ MPa$
Dimensioning strength	$f_{pd} := \frac{f_{p0.1k}}{\gamma_s} = 1434.78\ MPa$
Greatest allowed jacking stress	$\sigma_{p.max} := min\left(0.8 \cdot f_{pk}, 0.9 \cdot f_{p0.1k}\right) = 1485\ MPa$
Initial strain difference	$\varepsilon_{p0} := \frac{\sigma_{p.max}}{E_p} = 0.0076$
Effective strain difference (assuming 15% loss)	$\varepsilon'_{p0} := 0.85 \cdot \varepsilon_{p0} = 0.006$
Cable diameter	$\text{Ø}_p := 16\ mm$
Cable duct diameter	$\text{Ø}_{p.duct} := 20\ mm$
Cable length	$L_p := L = 35.000\ m$
Number of cables	$n_p := 18$

G.4 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.8\ MPa$

Reinforcement bar diameter $Ø_s := 10\ mm$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars $d := t - c - \frac{10\ mm}{2} = 120.0\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 95.58\ mm^2$$

To meet this requirement there is enough with 2xØ10 traditional reinforcement bars.

Effective reinforcement area $A_{s.eff} := 2 \cdot 70\ mm^2 = 140.0\ mm^2$

G.5 Loads (FEM-Design)

G.5.1 Ultimate limit state

Moment in x direction (hogging)	$M_{x.HOG.ULS} := 44.3\ kN \cdot m$
Moment in x direction (sagging)	$M_{x.SAG.ULS} := 17.48\ kN \cdot m$
Moment in y direction (hogging)	$M_{y.HOG.ULS} := 41.6\ kN \cdot m$
Moment in y direction (sagging)	$M_{y.SAG.ULS} := 61.0\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.ULS} := 2607.4\ kN$
Axial force in x direction (tension)	$N_{x.T.ULS} := 1930.6\ kN$
Axial force in y direction (compression)	$N_{y.C.ULS} := 746.9\ kN$
Axial force in y direction (tension)	$N_{xyT.ULS} := 529.2\ kN$
Shear force in x direction	$V_{x.ULS} := 260.2\ kN$
Shear force in y direction	$V_{y.ULS} := 283.3\ kN$

G.5.2 Serviceability limit state

Moment in x direction (hogging)	$M_{x.HOG.SLS} := 34.1\ kN \cdot m$
Moment in x direction (sagging)	$M_{x.SAG.SLS} := 14.4\ kN \cdot m$
Moment in y direction (hogging)	$M_{y.HOG.SLS} := 33.9\ kN \cdot m$
Moment in y direction (sagging)	$M_{y.SAG.SLS} := 45.8\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.SLS} := 1983.4\ kN$
Axial force in x direction (tension)	$N_{x.T.SLS} := 1443.9\ kN$
Axial force in y direction (compression)	$N_{y.C.SLS} := 568.2\ kN$
Axial force in y direction (tension)	$N_{xyT.SLS} := 400.2\ kN$
Shear force in x direction	$V_{x.SLS} := 201.6\ kN$
Shear force in y direction	$V_{y.SLS} := 211.9\ kN$

G.6 Reinforcement forces

G.6.1 Prestressed reinforcement

Cross section area of cable $A_p := 150\ mm^2$

Additional stress for un-injected prestressed cables $\Delta\sigma_p := 100\ MPa$

Dimensioning force in prestressed reinforcement, per cable $S_{p.n} := \left(\varepsilon'_{p0} \cdot E_p + \Delta\sigma_p\right) \cdot \dfrac{A_p}{\gamma_s} = 177.7\ kN$

Dimensioning prestressed force for moment capacity $S_p := n_p \cdot S_{p.n} = 3198.3\ kN$

G.6.2 Traditional reinforcement

Force in effective minimum reinforcement $S_s := f_{yd} \cdot A_{s.eff} = 60.870\ kN$

G.6.3 Spacing of prestressed reinforcement bars

Spacing of post tensioning bars:

$A_{s.bun} := 3 \cdot A_p = 450.00\ mm^2$

Moment utilization $u_m := \dfrac{M_{x.HOG.ULS}}{M_{Rd}} = 0.05$

$\text{Ø}_{bun} := \sqrt{\dfrac{4}{\pi} \cdot A_{s.bun}} = 23.94\ mm$

Bars can be bundled together in 3s with a maximum distance between them as Ø_{bundle}.

$n_b := \dfrac{n_p}{3} = 6$

Total distance of bars and spaces: $s := n_p \cdot \text{Ø}_p + \text{Ø}_{bun} \cdot n_b = 431.619\ mm$

$<\quad b - 2\ c = 550.000\ mm$ OK

G.7 Moment capacity

Width of tension zone for fiber contribution $b_{fy} := 0.5 \cdot L = 17.5\ m$

Width of compression zone $b_y := 0.5 \cdot L = 17.5\ m$

Heigt of compression zone: $\alpha d_{ys} := \dfrac{t \cdot b_{fy} \cdot f_{Ftud.m} + S_s + S_p}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 13.1\ mm$

Cross section in tension, low height of compression zone is predictable. Also because of the heigh concrete strength and thus a smaller compression zone needed.

Effective height of prestressed reinforcement $d_p := t - c - \O_s - \dfrac{\O_{p.duct}}{2} = 105.0\ mm$

Effective height of traditional reinforcement $d_s := t - c - \frac{\text{Ø}_s}{2} = 120.0\ mm$

Tension resultant from fiber reinforcement $S_f := (t - \alpha d_{ys}) \cdot b_{fy} \cdot f_{Ftud.m} = 7094\ kN$

Capacity from fiber reinforcement $M_{Rd.f} := S_f \cdot (0.5 \cdot t + 0.1 \cdot \alpha d_{ys}) = 541.3\ kN \cdot m$

Capacity from traditional reinforcement $M_{Rd.s} := S_s \cdot (d_s - 0.4 \cdot \alpha d_{ys}) = 7\ kN \cdot m$

Capacity from prestressed reinforcement $M_{Rd.p} := S_p \cdot (d_p - 0.4 \cdot \alpha d_{ys}) = 319.1\ kN \cdot m$

Total moment capacity $M_{Rd} := M_{Rd.f} + M_{Rd.s} + M_{Rd.p} = 867.4\ kN \cdot m$

The bottom chords moment capacity is very much larger than the induced moment on the elements. That is because they are mainly to stiffen the entire bridge structure, as they are one of the main parts that counter the deflection of the bridge. The post tensioning solution raises the whole bridge a total of 7.1 mm short term and 6.8 mm long term.

G.8 Shear force capacity

Distance to middle of reinforcement bar

$$d := t - c - \frac{\varnothing_s}{2} = 120.0\ mm$$

Dimensioning shear force

$$V_{y.ULS} := 283.3\ kN$$

Induced shear stress on cross section

$$\tau_{Ed} := \frac{V_{y.ULS}}{b \cdot d} = 3.935\ MPa$$

Aggregate size. 20mm + Reinforcement bar diameter

$$d_{dg} := 20\ mm + \varnothing_s = 30.0\ mm$$

Minimum concrete shear resistance

$$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.60$$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):

$$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.251$$

Reinforcement ratio

$$\rho_l := \frac{A_{s.eff}}{b \cdot d} = 0.002$$

Shear force resistance without fiber

$$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$$

$$\tau_{Rd.c} := 0.68\ MPa$$

Shear force resistance with fiber contribution

$$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 4.68\ MPa$$

Shear force utilization

$$u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.84$$

G.9 Tension force utilization

Dimensioning tension force in ULS $N_{EdT} := N_{x.T.ULS} + \frac{M_{y.SAG.ULS}}{t} = 2337.3\ kN$

Concrete tension capacity $N_{RdT.c} := b \cdot t \cdot \frac{f_{ctk0.05}}{\gamma_c} = 214.44\ kN$

Traditional reinforcement tension capacity $N_{RdT.s} := A_{s.eff} \cdot f_{yd} = 60.87\ kN$

Prestressed reinforcement tension capacity $N_{RdT.p} := n_p \cdot S_{p.n} = 3198.3\ kN$

Fiber reinforcement tension capacity $N_{RdT.f} := b \cdot t \cdot f_{Ftud} = 177.6\ kN$

Total tension capacity $N_{RdT} := N_{RdT.c} + N_{RdT.s} + N_{RdT.p} + N_{RdT.f} = 3651.2\ kN$

Tension force utilization $u_T := \frac{N_{EdT}}{N_{RdT}} = 0.64$

G.10 Compression utilization

Good capacity for compression. Assumed to be appropriately supported in all directions due to lateral and horizontal trusses attached to both axes, which removes the problem of buckling of the chords. They are mainly in tension through the span of the bridge.

$$N_{RdC} := b \cdot t \cdot f_{cd} = 5100\ kN \quad \gg \quad N_{x.C.ULS} = 2607.4\ kN$$

Compression force utilization $u_C := \frac{N_{x.C.ULS}}{N_{RdC}} = 0.51$

G.11 Control for cracking in SLS

As the cross section contains bar reinforcement, there is a requirement for control of crack widths in the service limit state.

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $\O_s = 10\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min\left(c + 5\ \O_p, 10\ \O_p, 3.5\ c\right) = 87.5\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 52500\ mm^2$

Ratio of bond strength of prestressing and reinforcing steel [Tab. 10.1] $\xi := 0.6$

Adjusted ratio of bond strength taking into account the different diameters of prestressing and traditional reinforcing steel $\xi_1 := \xi \cdot \frac{\O_s}{\O_p} = 0.375$

Ratio of steel and concrete tension area $\rho_{c.ef} := \frac{A_{s.eff} + \xi_1 \cdot n_p \cdot A_p}{A_{c.eff}} = 0.022$

Largest distance between cracks

$$s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \frac{\O_s}{\rho_{c.ef}}\right) \cdot \left(1 - \frac{f_{Fts.ef}}{f_{ctm}}\right) = 52.4\ mm$$

Simplified calculation of stress in reinforcement bars (traditional + prestressed):

$N_{EdT} = 2337.3\ kN$

$$\sigma_s := \frac{N_{EdT}}{A_{s.eff} + n_p \cdot A_p} = 823.0\ MPa$$

Simplified reduction of σ_s to account for the fibers contribution. < 0,8 fyk for each bar [EN1992-1-1 TABLE 9.1])

$\sigma_s := 750\ MPa$

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term

$k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \frac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 0.0031$$

or

$$\varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 0.0021$$

$\varepsilon_{dif} := 0.0031$

Minimum concrete cover

$c_{min.dur} := 25\ mm$

Reduction factor for crack width limit

$$k_{surf} := \frac{c}{\left(10\ mm + c_{min.dur}\right)} = 0.714$$

Crack width limit

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.214\ mm$

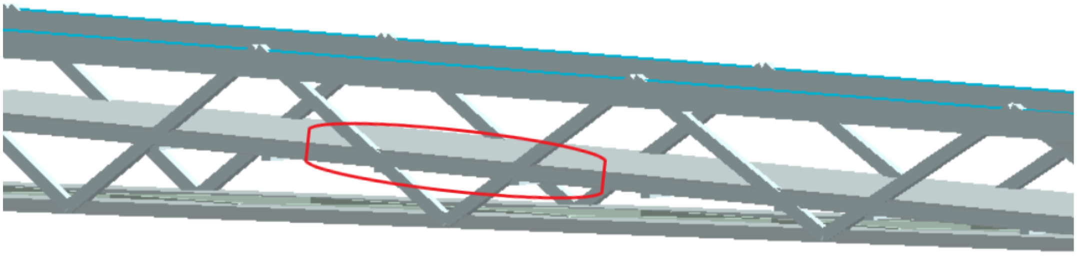
Crack width

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.162\ mm$

Utilization

$$u_w := \frac{w_k}{w_{lim}} = 0.76$$

H – Design of Interface Shear Connection



In the design of the bridge, there is longitudinal beams which goes through the whole length of the bridge and connects the transverse smaller beams that carries the bridge walkway deck. These longitudinal beams are cast inside the lateral diagonal truss, in such a way that the downwards forces are transmitted as tension or compression in the truss elements. There is not a significant shear force interface between these elements.

However, the transverse beams will need to be cast with shear force reinforcement connecting them to the longitudinal beam and lateral truss. This reinforcement is needed due to the forces coming from the walkway deck that is transmittet throughout the transverse beam pattern. These beams are however spaced close and the forces in each are not great.

Shear force at interface [FEM-Design] $V_{Ed} := 33.4\ kN$

Area of interface (area of transverse beam) $A_i := (120 \cdot 150)\ mm^2 = 18000\ mm^2$

Design value of interface shear force $\tau_{Ed} := \frac{V_{Ed}}{A_i} = 1.856\ MPa$

$$\tau_{Rdi} = c_{v1} \sqrt{(f_{ck})}/\gamma_c + \mu_v\, \sigma_n + \rho_i\, f_{yd}\,(\mu_v \sin\alpha + \cos\alpha) \;<\; 0{,}25 f_{cd}$$

Roughness factors (very smooth) $\mu_v := 0.5$ $C_{v1} := 0.0095$

Formula factors: $f_{ck} := 120\ MPa$ $\gamma_c := 1.5$ $\alpha := 90\ °$

$f_{yd} := 435\ MPa$

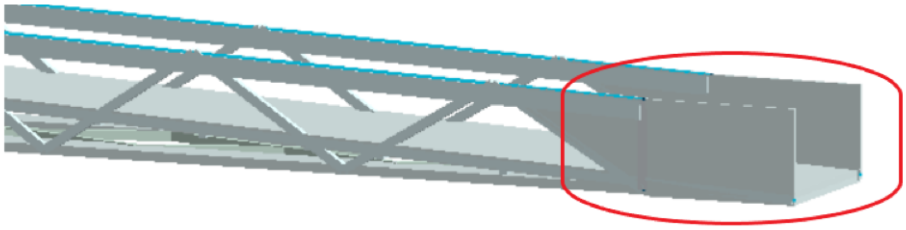
Necessary reinforcement ratio $\rho_i := \frac{\tau_{Ed} - \boxed{C_{v1} \cdot \frac{\sqrt{f_{ck}}}{\gamma_c}}}{f_{yd}\,(\mu_v \cdot \sin(\alpha) + \cos(\alpha))}$ $\rho_i := 0.00841$

Necessary interface reinforcement $A_{si} := \rho_i \cdot A_i = 151.38\ mm^2$

Present reinforcement in transverse beam $A_s := 2 \cdot 79\ mm^2 = 158\ mm^2$

The present reinforcement in transverse beams can be anchored in the perpendicular concrete elements and create a satisfactory shear force reinforcement for the induced shear force in ultimate limit state. While combining them with Ø8 or Ø10 stirrups there will be sufficient reinforcement. The extra anchoring length of the present reinforcement will satisfy the interface shear capacity by EN1992-1-1 8.2.6.

I – Design of Support Walls



I.1 Wall geometry and concrete data

Wall length	$L := 2.5\ m$
Wall thickness	$t := 120\ mm$
Wall height	$h := 1.4\ m$
Cover	$c := 25\ mm$
Critical buckling length	$L_{cr} := 2.5\ m$
Cross section area	$A := t \cdot h = 0.168\ m^2$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.667\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

I.2 Fiber reinforcement

See attachment A.2.

I.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000 \ MPa$

Characteristic yield strength $f_{yk} := 500 \ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.8 \ MPa$

Reinforcement bar diameter $Ø_s := 24 \ mm$

Strain $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars (assuming Ø16 reinforcement) $d := t - c - \frac{16 \ mm}{2} = 87 \ mm$

Minimum reinforcement per meter in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot h \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot h \cdot d}{f_{yk}}\right) = 161.689 \ mm^2$$

However:

Minimum reinforcement for fiber reinforcement horizontal walls:

$$A_{s.min} := 0.3 \cdot A \cdot \frac{\left(f_{ctm} - f_{Ftud}\right)}{f_{yk}} = 315.75 \ mm^2$$

$$A_{s.eff} := 6 \cdot 452 \ mm^2 = 2712 \ mm^2$$

Effective necessary reinforcement after tension control.

I.4 Loads (FEM-Design)

I.4.1 Ultimate limit state

Moment in y direction	$M_{y.ULS} := 56.1\ kN \cdot m$
Moment in x direction	$M_{x.ULS} := 14.2\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.ULS} := 1719.3\ kN$
Axial force in x direction (tension)	$N_{x.T.ULS} := 4068.4\ kN$
Axial force in y direction (compression)	$N_{y.C.ULS} := 769.3\ kN$
Axial force in y direction (tension)	$N_{y.T.ULS} := 485.6\ kN$
Shear force in x direction	$V_{x.ULS} := 62.7\ kN$
Shear force in y direction	$V_{y.ULS} := 323.8\ kN$

I.4.2 Serviceability limit state

Moment in y direction	$M_{y.SLS} := 42.3\ kN \cdot m$
Moment in x direction	$M_{x.SLS} := 10.8\ kN \cdot m$
Axial force in x direction (compression)	$N_{x.C.SLS} := 1308.2\ kN$
Axial force in x direction (tension)	$N_{x.T.SLS} := 3092.1\ kN$
Axial force in y direction (compression)	$N_{y.C.SLS} := 585.3\ kN$
Axial force in y direction (tension)	$N_{y.T.SLS} := 368.9\ kN$
Shear force in x direction	$V_{x.SLS} := 48.1\ kN$
Shear force in y direction	$V_{y.SLS} := 246.3\ kN$

I.5 Moment capacity about y axis (strong axis)

Force in effective minimum reinforcement $S_s := f_{yd} \cdot A_{s.eff} = \left(1.18 \cdot 10^{3}\right)\ kN$

Width of tension zone $b_{fy} := 0.5 \cdot L = 1.3\ m$

Width of compression zone $b_y := 0.5 \cdot L = 1.3\ m$

Heigt of compression zone: $$\alpha dy_{ys} := \frac{h \cdot b_{fy} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 105.3\ mm$$

Because of the high concrete strength, a smaller compression zone develops.

Effective height of traditional reinforcement $d_s := h - c - \frac{Ø_s}{2} = 1363\ mm$

Tension resultant from fiber reinforcement $S_f := \left(h - \alpha dy_{ys}\right) \cdot b_{fy} \cdot f_{Ftud.m} = 4790.2\ kN$

Capacity from fiber reinforcement $M_{Rd.f} := S_f \cdot \left(0.5 \cdot h + 0.1 \cdot \alpha dy_{ys}\right) = 3403.6\ kN \cdot m$

Capacity from traditional reinforcement $M_{Rd.s} := S_s \cdot \left(d_s - 0.4 \cdot \alpha dy_{ys}\right) = \left(1.6 \cdot 10^{3}\right)\ kN \cdot m$

Total moment capacity $M_{Rd.y} := M_{Rd.f} + M_{Rd.s} = 4961.1\ kN \cdot m$

Utilization $$u_m := \frac{M_{y.ULS}}{M_{Rd.y}} = 0.01$$

Very low utilization. Standing walls have good capacity against moment about the strong axis.

I.6 Moment capacity about x axis (weak axis)

Force in effective minimum reinforcement (same reinforcement in x direction, but not room for full capacity. Assuming conservatively only 5% reinforcement capacity.)

$$S_s := 0.05\ f_{yd} \cdot A_{s.eff} = 58.96\ kN$$

Width of tension zone

$$b_{fy} := \frac{1.4\ m}{2} = 0.7\ m$$

Width of compression zone

$$b_y := \frac{1.4\ m}{2} = 0.7\ m$$

Heigt of compression zone:

$$\alpha dx_{ys} := \frac{t \cdot b_{fy} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 9.1\ mm$$

Because of the high concrete strength, a smaller compression zone develops. A low amount of effective reinforcement also contributes to the low value.

Effective height of traditional reinforcement

$$d_s := t - c - \frac{Ø_s}{2} = 83\ mm$$

Tension resultant from fiber reinforcement

$$S_f := \left(t - \alpha dx_{ys}\right) \cdot b_{fy} \cdot f_{Ftud.m} = 229.8\ kN$$

Capacity from fiber reinforcement

$$M_{Rd.f} := S_f \cdot \left(0.5 \cdot t + 0.1 \cdot \alpha dx_{ys}\right) = 14.0\ kN \cdot m$$

Capacity from traditional reinforcement

$$M_{Rd.s} := S_s \cdot \left(d_s - 0.4 \cdot \alpha dx_{ys}\right) = 4.7\ kN \cdot m$$

Total moment capacity

$$M_{Rd.x} := M_{Rd.f} + M_{Rd.s} = 18.7\ kN \cdot m$$

Utilization

$$u_m := \frac{M_{x.ULS}}{M_{Rd.x}} = 0.76$$

I.7 Axial force capacity in x direction

Axial force cross section area $A_N := t \cdot h = 0.168\ m^2$

Dimensioning tension force $N_{Ed.T} := \frac{M_{y.ULS}}{h} + N_{x.T.ULS} = 4108.5\ kN$

Dimensioning compression force $N_{Ed.C} := \frac{M_{y.ULS}}{h} + N_{x.C.ULS} = 1759.4\ kN$

Tension resultant from fiber reinforcement (tension force) $S_f := t \cdot h \cdot f_{Ftud} = 331.5\ kN$

Force in effective minimum reinforcement $S_s := 2 \cdot 1.4\ f_{yd} \cdot A_{s.eff} = 3301.57\ kN$

Reinforcement net with $A_{s.eff}$ per meter on both sides with 1.4m effective height.

Tension capacity of concrete $N_{Rd.cs} := t \cdot h \cdot f_{ctk0.05} = 600.44\ kN$

Necessary area of reinforcement (per meter) to withstand the remaining tension force $A_{s.eff} := \frac{N_{Ed.T} - S_f - N_{Rd.cs}}{2 \cdot 1.4 \cdot f_{yd}} = 2609.28\ mm^2$

New reinforcement, 6xØ24 per meter. cc175mm or cc200mm. $A_{s.eff} := 6 \cdot 452\ mm^2 = 2712\ mm^2$

Dimensioning tension capacity $N_{Rd.T} := S_f + N_{Rd.cs} + 2 \cdot 1.4 \cdot f_{yd} \cdot A_{s.eff} = 4233.52\ kN$

Dimensioning compression capacity (without reinforcement) $N_{Rd.C} := h \cdot t \cdot f_{cd} = 9520\ kN$

Utilizations:

Tension utilization $u_T := \frac{N_{Ed.T}}{N_{Rd.T}} = 0.97$

Compression utilization $u_C := \frac{N_{Ed.C}}{N_{Rd.C}} = 0.18$

I.8 Axial force capacity in y direction

Axial force cross section area $A_N := t \cdot h = 0.168\ m^2$

Dimensioning tension force $N_{Ed.T} := \dfrac{M_{x.ULS}}{t} + N_{y.T.ULS} = 603.9\ kN$

Dimensioning compression force $N_{Ed.C} := \dfrac{M_{x.ULS}}{t} + N_{y.C.ULS} = 887.6\ kN$

Tension resultant from fiber reinforcement $S_f := \left(h - \alpha dx_{ys}\right) \cdot t \cdot f_{Ftud.m} = 494\ kN$

Force in effective minimum reinforcement $S_s := f_{yd} \cdot A_{s.eff} = \left(1.18 \cdot 10^3\right)\ kN$

Reinforcement net with $A_{s.eff}$ per meter on both sides with 1.4m effective height.

Dimensioning tension capacity $N_{Rd.T} := S_f + S_s = 1673.18\ kN$

Dimensioning compression capacity (without reinforcement) Assuming flat load application $N_{Rd.C} := L \cdot h \cdot f_{cd} = 198333.333\ kN$

Utilizations:

Tension utilization $u_T := \dfrac{N_{Ed.T}}{N_{Rd.T}} = 0.361$

Compression utilization $u_C := \dfrac{N_{Ed.C}}{N_{Rd.C}} = 0.004$

Most axial force working on the edge of the element making the moment about the x axis. Compression utilization will be neglected here.

I.9 Shear force capacity

Distance to middle of reinforcement bar

$$d := h - c - \frac{Ø_s}{2} = 1363\ mm$$

Dimensioning shear stress on cross section

$$\tau_{Ed} := \frac{V_{y.ULS}}{t \cdot d} = 1.98\ MPa$$

Aggregate size. 20mm + Reinforcement bar diameter

$$d_{dg} := 20\ mm + Ø_s = 44\ mm$$

Minimum concrete shear resistance

$$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 0.57$$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):

$$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.25$$

Reinforcement ratio

$$\rho_l := \frac{A_{s.eff} \cdot 1.4}{t \cdot d} = 0.02$$

As x 1.4 because of the reinforcement being per meter and the effective height is 1.4m.

Shear force resistance without fiber

$$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$$

$$\tau_{Rd.c} := 0.74\ MPa$$

Shear force resistance with fiber contribution

$$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 4.92\ MPa$$

Shear force utilization:

$$u_s := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.4$$

I.10 Control for cracking in SLS

As the cross section contains bar reinforcement, there is a requirement for control of crack widths in the service limit state.

$c := 25\ mm$ $\qquad$ $k_b := 0.8$ (good fastening)

$f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $\quad h_{c.eff} := min\left(c + 5\ Ø_s, 10\ Ø_s, 3.5\ c\right) = 87.5\ mm$

Effective area of concrete in tension (per meter) $\quad A_{c.eff} := t \cdot h_{c.eff} = 0.01\ m^2$

Ratio of steel and concrete tension area $\quad \rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.26$

$$s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \frac{Ø_s}{\rho_{c.ef}}\right) \cdot \left(1 - \frac{f_{Fts.ef}}{f_{ctm}}\right) = 0.02\ m$$

$N_{Y.C} := 1318.1\ kN$ $\qquad$ $N_{x.TSLS} := 3092.1\ kN$ $\qquad$ $M_{y.SLS} = 42.3\ kN \cdot m$

Tension stress in reinforcement bars $\quad \sigma_s := \dfrac{\dfrac{N_{x.TSLS}}{2} + \dfrac{M_{y.SLS}}{h}}{1.4 \cdot A_{s.eff}} = 415.16\ MPa$

Reduced stress for fiber contribution $\quad \sigma_s := 400.0\ MPa$

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term $\quad k_t := 0.4$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \dfrac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \dfrac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 1.82 \cdot 10^{-3}$$ or: $$\varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 1.14 \cdot 10^{-3}$$

$$\varepsilon_{dif} := 1.82 \cdot 10^{-3}$$

Minimum concrete cover $c_{min.dur} := 25 \ mm$

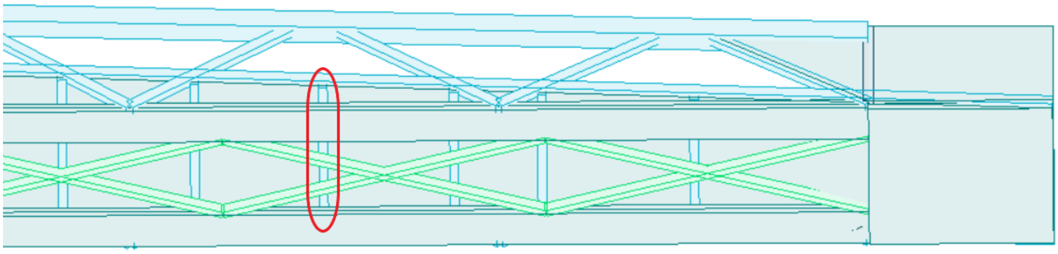
Reduction factor for crack width limit $k_{surf} := \dfrac{c}{\left(10 \ mm + c_{min.dur}\right)} = 0.714$

Crack width limit $w_{lim} := 0.3 \ mm \cdot k_{surf} = 0.214 \ mm$

Crack width $w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.041 \ mm$

Utilization $u_w := \dfrac{w_k}{w_{lim}} = 0.19$

J – Design of Transverse Deck Beams



J.1 Bar geometry and concrete data

Bar geometry

Bar length	$L := 2.62\ m$
Bar width	$b := 120\ mm$
Bar height	$h := 150\ mm$
Cover	$c := 25\ mm$
Area	$A := b \cdot h = 18000\ mm^2$

Concrete data

Material factor	$\gamma_c := 1.5$
Characteristic compressive strength	$f_{ck} := 100\ MPa$
Medium tensile strength	$f_{ctm} := 1.1 \cdot \sqrt[3]{100}\ MPa = 5.11\ MPa$
Dimensioning compressive strength	$f_{cd} := \dfrac{0.85 \cdot f_{ck}}{\gamma_c} = 56.667\ MPa$
Modulus of elasticity	$E_{cm} := 45000\ MPa$
Yield strain for compression	$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$
Characteristic tension stength	$f_{ctk0.05} := 0.7 \cdot f_{ctm} = 3.57\ MPa$

J.2 Fiber reinforcement

See attachment A.2.

J.3 Passive reinforcement and minimum reinforcement

Modulus of elasticity $E_s := 210000\ MPa$

Characteristic yield strength $f_{yk} := 500\ MPa$

Material factor $\gamma_s := 1.15$

Dimensioning yield strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.8\ MPa$

Reinforcement bar diameter $Ø_s := 10\ mm$

Strain $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Minimum reinforcement [NB38]

Assumed distance from center reinforcement bars (assuming Ø16 reinforcement) $d := h - c - \frac{10\ mm}{2} = 120\ mm$

Minimum reinforcement in a fiber reinforced concrete cross section to maintain capacity after cracking:

$$A_{s.min} := \max\left(0.26 \cdot b \cdot d \cdot \frac{\left(f_{ctm} - 2.15 \cdot f_{Ftud}\right)}{f_{yk}}, \frac{0.13 \cdot f_{ctm} \cdot b \cdot d}{f_{yk}}\right) = 19.12\ mm^2$$

To meet this requirement there is enough with 2xØ10 traditional reinforcement bars.

Effective reinforcement area $A_{s.eff} := 2 \cdot 79\ mm^2 = 158\ mm^2$

J.4 Loads (FEM-Design)

J.4.1 Ultimate limit state

Moment in y direction (sagging) $M_{y.ULS} := 0.6\ kN \cdot m$

Moment in z direction $M_{z.ULS} := 0.15\ kN \cdot m$

Axial force (tension) $N_{Ed.ULS} := 15.1\ kN$

Shear force $V_{Ed.ULS} := 1.7\ kN$

J.4.2 Serviceability limit state

Moment in y direction $M_{y.SLS} := 0.5\ kN \cdot m$

Moment in z direction $M_{z.SLS} := 0.12\ kN \cdot m$

Axial force (tension) $N_{Ed.SLS} := 11.5\ kN$

Shear force $V_{Ed.SLS} := 1.3\ kN$

J.5 Shear force capacity

Distance to middle of reinforcement bar

$$h' := d - c - \frac{10\ mm}{2} = 90.0\ mm$$

Dimensioning shear force

$$V_{Ed.ULS} = 1.7\ kN$$

Induced shear stress on cross section

$$\tau_{Ed} := \frac{V_{Ed.ULS}}{b \cdot h'} = 0.16\ MPa$$

Aggregate size. 20mm + Reinforcement bar diameter

$$d_{dg} := 20\ mm + Ø_s = 30.0\ mm$$

Minimum concrete shear resistance

$$\tau_{Rdc.min} := \frac{10}{\gamma_c} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} = 1.6$$

Factor for reducing/increasing the concretes effect on the capacity while having fiber reinforement (fFtud = 1.973):

$$\eta := \frac{1}{1 + 0.43 \cdot 1.973^{2.85}} = 0.25$$

Reinforcement ratio

$$\rho_l := \frac{A_{s.eff}}{b \cdot d} = 0.011$$

Shear force resistance without fiber

$$\tau_{Rd.c} := \frac{0.6}{\gamma_c} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\frac{1}{3}}$$

$$\tau_{Rd.c} := 1.21\ MPa$$

Shear force resistance with fiber contribution

$$\tau_{Rd.cF} := \frac{\tau_{Rd.c}}{\eta} + f_{Ftud} = 6.79\ MPa$$

Shear force utilization

$$u_{shear} := \frac{\tau_{Ed}}{\tau_{Rd.cF}} = 0.02$$

J.6 Moment capacity

Force in effective minimum reinforcement $S_s := f_{yd} \cdot A_{s.eff} = 68.7\ kN$

Width of tension zone $b_{fy} := 0.5 \cdot L = 1.3\ m$

Width of compression zone $b_y := 0.5 \cdot L = 1.3\ m$

Heigt of compression zone:

$$\alpha d_{ys} := \frac{h \cdot b_{fy} \cdot f_{Ftud.m} + S_s}{0.8 \cdot b_y \cdot f_{cd} + b_{fy} \cdot f_{Ftud.m}} = 10.3\ mm$$

Cross section in tension, low height of compression zone is predictable. Also because of the heigh concrete strength and thus a smaller compression zone needed.

Effective height of traditional reinforcement

$$d_s := h - c - \frac{Ø_s}{2} = 120\ mm$$

Tension resultant from fiber reinforcement

$$S_f := \left(h - \alpha d_{ys}\right) \cdot b_{fy} \cdot f_{Ftud.m} = 541.8\ kN$$

Capacity from fiber reinforcement

$$M_{Rd.f} := S_f \cdot \left(0.5 \cdot h + 0.1 \cdot \alpha d_{ys}\right) = 41.2\ kN \cdot m$$

Capacity from traditional reinforcement

$$M_{Rd.s} := S_s \cdot \left(d_s - 0.4 \cdot \alpha d_{ys}\right) = 8\ kN \cdot m$$

Total moment capacity

$$M_{Rd} := M_{Rd.f} + M_{Rd.s} = 49.2\ kN \cdot m$$

Utilization

$$u_m := \frac{M_{y.ULS}}{M_{Rd}} = 0.012$$

Very low utilization. Good capacity.

J.7 Control for cracking in SLS

Cover $c := 25\ mm$

Reinforcement fastening factor $k_b := 0.8$ (good fastening)

Reinforcement bar diameter $Ø_s = 10\ mm$

Uniaxial fiber residual tensile strength in SLS, determined with k0 = 1.0 $f_{Fts.ef} := 3.6\ MPa$

Effective height of concrete in tension: $h_{c.eff} := min\left(c + 5\ Ø_s, 10\ Ø_s, 3.5\ c\right) = 75.0\ mm$

Effective area of concrete in tension $A_{c.eff} := b \cdot h_{c.eff} = 9000\ mm^2$

Ratio of steel and concrete tension area $\rho_{c.ef} := \dfrac{A_{s.eff}}{A_{c.eff}} = 0.018$

Largest distance between cracks

$$s_{r.max.cal} := \left(2\ c + 0.35 \cdot k_b \cdot \frac{Ø_s}{\rho_{c.ef}}\right) \cdot \left(1 - \frac{f_{Fts.ef}}{f_{ctm}}\right) = 61.8\ mm$$

Simplified calculation of stress in reinforcement bars

$$N_{EdT} := \frac{M_{y.SLS}}{h - 2\ c} + \frac{N_{Ed.SLS}}{2} = 10.8\ kN$$

$$\sigma_s := \frac{N_{EdT}}{\dfrac{A_{s.eff}}{2}} = 136.1\ MPa$$

Factor depending on the duration of the load. Equal to 0.6 for short-term load and 0.4 for long term $k_t := 0.4$

Steel modulus of elasticity $E_s := 210000\ MPa$

Concrete modulus of elasticity $E_{cm} := 45000\ MPa$

Strain ratio:

$$\varepsilon_{dif} := \frac{\sigma_s - k_t \cdot \dfrac{f_{ctm}}{\rho_{c.ef}} \cdot \left(1 + \dfrac{E_s}{E_{cm}} \cdot \rho_{c.ef}\right)}{E_s} = 4.863 \cdot 10^{-5} \qquad \text{or} \qquad \varepsilon_{dif} := 0.6 \cdot \frac{\sigma_s}{E_s} = 3.888 \cdot 10^{-4}$$

$\varepsilon_{dif} := 3.888 \cdot 10^{-4}$

Crack width:

$c_{min.dur} := 25\ mm$

$$k_{surf} := \frac{c}{(10\ mm + c_{min.dur})} = 0.71$$

$w_{lim} := 0.3\ mm \cdot k_{surf} = 0.21\ mm$

$w_k := s_{r.max.cal} \cdot \varepsilon_{dif} = 0.02\ mm$

Utilization $u_w := \dfrac{w_k}{w_{lim}} = 0.112$

J.8 Tension force utilization

Tension force utilization only concrete

$$u_T := \frac{N_{EdT}}{A \cdot \dfrac{f_{ctk0.05}}{\gamma_c}} = 0.251$$

Tension force utilization only fiber reinforcement

$$u_T := \frac{N_{EdT}}{f_{Ftud} \cdot A} = 0.303$$

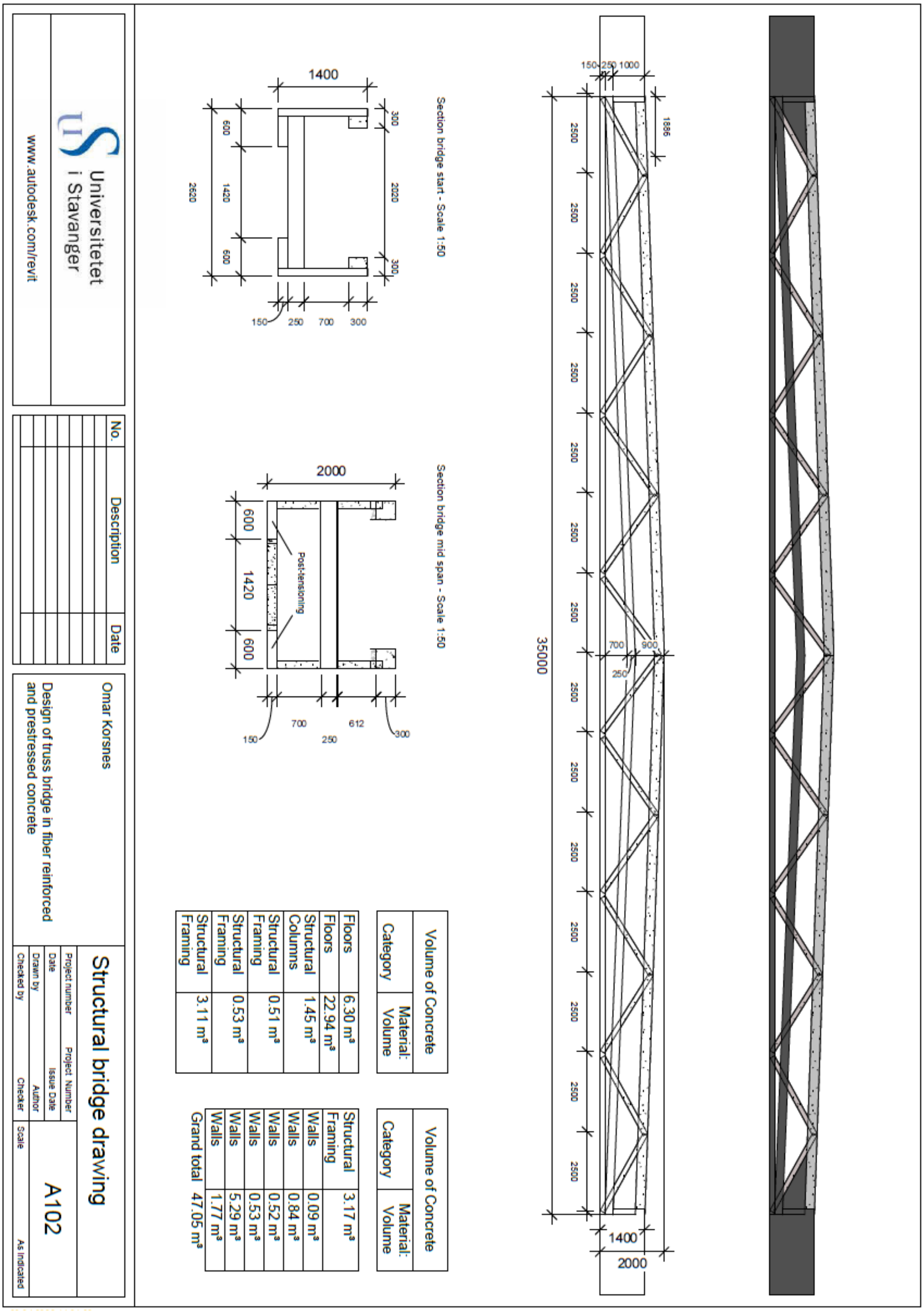
Tension force utilization only traditional reinforcement

$$u_T := \frac{N_{EdT}}{A_{s.eff} \cdot f_{yd}} = 0.156$$

Tension force utilization total

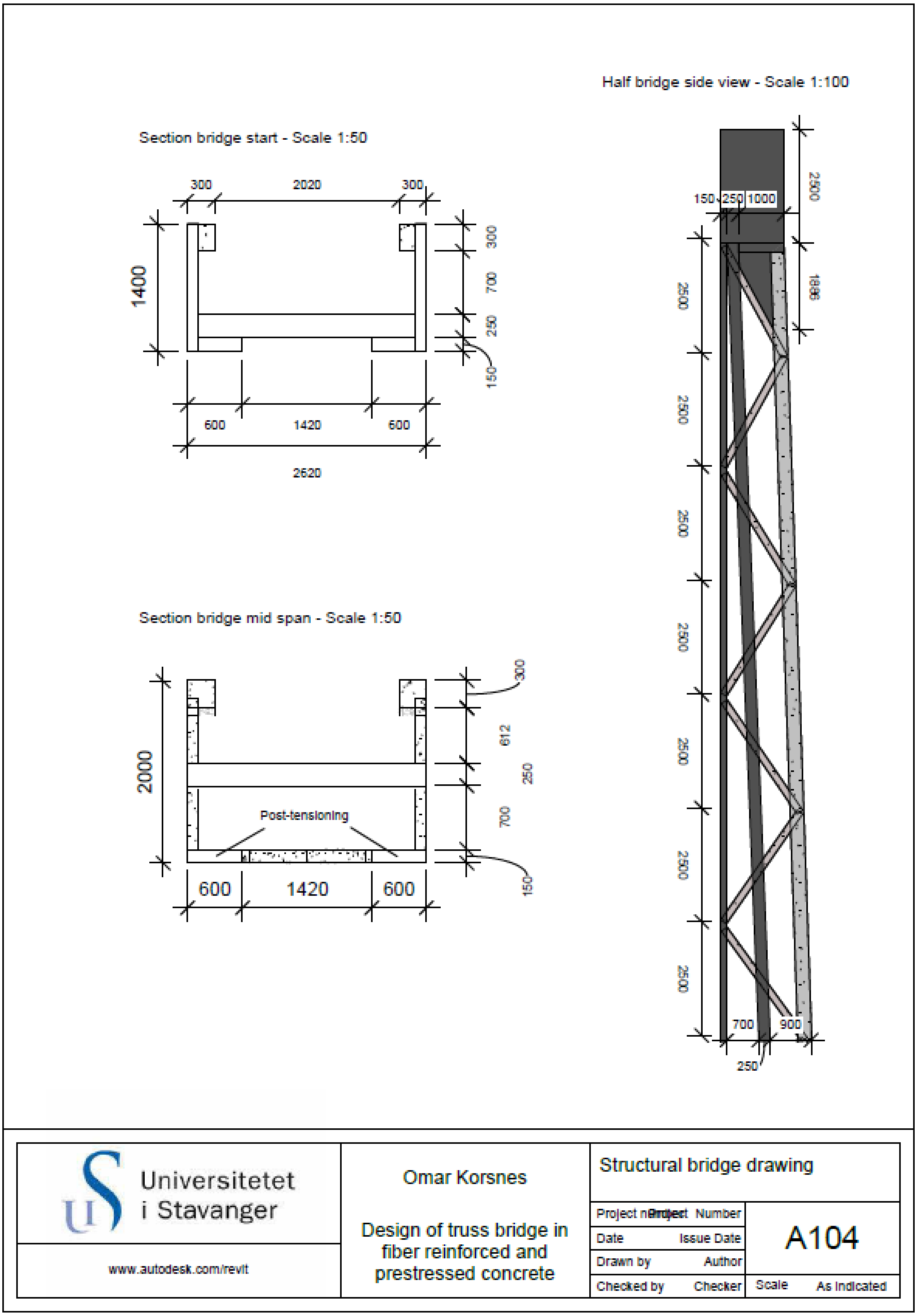
$$u_T := \frac{N_{EdT}}{A \cdot \dfrac{f_{ctk0.05}}{\gamma_c} + A_{s.eff} \cdot f_{yd} + f_{Ftud} \cdot A} = 0.073$$

K – Bridge design Revit drawings



Volume of Concrete	
Category	Material: Volume
Floors	6.30 m³
Floors	22.94 m³
Structural Columns	1.45 m³
Structural Framing	0.51 m³
Structural Framing	0.53 m³
Structural Framing	3.11 m³

Volume of Concrete	
Category	Material: Volume
Structural Framing	3.17 m³
Walls	0.09 m³
Walls	0.84 m³
Walls	0.52 m³
Walls	0.53 m³
Walls	5.29 m³
Walls	1.77 m³
Grand total	47.05 m³


Half bridge side view - Scale 1:100
Section bridge start - Scale 1:50
300
2020
300
1400
300
700
250
150
600
1420
600
2620
Section bridge mid span - Scale 1:50
2000
300
612
250
700
150
Post-tensioning
600
1420
600
2500
150
250
1000
1886
2500
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i Stavanger
www.autodesk.com/revit
Omar Korsnes
Design of truss bridge in fiber reinforced and prestressed concrete
Structural bridge drawing
Project number
Project Number
Date
Issue Date
Drawn by
Author
Checked by
Checker
A104
Scale
As indicated