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REPORT

THIN BALCONY SLABS
STRUCTURAL DESIGN REPORT

REPORT

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THIN BALCONY SLABS

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STRUCTURAL DESIGN REPORT

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Revision	Date	Reason for issue	Made by	Cont. by	App. by
5	26.10.22	Addition regarding placement of load between supports	MEN	OHHK	OHHK
4	04.07.22	Evaluation of thin balcony slabs with skirts, increased capacity of cantilever beams	OHHK	MEN	MEN
3	04.07.2018	Dimensioning of alternative railing support	OHHK	OBJ/LARN	OEL
2	10.01.2017	Introduction of railing support	OHHK	JSP	MEN
1	21.12.2016	For client review	OHHK	JSP	MEN

CONTENT

1 INTRODUCTION

Invisible connections (iC) delivers a steel connection for pre-fabricated cantilevered balconies. The capacity of this steel connection has been provided by iC.

iC has requested that the balcony concrete slabs are designed as thin as possible, with the aim to reduce the dead weight and increase the size of the balconies.

Dr.techn.Olav Olsen AS shall investigate solutions with reinforcement nets and also investigate the possibilities for using fiber reinforcement with an unreinforced cover to avoid protruding fibers.

Dr.techn.Olav Olsen AS shall, additionally, investigate the balcony railing fastening and come up with a railing fastening arrangement.

In revision 3 of the report structural calculations for an alternative railing fastening arrangement are carried out. The anchorage length in the original railing fastening arrangement is altered.

In revision 4 of the report balcony decks with skirts are evaluated. The balcony slabs have skirts on the three edges not facing the building. Additionally, the maximum bending moment and maximum shear force for the cantilever beams are increased. In revision 5 we have investigated the effect of concentrating the payload only between the supports.

2 REVISION HISTORY

Rev. 01: This is the first issue of the report.

Rev. 02: The weight of the balcony slabs coating is reduced to zero, and the concrete part of the railing fastening arrangement is dimensioned.

Rev. 03: An alternative railing fastening arrangement is dimensioned, and an anchorage length in the original railing fastening arrangement is altered.

Rev. 04: Balcony decks with skirts on three edges are evaluated considering dimensioning and load carrying capacity. Maximum bending moment and maximum shear for the cantilever beams are increased.

Rev. 05: Load effect of payload concentrated between supports is calculated and the tables are updated accordingly.

3 SUMMARY

OO has investigated several types of balcony slab designs. Fiber reinforcement was ruled out at an early stage, and traditional reinforcing mesh are used instead.

The chosen reinforcing mesh must be available as standard commercial products and be well suited for industrial production. A too thin reinforcing mesh will require considerable supporting.

A balcony slab supporting arrangement with two cantilever beams is investigated. The positioning of the cantilever beams is carried out in a way that produce equal values of the field moment and support moment.

A reinforcement solution using one reinforcement net only has been found possible. The reinforcement net must be localized in such a manner that the load carrying top layer is in the middle of the plate. With this arrangement, the reinforcement net will be slightly eccentrically placed.

The chosen reinforcement net has the same dimensions for all concrete slabs (i.e., balcony widths). The chosen reinforcing mesh is Φ8mm with mask width 100 mm x 100 mm. The same mesh is used for all balcony widths because types of commercially available reinforcing mesh are limited, and using the same mesh is less complicated for industrial production. The balcony slabs also need to have sufficient capacity to resist concrete crushing and avoid unacceptable crack widths.

If one wishes to use other mesh widths or thread diameters, an evaluation of alternative reinforcing mesh can be carried out at a later stage.

The total plate thickness becomes 88.8 mm. The largest contribution comes from concrete cover demands, 60 mm all together. The plates for shallow balconies (small width) will have the highest utilization because they are the longest and will have highest values for moment and shear forces. Maximum length for different balcony widths is given in

Table 1.

The balcony width in this context is the distance from the outermost part of the fastening plate for the cantilever beams to the outermost edge of the balcony plate. The balcony slab width will be somewhat smaller.

Maximum allowable moment for cantilever bolt connection: 60 kNm

Maximum allowable shear force for cantilever bolt connection: 70 kN

Weight of reinforced concrete plate: 226.3 kg/m^2

Weight of balcony railing: 19.1 kg/m^2

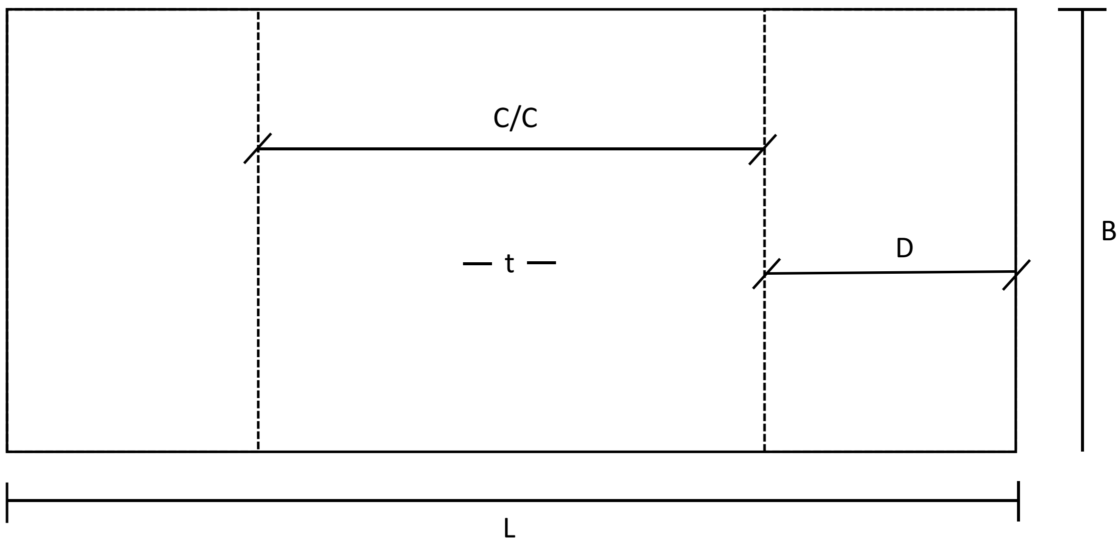
Weight of balcony coating: 0 kg/m^2

General live load: 407.7 kg/m^2

> *Table 1: Maximum balcony length for different balcony widths*

Balcony width B [m]	Maximum balcony length L [m]	Slab thickness t [mm]	Distance between cantilever beams C/C [m]	Distance from edge D [m]
1.500	6.200*	89	3.632	1.284
1.800	6.200*	89	3.632	1.284
2.100	6.122*	89	3.586	1.268
2.400	4.687	89	2.746	0.971
2.700	3.704	89	2.170	0.767
3.000	3.000	89	1.757	0.621
3.400	2.336	89	1.368	0.484
3.800	1.870	89	1.095	0.387

*If a concentration of payload between the supports is relevant the balcony slab will not satisfy Ultimate Limit State (ULS) for these lengths. Maximum length will in that case be approximately 5m.



> *Figure 1: Balcony dimensions*

A solution for balcony railing support is suggested. The distance between the suggested balcony railing support poles is 1000 mm. The balcony railing support is sketched in Figure 2.

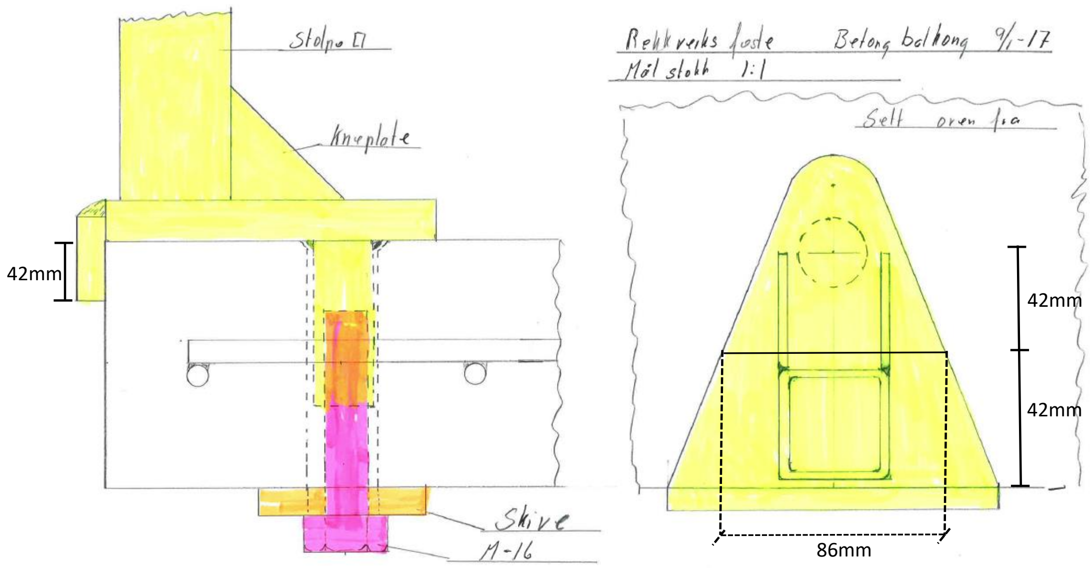
The solution consists of a stiffened steel support plate with triangular shape. Under the support plate a hole with 26 mm diameter is casted. The hole is casted using a plastic sleeve. In the casted hole a screw socket for M16 bolts is placed. The screw socket is fastened to the support plate with full capacity for an M16 bolt.

The M16 bolt is screwed in the screw socket from the underside of the plate. Between the bolt head and the underside of the plate a washer with sufficient thickness and diameter to avoid crushing of the concrete must be used. Considerations regarding bolt head diameter, bolt head thickness, washer diameter and washer thickness have not been carried out.

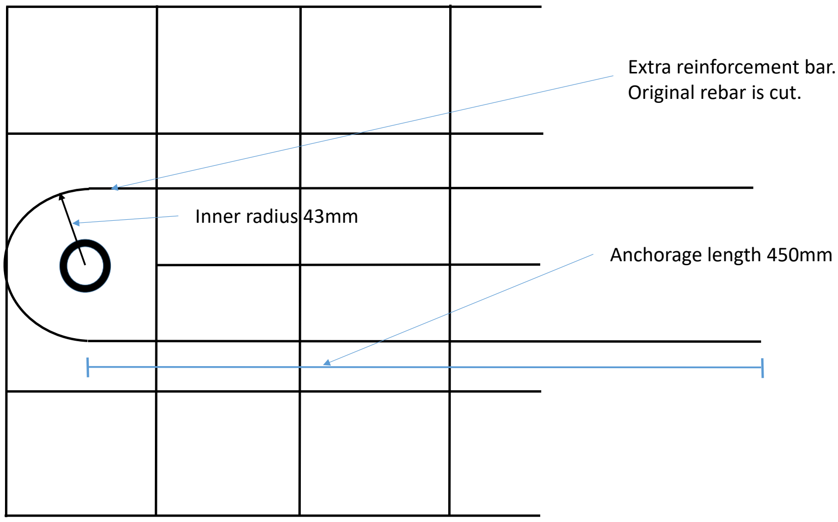
The moment that shall be transferred through the balcony railing support is 1800 Nm, and the shear force is 1500 N (see Chapter 9.3).

The moment is assumed carried as tension in the M16-bolt combined with a compression zone in the concrete under the support plate. The dimensioning of the necessary compression zone area for the concrete is done in Chapter 9.7. The plate thickness is not calculated, but the plate needs to be thick enough to give an even enough concrete pressure not to crush the concrete. The dimensioning of the support plate must be done considering the stiffening arrangement on the plate and the railing post cross section properties.

The shear force is assumed carried through the threaded sleeve for the M16 bolt. An extra rebar with a diameter of 8 mm must be laid around the casted hole (i.e., the threaded sleeve for the M16 bolt) in a semicircle with a radius of 43 mm. This is done in the same layer as the transverse reinforcement. The original rebar in the reinforcement net is cut (see Figure 3).

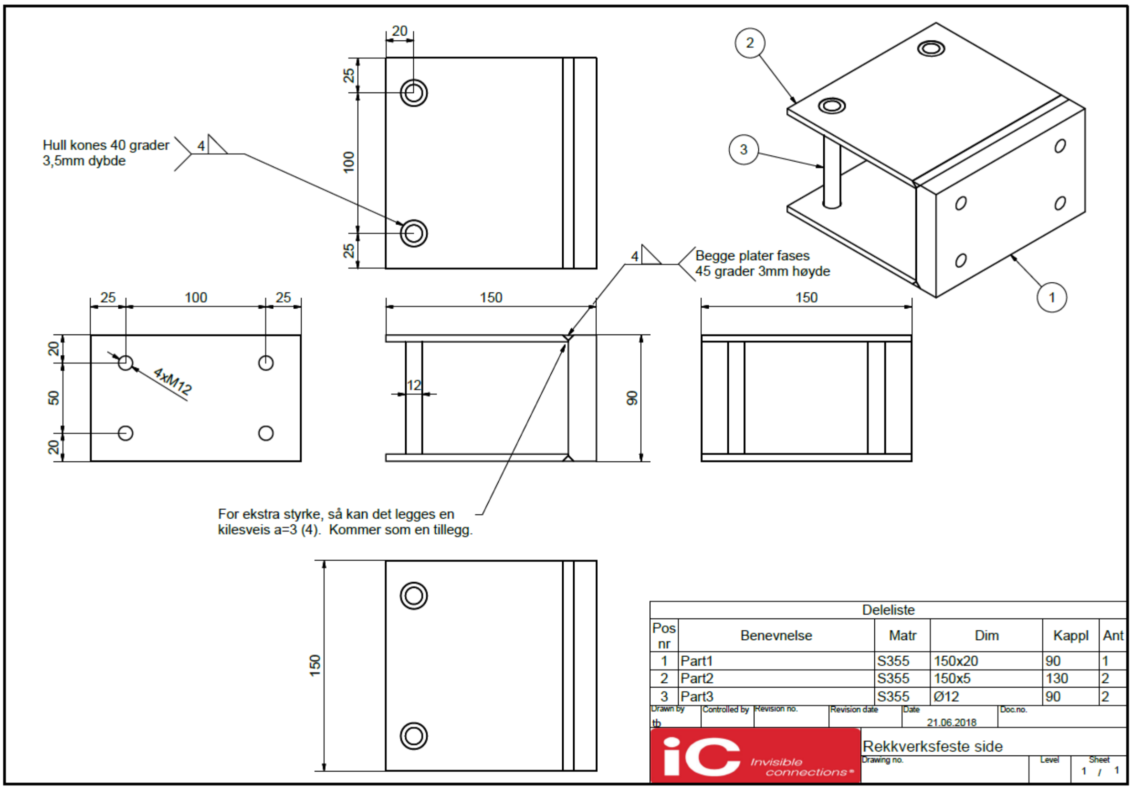


> *Figure 2: Suggested balcony railing support*



> *Figure 3: Placement in the reinforcement net*

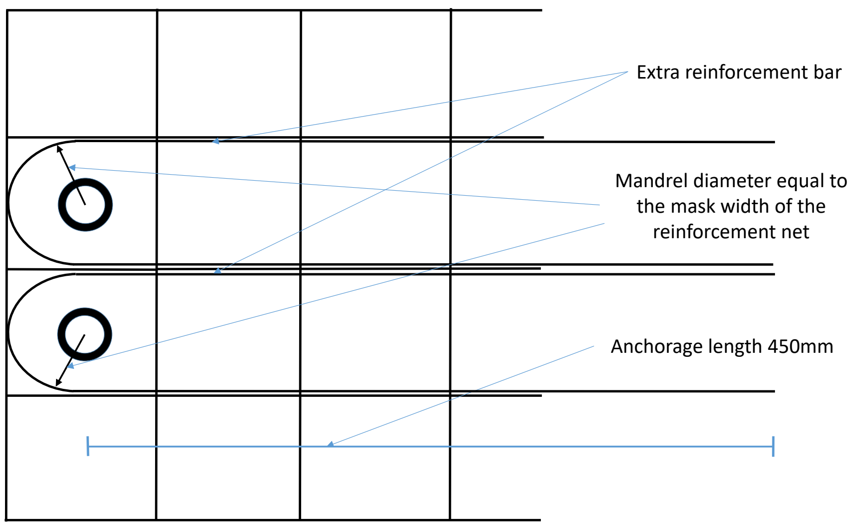
In Revision 3 of the report an alternative balcony railing support structure is dimensioned. The alternative balcony railing support structure is shown in Figure 4.



> *Figure 4: Alternative railing support structure*

The alternative support structure is cast into the concrete plate. It consists of a vertical front plate with four threaded holes and two horizontal plates that are welded to upper and lower edge of the front plate respectively. Between the two horizontal plates runs two reinforcement bars with 12 mm diameter.

The vertical reinforcement bars are placed in the middle of the masks of the reinforcement net (see Figure 5). Extra reinforcement bars with a mandrel diameter equal to the mask with of the reinforcement net are inserted. The anchorage length of the extra reinforcement bars is set to 450 mm. The upper layer (transverse layer) must be placed above the center of the concrete plate, not under.



> *Figure 5:Location of vertical reinforcement bars in the reinforcement net*

This support structure can be used for many types of railing because it is very flexible when it comes to railing design. It is important that the fastening bolts has correct length. The whole front plate thickness must be used as threading support.

By casting the suggested supporting structure into the balcony deck the concrete cover requirements are no longer fulfilled as they are specified by Norwegian Standard (ref. 1). Despite this nonconformity the railing support is evaluated to be very robust with good capacity to transfer the railing forces. It is important to note that the produced units must be checked very thoroughly for good production execution. The casted concrete plates need to be controlled. A procedure to avoid confined air near the plate top must be prepared. An alternative may be to introduce air vent holes in the top plate.

If skirts are introduced on the three plate edges not facing the building, the self-weight of the balcony deck will increase. The cantilever beams will have increased loading. There will also be increased loading on the concrete plate due to higher self-weight.

Maximum length of the balconies (alternatively width) must be somewhat reduced to compensate for the increased loading on the cantilever beams. Quantification of the reduction requirement has not been carried out.

The way the concrete calculations have been carried out (with hand calculations), forces and moments at the most critical locations will not change substantially, and not necessarily in a favorable manner. For the longest balcony plates there might be a need for length reduction. Quantification of the reduction requirement has not been carried out. Short balcony decks are oversized.

4 SUGGESTIONS FOR FURTHER WORK

Maximum balcony length for different balcony widths, slab thickness and amount of reinforcement have been calculated.

A suggestion for railing support design has been established. An M16 bolt with a yield stress of 355 MPa has been chosen for the support design. The width of the triangular support plate is determined, but not its thickness. There are several other steel details for the balcony railing support design that are not decided and dimensioned. All steel dimensioning must be done before the balcony can be built.

The cantilever design is not determined. In the analyses carried out it is assumed that the cantilever beams go as far out as the width of the balcony slab and the support area is continuous and large enough to ensure that the extra local stresses from the cantilever beams are negligible.

Evaluations regarding the support structure will require use of finite element modelling of the concrete plate and support structure.

In the analyses a uniformly distributed vertical loading is assumed. In practice the vertical loading will be somewhat unevenly distributed, and there will be a component in the horizontal direction. Some considerations regarding skew loading and alternative loading directions should be carried out. As a minimum, the support structure should be designed to resist a certain amount of horizontal loading (from wind).

One should also investigate whether the slabs should be designed with pre-camber. The same goes for inclination of the slab to take care of run-off. Both measures are important for drainage of the water in desired direction.

5 MATERIALS, MATERIAL FACTORS AND LOAD FACTORS

5.1 Materials

5.1.1 Concrete

The chosen concrete quality is B35.

5.1.2 Reinforcement Steel

The yield stress for the reinforcement steel is set to 500 MPa.

5.1.3 Construction Steel

The yield stress for the construction steel is set to 355 MPa.

5.2 Material Factors

5.2.1 Concrete

The material factor for the concrete is 1.5 (see Table NA.2.1N in ref. /1/).

5.2.2 Reinforcement Steel

The material factor for the reinforcement steel is 1.15 (see Table NA.2.1N in ref. /1/).

5.2.3 Construction Steel

The material factor for the construction steel is 1.25 (see NA.6.1 in ref /2/).

5.3 Load Factors

Load factor for permanent loads is set to 1.2, and load factor for live loads is set to 1.5. These load factors are given by iC. This is in accordance with combination rule 6.10a+6.10b in NS-EN 1990. In several countries in Europe combination rule 6.10 is used, which gives a load factor 1.35 for self-weight.

6 LOADS

6.1 Permanent Loads

6.1.1 Weight of reinforced Concrete

The weight of the reinforced concrete is set to 25 kN/m^3 (2548 kg/m^3). With a plate thickness of 0.0888m the weight of the concrete plate becomes 25 kN/m^3*0.0888 m = 2.22 kN/m^2 (226.3 kg/m^2).

6.1.2 Weight of Balcony Railing

The weight of the balcony handrail is uncertain, but it is temporarily chosen to be 0.1875 kN/m^2 (19.1 kg/m^2).

6.1.3 Total permanent Loading

The total permanent loading is the sum of the weight of the reinforced concrete and the weight of the handrail: 2.22 kN/m^2 + 0.1875 kN/m^2 =2.4075 kN/m^2 (245.4 kg/m^2).

6.2 Live Loads

6.2.1 General Live Load

The general live load is 4.0 kN/m^2 (407.7 kg/m^2).

6.2.2 Balcony Coating

The weight of the balcony coating is chosen to be 0 kN/m^2 (0 kg/m^2).

6.2.3 Total Live Load

The total live load is the sum of the general live load and the weight of the balcony coating: 4.0 kN/m^2 + 0 kN/m^2 = 4.0 kN/m^2 (407.7 kg/m^2).

6.3 Total Load

6.3.1 Non-factorized total Load

Non-factorized total load is found by adding total permanent loading and summed live loading: 2.4075 kN/m^2 + 4.0 kN/m^2 = 6.4075 kN/m^2 (653.16 kg/m^2).

6.3.2 Factorized total Load

Factorized permanent loads: 2.4075 kN/m^2*1.2 = 2.889 kN/m^2 (294.5 kg/m^2)

Factorized live loads: 4.0 kN/m^2*1.5 = 6.00 kN/m^2 (611.6 kg/m^2)

The factorized total load is found by adding factorized permanent loads and factorized live loads: 2.889 kN/m^2 + 6.00 kN/m^2 = 8.889 kN/m^2 (906.1 kg/m^2).

7 UTILIZATION OF BOLTS

7.1 Capacity of Bolt Connections

The maximum allowable forces in the bolt connection (M24 bolts) are given by iC and are as follows:

Vertical force: 70.0 kN.

Horizontal force: 20 kN (this force is not used in the calculations).

Clamping moment: 60 kNm.

All these forces are assumed to be able to occur simultaneously with their maximum values. This is an essential assumption that sets the basis for the structural dimensioning and should be closely safeguarded. All loads are also assumed to include material factors. Collapse loads will be higher than the given loads.

7.2 Maximum Length of Balconies

The maximum length of the balconies, as a function of the width, where the bolt connections are fully utilized, can now be found. Both maximum allowable moment and maximum allowable vertical force (shear force) must be checked.

From elementary beam theory the following expression can be found:

$$L_{max,moment} = \frac{M_{max}}{0.5 \cdot q \cdot B^2}$$

$$L_{max,shear} = \frac{V_{max}}{q \cdot B}$$

$L_{max,moment}$ = Maximum allowable length where the bolt connection is fully utilized regarding moment

M_{max} = Maximum allowable moment for the cantilever beams, this is the sum of the maximum allowable moment for both beams

q = factorized total load

B = chosen balcony width

$L_{max,shear}$ = Maximum allowable length where the bolt connection is fully utilized regarding vertical force

V_{max} = Maximum allowable shear force for the cantilever beams, this is the sum of the maximum allowable shear force for both beams

> *Table 2: Maximum allowable balcony length, for a given balcony width, where the bolt connections are fully utilized regarding moment*

$M_{max, bolt}$ [kNm]	$M_{max, balcony}$ [kNm]	q [kN/m²]	B [m]	$L_{max, moment}$ [m]
60	120	8.889	1.500	12.000
60	120	8.889	1.800	8.333
60	120	8.889	2.100	6.122
60	120	8.889	2.400	4.687
60	120	8.889	2.700	3.704
60	120	8.889	3.000	3.000
60	120	8.889	3.400	2.336
60	120	8.889	3.800	1.870

> *Table 3: Maximum allowable balcony length, for a given balcony width, where the bolt connections are fully utilized regarding vertical force (shear force)*

$V_{max, bolt}$ [kN]	$V_{max, balcony}$ [kN]	q [kN/m²]	B [m]	$L_{max, shear}$ [m]
70	140	8.889	1.500	10.500
70	140	8.889	1.800	8.750
70	140	8.889	2.100	7.500
70	140	8.889	2.400	6.562
70	140	8.889	2.700	5.833
70	140	8.889	3.000	5.250
70	140	8.889	3.400	4.632
70	140	8.889	3.800	4.145

The shortest balcony length from Table 2 and Table 3 must be chosen. Additionally, the concrete slab must have sufficient capacity. This is checked in Chapter 8.

> *Table 4: Maximum allowable balcony length, for a given balcony width, where the bolt connections are fully utilized*

q [kN/m²]	B [m]	L max, bolt forces [m]	Dimensioning bolt force
8.889	1.500	10.500	Shear force
8.889	1.800	8.333	Moment
8.889	2.100	6.122	Moment
8.889	2.400	4.687	Moment
8.889	2.700	3.704	Moment
8.889	3.000	3.000	Moment
8.889	3.400	2.336	Moment
8.889	3.800	1.870	Moment

8 DIMENSIONING OF REINFORCED CONCRETE SLAB

8.1 General

The concrete slab is dimensioned using standard reinforcing mesh only. This will simplify the production of the balconies.

Typical standard reinforcing mesh from Norwegian suppliers are given in Figure 6 and Figure 7. The assortment is limited. For this reason, the same reinforcing mesh is chosen for all concrete slabs, Φ8mm with mask width 100 mm x 100 mm. The next standard mesh with a mask width of 100 mm x 100 mm has a bar diameter of 4 mm. A bar diameter of 4 mm is regarded problematic during production execution because comprehensive propping is needed to fulfill the strict requirements for production tolerances (se Chapter 8.2).

Armeringsnett B500NA

NS 3576-1

	Format	Tråd	Masker	Vekt		
Type	m	mm	mm	kg	stk/pk	vekt/pk
P091	2,0 x 5,0	3,4	100x100	14,2	70	994
P091S	1,2 x 2,4	4	100x100	4,1	50	205
K131	2,0 x 5,0	5	150x150	21	50	1050
K131S	1,2 x 2,4	5	150x150	5,91	50	295
K189	2,0 x 5,0	6	150x150	30,2	40	1208
K257	2,0 x 5,0	7	150x150	41,1	30	1233
K335	2,0 x 5,0	8	150x150	53,7	20	1074
K402	2,0 x 5,0	8	125x125	66	15	990
K503	2,0 x 5,0	8	100x100	79	15	1185

Tilbake

> *Figure 6: Reinforcing mesh from Bergen Armering AS*

ARMERINGSNETT B 500 NA

Kvalitet: B 500 NA
Etter NS 3576-1

Type	Format m	Tråd mm	Masker mm	Vekt kg	stk/pk	vekt/pk	
P091	2,0 x 5,0	3,4	100x100	14,2	70	994	
P091S	1,2 x 2,4	4	100x100	4,1	50	205	4mm tråd med kam
K131	2,0 x 5,0	5	150x150	21	50	1050	
K131S	1,2 x 2,4	5	150x150	5,91	50	295	
K189	2,0 x 5,0	6	150x150	30,2	40	1208	
K257	2,0 x 5,0	7	150x150	41,1	30	1233	
K335	2,0 x 5,0	8	150x150	53,7	20	1074	
K402	2,0 x 5,0	8	125x125	66	15	990	
K503	2,0 x 5,0	8	100x100	79	15	1185	

Prosjektnett leveres fra vår verksforbindelse. Max bredde 3000 mm og max lengde 12000 mm.
Det meste kan skreddersys etter dine behov.

> *Figure 7: Reinforcing mesh from Stene Stål Produkter AS*

8.2 Requirements for Concrete Cover

In addition to necessary concrete thickness for structural strength, concrete cover requirements must be fulfilled, see ref. /1/, Chapter 4.4.1 and Chapter NA.4.4.1.2. Exposure class XC2, XC3, XC4 is chosen (structures near or on sea must use exposure class XS1, which gives other concrete cover values). Minimum concrete cover thickness becomes 25 mm for a project with a design life of 50 years.

An upper limit for production deviation must be established. This upper limit is set to 5 mm. Total concrete cover requirement becomes 25 mm + 5 mm = 30 mm. With a maximum production deviation of 5 mm the production execution must have a quality assurance system where control procedures include measurements of achieved concrete cover thickness. Accurate measuring techniques shall be used, and all parts not meeting the requirements shall either be rejected or improved. If not, project deviation requirement for concrete cover must be increased to 10 mm.

8.3 Requirement for anchorage of mesh

The reinforcing mesh must be closed using four reinforcement bars with size Φ8mm. The four reinforcement bars are placed along the four edges of the reinforcement nets and welded to all protruding bars.

The welding procedure must follow NS-EN ISO 17660-1. The details in this procedure have not been examined in this report.

8.4 Requirements for structural Strength

8.4.1 Bearing Location

The cantilever beams are positioned in such a manner that the field moment and support moment become equal in magnitude when considering an evenly distributed load:

From elementary beam theory the cantilever beams will have the following location:

Position 1: $\frac{1}{2} \cdot (\sqrt{2} - 1)L = 0.2071L$

Position 2: $\frac{1}{2} \cdot (3 - \sqrt{2})L = 0.7929L$

The distance between the cantilever beams becomes: $L_{eff} = (2 - \sqrt{2})L = 0.5858L$

L is the length of the balcony.

> *Table 5: Support location for different balcony lengths*

L [m]	C/C Distance between cantilever beams [m]	D Distance from edge [m]
10.500	6.151	2.175
8.333	4.881	1.726
6.200	3.632	1.284
6.122	3.586	1.268
4.687	2.746	0.971
3.704	2.170	0.767
3.000	1.757	0.621
2.336	1.368	0.484
1.870	1.095	0.387

8.4.2 Cross-Section Forces in Concrete Plates

The cross-section forces are found from elementary beam theory. We have considered two different load cases. When the load is evenly distributed across the plate the field moment is equal to the support moment.

$$M = M_{field} = M_{support} = \frac{1}{8} \cdot (3 - 2\sqrt{2}) \cdot qL^2 = 0.02145qL^2$$

This is correct for self-weight in any case, but one could consider the payload to only act between the supports. In that case the support moment disappear and the maximum field moment is given by the following equation:

$$M_{q,felt} = \frac{1}{8} q L_{felt}^2 = \frac{1}{8} q \left[\left(2 - \sqrt{2}\right) L \right]^2 = \frac{6 - 4\sqrt{2}}{8} q L^2$$

This can be an overly conservative approach as there normally is no boundaries at a balcony leading to such a distribution of the load. NS-EN 1990 3.2 states that "the selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure". And one could argue that it is not reasonable to assume such a design situation.

However, there could be situations where furniture, or other items, makes it completely reasonable to assume such a load distribution. In that case the contribution to the design moment from the payload will be doubled.

The shear force reaches maximum value at the supports. The shear force is the same independent of the load distribution.

$$V = V_{support} = \frac{1}{2} \cdot \left(2 - \sqrt{2}\right) \cdot qL = 0.2929qL$$

At the supports the cross-section moment also reaches maximum value, and the utilization is highest at the supports. At the midpoint of the balconies the shear force is zero.

> *Table 6: Cross-section forces in the concrete plate for different balcony lengths*

		Evenly distributed load		Concentration of load between supports	
L [m]	**q [kN/m²]**	**M [kNm/m]**	**V [kN/m]**	**M [kNm/m]**	**V [kN/m]**
10.500	8.889	21.017	27.337	35.205	27.337
8.333	8.889	13.238	21.696	22.173	21.696
6.200	8.889	7.328	16.142	12.275	16.142
6.122	8.889	7.146	15.940	11.968	15.940
4.687	8.889	4.189	12.204	7.015	12.204
3.704	8.889	2.615	9.643	4.381	9.643
3.000	8.889	1.716	7.810	2.874	7.810
2.336	8.889	1.040	6.081	1.742	6.081
1.870	8.889	0.666	4.868	1.117	4.868

8.4.3 Crack Control

Crack control is based on ref. /1/, Chapter NA.7.3.1. The limiting crack width values for exposure class XC1, XC2, XC3, XC4 becomes:

$$w_{max} = 0.3mm \cdot \frac{30mm}{25mm} = 0.36mm$$

8.4.4 Strength Documentation

The longest concrete slab will have the highest utilization. If the preferred balcony length of 10.500 m is used, the concrete balcony slab will be over-utilized.

Following the assumption of an evenly distributed load the balcony length must be reduced to 6.200 meter for the concrete plate to have an acceptable utilization. If the payload is concentrated between the supports the maximum length is reduced to about 5m. The shorter balconies will have lower utilization than the longest balcony, and they will have acceptable utilization until the reinforcement net is altered in an unfavorable fashion.

The dimensioning is carried out using ShellDesign. This is a program that carries out non-linear cross section control. The program is developed by Dr.techn.Olav Olsen. The calculation results can be found in Appendix A.

A hand calculation is also carried out. It can be found in Appendix B.

9 DESIGN OF BALCONY RAILING SUPPORT

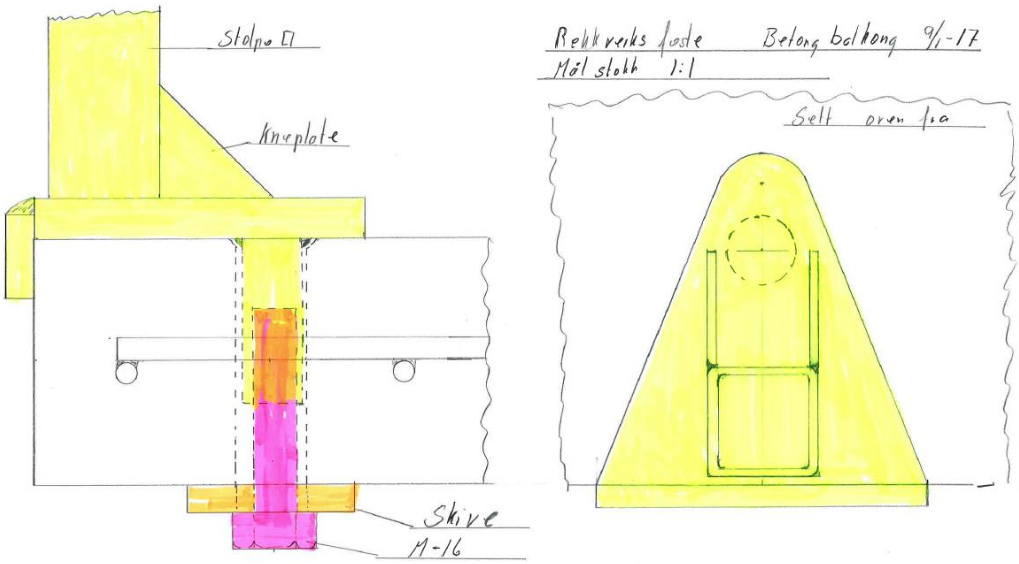
9.1 General

Railing support means the fastening point for the vertical railing poles against the balcony slab.

The vertical railing poles are assumed to be steel poles that are welded to a steel plate. The connection between the railing pole and the steel plate is reinforced with a stiffener arrangement. At the underside of the steel plate there is a screw socket for an M16 bolt. The screw socket is place in a casted through hole in the concrete slab. The steel plate is then bolted to the concrete plate with an M16 bolt that is screw in from the underside.

9.2 Design of fastening Point

The basis for support point design is specified by iC. The information handed over is «veiledning om tekniske krav til byggverk, §12-17. Rekkverk (ref. /3/), Norsk Standard NS-EN 1991-1-1:2002+NA:2008. Eurokode 1 (ref. /4/)» and a handmade sketch.



> *Figure 8: Handmade sketch of fastening point for railing pole*

9.3 Loads on Railing Pole

The horizontal load on the balcony railing is 1KN/m (Kategori A, Table NA.6.12 i ref. /4/). Load factor is chosen equal to the load factor for live loads, 1.5. The distance between the railing poles is 1.0 m.

The factorized horizontal load on the balcony railing pole becomes:

F=1000 N/m*1.0 m*1.5=1500 N.

Pole height is set to h=1.2 m. This is the minimum requirement to railing height if the height above ground level is more than 10.0 m (see ref. /3/).

The moment in the base of the railing pole will be:

M=1500 N*1.2 m=1800 Nm

9.4 Requirement for Railing pole

The balcony railing pole must have a cross section modulus that is large enough not to exceed the dimensioning equivalent von Mises stress. The dimensioning equivalent von Mises stress is equal to the yield stress for construction steel divided by the material factor for construction steel.

σ_{dim} =355 MPa/1.25=284 MPa

The dominating contribution to equivalent von Mises stress comes from bending stresses. For the bending stress not to exceed the maximum allowable equivalent von Mises stress, the cross section modulus must be equal to or greater than:

$W_{min}=M/\sigma_{dim} = 1800\ Nm/284*10^6\ N/m^2=6.338*10^{-6}\ m^3$

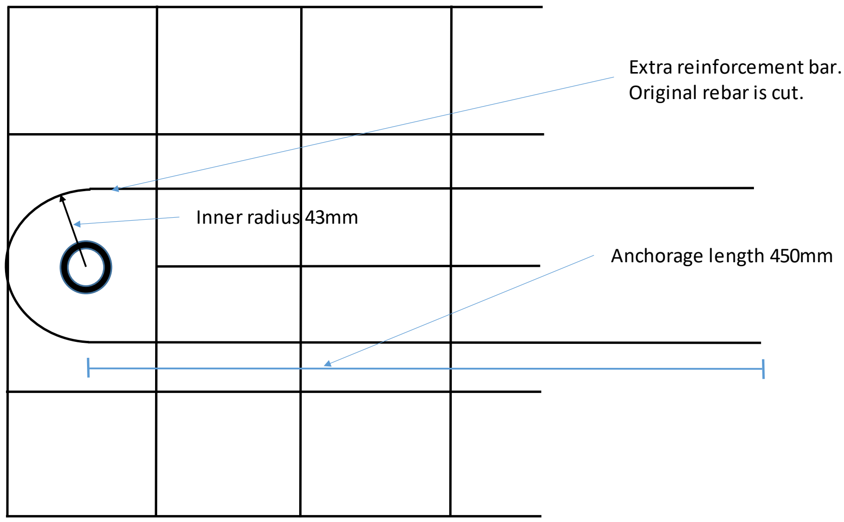
The balcony railing pole most also be able to transfer a shear force of 1500 N.

9.5 Requirements for Screw Socket of M16 bolt

The dimensions for the screw socket for the M16-bolts are not specified, but the screw socket must have a cross section area that is greater than the cross section area for the M16-bolt, and there must sufficient structural capacity to transfer all forces from the M16-bolt and the connection plate. The outer diameter must be small enough for the screw socket to fit withing the fabrication tolerances in a hole with a diameter equal to 26 mm.

9.6 Location of Screw Socket for M16 bolt

The center of the screw socket for the M16-bolt is placed 84 mm from the edge of the balcony. This is the middle point between the two outmost reinforcement bars going in the balcony longitudinal direction. The longitudinal location is the middle point for a transversal reinforcement bar. This reinforcement bar is cut and replaced by a new reinforcement bar with a dimeter of 8 mm (se Figure 9).



> *Figure 9: Location in reinforcement net*

9.7 Dimensioning of Connection Plate

A balanced cross section is chosen. The pressure zone height factor is set to 0.5. The pressure zone height then becomes half the distance between the center of the M16 bolt and the edge of the concrete plate.

x=α*d=0.5*84 mm =42 mm.

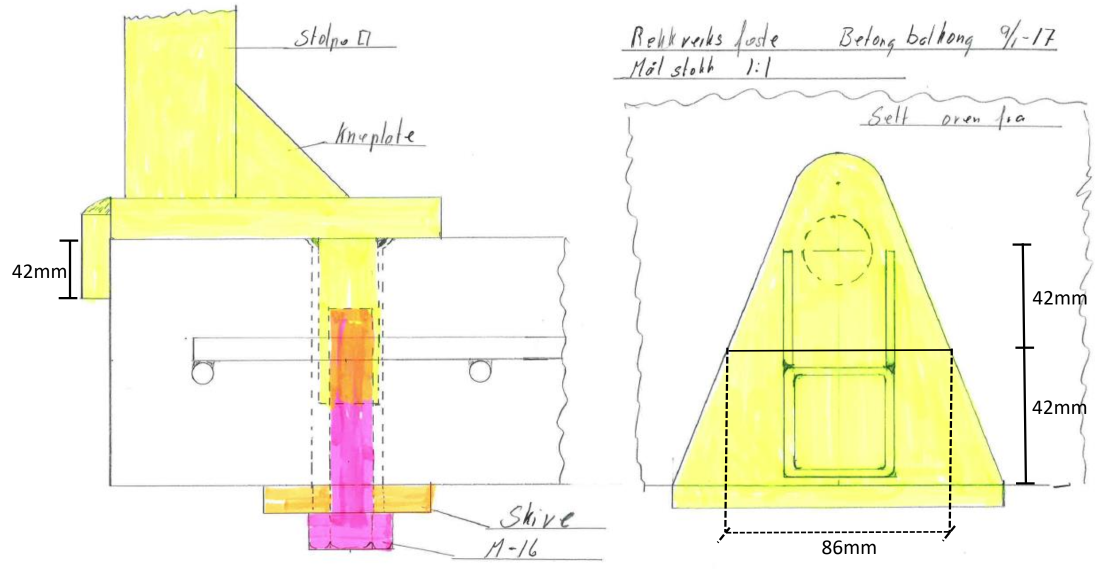
With the given material parameters (see Chapter 5 and ref. /1/) the allowable stresses in the M16 bolt becomes, f_{sd} = 284 MPa. The dimensioning compressive strength for B35 concrete is, f_{cd}=19.8 MPa.

The cross-section area for the M16-bolt is A_s=201.1 mm^2.

To achieve the required balanced cross section the width must be equal to (see ref. /1/, Chapter 3.1.7 for establishment of design parameters):

$$b = \frac{f_{sd} \cdot A_s}{\lambda \cdot \eta \cdot \alpha \cdot f_{cd} \cdot d} = \frac{284 \frac{N}{mm^2} \cdot 201.1 mm^2}{0.8 \cdot 1.0 \cdot 0.5 \cdot 19.8 \frac{N}{mm^2} \cdot 84 mm} = 85.69 mm$$

Based on these calculations a width of 86 mm is chosen.



> *Figure 10: Chosen dimensions for connection plate*

The vertical steel plate at the front of the balcony concrete plate shall ideally have the same height as the pressure zone height, 42 mm.

The moment capacity becomes:

$$M_{Rd} = 0.8\alpha \cdot (1 - 0.4\alpha) f_{cd} b d^2 = 0.8 \cdot 0.5 \cdot (1 - 0.4 \cdot 0.5) \cdot 19.8 \frac{N}{mm^2} \cdot 85.69mm \cdot (84mm)^2 = 3837227Nmm = 3837Nm$$

The given design moment is M=1800 Nm.

The utilization becomes:

$$\eta_{connection} = \frac{M}{M_{Rd}} = \frac{1800Nm}{3837Nm} = 0.469$$

Because this is not an ideal, balanced, reinforced concrete cross section, the utilization should be well below 1.0. A utilization equal 0.469 is evaluated to be acceptable.

Additionally, concrete crushing must be checked. This is done for a fully utilized cross section. Because the pressure zone height factor is equal to 0.5, the steel strain is equal to the concrete strain. The steel strain for a balanced reinforced concrete cross section is equal to the yield strain:

$$\varepsilon_{steel} = \varepsilon_{concrete} = \frac{\sigma_{steel}}{E_{steel}} = \frac{284 \frac{N}{mm^2}}{210000 \frac{N}{mm^2}} = 0.001352 = 1.35 \cdot 10^{-3}$$

This strain is lower than allowable concrete strain, which is $3.5*10^{-3}$ (see Table 3.1 i ref. /1/).

9.8 Check of Reinforcement Stress in extra Reinforcement Bars

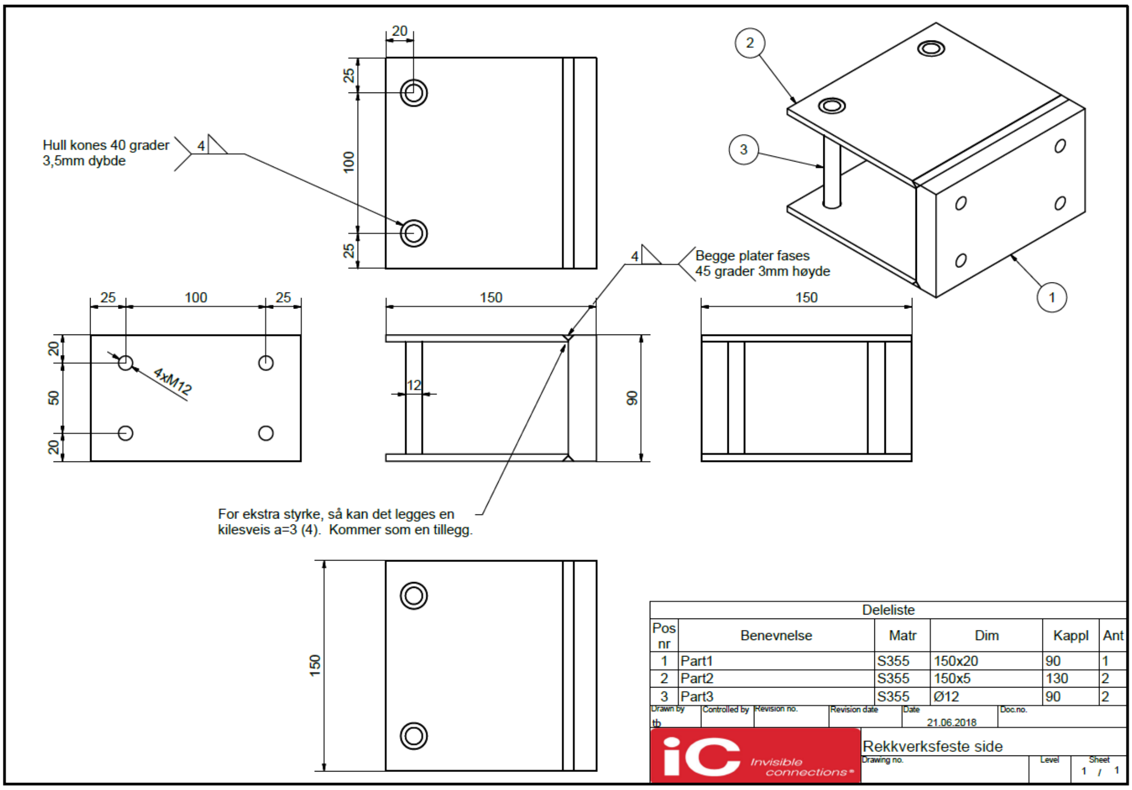
The shear force of 1500 N shall be distributed to two reinforcement bars with 8 mm diameter. Total cross section area is 100.5 mm^2.

The stress in the reinforcement bars becomes 1500 N/100.5 mm^2=14.9 MPa. This is much lower than the allowable stress of 434.7 MPa.

10 DIMENSIONING OF ALTERNATIVE BALCONY RAILING SUPPORT

10.1 General

An alternative to the pole fastening point presented in Chapter 9 is evaluated. A drawing of this railing support connection has been sent to Dr.techn.Olav Olsen (see Figure 11).



> *Figure 11: Alternative railing support*

The alternative railing support is casted into the concrete. It consists of a vertical front plate with four threaded holes and two horizontal plates that are welded to the upper longitudinal edge and lower longitudinal edge of the front plate respectively. Between the horizontal plates runs two reinforcement bars with a diameter of 12 mm.

The railing support design can be used for many types of railing because it is very flexible regarding geometric shapes of railing designs.

10.2 Forces and Stresses in tightened M12 Bolts

The horizontal load on the balcony railing is set to 1500 N (see Chapter 9.3). The moment at the pole base is slightly higher than the moment found in Chapter 9.3 because the pole height increases with half the plate thickness.

H=1.2 m+0.09 m/2=1.245 m

The updated moment becomes:

$M=F*H=1500\ N*1.245\ m=1867.5\ Nm$

This moment is applied by two force couples in the M12-bolts.

The distance between the upper and lower M12-bolts is:

$h_{bolt}=0.05\ m$

This gives the following axial forces in the M12-bolts:

$F_{bolt}=M/(2*h_{bolt})=1867.5\ Nm/(2*0.05\ m)=18675\ N$

The M12-bolts have a cross section area equal to 113.1 mm^2. The axial stresses in the M12-bolt becomes:

$\sigma_{bolt}=F_{bolt}/A_{bolt}=18675\ N/113.1\ mm^2=165.1\ MPa$

A stress value of 165.1 MPa is lower than the dimensioning equivalent stress of 284 MPa (see Chapter 9.4).

If there is a pure vertical force on the balcony railing, which is the type of loading giving the highest shear force, the average shear stress in the four M12-bolt becomes:

$$\tau_{bolt}=\frac{F}{4\cdot A_{bolt}}=\frac{1500N}{4\cdot 113.1mm^2}=3.316N/mm^2$$

This average shear stress with a pure vertical load on the balcony railing is so low that there is no need for stress combination considerations with axial loading before axial stress with a pure horizontal load is closer to the dimensioning axial stress.

10.3 Forces and stresses in the transition Area between the front Plate and the horizontal Plates

The vertical distance between the centers of the horizontal plates is 85 mm. These plates shall transfer the same moment as the M12-bolts, 1867.5 Nm.

The axial force in each horizontal plate becomes:

$F_{plate}=1867.5\ Nm/0.085\ m=21970.6\ N$

The front area of the two plates is:

$A_{front}=150\ mm*5\ mm=750\ mm^2$

Average stress in the two plates becomes:

$\sigma_{plate,average}=F_{plate}/A_{front}=21970.6\ N/750\ mm^2=29.29\ MPa$

Then there is a question regarding how much of the horizontal plates that is activated from the bolt holes to the plate edge. With 45 degrees spreading from the center of each bolt to the plate edge (which is located 20 mm from the bolt center) it becomes:

L_{active}=4*20 mm=80 mm

The axial stresses in the active parts of the horizontal plates becomes:

σ_{plate}= $\sigma_{plate,average}$ (L/L_{active})=29.29 MPa (150 mm/80 mm)=54.93 MPa

The stresses in the plates in the interface area with the vertical reinforcement bars are checked in Chapter 10.5.1. The weld is considered as the weakest point.

10.4 Shear Capacity of vertical Reinforcement Bars

Each reinforcement bar shall transfer half of the axial force in the horizontal plates as shear force.

Q=F_{plate}/2=21970.6 N/2=10985.3 N

The shear capacity for the reinforcement bar and surrounding concrete is checked according to Eurocode 4 (ref. /5/), Chapter 6.6.3.1. The standard does not mention dowels with a thickness less than 16 mm. For this reason the shear capacity of the reinforcement bar and the surrounding concrete is calculated in the same fashion as for a 16 mm reinforcement bar.

The reinforcement bar is checked first:

$$P_{Rd} = \frac{0.8 \cdot f_u \cdot \pi \cdot d^2/4}{\gamma_v} = \frac{0.8 \cdot 500N/mm^2 \cdot \pi \cdot (16mm)^2/4}{1.25} = 64339.8N$$

The reinforcement bar with 16 mm diameter has a shear capacity equal to 64640 N. This is higher than the shear capacity requirement of 10985 N.

Subsequently the concrete is checked:

h_{sc}/d=80 mm/16 mm=5

This gives $\alpha = 1$.

$$P_{Rd} = \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v} = \frac{0.29 \cdot 1 \cdot (16mm)^2 \cdot \sqrt{35N/mm^2 \cdot 34000N/mm^2}}{1.5} = 53990.9N$$

If a 16 mm reinforcement bar is used, the concrete can transfer a shear force of 53991 N to the reinforcement bar. This is higher than the shear capacity requirement which is 10985 N.

If the formulas to be used, had been valid for a 12 mm reinforcement bar, the shear capacity to the reinforcement bar would have been 36191 N. The concrete capacity would have been 30370 N. Both these values are higher than the shear capacity requirement of 10985 N.

It is therefore highly likely that 12 mm vertical reinforcement bars will be sufficient.

10.5 Weld Dimensioning

10.5.1 Weld between vertical Front Plate and horizontal Plates

The weld on the received drawing (se Figure 11) is set as 4 mm fillet weld. This must be changed to 4 mm butt weld.

The same active length as found in Chapter 10.3 is used.

L_{active}=80 mm

The area of the weld becomes:

$$A_{weld} = L_{active} \cdot a = 80mm * 4mm = 320mm^2$$

The weld shall transfer the axial force, F_{plate}=21971 N, found in Chapter 10.3.

Average axial stress in the weld becomes:

$$\sigma_{weld} = \frac{F_{plate}}{A_{weld}} = \frac{21971N}{320mm^2} = 72.8MPa$$

Average axial stress in the weld is significantly lower than the allowable stress for construction steel, 284 MPa (see Chapter 9.4).

10.5.2 Weld between vertical Reinforcement Bars and horizontal Plates

The weld on the received drawing (see Figure 11) is given as a fillet weld with throat thickness 4 mm.

The area of the weld becomes:

$$A_{weld} = a \cdot \pi \cdot D = 4mm * \pi * 12mm = 150.8mm^2$$

This area shall transfer the shear force, Q=10985 N, found in Chapter 10.4.

Average shear stress in the weld becomes:

$$\tau_{weld} = \frac{Q}{A_{sveis}} = \frac{10985N}{150.8mm^2} = 72.8MPa$$

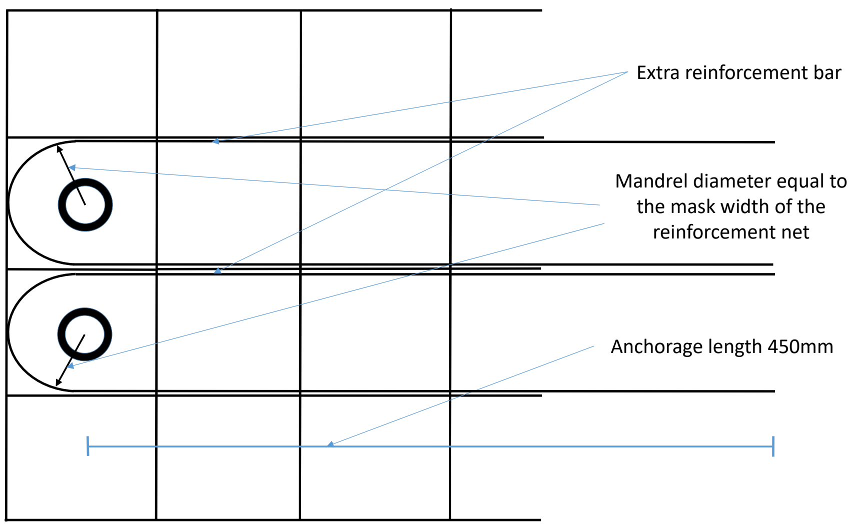
Average allowable shear stress in the weld is chosen equal to allowable shear stress for construction steel.

$$\tau_{weld,max} = \frac{1}{\sqrt{3}} \cdot \frac{355MPa}{1.25} = 164.0MPa$$

Average shear stress in the weld is less than half the allowable shear stress in the weld, and the weld is considered as strong enough.

10.6 Placing in Mesh

The vertical reinforcement bars are aligned with the center of the reinforcing mesh (see Figure 12). Extra reinforcement bars with a mandrel diameter equal to the mask width of the reinforcement net are put in. The anchorage length is set to 450 mm.

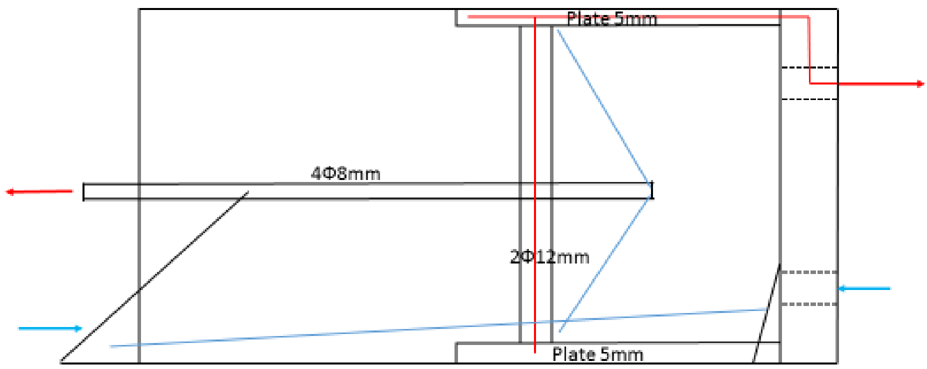


> *Figure 12: Placing of vertical reinforcement bars in the reinforcement net*

10.7 Check of critical Cross Section

10.7.1 General

Critical section will be localized where the railing support ends. It is assumed that the extra reinforcement bars have the same location as the reinforcing mesh. The upper layer of the mesh must be positioned above the center line of the concrete plate.



> *Figure 13: Sketch of the force transfer in the critical section*

Critical section is checked in ShellDesign. Acceptable utilizations are found.

10.7.2 Check of critical Section with centric Load Transfer

PRINT SHEET 1

ShellDesign Section Capacity Check

INPUT | SECTION PLOT

Design parameters:

Design code	=	EC
Limit state	=	ULS

Section geometry:

Thickness	=	0.09	m
Width	=	0.2	m

Materials:

Concrete grade	=	B35	
Reinforcement yielc	=	500	MPa

Material factors:

Concrete	=	1.5
Reinforcement	=	1.15

Longitudinal Reinforcement:

Actual nominal cove = 0.0402 m

Top layers

Layer 1 (1-direction)	=	4	ø	8	c	200	mm
Layer 2 (2-direction)	=		ø		c		mm
Layer 3 (1-direction)	=		ø		c		mm
Layer 4 (2-direction)	=		ø		c		mm

Bottom layers

Layer 1 (1-direction)	=		ø		c		mm
Layer 2 (2-direction)	=		ø		c		mm
Layer 3 (1-direction)	=		ø		c		mm
Layer 4 (2-direction)	=		ø		c		mm

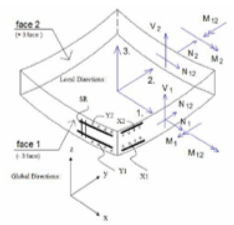
Shear Reinforcement:

Shear Reinforcemer = ø c c

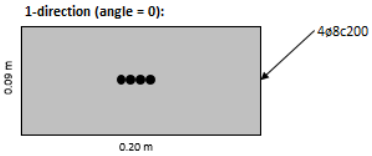
Design loads:

N1	=	1.5	kN
N2	=	0	kN
N12	=	0	kN
M1	=	-1.8675	kNm
M2	=	0	kNm
M12	=	0	kNm
V1	=	0	kN
V2	=	0	kN

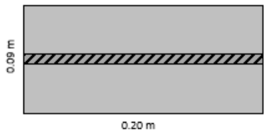
Load definitions:



1-direction (angle = 0):



2-direction (angle = 90):



Project:	10101 Project
Part:	Section 101
Responsible:	JSP
Date:	01.01.2000
Checked sign:	BSN

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Plot | Run SD

NB! Additional forces in the longitudinal reinforcement due to shear forces, ref. EC2 Sec. 6.2.1(7) / NS 3473 Sec. 12.3.4, are not accounted for in ShellDesign and should be handled by the designer. See ShellDesign User Manual for more information.

> *Figure 14: Check of critical section in ShellDesign, page 1 of 2*

PRINT SHEET 2

ShellDesign Section Capacity Check

SECTION RESPONSE | SECTION STRAINS

Design results:

Concrete:	**UR:**
Principal stress bottom 1:	0.000
Principal stress bottom 2:	0.912
Principal stress top 1:	0.000
Principal stress top 2:	0.000

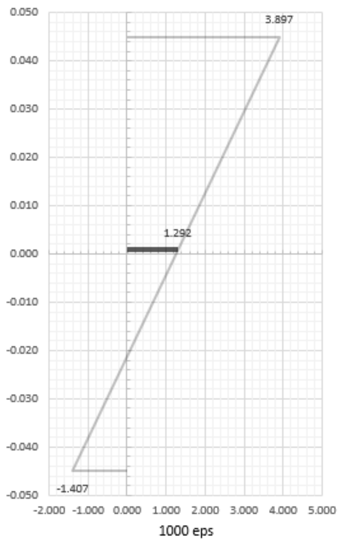
Reinforcement:

Bottom layers

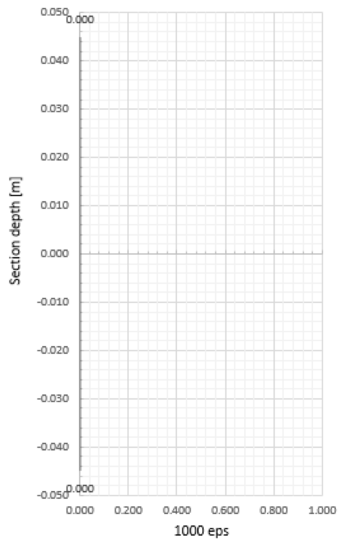
Top layers

Layer 1 (1-direction):	0.594

Strain 1-dir:

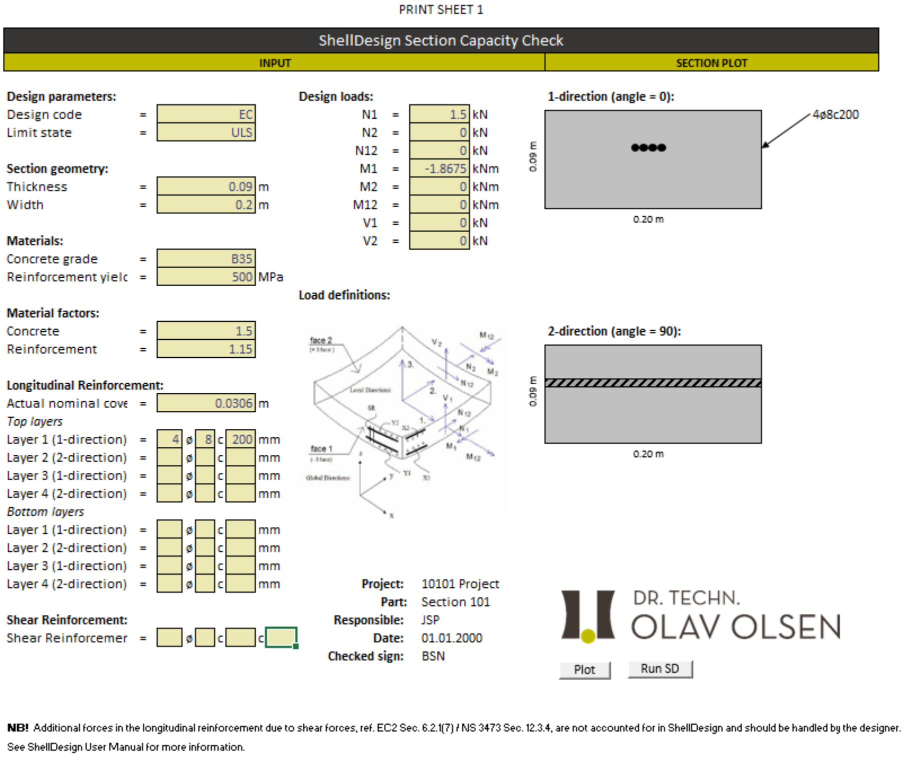


Strain 2-dir:

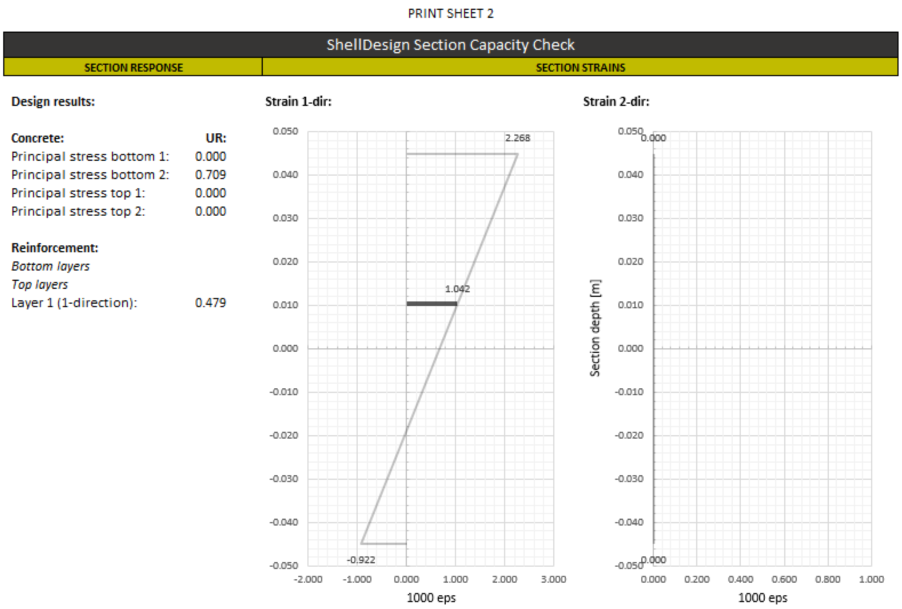


> *Figure 15: Check of critical section in ShellDesign, page 2 of 2*

10.7.3 Check of critical section with load Transfer in upper Net Layer



> *Figure 16: Check of critical section in ShellDesign, page 1 of 2*

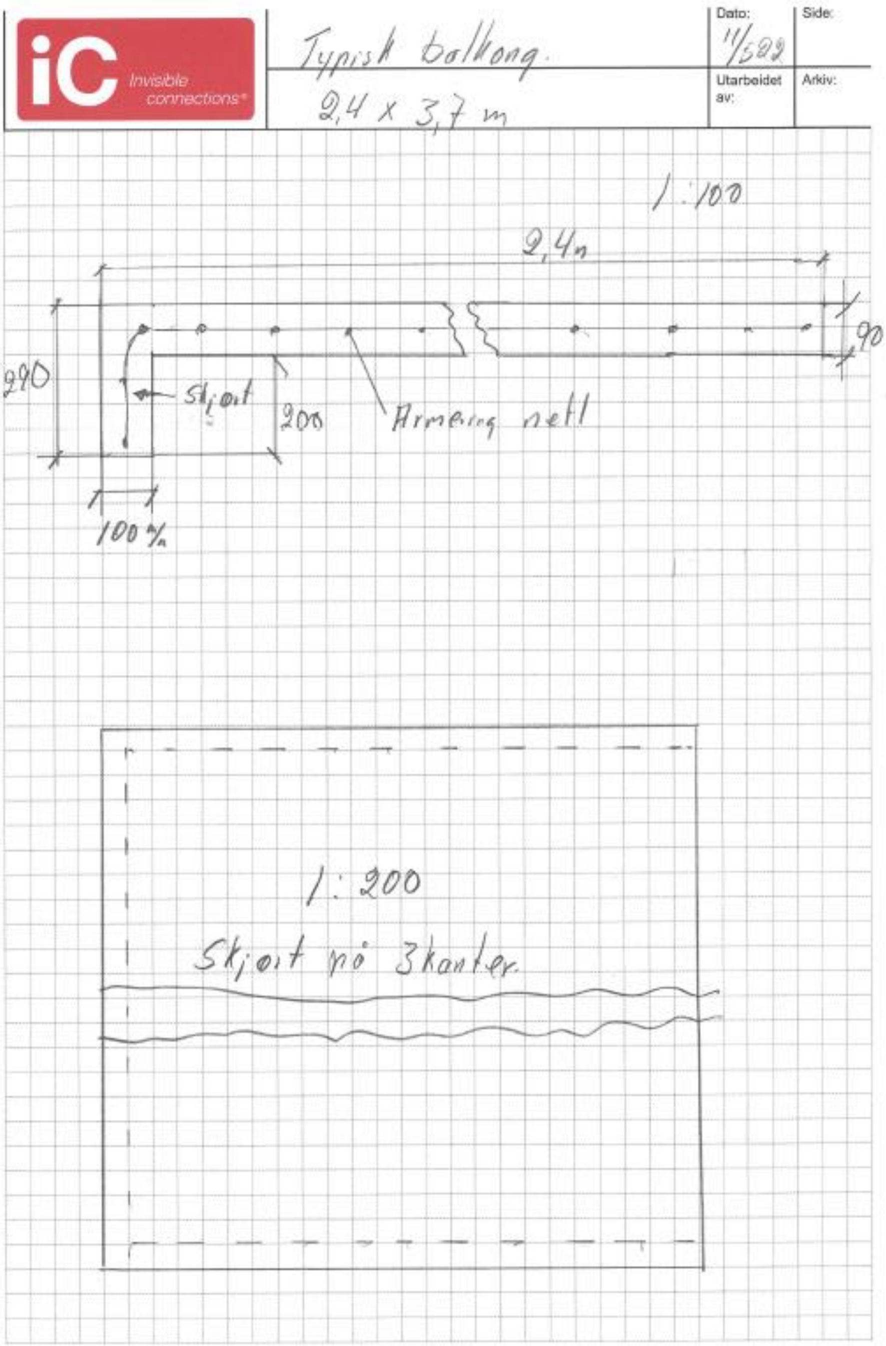


> *Figure 17: Check of critical section in ShellDesign, page 2 of 2*

11 EVALUATION OF THIN BALCONIES WITH SKIRT

11.1 General

From iC we have received a drawing of a typical balcony deck with skirt (sent Dr.techn.Olav Olsen 11. May 2022).



> Figure 18: Typical balcony deck with skirt

The skirt itself has a width of 100 mm and a height of 200 mm. It is placed on the underside of the balcony slab along the three edges not facing the building.

The reinforcing mesh is bended to continue into the skirt.

A short evaluation of structural implications regarding load carrying capacities are carried out in the following.

11.2 Utilization of Cantilever Beams

The bolt connections for the cantilever beams will experience increased loading due to increased self-weight from the balcony skirts. Maximum length of the balcony slab (alternatively width) must be somewhat reduced to compensate for the self-weight increase.

Quantification of the reduction requirement has not been carried out.

11.3 Utilization of Concrete Plate

The self-weight of the concrete slab will increase due to the weight of the concrete skirts. The loading on the concrete plate will be somewhat increased.

The way the concrete calculations have been performed (by hand calculations), the forces and moments in the most critical cross section will not change substantially, and not necessarily in the positive direction (due to the increased self-weight from the skirts).

For the longest balcony plates there might be a need for length reduction. Quantification of the reduction requirement has not been carried out. Short balconies are oversized.

12 REFERENCES

/1/	Norsk Standard. NS-EN 1992-1-1:2004+NA:2008. Eurokode 2: Prosjektering av betongkonstruksjoner. Del 1-1: Allmenne regler og regler for bygninger
/2/	Norsk Standard. NS-EN 1993-1-1:2005+A1:2014+NA:2015. Eurokode 3: Prosjektering av stålkonstruksjoner. Del 1-1: Allmenne regler og regler for bygninger
/3/	Direktoratet for byggkvalitet. Veiledning om tekniske krav til byggverk. §12-17 Rekkverk.
/4/	Norsk Standard NS-EN 1991-1-1:2002+NA:2008. Eurokode 1: Laster på konstruksjoner – Del 1-1: Allmenne laster – Tetthet, egenvekt og nyttelaster i bygninger.
/5/	Norsk Standard NS-EN 1994-1-1:2004+NA2009. Eurokode 4: Prosjektering av samvirkekonstruksjoner av stål og betong – Del 1-1: Allmenne regler og regler for bygninger.

ATTACHMENT A

A.1 ShellDesign, B=1.5m, L=6.20m, ULS

ShellDesignConcrete : 4.4.2/160310 , run date 20170105 , time 12:56

ShellDesign Section Capacity Check

INPUT | SECTION PLOT

Design parameters:

Design code	=	EC	
Limit state	=	ULS	

Section geometry:

Thickness	=	0.0888	m
Width	=	1.0	m

Materials:

Concrete grade	=	B35	
Reinforcement yield	=	500	MPa

Material factors:

Concrete	=	1.5
Reinforcement	=	1.15

Longitudinal Reinforcement:

Actual nominal cover	=	0.0396	m					
Top layers								
Layer 1 (1-direction)	=	1	ø	8	c	100	mm	
Layer 2 (2-direction)	=		ø		c		mm	
Layer 3 (1-direction)	=		ø		c		mm	
Layer 4 (2-direction)	=		ø		c		mm	
Bottom layers								
Layer 1 (1-direction)	=		ø		c		mm	
Layer 2 (2-direction)	=		ø		c		mm	
Layer 3 (1-direction)	=		ø		c		mm	
Layer 4 (2-direction)	=		ø		c		mm	

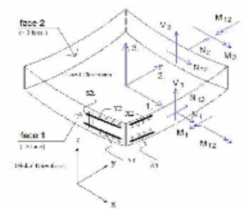
Shear Reinforcement:

Shear Reinforcement	=		ø		c		c

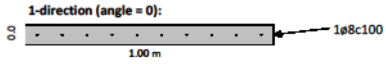
Design loads:

N1	=		kN
N2	=		kN
N12	=		kN
M1	=	-7.32816	kNm
M2	=		kNm
M12	=		kNm
V1	=	16.14187	kN
V2	=		kN

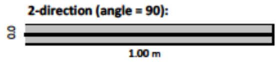
Load definitions:



1-direction (angle = 0):



2-direction (angle = 90):



Project: 10101 Project
Part: Section 101
Responsible: JSP
Date: 01.01.2000
Checked sign: BSN

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Page 1 of 2

ShellDesignConcrete : 4.4.2/160310 , run date 20170105 , time 12:56

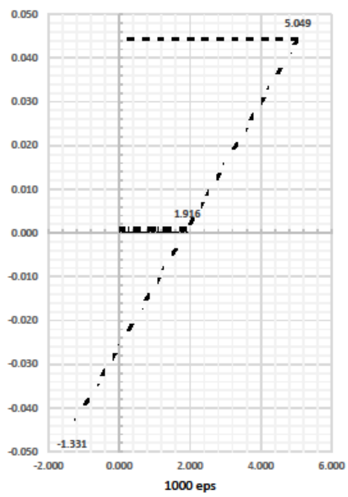
ShellDesign Section Capacity Check

SECTION RESPONSE | SECTION STRAINS

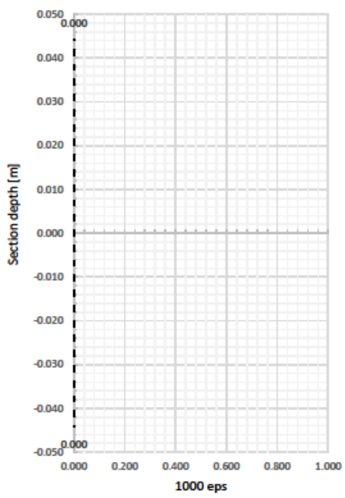
Design results:

Concrete:	**UR:**
Principal stress bottom 1:	0.000
Principal stress bottom 2:	0.888
Principal stress top 1:	0.000
Principal stress top 2:	0.000
Reinforcement:	
Bottom layers	
Top layers	
Layer 1 (1-direction):	0.882
Shear:	
Shear concrete tensile:	0.527
Shear concrete compression:	0.112
Shear reinforcement:	0.000

Strain 1-dir:



Strain 2-dir:



Page 2 of 2

A.2 ShellDesign, B=1.5m, L=6.20m, SLS

ShellDesignConcrete : 4.4.2/160310 , run date 20170105 , time 13:03

ShellDesign Section Capacity Check

INPUT | **SECTION PLOT**

Design parameters:

Design code	=	EC	
Limit state	=	CRW	

Section geometry:

Thickness	=	0.0888	m
Width	=	1.0	m

Materials:

Concrete grade	=	B35	
Reinforcement yield	=	500	MPa

Material factors:

Concrete	=	1
Reinforcement	=	1

Longitudinal Reinforcement:

Actual nominal cover	=	0.0396 m				
Top layers						
Layer 1 (1-direction)	=	1	ø 8	c	100	mm
Layer 2 (2-direction)	=		ø	c		mm
Layer 3 (1-direction)	=		ø	c		mm
Layer 4 (2-direction)	=		ø	c		mm
Bottom layers						
Layer 1 (1-direction)	=		ø	c		mm
Layer 2 (2-direction)	=		ø	c		mm
Layer 3 (1-direction)	=		ø	c		mm
Layer 4 (2-direction)	=		ø	c		mm

Shear Reinforcement:

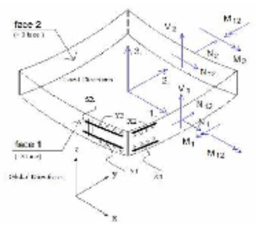
Shear Reinforcement = ø c c

Crack width data:

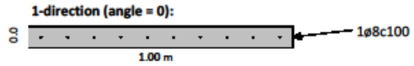
Design loads:

N1	=		kN
N2	=		kN
N12	=		kN
M1	=	-7.32816	kNm
M2	=		kNm
M12	=		kNm
V1	=	16.14187	kN
V2	=		kN

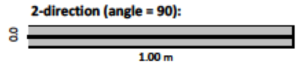
Load definitions:



1-direction (angle = 0):



2-direction (angle = 90):



Project:	10101 Project
Part:	Section 101
Responsible:	JSP
Date:	01.01.2000
Checked sign:	BSN

DR. TECHN. OLAV OLSEN

Page 1 of 2

ShellDesignConcrete : 4.4.2/160310 , run date 20170105 , time 13:03

ShellDesign Section Capacity Check

SECTION RESPONSE | **SECTION STRAINS**

Design results:

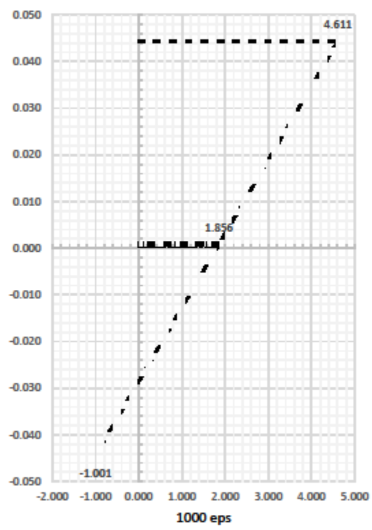
Crack widths [mm]:

Top face (angle = 0.0): 0.317

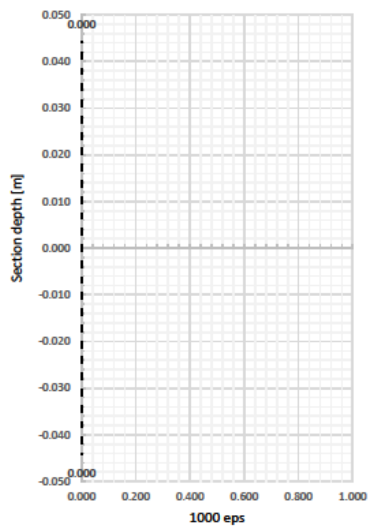
Crack widths UR:

Top face (angle = 0.0): 0.881

Strain 1-dir:



Strain 2-dir:



Page 2 of 2

ATTACHMENT B

A.3 Hand calculation, B=1.5m, L=6.20m, ULS

	A	B	C	D	E	F	G
1							
2							
3	h	88.8	mm	tverrsnittshøyde			
4	Ø	8	mm	armeringsdiameter			
5	cc	100	mm				
6	As	503	mm2/m	tv.sn.areal hoverarmering			
7	d_inf	44.4	mm	statisk høyde for støttemoment			
8	d_sup	44.4	mm	statisk høyde for feltmoment			
9	c	30	mm	minimumsoverdekning			
10	c act.	30	mm	effektiv betongoverdekning ved sentrisk arm.nett			
11		**OK**					
12							
13	f_sd	435	MPa	dimensjonerende arm.fasthet			
14	f_cd	19	MPa	dimensjonerende betongfasthet			
15	E_s	210000	MPa				
16	eps_cu	0.00350					
17	eps_su	0.00207					
18	x/d max	0.63		kriterium for sprødbrud			
19							
20							
21	**Kapasitetskontroll for støttemoment**				**Kapasitetskontroll for feltmoment**		
22	M	7.33	kNm		M	7.33	kNm
23	F_sd	219	kN/m		F_sd	219	kN/m
24	F_cd	219	kN/m		F_cd	219	kN/m
25							
26	x faktor	0.8			x faktor	0.8	
27	x	15	mm		x	15	mm
28	d_inf	44.4	mm		d_sup	44.4	mm
29	x_max	28	mm		x_max	28	mm
30	Sprødbrudd	**0.52**			Sprødbrudd	**0.52**	
31	z	39	mm		z	39	mm
32	M_Rd	**8.4**	kNm/m		M_Rd	8.4	kNm/m
33	UR	**0.87**			UR	**0.87**	
34							