Universitetet
i Stavanger

FACULTY OF SCIENCE AND TECHNOLOGY

MASTER'S THESIS

Study programme/specialization: Konstruksjoner og materialer /byggkonstruksjoner	Spring / Autumn semester, 2017 Open/~~Confidential~~
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Title of master's thesis: Structural performance of a prefabricated concrete beam with longitudinal cavities	
Credits: 30	
Keywords: Prefabricated concrete elements Non-linear Finite Element Method Continuous beam Weight reduction of beams	Number of pages: 51 + supplemental material/other: 52-150 Stavanger, 15/06-17 date/year

Title page for Master's Thesis
Faculty of Science and Technology

Summary

Precast concrete elements are getting more and more popular in the construction industry. The precast elements are casted into many different shapes and sizes, which includes walls, beams, columns, slabs and more. There are many advantages of using them, although there are also many limitations. Moreover, concrete elements pass different stages such as casting, storing, transporting and disassembling. Especially the last two stages, brings several limitations regarding size and especially weight to the picture.

A specific project turned out to have some problems regarding the weight of a large continuous beam with cross-shaped (special DLB) cross section, carrying hollow decks on both corbels, created the motivation for the topic of this thesis. Many different ideas on how to reduce the weight of the beam were mentioned. Use of lightweight concrete, use pf pre-stressing beams, hollow sections, geometrical changes and many others were proposed. Inspiration from hollow decks and IB-beams created the final idea of using longitudinal cavities as a measure. Originally, the beam was designed using a Gerber system, a system which allows for splitting a continuous beam in parts. It also had transverse cavities. However, for simplicity, the original beam has been re-designed for a case without either Gerber or transverse cavities.

The beams have been designed using rules given in Eurocode 2 [1] and design guidelines given in the concrete element book [2] [3]. Also, a publication [4] regarding where the Euler-Bernoulli beam theory is valid for concrete structures has been used in the design phase. Practical design with regards to production has had a very high focus. Preliminary control and calculations for design were mostly done using simple software [5] for design of concrete structures directly towards Eurocode 2 with Norwegian annex.

Through discussion and trial & error, a design with a cavity below the neutral axis were proposed, which then maintained almost all its theoretical bending moment capacity (cracked section assumed). As the initial beam were assumed to be sufficiently constrained in the lateral direction, the shear capacity then turned out to be the main issue for design. Reduction in shear capacity introduced further limits to where the design may be used.

The initial beam cross section consists of a rectangular beam 520mm*980mm with rectangular corbels 150mm*250mm on each side starting at 350mm above the most bottom part. Space for two layers of ø32 in both top and bottom, shear rebar ø16, and corbel rebar both in the transverse ø12 and longitudinal direction. For the modified cross section, the bottom part of corbels was angled at 45 degrees. A cylindrical cavity ø300 were added and corbel rebar adjusted accordingly. Longitudinal and shear rebar were unaffected. The cavities were added at a distance equivalent to effective depth, "d", of the cross section from the face of each support, unless the shear capacity of the cross section were violated, in which it was moved accordingly. In sum, these modifications reduced the total weight of the continuous beam by ~9%.

To study the structural behavior of the proposed beam, numerical analysis was then carried out using a non-linear finite element software (i.e. ATENA [6]) specifically aimed at concrete structures. Two models were produced for comparison, and two extra for control of shear rebar and mesh size. All the numerical results produced a way too high crack width, though all models mostly passed criteria for deflection, crushing of concrete and yielding of rebar. Comparatively, the higher range of values in the modified beam had mostly ~5-10% higher values for crack width, deflection and compressive stress in concrete.

In sum, this design approach seems to be a good measure, though the economical and other aspects have not been analyzed in detail. Further studies must be performed for a proper conclusion.

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Preface

First and foremost, I would like to thank Norconsult in Stavanger for the office space during the thesis. Second, I would like to thank my supervisor, Tore Larsen Aspmo and the rest of the civil engineering division at Norconsult in Stavanger, for help and tips during the thesis.

I would also like to direct a thank you to my supervisor at University of Stavanger (UiS), Samindi Samarakoon for valuable tips and help with writing the thesis. Additionally, I would like to thank UiS for lending me a license for both ATENA and GiD.

A thank you is also given to Dobromil Pryl at Cervenka Consulting for valuable help through email in learning ATENA- GiD.

Lastly, I would like to thank my family, friends and fellow students for a happy time while studying.

The topic of the thesis was not easily chosen as trying out a completely new approach to something is very hard. A lot of time were used during preliminary stages for actually figuring out what to do, simplifications to make and where to place the focus, which has been extremely frustrating at times. This has shown me a lot of what initial studies of figuring out a new method may involve in terms of work load.

Additionally, before the thesis, I knew very little about using a non-linear finite element software, non-linear finite element analysis in general and using the strut & tie model, along with other topics. Also, I had very few people to ask regarding the use of ATENA as it was only recently introduced to UiS, which caused a lot of user error. Consequently, the learning curve has been very steep, but I am happy for what I have learned during my time writing.

The thesis has been written as if the reader is a fellow student.

1. Introduction

1.1. Background

1.1.1. About precast concrete elements

Precast concrete elements are getting more and more popular within the construction industry. Many structures where it is normal to cast in-situ are now set up using either precast elements or a hybrid. The background for using precast elements is that they represent a rational, economic and timesaving method of construction. [7]

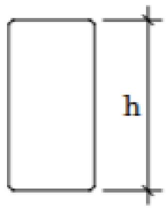
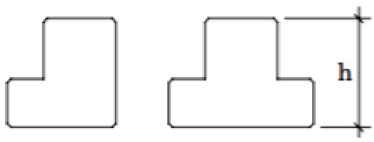
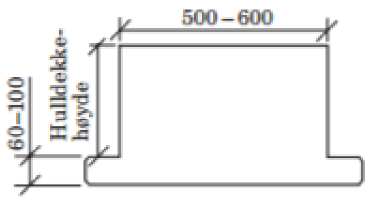
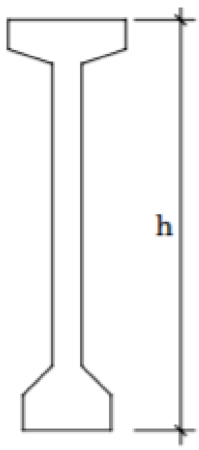
Regardless of all the advantages of precast concrete elements, it is important to know that there are many limitations to their use. For instance, precast elements are very easily subjected to geometrical and placement imperfections, which may cause eccentricities and different resistance conditions. Buildings with precast elements also tend to behave differently with respect to transverse loads from earthquakes and wind.

Figure 1 – Precast elements [7]

1.1.2. Precast reinforced concrete beams

Precast beams are easily produced in a high variety of c/s (cross section) types and sizes, some of which are given in Table 1.

Table 1 – Some of the most usual beams used for buildings [7]

BEAM TYPE	DESIGNATION	SKETCH (mm)	NORMAL C/S HEIGHT (mm)	NORMAL SPAN WIDTHS (not maximum) (m)
Rectangular	RB		300-800	4-12
Rectangular flange beam	LB DLB		300-800	4-12
Rectangular low flange beam with special corbel (LB/DLB for hollow decks)	LFB		260-500	4-8
I-beam	IB		600-2000	10-30

1.1.3. Challenges and possible solutions for precast reinforced concrete beams

Although very large and long beams can be precast at factories, as may be seen in Table 1, they must be transported to and at the site in a safe and economical manner. Transportation and assembling of long and heavy beams could easily become very expensive and difficult. Especially continuous beams are subjected to these problems.

A possible way to solve issues regarding transportation and assembling of continuous beams, is to use a Gerber system. This system allows for splitting the beam in parts and still maintain most of the benefits of continuous beams. Although it seems very versatile at first glance, it causes some problems by itself.

Weight reduction is also a measure which may solve some problems. It may be done using light weight aggregate, pre-stressed concrete, hollow sections. Regardless of method, weight reduction usually has a backside in terms of loss in capacity or increase in cost.

1.2. Scope of the thesis

The goal is to optimize a pre-designed continuous concrete beam with respect to weight. The cross section of the beam initially shaped like a cross and is supposed to carry the weight of hollow decks on its corbels. The cross section will be changed by using a cylindrical cavity through parts of the beam. The corbels will also be slightly modified.

Pre-calculations are either done by hand or by use of software called K-bjelke (bjelke=beam) and E-bjelke [5], which uses Eurocode 2 with Norwegian annex [1] directly. Numerical calculations will be done using a non-linear finite element software, i.e. ATENA [6]. Characteristic loading will be applied for the numerical analysis.

Then the results for the modified design will be compared with the results from the initial design.

1.3. Objective

The reference beam is a continuous beam with a total length 46.79 m, consisting of 5 spans of lengths ranging from 6m to 12m. The beam consists of 5 parts using a Gerber system. The main part the cross section is 520mm*980mm. Geometric details are given in 2.6 and 3.2.

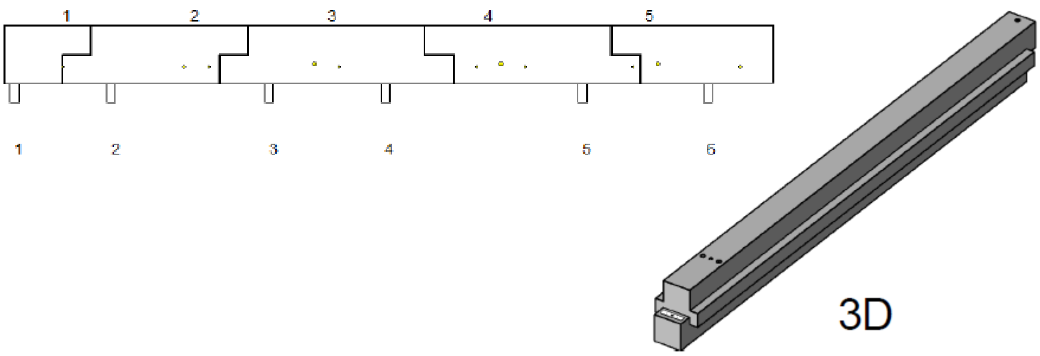


Figure 2 – (a) continuous beam (b) part of the beam, original design

The task will be to analyze the specific case of a continuous span for possible modifications in terms of weight reduction. One measure is to be analyzed in detail for comparison with the original design. To ease the analysis, the beam is modelled as a continuous beam, i.e. without joints. This will most notably cause a fall in moments, which may be seen in Figure 3, and in consequence, also the required main reinforcement.

The reason for the change in moments is that while using pinned joints, the zero moment will be forced into a specific position regardless of unfavorable distributed loads. Comparatively, the zero moment may move freely in a continuous beam.

The initial beam also has several transverse cavities, which in general lowers the capacity. These will be disregarded in the analysis.

All changes will be adjusted for in calculations.

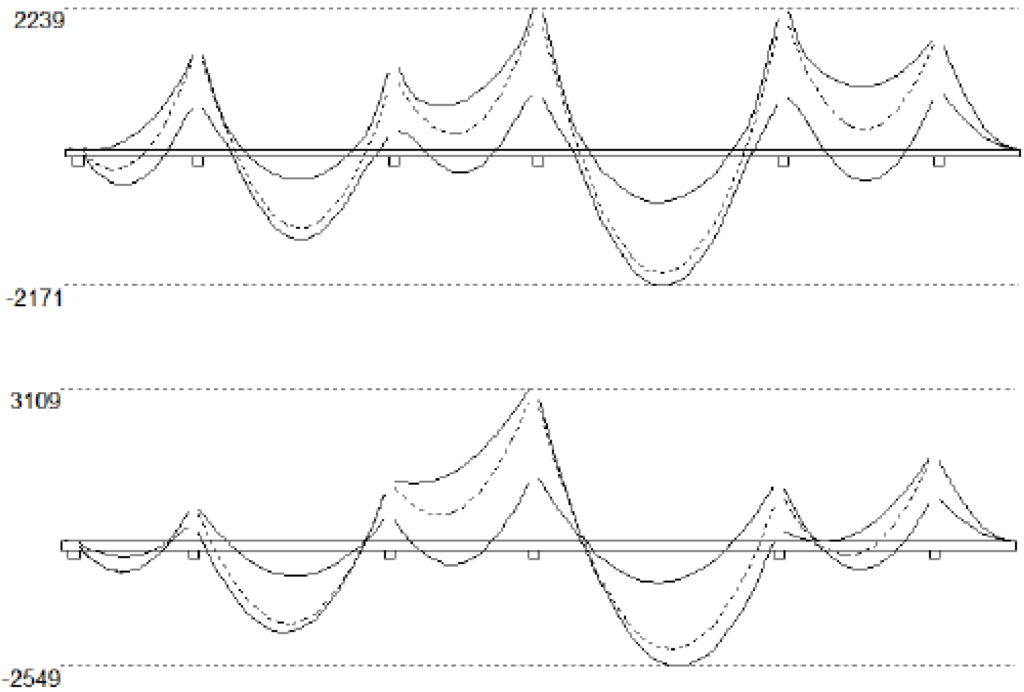


Figure 3 – Bending moment envelope without and with joints (kNm)

1.4. Weight reduction measures for precast concrete beam

Figure 4 shows an overview of the specific case. The structure which is to be built is enclosed in thick lines. The position of the crane is limited to two specific locations shown with purple circles. The distance of which the crane can carry a certain load is also limited, this is shown by using circles around the crane positions. However, some concrete beams' position and weight is not in favor of the selected crane, most notably to the right part in which the cranes cannot reach. One solution is to use a larger, but costlier crane, a measure which is generally not preferable. Weight reduction is another alternative which may be both cheaper and easier, and therefore preferable.

Detailed overview of planned beam element distribution may be found in 8.5.

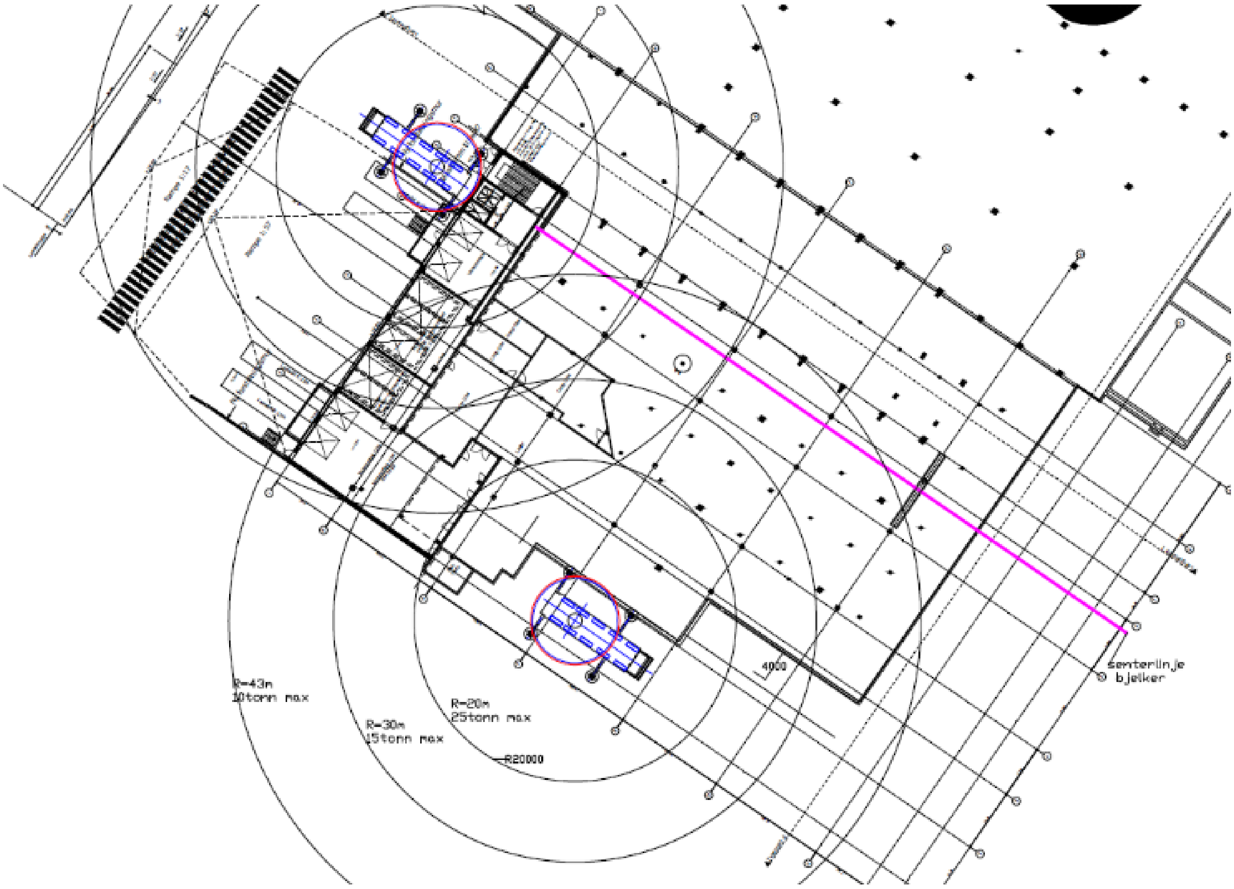


Figure 4 - Crane circles

There are many possible ways to reduce the weight in concrete beams. Some includes pure geometrical alterations, others include changes within the materials and so on. Many methods may even be combined. However, with every advantage some disadvantages always come along for the ride. That being either loss in capacity or increase in cost.

Some weight reduction measures/examples/ideas include:

- Lightweight aggregate
- Hollow sections
    - Transverse cavity
    - Longitudinal cavity

  - Bubble cavity
- Geometrical changes
- Steel profile within (i.e. composite steel and concrete beam)
- Pre-stressed concrete

1.5. Limitations

Limitations are mainly set by NS-EN 1992-1-1:2004+NA:2008 [1] along with Eurocodes above it. Design guidelines are given in Betongelementboken, mostly B [2] and C [3], and Betongkonstruksjoner [8].

In design, the main measure is only to be applied where Euler-Bernoulli beam theory is valid. This is to avoid the critical shear area, which is within a distance equivalent to the effective depth, "d", from the face of the supports and face joints. However, this is only in addition to avoid conflicts with required stiffening at the column support. [4]

The proposed cavity will also be placed below the equivalent compressive stress block, in positive bending, for both cracked and un-cracked section to limit reduction in capacity and increase of crack width and deflection.

2. Literature review

2.1. Developments in precast concrete beams

Through history, many different c/s types and sizes has been used in practice, some of which can be seen in Table 1. There are fewer and fewer limits in possibilities in making precast beams. However, the main limitation has mostly been the effectiveness in production. Though for instance, a new method to save resources is pre-set steel formwork for frequent beam geometries.

Regarding structural analysis, continuous beams have significantly less deformations than the corresponding simply supported beams, one can therefore reduce building height. This has led to an increase in use of floor-high columns and continuous beams, especially in combination with low corbel heights for support of hollow decks. [3]

2.2. Method of continuous beams

2.2.1. Gerber system

Using Gerber systems is a normal way to ease transportation and montage of precast continuous beams. Most notably, it allows for splitting the beam in parts. Splitting is done at approximately zero moment in the BMD (Bending Moment Diagram) and with as low shear force as possible. The parts are then joined together in-situ.

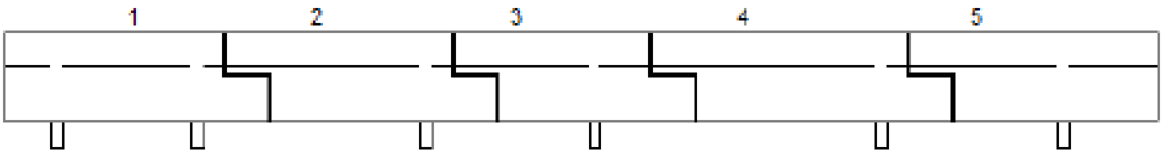


Figure 5 - Gerber system example

Regardless of the usefulness of a Gerber system, it will cause some changes and there are some extra measures required. For instance, the joints must be designed in specific ways. The joints are also mainly supposed to transfer vertical shear force and are therefore designed accordingly. This will lock the zero moments in place, regardless of the load distribution, and consequently cause changes in the Bending Moment Diagram.

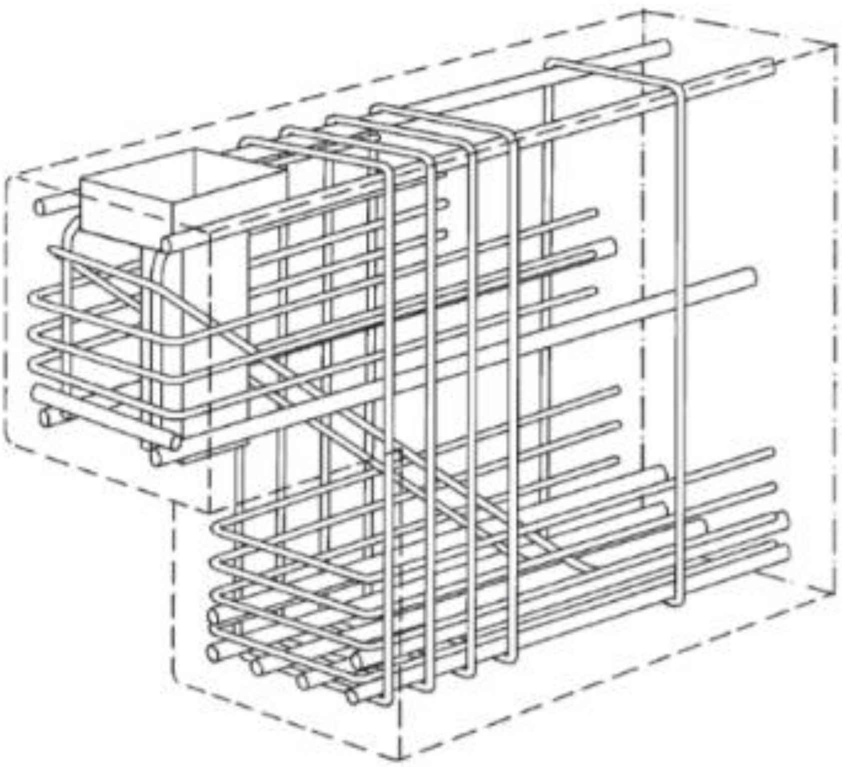


Figure 6 - Reinforced joint [3]

2.2.1.1. Joints

There are many different types of joints for concrete beams, two of which are shown in Figure 7. These joints are normally calculated using strut & tie models and require additional reinforcement. This reinforcement will usually take up a lot of space, which is exemplified in Figure 6.

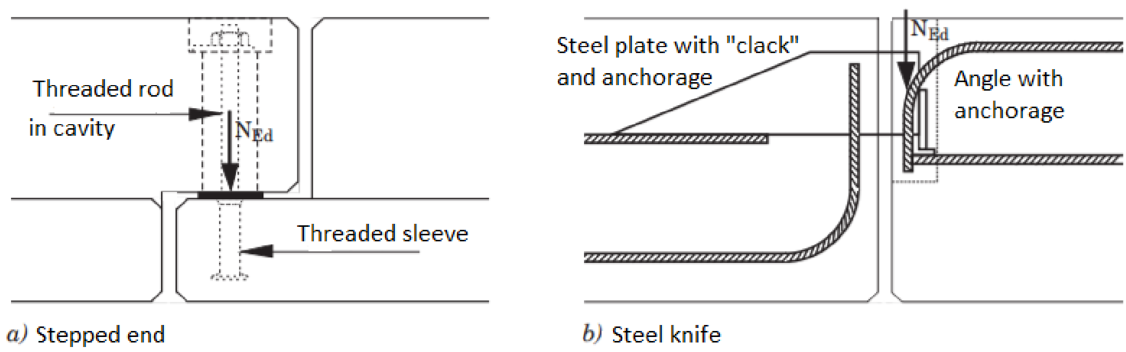


Figure 7 - Examples of joints in continuous beams at zero moment [3]

2.2.2. Transport, storage and assembling

When the beam parts are transported and stored, the supports are generally not the same size nor at the same place the final supports. Therefore, the beam parts must also be designed to withstand different moment distributions compared to the final placement. Although in these preliminary stages, self-weight is the main concern.

When assembling, the loadings may also be very different then the final one. For instance, all precast elements are not placed at the same time, but in steps. Therefore, eccentric loading may occur and cause torsional moments and other unwanted effects. This is normally solved using temporary supports or other various methods.

One cannot assume that the elements will always be made or placed with millimeter precision every time. To counter this, as with cast in-situ elements, tolerances should be initiated. Tolerances are usually held through conservatism in design, i.e. worst-case scenario. [3]

2.2.3. Corbels on beams

Corbels on beams may be modelled similarly to corbels on columns. Referring to Figure 8, a_0/d should be in the range of 0.4 to 0.6 to reduce effects of tolerance deviations and for practical rebar placement. [3]

As with joints, corbels are usually calculated using strut and tie models. Depending on the length vs height of the corbel, it may rather be designed as a cantilever beam.

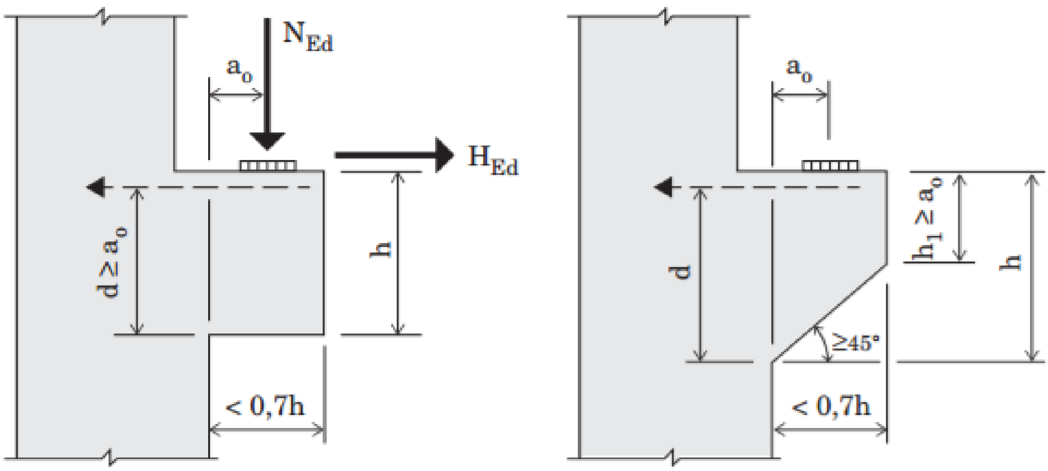


Figure 8 - Recommended design of corbels [3]

Figure 9 shows a simplified model of the functional principle of a corbel. The model is made using elastic models. Not all trajectories are shown.

As one can see, the bottom outermost part of the corbel is inactive. This makes room for reduction in cross sectional area. Though, rectangular shapes are usually made due to the fact that they are easier to both calculate and produce. These are resource uses which usually outweigh the cost of resources saved in materials in these cases. However, 45 degree angles are not very much harder to accomplish.

Precast elements will at some point be lifted out of the formwork, a process which causes friction and vacuum between the element and the formwork. This effect is usually very small, but is important to be aware of. Small surface area is therefore preferred.

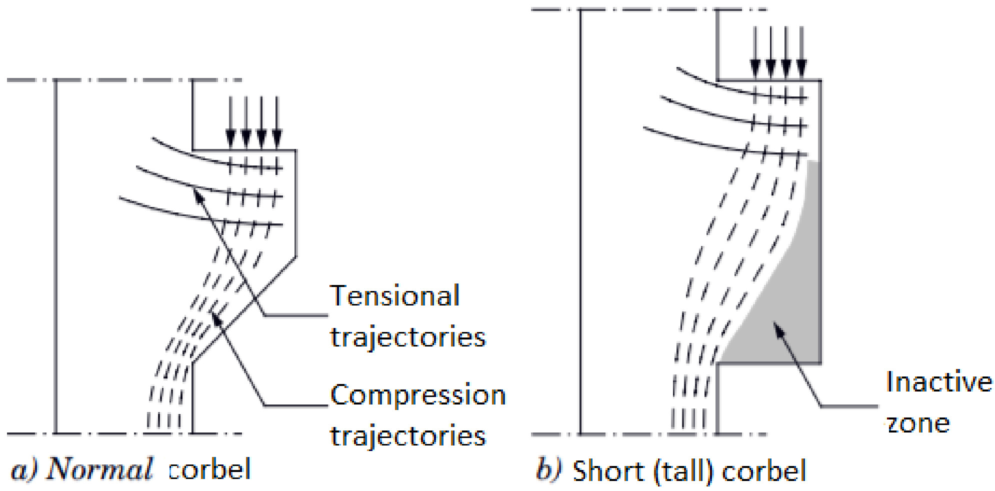


Figure 9 - Elastic model of corbel [3]

2.2.4. B- and D-regions

Regions in which the Bernoulli hypothesis of plane strain distribution is assumed valid, may be referred to as B-regions (beam or Bernoulli regions) as given in Figure 10. In these regions, internal state of stress can easily be derived from sectional forces, i.e. bending, shear, torsion and axial.

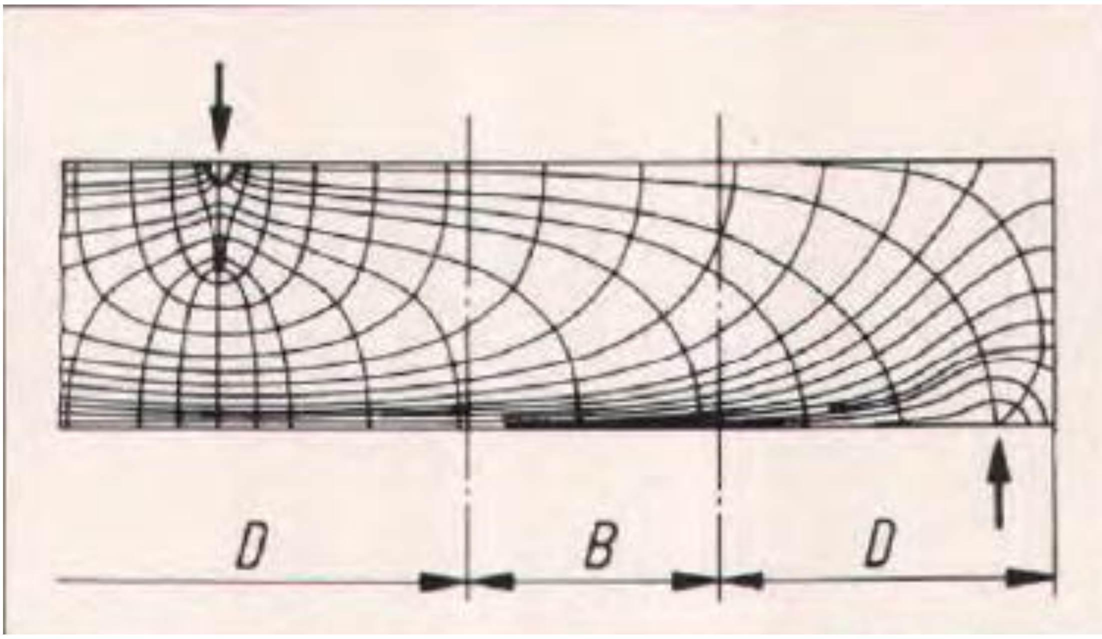


Figure 10 - Stress trajectories in B- and D-regions [4]

However, the standard methods are not applicable to all other regions and details of structures, where strain distribution is strictly non-linear. They are not applicable near concentrated loads, bends, corners, openings, and other discontinuities. These regions may be referred to as D-regions (discontinuity, disturbance, or detail).

For beams, the D-region extends from the discontinuity itself to a distance equivalent to the depth of the beam. Normally, cracked section design is applied when designing beams, this induces an effective depth, “d”, of the beam which is the distance from the top compressive fiber to the centroid of the tensional rebar. If cracked section design is assumed, the discontinuity region reduces to “d” from the support. [4]

2.3. Distortional Energy Density (von Mises) Criterion

The von Mises Criterion is a failure criteria which applies to ductile materials like metals. It states that, for a single point, yielding begins when the distortional strain-energy density is equal to distortional strain-energy density when the material yields in uniaxial compression/tension. In short, it is a method which accounts for all principal stresses into one single value for control. It is similar to the Maximum shear-stress criterion (Tresca). [9]

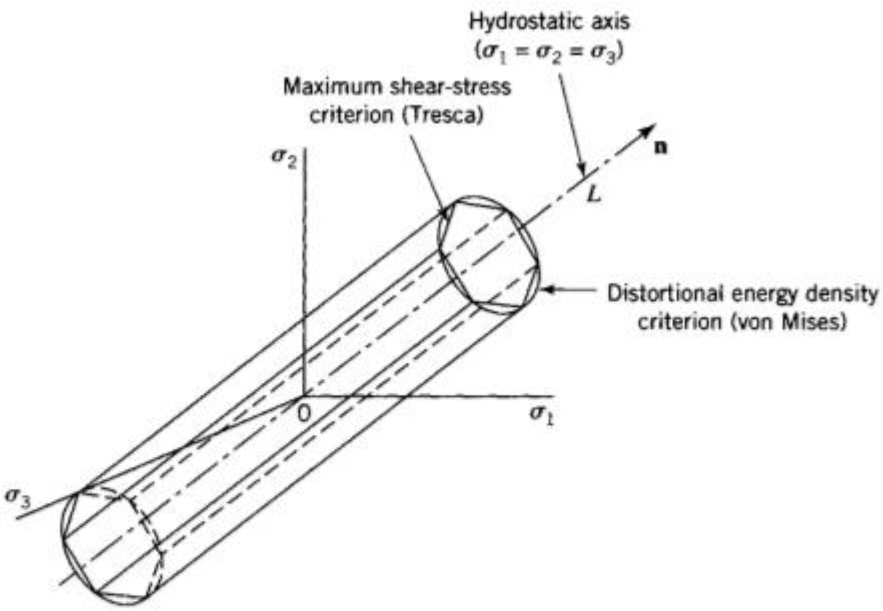


Figure 11 - Yield surface in principal stress space [9]

2.4. Numerical modelling of precast concrete beams

Concrete has a non-linear stress/strain distribution, and unlike steel, its capacity differs dramatically in compression vs tension and shear. For instance, although very over-simplified, it may be said that the strength in tension is approximately 10% of strength in compression. Concrete is also a material which changes over time and many different properties and adjustments comes into play. In further addition, concrete is often combined with reinforcement steel bars to counter its low shear and tensional capacity. Due to, but not limited by, these factors, concrete is a difficult material to model correctly.

The finite element method (FEM) is a very popular tool to use for structural analysis. A method which builds on the principle of splitting a structure into smaller, but finite elements and nodes, at which parameters like stress, strain, displacements and many others may be calculated. FEM is a tool which may also be used for non-linear analysis, which is very welcome in terms of concrete structures. FEM is available through a wide range of software.

A reliable element for the task should be chosen to avoid bad results. Elements vary in both geometry and order, including, combinations of line segments, triangular shapes, quadrilateral shapes, and a jungle of others. In bending, the use of quadrilateral shapes is preferred over triangular ones. This is mainly due that triangular elements like triangles or tetrahedral are very stiff in bending and fails to display displacements in a proper way. 3-dimensional quadrilateral elements may be called hexahedral or brick elements, which is illustrated in Figure 13. However, hexahedral elements may not show other values properly. To sum it up, there are no “one size fits all” in FEM.

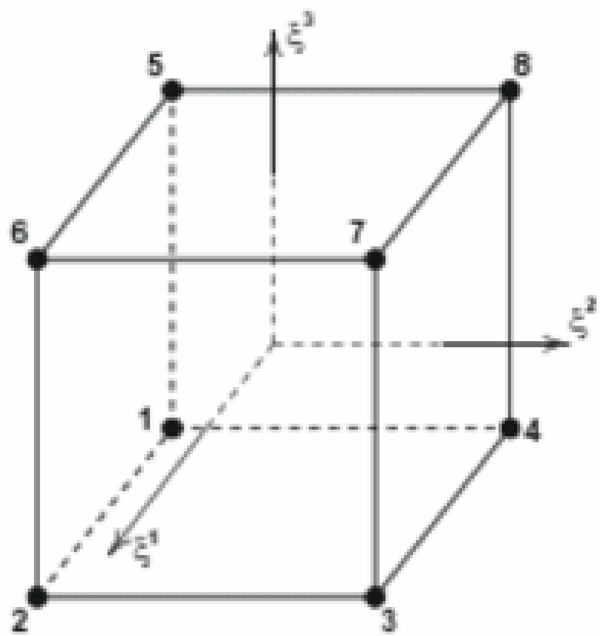


Figure 12 - Hexahedral/brick element [13]

Sizes and number of elements are very important aspects of FEM. How fine a mesh needs to be depends largely on the case, type of elements, and what results are interesting. A too fine mesh may also cause wrong results. For instance, stresses are usually too high when using a too fine mesh. Though, probably the most important aspect regarding number of elements is that a higher number of elements generally causes the need for more computing power. For this reason, applying symmetry where possible is a good practice. The mesh may also be refined at important areas, which includes among others, at corners, concentrated loads or other input, or changes in general geometry or material.

For bending, an appropriate minimum of elements per length is 4 to 6. For instance, this may be visualized through a simply supported beam, where a one can see that the stiffness approximates sufficiently at 4+ elements. However, this also bring the aspects ratio into the picture, which is exemplified in Figure 13. To avoid computational problems within an element, the longest length should not exceed 10 times the shortest length.

Just to notify, this is only an extremely short and simplified summary of FEM, how it works and some important properties to be aware of. There are many books of several hundred pages which explains it in further detail.

[10] [11] [12] [13]

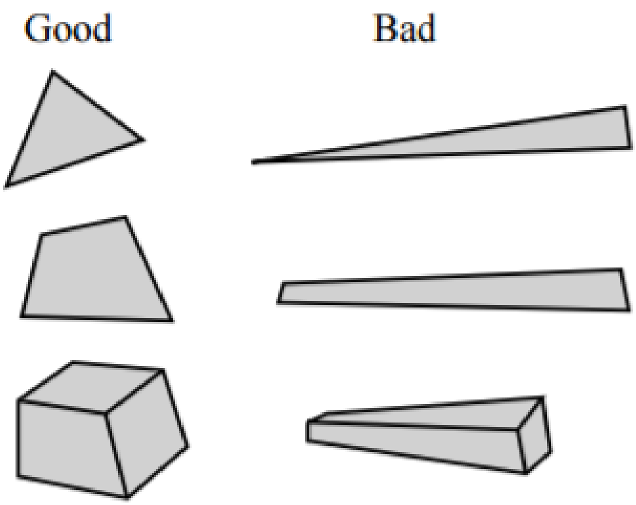


Figure 13 - Aspects ratio, good vs bad [11]

2.5. Non-linear FE analysis using ATENA

2.5.1. Using ATENA

ATENA is a FEM software specifically designed for nonlinear analysis of reinforced concrete structures. Good default values for parameters in concrete structures are given and the software also allows for control towards EC2 [1]. [6]

Recommendations regarding 3D beam elements

- A minimum of 4 elements per thickness
- A minimum of 4 to 6 elements per length
- Limit aspects ratio to a maximum of 4

[14] [15]

Theory may be found in [16].

Pre-processing (up to and including meshing) in ATENA builds on a non-linear FEM software called GiD, which is initially a very general software. ATENA places a plugin within GiD, which applies possibilities of using pre-defined materials like concrete, reinforcement bar and others. Meshing may be done in either a structured, unstructured or semi-structured manner. Semi-structured means for example unstructured cross section and structured in the longitudinal direction.

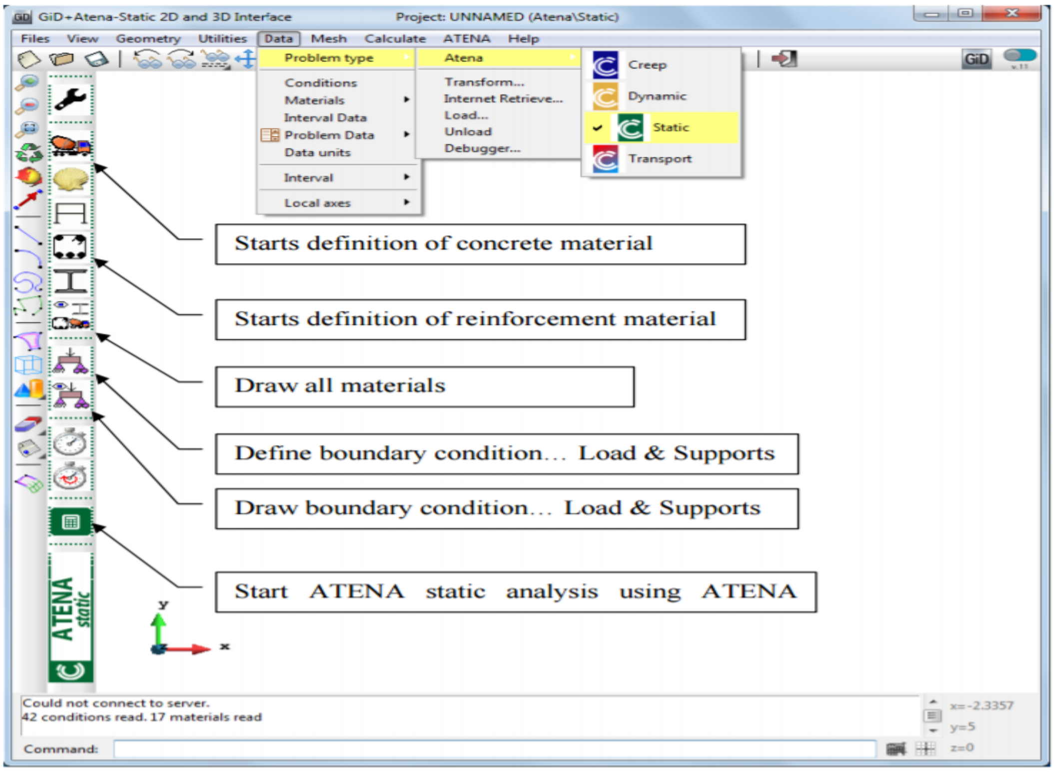


Figure 14 - Graphical user interface of ATENA-GiD [14]

Post-processing (after meshing) is done through the ATENA software itself which opens a new window. In this interface, one may analyze several results like displacement, crack width, and other useful results. It is also possible, through monitors, to create graphs and simulations for analysis of the behavior of the structure.

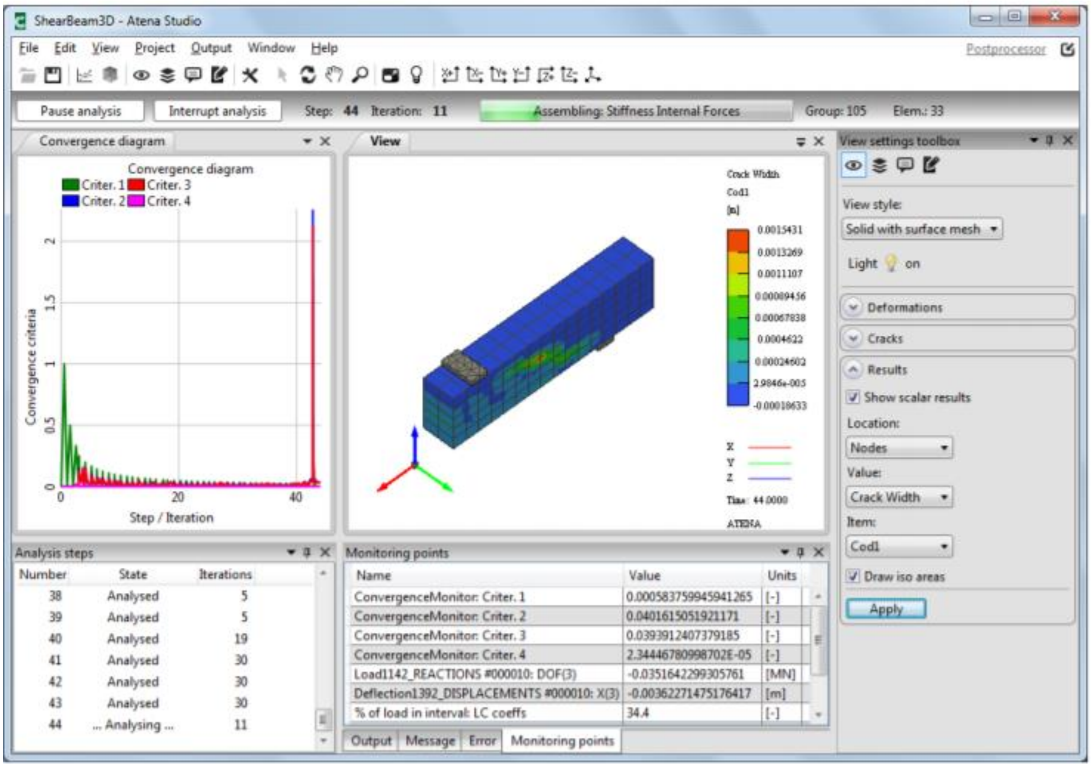


Figure 15 - ATENA interface [14]

GiD also has its own post-processor, though compared to ATENA, it is missing some tools for analysis.

2.5.2. Material models in ATENA

Concrete has non-linear behavior and one should therefore design accordingly. In ATENA, the biaxial stress state of concrete is described using a so-called effective stress σ_c^{ef}, along with equivalent uniaxial strain ε^{eq}. The effective stress is in most cases one of the principal stresses. Equivalent uniaxial strain is introduced to eliminate the Poisson's effect in plane stress state.

$\varepsilon^{eq} = \frac{\sigma_{ci}}{E_{ci}}$ may be considered as the strain produced by the governing stress, σ_{ci}, through a uniaxial test with an elastic modulus E_{ci} in the direction i. This assumes that nonlinearity representing a damage is only caused by σ_{ci}.

A complete equivalent uniaxial stress-strain diagram for concrete is given in Figure 16. Numbers within the diagram are used in results to indicate damage of the concrete. [16]

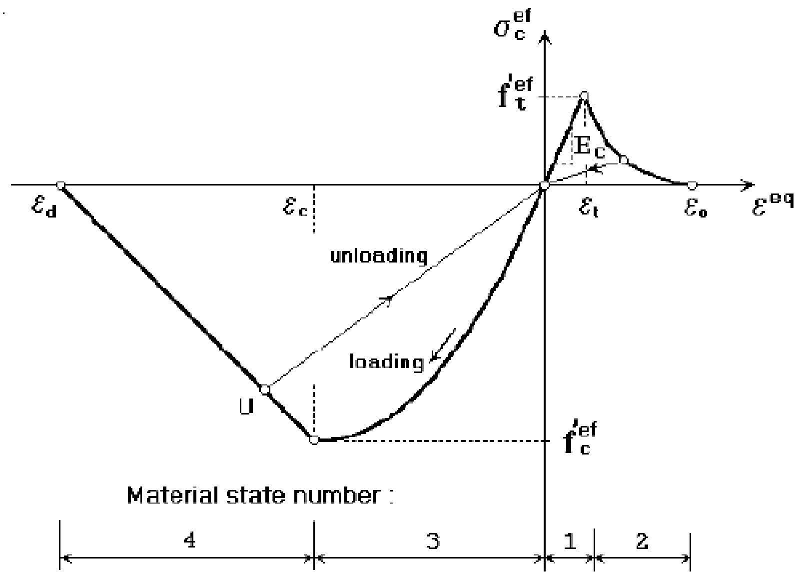


Figure 16 -Uniaxial stress-strain law for concrete [16]

Reinforcement on the other hand may be assumed to be linear in the elastic state. However, in the plastic state, steel is not linear, something which ATENA is able to consider approximately. Reinforcement in ATENA may be modelled as either discrete or smeared. Discrete modelling models the rebar as truss elements. Smeared modelling models the rebar in layers, i.e. for beams, all rebar in one layer of rebar is assumed evenly distributed along the breadth. Linear stress-strain behaviour is assumed for both cases.

ATENA may use either a bilinear or a multi-linear behavior for reinforcement. The multi-linear behavior allows for modelling of all four stages of steel behavior; elastic, yield plateau, hardening and fracture. The multi-linear behavior is shown in Figure 17. [16]

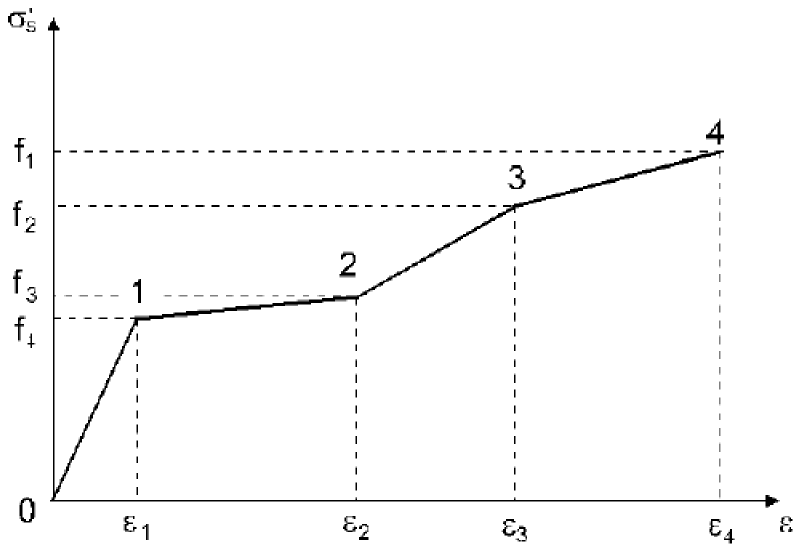


Figure 17 - Multi-linear stress-strain law for reinforcement [16]

Discrete model along a multi-linear stress-strain distribution with 2 multi-linear values are used in the thesis.

2.5.3. Cracking models in ATENA

Two approaches for crack modelling are available in ATENA, fixed crack model and the rotated crack model. Both models initiate cracks when principal stress exceed the tensional strength. An assumption of uniform distribution of cracks within the material volume is also made.

For fixed crack model, the crack direction is given by the principal stress direction at the moment of crack initiation. This direction is fixed regardless of further loading and possible change in principal stress direction. Consequently, shear stresses may induce along the crack surface.

As with the fixed crack model, the rotated crack model models the crack in the direction of principal stress at crack initiation. However, the difference comes with change in the principal stress direction after the crack has been initiated. In the rotated crack model, the crack follows the direction of the principal stress after initiation. Shear stresses does consequently not occur at the crack surface. [16]

The fixed crack model is used in the thesis.

2.5.4. Convergence criteria in ATENA

The are several methods to analyze a set of nonlinear equations, which uses both direct and iterative solvers, many of which are available in ATENA. Regardless of the solver, they must solve a set of linear algebraic equations, i.e. $[A][x] = [b]$. There pros and cons with the use of direct vs iterative solvers, which will not be explained in detail. However, it is advised to use iterative solvers for advanced problems. Some methods which uses iterative solvers, most notably the Newton-Raphson Method and the Arc-Length Method, are available in ATENA, both in their original- and modified form.

It is important to ensure convergence in numerical analysis as divergence is usually fatal to the numerical results. During analysis, convergence is measured using a rate of convergence. Numerical analysis is produced using several iterations of which is calculated.

The most widely used method, the Newton-Raphson method works in principle through use of the following controls for each iteration.

- Norm of deformation change from the last step
- Norm of out-of-balance forces
- Out-of-balance energy
- Out-of-balance forces in terms of max components.

In ATENA, convergence limits ε are by default set to 0.01, but may be changed. The un-modified Newton-Raphson method is illustrated in Figure 18. [16]

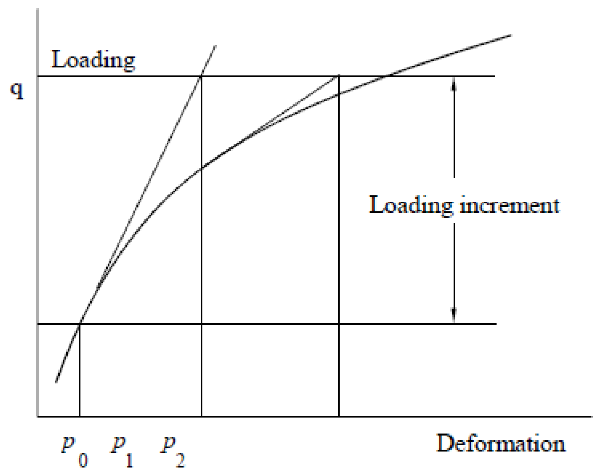


Figure 18 - Newton-Raphson method [16]

The second most used method, the Arc-Length method, is newer and is in general more robust and computationally efficient than the Newton-Raphson method. The Arc-Length method observes the complete load-displacement relationship, in contrast to the Newton-Raphson method which applies constant load increments. I.e. the method does not only fix the load, but also the displacement at the end of each step. Mathematically, this is done through the addition of a new degree of freedom. Controls for each step may either be the same as for the

Newton-Raphson method, or a variation. The Arc-Length method is illustrated in Figure 19. [16]

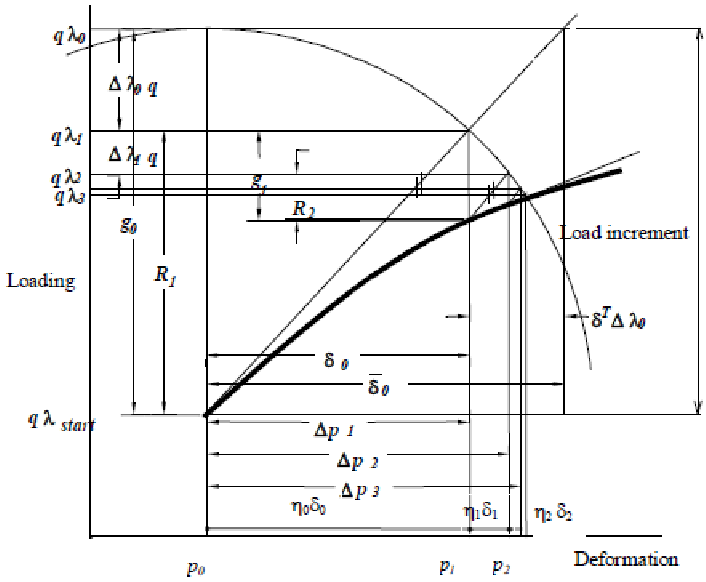


Figure 19 - Arc-Length method [16]

The Newton-Raphson method is used in the thesis.

2.6. Proposed modified beam

This idea takes inspiration in many different elements. Most notably hollow decks, but also IB-beams, as shown in Figure 22. Which is essentially an I-beam with a c/s change close to the ends. There are very few examples on longitudinal cavities in concrete beams, which is the primary cause of why I chose this approach. The idea is also that this cross section will maintain most of the initial capacity.

Further inspiration came from [17].

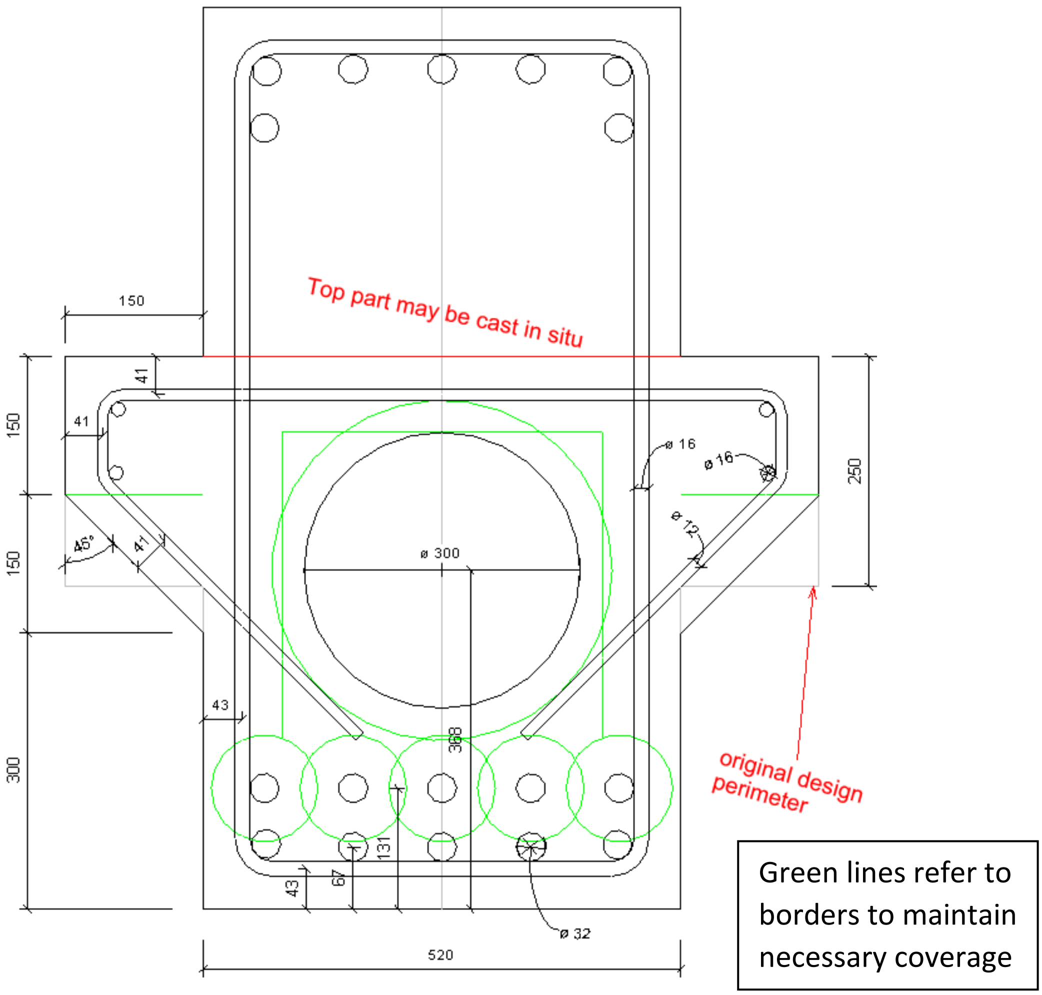


Figure 20 – Cross section proposal at spans (dimensions in mm)

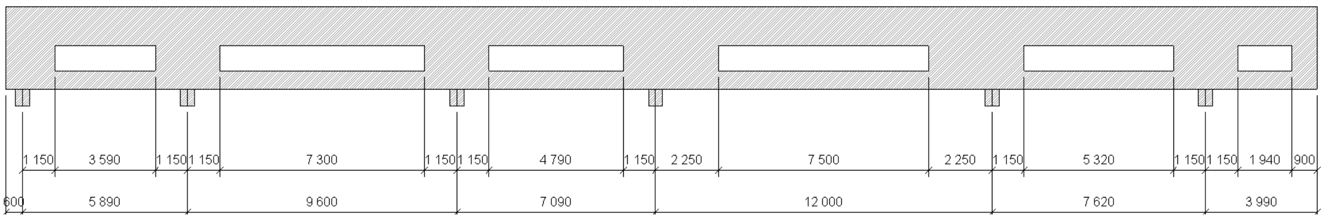


Figure 21 – Continuous beam with longitudinal cavities (dimensions in mm)

Some properties/notes/arguments regarding proposed beam:

- Cavity starts at least a distance, “d”, from the face of the support. It also must stop before the rebar related to a joint. Unless the general shear capacity of the cross section is violated, in which it sets the limit.
- Corbel rebar in elevation is designed to withstand the load from the hollow decks acting on the corbels.
- Moment capacity marginally changed
- Corbel is set sufficiently large to avoid spalling
- Corbel rebar is sufficiently anchored
- Cavity below equivalent compressive stress block both for cracked and un-cracked c/s in positive bending.
- Cross section is not designed for torsion as hollow decks placed on both corbels are held in place by a torsion lock. Principle may be seen in 8.5.
- Excessive concrete is removed from bottom of corbels
- Angles and lengths has been optimized for simple production
- May possibly be combined with transverse cavities. Though, that will normally cause a discontinuity and the case will have to be analyzed separately.
- Cavities in top part is avoided due to
    - Space for torsion lock
    - Top part may be cast in-situ
    - Negative moments mainly over the supports within the discontinuity region
- Proposal opens for in-situ cast of the compression zone/top part. This would induce the need for additional shear rebar in the beam, along with detailed analysis of the assembly phase.
- Allows for in-situ cast of corbels.
- a_0/d ratio for corbel is 0.386, which barely outside the interval 0.4 to 0.6. Non-conservatively assumed to be ok.
- Allows for pre-stressing.
- Cavity size is chosen to be a size which is both as practical as possible, but also as large as possible. In principle, it could have been made larger.
- Cylindrical shaped cavity is mainly chosen to minimize stress concentrations, but also for practical design.
- In-plane corbel reinforcement are normally made like a square because it lowers the need for anchorage and are easier to make, not to forget that it helps a little with respect to torsion. However, as this is a very large cross section, the reduction potential is smaller. Plus, a corbel with these geometries may be assumed to transfers torsion into the shear rebar either way.

[1] [3] [2]

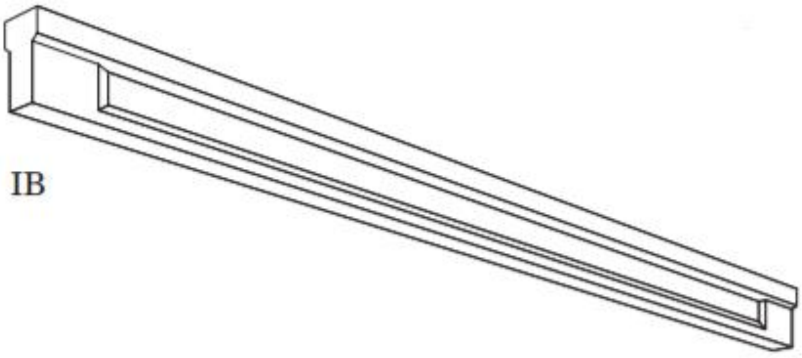


Figure 22 - IB beam [7]

3. Analysis of proposed reinforced concrete beam

3.1. Boundary conditions

Basically, the loads are transferred primarily from hollow decks on either side of the beam as given in Table 2. The beam is constrained in the lateral direction by torsion locks between the hollow decks and the top part of the beam. Principle may be seen in 8.5.

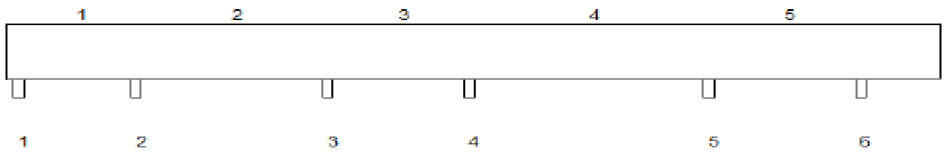
Table 2 - Loads

[kN/m]	Left corbel	Middle	Right corbel
Permanent load	46.2	6.13	43.8
Variable load	37.7	5	35.8

3.2. Numerical models using ATENA

Using ATENA software, the beam was modelled and all the information used during modelling is given in Table 3. Cross sections with belonging boundary conditions are designed according to Figure 20 and Figure 24.

Table 3 - Continuous beam



Field number	Left cantilever	1	2	3	4	5	Right cantilever
Span width [mm]	600	5890	9600	7090	12000	7620	3990
Support number	All transverse	1	2	3	4	5	6
Column diameter [mm]	520	500	500	500	500	500	500

For the modified beam, a cavity is modelled throughout the beam and then re-filled with elements shown in Figure 23.

The mesh is semi-structured with structured direction along the beam. Element type is hexahedral. Unstructured mesh size for cross section is 0.2, while structured mesh size is ~0.4.

Shear rebar: c140

Corbel rebar: c420

Only fully loaded beam with characteristic load will be checked.

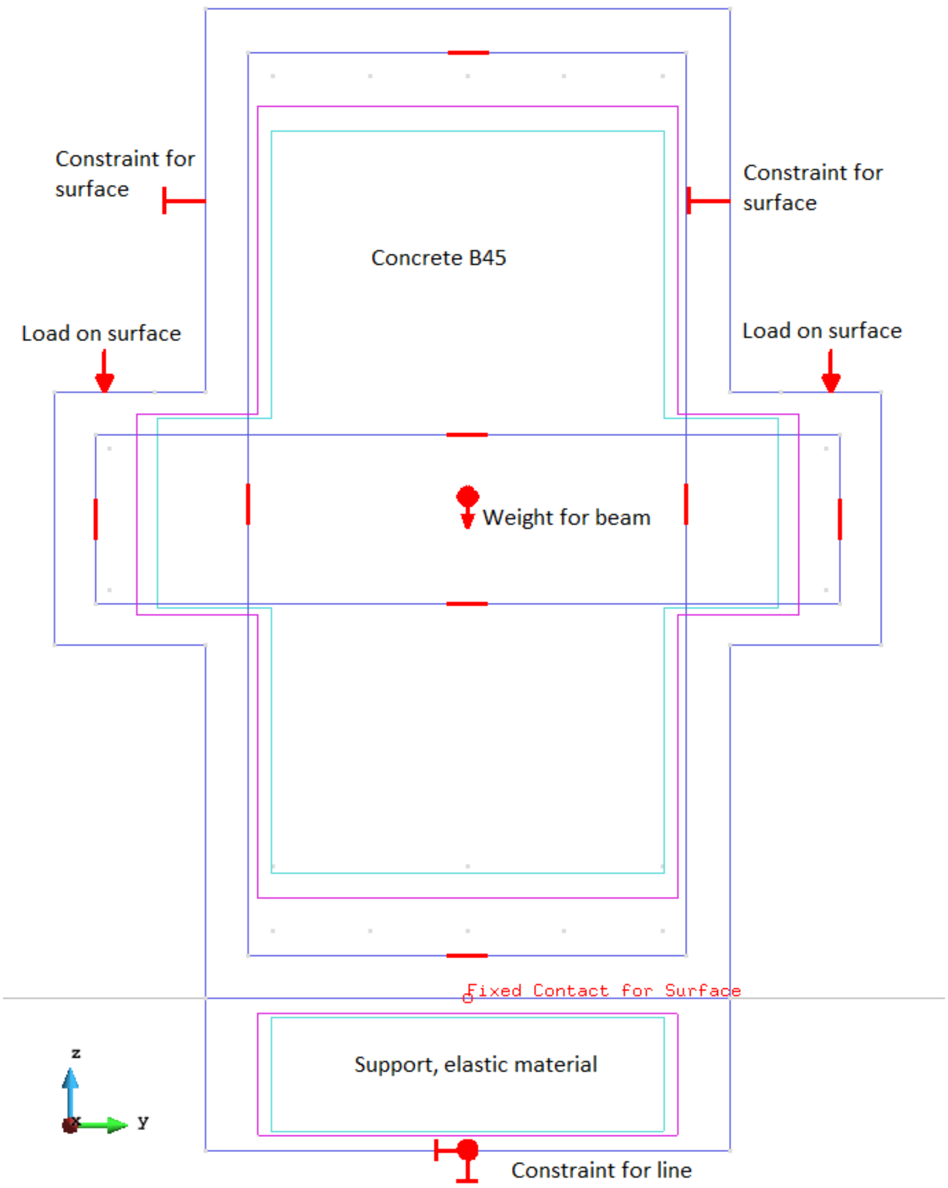


Figure 24 - Cross section with boundary conditions

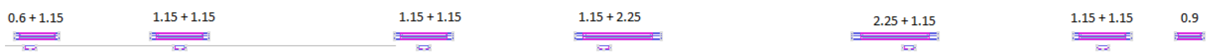


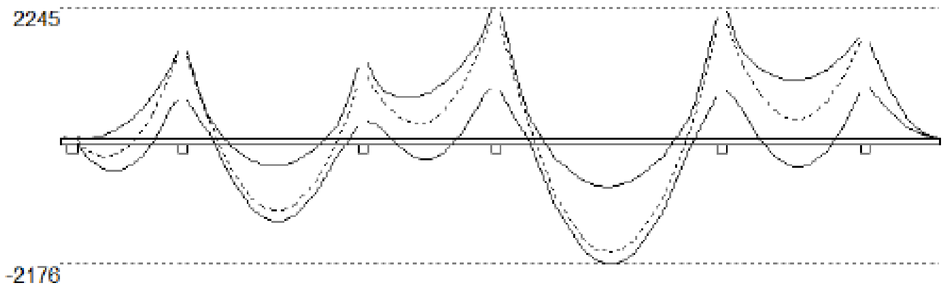
Figure 23 - Non-cavities in modified beam with distances [m] from center of support

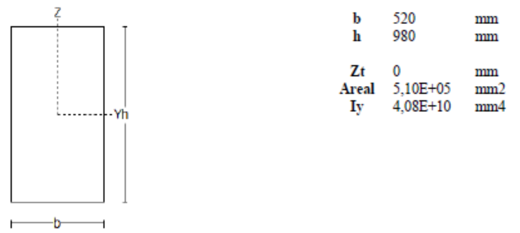
4. Analysis of the beam and results

4.1. Limits and pre-calculations

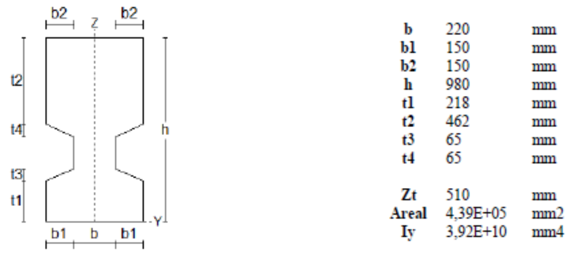
Manual calculations have been carried out mostly using simple software for analysis of concrete structures directly according to EC2 with Norwegian annex, i.e. K-bjelke, E-bjelke and BTSNITT. Basically, the following is a very short summary of 8.2.

Bending moment envelope in ULS [kNm]





Original cross section

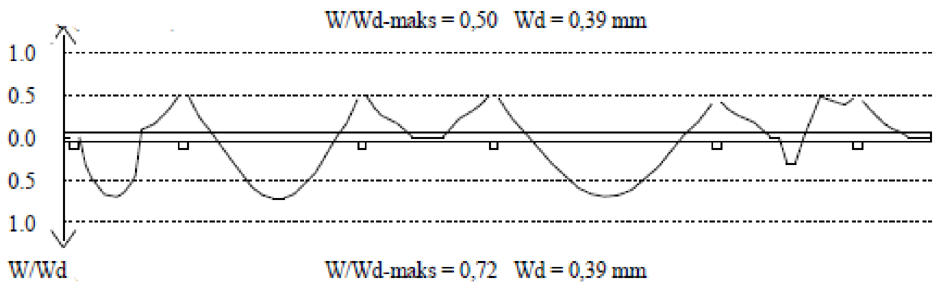


Equivalent modified cross section

The cavity is fully under the centroid (cracked section) at positive bending, therefore the positive moment capacity is unaffected.

If 5 rebar in top and 5+3 in bottom, positive moment capacity both beams: 2290 kNm

Crack width control for both cross sections



Shear capacity original cross section (vertical shear rebar)

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot(\theta) = \frac{2871}{140} 814 * 434.78 = 7258\ kN$$

$$V_{Rd,max} = \frac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot(\theta) + \tan(\theta)} = 1 * 520 * 841 * 0.6 * 25.5 = 6476\ kN$$

Shear capacity: 6476 kN

Shear capacity modified cross section (vertical shear rebar)

$$V_{Rd,s} = 7258\ kN$$

$$V_{Rd,max} = 1 * 220 * 841 * 0.6 * 25.5 = 2740\ kN$$

Shear capacity: 2740kN

Which is much lower then the original, need to be aware.

Design values in ULS

Compressive strength of concrete (B45): $f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_C} = \frac{0.85*45MPa}{1.5} = 25.5\ MPa$

Yielding strength of steel (B500NC): $f_{yd} = \frac{f_{yk}}{\gamma_S} = \frac{500MPa}{1.15} = 434.78\ MPa$

Design values in SLS

Compressive strength of concrete (B45): $f_{cd,SLS} = k_1 f_{ck} = 0.6 * 45MPa = 27MPa$

Yield strength of steel (B500NC): $f_{yd,SLS} = k_4 f_{yk} = 1 * 500Mpa = 500MPa$

Table 4 – Deflection, worst-case [mm]

Span Ref: Table 3	Left cantilever	1	2	3	4	5	Right cantilever
Deflection	0	0	18	-5	33	-6	29
L/250	2.4	23.6	38.4	28.4	48	30.5	31.9
Factor	0	0	0.47	-0.18	0.69	-0.20	0.91

Note: Negative values refer to most extreme negative deflection, 0 otherwise

Most of the deflected values are well under the limit.

4.2. Number coding for results

The following coding has been used to present the results from numerical analysis. Basically, both continuous beam without longitudinal cavities (i.e. reference beam) and modified continuous beam have been modelled using ATENA software.

First number

- 0xx: Reference beam
- 1xx: Reference beam with variable distribution of rebar
- 2xx: Modified beam
- 3xx: Modified beam with finer mesh

Second number

- x0x: Crack width in longitudinal direction
- x1x: Cracking pattern
- x2x: Deflection
- x3x: Compressive stress in concrete
- x4x: von Mises stress in all rebar
- x5x: von Mises stress in longitudinal rebar
- x6x: von Mises stress in shear and corbel rebar

Third number

- xx0: Whole beam
- xx1-xx9: Details

Mesh for each particular analysis is given at the start.

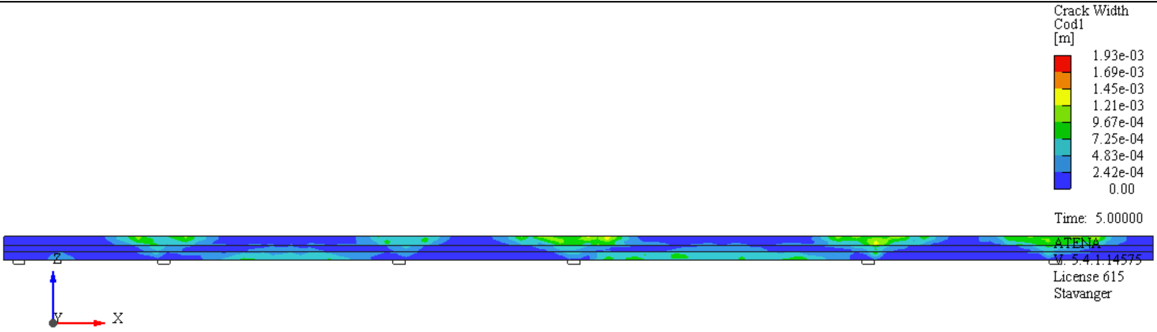
4.3. Reference beam

4.3.1. ATENA model of reference beam

Mesh

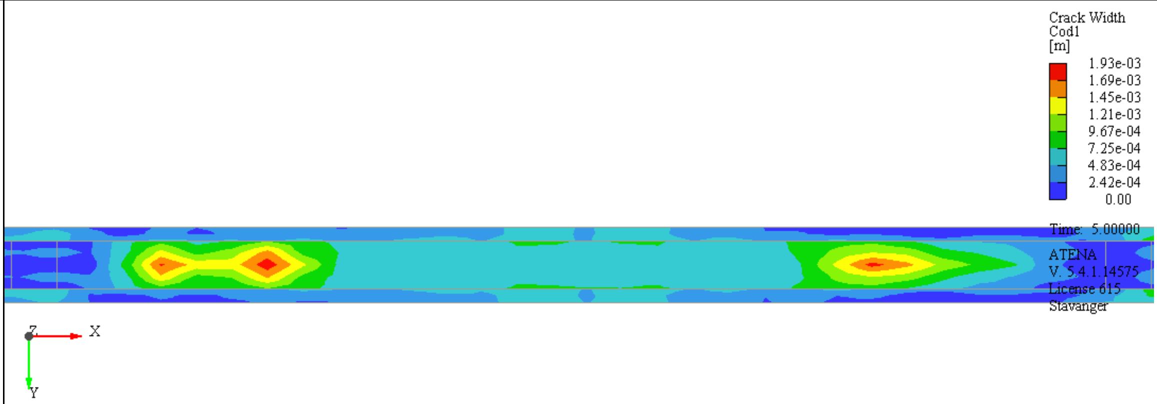
- Unstructured size yz-plane: 0.2
- Structured size x-axis: 120 per total length
- Element type: Hexahedral

000



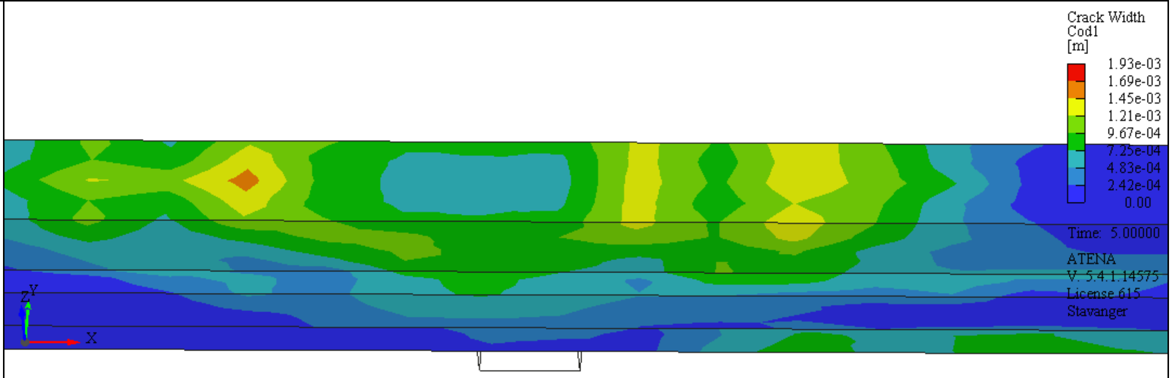
Very high crack width, only darkest blue colour is under the limit

001



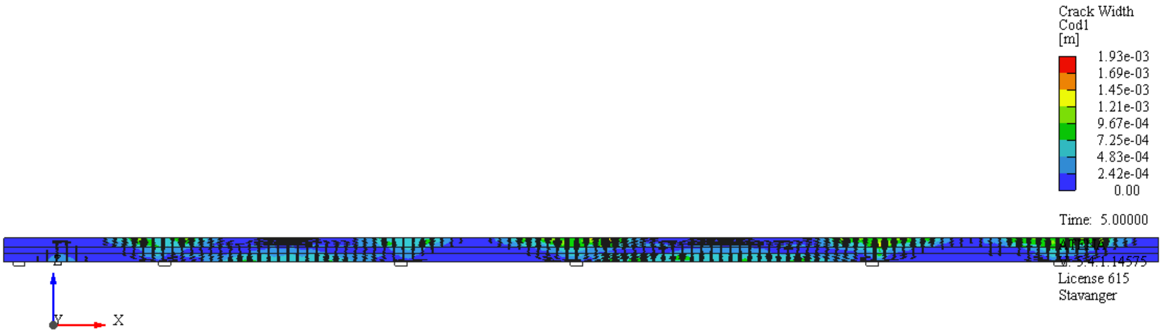
Under span 4: Some crack concentrations. Similarly in span 2.

002



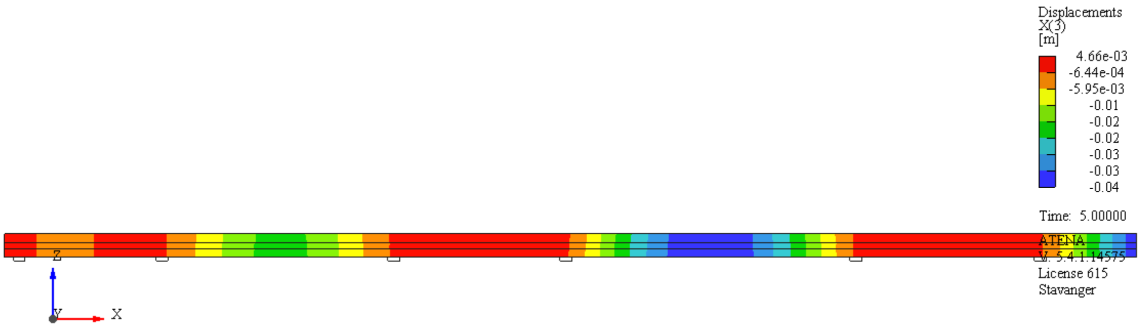
Support 4: Critical crack concentrations. Similar at other supports.

010



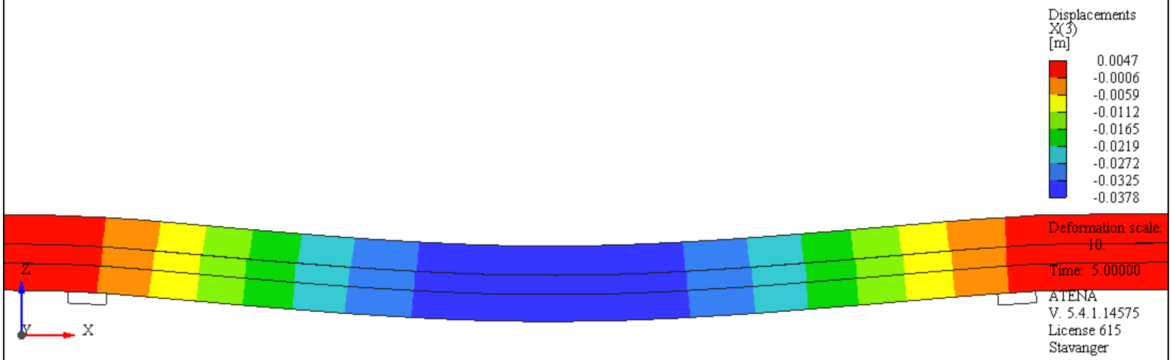
As expected

020



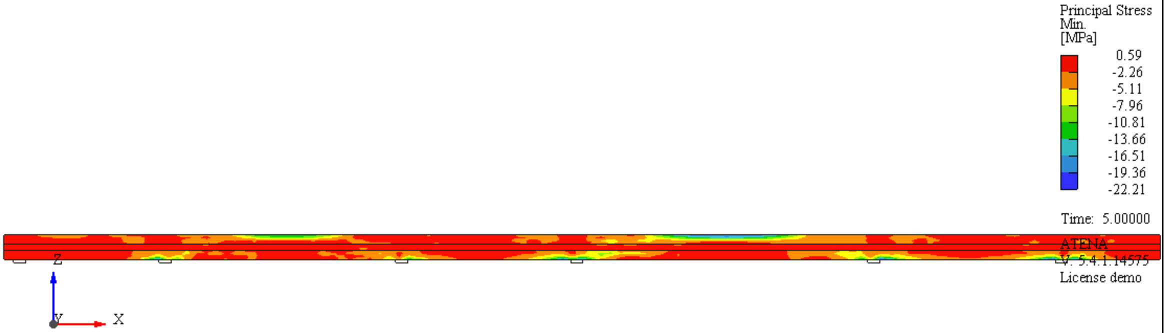
Right cantilever is above limit

021

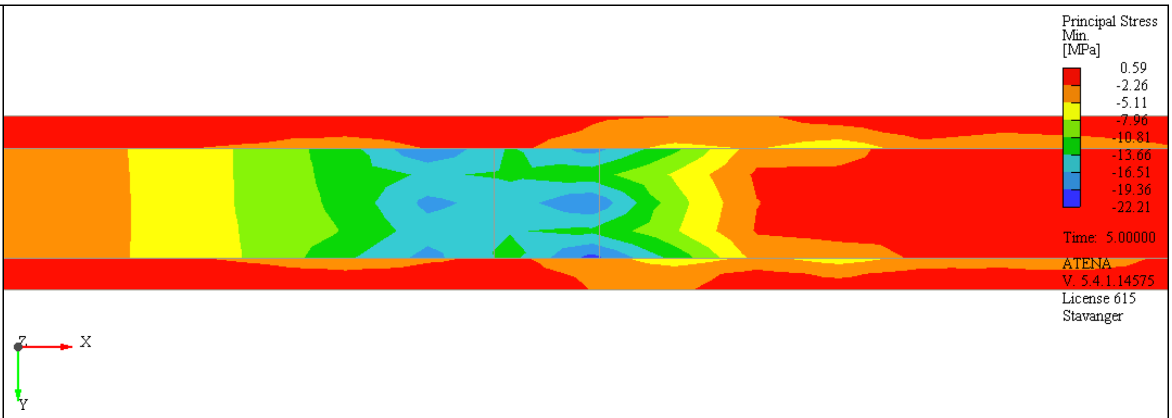


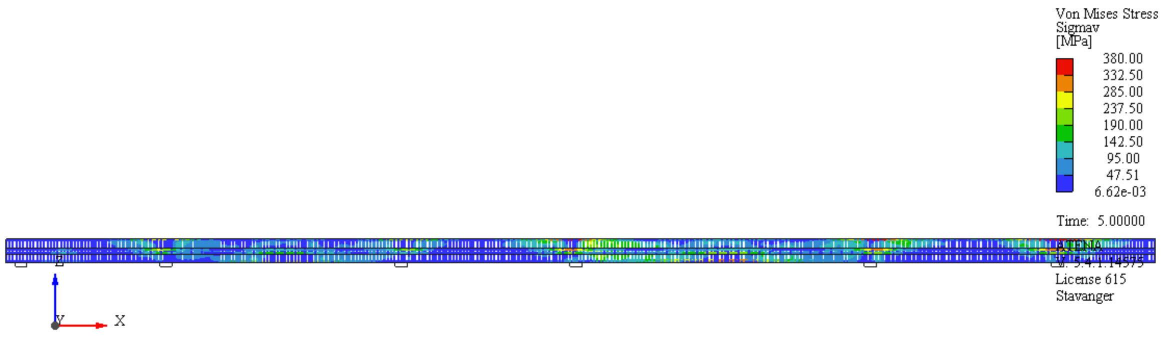
Span 4. A little higher than pre-calculations, but still under limit.

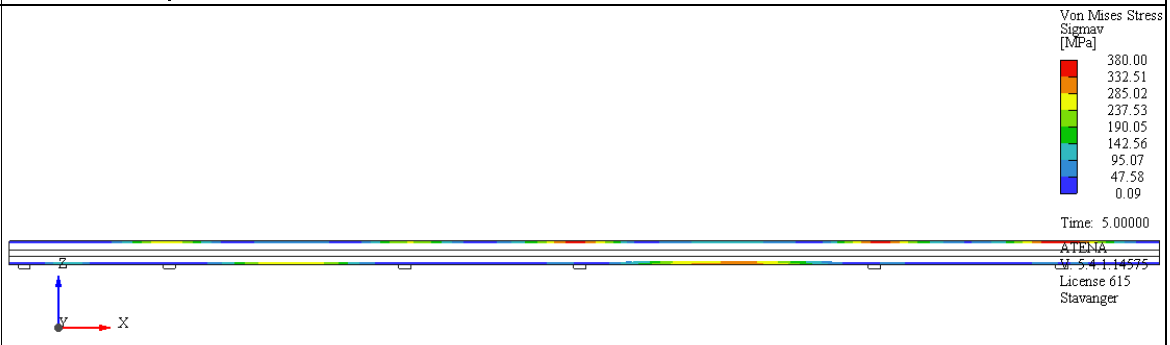
030

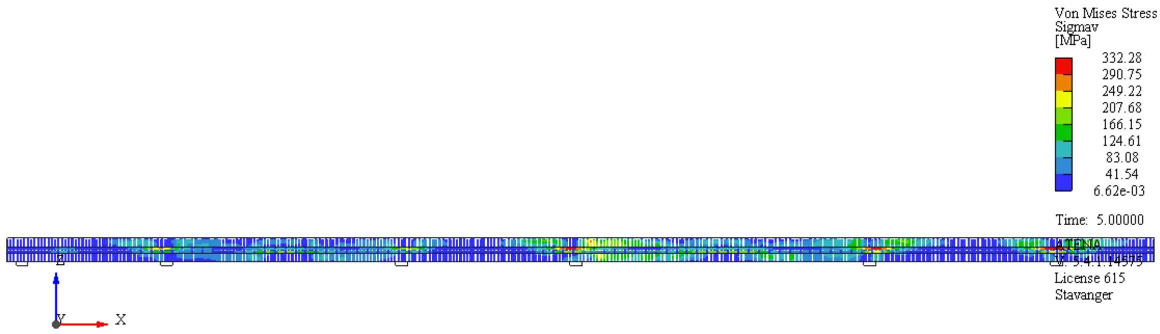


Comfortably under limit for most part of the beam

<table>
<tr><td>031</td><td>

Support 4. High concentrations. Similar at other supports.</td></tr>
<tr><td>040</td><td>

Comfortably under the limit</td></tr>
<tr><td>050</td><td>

Higher stresses in main rebar</td></tr>
<tr><td>060</td><td>

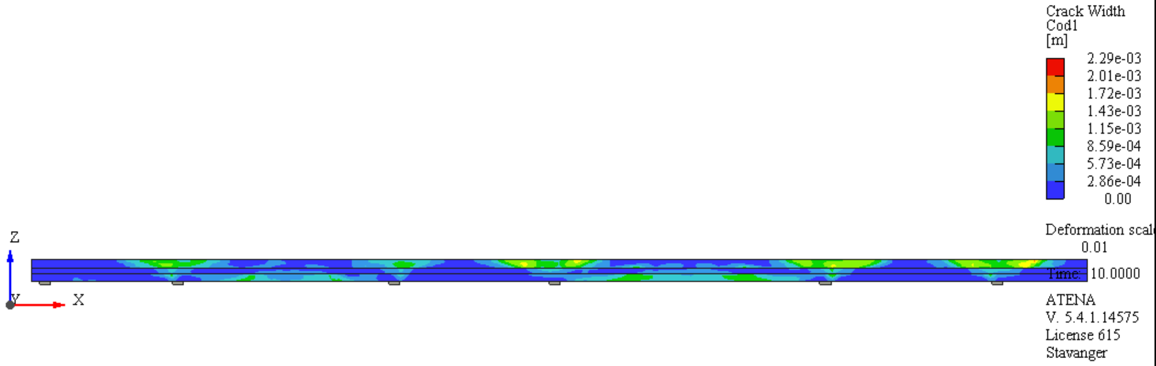
A little lower than in main rebar</td></tr>
</table>

4.3.2. ATENA model of reference beam with variable distribution of shear rebar

Mesh

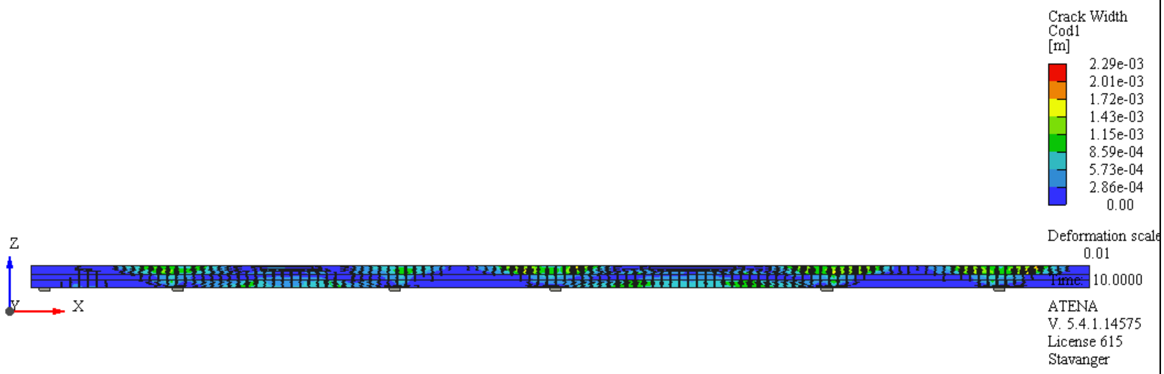
- Unstructured size yz-plane: 0.2
- Structured size x-axis: 120 per total length
- Element type: Hexahedral

100



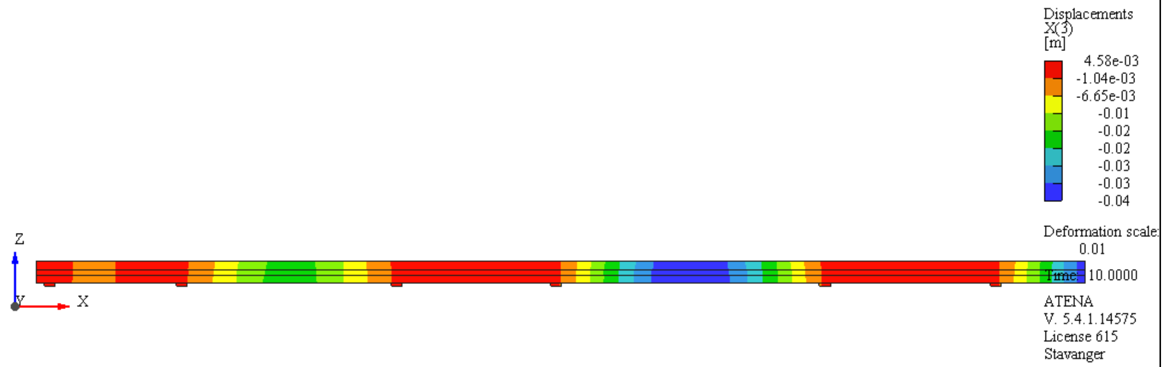
Very high crack width, only darkest blue colour is under the limit. A little higher then 000.

110

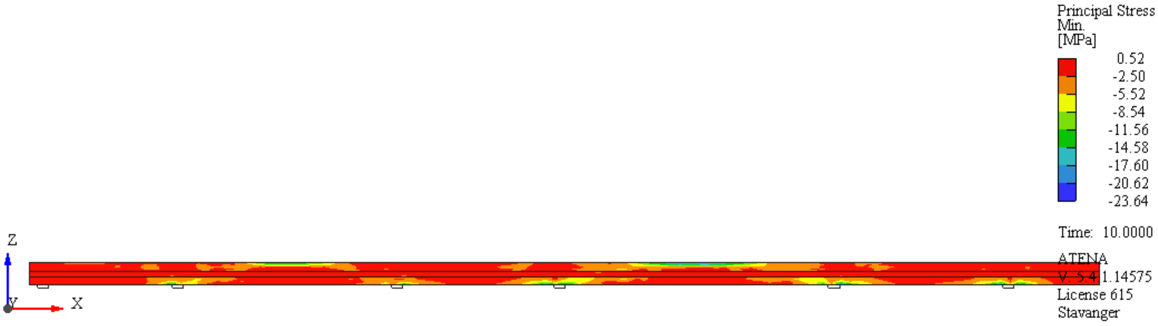


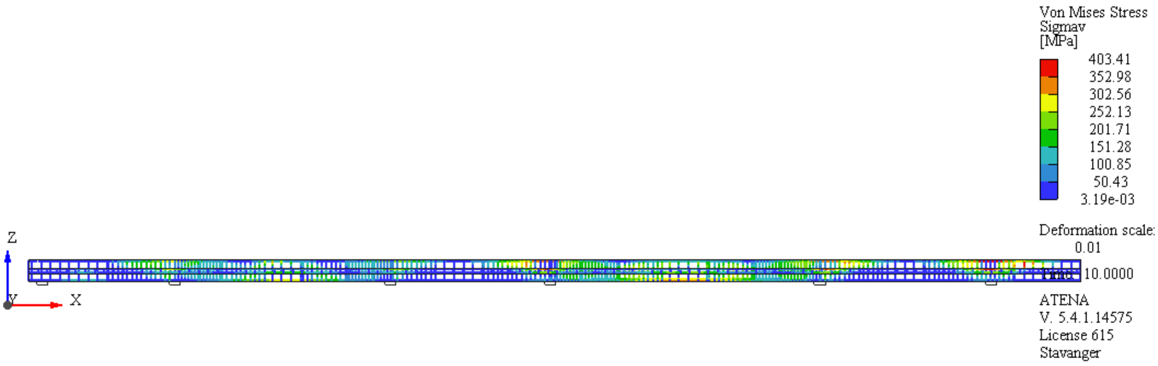
As expected

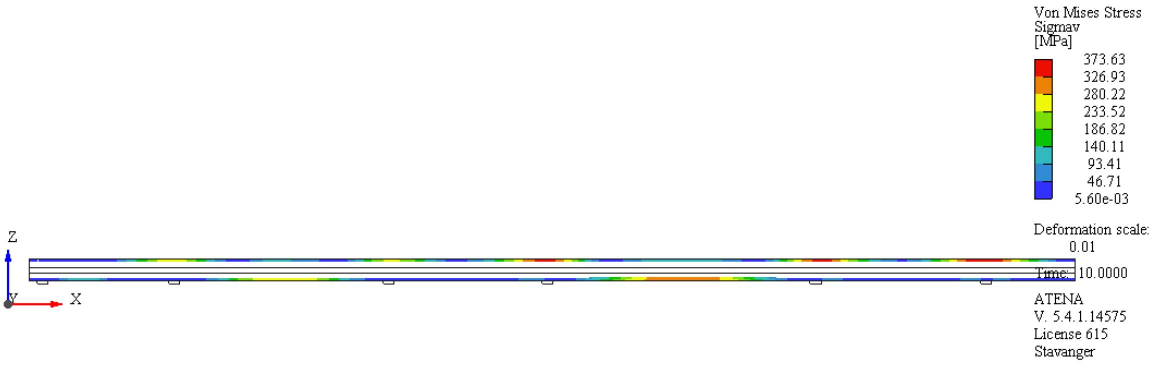
120

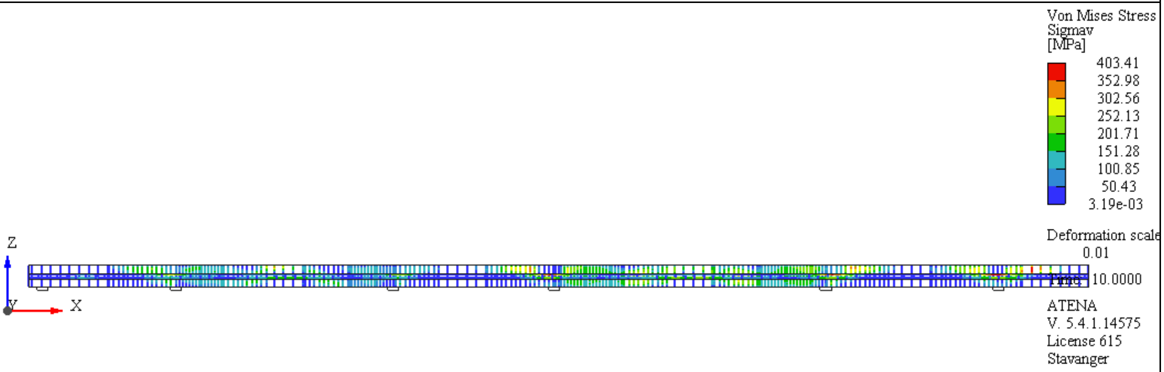


Very much alike 020

<table>
<tr><td>130</td><td>

Marginally higher then 030</td></tr>
<tr><td>140</td><td>

A bit higher then 040, but still well under the limit.</td></tr>
<tr><td>150</td><td>

Main rebar lowest, a little lower then 050</td></tr>
<tr><td>160</td><td>

Higher then main rebar</td></tr>
</table>

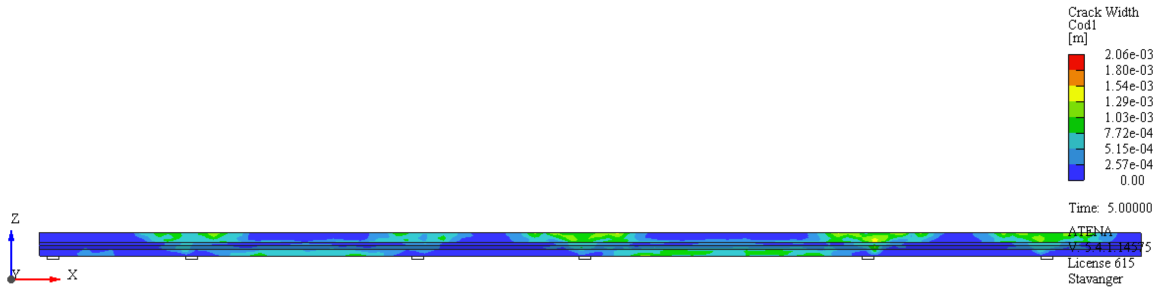
4.4. Modified beam

4.4.1. ATENA model of modified beam

Mesh

- Unstructured size yz-plane: 0.2
- Structured size x-axis: 120 per total length ~0.4
- Element type: Hexahedral

200

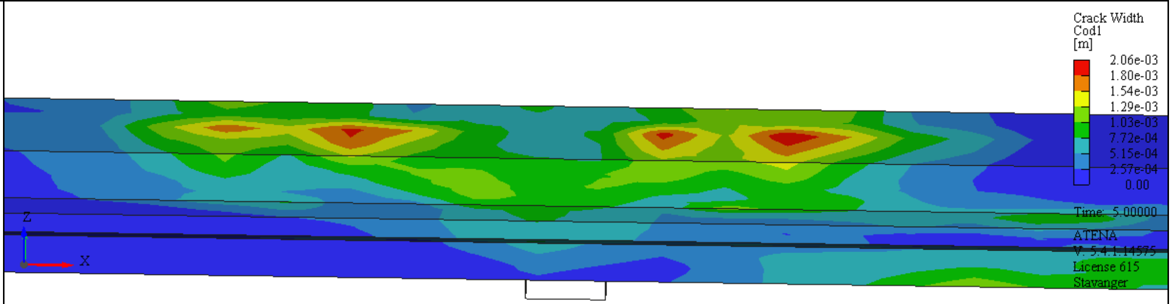


6.7% higher then 000. Quite high

201

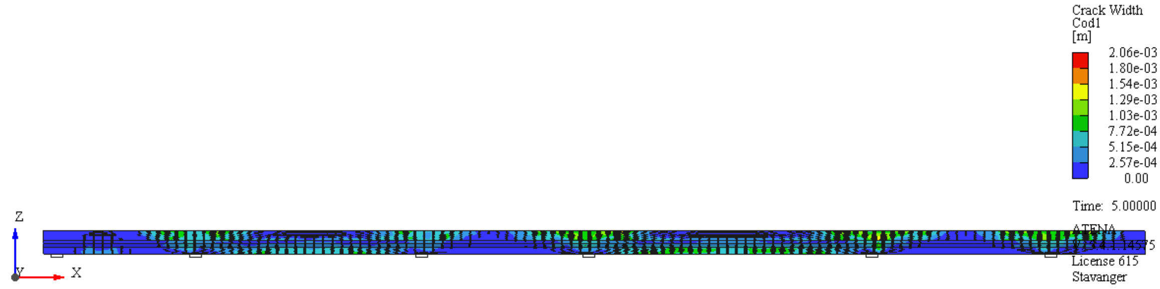
Under span 4: Some crack concentrations. Similarly in span 2. Similar patterns to 001

202

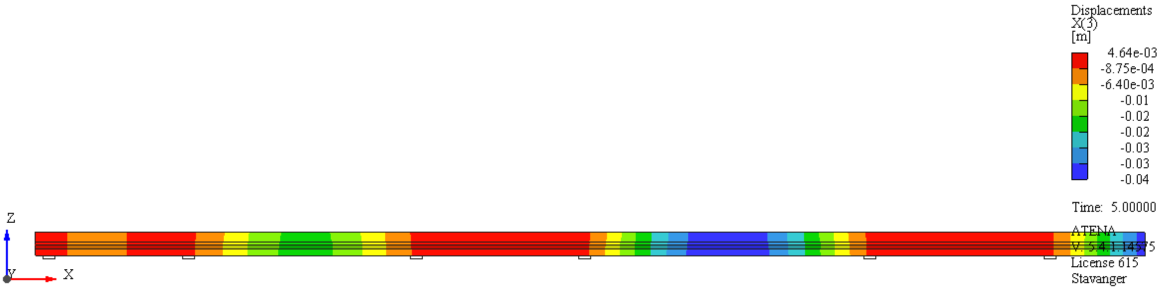


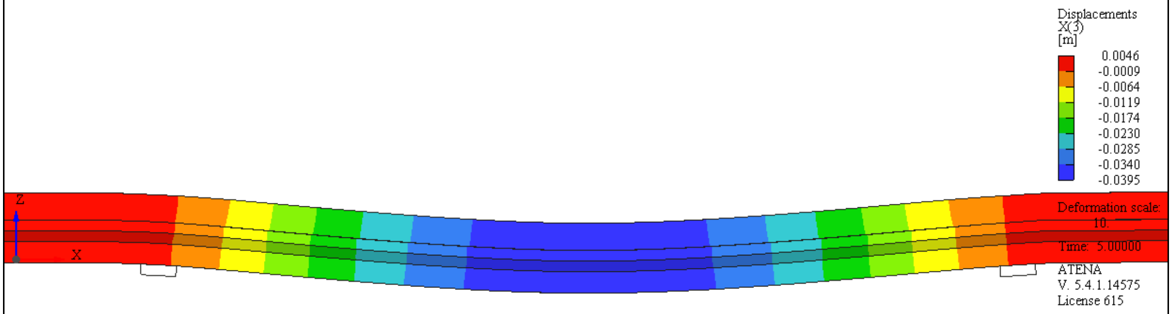
Support 4: Higher crack concentrations then 002

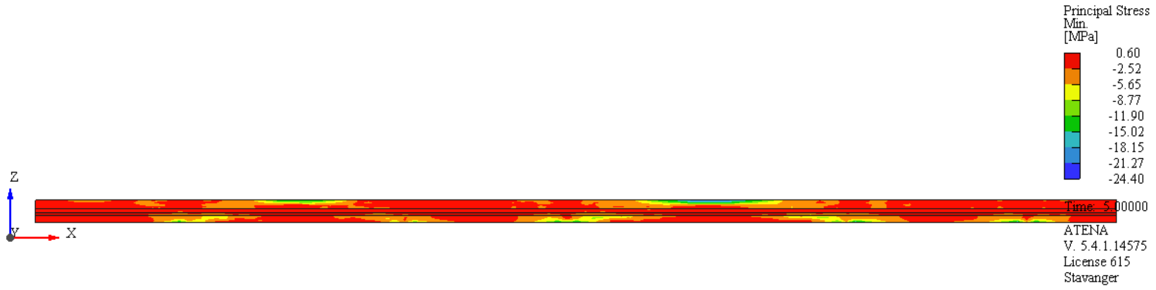
210

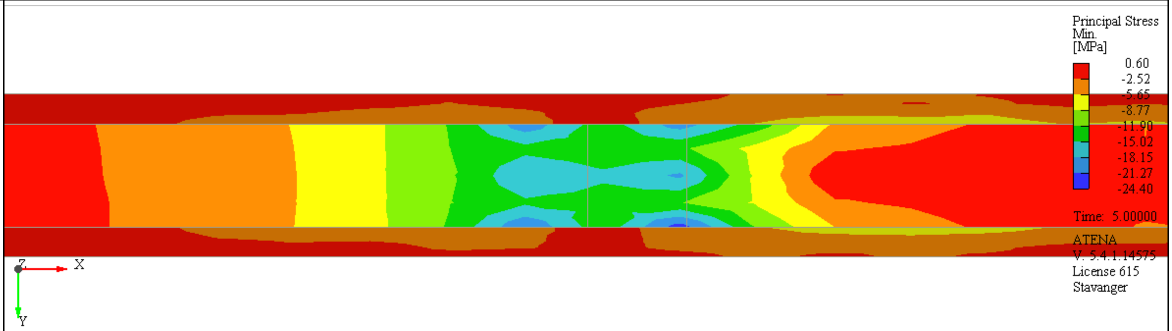


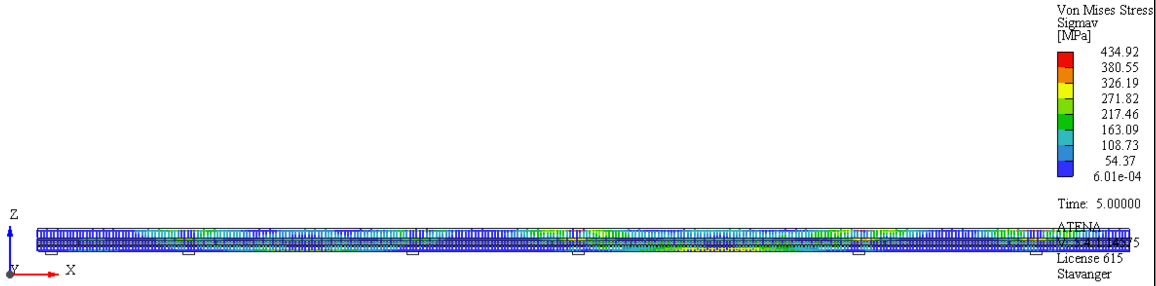
As expected

<table>
<tr><td>220</td><td>

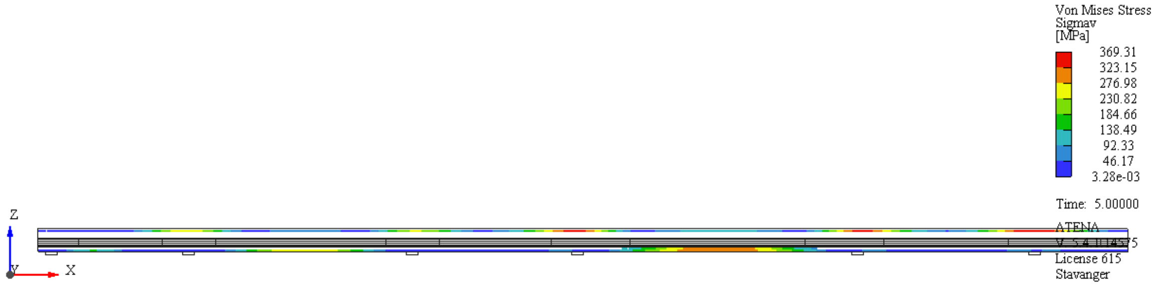
Very similar distribution as 020</td></tr>
<tr><td>221</td><td>

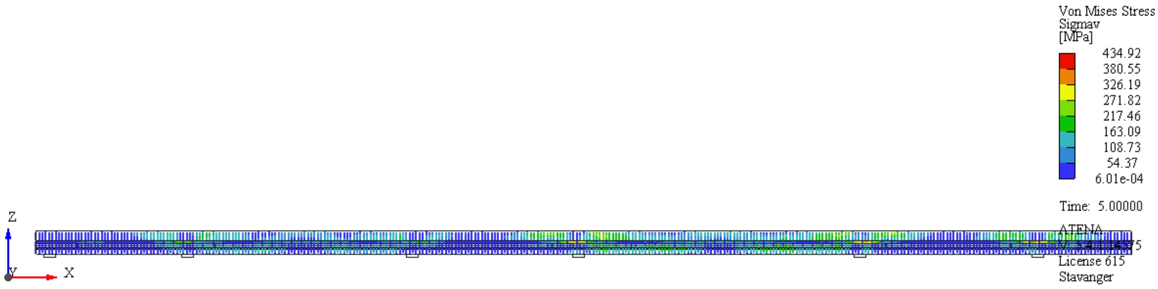
Span 4: Even higher than 021, but still under the limit.</td></tr>
<tr><td>230</td><td>

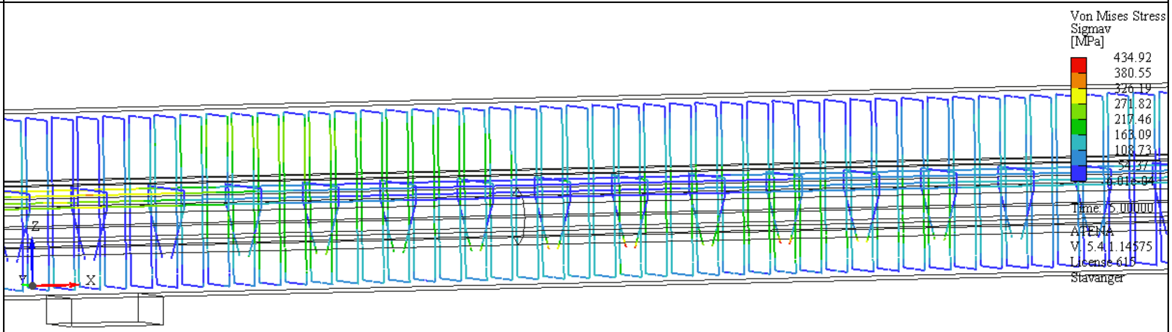
Higher them 030, close to the limit.</td></tr>
<tr><td>231</td><td>

Support 4: Pattern similar to 031</td></tr>
<tr><td>240</td><td>

A little high</td></tr>
</table>

<table>
<tr><td>250</td><td>

Stresses in main rebar are well below the limit</td></tr>
<tr><td>260</td><td>

Stresses in other rebar seem ok, other than the value shown.</td></tr>
<tr><td>261</td><td>

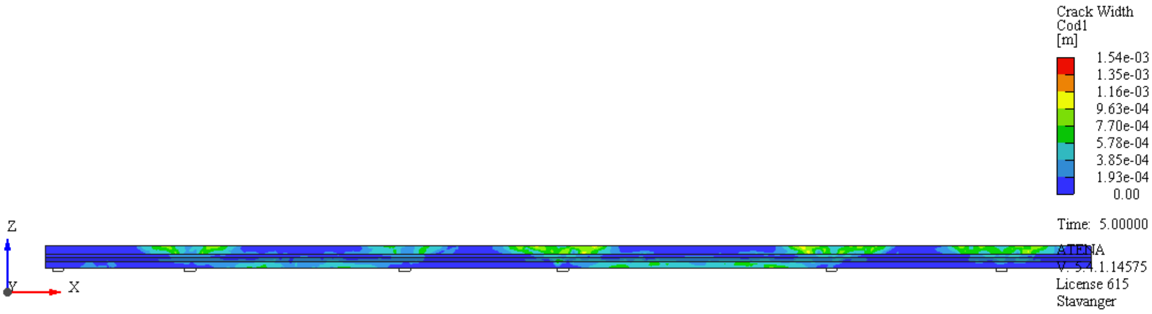
Stress concentration at corbel rebar tips.</td></tr>
</table>

4.4.1. ATENA model of modified beam with finer mesh

Mesh

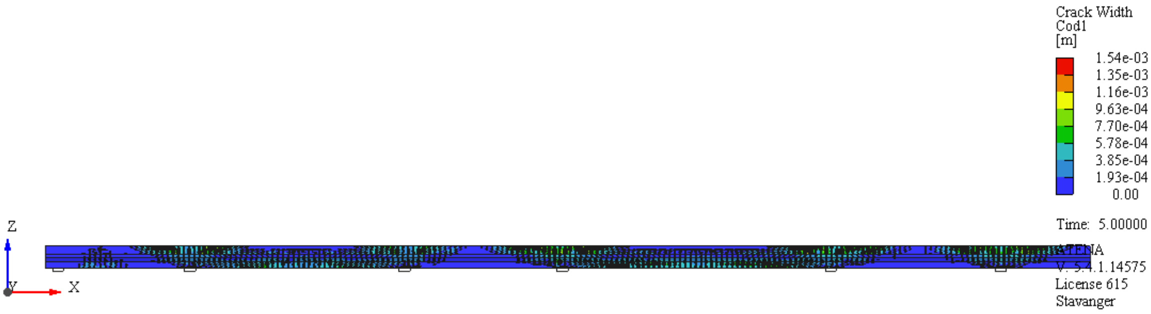
- Unstructured size yz-plane: 0.15
- Structured size x-axis: 200 per total length ~0.234
- Element type: Hexahedral

300



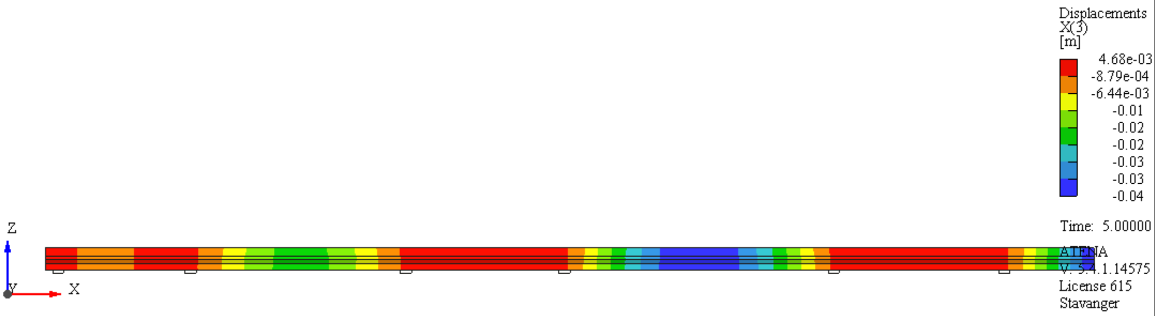
Crack width reduced by 25% compared to 200

310



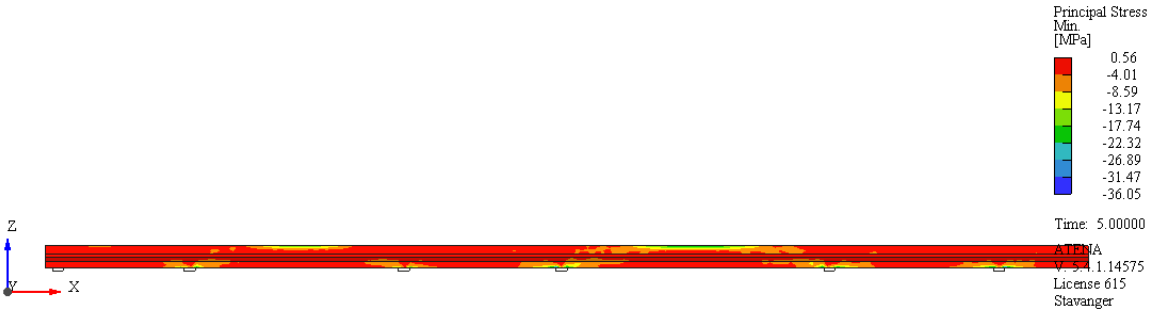
As expected

320

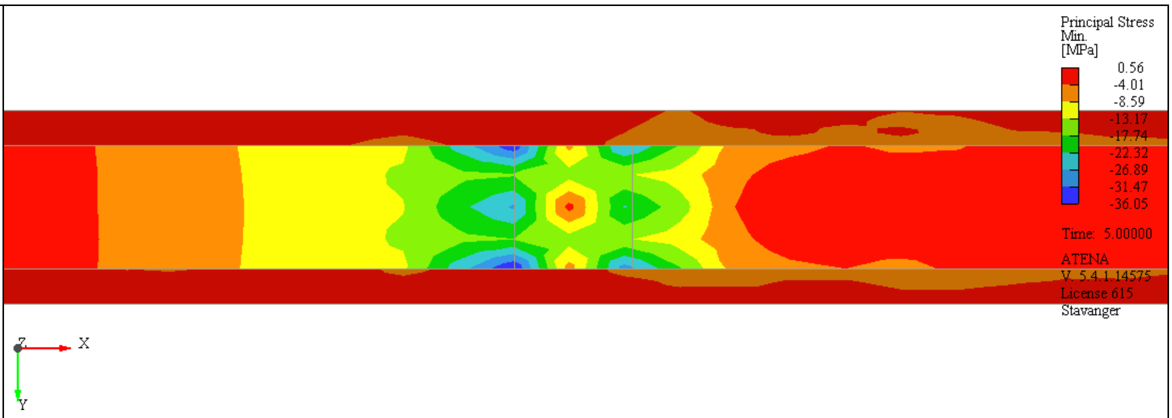
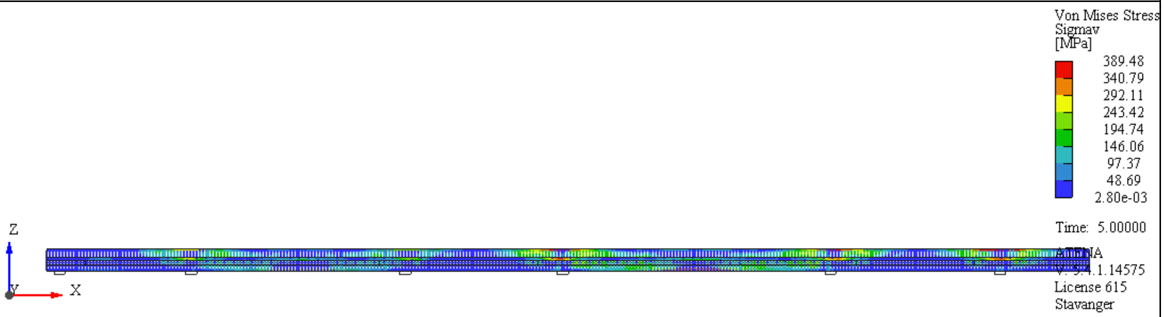
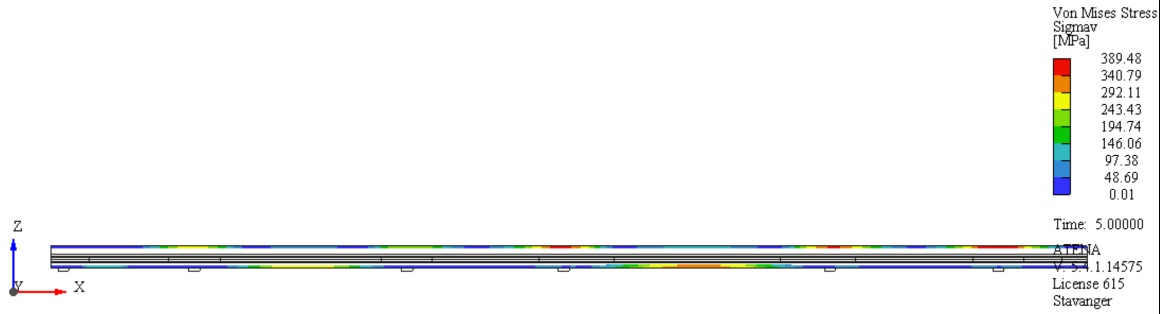
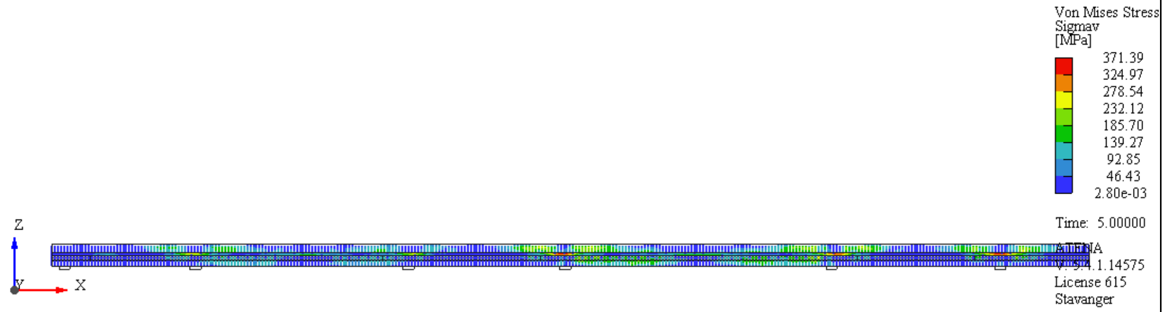


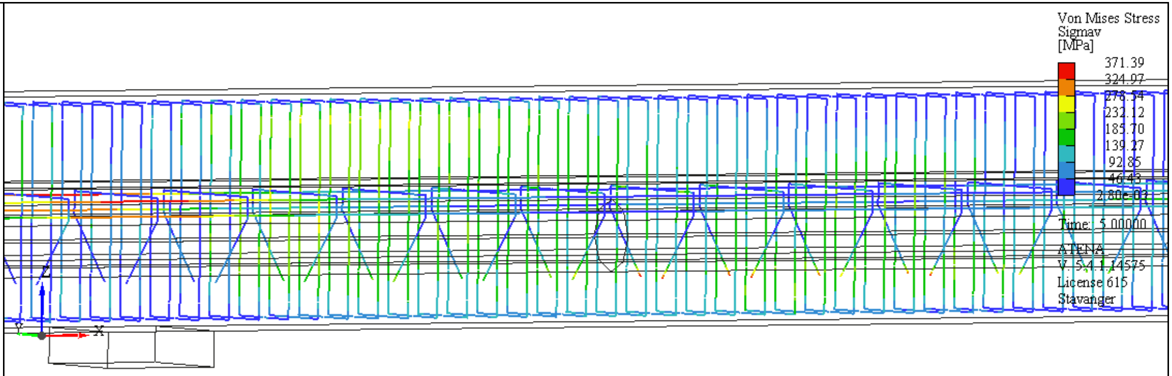
Not much changed from 220

330



Very much higher then 230, though quite low in the beam in general

331	 Support 4: As with 231, there are some stress concentrations, although very much higher.
340	 Compared to 240, no not as high in this case. Although, it is still higher then 050
350	 Main rebar has the highest stresses
360	 Highest stresses are quite a lot lower than for 260

361	 Compared to 261, there are still stress concentrations at tips, but not as high.

4.5. Summary and general comments on results

Summary of results from ATENA has been given in Table 5.

Table 5 - Maximum values

Model reference number and names		Crack width (span 4) [m]	Deflection (span 4) [m]	Compressive stress in concrete [MPa]	Max stress main rebar [MPa]	Max stress corbel- and shear rebar [MPa]
	Limit	0.39e-03	-0.0480	-27	500	500
	Pre-calculations	0.28e-03	-0.033	N.A.	N.A.	N.A.
0	Reference beam	1.93e-03	-0.0378	-22.21	380.00	332.28
1	Reference beam with variable distribution of shear rebar	2.29e-03	-0.04	-23.64	373.63	403.41
2	Modified beam	2.06e-03	-0.0395	-24.40	369.31	434.92
3	Modified beam with finer mesh	1.54e-03	-0.04	-36.05	389.48	371.39

Notes on results

- Crack width is considerably higher than the limit for all ATENA-calculated values. It seems that this decreases with finer mesh. Ref Table 5.
- There are some concentrations of crack width on top of each support. There are usually more stiffening at supports, which may possibly lower the stresses. This has not been applied to this model.
- The distribution of shear rebar seems to affect the results even though it in theory should not. Shear rebar were distributed according to the shear envelope.
- This model applies full distributed loading on all spans. In other words, critical load arrangement is not analyzed, which could increase all values.
- Cantilever part is not analyzed in detail due to support from above which is not included in the model.
- Modified beam with finer mesh has very high stress concentrations by supports.

5. Discussion

Even though the other results are mostly fine, the models show a tendency to have very high crack widths, which is of course very problematic. A smaller mesh (ref model 3) seem to solve it partly with regards to crack width, but a finer mesh may also induce unnatural stress concentrations like the one seen in compressive stress of concrete. In addition, with respect to aspects ratio of element size, refinement in one direction may cause numerical problems.

However, if one chooses to compare results in model 0 to model 2, there are not very big differences, which is in general is a positive sign. As expected, the higher range of values for model 2 turned out a slightly higher then model 0, which seem to increase in a range of 5-10+%.

A high stress occurred in the tip of the rebar for model 2 and 3, which most likely occurred due to the steep change at the border between cross section with and without cavity. This may possibly be solved by rounding of the ends of the cavity. In general, rounding off edges helps reduce stress concentrations.

With reference to 001 and 201, there are some crack concentrations under span 4. One would think that this is caused by the start of the cavity, but as mentioned, this also happens for model 0 which does not have any cavities within. Another reason for this could be due to the change in longitudinal rebar, but as this also happens at span 2, which does not have any change in longitudinal rebar, it is unlikely to be the main cause.

A beam with longitudinal cavity gets reduction in shear capacity compared to a rectangular section in the same way an IB beam has reduction in shear capacity, due to reduction in effective breadth. The shear capacity happened to set the premises at many points, a lot more than the bending capacity which happened to stay close to the same, in the same manner as an IB beam.

Also, as seen in the results, there are clearly cracking patterns very alike punching shear patterns. Punching shear is a parameter which is usually only analyzed in terms of plates and has therefore not been considered for the analysis. For punching shear, a circumference of radius 2 times "d" are usually added, i.e. twice the size of what has been used in this proposal. In afterthought, it may be a topic for further analysis.

Loading for the numerical analysis were interpreted as characteristic load combination. In addition, full load was placed in all spans. This simplification does indeed affect the results. ULS usually has higher factors which would most likely have caused the stresses to increase. In contrast, it is a conservative approach with regards to SLS, as in quasi-permanent load combination, variable load usually gets reduced by a factor ψ_2. Which in turn causes higher values for both crack width and deflection. Although variable load in all spans causes a little lower crack width and deflection. This was done due that the main goal was to compare the original beam to the modified one with a similar mesh. I.e. model 0 with model 2. [1]

The mesh could have been made better, though with regards to meshing guidelines and available computing power, better results were hard to obtain. Most likely, a preliminary study could instead have been conducted using a simply supported beam or a continuous beam over two spans for analysis of the principle itself.

Coverage has been a topic for discussion in preliminary studies of this thesis. For precast elements, the standard added 10mm may be reduced to 5mm due to the nice environments in the factory. Also, as the cavity will be fully within the beam, fire resistance will not be an issue, which may affect the coverage. Additionally, the exposure class is therefore also a parameter which could also be lowered. To sum up, mainly the rebar size will set the premises for coverage. Due to these arguments, the hole may be increased even further.

As this is definitely not a well-established design approach, much focus has gone into how to actually accomplish this in a cheap and practical manner. As mentioned earlier in the thesis, many proposals have been made regarding material. The initial and possibly cheapest alternative is to use a cardboard pipe or PVC pipe with a cap on each end. This seem like a good idea at first, but it may cause some additional problems and therefore adjustments. First, a pipe of that size may not tackle the uneven pressure from the liquid concrete in the casting phase. Buoyancy of the pipe must also be taken care of and it may be very hard to hold a pipe in place without a proper way to keep it rigid. It may not be possible with for example, screws. However, keeping it in place could be done using steel wire around it, which is in general not a very good way. Secondly, like a problem with regards to hollow decks, there will be moisture from the concrete which needs to be drained out of the pipe.

Another proposal is to use a special kind of Styrofoam, which is widely used as insulation in precast concrete elements, and shape it into a cylinder. It does not have as many of the problems as a pipe. For instance, it allows for rods through its volume to keep it rigid and it also does not have the problem with moisture which needs to be drained out. It also allows for rounding the edges of the cavity to limit stress concentrations there. Though, the backside is that it is much more expensive.

A different side-note is to possibly use a hollow steel profile within (i.e. composite), which may even help with the structural capacity. Though, steel is heavier and more expensive then concrete and for this to be a viewable alternative, the cross section should be even further reduced, and so on. Which in turn makes a completely different analysis.

Like hollow decks, the cavity may be possible to use for other purposes like ventilation and such. Though, it rarely happens for hollow decks as it is simply a lot easier to lay it on the outside. Inside a cavity, modifications and maintenance is much harder, not to forget problems with moisture as mentioned. As to do the same with a beam would most likely be even more impractical, the potential may be disregarded.

Some other pros and cons

- Save
    - Smaller crane
    - Transport

    - Weight on structure
- Extra cost
    - Production
    - Additional rebar
    - Materials for cavity
    - Maintenance

6. Conclusion

This method of weight reduction for cross beams seems to have a potential. For this case, the weight reduction potential for the whole continuous beam is about 9%. Upon further optimization and analysis, this percentage could go either way. For the general case, this percentage is of course subjected to change. However, the cavity in the bottom could possibly be used in conjunction with other weight reduction measures.

In general, critical values seem to increase ~5-10%

How to do this in a cost-effective way seem to be the main wall between whether it is worthwhile or not. For example, in case another element is to be placed at the approximate same position as a beam part, but are heavier than the beam part, a larger crane is needed regardless. However, hollow decks are the elements usually used in conjunction with continuous beams, and a single hollow deck is usually lighter than a part of its supporting concrete beam, i.e. not a frequent problem.

It may also help with regards to weight of the structure. Say reducing the size of the foundation by a few percentages may help a little. Ability to transport one more element per load may also help a bit. 9% weight may also be the difference between two crane sizes. Anyhow, structural analysis and production may take longer along with higher material cost. This, along with many other factors come into play.

Based on this preliminary study, it is very hard to say whether it is a good design approach or not. Further studies in conjunction with other theories must be conducted before a concrete conclusion may be drawn. Cost is also an important aspect. Anyhow, in sum, there are not any clear obstacles in the way of studying this approach in further detail.

Due to the need for coverage between the cavity and the rebar, this is an approach seem applicable mostly to beams which has space for a sufficiently large cavity.

Some possible topics for further studies:

- Combinations with
    - Pre-stressed steel
    - Transverse cavities
    - Top part cast in-situ
    - Fiber reinforcement to limit crack width
- Stress concentrations
- Shear and torsion capacities
- Model with pinned joints

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8. Appendix

8.1. Conference paper title and extended abstract

Weight reduction of pre-fabricated reinforced concrete beams using longitudinal cavities

Pre-fabricated reinforced concrete beams are widely used within the construction industry. The reason behind this is due that pre-fabricated elements represent a rational, timesaving and economic method of construction. A wide variety of shapes and sizes are available, which includes among other walls, beams, columns and slabs. Regardless of all the benefits, there are still some additional aspects to be aware of. For instance, weight is a much more important property for pre-fabricated then in-situ cast, as they must be both transported and assembled. In addition, weight reduction intuitively lowers loading on the structure itself.

As for weight of reduction of concrete elements, there are many different methods available. Lightweight aggregate, pre-stressing, geometrical changes and several others. Most weight reduction measures tend to reduce the structural capacity so it is important to do it correctly.

Beams are normally very sensitive to changes in material as they are usually subjected to very high stresses compared to volume. Beams are therefore often subjected to geometrical changes for optimization instead. To reduce building height, cross shaped or inverted T beams are very popular to use, but sometimes they become very large and heavy with long spans. Not very many optimization measures exist for these kinds of beams due to the multiple load effects, especially not under the corbels. Intuitively, this should be possible so the motivation was initiated.

A design proposal for large cross sections were constructed, consisting of a cylindrical cavity within the beam with some optimizations of the corbels to allow corbel rebar to pass a sufficient distance from the cavity. The cavity was only to be applied at a distance effective depth, “d”, of the beam from the face of the supports, unless the general shear capacity of the cross section is violated, in which the start of the cavity was altered accordingly. For the specific case, this measure reduced the weight of the beam by ~9%.

The design was achieved through analysis of the necessary coverage and anchorage. Along with structural analysis of the cross section in terms shear and bending resistance and resistance for the corbel through a simple strut & tie model. The cavity itself were assumed to be applied only within length which the Euler-Bernoulli beam theory are valid, or in which the reduced breadth was sufficient in terms of shear resistance. Numerical analysis of both initial and new design was then carried out using a non-linear finite element software.

The numerical analysis produced very high values for crack width for both. Comparatively, highest range of values for crack width, deflection and compressive stress in concrete increased by ~5-10%. Some stress concentrations also occurred.

Due to the need of coverage between cavity and rebar, this approach is mostly applicable to cross sections sufficiently large enough to have a cavity within.

8.2. Calculations

8.2.1. Initial beam

Includes

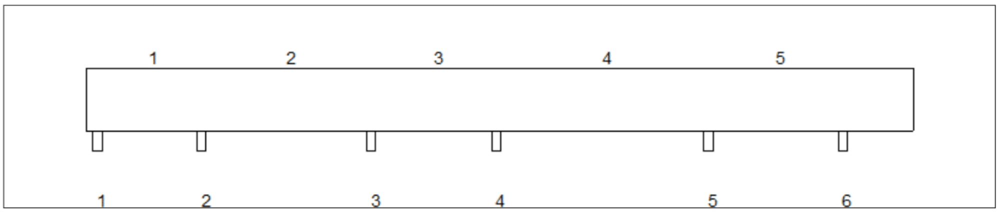
- K-bjelke calculation
- E-bjelke calculation
- BTSNITT calculation

Tittel			Side 1
Prosjekt	Ordre	Sign	Dato 30-05-2017

Dataprogram: K-Bjelke versjon 6.3.3 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1:2004 + NA:2008 og NS-EN 1990:2002
Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\uten momentledd.kbj

INNHOLD

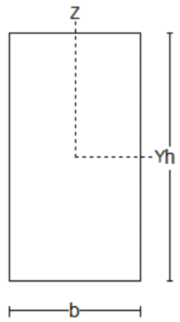
1.0 BJELKE MED 6 OPPLEGGSPUNKTER



1.1 SPENNVIDDER [mm], OG TVERRSNITTYPER

Felt nr	v.utkr.	1	2	3	4	5	h.utkr.
Spennvidde	600	5890	9600	7090	12000	7620	3990
Tverrsnittype	1	1	1	1	1	1	1

Tverrsnittype 1

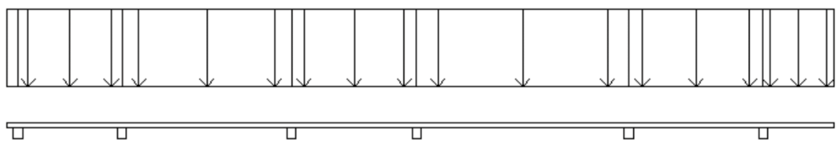


b	520	mm
h	980	mm
Zt	0	mm
Areal	5,10E+05	mm2
Iy	4,08E+10	mm4

1.2 SØYLER OG OPPLEGGSPUNKT [mm]

Opplegg	Søyler på bjelkens underside				Søyler på bjelkens overside			
nr	kode	lengde	h/diameter	b(tverretn)	kode	lengde	h/diameter	b(tverretn)
1	Fri		500					
2	Fri		500					
3	Fri		500					
4	Fri		500					
5	Fri		500					
6	Fri		500					

1.3 LASTBILDE



Lastfaktorer

	Nedbøyning	Risskontroll	Bruddgrense
Permanent last	1,00	1,00	1,20
Variabel last	0,60	0,60	1,50

PSI-Faktor Kategori D : butikker
Krav maks.nedbøyning Konstruksjoner med alminnelige brukskrav eller estetiske krav

Pålitelighetsklasse: 3 Bjelkens romvekt: 2500 kg/m3

Jevnt fordelt last (kN/m)

Felt nr	Egenvekt	Permanent last	Variabel last
v. utkrag.	12,74	90,00	73,40
1	12,74	90,00	73,40
2	12,74	90,00	73,40
3	12,74	90,00	73,40
4	12,74	90,00	73,40
5	12,74	90,00	73,40
h. utkrag.	12,74	90,00	73,40

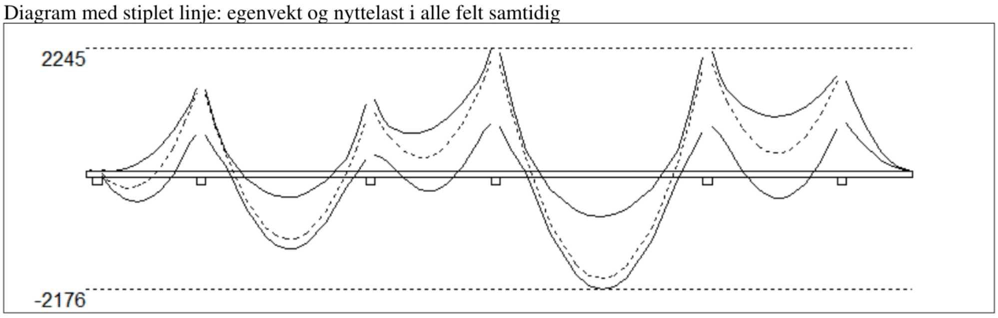
1.4 MATERIALDATA

Korreksjonsfaktor for Emodul pga tilslag	1	Eksponeringsklasse	XC1	XC1
Materialkoeffisient betong	1,5	Lite korrosjonsømfintlig armering		
Materialkoeffisient stål	1,15	Dimensjonerende levetid		50
Betongkvalitet	B45 (C45/55)			
Tilslagets spesifikke tyngde (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	N	**Min. overdekning**	**uk**	**ok**
Armering flytegrense	500	Min krav	15	15
Bøyler flytegrense	500	Toleransekrav +/-	10	10
Relativ fuktighet %	40	Min. nominell overdekning	25	25
Betongens alder ved pålastning (døgn)	28			
Effektiv høyde, h0 (EN 1992-1-1 3.1.4(5))	340			
største tilslagsstørrelse, dg(mm)	22	Kryptall, FI 28_5000		1,64
Korttids Emodul, Ecm	36300	Svinntøyning, FI 0_28		-0,00009
Trykkfasthet, fcd	25,5	Svinntøyning, FI 28_5000		-0,0003
Middel verdi av strekkfasthet, fctm	3,8			
Strekkfasthet, fctd	1,51			

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

2.1 MOMENTDIAGRAMMER FOR MAKS OG MIN MOMENT I BRUDDGRENSETILSTAND, MED NYTTELAST I UGUNSTIGE FELT

Diagram med stiplet linje: egenvekt og nyttelast i alle felt samtidig



Største negative feltmomenter (strekk i uk)(kNm)

	Bruksgrense		Bruddgrense	
Felt	Mg	Mg+Mp	Mg	Mg+Mp
1	-140	-292	-168	-564
2	-551	-856	-661	-1423
3	0	-95	0	-371
4	-870	-1323	-1044	-2176
5	0	-125	0	-497

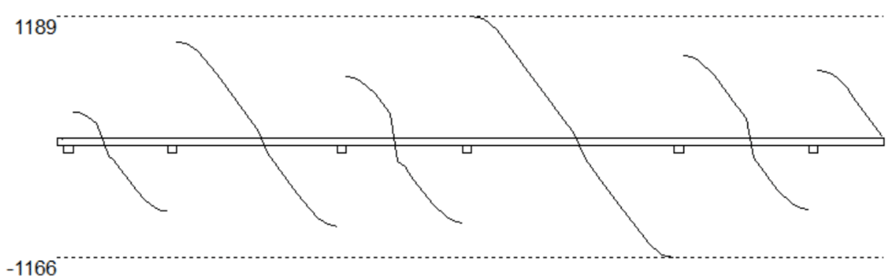
Mg: permanent last   Mp: variabel last

Største positive momenter ved kant av opplegg (kNm)

	Bruksgrense		Bruddgrense	
Opplegg	Mg	Mg+Mp	Mg	Mg+Mp
1	6	9	8	18
2	632	939	758	1526
3	459	760	551	1303
4	899	1365	1078	2245
5	855	1312	1026	2169
6	727	1076	873	1745

2.2 SKJÆRKRAFTDIAGRAM I BRUDDGRENSETILSTAND

MED NYTTELAST I UGUNSTIGSTE FELT.
REDUSERT SKJÆRKRAFT MOT OPPLEGG.



Største skjærkraft i bruddgrensetilstand (kN)

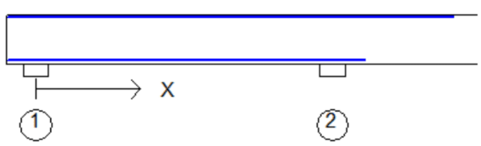
	Venstre side av opplegg		Høyre side av opplegg	
Opplegg	Vgamma	Vredusert	Vgamma	Vredusert
1	-81	-8	468	255
2	-925	-712	1151	938
3	-1074	-861	813	600
4	-1039	-826	1402	1189
5	-1380	-1166	1021	808
6	-913	-700	873	659

3.1-1 BESTEMT ARMERING I FELT

Kantavstand er avstand fra senter av armering til underkant eller overkant
Toleranseavvik for overdekning: +/- 10 mm
X1 og X2 er regnet fra senter av venstre opplegg i betraktet felt.

Bestemt armering i overkant i felt nr: 1

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-575	8300	8875	45	65



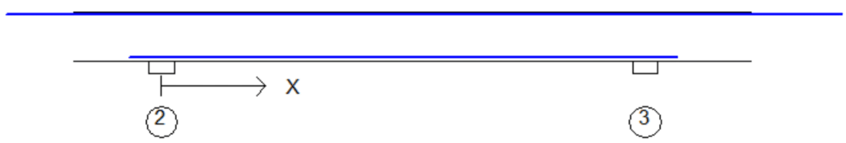
Bestemt armering i underkant i felt nr: 1

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand

2	32	1	-575	6530	7105	45	65

Bestemt armering i overkant i felt nr: 2

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-3080	13510	16590	45	65

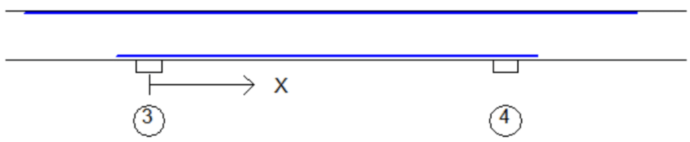


Bestemt armering i underkant i felt nr: 2

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-640	10240	10880	45	65
2	32	1	1600	8790	7190	45	65

Bestemt armering i overkant i felt nr: 3

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
4	32	1	-2470	9700	12170	45	65

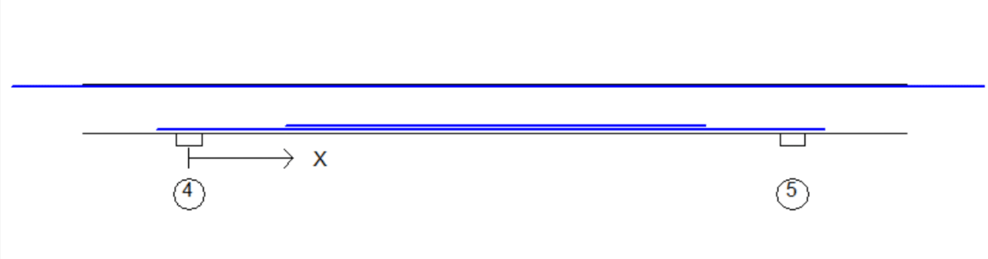


Bestemt armering i underkant i felt nr: 3

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-640	7730	8370	45	65

Bestemt armering i overkant i felt nr: 4

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-3520	15800	19320	45	65

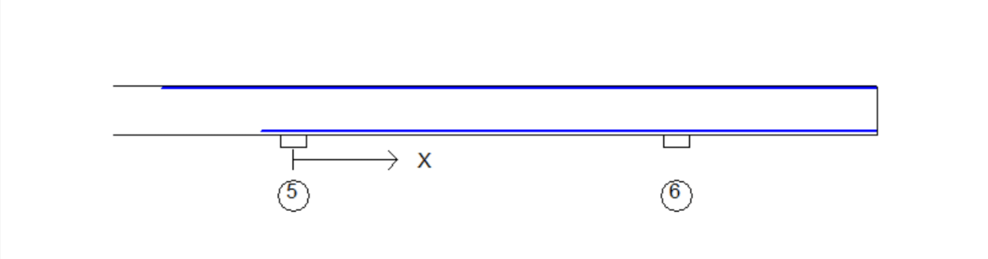


Bestemt armering i underkant i felt nr: 4

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-640	12640	13280	45	65
2	32	1	1910	10260	8350	45	65
3	32	2	1910	10260	8350	117	137

Bestemt armering i overkant i felt nr: 5

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
5	32	1	-2620	11585	14205	45	65



Bestemt armering i underkant i felt nr: 5

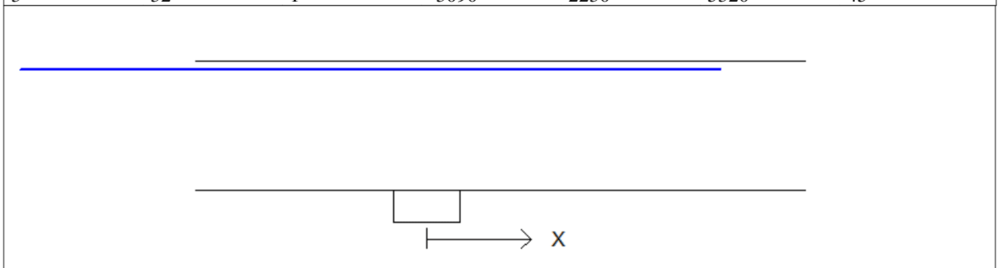
Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-640	11585	12225	45	65

3.1-2 BESTEMT ARMERING I OVERKANT VED OPPLEGG

Denne armeringen kommer i tillegg til overkantarmering i felt.

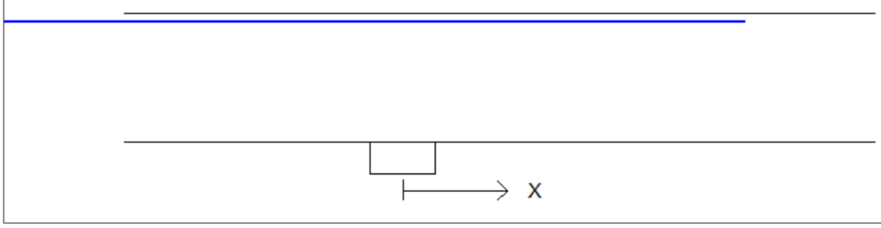
Støttearmering over opplegg nr: 2

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-3090	2230	5320	45



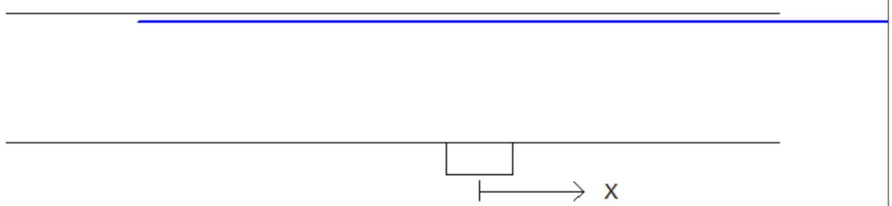
Støttearmering over opplegg nr: 4

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-3470	2600	6070	45



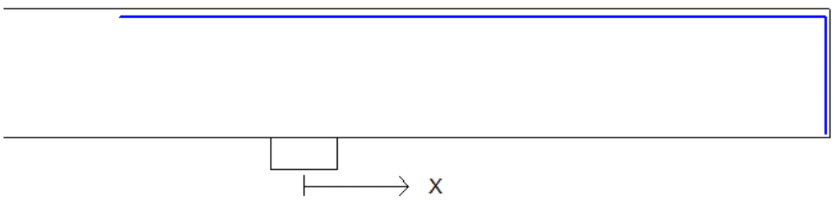
Støttearmering over opplegg nr: 5

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-2590	3590	6180	45



Støttearmering over opplegg nr: 6 *Forankring = forankringsfaktor for høyre bjelkenende (0-1)*

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	Overdekning	Forankring
12	16	1	-1400	3965	55	1



3.2 FORANKRINGSLENGDE OG UTNYTTELSE AV ARMERING

D: armeringsdiameter
Forankringslengde i underkant: 30 x D   Forankringslengde i overkant: 43 x D
Kapasitetskurver for moment (M/Md):
- Det er tatt hensyn til skjærkraftbidrag
- M/Md (uk) viser utnyttelse av bestemt armering i uk
- M/Md (ok) viser utnyttelse av bestemt armering i ok

3.3 FORANKRINGSARMERING (bøyler) I UNDERKANT VED ENDEOPPLEGG

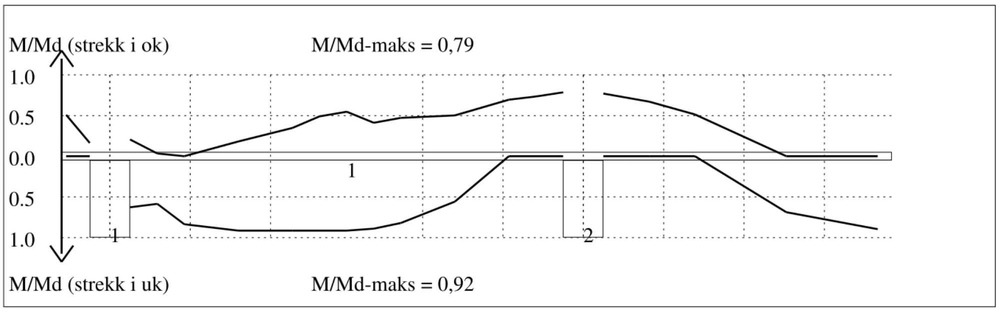
Opplegg nr 1
Det trengs ikke forankringsbøyler.
Opplegg nr 6
Det trengs ikke forankringsbøyler.

3.4 MINIMUMSARMERING (mm2)

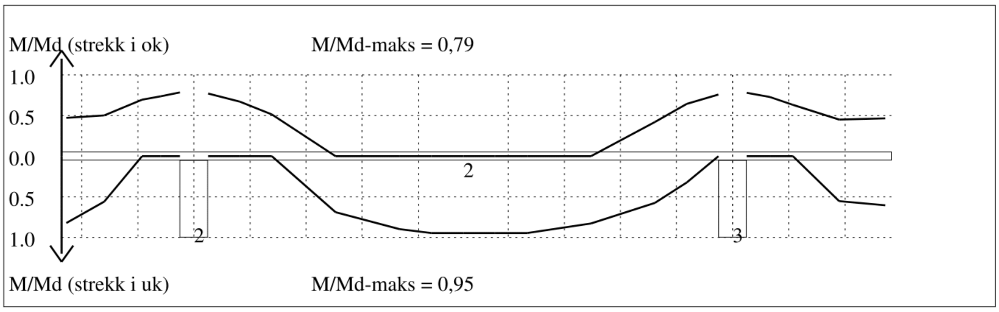
Det er regnet med minst 2 stenger inn over opplegg

Felt nr	Uk-venstre opplegg	Uk-høyre opplegg	Underkant i felt	Overkant i felt
1	1608	1608	939	939
2	1608	1608	939	939
3	1608	1657	939	939
4	1608	1608	939	939
5	1657	1608	939	939

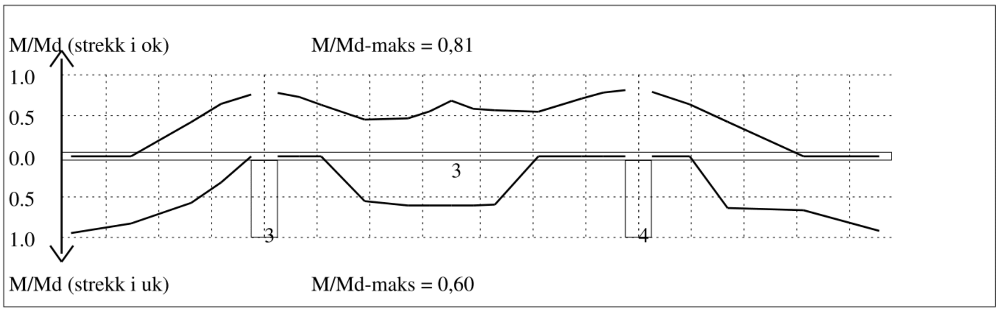
4.1 MOMENTKONTROLL



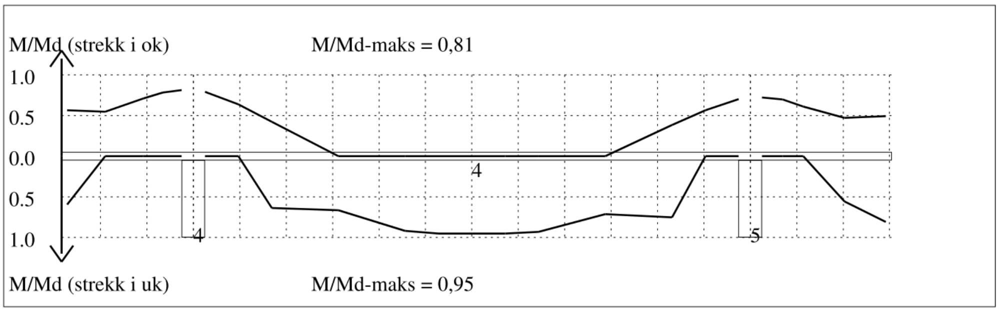
Momentkontroll for felt nr 1   Avstand mellom vertikalstreker = 1.0 m



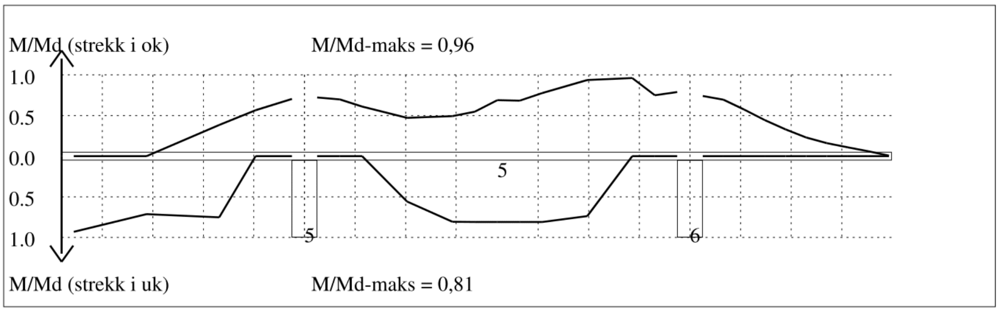
Momentkontroll for felt nr 2   Avstand mellom vertikalstreker = 1.0 m



Momentkontroll for felt nr 3 Avstand mellom vertikalstreker = 1.0 m



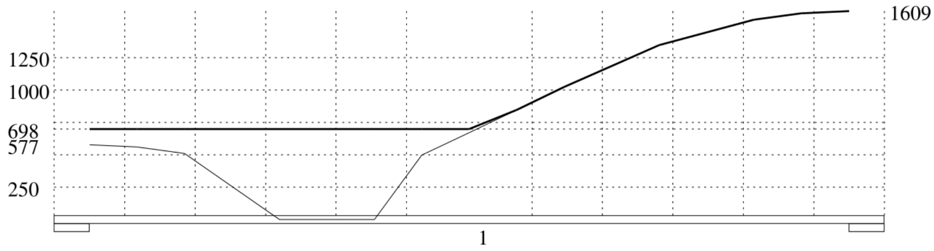
Momentkontroll for felt nr 4 Avstand mellom vertikalstreker = 1.0 m



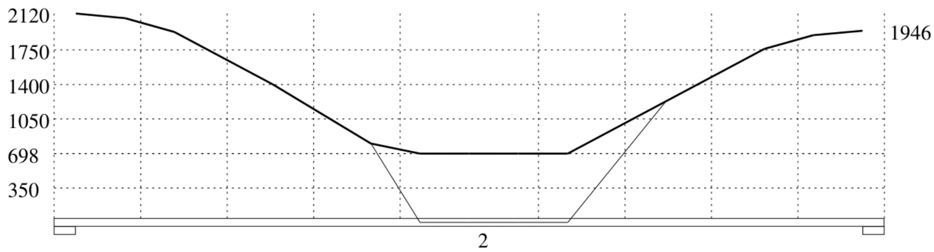
Momentkontroll for felt nr 5 Avstand mellom vertikalstreker = 1.0 m

4.2 SKJÆRARMERING

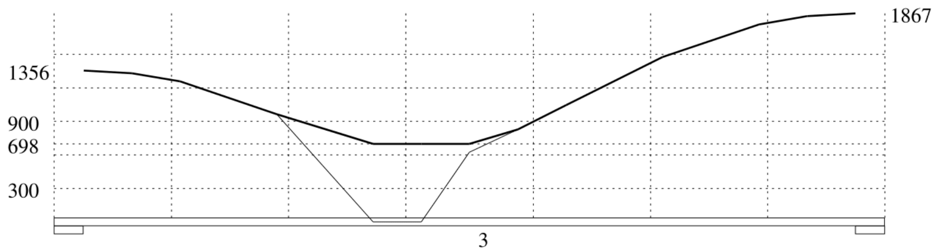
Skjærarmering i felt nr 0 (minimum skjærarmering) = 698 mm2/m   Maks bøyleavstand = 510mm
Maks. statisk nødvendig skjærarmering = 0 mm2/m



Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 510mm
Skjærarmering (mm2/m) for felt nr 1   Avstand mellom vertikalstreker = 0.5 m

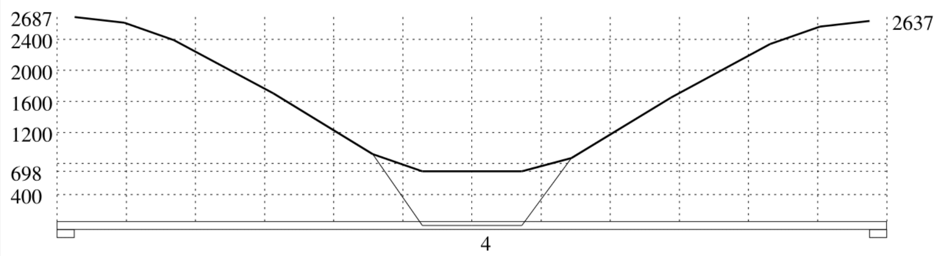


Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 510mm
Skjærarmering (mm2/m) for felt nr 2   Avstand mellom vertikalstreker = 1.0 m



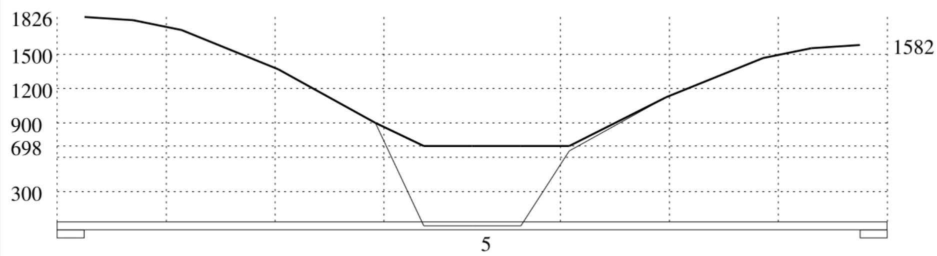
Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 510mm
Skjærarmering (mm2/m) for felt nr 3   Avstand mellom vertikalstreker = 1.0 m

Tittel			Side 11
Prosjekt	Ordre	Sign	Dato 30-05-2017



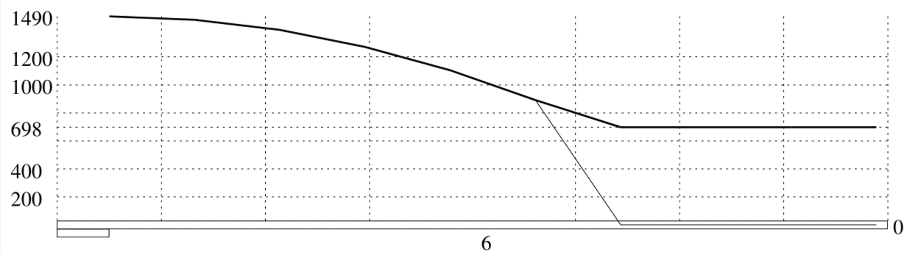
Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 494mm

Skjærarmering (mm2/m) for felt nr 4   Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 510mm

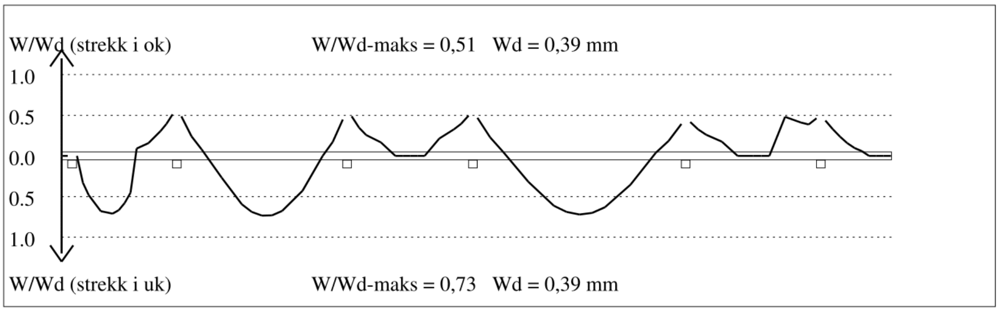
Skjærarmering (mm2/m) for felt nr 5   Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 698 mm2/m   Maks bøyleavstand = 510mm

Skjærarmering (mm2/m) for felt nr 6   Avstand mellom vertikalstreker = 0.5 m

4.3 RISSKONTROLL



4.4 NEDBØYNINGER I BRUKSGRENSETILSTAND (mm)

	Permanent last		Permanent + variabel last (lang tid)	
Felt	Kort tid	Lang tid	Nyttelast i alle felt	Nyttelast i betraktet felt
V. utkrager	0	0	0	0
1	0	-1	-1	0
2	9	13	17	18
3	-2	-4	-5	-1
4	16	23	31	33
5	-2	-5	-6	0
H. utkrager	12	22	28	29

5.1 OPPLEGGSKREFTER I BRUKSGRENSETILSTAND (kN og kNm) (alle lastfaktorer = 1)

Ng,Mg: fra egenvekt. Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-242	0,00	-173	0,00	-47	0,00	-194	0,00
2	-939	0,00	-671	0,00	-257	0,00	-423	0,00
3	-769	0,00	-550	0,00	-416	0,00	-288	0,00
4	-1051	0,00	-751	0,00	-273	0,00	-562	0,00
5	-1022	0,00	-730	0,00	-514	0,00	-335	0,00
6	-783	0,00	-560	0,00	-249	0,00	-386	0,00

5.2 OPPLEGGSKREFTER I BRUDDGRENSETILSTAND (kN og kNm)

Ng,Mg: fra egenvekt. Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-291	0,00	-260	0,00	-70	0,00	-290	0,00
2	-1127	0,00	-1006	0,00	-385	0,00	-634	0,00
3	-923	0,00	-824	0,00	-625	0,00	-433	0,00
4	-1262	0,00	-1127	0,00	-410	0,00	-844	0,00
5	-1226	0,00	-1095	0,00	-771	0,00	-503	0,00

6	-940	0,00	-839	0,00	-374	0,00	-579	0,00

Dataprogram: E-BJELKE versjon 6.5.6 Laget av Sletten Byggdata
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002 + NA:2008
Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\Ny geometri\Original.ebj

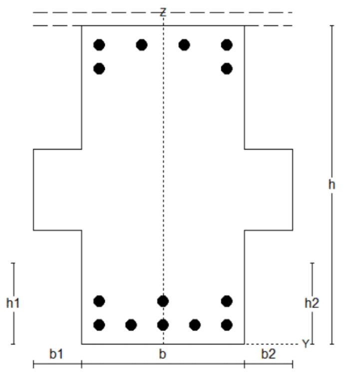
INNHOLD

1.0 Materialdata

Korreksjonsfaktor for Emodul pga tilslag	1	**Data vedr. spennarmert element**		
Materialkoeffisient betong	1,5	Sylindertrykkfasthet ved avspenning (fckj)		24
Materialkoeffisient stål	1,15	Sylindertrykkfasthet ved transport(fckj)		32
Betongkvalitet	B45(C45/55)	Betongens alder ved avspenning (døgn)		1
Densitet (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	R	Eksponeringsklasser	uk:XC1	ok:XC1
Armering flytegrense	500	Lite korrosjonsømfintlig armering		
Bøyler flytegrense	500	Dimensjonerende levetid		50
Relativ fuktighet i lagringsperioden %	50			
Relativ fuktighet i ferdig bygg %	40	**Min. overdekning (mm)**	**uk**	**ok**
Betongens alder ved pålastning (døgn)	28	Min. krav	15	15
Effektiv høyde, h0 (EN1992-1-1 3.1.4(5))	317	Toleranse	10	10
		Nominell overdekning	25	25
Korttids Emodul, Ecm	36300			
Dimensjonerende trykkfasthet, fcd	25,5			
Aksial strekkfasthet, fctm	4			
Dimensjonerende strekkfasthet, fctd	1,51			
Kryptall, FI 0_28	0,93	Svinntøyning, 0_28		-0,0001
Kryptall, FI 28_9000	1,64	Svinntøyning, 0_9000		-0,00053

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

1.1 Tverrsnitt (med samvirkepåstøp vist stiplet)

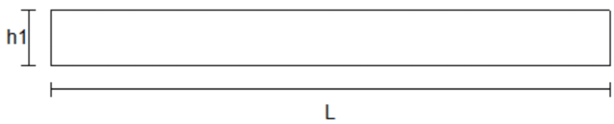


b	500	mm
h	980	mm
b1	150	mm
h1	250	mm
b2	150	mm
h2	250	mm
Zt	488	mm
Areal	5,64E+05	mm2
Ic	3,96E+10	mm4

1.2 Armeringsdata

Kant	Lag nr	Kantavstand	Slakkarmering	Spennarmering
ok	1	60	4d 32	
ok	2	132	2d 32	
uk	1	60	5d 32	
uk	2	132	3d 32	

1.3 Bjelkeprofil



L	9835	mm
h1	980	mm
h2	980	mm

Elementvekt: 13,9 tonn

Utkragerlengde (mm)

	Venstre ende	Høyre ende
Utløfting	2450	2450
Lagring	2450	2450
Transport	2450	2450
Ferdig montert	1200	110

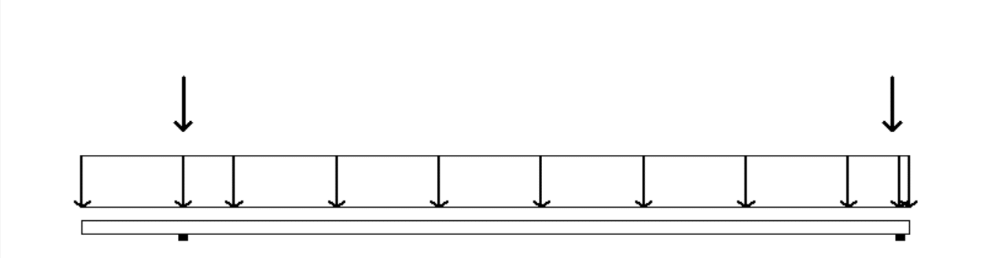
Minste effektive oppleggsbredde: 200 mm

1.4 Lastfaktor og pålitelighetsklasse

	Lastfaktor			BENYTTES:
	Nedbøyning	Risskontroll	Bruddgr. B1	Bruddgr. B2
Permanent last	1,00	1,00	1,35	1,20
Variabel last	1,00	0,60	1,05	1,50

Pålitelighetsklasse	3
PSI -faktor	Kategori D : butikker
Krav til maks. nedbøyning	Konstruksjoner der nedbøyning fører til skader
Formsug ved avforming	1,00 kN/m
Elementets romvekt	2500 kg/m3
Horisontalkraft i oppleggspunkt (H/N)	0,20

1.6 Egenvekt, permanent last og nyttelast



Jevnt fordelt last (kN/m)

Last på bjelken

	v. utkrager	midtfelt	h. utkrager
Egenvekt	14,13	14,13	14,13
Permanent last	6,13	6,13	6,13
Variabel last	5,00	5,00	5,00

Last på venstre hylle

Permanent last	46,20	46,20	46,20
Variabel last	37,70	37,70	37,70

Last på høyre hylle

Permanent last	43,80	43,80	43,80
Variabel last	35,80	35,80	35,80

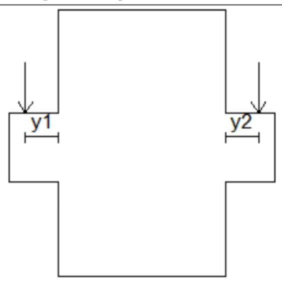
Punktlaster

Permanent last G (kN)	Variabel last P (kN)	Avstand til venstre ende: x (mm)	Lastbredde b (mm)	Lastplassering
0,00	194,50	1200	200	høyre hylle
0,00	194,50	1200	200	venstre hylle
0,00	194,50	9635	200	høyre hylle
0,00	194,50	9635	200	venstre hylle

1.7 Samvirkepåstøp (med dynamisk last)

Bredde av påstøp	800	mm	Betongkvalitet	B45(C45/55)
Tykkelse av påstøp, tp	40	mm	Antall armeringsjern	0
Fra ok bjelke til uk påstøp	0	mm		
			Fugetype: Svært glatt	
Påført egenvekt: Lastandel etter samvirke	0,0		Effektiv fugebredde	500 mm

2.1 Bjelkehylle



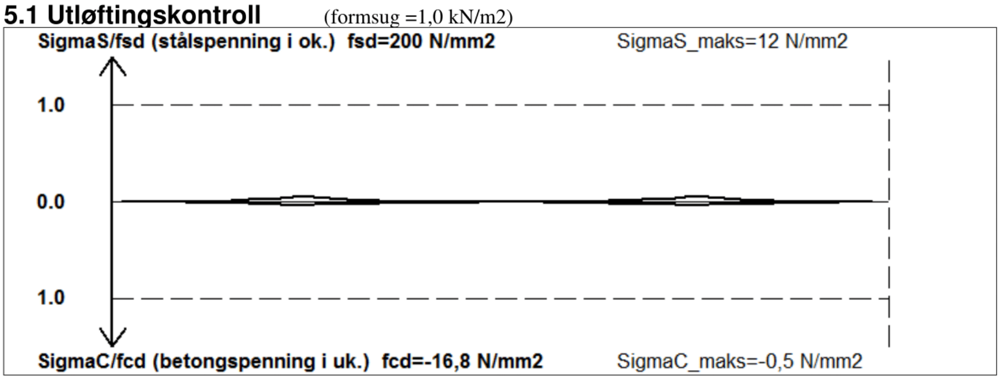
Hylle er ikke fastlåst mot vridning
Bjelken er ikke beregnet for torsjon
Ubalansert moment mot venstre (bruddgrense): 2,01 kNm/m
Det bør forankres mellom dekke og bjelke for momentet

Horisontalkraft på hylle (H/N)	0,200	
y1 _	100	mm
y2	100	mm

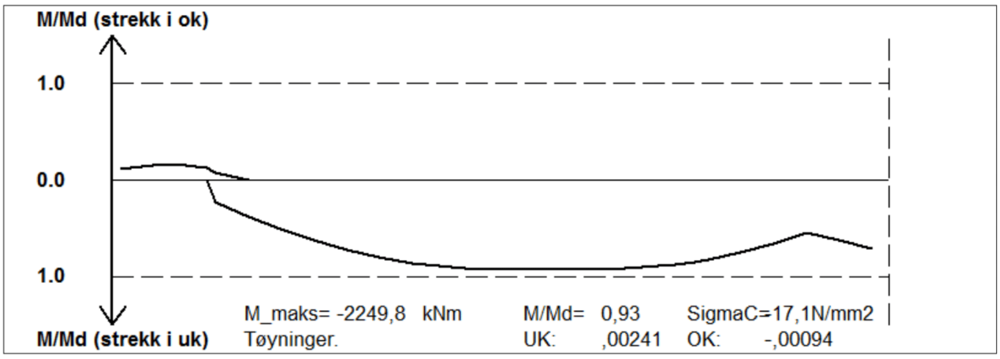
Horisontalarmering i bjelkehylle		
Overdekning	25	mm
Armeringsdiameter	12	mm

Tittel			Side 5
Prosjekt	Ordre	Sign	Dato 30-05-2017

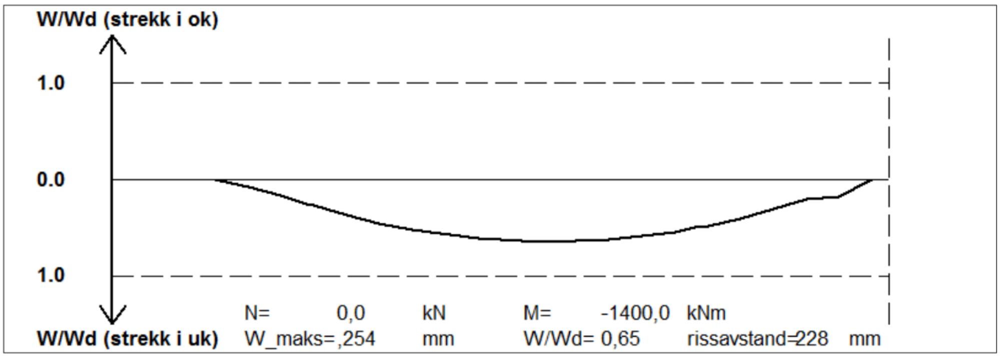
5.1 Utløftingskontroll (formsug =1,0 kN/m2)



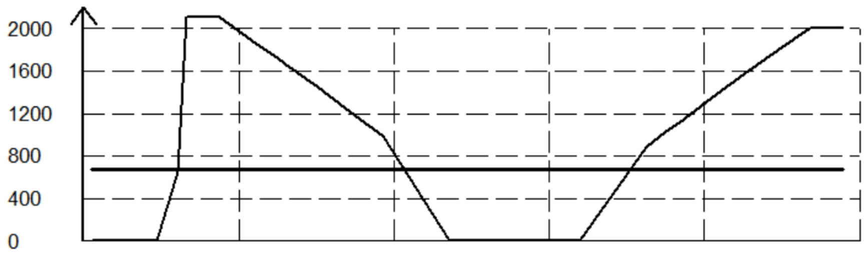
5.2 Momentkontroll



5.3 Risskontroll



5.4.0 Skjærarmering

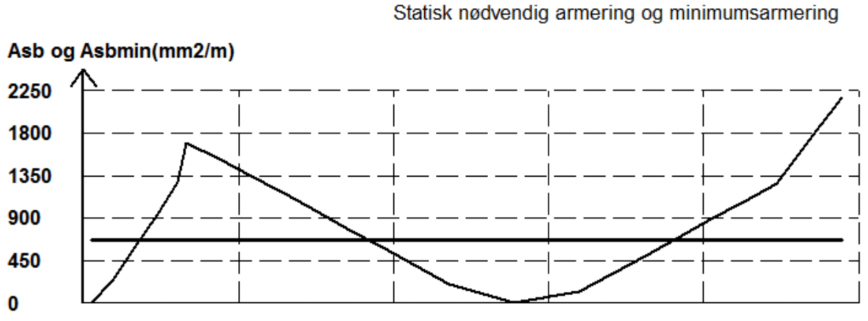


5.4.1 Skjærkraftkontroll

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert skjærkraft (kN)	Vrd,max trykk kap. (kN)	Vrd,c (kN)	Statisk nødvendig skjærarmer. (mm2/m)	Minimums-armering (mm2/m)	Maks bøyleavstand (mm)
100	25,0	24,9	2474,0	187,9	0	671	485
375	93,7	91,8	2474,0	187,9	0	671	485
650	162,4	156,8	2474,0	219,7	0	671	485
924	231,1	219,8	2474,0	248,7	0	671	485
1199	299,8	280,8	2474,0	272,2	648	671	485
1716	-963,9	-948,4	2576,2	324,3	2103	671	485
2133	-859,8	-852,4	2576,2	324,3	1890	671	485
2450	-780,4	-777,2	2576,2	324,3	1723	671	485
2549	-755,7	-753,5	2576,2	324,3	1670	671	485
2965	-651,6	-651,5	2576,2	324,3	1444	671	485
3381	-547,5	-547,5	2576,2	324,3	1214	671	485
3798	-443,4	-443,4	2576,2	324,3	983	671	485
4630	-235,3	-235,3	2576,2	324,3	0	671	485
5463	-27,1	-27,1	2576,2	324,3	0	671	485
6295	190,9	190,9	2576,2	324,3	0	671	485
7128	399,1	399,1	2576,2	324,3	885	671	485
7385	463,5	463,5	2576,2	324,3	1028	671	485
7544	503,2	503,2	2576,2	324,3	1116	671	485
7960	607,3	607,2	2576,2	324,3	1346	671	485
8376	711,4	709,1	2576,2	324,3	1572	671	485
8793	815,4	808,1	2576,2	324,3	1791	671	485
9209	919,5	904,1	2577,6	282,4	2003	671	486

Skjærameringen helningsvinkel med bjelkeakse: 90 grader
Trykkdiagonalens helningsvinkel med bjelkeakse: 39 grader

5.5.0 Skjærarmering gjennom støpeskjøt



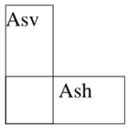
5.5.1 Skjærarmering gjennom støpeskjøt (Fordeling: Se NS-EN 1992 Figur 6.10)

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert Vrd,max (N/mm2)	Statisk nødvendig skjærarmer.	Minimums-armering (mm2/m)	Maks bøyleavstand (mm)
100	25,0	0,05	0	671	500
375	93,7	0,21	242	671	500
650	162,4	0,36	588	671	500
924	231,1	0,51	934	671	500
1199	299,8	0,66	1280	671	500
1300	-1068,0	0,84	1697	671	500
1716	-963,9	0,76	1519	671	500
2133	-859,8	0,68	1342	671	500
2450	-780,4	0,62	1203	671	500
2549	-755,7	0,60	1157	671	500
2965	-651,6	0,52	966	671	500
3381	-547,5	0,44	775	671	500
3798	-443,4	0,35	584	671	500
4630	-235,3	0,19	202	671	500
5463	-27,1	0,02	0	671	500
6295	190,9	0,15	120	671	500
7128	399,1	0,32	503	671	500
7385	463,5	0,37	621	671	500
7544	503,2	0,40	694	671	500
7960	607,3	0,48	885	671	500
8376	711,4	0,57	1076	671	500
8793	815,4	0,65	1267	671	500
9209	919,5	0,85	1715	671	500
9625	1023,6	1,05	2174	671	500

Minimumsarmering: basert på NS-EN 1992(NA.9.5N). Maks bøyleastand: basert på tidligere praksis (NS 3473 12.7.2)

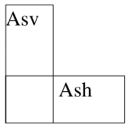
5.6 Forankringsarmering (på grunn av skjærkraft og horisontalkraft i oppleggspunkt)

Forankringsbøyler i v. ende, underkant	0	mm2:	
Forankringsbøyler i h. ende, underkant	798	mm2:	4 bøyler d 12, L=610 mm avstand til kant: 50 mm



5.7.0 Hyllearmering p.g.a. jevnt fordelt last: venstre hylle. (se også punkt 2.1)

Sted	Ngamma kN/m	Asv(oppheng.) mm2/m	Ash(ok hylle) mm2/m	Trykkbrudd Ngamma/Nd	Strekkbrudd Vred/Vrdc
V. utkrager	112,0	280	257	0,084	0,200
Midtfelt	112,0	280	257	0,084	0,200



5.7.1 Hyllearmering p.g.a. jevnt fordelt last: høyre hylle. (se også punkt 2.1)

Sted	Ngamma kN/m	Asv(oppheng.) mm2/m	Ash(ok hylle) mm2/m	Trykkbrudd Ngamma/Nd	Strekkbrudd Vred/Vrdc
V. utkrager	106,3	266	243	0,079	0,190
Midtfelt	106,3	266	243	0,079	0,190

5.7.2 Hyllearmering p.g.a. punktlaster (se også punkt 2.1)

Last nr	Hylle	Avst.til v. ende(mm)	Ngamma (kN)	Oppheng. Asv(mm2)	Ok hylle Ash(mm2)	Fordelings - br.(mm)	Trykkbrud d N/Nd	Strekkbr. Vred/Vrdc
1	høyre	1200	291,8	729	709	450	0,484	0,992
2	venstre	1200	291,8	729	709	450	0,484	0,992
3	høyre	9635	291,8	729	709	450	0,484	0,992
4	venstre	9635	291,8	729	709	450	0,484	0,992

6.1 Nedbøyning (mm)

(G1=egenvekt av bjelken G2=påført permanent last P=variabel last)			
	V. utkrager	Midtfelt	H. utkrager
Avforming	0	0	
G1: ved montasje	0	1	
G1+G2: ved montasje	-5	10	
G1+G2+P.langtidsdel ved montasje	-7	14	
G1+G2 etter lang tid	-7	15	
G1+G2+P_langtidsdel etter lang tid	-9	18	
G1+G2+P_total etter lang tid	-10	21	

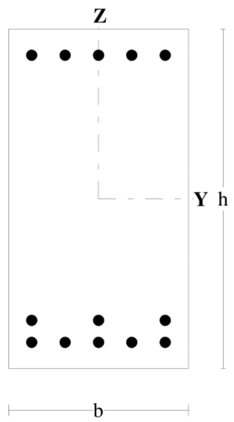
7.1 Oppleggskrefter (kN) (alle lastfaktorer = 1 i bruksgrense)

	------------- Bruksgrense -------------			------------- Bruddgrense -------------		
	Permanent last	Variabel	All last	Permanent last	Variabel	All last
v. opplegg	611,5	828,5	1440,0	733,8	1242,7	1976,5
h. opplegg	472,9	721,6	1194,4	567,4	1082,3	1649,8

Tittel			Side 1
Prosjekt	Ordre	Sign	Dato 11-06-2017

Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\Ny geometri\originalt tverrsnitt.bts
Dataprogram: BTSNITT versjon 6.3.3 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002 + NA:2008

Tverrsnitt



b	520	mm
h	980	mm
Zt	0	mm
Yt	0	mm
Areal	5,10E+05	mm2
Iy	4,08E+10	mm4
Iz	1,15E+10	mm4

Maks. bøyleavstand: 400 mm
Spesielle krav: endesoner og seismisk
Se NS-EN 1992-1-1 9.5.3 og NA.9.5.3(3)
bøylearmering d16

Armeringsdata

Kant	Lag nr	Kantavst.	Slakkarmering	Spennarmering
ok	1	75	5d 32	
uk	1	75	5d 32	
uk	2	139	3d 32	

Materialdata

Korreksjonsfakt. for Emodul pga tilslag	1,00	Eksponeringsklasse	XC1
Materialfaktor betong	1,50	Lite korrosjonsømfintlig armering	
Materialfaktor stål	1,15	Dimensjonerende levetid 100 år	
Betongkvalitet	B45 (C 45/55)		
Densitet kg/m3	2400	**Minimum overdekning**	
Sement i fasthetsklasse	N	Min. krav	25
Armering flytegrense	500	Toleranse	10
Skjærarmering flytegrense	500	Min. nominell overdekning	35
Relativ fuktighet	40%		
Betongens alder ved pålastning (døgn)	28		
Effektiv høyde, h0 (NS-EN 1992-1-1 (B.6))	340		

NA.6.2.2(1)Følgende krav til tilslag er oppfyllt
(1.Største tilslag etter NS-EN 12620 D>=16mm. 2.Det grove tilslaget>=50% av total tilslagsmengde.
3.Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet)

Korttids Emodul, Ecm	36300	Kryptall, FI 0_28	1,18
Trykkfasthet, fcd	25,5	Kryptall, FI 28_5000	1,64
Middelverdi av strekkfasthet, fctm	3,80	Svinntøyning, 0_28	-,00009
Strekkfasthet, fctd	1,51	Svinntøyning, 28_25000	-,00030

Pålitelighetsklasse: 3

Lastfaktorer	Bruksgrense	Risskontroll	Bruddgrense B1	Bruddgrense B2	**PSI-Faktor:** Kategori D - Butikker
Permanent last (G)	1,00	1,00	1,35	1,20	**Krav maks.nedbøyning:**
Variabel last (P)	0,60	0,60	1,05	1,50	Alminnelige bruks-/estetiske krav

Snittkrefter. Lasttilfelle nr 1

Permanent last

Mg_Y	-855,0 kNm
Ng	0,0 kN

Variabel last

Mp_Y	-755,0 kNm
Np	0,0 kN

Positiv moment-og kraftvektorer i Y og Z-retning. Positiv Mg_Y,Mp_Y gir strekk i ok

Dimensjonerende snittkrefter

Momentkontroll: Programmet regner ikke med ekstra momentbidrag fra skjærkraften (NS-EN 1992 6.2.3(7))

Momentkontroll. Lasttilfelle nr 1

N+Nsp+tap	0,0
M+Msp+tap	-2158,5
M/Md	0,95
tøyning i ok	-,00126
tøyning i uk	,00268
SigmaC i ok	-21,08
SigmaC i uk	0,00
SigmaS i ok	

Skjærkontroll. Lasttilfelle nr 1

Vgamma (kN)	0,0
Vredusert (kN)	0,0
Vccd Trykkbr.	2529,9
Vcd (uarmert).	323,3
Stat.nødv(mm2/m)	0
Min.arm. (mm2/m)	698
Maks bøyleavstand	484

Risskontroll. Lasttilfelle nr 1

N (kN)	0,0
M (kNm)	-1308,0
Min. overdekning	35
Overdekning (mm)	35
Største rissavstand (mm)	291
Beregnet rissvidde(mm)	0,317
tillatt rissvidde	0,390

8.2.2. Modified beam

Includes

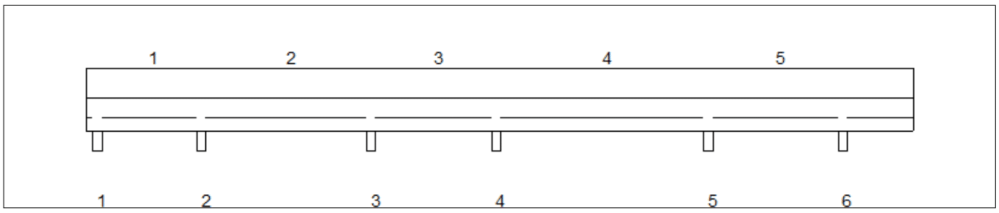
- K-bjelke calculation
- E-bjelke calculation
- BTSNITT calculation

Tittel			Side 1
Prosjekt	Ordre	Sign	Dato 09-06-2017

Dataprogram: K-Bjelke versjon 6.3.3 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1:2004 + NA:2008 og NS-EN 1990:2002
Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\uten momentledd eq cs.kbj

INNHOLD

1.0 BJELKE MED 6 OPPLEGGSPUNKTER

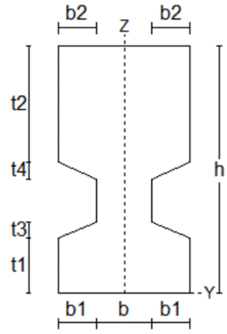


1.1 SPENNVIDDER [mm], OG TVERRSNITTYPER

Felt nr	v.utkr.	1	2	3	4	5	h.utkr.
Spennvidde	600	5890	9600	7090	12000	7620	3990
Tverrsnittype	1	1	1	1	1	1	1

Tittel			Side 2
Prosjekt	Ordre	Sign	Dato 09-06-2017

Tverrsnittype 1

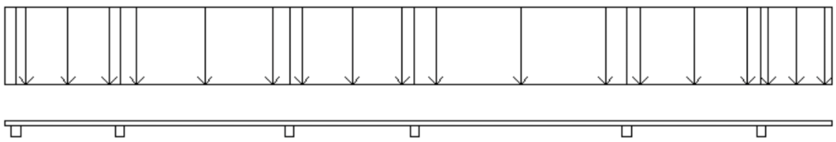


b	220	mm
b1	150	mm
b2	150	mm
h	980	mm
t1	218	mm
t2	462	mm
t3	65	mm
t4	65	mm
Zt	510	mm
Areal	4,39E+05	mm2
Iy	3,92E+10	mm4

1.2 SØYLER OG OPPLEGGSPUNKT [mm]

Opplegg	Søyler på bjelkens underside				Søyler på bjelkens overside			
nr	kode	lengde	h/diameter	b(tverretn)	kode	lengde	h/diameter	b(tverretn)
1	Fri		500					
2	Fri		500					
3	Fri		500					
4	Fri		500					
5	Fri		500					
6	Fri		500					

1.3 LASTBILDE



Lastfaktorer

	Nedbøyning	Risskontroll	Bruddgrense
Permanent last	1,00	1,00	1,20
Variabel last	0,60	0,60	1,50

PSI-Faktor Kategori D : butikker
Krav maks.nedbøyning Konstruksjoner med alminnelige brukskrav eller estetiske krav

Pålitelighetsklasse: 3	Bjelkens romvekt: 2500 kg/m3

Jevnt fordelt last (kN/m)

Felt nr	Egenvekt	Permanent last	Variabel last
v. utkrag.	10,98	90,00	73,40
1	10,98	90,00	73,40
2	10,98	90,00	73,40
3	10,98	90,00	73,40
4	10,98	90,00	73,40
5	10,98	90,00	73,40
h. utkrag.	10,98	90,00	73,40

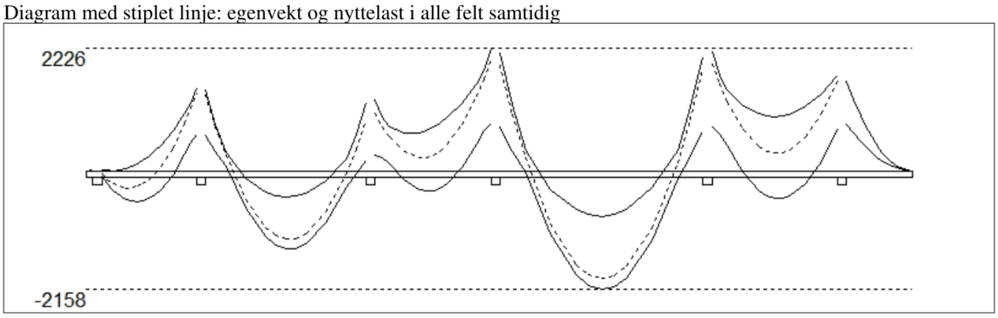
1.4 MATERIALDATA

Korreksjonsfaktor for Emodul pga tilslag	1	Eksponeringsklasse	XC1	XC1
Materialkoeffisient betong	1,5	Lite korrosjonsømfintlig armering		
Materialkoeffisient stål	1,15	Dimensjonerende levetid		50
Betongkvalitet	B45 (C45/55)			
Tilslagets spesifikke tyngde (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	N	**Min. overdekning**	**uk**	**ok**
Armering flytegrense	500	Min krav	15	15
Bøyler flytegrense	500	Toleransekrav +/-	10	10
Relativ fuktighet %	40	Min. nominell overdekning	25	25
Betongens alder ved pålastning (døgn)	28			
Effektiv høyde, h0 (EN 1992-1-1 3.1.4(5))	259			
største tilslagsstørrelse, dg(mm)	22	Kryptall, FI 28_5000		1,71
Korttids Emodul, Ecm	36300	Svinntøyning, FI 0_28		-0,0001
Trykkfasthet, fcd	25,5	Svinntøyning, FI 28_5000		-0,00032
Middel verdi av strekkfasthet, fctm	3,8			
Strekkfasthet, fctd	1,51			

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

2.1 MOMENTDIAGRAMMER FOR MAKS OG MIN MOMENT I BRUDDGRENSETILSTAND, MED NYTTELAST I UGUNSTIGE FELT

Diagram med stiplet linje: egenvekt og nyttelast i alle felt samtidig



Største negative feltmomenter (strekk i uk)(kNm)

	Bruksgrense		Bruddgrense	
Felt	Mg	Mg+Mp	Mg	Mg+Mp
1	-137	-290	-165	-561
2	-541	-846	-650	-1412
3	0	-97	0	-373
4	-855	-1308	-1026	-2158
5	0	-127	0	-500

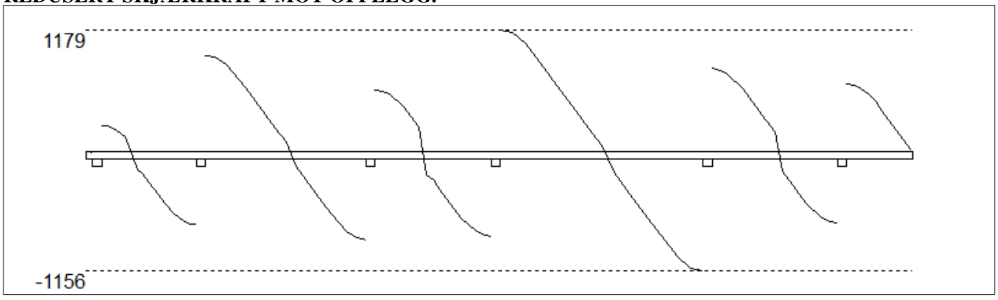
Mg: permanent last   Mp: variabel last

Største positive momenter ved kant av opplegg (kNm)

	Bruksgrense		Bruddgrense	
Opplegg	Mg	Mg+Mp	Mg	Mg+Mp
1	6	9	7	19
2	621	928	745	1513
3	451	752	541	1293
4	883	1350	1060	2226
5	841	1298	1009	2151
6	715	1064	858	1730

2.2 SKJÆRKRAFTDIAGRAM I BRUDDGRENSETILSTAND

MED NYTTELAST I UGUNSTIGSTE FELT.
REDUSERT SKJÆRKRAFT MOT OPPLEGG.



Største skjærkraft i bruddgrensetilstand (kN)

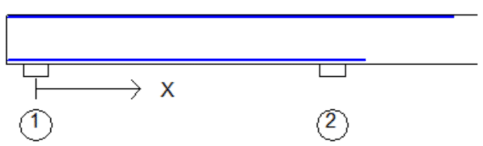
	Venstre side av opplegg		Høyre side av opplegg	
Opplegg	Vgamma	Vredusert	Vgamma	Vredusert
1	-81	-8	465	254
2	-917	-705	1141	930
3	-1065	-854	807	596
4	-1031	-819	1390	1179
5	-1368	-1156	1013	802
6	-906	-695	865	653

3.1-1 BESTEMT ARMERING I FELT

Kantavstand er avstand fra senter av armering til underkant eller overkant
Toleranseavvik for overdekning: +/- 10 mm
X1 og X2 er regnet fra senter av venstre opplegg i betraktet felt.

Bestemt armering i overkant i felt nr: 1

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-575	8300	8875	45	65



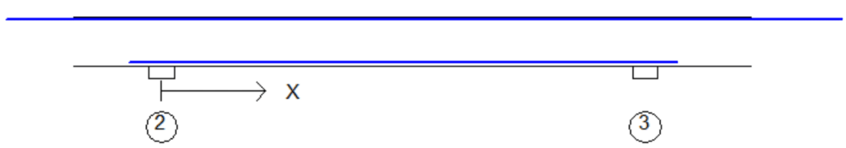
Bestemt armering i underkant i felt nr: 1

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand

2	32	1	-575	6530	7105	45	65

Bestemt armering i overkant i felt nr: 2

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-3080	13510	16590	45	65

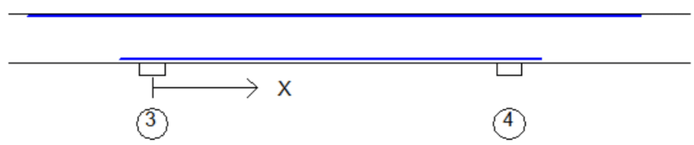


Bestemt armering i underkant i felt nr: 2

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-640	10240	10880	45	65
2	32	1	1600	8790	7190	45	65

Bestemt armering i overkant i felt nr: 3

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
4	32	1	-2470	9700	12170	45	65

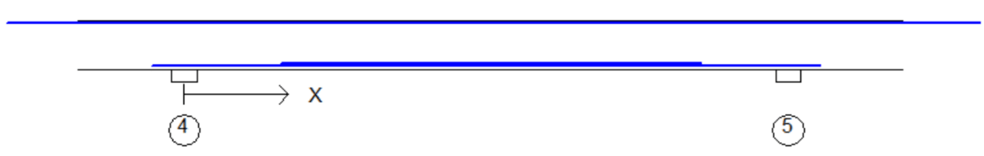


Bestemt armering i underkant i felt nr: 3

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-640	7730	8370	45	65

Bestemt armering i overkant i felt nr: 4

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-3520	15800	19320	45	65

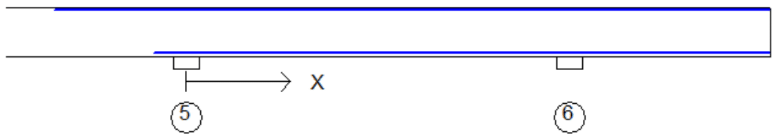


Bestemt armering i underkant i felt nr: 4

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
3	32	1	-640	12640	13280	45	65
2	32	1	1910	10260	8350	45	65
3	32	2	1910	10260	8350	111	131

Bestemt armering i overkant i felt nr: 5

Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
5	32	1	-2620	11585	14205	45	65



Bestemt armering i underkant i felt nr: 5

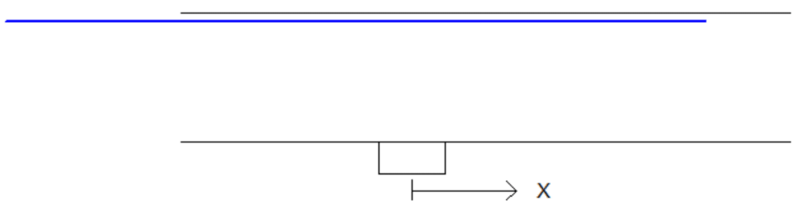
Antall	Diameter	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning	Kantavstand
2	32	1	-640	11585	12225	45	65

3.1-2 BESTEMT ARMERING I OVERKANT VED OPPLEGG

Denne armeringen kommer i tillegg til overkantarmering i felt.

Støttearmering over opplegg nr: 2

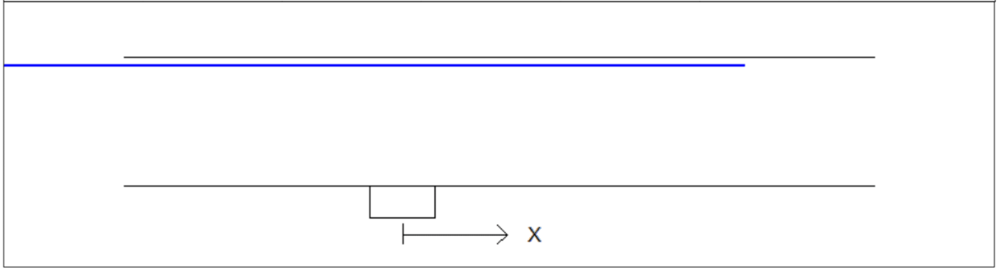
Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-3090	2230	5320	45



Tittel			Side 7
Prosjekt	Ordre	Sign	Dato 09-06-2017

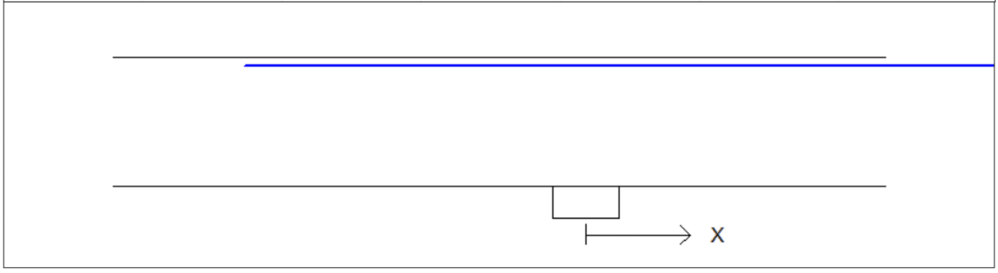
Støttearmering over opplegg nr: 4

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-3470	2600	6070	45



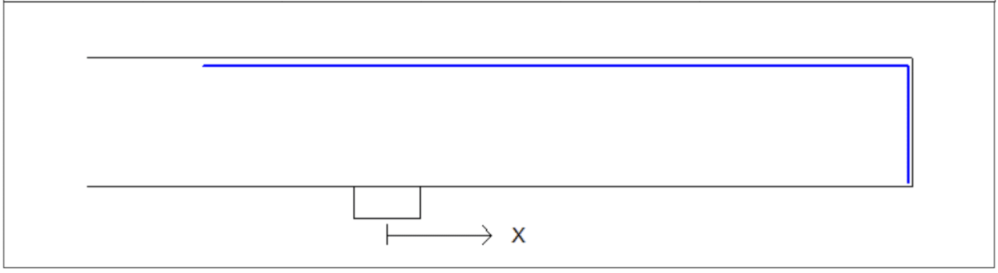
Støttearmering over opplegg nr: 5

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	L (mm)	Overdekning
3	32	1	-2590	3590	6180	45



Støttearmering over opplegg nr: 6 *Forankring = forankringsfaktor for høyre bjelkenende (0-1)*

Antall	Ø (mm)	Lag	X1 (mm)	X2 (mm)	Overdekning	Forankring
12	16	1	-1400	3965	55	1



3.2 FORANKRINGSLENGDE OG UTNYTTELSE AV ARMERING

D: armeringsdiameter
Forankringslengde i underkant: 30 x D   Forankringslengde i overkant: 43 x D
Kapasitetskurver for moment (M/Md):
- Det er tatt hensyn til skjærkraftbidrag
- M/Md (uk) viser utnyttelse av bestemt armering i uk
- M/Md (ok) viser utnyttelse av bestemt armering i ok

3.3 FORANKRINGSARMERING (bøyler) I UNDERKANT VED ENDEOPPLEGG

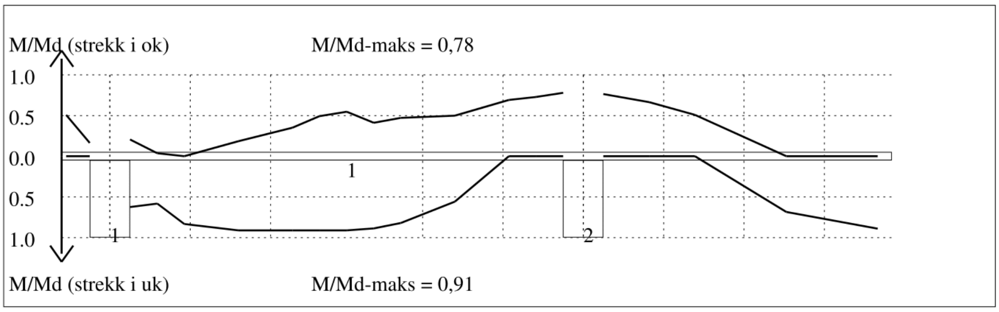
Opplegg nr 1
Det trengs ikke forankringsbøyler.
Opplegg nr 6
Det trengs ikke forankringsbøyler.

3.4 MINIMUMSARMERING (mm2)

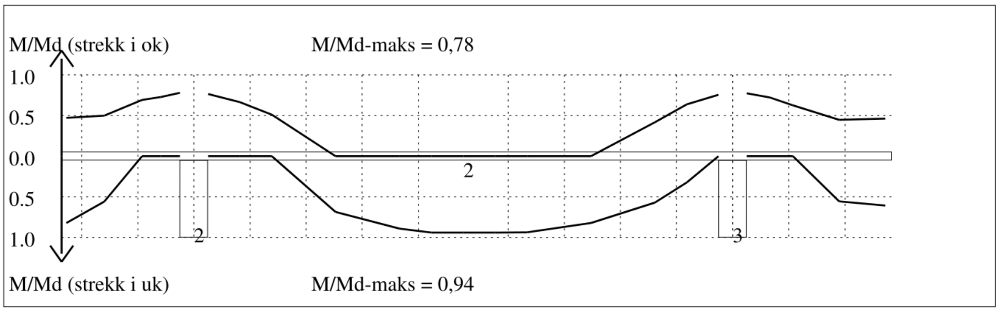
Det er regnet med minst 2 stenger inn over opplegg

Felt nr	Uk-venstre opplegg	Uk-høyre opplegg	Underkant i felt	Overkant i felt
1	1608	1608	770	770
2	1608	1608	770	770
3	1608	1608	770	770
4	1608	1608	770	770
5	1608	1608	770	770

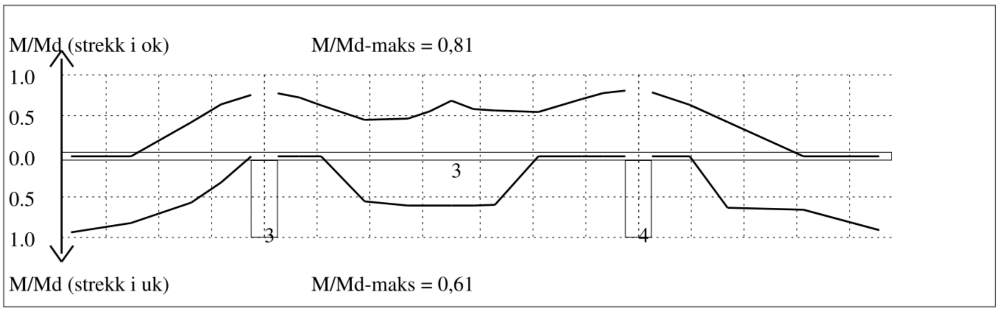
4.1 MOMENTKONTROLL



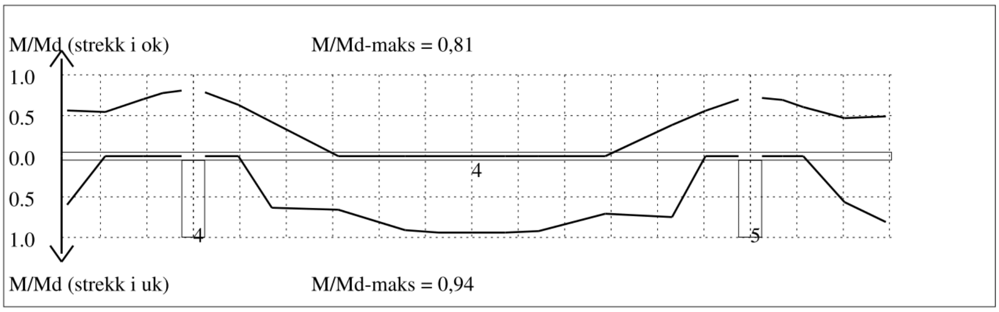
Momentkontroll for felt nr 1 Avstand mellom vertikalstreker = 1.0 m



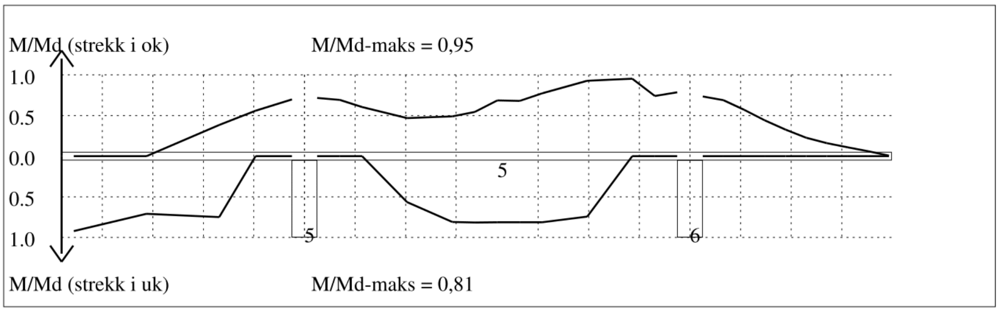
Momentkontroll for felt nr 2 Avstand mellom vertikalstreker = 1.0 m



Momentkontroll for felt nr 3 Avstand mellom vertikalstreker = 1.0 m



Momentkontroll for felt nr 4 Avstand mellom vertikalstreker = 1.0 m

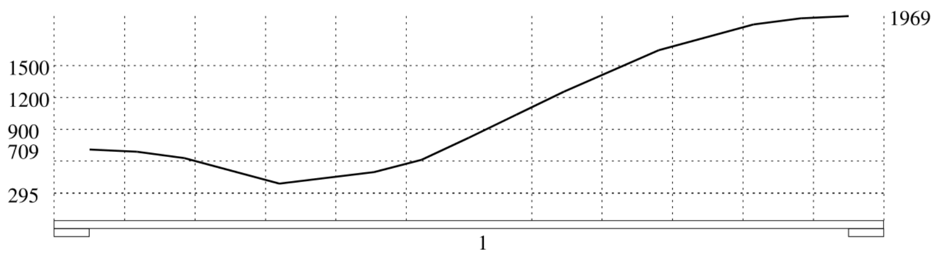


Momentkontroll for felt nr 5 Avstand mellom vertikalstreker = 1.0 m

4.2 SKJÆRARMERING

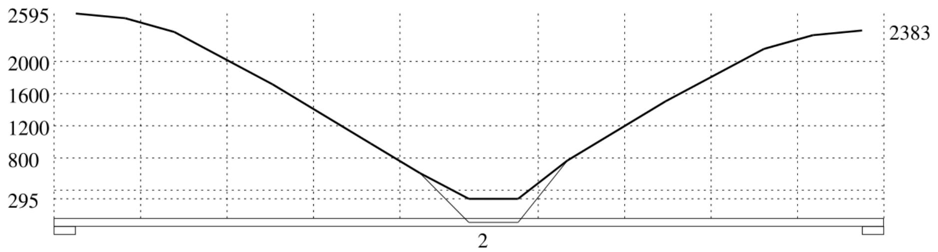
Skjærarmering i felt nr 0 (minimum skjærarmering) = 295 mm2/m   Maks bøyleavstand = 510mm

Maks. statisk nødvendig skjærarmering = 0 mm2/m



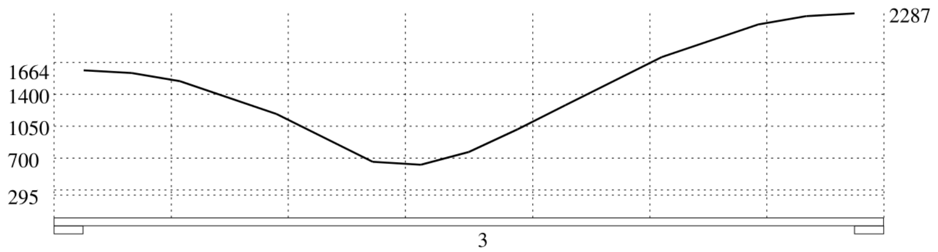
Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 510mm

Skjærarmering (mm2/m) for felt nr 1   Avstand mellom vertikalstreker = 0.5 m



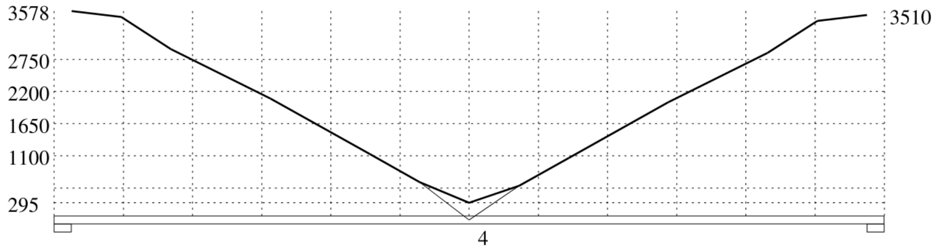
Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 510mm

Skjærarmering (mm2/m) for felt nr 2   Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 510mm

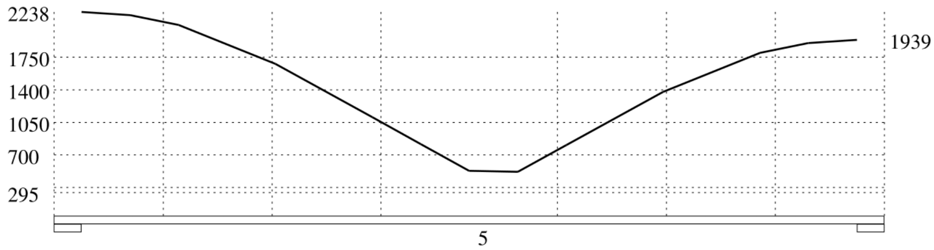
Skjærarmering (mm2/m) for felt nr 3   Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 495mm

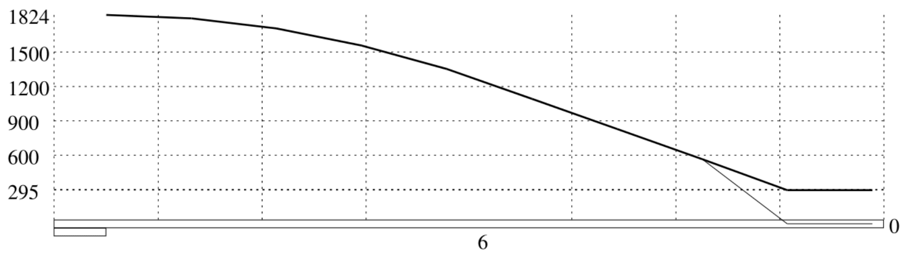
Skjærarmering (mm2/m) for felt nr 4   Avstand mellom vertikalstreker = 1.0 m

Trykkbruddkapasiteten er overskredte. V/Vd =1,10



Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 510mm

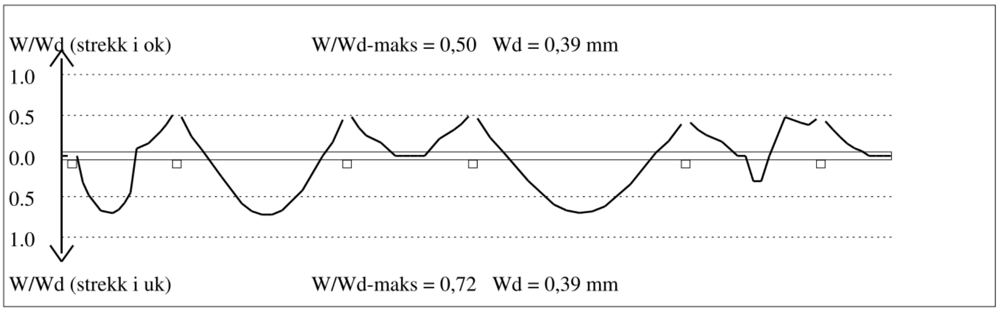
Skjærarmering (mm2/m) for felt nr 5   Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 295 mm2/m   Maks bøyleavstand = 510mm

Skjærarmering (mm2/m) for felt nr 6   Avstand mellom vertikalstreker = 0.5 m

4.3 RISSKONTROLL



4.4 NEDBØYNINGER I BRUKSGRENSETILSTAND (mm)

	Permanent last		Permanent + variabel last (lang tid)	
Felt	Kort tid	Lang tid	Nyttelast i alle felt	Nyttelast i betraktet felt
V. utkrager	0	0	0	0
1	0	-1	-1	0
2	9	13	17	18
3	-2	-4	-5	-1
4	16	24	32	33
5	-2	-5	-7	0
H. utkrager	12	23	29	30

5.1 OPPLEGGSKREFTER I BRUKSGRENSETILSTAND (kN og kNm) (alle lastfaktorer = 1)

Ng,Mg: fra egenvekt. Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-238	0,00	-173	0,00	-47	0,00	-194	0,00
2	-923	0,00	-671	0,00	-257	0,00	-423	0,00
3	-756	0,00	-550	0,00	-416	0,00	-288	0,00
4	-1033	0,00	-751	0,00	-273	0,00	-562	0,00
5	-1004	0,00	-730	0,00	-514	0,00	-335	0,00
6	-770	0,00	-560	0,00	-249	0,00	-386	0,00

5.2 OPPLEGGSKREFTER I BRUDDGRENSETILSTAND (kN og kNm)

Ng,Mg: fra egenvekt. Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-286	0,00	-260	0,00	-70	0,00	-290	0,00
2	-1107	0,00	-1006	0,00	-385	0,00	-634	0,00
3	-907	0,00	-824	0,00	-625	0,00	-433	0,00
4	-1240	0,00	-1127	0,00	-410	0,00	-844	0,00
5	-1205	0,00	-1095	0,00	-771	0,00	-503	0,00

6	-924	0,00	-839	0,00	-374	0,00	-579	0,00

Tittel			Side 1
Prosjekt	Ordre	Sign	Dato 30-05-2017

Dataprogram: E-BJELKE versjon 6.5.6 Laget av Sletten Byggdata
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002 + NA:2008
Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\Ny geometri\Mod1.ebj

INNHOLD

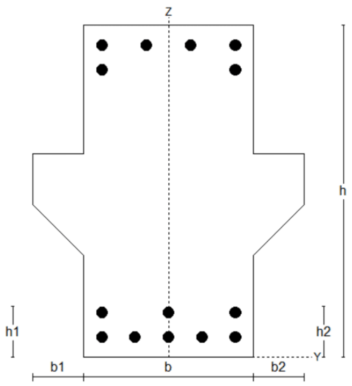
1.0 Materialdata

Korreksjonsfaktor for Emodul pga tilslag	1	**Data vedr. spennarmert element**		
Materialkoeffisient betong	1,5	Sylindertrykkfasthet ved avspenning (fckj)		24
Materialkoeffisient stål	1,15	Sylindertrykkfasthet ved transport(fckj)		32
Betongkvalitet	B45(C45/55)	Betongens alder ved avspenning (døgn)		1
Densitet (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	R	Eksponeringsklasser	uk:XC1	ok:XC1
Armering flytegrense	500	Lite korrosjonsømfintlig armering		
Bøyler flytegrense	500	Dimensjonerende levetid		50
Relativ fuktighet i lagringsperioden %	50			
Relativ fuktighet i ferdig bygg %	40	**Min. overdekning (mm)**	**uk**	**ok**
Betongens alder ved pålastning (døgn)	28	Min. krav	15	15
Effektiv høyde, h0 (EN1992-1-1 3.1.4(5))	329	Toleranse	10	10
		Nominell overdekning	25	25
Korttids Emodul, Ecm	36300			
Dimensjonerende trykkfasthet, fcd	25,5			
Aksial strekkfasthet, fctm	4			
Dimensjonerende strekkfasthet, fctd	1,51			
Kryptall, FI 0_28	0,92	Svinntøyning, 0_28		-0,0001
Kryptall, FI 28_9000	1,63	Svinntøyning, 0_9000		-0,00053

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

Tittel			Side 2
Prosjekt	Ordre	Sign	Dato 30-05-2017

1.1 Tverrsnitt

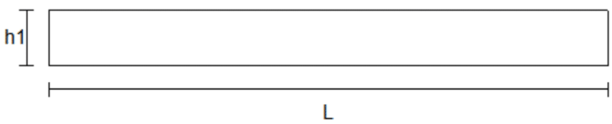


b	500	mm
h	980	mm
b1	150	mm
h1	150	mm
b2	150	mm
h2	150	mm
Zt	489	mm
Areal	5,57E+05	mm2
Ic	3,96E+10	mm4

1.2 Armeringsdata

Kant	Lag nr	Kantavstand	Slakkarmering	Spennarmering
ok	1	60	4d 32	
ok	2	132	2d 32	
uk	1	60	5d 32	
uk	2	132	3d 32	

1.3 Bjelkeprofil



L	9835	mm
h1	980	mm
h2	980	mm

Elementvekt: 13,2 tonn

Utkragerlengde (mm)

	Venstre ende	Høyre ende
Utløfting	2450	2450
Lagring	2450	2450
Transport	2450	2450
Ferdig montert	1200	110

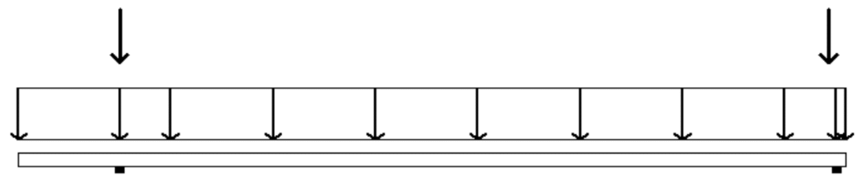
Minste effektive oppleggsbredde: 200 mm

1.4 Lastfaktor og pålitelighetsklasse

	Lastfaktor			BENYTTES:
	Nedbøyning	Risskontroll	Bruddgr. B1	Bruddgr. B2
Permanent last	1,00	1,00	1,35	1,20
Variabel last	1,00	0,60	1,05	1,50

Pålitelighetsklasse	3
PSI -faktor	Kategori D : butikker
Krav til maks. nedbøyning	Konstruksjoner der nedbøyning fører til skader
Formsug ved avforming	1,00 kN/m
Elementets romvekt	2500 kg/m3
Horisontalkraft i oppleggspunkt (H/N)	0,20

1.6 Egenvekt, permanent last og nyttelast



Jevnt fordelt last (kN/m)

Last på bjelken

	v. utkrager	midtfelt	h. utkrager
Egenvekt	13,38	13,38	13,38
Permanent last	6,13	6,13	6,13
Variabel last	5,00	5,00	5,00

Last på venstre hylle

Permanent last	46,20	46,20	46,20
Variabel last	37,70	37,70	37,70

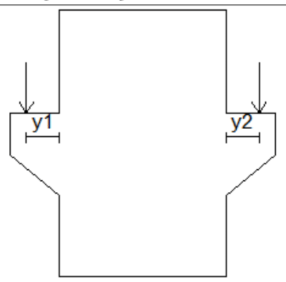
Last på høyre hylle

Permanent last	43,80	43,80	43,80
Variabel last	35,80	35,80	35,80

Punktlaster

Permanent last G (kN)	Variabel last P (kN)	Avstand til venstre ende: x (mm)	Lastbredde b (mm)	Lastplassering
0,00	194,50	1200	200	høyre hylle
0,00	194,50	1200	200	venstre hylle
0,00	194,50	9635	200	høyre hylle
0,00	194,50	9635	200	venstre hylle

2.1 Bjelkehylle



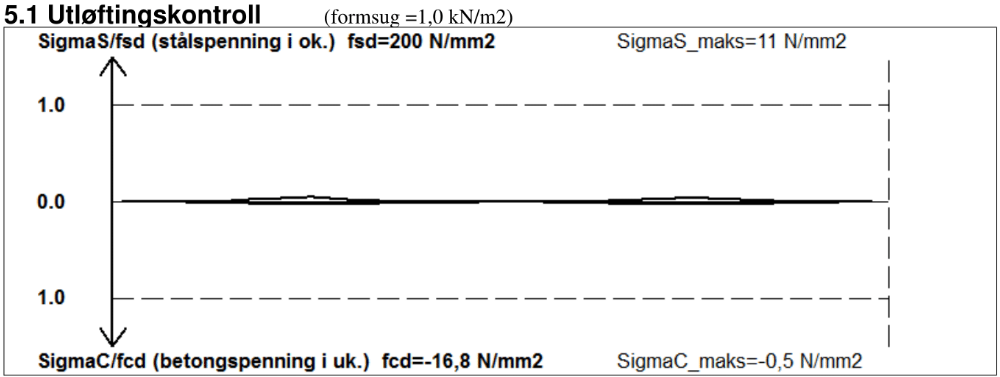
Hylle er ikke fastlåst mot vridning
Bjelken er ikke beregnet for torsjon
Ubalansert moment mot venstre (bruddgrense): 2,01 kNm/m
Det bør forankres mellom dekke og bjelke for momentet

Horisontalkraft på hylle (H/N)	0,200	
y1 _	100	mm
y2	100	mm

Horisontalarmering i bjelkehylle		
Overdekning	25	mm
Armeringsdiameter	12	mm

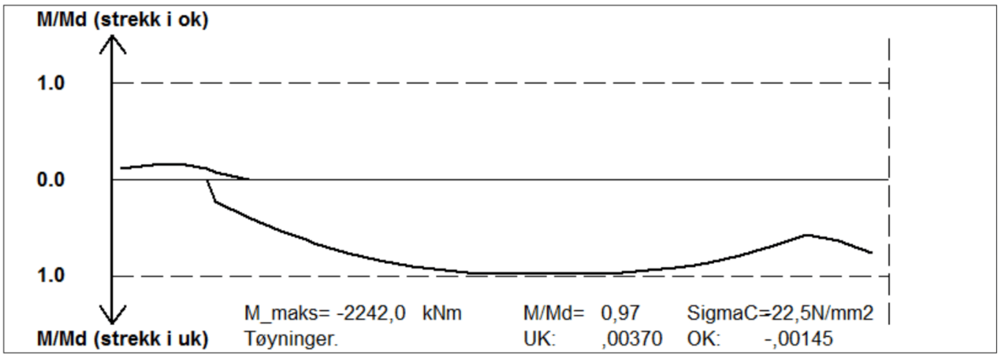
5.1 Utløftingskontroll

(formsug =1,0 kN/m2)

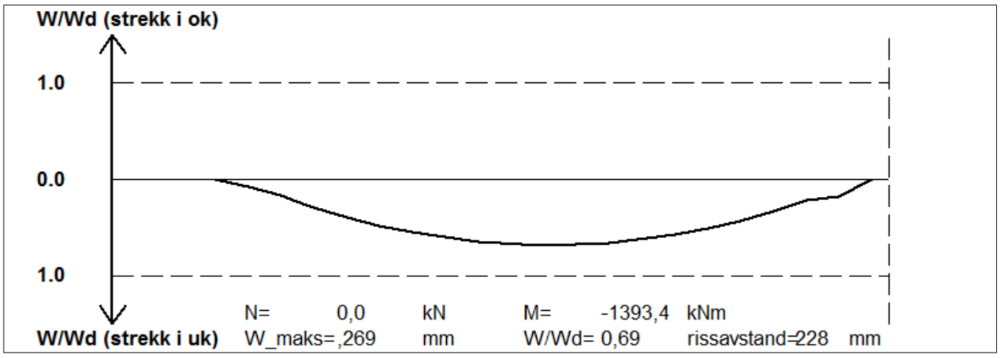


Tittel			Side 5
Prosjekt	Ordre	Sign	Dato 30-05-2017

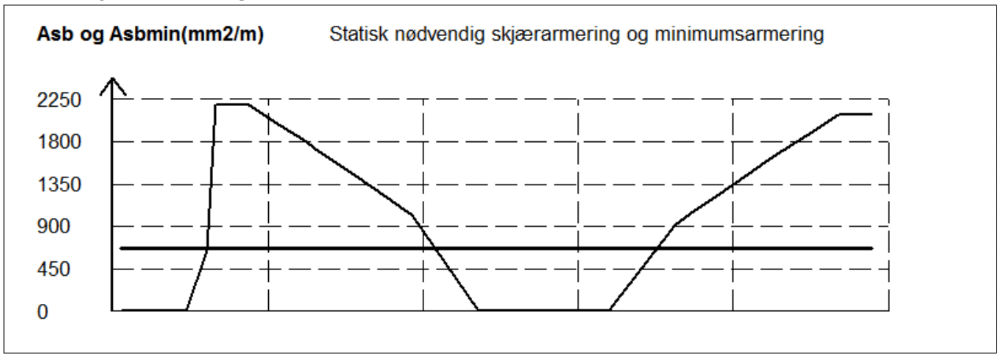
5.2 Momentkontroll



5.3 Risskontroll



5.4.0 Skjærarmering



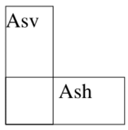
5.4.1 Skjærkraftkontroll

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert skjærkraft (kN)	Vrd,max trykk kap. (kN)	Vrd,c (kN)	Statisk nødvendig skjærarmer. (mm2/m)	Minimums-armering (mm2/m)	Maks bøyleavstand (mm)
100	24,9	24,8	2474,0	187,9	0	671	485
375	93,4	91,6	2474,0	187,9	0	671	485
650	161,8	156,4	2474,0	219,7	0	671	485
924	230,3	219,3	2474,0	248,7	0	671	485
1199	298,7	280,2	2474,0	272,2	647	671	485
1716	-960,4	-945,4	2465,7	317,1	2190	671	485
2133	-856,7	-849,6	2465,7	317,1	1968	671	485
2450	-777,6	-774,5	2465,7	317,1	1794	671	485
2549	-753,0	-750,8	2465,7	317,1	1739	671	485
2965	-649,3	-649,2	2465,7	317,1	1504	671	485
3381	-545,6	-545,6	2465,7	317,1	1264	671	485
3798	-441,9	-441,9	2465,7	317,1	1023	671	485
4630	-234,4	-234,4	2465,7	317,1	0	671	485
5463	-27,0	-27,0	2465,7	317,1	0	671	485
6295	190,3	190,3	2465,7	317,1	0	671	485
7128	397,7	397,7	2465,7	317,1	921	671	485
7385	461,8	461,8	2465,7	317,1	1070	671	485
7544	501,4	501,4	2465,7	317,1	1161	671	485
7960	605,1	605,0	2465,7	317,1	1401	671	485
8376	708,8	706,6	2465,7	317,1	1637	671	485
8793	812,5	805,4	2465,7	317,1	1865	671	485
9209	916,2	901,2	2469,4	279,2	2084	671	486

Skjærameringen helningsvinkel med bjelkeakse: 90 grader
Trykkdiagonalens helningsvinkel med bjelkeakse: 39 grader

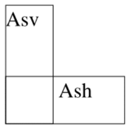
5.6 Forankringsarmering (på grunn av skjærkraft og horisontalkraft i oppleggspunkt)

Forankringsbøyler i v. ende, underkant	0	mm2:	
Forankringsbøyler i h. ende, underkant	877	mm2:	4 bøyler d 12, L=650 mm avstand til kant: 50 mm



5.7.0 Hyllearmering p.g.a. jevnt fordelt last: venstre hylle. (se også punkt 2.1)

Sted	Ngamma kN/m	Asv(oppheng.) mm2/m	Ash(ok hylle) mm2/m	Trykkbrudd Ngamma/Nd	Strekkbrudd Vred/Vrdc
V. utkrager	112,0	280	214	0,073	0,175
Midtfelt	112,0	280	214	0,073	0,175



5.7.1 Hyllearmering p.g.a. jevnt fordelt last: høyre hylle. (se også punkt 2.1)

Sted	Ngamma kN/m	Asv(oppheng.) mm2/m	Ash(ok hylle) mm2/m	Trykkbrudd Ngamma/Nd	Strekkbrudd Vred/Vrdc
V. utkrager	106,3	266	203	0,069	0,166
Midtfelt	106,3	266	203	0,069	0,166

5.7.2 Hyllearmering p.g.a. punktlaster (se også punkt 2.1)

Last nr	Hylle	Avst.til v. ende(mm)	Ngamma (kN)	Oppheng. Asv(mm2)	Ok hylle Ash(mm2)	Fordelings - br.(mm)	Trykkbrud d N/Nd	Strekkbr. Vred/Vrdc
1	høyre	1200	291,8	729	557	500	0,379	0,912
2	venstre	1200	291,8	729	557	500	0,379	0,912
3	høyre	9635	291,8	729	557	500	0,379	0,912
4	venstre	9635	291,8	729	557	500	0,379	0,912

6.1 Nedbøyning (mm)

(G1=egenvekt av bjelken G2=påført permanent last P=variabel last)			
	V. utkrager	Midtfelt	H. utkrager
Avforming	0	0	
G1: ved montasje	0	1	
G1+G2: ved montasje	-5	10	
G1+G2+P.langtidsdel ved montasje	-7	14	
G1+G2 etter lang tid	-7	14	
G1+G2+P_langtidsdel etter lang tid	-9	18	
G1+G2+P_total etter lang tid	-10	21	

7.1 Oppleggskrefter (kN)   (alle lastfaktorer = 1 i bruksgrense)

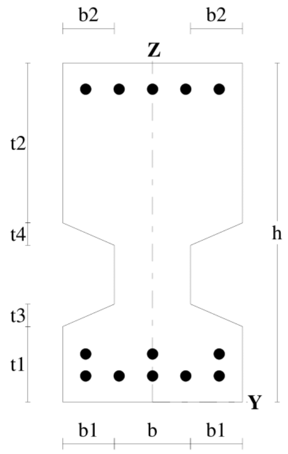
	------------- Bruksgrense -------------			------------- Bruddgrense -------------		
	Permanent last	Variabel	All last	Permanent last	Variabel	All last
v. opplegg	607,3	828,5	1435,8	728,8	1242,7	1971,5
h. opplegg	469,6	721,6	1191,2	563,6	1082,3	1645,9

Tittel	Side
	1

Prosjekt	Ordre	Sign	Dato
			09-06-2017

Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\Ny geometri\Ekvivalent tverrsnitt.bts
Dataprogram: BTSNITT versjon 6.3.3 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002 + NA:2008

Tverrsnitt



b	220	mm
b1	150	mm
b2	150	mm
h	980	mm
t1	218	mm
t2	462	mm
t3	65	mm
t4	65	mm
Zt	510	mm
Yt	0	mm
Areal	4,39E+05	mm2
Iy	3,92E+10	mm4
Iz	8,76E+09	mm4

Maks. bøyleavstand: 400 mm
Spesielle krav: endesoner og seismisk
Se NS-EN 1992-1-1 9.5.3 og NA.9.5.3(3)
bøylearmering d16

Armeringsdata

Kant	Lag nr	Kantavst.	Slakkarmering	Spennarmering
ok	1	75	5d 32	
uk	1	75	5d 32	
uk	2	139	3d 32	

Materialdata

Korreksjonsfakt. for Emodul pga tilslag	1,00	Eksponeringsklasse	XC1
Materialfaktor betong	1,50	Lite korrosjonsømfintlig armering	
Materialfaktor stål	1,15	Dimensjonerende levetid 100 år	
Betongkvalitet	B45 (C 45/55)		
Densitet kg/m3	2400	**Minimum overdekning**	
Sement i fasthetsklasse	N	Min. krav	25
Armering flytegrense	500	Toleranse	10
Skjærarmering flytegrense	500	Min. nominell overdekning	35
Relativ fuktighet	40%		
Betongens alder ved pålastning (døgn)	28		
Effektiv høyde, h0 (NS-EN 1992-1-1 (B.6))	259		

NA.6.2.2(1)Følgende krav til tilslag er oppfyllt
(1.Største tilslag etter NS-EN 12620 D>=16mm. 2.Det grove tilslaget>=50% av total tilslagsmengde.
3.Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet)

Korttids Emodul, Ecm	36300	Kryptall, FI 0_28	1,29
Trykkfasthet, fcd	25,5	Kryptall, FI 28_5000	1,71
Middelverdi av strekkfasthet, fctm	3,80	Svinntøyning, 0_28	-,00010
Strekkfasthet, fctd	1,51	Svinntøyning, 28_25000	-,00032

Pålitelighetsklasse: 3

Lastfaktorer	Bruksgrense	Risskontroll	Bruddgrense B1	Bruddgrense B2	**PSI-Faktor:** Kategori D - Butikker
Permanent last (G)	1,00	1,00	1,35	1,20	**Krav maks.nedbøyning:**
Variabel last (P)	0,60	0,60	1,05	1,50	Alminnelige bruks-/estetiske krav

Snittkrefter. Lasttilfelle nr 1

Permanent last

Mg_Y	-855,0 kNm
Ng	0,0 kN

Variabel last

Mp_Y	-755,0 kNm
Np	0,0 kN

Positiv moment-og kraftvektorer i Y og Z-retning. Positiv Mg_Y,Mp_Y gir strekk i ok

Dimensjonerende snittkrefter

Momentkontroll: Programmet regner ikke med ekstra momentbidrag fra skjærkraften (NS-EN 1992 6.2.3(7))

Momentkontroll. Lasttilfelle nr 1

N+Nsp+tap	0,0
M+Msp+tap	-2158,5
M/Md	0,95
tøyning i ok	-,00126
tøyning i uk	,00268
SigmaC i ok	-21,08
SigmaC i uk	0,00
SigmaS i ok	

Skjærkontroll. Lasttilfelle nr 1

Vgamma (kN)	0,0
Vredusert (kN)	0,0
Vccd Trykkbr.	1070,3
Vcd (uarmert).	153,9
Stat.nødv(mm2/m)	0
Min.arm. (mm2/m)	295
Maks bøyleavstand	484

Risskontroll. Lasttilfelle nr 1

N (kN)	0,0
M (kNm)	-1308,0
Min. overdekning	35
Overdekning (mm)	35
Største rissavstand (mm)	289
Beregnet rissvidde(mm)	0,317
tillatt rissvidde	0,390

8.2.3. Example with moment joints

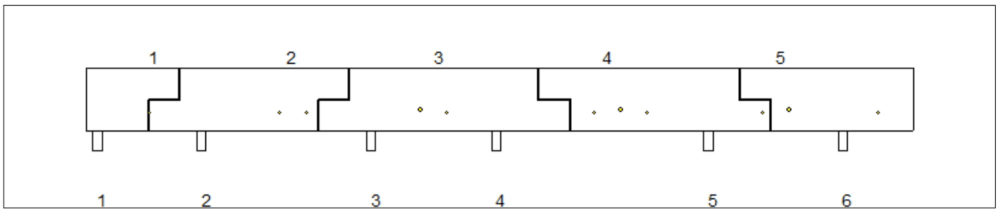
Includes:

- K-bjelke calculation

Dataprogram: K-Bjelke versjon 6.3.3 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1:2004 + NA:2008 og NS-EN 1990:2002
Data er lagret på fil: C:\Users\TOGVI\OneDrive\Masteroppgave\00 forhåndsberegninger\med momentledd.kbj

INNHOLD

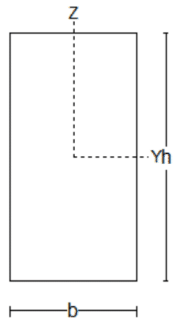
1.0 BJELKE MED 6 OPPLEGGSPUNKTER



1.1 SPENNVIDDER [mm], TVERRSNITTYPER OG MOMENTLEDD

Felt nr	v.utkr.	1	2	3	4	5	h.utkr.
Spennvidde	600	5890	9600	7090	12000	7620	3990
Tverrsnittype	1	1	1	1	1	1	1
Momentledd. Avst. fra v.ende i felt		4690	8400		2400	1800	

Tverrsnittype 1

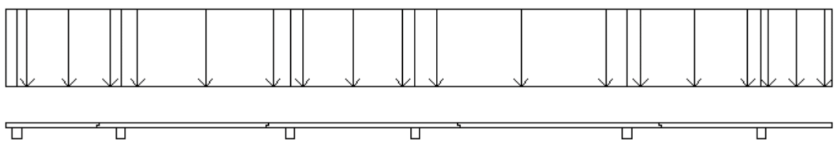


b	500	mm
h	980	mm
Zt	0	mm
Areal	4,90E+05	mm2
Iy	3,92E+10	mm4

1.2 SØYLER OG OPPLEGGSPUNKT [mm]

Opplegg	Søyler på bjelkens underside				Søyler på bjelkens overside			
nr	kode	lengde	h/diameter	b(tverretn)	kode	lengde	h/diameter	b(tverretn)
1	Fri		500					
2	Fri		500					
3	Fri		500					
4	Fri		500					
5	Fri		500					
6	Fri		500					

1.3 LASTBILDE



Lastfaktorer

	Nedbøyning	Risskontroll	Bruddgrense
Permanent last	1,00	1,00	1,20
Variabel last	0,60	0,60	1,50

PSI-Faktor Kategori D : butikker
Krav maks.nedbøyning Konstruksjoner med alminnelige brukskrav eller estetiske krav

Pålitelighetsklasse: 3	Bjelkens romvekt: 2500 kg/m3

Jevnt fordelt last (kN/m)

Felt nr	Egenvekt	Permanent last	Variabel last
v. utkrag.	12,25	90,00	73,40
1	12,25	90,00	73,40
2	12,25	90,00	73,40
3	12,25	90,00	73,40
4	12,25	90,00	73,40
5	12,25	90,00	73,40
h. utkrag.	12,25	90,00	73,40

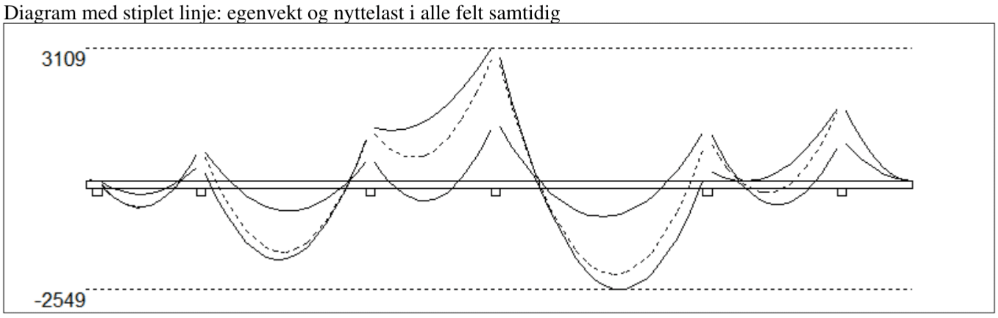
1.4 MATERIALDATA

Korreksjonsfaktor for Emodul pga tilslag	1	Eksponeringsklasse	XC1	XC1
Materialkoeffisient betong	1,5	Lite korrosjonsømfintlig armering		
Materialkoeffisient stål	1,15	Dimensjonerende levetid		50
Betongkvalitet	B45 (C45/55)			
Tilslagets spesifikke tyngde (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	N	**Min. overdekning**	**uk**	**ok**
Armering flytegrense	500	Min krav	15	15
Bøyler flytegrense	500	Toleransekrav +/-	10	10
Relativ fuktighet %	40	Min. nominell overdekning	25	25
Betongens alder ved pålastning (døgn)	28			
Effektiv høyde, h0 (EN 1992-1-1 3.1.4(5))	331			
største tilslagsstørrelse, dg(mm)	22	Kryptall, FI 28_5000		1,65
Korttids Emodul, Ecm	36300	Svinntøyning, FI 0_28		-0,00009
Trykkfasthet, fcd	25,5	Svinntøyning, FI 28_5000		-0,0003
Middel verdi av strekkfasthet, fctm	3,8			
Strekkfasthet, fctd	1,51			

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm   (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

2.1 MOMENTDIAGRAMMER FOR MAKS OG MIN MOMENT I BRUDDGRENSETILSTAND, MED NYTTELAST I UGUNSTIGE FELT

Diagram med stiplet linje: egenvekt og nyttelast i alle felt samtidig



Største negative feltmomenter (strekk i uk)(kNm)

	Bruksgrense		Bruddgrense	
Felt	Mg	Mg+Mp	Mg	Mg+Mp
1	-272	-393	-326	-629
2	-732	-1119	-878	-1846
3	0	-59	0	-477
4	-964	-1516	-1157	-2549
5	-122	-279	-146	-554

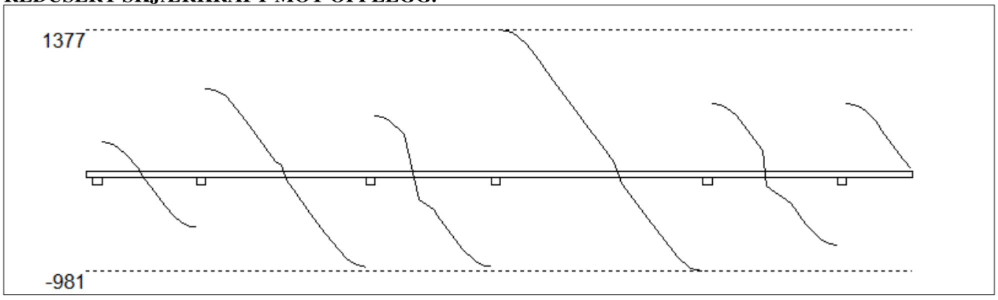
Mg: permanent last   Mp: variabel last

Største positive momenter ved kant av opplegg (kNm)

	Bruksgrense		Bruddgrense	
Opplegg	Mg	Mg+Mp	Mg	Mg+Mp
1	6	9	7	14
2	270	393	324	668
3	479	746	574	1244
4	1243	1890	1492	3109
5	367	618	440	1117
6	715	1043	858	1687

2.2 SKJÆRKRAFTDIAGRAM I BRUDDGRENSETILSTAND

MED NYTTELAST I UGUNSTIGSTE FELT.
REDUSERT SKJÆRKRAFT MOT OPPLEGG.



Største skjærkraft i bruddgrensetilstand (kN)

	Venstre side av opplegg		Høyre side av opplegg	
Opplegg	Vgamma	Vredusert	Vgamma	Vredusert
1	-81	-8	496	284
2	-762	-550	1017	804
3	-1148	-936	744	531
4	-1147	-934	1590	1377
5	-1194	-981	870	657
6	-937	-725	871	658

3.1 ARMERING I FELT OG VED OPPLEGG

Kantavstand er avstand fra senter av armering til underkant eller overkant
Toleranseavvik for overdekning: +/- 10 mm

Feltarmering i underkant og overkant

	Felt	Lag	Kantavstand	Antall	Diameter	Overdekning
uk	1	1	65	3	32	45
uk	2	1	65	5	32	45
uk	2	2	137	2	32	117
uk	3	1	65	2	32	45
uk	4	1	65	5	32	45
uk	4	2	137	5	32	117
uk	5	1	65	2	32	45
ok	1	1	65	2	32	45
ok	2	1	65	2	32	45
ok	3	1	65	5	32	45
ok	3	1	65	2	32	45
ok	4	1	65	3	32	45
ok	5	1	65	2	32	45

Overkantarmering ved opplegg

Opplegg	Lag	Kantavstand	Antall	Diameter	Overdekning
1	1	65	2	32	45
2	1	65	3	32	45
3	1	65	5	32	45
4	1	65	5	32	45
4	2	150	5	32	130
4	3	240	5	32	220
5	1	65	5	32	45
6	1	65	5	32	45
6	2	150	2	32	130

3.2 FORANKRINGSLENGDE OG AVKORTING AV ARMERING

Forutsetning vedr. forankringslengde: Maks. halvparten av armeringen i felt eller over opplegg kan bli avkortet
D: armeringsdiameter
Forankringslengde i underkant: 30 x D   Forankringslengde i overkant: 43 x D
Kapasitetskurvene for moment,(M/Md), kan benyttes til å avkorte armering. Det er tatt hensyn til skjærkraftbidrag
Forskyv endepunktene minst 3xD forbi teoretisk punkt (toleranseavvik)
M/Md for uk viser statisk nøvendig andel av beregnet feltarmering i uk
M/Md for ok viser statisk nøvendig andel av beregnet overkantarmering ved opplegg
M/Md for ok midt i felt kan eventuelt vise nøvendig andel av beregnet feltarmering i ok

3.3 FORANKRINGSARMERING (bøyler) I UNDERKANT VED ENDEOPPLEGG

Det forutsettes at feltarmeringen i underkant av endefelt avsluttes som rette stenger, 50 mm fra bjelkeende
Opplegg nr 1
Det trengs ikke forankringsbøyler.
Opplegg nr 6
Det trengs ikke forankringsbøyler.

3.4 MINIMUMSARMERING (mm2)

Det er regnet med minst 2 stenger inn over opplegg

Felt nr	Uk-venstre opplegg	Uk-høyre opplegg	Underkant i felt	Overkant i felt
1	1608	1608	903	903
2	1608	1608	903	903
3	1608	1753	903	903
4	1608	1608	903	903
5	1608	1608	903	903

3.6 UTSPARINGER

Utsparing nr	Felt nr	Avst. til v. ende i felt(mm)	Avst. til uk bjelke a (mm)	Bredde b (mm)	Høyde h (mm)	Type	Dim. skjærkrefter (kN) Under utsparing	Over utsparing
1	1	2945	200	200	200	sirkel	0,0	158,5
2	2	4425	200	200	200	sirkel	0,0	57,5
3	2	5925	200	200	200	sirkel	0,0	374,5
4	3	2795	200	300	300	sirkel	0,0	306,8
5	3	4295	200	200	200	sirkel	0,0	578,3
6	4	5500	200	200	200	sirkel	0,0	344,5
7	4	7000	200	300	300	sirkel	0,0	128,7
8	4	8500	200	200	200	sirkel	0,0	460,8
9	5	3060	200	200	200	sirkel	0,0	193,0
10	5	4560	200	300	300	sirkel	0,0	318,4
11	6	1995	200	200	200	sirkel	99,5	341,6

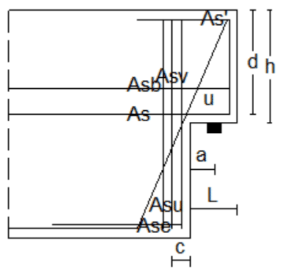
Vinkel mellom trykkdiagonal og bjelkeakse: grader: (økes hvis trykkbruddkapasitet er for liten)

3.7 STATISK NØDVENDIG ARMERING RUNDT UTSPARINGER (vierendelteori)

Utsparing nr	Vert. bøyler pr. side (mm2)	Vert. bøyler over utsp. (mm2/m)	Vert. bøyler under utsp. (mm2/m)	-----------------Horisontalarmering -------------------- Totalt i ok.bjelke (mm2)	ok. utsp. (mm2)	uk. utsp. (mm2)	Totalt i uk.bjelke (mm2)	Trykkbrudd V/Vd under/over
1	177	0	0	903	365	0	1859	0,00 / 0,10
2	62	0	0	903	132	0	5584	0,00 / 0,04
3	407	1968	0	903	862	0	5257	0,00 / 0,23
4	413	2000	0	4736	706	0	1409	0,00 / 0,24
5	645	3038	0	6640	1330	0	1320	0,00 / 0,36
6	366	1810	0	903	793	0	7430	0,00 / 0,21
7	168	0	0	903	296	0	7846	0,00 / 0,10
8	490	2421	0	1060	1060	0	7733	0,00 / 0,29
9	215	1014	0	903	444	0	1639	0,00 / 0,12
10	428	2076	0	1936	733	0	1639	0,00 / 0,24
11	381	1795	0	2022	786	482	903	0,24 / 0,21

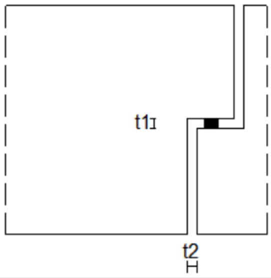
Minimum skjærarmering: Se EN 1992-1-1 NA.9.2.2 (5)   Maks. bøyleavstand: Se EN 1992-1-1 9.2.2 (6)

3.9.1 BJELKENESE. FELT NR 1



h	485	mm
d	448	mm
L	250	mm
a	130	mm
c	97	mm
Ant.skråarmeringsjern	2	
Helningsvinkel, u	62	grader
Horisontalkraft / Oppleggskraft	0,2	

Tittel			Side 7
Prosjekt	Ordre	Sign	Dato 19-05-2017



Fugetykkelse mellom bjelkeneser

t1	10	mm
t2	15	mm

Bjelkenese: Armeringsdata

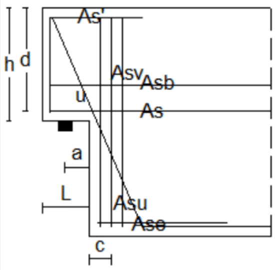
Diameter for skråarmering	16	mm
Diameter for horisontalarmering i uk nese	20	mm
Diameter for horisontale bøyler i uk nese	16	mm

Armering av bjelkenese

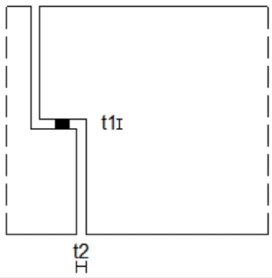
Dim. oppleggskraft:		570,6	kN	Dimensjonerende stålspenning: 387N/mm2
Skråarmeringens andel:		142,0	kN	
Dim. horisontalkraft:		114,1	kN	
Horisontalarmering i uk nese:	As	940	mm2	3d 20, L=1297 mm
Horisontale bøyler i nese:	Asb	322	mm2	1 bøyler d 16, L=1297 i nedre halvdel
Trykkarmering i ok nese:	As'	442	mm2	
Skråarmering:	Asu	402	mm2	2d 16, Forankringslengde i uk =487 mm
Vertikale bøyler ved kant nese:	Asv	1107	mm2	
Forankringsarmering i uk bjelke	Ase	1107	mm2	3 bøyler d 16, L=700 mm avstand til kant: 60 mm

Armeringsutførelse: Hovedstrekkarmering, As, må forankres korrekt.
Se Betongelementboken, Bind C, 7.4.2 og 8.2.2

3.9.2 BJELKENESE. FELT NR 4



h	485	mm
d	444	mm
L	250	mm
a	130	mm
c	120	mm
Ant.skråarmeringsjern	4	
Helningsvinkel, u	62	grader
Horisontalkraft / Oppleggskraft	0,2	



Fugetykkelse mellom bjelkeneser

t1	10	mm
t2	15	mm

Bjelkenese: Armeringsdata

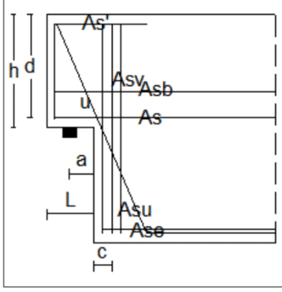
Diameter for skråarmering	16	mm
Diameter for horisontalarmering i uk nese	25	mm
Diameter for horisontale bøyler i uk nese	16	mm

Armering av bjelkenese

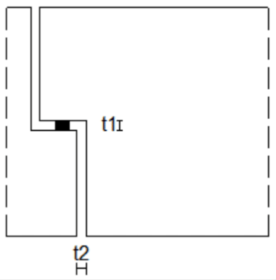
Dim. oppleggskraft:		1118,6	kN	Dimensjonerende stålspenning: 400N/mm2
Skråarmeringens andel:		284,0	kN	
Dim. horisontalkraft:		223,7	kN	
Horisontalarmering i uk nese:	As	1906	mm2	4d 25, L=1540 mm
Horisontale bøyler i nese:	Asb	673	mm2	2 bøyler d 16, L=1540 i nedre halvdel
Trykkarmering i ok nese:	As'	439	mm2	
Skråarmering:	Asu	804	mm2	4d 16, Forankringslengde i uk =510 mm
Vertikale bøyler ved kant nese:	Asv	2086	mm2	
Forankringsarmering i uk bjelke	Ase	2086	mm2	6 bøyler d 16, L=700 mm avstand til kant: 60 mm

Armeringsutførelse: Hovedstrekkarmering, As, må forankres korrekt.
Se Betongelementboken, Bind C, 7.4.2 og 8.2.2

3.9.3 BJELKENESE. FELT NR 5



h	485	mm
d	444	mm
L	250	mm
a	130	mm
c	97	mm
Ant.skråarmeringsjern	2	
Helningsvinkel, u	62	grader
Horisontalkraft / Oppleggskraft	0,2	



Fugetykkelse mellom bjelkeneser

t1	10	mm
t2	15	mm

Bjelkenese: Armeringsdata

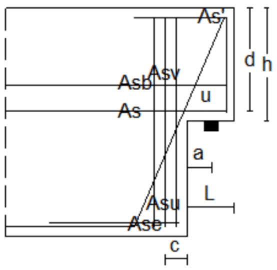
Diameter for skråarmering	16	mm
Diameter for horisontalarmering i uk nese	25	mm
Diameter for horisontale bøyler i uk nese	16	mm

Armering av bjelkenese

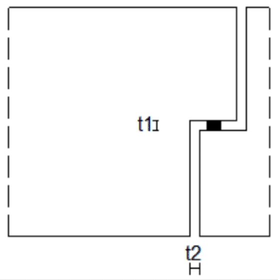
Dim. oppleggskraft:		538,7	kN	Dimensjonerende stålspenning: 309N/mm2
Skråarmeringens andel:		142,0	kN	
Dim. horisontalkraft:		107,7	kN	
Horisontalarmering i uk nese:	As	1103	mm2	3d 25, L=1495 mm
Horisontale bøyler i nese:	Asb	377	mm2	1 bøyler d 16, L=1495 i nedre halvdel
Trykkarmering i ok nese:	As'	439	mm2	
Skråarmering:	Asu	402	mm2	2d 16, Forankringslengde i uk =487 mm
Vertikale bøyler ved kant nese:	Asv	1284	mm2	
Forankringsarmering i uk bjelke	Ase	1284	mm2	4 bøyler d 16, L=1044 mm avstand til kant: 60 mm

Armeringsutførelse: Hovedstrekkarmering, As, må forankres korrekt.
Se Betongelementboken, Bind C, 7.4.2 og 8.2.2

3.9.4 BJELKENESE. FELT NR 2



h	485	mm
d	444	mm
L	250	mm
a	130	mm
c	120	mm
Ant.skråarmeringsjern	4	
Helningsvinkel, u	62	grader
Horisontalkraft / Oppleggskraft	0,2	



Fugetykkelse mellom bjelkeneser

t1	10	mm
t2	15	mm

Bjelkenese: Armeringsdata

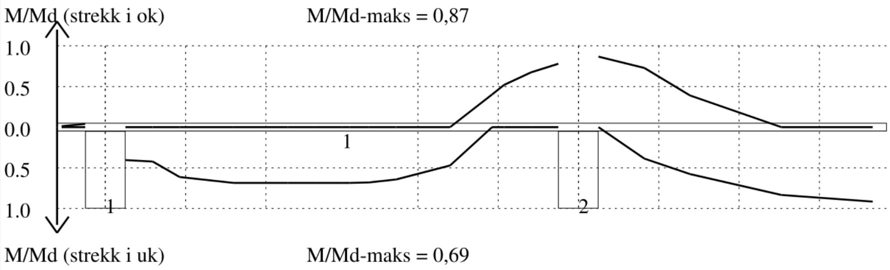
Diameter for skråarmering	16	mm
Diameter for horisontalarmering i uk nese	25	mm
Diameter for horisontale bøyler i uk nese	16	mm

Armering av bjelkenese

Dim. oppleggskraft:		956,5	kN	Dimensjonerende stålspenning: 400N/mm2
Skråarmeringens andel:		284,0	kN	
Dim. horisontalkraft:		191,3	kN	
Horisontalarmering i uk nese:	As	1563	mm2	4d 25, L=1540 mm
Horisontale bøyler i nese:	Asb	543	mm2	2 bøyler d 16, L=1540 i nedre halvdel
Trykkarmering i ok nese:	As'	439	mm2	
Skråarmering:	Asu	804	mm2	4d 16, Forankringslengde i uk =510 mm
Vertikale bøyler ved kant nese:	Asv	1681	mm2	
Forankringsarmering i uk bjelke	Ase	1681	mm2	5 bøyler d 16, L=700 mm avstand til kant: 60 mm

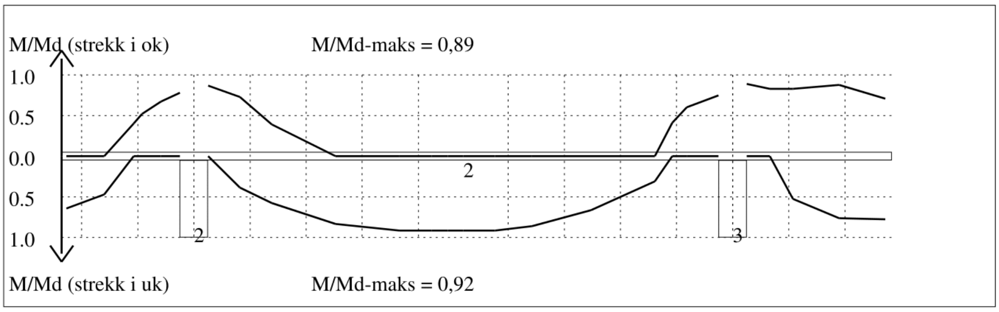
Armeringsutførelse: Hovedstrekkarmering, As, må forankres korrekt.
Se Betongelementboken, Bind C, 7.4.2 og 8.2.2

4.1 MOMENTKONTROLL

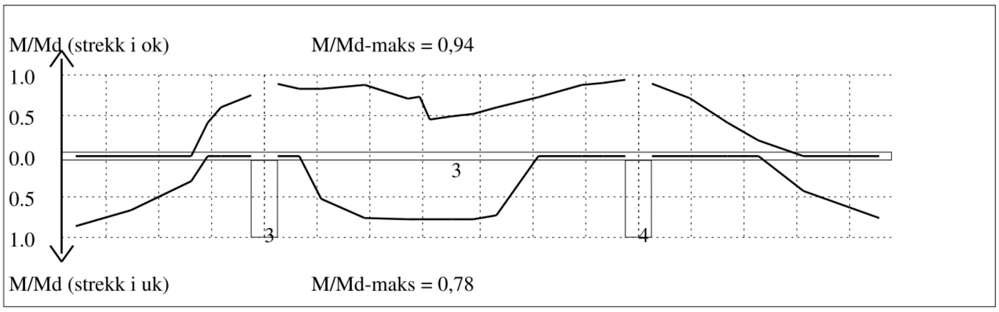


Momentkontroll for felt nr 1 Avstand mellom vertikalstreker = 1.0 m

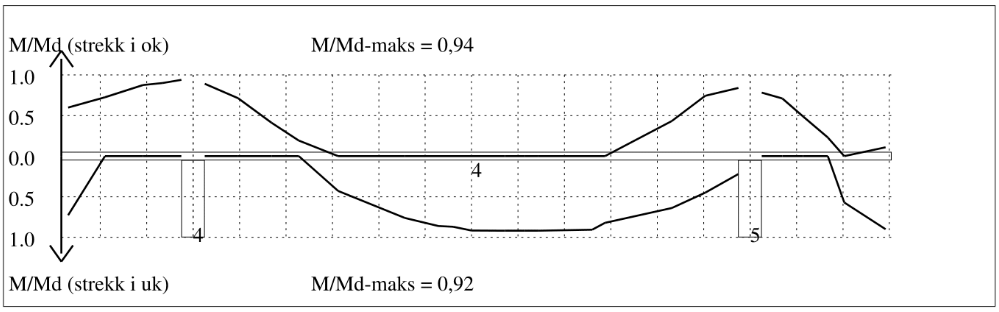
Tittel			Side 11
Prosjekt	Ordre	Sign	Dato 19-05-2017



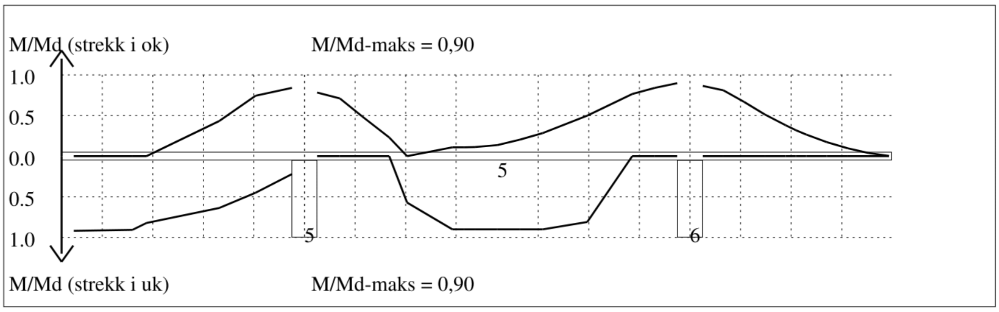
Momentkontroll for felt nr 2 Avstand mellom vertikalstreker = 1.0 m



Momentkontroll for felt nr 3 Avstand mellom vertikalstreker = 1.0 m



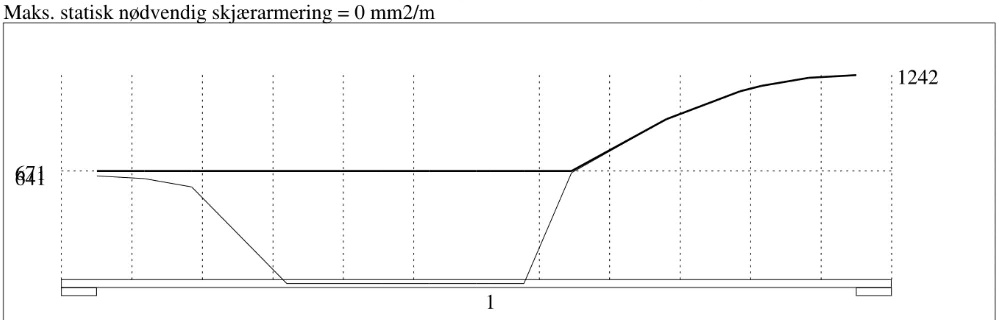
Momentkontroll for felt nr 4 Avstand mellom vertikalstreker = 1.0 m



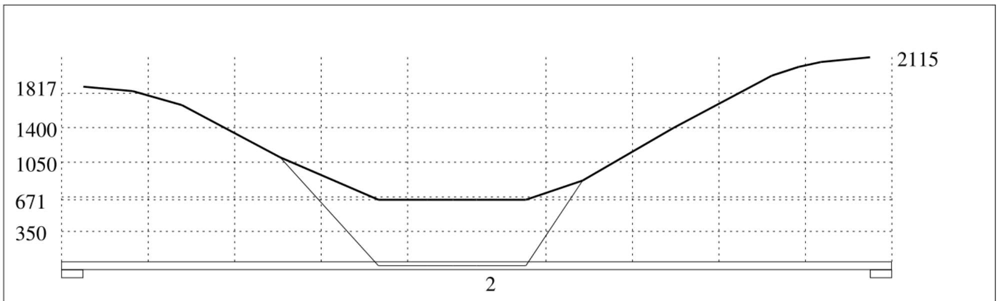
Momentkontroll for felt nr 5 Avstand mellom vertikalstreker = 1.0 m

4.2 SKJÆRARMERING

Skjærarmering i felt nr 0 (minimum skjærarmering) = 671 mm2/m Maks bøyleavstand = 510mm
Maks. statisk nødvendig skjærarmering = 0 mm2/m

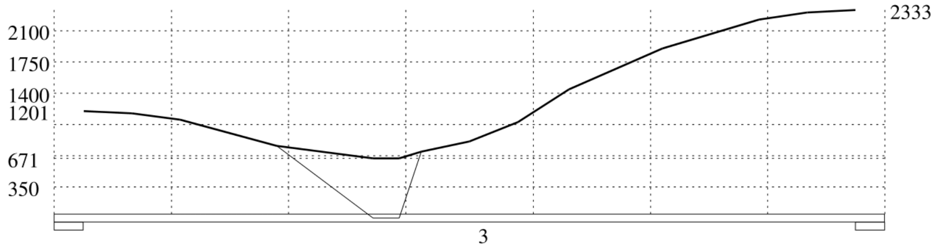


Minimum skjærarmering = 671 mm2/m Maks bøyleavstand = 510mm
Skjærarmering (mm2/m) for felt nr 1 Avstand mellom vertikalstreker = 0.5 m



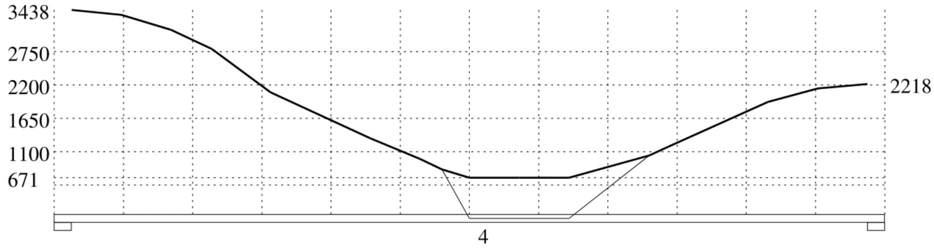
Minimum skjærarmering = 671 mm2/m Maks bøyleavstand = 498mm
Skjærarmering (mm2/m) for felt nr 2 Avstand mellom vertikalstreker = 1.0 m

Tittel			Side 13
Prosjekt	Ordre	Sign	Dato 19-05-2017



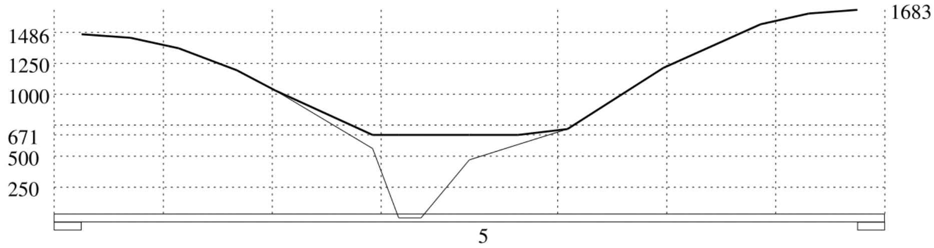
Minimum skjærarmering = 671 mm2/m Maks bøyleavstand = 458mm

Skjærarmering (mm2/m) for felt nr 3 Avstand mellom vertikalstreker = 1.0 m



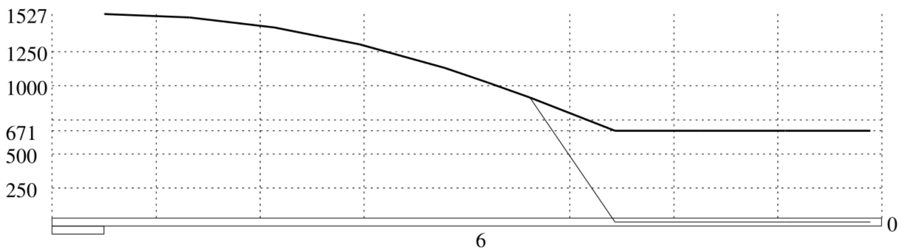
Minimum skjærarmering = 671 mm2/m Maks bøyleavstand = 458mm

Skjærarmering (mm2/m) for felt nr 4 Avstand mellom vertikalstreker = 1.0 m



Minimum skjærarmering = 671 mm2/m Maks bøyleavstand = 495mm

Skjærarmering (mm2/m) for felt nr 5 Avstand mellom vertikalstreker = 1.0 m

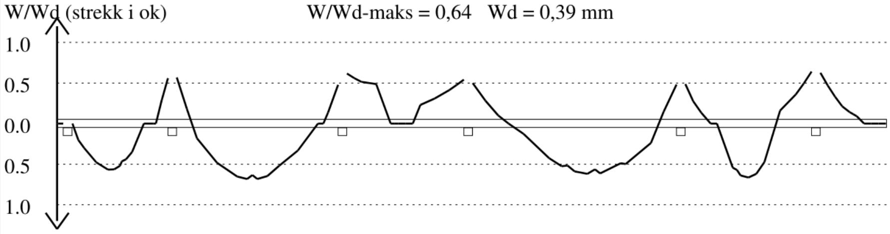


Minimum skjærarmering = 671 mm2/m   Maks bøyleavstand = 495mm

Skjærarmering (mm2/m) for felt nr 6   Avstand mellom vertikalstreker = 0.5 m

4.3 RISSKONTROLL

W/Wd (strekk i ok)   W/Wd-maks = 0,64   Wd = 0,39 mm



W/Wd (strekk i uk)   W/Wd-maks = 0,68   Wd = 0,39 mm

4.4 NEDBØYNINGER I BRUKSGRENSETILSTAND (mm)

	Permanent last		Permanent + variabel last (lang tid)	
Felt	Kort tid	Lang tid	Nyttelast i alle felt	Nyttelast i betraktet felt
V. utkrager	-1	-1	-2	-1
1	2	3	4	4
2	10	16	21	22
3	-3	-6	-8	0
4	16	24	31	34
5	1	1	2	3
H. utkrager	9	16	21	24

5.1 OPPLEGGSKREFTER I BRUKSGRENSETILSTAND (kN og kNm) (alle lastfaktorer = 1)

Ng,Mg: fra egenvekt.   Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-305	0,00	-219	0,00	-47	0,00	-172	0,00
2	-831	0,00	-596	0,00	-291	0,00	-308	0,00
3	-756	0,00	-543	0,00	-456	0,00	-260	0,00
4	-1168	0,00	-838	0,00	-260	0,00	-678	0,00

5	-879	0,00	-631	0,00	-352	0,00	-398	0,00
6	-845	0,00	-607	0,00	-214	0,00	-393	0,00

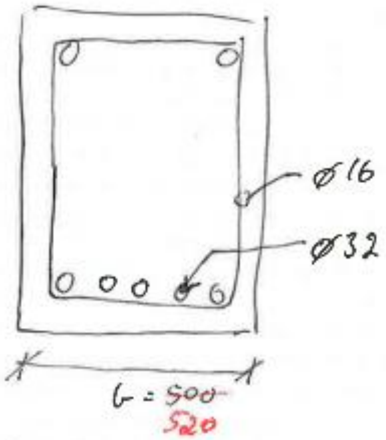
5.2 OPPLEGGSKREFTER I BRUDDGRENSETILSTAND (kN og kNm)

Ng,Mg: fra egenvekt.   Np,Mp: fra nyttelast

					Variabel last i ett felt ved siden av oppleggspunkt			
Oppleggs-punkt	Permanent last i alle felt		Variabel last i alle felt		Variabel last i venstre felt		Variabel last i høyre felt	
	Ng (kN)	Mg (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)	Np (kN)	Mp (kNm)
1	-366	0,00	-328	0,00	-70	0,00	-258	0,00
2	-997	0,00	-894	0,00	-437	0,00	-462	0,00
3	-907	0,00	-814	0,00	-684	0,00	-390	0,00
4	-1401	0,00	-1258	0,00	-390	0,00	-1016	0,00
5	-1055	0,00	-947	0,00	-528	0,00	-597	0,00
6	-1014	0,00	-910	0,00	-320	0,00	-590	0,00

8.2.4. Draft calculations

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
Ktr./ *Chkd*	Dato/ *Date*	Overdekning	Prosj.nr./ *Proj.no*
			Ref.



Overdekning

$c_{min} = \max\{c_{min,b}; c_{min,dur}; 10\}$

Ø32: $c_{min} = \max\{32; 25; 10\} = 32$

Ø16: $c_{min} = \max\{16; 25; 10\} = 25$

$c_{nom} = c_{min} + \Delta c_{dev}$

Ø32: $c_{nom} = 32 + 10 = 42$

Ø16: $c_{nom} = 25 + 10 = 35$ ← dimensjonerende i utk

K-bøyle

$35 + 16 = 51 \quad > 42$

Avstand til kant: $51 + \frac{32}{2} = 67 \quad > 65$

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page* Prosj.nr./ *Proj.no*
Ktr./ *Chkd*	Dato/ *Date*		

Ref.

Avstand mellom lag

$$a_v = \max\{1{,}5\phi \,;\, d_g + 5 \,;\, 20\}$$
$$= 48$$

$$a_{v,max} = 32 < 48$$

$$\rightarrow a_v = 32$$

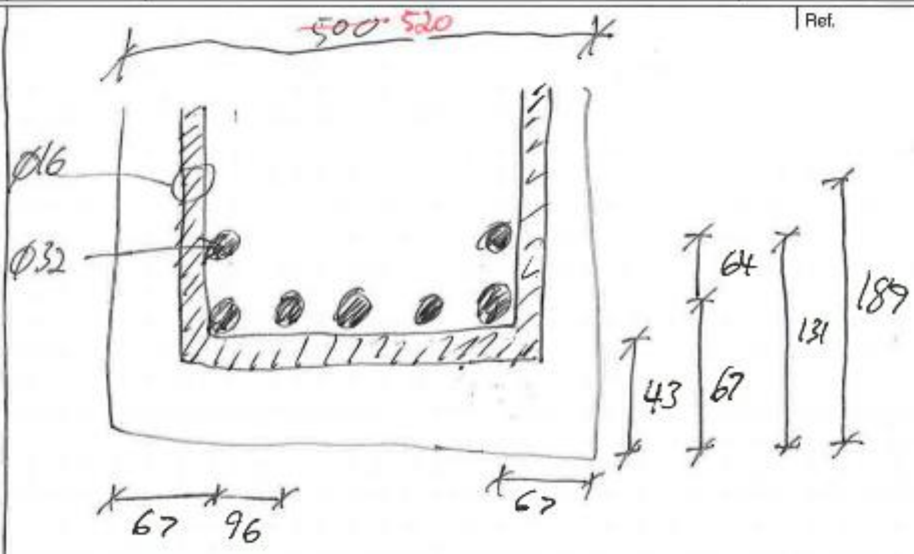
Avstand innad i lag

$$a_h = \max\{2{,}0\phi \,;\, d_g + 5 \,;\, 20\}$$

$$\rightarrow a_h = 64$$

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page* Prosj.nr./ *Proj.no*
Ktr./ *Chkd*	Dato/ *Date*		

Ref.



Maks antall stenger i hvert lag

$$\frac{b - 2\cdot\left(c_{nom,\phi 16} + \phi 16 + \frac{\phi 32}{2}\right)}{a_h + \phi 32} + 1$$

$$= \frac{500 - 2\cdot 67}{64 + 32} + 1 = 4.8125$$

for å ha 5 stenger $\rightarrow$ $\underline{b = 518}$

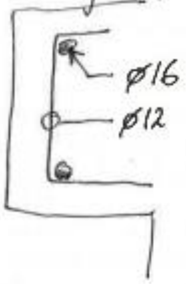
$\rightarrow$ velger 520

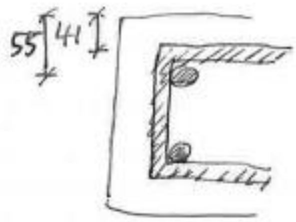
Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
			Prosj.nr./ *Proj.no*
Ktr./ *Chkd*	Dato/ *Date*		

Ref.

Konsoll overdekning

* antar $d_g = 16$ pga SKB





$c_{min} = max\{c_{min,b} ; c_{min,dur} ; 10\}$

ø16: $c_{min} = max\{16 ; 25 ; 10\} = 25$

ø12: $c_{min} = max\{12 ; 25 ; 10\} = 25$

$c_{nom} = c_{min} + \Delta c_{dev}$

$= 25 + 10 = 35$

$35 + 12 = 47$

Avstand til kant: $47 + \frac{16}{2} = 55$

Avstand mellom lag:

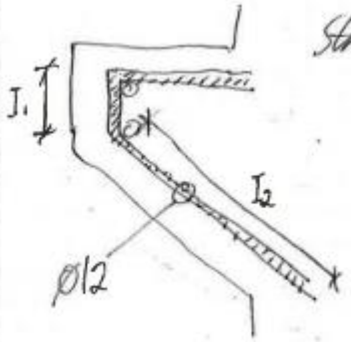
$a_v = max\{1{,}5\,ø ; \underbrace{d_g}_{16^* (SKB)} + 5 ; 20\} = 24$

Minste høyde konsoll: $2 \cdot 55 + 24 + 16 = \underline{150}$

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
			Prosj.nr./ *Proj.no*
Ktr./ *Chkd*	Dato/ *Date*		

Ref.

EC2



Strekk forankring av bøyle

$I_1 = 91{,}8\,mm$

$I_{bd} = \overbrace{I_1 + I_2}$ — Konservativt

3.1.6 $\quad f_{ctd} = \dfrac{\alpha_{ct}}{\gamma_c} f_{ctk\,0,05} = \dfrac{0.85}{1.5} \cdot 2.7 = 1.53\ MPa$ — 845

8.4.2 $\quad f_{bd} = 2.25\ \eta_1 \eta_2\ f_{ctd}$

$= 2.25 \cdot 1.0 \cdot 1.0 \cdot 1.53 = 3.4425\ MPa$

$f_{yd} = 434.7$

8.4.3 $\quad \sigma_{sd} = \dfrac{F}{A_{sh}} = \cancel{\dfrac{112 \cdot 10^3}{214}} = \cancel{523{,}4\ MPa}$

$l_{b,rqd} = \dfrac{\varnothing}{4} \cdot \dfrac{\sigma_{sd}}{f_{bd}} = \dfrac{12}{4} \cdot \dfrac{\overset{434.7}{\cancel{523.4}}}{3.4425} = \overset{379}{\cancel{456}}\ mm$

8.4.4 (2)

Forenklet

$I_{bd} = \alpha_1 I_{b,rqd} = 1.0 \cdot \overset{379}{\cancel{456}} = \overset{379}{\cancel{456}}\ mm$

$\uparrow c_d = 35 \not> 3\varnothing \rightarrow \alpha_1 = 1.0$

velger: $\dfrac{1700 - 738}{2} = 91.8 + 389.2 = \underline{481\ mm}$ — konservativt

$\Rightarrow$ 1700 mm bøyle

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			Prosj.nr./ *Proj.no*
Ktr./ *Chkd*	Dato/ *Date*		

Ref.

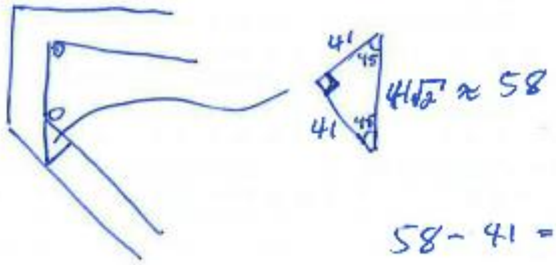
EC2

Tabell NA.8.1 Nc

Dordiameter

12 mm → 32 mm dordiameter

16 mm → 50mm dordiameter

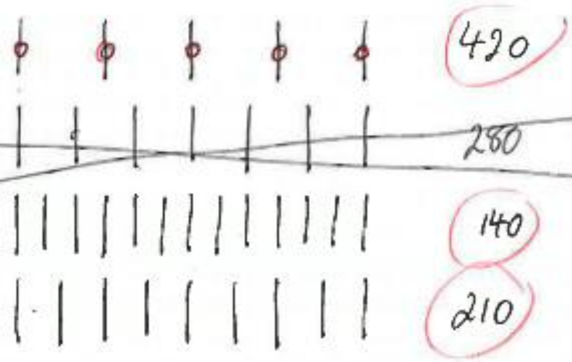


$58 - 41 = 17$

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
Ktr./ *Chkd*	Dato/ *Date*	Bøylefordeling	Proj.nr./ *Proj.no*

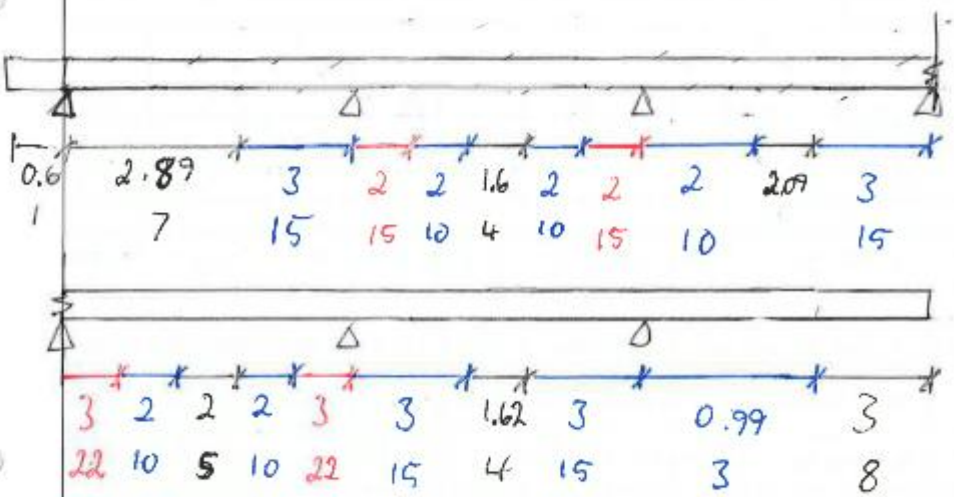
Ref.

s	140	~~280~~	420 $>500, 494$	210
dobbel $A_{sv,ø16}$	2871 >2687		957 >698	1914
A_{sh}			269 >257	



$= 140 \cdot 1.5$

c420 c210 c140



Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
Ktr./ *Chkd*	Dato/ *Date*	Cracked section (Cr) at SLS	Prosj.nr./ *Proj.no*

Ref.

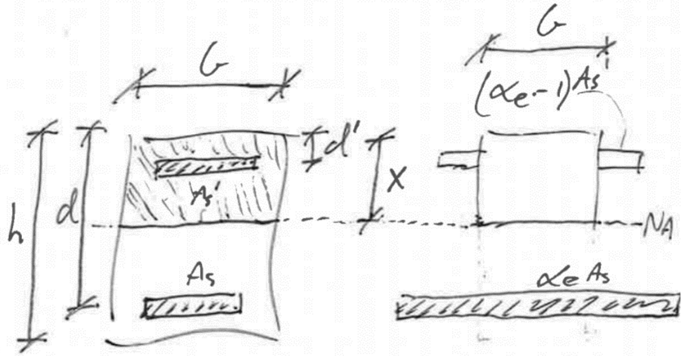
$$(\alpha_e - 1) A_s' (x - d') + (b x) \frac{x}{2} = \alpha_e A_s (d - x)$$

$$(\alpha_e - 1) A_s' x - (\alpha_e - 1) A_s' d' + \frac{b}{2} x^2 = \alpha_e A_s d - \alpha_e A_s x$$

$$\frac{b}{2} x^2 + (\alpha_e - 1) A_s' x + \alpha_e A_s x - (\alpha_e - 1) A_s' d' - \alpha_e A_s d = 0$$

$$\underbrace{\frac{b}{2}}_{a} x^2 + \underbrace{\left[(\alpha_e - 1) A_s' + \alpha_e A_s\right]}_{b} x + \underbrace{\left(-\left[(\alpha_e - 1) A_s' d' + \alpha_e A_s d\right]\right)}_{c} = 0$$

$$\frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$



Cracked section (Cr) at SLS

$$\frac{1}{r_{cr}} = \frac{M_{Ed, SLS}}{E_{c,eff} \cdot I_{cr}}$$

$$\left[I_{cr} = \frac{1}{12} b x^3 + (b x) \left(\frac{x}{2}\right)^2 + \alpha_e A_s (d - x)^2 + (\alpha_e - 1) A_s' (x - d')^2 \right]$$

$$\alpha_e = \frac{E_s}{E_{c,eff}} \qquad E_{c,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)}$$

Sign.	Dato/ *Date*	Prosjekt/ *Project*	Side/ *Page*
Ktr./ *Chkd*	Dato/ *Date*		Prosj.nr./ *Proj.no*

Ref.

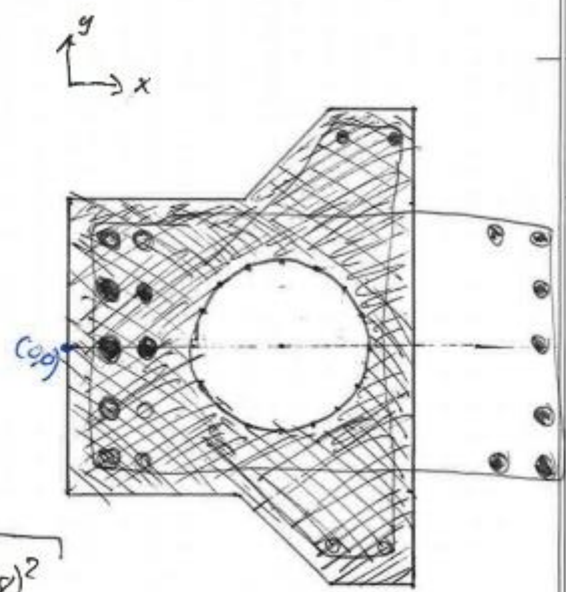
$$\boxed{\begin{aligned} y^2 + (x-368)^2 &= 150^2 \\ y &= \pm\sqrt{150^2-(x-368)^2} \end{aligned}}$$

$y(300) = 218$

$y = a + x$

$218 = a + 300$

$a = -82$



$0 \le x \le 218: \quad \frac{y}{2} = 260$

$218 \le x \le 300: \quad \frac{y}{2} = 260 - \sqrt{150^2-(x-368)^2}$

$300 \le x \le 450: \quad \frac{y}{2} = -82 + x - \sqrt{150^2-(x-368)^2}$

$450 \le x \le 518: \quad \frac{y}{2} = 410 - \sqrt{150^2-(x-368)^2}$

$518 \le x \le 600: \quad \frac{y}{2} = 410$

$$\int_0^{600} \frac{y}{2}\,dx = \overbrace{65400 + 13487.5}^{=28887.5} + 22455 + 21865 + 33620 = 156827.5 = 78413.75 \cdot 2$$

8.2.5. Neutral axis

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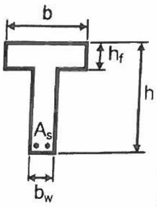
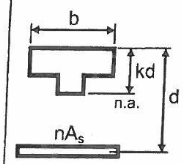
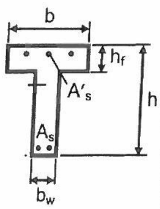
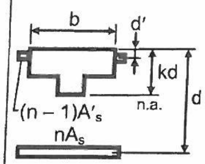
Moment of Inertia of Cracked Section Transformed to Concrete, I_{cr} (continued)

Gross Section	Cracked Transformed Section	Cracked Moment of Inertia, I_{cr}
		$I_{cr} = \frac{(b-b_w)h_f^3}{12} + \frac{b_w(kd)^3}{3} + (b-b_w)h_f\left(kd - \frac{h_f}{2}\right)^2 + nA_s(d-kd)^2$ where $kd = \frac{\sqrt{C(2d + h_f f) + (1+f)^2} - (1+f)}{C}$
		$I_{cr} = \frac{(b-b_w)h_f^3}{12} + \frac{b_w(kd)^3}{3} + (b-b_w)h_f\left(kd - \frac{h_f}{2}\right)^2 + nA_s(d-kd)^2 + (n-1)A'_s(kd-d')^2$ where $kd = \frac{\sqrt{C(2d + h_f f + 2rd') + (1+r+f)^2} - (1+r+f)}{C}$

$n = E_s / E_c$ $= \alpha_e$ ($n = \alpha_e$) → effective modular ratio

$C = b_w / (nA_s)$

$f = h_f (b - b_w) / (nA_s)$

$r = (n-1)A'_s / (nA_s)$

$E_c = E_{c,eff}$ or E_{middle}.

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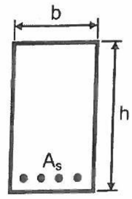
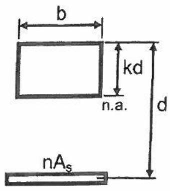
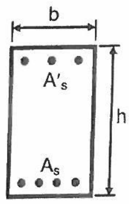
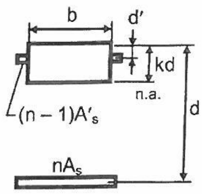
DESIGN AID J.1-14

Moment of Inertia of Cracked Section Transformed to Concrete, I_{cr}

Gross Section	Cracked Transformed Section	Cracked Moment of Inertia, I_{cr}
		$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d-kd)^2$ where $kd = \frac{\sqrt{2dB+1}-1}{B}$
		$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d-kd)^2 + (n-1)A'_s(kd-d')^2$ where $kd = \frac{\sqrt{2dB\left(1+\frac{rd'}{d}\right)+(1+r)^2}-(1+r)}{B}$

---continued next page---

$n = E_s / E_c = \alpha_e$ $\quad$ $(n = \alpha_e) \rightarrow$ (Effective modular ratio)

$B = b/(nA_s)$

$r = (n-1)A'_s/(nA_s)$

$E_c = E_{c,eff}$ or E_{middle}

American Concrete Institute®

Design Aid created by members of ACI Committee 314

Doubly reinforced rectangular section section

Input parameters		**Output parameters**		**Cracked section**	
Long d	32	Abar	804.2477	a	250
Asw	16	Abar	201.0619	b	171716.858
h	980	A	490000	c	-106591404
b	500				
				NA y-	394.34
As1 NO	5	d	880	stress bloc	315.47
As2 NO	5	As	8042.477		
As'1 NO	5	d'	64	Under stress block	
As'2 NO	0	As'	4021.239		
				Un-cracked section	
Layer1	64	Cavity top	466		
Layer2	136			e	29.55
				NA y-	519.55
Es	200	Ec,eff	13.7292	stress bloc	415.64
Ecm	36.3	alpha e	14.56749		
Creep c	1.644			Under stress block	

8.3. ATENA help files

8.3.1. Positions

Felt nr	Spennvidde		Spennvidde kummulativ		Posisjon momentledd (ref midt på hylla)	
v.utkr.	600	0.6	600	0.6		
1	5890	5.89	6490	6.49	11180	11.18
2	9600	9.6	16090	16.09	24490	24.49
3	7090	7.09	23180	23.18		
4	12000	12	35180	35.18	37580	37.58
5	7620	7.62	42800	42.8	44600	44.6
h.utkr.	3990	3.99	46790	46.79		

Spennvidde kummulativ (fra første opplegg)
5.89
15.49
22.58
34.58
42.2

80		5025			Bøyler til innsetting i ATENA		
90	4467		Spenn		Posisjon siste		Antall stenger inkl siste
100				4020			
110	3655	420	0.6		0.6	2	698
120	3350	420	2.89		3.49	7	1609
130	3092	210	3		6.49	15	2120
140	2871	140	2		8.49	15	1946
150	2680	210	2		10.49	10	1356
160	2513	1.6		12.09		4	1867
170	2365	2		14.09		10	2687
180	2233	2		16.09		15	2637
190	2116	2		18.09		10	1826
200	2010	2.09		20.18		5	1582
210	1914	3		23.18		15	1490
220	1827	3		26.18		22	698
230	1748		2		28.18		10
240	1675		2		30.18		5
250	1608		2		32.18		10
260	1546	3		35.18		22	Hylle
270	1489	3		38.18		15	112
280	1436		1.62		39.8		4
290	1386		3		42.8		15
300	1340		0.99		43.79		5
310	1297		3		46.79		8
320				1256			
330				1218			
340				1182			
350				1149			
360				1117			
370				1086			
380				1058			
390				1031			
400				1005			
410				980			
420				957			
430				935			
440				914			

Posisjoner lengdearmering x-retning

U.k.	Fra	Til	Lengde	Fra	Til	Lengde
1 kant 2	0.025		46.765		46.74	
1 midt 3	5.85	16.73	10.88	22.54	28.73	6.19
2 midt 3	25.09		33.44		8.35	
O.k.						
1 kant 2	0.025		46.765		46.74	
1 mellom 2	3.4	8.72	5.32	13.62	46.765	33.145
1 midt 1	3.4	8.72	5.32	19.71	38.77	19.06

Opplegg nr kant 2

v.utkr. 600 1 x x

1 6490 2

2 16090 3 midt 3

3 23180 4 x x x

4 35180 5

5 42800 6 mellom 2

h.utkr. 46790 x x

midt 1

x

Spennvidde	Startposisjon ikke utsparing		Sylinderhøyde
0.6	0		1.75
6.49	5.34		2.3
16.09	14.94		2.3
23.18	22.03		3.4
35.18	34.03		3.4
42.8	41.65		2.3
46.79	45.64		1.15
16.6		30.19	
Volum m		24.8685	
Volum i		27.35343	
% besparelse		9.084528	
d	0.881		Ikke utsparing sylinderhøyde
velger	0.9		2.3
Søylebr.		0.5	

8.3.2. Modelling

3D initial

Geometry original design (remember layers)

Beam

Plane geometry at x=0

0,.26,0

0,.26,.35

0,.41,.35

0,.41,.6

0,.31,.6

0,.26,.6

0,.26,.98

0,-.26,.98

0,-.26,.6

0,-.31,.6

0,-.41,.6

0,-.41,.35

0,-.26,.35

0,-.26,0

0,.26,0

Extrude volume x-dir

46.79

Supports (steel plates)

Line

.35,-.26,0

.35,.26,0

Extrude surface x-direction

.5

Extrude volume z-direction

-.15

Edit bottom surface in two (v sense)

Copy volume (incl.lower entities) in x-direction

5.89

15.49

22.58

34.58

42.2

Longitudinal reinforcement

Interval/layer 1

.025,-.193,.067

.025,.193,.067

.025,-.193,.913

.025,.193,.913

Corbel

.025,-.355,.545

.025,-.355,.405

.025,.355,.545

.025,.355,.405

Extrude lines x

46.74

Interval/layer 2

5.85,-.096,.067

5.85,0,.067

5.85,.096,.067

Extrude lines x

10.88

Extrude same points x

16.69

Extrude the new points into lines x

13.28

Interval/layer 3

25.09,-.193,.131

25.09,0,.131

25.09,.193,.131

Extrude lines x

8.35

Interval 4

Layer 4

3.4,-.096,.913

3.4,.096,.913

Layer 5

3.4,0,.913

Extrude lines x

5.32

Extrude midend (4) points x

10.22

Extrude new point lines x

33.145

Extrude mid (5) point x

16.31

Extrude new point lines x

19.06

Shear reinforcement (remember layering)

0,.217,.043

0,.217,.937

0,-.217,.937

0,-.217,.043

0,.217,.043

Extrude lines x (+ layering)

Layer 140

8.49

16.09

26.18

35.18

Layer 210

6.49

10.49

14.09

18.09

23.18

28.18

32.18

38.18

42.8

43.79

Layer 420

3.49

12.09

20.18

30.18

39.8

46.79

Layer all, then extrude in number of times (negative x-dir) needed to reach next

Corbel reinforcement (remember layers)

0,.369,.391

0,.369,.559

0,-.369,.559

0,-.369,.391

0,.369,.391

Extrude array in x-direction (111 times)

.420

Materials

B45

ø32 (class A) longitudinal rebar

ø16 (class A) longitudinal corbel rebar + shear rebar

ø12 (class A) corbel rebar

Supports steel

Constraints (all intervals)

Contact for surfaces between beam (m) and supports (s)

Fixed contact for line on mid bottom line of supports

Fixed contact for surfaces on top sides of beam

Constraints (interval 1)

Weight of volume 25 kN/m3

Dead load

Left kN/m2

-462

Right kN/m2

-438

Constraints (interval 2)

Variable load

Left kN/m2

-377

Right kN/m2

-358

Constraints (interval 3)

Variable load

Left kN/m2

-377

Meshing for beam

Mesh

SemiStructured

Set

Structured Direction

Select ONE of the longitudinal lines of beam

Confirm

Set surfaces to Quadrilateral

NO divisions on all rebar to 1

NO divisions on all longitudinal lines (BEAM) to 120

Set volume to Hexahedral

3D modified

Refers to use editing "3D geometry"

Delete Beam volume

Delete surfaces, lines and points which refers to the changed part of the c/s. Don't delete bottom or any of the top surfaces!

Create new lines

0,.26,0

0,.26,.3

0,.41,.45

0,.41,.6

Same for x=46.79 and negative y

Lines in between points in longitudinal direction

New surfaces

New volume

Cavity

Non-cavities

remember layer

Create polygons 8 sides radius .15

Starting

Length

0,0,.368

Copy to x-pos

5.34

14.94

22.03

34.03

41.65

45.64

Extrude volumes in x-dir

1.75

2.3

2.3

2.3

2.3

2.3

1.15

Bottom longitudinal corbel rebar

Move points to

.025,.355,.481

46.765,.355,.481

.025,-.355,.481

46.765,-.355,.481

Corbel rebar 420

0,.094,.192

0,.369,.467

0,.369,.559

0,-.369,.559

0,-.369,.467

0,-.094,.192

Extrude array in x-direction (111 times) .420

Material: ø12

Mesh: 1 division

Remaining

Constraints for volume: Weigth

Material for volume: B45

Mesh

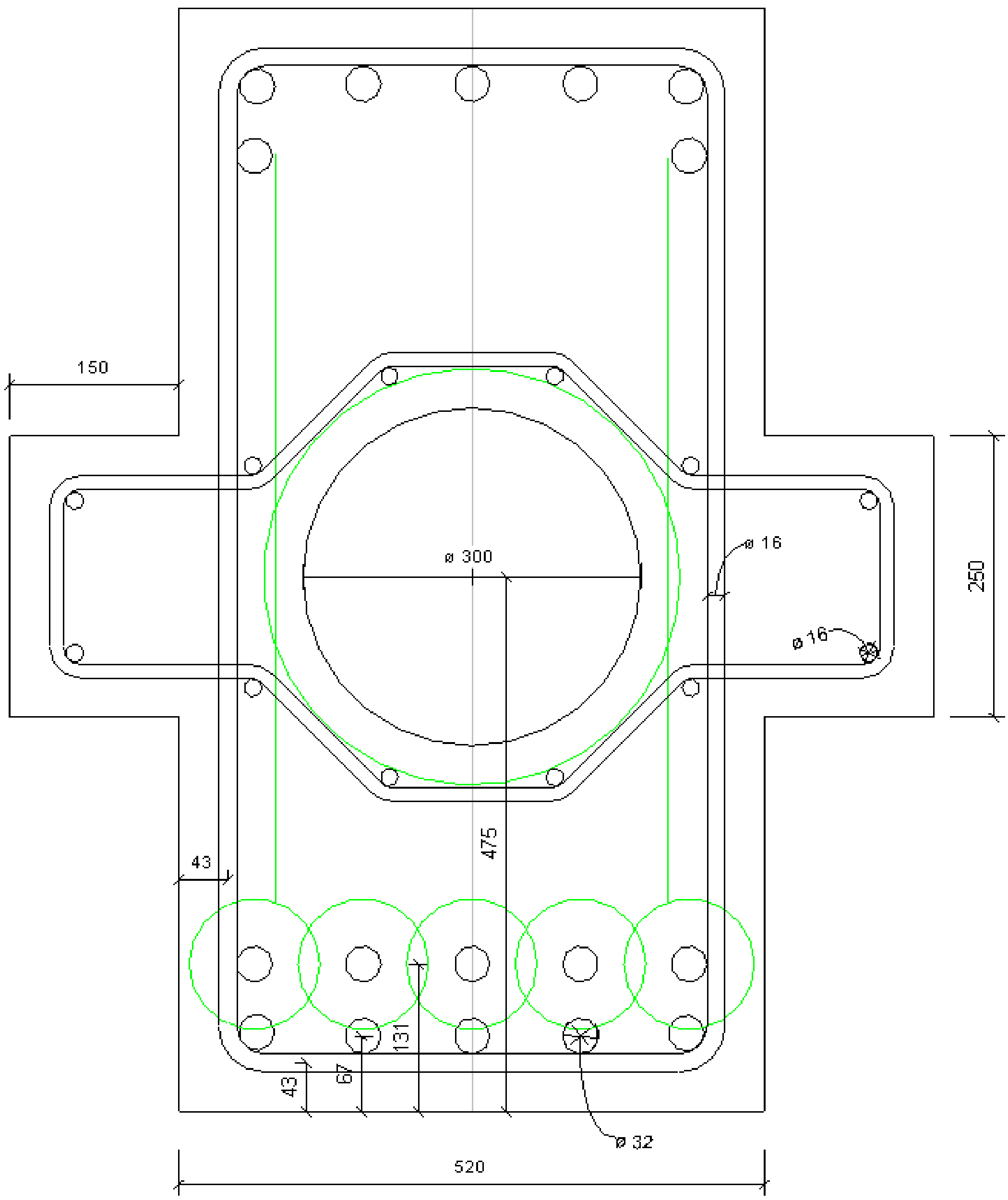
Possibly change sizes

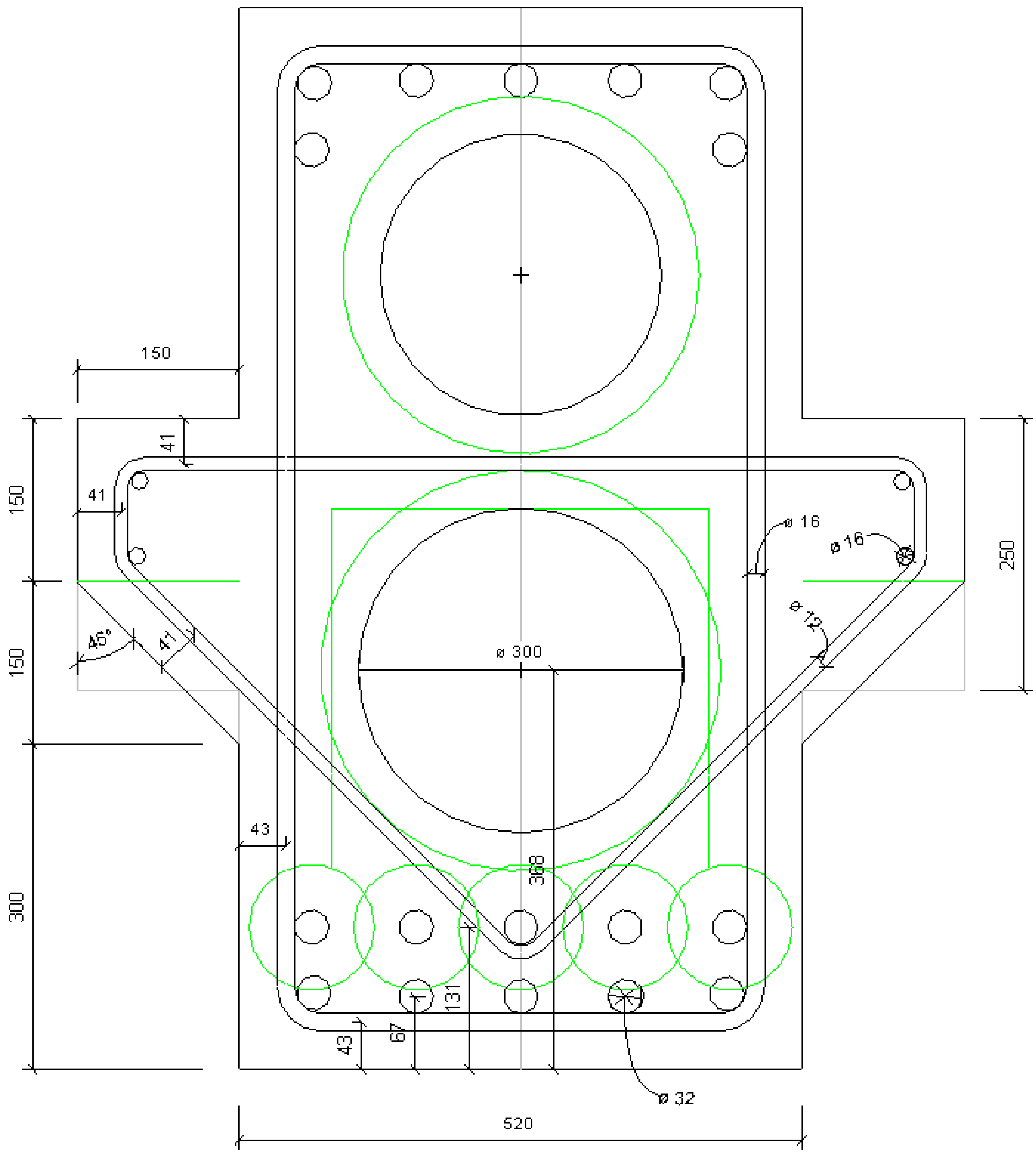
NO divisions for new longitudinal lines: 120

Quadrilateral elements for surfaces

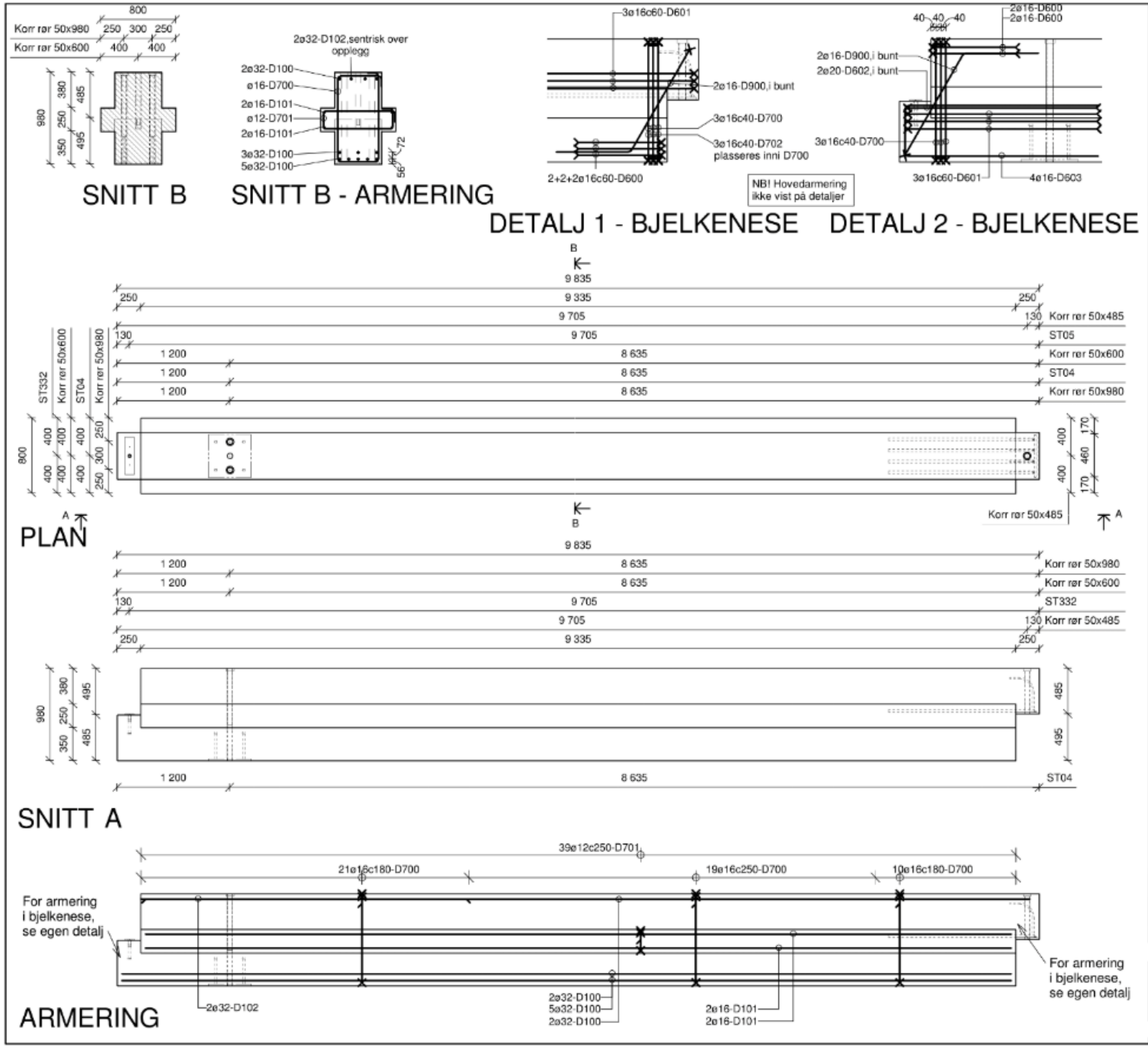
Hexahedral for volume

8.4. Preliminary ideas


150
ø 300
ø 16
250
ø 16
475
43
131
67
43
ø 32
520


150
41
150
41
45°
41
150
250
ø 16
ø 16
ø 12
ø 300
300
43
368
131
67
43
ø 32
520

8.5. Miscellaneous



Deleliste

	Ant.	ELI-Kode
7,5 T Galv FS	2	
1988 M24x200	1	
Korr rør 50x485	1	
Korr rør 50x600	1	
Korr rør 50x980	2	
ST04	1	
ST05	1	
ST06	1	

ANVISNINGER

Produsent plasserer løftepunkter selv

Kanter avfases: 10 mm

ELI-Kode

Overflate: Glatt Grå

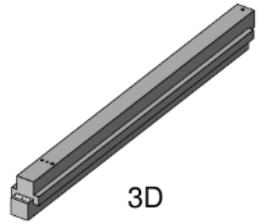
Betongkvalitet: B45 - M45-Res 5

Armeringskvalitet: B500NC Kamstål

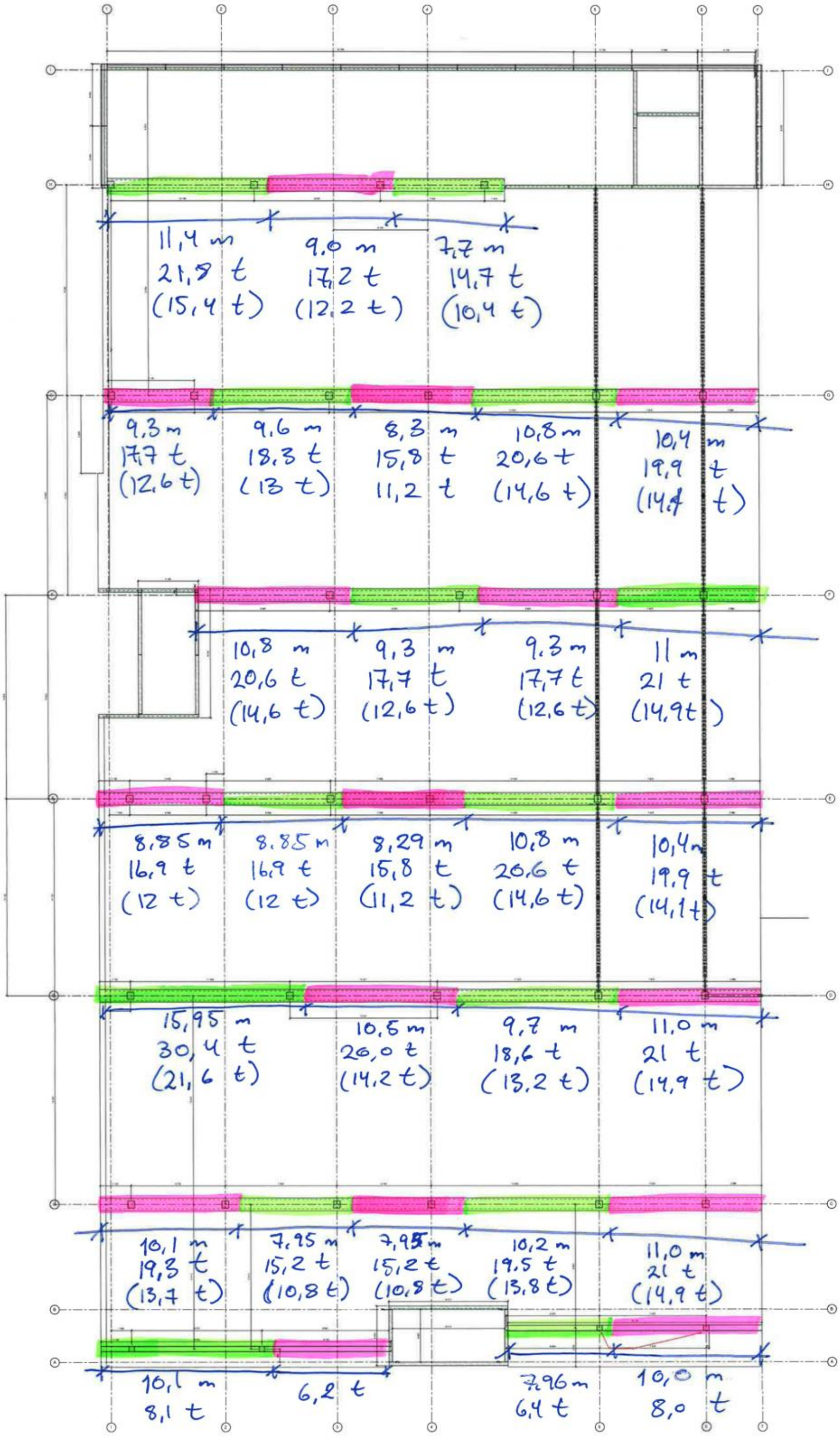
Overdekning: 25 ± 5mm

Bøyeliste: Se tilhørende bøyeliste

Produkt-ID	Antall	Teoretisk Vekt
	1	13,53 t



3D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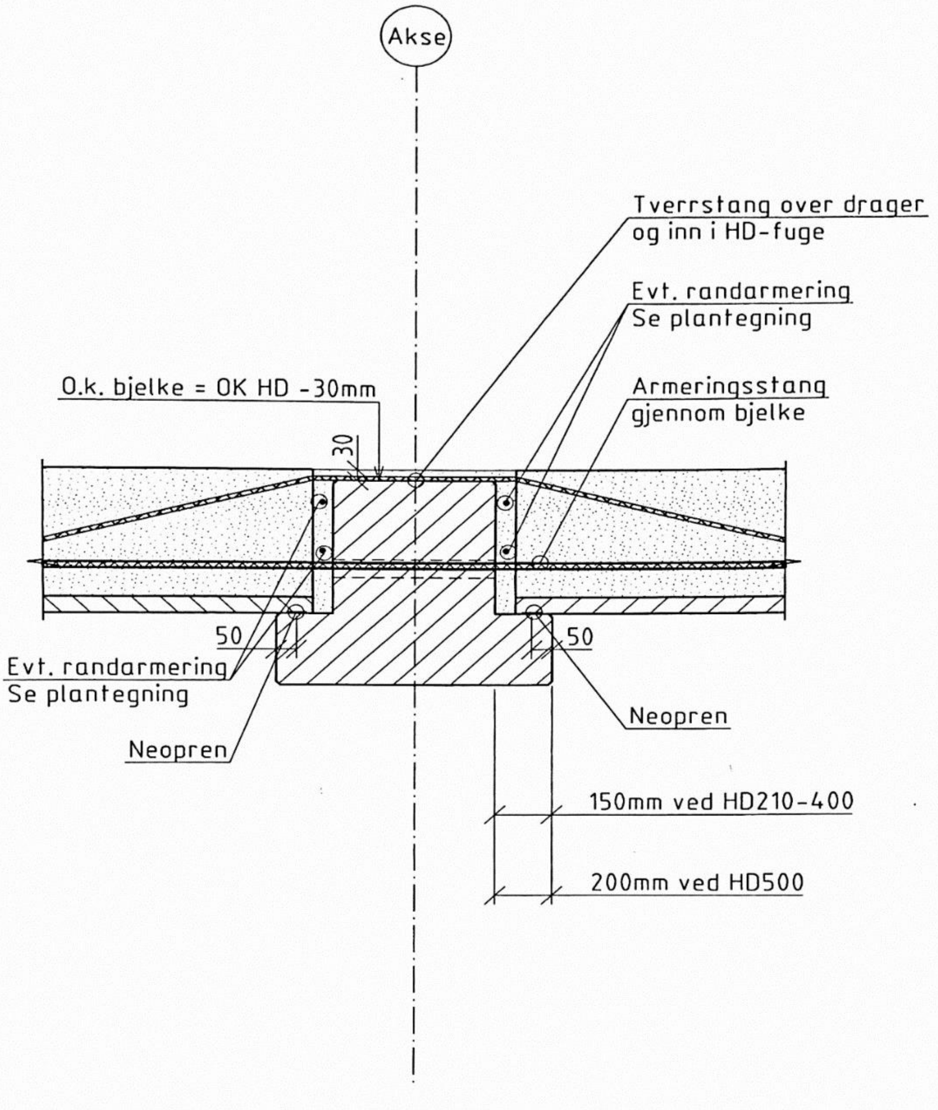
21,8 t
(15,4 t)
9,0 m
17,2 t
(12,2 t)
7,7 m
14,7 t
(10,4 t)
9,3 m
17,7 t
(12,6 t)
9,6 m
18,3 t
(13 t)
8,3 m
15,8 t
11,2 t
10,8 m
20,6 t
(14,6 t)
10,4 m
19,9 t
(14,1 t)
10,8 m
20,6 t
(14,6 t)
9,3 m
17,7 t
(12,6 t)
9,3 m
17,7 t
(12,6 t)
11 m
21 t
(14,9 t)
8,85 m
16,9 t
(12 t)
8,85 m
16,9 t
(12 t)
8,29 m
15,8 t
(11,2 t)
10,8 m
20,6 t
(14,6 t)
10,4 m
19,9 t
(14,1 t)
15,95 m
30,4 t
(21,6 t)
10,5 m
20,0 t
(14,2 t)
9,7 m
18,6 t
(13,2 t)
11,0 m
21 t
(14,9 t)
10,1 m
19,3 t
(13,7 t)
7,95 m
15,2 t
(10,8 t)
7,95 m
15,2 t
(10,8 t)
10,2 m
19,5 t
(13,8 t)
11,0 m
21 t
(14,9 t)
10,1 m
8,1 t
6,2 t
7,96 m
6,4 t
10,0 m
8,0 t



Rev.	Beskrivelse	Sign.	Dato
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–	–	–	–
–	–	–	–
–	–	–	–

STANDARD MONTASJEDETALJ

DLB/HD

OPPLEGG AV HD PÅ DLB MED UBALANSERT SPENN

Konstr.

Kontr.

Godkj.

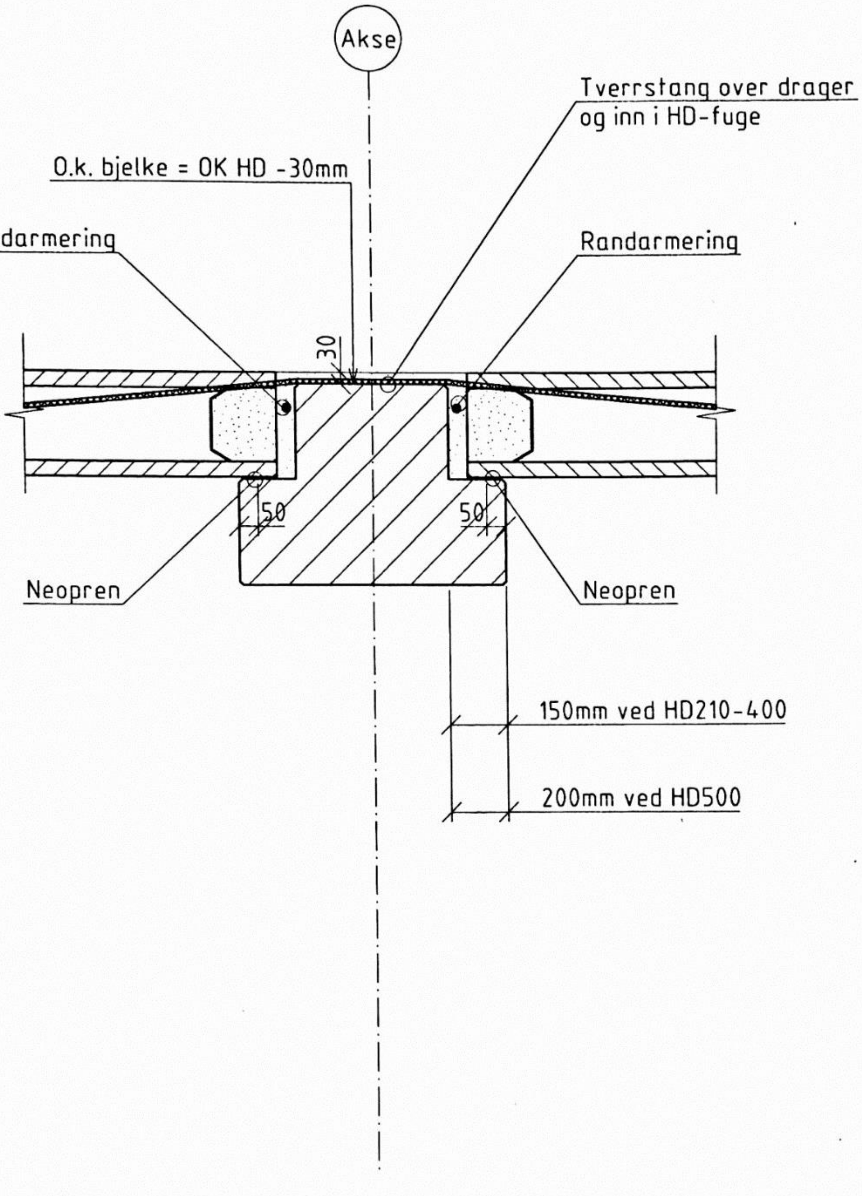
Dato 20.09.08

Målestokk 1:15

Prosj. nr.

Tegn. nr.

Rev. 0



Rev.	Beskrivelse	Sign.	Dato
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-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

STANDARD MONTASJEDETALJ

DLB/HD

OPPLEGG AV HD PÅ DLB MED BALANSERT SPENN

Konstr.	
Kontr.	
Godkj.	
Dato	20.09.08
Målestokk	1:15
Prosj. nr.	
Tegn. nr.	
Rev.	0