EUROCODES
Background and Applications

"Dissemination of information for training" workshop

18-20 February 2008

Brussels

EN 1992
Eurocode 2: Design of concrete structures

Organised by
European Commission: DG Enterprise and Industry, Joint Research Centre

with the support of
CEN/TC250, CEN Management Centre and Member States

Wednesday, February 20 – Palais des Académies

EN 1992 - Eurocode 2: Design of concrete structures
Baron Lacquet room

9:00-10:30	EN1992-1-1	J. Walraven *TU Delft*
10:30-11:00	Coffee	
11:00-12:00	EN1992-1-1	J. Walraven *TU Delft*
12:00-13:30	Lunch	
13:30-15:30	EN1992-2	G. Mancini *Politecnico di Torino*
15:30-16:00	Coffee	
16:00-17:00	EN1992-3	T. Jones *Arup*

All workshop material will be available at
http://eurocodes.jrc.ec.europa.eu

EN1992-1-1

J. Walraven
TU Delft

Eurocode 2: Design of concrete structures EN1992-1-1

Symposium Eurocodes: Backgrounds and Applications, Brussels 18-20 February 2008

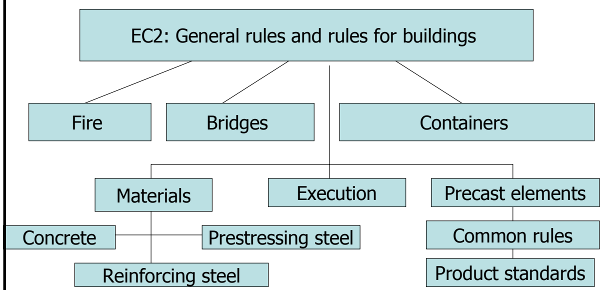
J.C. Walraven

02 February 2008

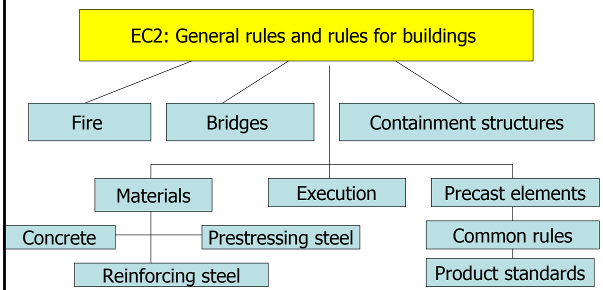
Requirements to a code

1. Scientifically well founded, consistent and coherent
2. Transparent
3. New developments reckognized as much as possible
4. Open minded: models with different degree of complexity allowed
5. As simple as possible, but not simplier
6. In harmony with other codes

EC-2: Concrete Structures



EC-2: Concrete Structures



EN 1992-1-1 "Concrete structures" (1)

Content:

1. **General**
2. **Basics**
3. **Materials**
4. **Durability and cover**
5. **Structural analysis**
6. **Ultimate limit states**
7. **Serviceability limit states**
8. **Detailing of reinforcement**
9. **Detailing of members and particular rules**
10. **Additional rules for precast concrete elements and structures**
11. **Lightweight aggregate concrete structures**
12. **Plain and lightly reinforced concrete structures**

EN 1992-1-1 "Concrete structures" (2)

Annexes:

- **A. Modifications of safety factor (I)**
- **B. Formulas for creep and shrinkage (I)**
- **C. Properties of reinforcement (N)**
- **D. Prestressing steel relaxation losses (I)**
- **E. Indicative strength classes for durability (I)**
- **F. In-plane stress conditions (I)**
- **G. Soil structure interaction (I)**
- **H. Global second order effects in structures (I)**
- **I. Analysis of flat slabs and shear walls (I)**
- **J. Detailing rules for particular situations (I)**

I = Informative
N = Normative

EN 1992-1-1 "Concrete structures" (3)

In EC-2 "Design of concrete structures –
Part 1: General rules and rules for buildings

109 national choices are possible

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Chapter: 3 Materials

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Delft University of Technology

Concrete strength classes

Concrete strength class C8/10 tot C100/115.
(Characteristic cylinder strength / char. cube strength)

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Concrete strength classes and properties

	Strength classes for concrete													
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0
$f_{ctk,0,05}$ (MPa)	1 1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6
E_{cm} (Gpa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2,5	2,6
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6
n	2,0									1,75	1,6	1,45	1,4	1,4
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6

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Design Strength Values

(3.1.6)

- Design compressive strength, f_{cd}

  $f_{cd} = \alpha_{cc}\, f_{ck}/\gamma_c$

- Design tensile strength, f_{ctd}

  $f_{ctd} = \alpha_{ct}\, f_{ctk,0.05}/\gamma_c$

α_{cc} (= 1,0) and α_{ct} (= 1,0) are coefficients to take account of long term effects on the compressive and tensile strengths and of unfavourable effects resulting from the way the load is applied (national choice)

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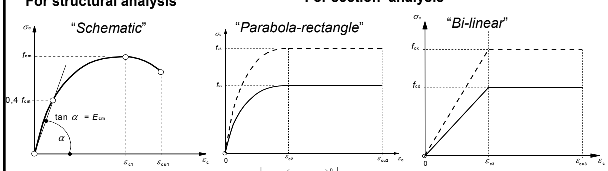
Concrete stress - strain relations (3.1.5 and 3.1.7)

For structural analysis

For section analysis



$\varepsilon_{c1}\ (^0/_{00}) = 0{,}7\ f_{cm}^{\ 0,31}$

$\varepsilon_{cu1}\ (^0/_{00}) =$

$2{,}8 + 27[(98\text{-}f_{cm})/100]^4\ f_{cm})/100]^4$

for $f_{ck} \geq 50$ MPa otherwise 3.5

$$\sigma_c = f_{cd}\left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \text{ for } 0 \leq \varepsilon_c < \varepsilon_{c2}$$

$$\sigma_c = f_{cd} \text{ for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2}$$

$n = 1{,}4 + 23{,}4\ [(90\text{-}\ f_{ck})/100]^4$
for $f_{ck} \geq 50$ MPa otherwise 2,0

$\varepsilon_{c2}\ (^0/_{00}) = 2{,}0 + 0{,}085(f_{ck}\text{-}50)^{0,53}$

for $f_{ck} \geq 50$ MPa otherwise 2,0

$\varepsilon_{cu2}\ (^0/_{00}) = 2{,}6 + 35\ [(90\text{-}f_{ck})/100]^4$
for $f_{ck} \geq 50$ MPa otherwise 3,5

$\varepsilon_{c3}\ (^0/_{00}) = 1{,}75 + 0{,}55\ [(f_{ck}\text{-}50)/40]$
for $f_{ck} \geq 50$ MPa otherwise 1,75

$\varepsilon_{cu3}\ (^0/_{00}) = 2{,}6+35[(90\text{-}f_{ck})/100]^4$
for $f_{ck} \geq 50$ MPa otherwise 3,5

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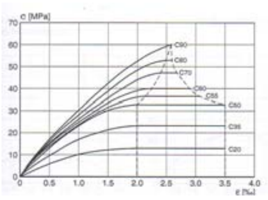
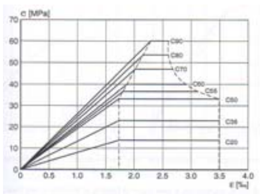
Concrete stress-strain relations

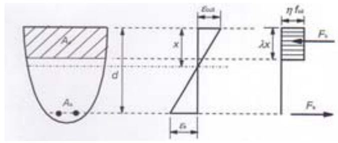
- Higher concrete strength show more brittle behaviour, reflected by shorter horizontal branche





Chapter 3.1: Concrete

Simplified σ - ε relation for cross sections with non rectangular cross-section



$\lambda = 0{,}8$ for $f_{ck} \leq 50$ MPa
$\lambda = 0{,}8 - (f_{ck}-50)/400$ for $50 \leq fck \leq 90$ MPa

$\eta = 1{,}0$ for $f_{ck} \leq 50$ MPa
$\eta = 1{,}0 - (f_{ck}-50)/200$ for $50 \leq fck \leq 90$ MPa

Shrinkage (3.1.4)

- The shrinkage strain ε_{cs} is composed of two components:
  $\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$

where
- drying shrinkage strain
  $\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd,0}$ where $\varepsilon_{cd,0}$ is the basic drying shrinkage strain
- autogenous shrinkage strain
  $\varepsilon_{ca}(t) = \beta_{as}(t) \cdot \varepsilon_{ca}(\infty)$

Autogenous shrinkage

Stichtse Bridge, 1997:
Autogenous shrinkage
20.10^{-3} after 2 days

Concrete strength f_c=90 MPa

Shrinkage (3.1.4)

$$\beta_{ds}(t,t_s) = \frac{(t-t_s)}{(t-t_s)+0{,}04\sqrt{h_0^3}}$$

where t = age of concrete at time considered, t_s= age at beginning of drying shrinkage (mostly end of curing)

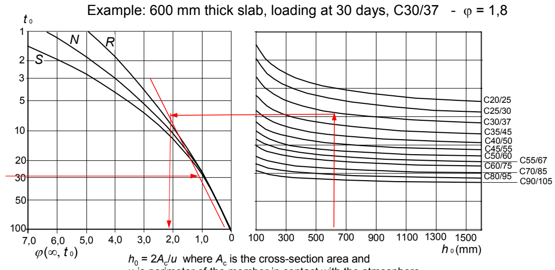
$$\varepsilon_{ca}(t) = \beta_{as}(t)\varepsilon_{ca}(\infty)$$

where

$$\varepsilon_{ca}(\infty) = 2{,}5(f_{ck}-10)\cdot 10^{-6} \quad \text{and} \quad \beta_{as}(t) = 1-\exp(-0{,}2t^{0{,}5})$$

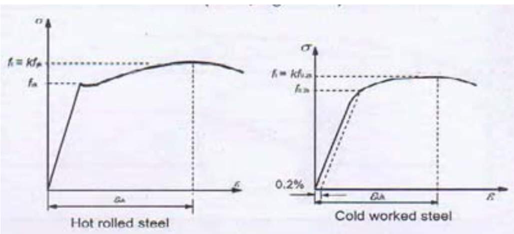
Creep of concrete (3.1.4)

Inside conditions – RH = 50%
Example: 600 mm thick slab, loading at 30 days, C30/37 - φ = 1,8



$h_0 = 2A_c/u$ where A_c is the cross-section area and u is perimeter of the member in contact with the atmosphere

Stress-strain relations for reinforcing steel



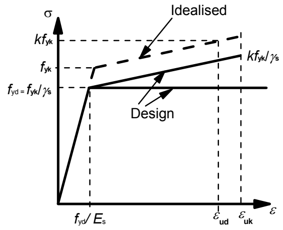
Reinforcement (2) – From Annex C

Product form	Bars and de-coiled rods			Wire Fabrics		
Class	A	B	C	A	B	C
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600					
$k = (f_t/f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35
Characteristic strain at maximum force, ε_{uk} (%)	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5
Fatigue stress range (N = 2 x 10^6) (MPa) with an upper limit of 0.6f_{yk}	150			100		

cold worked hot rolled seismic

Idealized and design stress strain relations for reinforcing steel

Alternative design stress/strain relationships are permitted:
- inclined top branch with a limit to the ultimate strain horizontal
- horizontal top branch with no strain limit



$k = (f_t/f_y)_k$

$\varepsilon_{ud} = 0.9\ \varepsilon_{uk}$

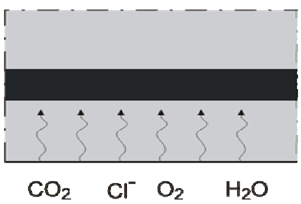
Durability and cover

Prof.dr.ir. J.C. Walraven

02 February 2008

Group Concrete Structures

Penetration of corrosion stimulating components in concrete



Deterioration of concrete

Corrosion of reinforcement by chloride penetration

Avoiding corrosion of steel in concrete

Design criteria
- Aggressivity of environment
- Specified service life

Design measures
- Sufficient cover thickness
- Sufficiently low permeability of concrete (in combination with cover thickness)
- Avoiding harmfull cracks parallel to reinforcing bars

- Other measures like: stainless steel, cathodic protection, coatings, etc.

Aggressivity of the environment

Main exposure classes:

- The exposure classes are defined in EN206-1. The main classes are:

- XO – no risk of corrosion or attack
- XC – risk of carbonation induced corrosion
- XD – risk of chloride-induced corrosion (other than sea water)
- XS – risk of chloride induced corrosion (sea water)
- XF – risk of freeze thaw attack
- XA – Chemical attack

Agressivity of the environment

Further specification of main exposure classes in subclasses (I)

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs

Procedure to determine $c_{min,dur}$

EC-2 leaves the choice of $c_{min,dur}$ to the countries, but gives the following recommendation:

The value $c_{min,dur}$ depends on the "structural class", which has to be determined first. If the specified service life is 50 years, the structural class is defined as 4. The "structural class" can be modified in case of the following conditions:

-The service life is 100 years in stead of 50 years
-The concrete strength is higher than necessary
- Slabs (position of reinforcement not affected by construction process
- Special quality control measures apply

The finally applying service class can be calculated with Table 4.3N

Table for determining final Structural Class

Structural Class							
Criterion	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class 1) 2)	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Final determination of $c_{min,dur}$ (1)

The value $c_{min,dur}$ is finally determined as a function of the structural class and the exposure class:

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Special considerations

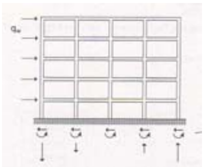
In case of stainless steel the minimum cover may be reduced. The value of the reduction is left to the decision of the countries (0 if no further specification).

Structural Analysis

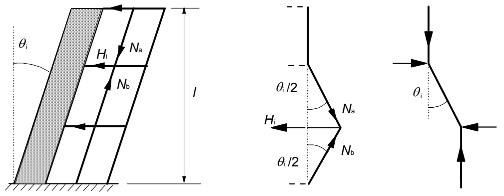
Methods to analyse structures

Linear elastic analysis

1. Suitable for ULS and SLS
2. Assumptions:
   - uncracked cross-sections
   - linear σ - ε relations
   - mean E-modulus
3. Effect of imposed deformations in ULS to be calculated with reduced stiffnesses and creep



Forces due to geometric imperfections on structures(5.2)



Bracing System

$H_i = \theta_i (N_b - N_a)$

Floor Diaphragm

$H_i = \theta_i (N_b + N_a)/2$

Roof

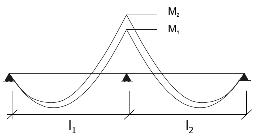
$H_i = \theta_i N_a$

Methods to analyse structures

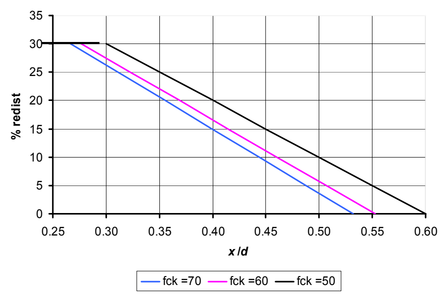
5.5 Linear elastic analysis with limited redistribution

1. Valid for $0{,}5 \leq l_1 / l_2 \leq 2{,}0$
2. Ratio of redistribution δ, with
   $\delta \geq k_1 + k_2\, x_u/d$ for $f_{ck} \leq 50$ MPa
   $\delta \geq k_3 + k_4\, x_u/d$ for $f_{ck} > 50$ MPa

   $\delta \geq k_5$ for reinforcement class B or C
   $\delta \geq k_6$ for reinforcement class A



Redistribution limits for Class B & C steel

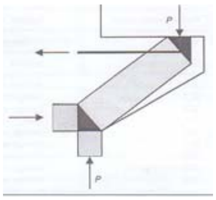


Methods to analyse structures

5.6 Plastic methods of analysis

(b) Strut and tie analysis
(lower bound)

- Suitable for ULS
- Suitable for SLS if compatibility is ensured (direction of struts oriented to compression in elastic analysis



Methods to analyse structures

Ch. 5.7 Nonlinear analysis

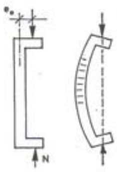
"Nonlinear analysis may be used for both ULS and SLS, provided that equilibrium and compatibility are satisfied and an adequate non-linear behaviour for materials is assumed. The analysis may be first or second order".

Chapter 5 "Structural analysis"

5.8 Second order effects with axial loads

- Slenderness criteria for isolated members and buildings (when is 2nd order analysis required?)
- Methods of second order analysis
  - General method based on nonlinear behaviour, including geometric nonlinearity
  - Analysis based on nominal stiffness
  - Analysis based on moment magnification factor
  - Analysis based on nominal curvature

Extended calculation tools are given



Methods of analysis

Lateral buckling of beams

No lateral buckling if:

- persistent situations: $\frac{l_{0t}}{b} \le \frac{50}{(h/b)^{1/3}}$ and $h/b \le 2,5$
- transient situations: $\frac{l_{0t}}{b} \le \frac{70}{(h/b)^{1/3}}$ and $h/b \le 3,5$

where:
l_{0t} is the distance between torsional restraints
h is the total depth of beam in central part of l_{0t}
b is the width of compression flange

Bending with or without axial force

Prof.dr.ir. J.C. Walraven

02 February 2008

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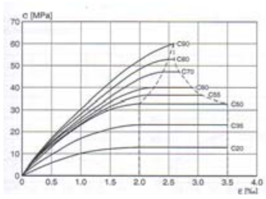
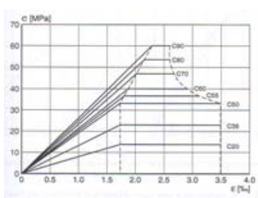
Delft University of Technology

Concrete design stress strain relations for different strength classes

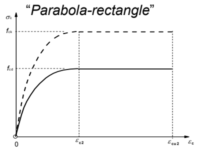
- Higher concrete strength shows more brittle behaviour, reflected by shorter horizontal branche





Concrete design stress - strain relations (3.1.5 and 3.1.7) for section analysis

"Parabola-rectangle"



$$\sigma_c = f_{cd}\left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \text{ for } 0 \le \varepsilon_c < \varepsilon_{c2}$$

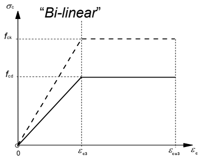
$$\sigma_c = f_{cd} \text{ for } \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2}$$

$n = 1,4 + 23,4\ [(90 - f_{ck})/100]^4$
for $f_{ck} \ge 50$ MPa otherwise 2,0

ε_{c2} ($^0/_{00}$) = 2,0 + 0,085(f_{ck}-50)^{0,53}
for $f_{ck} \ge 50$ MPa otherwise 2,0

ε_{cu2} ($^0/_{00}$) = 2,6 + 35 [(90-f_{ck})/100]^4
for $f_{ck} \ge 50$ MPa otherwise 3,5

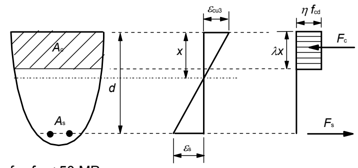
"Bi-linear"



ε_{c3} ($^0/_{00}$) = 1,75 + 0,55 [(f_{ck}-50)/40]
for $f_{ck} \ge 50$ MPa otherwise 1,75

ε_{cu3} ($^0/_{00}$) =2,6+35[(90-f_{ck})/100]^4
for $f_{ck} \ge 50$ MPa otherwise 3,5

Simplified concrete design stress block



$\lambda = 0,8$ for $f_{ck} \le 50$ MPa

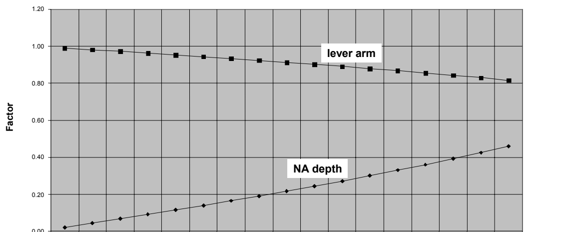
$= 0,8 - \frac{(f_{ck} - 50)}{400}$ for $50 < f_{ck} \le 90$ MPa

$\eta = 1,0$ for $f_{ck} \le 50$ MPa

$= 1,0 - (f_{ck} - 50)/200$ for $50 < f_{ck} \le 90$ MPa

Simplified factors for flexure (1)

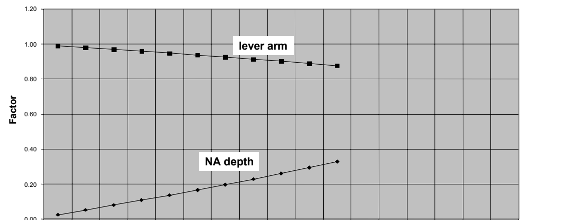
Factors for NA depth (n) and lever arm (=z) for concrete grade ≤ 50 MPa



M/bd^2f_{ck}	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.11	0.12	0.13	0.14	0.15	0.16	0.17
n	0.02	0.04	0.07	0.09	0.12	0.14	0.17	0.19	0.22	0.24	0.27	0.30	0.33	0.36	0.39	0.43	0.46
z	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.88	0.87	0.86	0.84	0.83	0.82

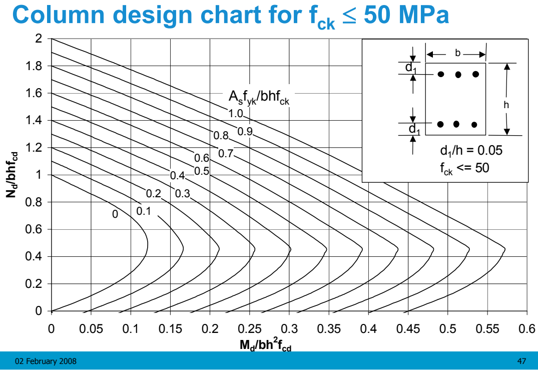
Simplified factors for flexure (2)

Factors for NA depth (=n) and lever arm (=z) for concrete grade 70 MPa

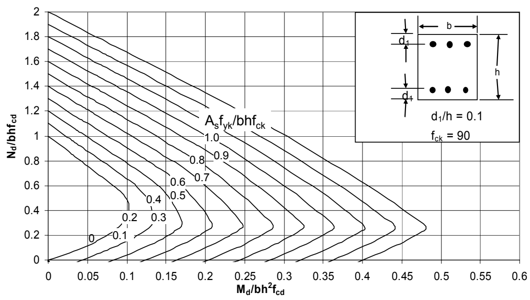


M/bd^2f_{ck}	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.11	0.12	0.13	0.14	0.15	0.16	0.17
n	0.03	0.05	0.08	0.11	0.14	0.17	0.20	0.23	0.26	0.29	0.33						
z	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.91	0.90	0.89	0.88						

Column design chart for $f_{ck} \le 50$ MPa



Column design chart for f_{ck} = 70 MPa



Shear

Prof.dr.ir. J.C. Walraven

02 February 2008

Group Concrete Structures

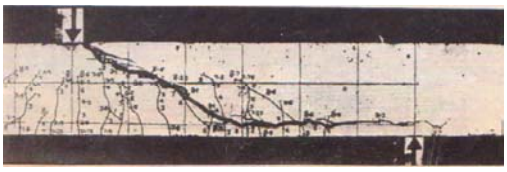
Principles of shear control in EC-2

Until a certain shear force $V_{Rd,c}$ no calculated shear reinforcement is necessary (only in beams minimum shear reinforcement is prescribed)

If the design shear force is larger than this value $V_{Rd,c}$ shear reinforcement is necessary for the full design shear force. This shear reinforcement is calculated with the variable inclination truss analogy. To this aim the strut inclination may be chosen between two values (recommended range $1 \leq \cot\ \theta \leq 2{,}5$)

The shear reinforcement may not exceed a defined maximum value to ensure yielding of the shear reinforcement

Concrete slabs without shear reinforcement



Shear resistance $V_{Rd,c}$ governed by shear flexure failure: shear crack develops from flexural crack

Concrete slabs without shear reinforcement

Prestressed hollow core slab

Shear resistance $V_{Rd,c}$ governed by shear tension failure: crack occurs in web in region uncracked in flexure

Concrete beam reinforced in shear

Shear failure introduced by yielding of stirrups, followed by strut rotation until web crushing

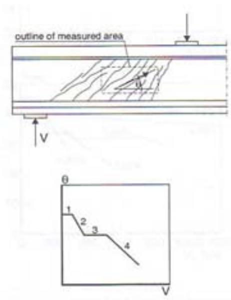
Principle of variable truss action

approach "Variable inclination struts":
a realistic

Stage 1: web uncracked in shear

Stage 2: inclined cracks occur
Stage 3: stabilized inclined cracks
Stage 4: yielding of stirrups, further rotation, finally web crushing

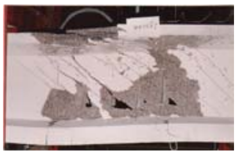
Strut rotation as measured in tests (TU Delft)



Principles of variable angle truss

Strut rotation, followed by new cracks under lower angle, even in high strength concrete (Tests TU Delft)

Web crushing in concrete beam



Web crushing provides maximum to shear resistance

Advantage of variable angle truss analogy

-Freedom of design:
- low angle θ leads to low shear reinforcement
- High angle θ leads to thin webs, saving concrete and dead weight

Optimum choice depends on type of structure

- Transparent equilibrium model, easy in use

Shear design value under which no shear reinforcement is necessary in elements unreinforced in shear (general limit)

$$V_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} b_w d$$

$C_{Rd,c}$ coefficient derived from tests (recommended 0,12)
k size factor = 1 + √(200/d) with d in meter
ρ_l longitudinal reinforcement ratio (≤0,02)
f_{ck} characteristic concrete compressive strength
b_w smallest web width
d effective height of cross section

Shear design value under which no shear reinforcement is necessary in elements unreinforced in shear (general limit)

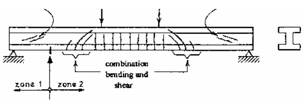
Minimum value for $V_{Rd,c}$:

$V_{Rd,c} = v_{min} b_w d$

Values for v_{min} (N/mm²)

	d=200	d=400	d=600	d=800
C20	0,44	0,35	0,25	0,29
C40	0,63	0,49	0,44	0,41
C60	0,77	0,61	0,54	0,50
C80	0,89	0,70	0,62	0,58

Shear design value under which no shear reinforcement is necessary in elements unreinforced in shear (special case of shear tension)



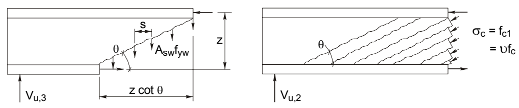
Special case of shear tension (example hollow core slabs)

$$V_{Rd,c} = \frac{I \cdot b_w}{S}\sqrt{(f_{ctd})^2 + \alpha_l \sigma_{cp} f_{ctd}}$$

I	*moment of inertia*
b_w	*smallest web width*
S	*section modulus*
f_{ctd}	*design tensile strength of concrete*
α_l	*reduction factor for prestress in case of prestressing strands or wires in ends of member*
σ_{cp}	*concrete compressive stress at centroidal axis ifor for fully developed prestress*

TU Delft

Design of members if shear reinforcement is needed ($V_{E,d}>V_{Rd,c}$)



For most cases:
-Assume cot θ = 2,5 (θ = 21,8⁰)
-Calculate necessary shear reinforcement
-Check if web crushing capacity is not exceeded ($V_{Ed}>V_{Rd,s}$)
-If web crushing capacity is exceeded, enlarge web width or calculate the value of cot θ for which $V_{Ed} = V_{Rd,c}$ and repeat the calculation

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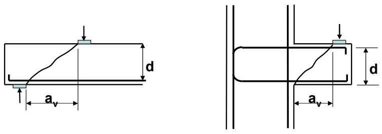
Special case of loads near to supports



For $a_v \leq 2d$ the contribution of the point load to the shear force V_{Ed} may be reduced by a factor $a_v/2d$ where $0.5 \leq a_v \leq 2d$ provided that the longitudinal reinforcement is fully anchored at the support. However, the condition

$V_{Ed} \leq 0{,}5 b_w d \upsilon f_{cd}$

should always be fulfilled

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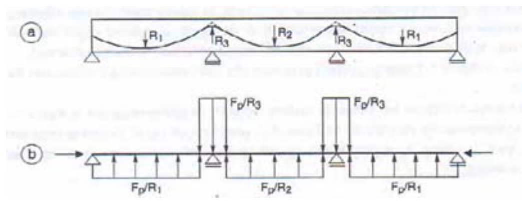
Influence of prestressing on shear resistance (1)

1. Prestressing introduces a set of loads on the beam



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Influence of prestressing on shear resistance (2)

Prestressing increases the load $V_{Rc,d}$ below which no calculated shear reinforcement is required

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d$$

k_1	*coefficient, with recommended value 0,15*
σ_{cp}	*concrete compressive stress at centroidal axis due to axial loading or prestressing*

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Influence of prestressing on shear resistance (3)

1. Prestressing increases the web crushing capacity

$$V_{Rd,\max} = \alpha_{cw} b_w z \nu f_{cd} / (\cot\theta + \tan\theta)$$

α_{cw} *factor depending on prestressing force*

$\alpha_{cw} =$	1	*for non prestressed structures*
	$(1+\sigma_{cp}/f_{cd})$	*for* $0{,}25 < \sigma_{cp} < 0{,}25 f_{cd}$
	$1{,}25$	*for* $0{,}25 f_{cd} < \sigma_{cp} < 0{,}5 f_{cd}$
	$2{,}5(1-\sigma_{cp}/f_{cd})$	*for* $0{,}5 f_{cd} < \sigma_{cp} < 1{,}0 f_{cd}$

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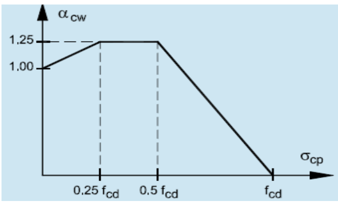
Increase of web crushing capacity by prestressing (4)



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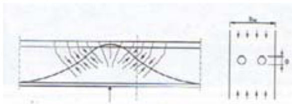
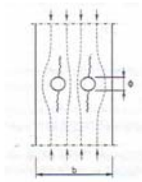
Influence of prestressing on shear resistance (4)

Reducing effect of prestressing duct (with or without tendon) on web crushing capacity

Grouted ducts $b_{w,nom} = b_w - \Sigma\phi$

Ungrouted ducts $b_{w,nom} = b_w - 1{,}2\,\Sigma\phi$





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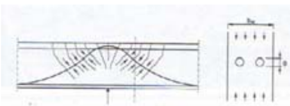
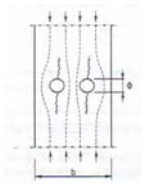
Influence of prestressing on shear resistance (5)

Reducing effect of prestressing duct (with or without tendon) on web crushing capacity

Grouted ducts $b_{w,nom} = b_w - \Sigma\phi$

Ungrouted ducts $b_{w,nom} = b_w - 1{,}2\,\Sigma\phi$





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Shear at the interface between concretes cast at different times

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Shear at the interface between concrete's cast at different times (Eurocode 2, Clause 6.5.2)

$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd}\,(\mu \cdot \sin\beta + \cos\beta) \leq 0{,}5\,\nu \cdot f_{cd}$

(=tan α)

f_{ctd} =concrete design tensile strength
σ_n = eventual confining stress, not from reinforcement
ρ= reinforcement ratio
β = inclination between reinforcement and concrete surface
f_{cd} = concrete design compressive strength
υ = 0,6 for $f_{ck} \leq 60$ MPa
= 0,9 – $f_{ck}/200 \geq 0{,}5$ for $f_{ck} \geq 60$ MPa

	c	μ
Very smooth	0,25	0,5
smooth	0,35	0,6
rough	0,45	0,7
indented	0,50	0,8

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Torsion

Prof.dr.ir. J.C. Walraven

02 February 2008

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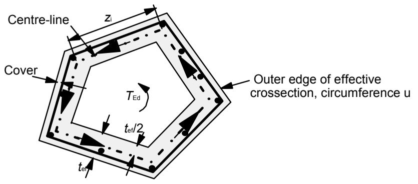
Modeling solid cross sections by equivalent thin-walled cross sections



Effective wall-thickness follows from $t_{ef,i}$=A/u, where;
A = total area of cross section within outer circumference, including hollow areas
U = outer circumference of the cross section

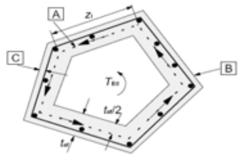
Design procedure for torsion (1)

Shear flow in any wall follows from:

$$\tau_{t,i} t_{ef,i} = \frac{T_{Ed}}{2A_k}$$

where

$\tau_{t,I}$ *torsional shear stress in wall I*
$t_{ef,I}$ *effective wall thickness (A/u)*
T_{Ed} *applied torsional moment*
A_k *area enclosed by centre lines of connecting walls, including hollow areas*



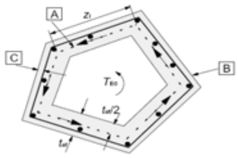
Design procedure for torsion (2)

Shear force V_{Ed} in wall i due to torsion is:

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i$$

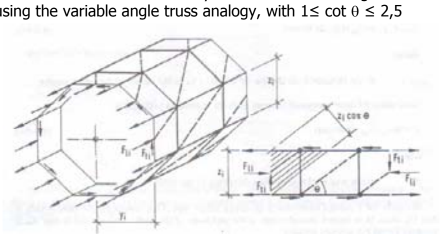
where

$\tau_{t,I}$ *torsional shear stress in wall i*
$t_{ef,I}$ *effective wall thickness (A/u)*
Z_i *inside length of wall I defined by distance of intersection points with adjacent walls*



Design procedure for torsion (3)

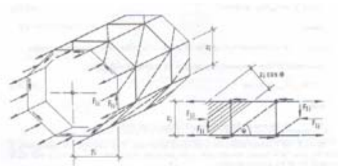
The shear reinforcement in any wall can now be designed like a beam using the variable angle truss analogy, with 1≤ cot θ ≤ 2,5



Design procedure for torsion (4)

The longitudinal reinforcement in any wall follows from:

$$\frac{\Sigma A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot\theta$$



where

u_k *perimeter of area* A_k
f_{yk} *design yield stress of steel*
θ *angle of compression struts*

Punching shear

Prof.dr.ir. J.C. Walraven

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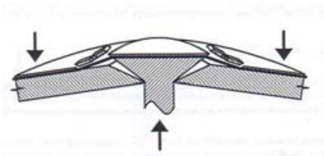
Design for punching shear

Most important aspects:

- Control perimeter
- Edge and corner columns
- Simplified versus advanced control methods



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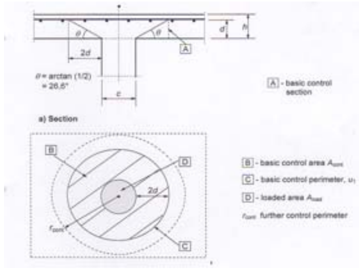
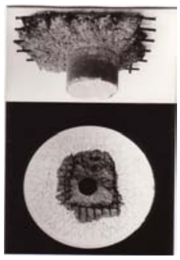
Definition of control perimeter





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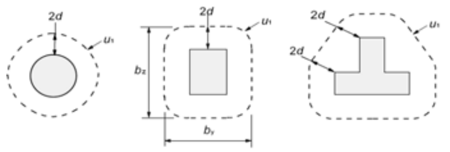
Definition of control perimeters

The basic control perimeter u_1 is taken at a distance 2,0d from the loaded area and should be constructed as to minimise its length



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Limit values for design punching shear stress in design

The following limit values for the punching shear stress are used in design:

If $v_{Ed} \leq v_{Rd,c}$ *no punching shear reinforcement required*

where:

$$v_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + 0{,}10 \sigma_{cp} \geq (v_{min} + 0{,}10 \sigma_{cp})$$

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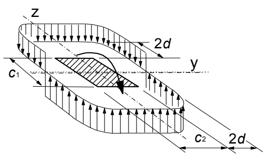
How to take account of eccentricity

More sophisticated method for internal columns:



$$\beta = 1 + 1{,}8 \sqrt{\left(\frac{e_y}{b_z}\right)^2 + \left(\frac{e_z}{b_y}\right)^2}$$

e_y and e_z eccentricities M_{Ed}/V_{Ed} along y and z axes

b_y and b_z dimensions of control perimeter

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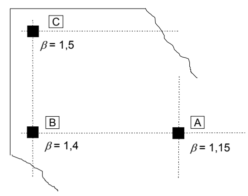
How to take account of eccentricity

Or, how to determine β in equation $v_{Ed} = \beta \frac{V_{Ed}}{u_i d}$



For structures where lateral stability does not depend on frame action and where adjacent spans do not differ by more than 25% the approximate values for β shown below may be used:

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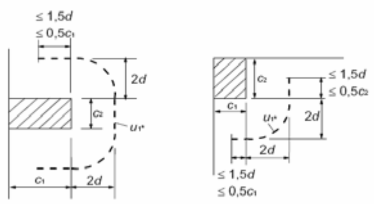
How to take account of eccentricity

Alternative for edge and corner columns: use perimeter u_{1*} in stead of full perimeter and assume uniform distribution of punching force



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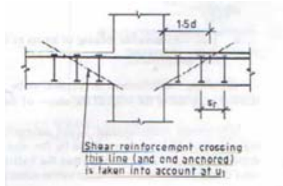
Design of punching shear reinforcement

If $v_{Ed} \geq v_{Rd,c}$ shear reinforcement is required.

The steel contribution comes from the shear reinforcement crossing a surface at 1,5d from the edge of the loaded area, to ensure some anchorage at the upper end. The concrete component of resistance is taken 75% of the design strength of a slab without shear reinforcement



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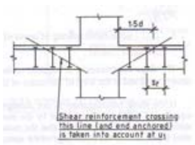
Punching shear reinforcement

Capacity with punching shear reinforcement

$V_u = 0{,}75V_{Rd,c} + V_S$

Shear reinforcement within 1,5d from column is accounted for with $f_{y,red} = 250 + 0{,}25d(\text{mm}) \leq f_{ywd}$



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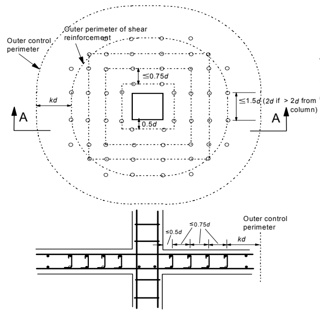
Punching shear reinforcement



The outer control perimeter at which shear reinforcement is not required, should be calculated from:

$u_{out,ef} = V_{Ed} / (v_{Rd,c}\, d)$

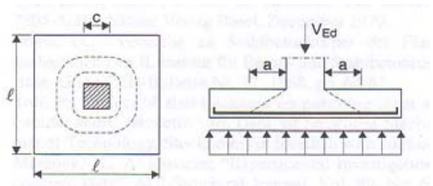
The outermost perimeter of shear reinforcement should be placed at a distance not greater than *kd* (*k* = 1.5) within the outer control perimeter.

Section A - A

TUDelft

Punching shear

- Column bases; critical parameters possible at a <2d
- $V_{Rd} = C_{Rd,c}\, k\, (100\rho f_{ck})^{1/3} \cdot 2d/a$



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Design with strut and tie models

Prof.dr.ir. J.C. Walraven

02 February 2008

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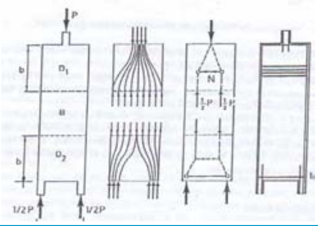
General idea behind strut and tie models

Structures can be subdivided into regions with a steady state of the stresses (B-regions, where "B" stands for "Bernoulii" and in regions with a nonlinear flow of stresses (D-regions, where "D" stands for "Discontinuity"



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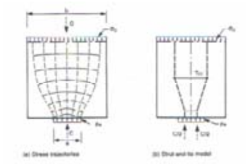
D-region: stress trajectories and strut and tie model



Steps in design:

1. Define geometry of D-region (Length of D-region is equal to maximum width of spread)
2. Sketch stress trajectories
3. Orient struts to compression trajectories
4. Find equilibrium model by adding tensile ties
5. Calculate tie forces
6. Calculate cross section of tie
7. Detail reinforcement

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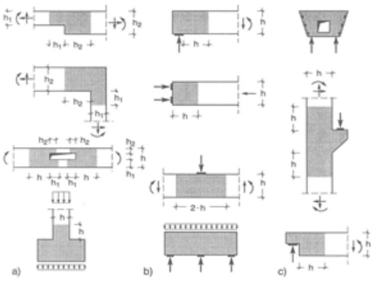
Examples of D-regions in structures



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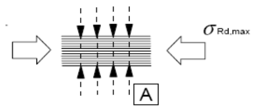
Design of struts, ties and nodes



A transverse compressive stress or no transverse stress

Struts with transverse compression stress or zero stress:

$\sigma_{Rd,max} = f_{cd}$

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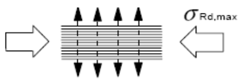
Design of struts, ties and nodes



Struts in cracked compression zones, with transverse tension

$\sigma_{Rd,max} = \upsilon f_{cd}$

Recommended value $\upsilon = 0{,}60\ (1 - f_{ck}/250)$

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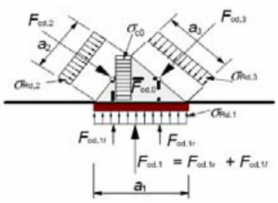
Design of struts, ties and nodes

Compression nodes without tie



$\sigma_{Rd,max} = k_1 \upsilon' f_{cd}$

where

$\upsilon' = 0{,}60\ (1 - f_{ck}/250)$

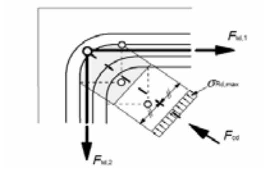
Recommended value

$K_1 = 1{,}0$

TUDelft

Design of struts, ties and nodes

Compression-Tension-Tension (CTT) node



$\sigma_{Rd,max} = k_3 \upsilon' f_{cd}$

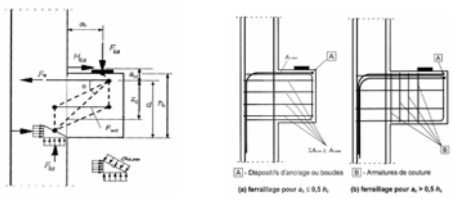
where

$\upsilon' = 0,60\ (1 - f_{ck}/250)$

Recommended value

$k_3 = 0,75$

Example of detailing based on strut and tie solution



Stress - strain relation for confined concrete (dotted line)

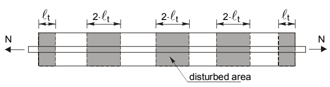
Crack width control in concrete structures

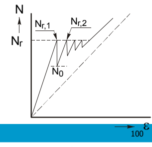
Prof.dr.ir. J.C. Walraven

02 February 2008

Theory of crack width control (4)

When more cracks occur, more disturbed regions are found in the concrete tensile bar. In the N-ε relation this stage (the "crack formation stage" is characterized by a "zig-zag"-line ($N_{r,1}$-$N_{r,2}$). At a certain strain of the bar, the disturbed areas start to overlap.
If no intermediate areas are left, the concrete cannot reach the tensile strength anymore, so that no new cracks can occur. The "crack formation stage" is ended and the stabilized cracking stage starts. No new cracks occur, but existing cracks widen.

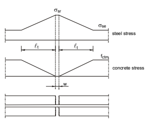




EC-formulae for crack width control (1)

For the calculation of the maximum (or characteristic) crack width, the difference between steel and concrete deformation has to be calculated for the largest crack distance, which is $s_{r,max} = 2l_t$. So

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) \qquad \text{Eq. (7.8)}$$



where
$s_{r,max}$ is the maximum crack distance
and
(ε_{sm} - ε_{cm}) is the difference in deformation between steel and concrete over the maximum crack distance.
Accurate formulations for $s_{r,max}$ and (ε_{sm} - ε_{cm}) will be given

EC-2 formulae for crack width control (2)

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0,6 \frac{\sigma_s}{E_s} \qquad \text{Eq. 7.0}$$

where: σ_s is the stress in the steel assuming a cracked section
α_e is the ratio E_s/E_{cm}
$\rho_{p,eff} = (A_s + \xi A_p)/A_{c,eff}$ (effective reinforcement ratio including eventual prestressing steel A_p
ξ is bond factor for prestressing strands or wires
k_t is a factor depending on the duration of loading (0,6 for short and 0,4 for long term loading)

EC-3 formulae for crack width control (4)

Maximum final crack spacing $s_{r,max}$

$$s_{r,max} = 3.4c + 0.425\, k_1 k_2 \frac{\phi}{\rho_{p,eff}} \qquad \text{(Eq. 7.11)}$$

where c is the concrete cover
Φ is the bar diameter
k_1 bond factor (0,8 for high bond bars, 1,6 for bars with an effectively plain surface (e.g. prestressing tendons)
k_2 strain distribution coefficient (1,0 for tension and 0,5 for bending: intermediate values van be used)

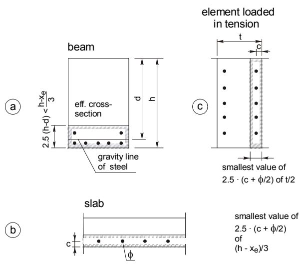
EC-2 requirements for crack width control (recommended values)

Exposure class	RC or unbonded PSC members	Prestressed members with bonded tendons
	Quasi-permanent load	Frequent load
X0,XC1	0.3	0.2
XC2,XC3,XC4	0.3	
XD1,XD2,XS1,XS2,XS3		Decompression

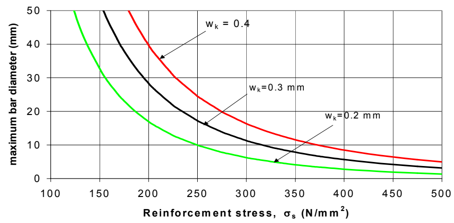
EC-2 formulae for crack width control (5)

In order to be able to apply the crack width formulae, basically valid for a concrete tensile bar, to a structure loaded in bending, a definition of the "effective tensile bar height" is necessary. The effective height $h_{c,ef}$ is the minimum of:

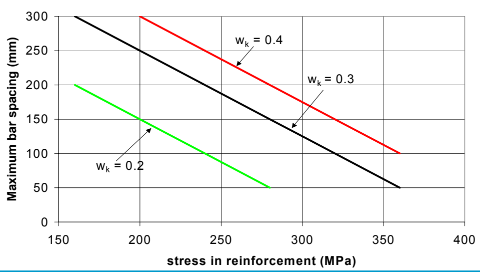
2,5 (h-d)
(h-x)/3
h/2



Maximum bar diameters for crack control (simplified approach 7.3.3)



Maximum bar spacing for crack control (simplified approach 7.3.3)

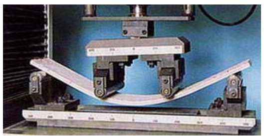


Deformation of concrete structures

Prof.dr.ir. J.C. Walraven
02 February 2008

Deformation of concrete

Reason to worry or challenge for the future?



Deflection of ECC specimen, V. Li, University of Michigan

Damage in masonry wall due to excessive deflection of lintel

Reasons for controling deflections (1)

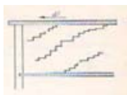
Appearance

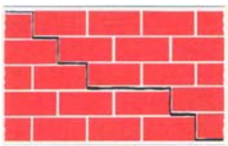
Deflections of such a magnitude that members appear visibly to sag will upset the owners or occupiers of structures. It is generally accepted that a deflection larger than span/250 should be avoided from the appearance point of view. A survey of structures in Germany that had given rise to complaints produced 50 examples. The measured sag was less than span/250 in only two of these.

Reasons for controling deflections (2)

Damage to non-structural Members

An important consequence of excessive deformation is damage to non structural members, like partition walls. Since partition walls are unreinforced and brittle, cracks can be large (several millimeters). The most commonly specified limit deflection is span/500, for deflection occurring after construction of the partitions. It should be assumed that all quasi permanent loading starts at the same time.





Reasons for controling deflections (3)

Collapse

In recent years many cases of collapse of flat roofs have been noted. If the rainwater pipes have a too low capacity, often caused by pollution and finally stoppage, the roof deflects more and more under the weight of the water and finally collapses. This occurs predominantly with light roofs. Concrete roofs are less susceptible for this type of damage

EC-2 Control of deflections

Deflection limits according to chapter 7.4.1

- Under the quasi permanent load the deflection should not exceed span/250, in order to avoid impairment of appearance and general utility

- Under the quasi permanent loads the deflection should be limited to span/500 after construction to avoid damage to adjacent parts of the structure

EC-2: SLS - Control of deflections

Control of deflection can be done in two ways

- By calculation
- By tabulated values

Calculating the deflection of a concrete member

The deflection follows from:

$\delta = \zeta\, \delta_{II} + (1 - \zeta)\delta_{I}$

δ deflection
δ_I deflection fully cracked
δ_{II} deflection uncracked
ζ coefficient for tension stiffening (transition coefficient)

$\zeta = 1 - \beta\, (\sigma_{sr}/\sigma_s)^2$

σ_{sr} steel stress at first cracking
σ_s steel stress at quasi permanent service load
β 1,0 for single short-term loading
0,5 for sustained loads or repeated loading

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Calculating the deflection of a concrete member

For pure bending the transition factor

$$\xi = 1 - \beta(\sigma_s / \sigma_r)^2$$

can as well be written as

$$\xi = 1 - \beta(M_{cr} / M)^2$$

where M_{cr} is the cracking moment and M is the applied moment

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Calculating the deflection of a concrete member

7.4.3 (7)

"The most rigorous method of assessing deflections using the method given before is to compute the curvatures at frequent locations along the member and then calculate the deflection by numerical integration.

In most cases it will be acceptable to compute the deflection twice, assuming the whole member to be in the uncracked and fully cracked condition in turn, and then interpolate using the expression:

$$\xi = 1 - \beta(M_{cr} / M)^2$$

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Cases where detailed calculation may be omitted

In order to simplify the design, expressions have been derived, giving limits of l/d for which no detailed calculation of the deflection has to be carried out.

These expressions are the results of an extended parameter analysis with the method of deflection calculation as given before. The slenderness limits have been determined with the criteria δ<L/250 for quasi permanent loads and δ<L/500 for the additional load after removing the formwork

The expressions, which will be given at the next sheet, have been calculated for an assumed steel stress of 310 MPa at midspan of the member. Where other stress levels are used, the values obtained by the expressions should be multiplied with $310/\sigma_s$

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Calculating the deflection of a concrete member

For span-depth ratios below the following limits no further checks is needed

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \le \rho_0 \qquad (7.16.a)$$

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if } \rho > \rho_0 \qquad (7.16.b)$$

l/d is the limit span/depth
K is the factor to take into account the different structural systems
ρ_0 is the reference reinforcement ratio = $\sqrt{f_{ck}}\, 10^{-3}$
ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)
ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

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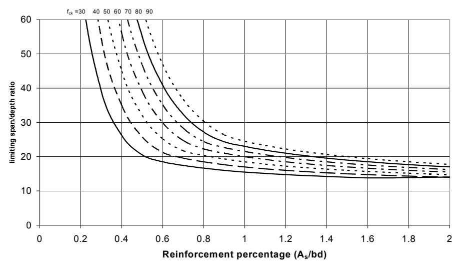
Previous expressions in a graphical form (Eq. 7.16):



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Lmit values for l/d below which no calculated verification of the deflection is necessary

The table below gives the values of K (Eq.7.16), corresponding to the structural system. The table furthermore gives limit l/d values for a relatively high (ρ=1,5%) and low (ρ=0,5%) longitudinal reinforcement ratio. These values are calculated for concrete C30 and σ_s = 310 MPa and satisfy the deflection limits given in 7.4.1 (4) and (5).

Structural system	K	ρ = 1,5%	ρ = 1,5%
Simply supported slab/beam	1,0	l/d=14	l/d=20
End span	1,3	l/d=18	l/d=26
Interior span	1,5	l/d=20	l/d=30
Flat slab	1,2	l/d=17	l/d=24
Cantilever	0,4	l/d= 6	l/d=8

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Bond and anchorage

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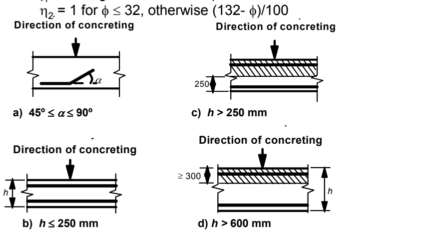
Ultimate Bond Stress, f_{bd} (8.4.2)

- The design value of the ultimate bond stress, f_{bd} = 2,25 $\eta_1\eta_2 f_{ctd}$ where f_{ctd} should be limited to C60/75
  - η_1 =1 for 'good' and 0,7 for 'poor' bond conditions
  - η_2 = 1 for $\phi \leq$ 32, otherwise (132- ϕ)/100



a) & b) 'good' bond conditions for all bars

c) & d) unhatched zone – 'good' bond conditions
hatched zone - 'poor' bond conditions

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Basic Required Anchorage Length, $l_{b,rqd}$ (8.4.3)

$$l_{b,rqd} = (\phi / 4)\ (\sigma_{sd} / f_{bd})$$

where σ_{sd} is the design stress of the bar at the position from where the anchorage is measured

- For bent bars $l_{b,rqd}$ should be measured along the centreline of the bar
- Where pairs of wires/bars form welded fabrics ϕ should be replaced by ϕ_n = $\phi\sqrt{2}$

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Design Anchorage Length, l_{bd} (8.4.4)

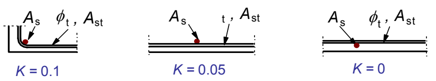
$$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd} \geq l_{b,min}$$

α_1 effect of bends — For straight bars α_1 = 1.0, otherwise 0.7

α_2 effect of concrete cover — α_2 = 1- 0.15(cover - ϕ)/ϕ $\geq$ 0.7 and $\leq$ 1.0

α_3 effect of confinement by transverse reinforcement (not welded)

α_3 = 1- $K\lambda$ $\geq$ 0.7 and $\leq$ 1.0 where λ = (ΣA_{st} - $\Sigma A_{st,min}$)/A_s



α_4 effect of confinement by welded transverse reinforcement — α_4 = 0.7

α_5 effect of confinement by transverse pressure

α_5 = 1 - 0.04p $\geq$ 0.7 and $\leq$ 1.0
where p is the transverse pressure (MPa) at ULS along l_{bd}

(α_2 α_3 α_5) $\geq$ 0.7 **$l_{b,min}$ > max(0.3l_b; 15ϕ, 100mm)**

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Design Lap Length, l_0 (8.7.3)

$$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$$

α_1 α_2 α_3 α_5 are as defined for anchorage length

α_6 = (ρ_1/25)^{0,5} but between 1,0 and 1,5
where ρ_1 is the % of reinforcement lapped within 0,65l_0 from the centre of the lap

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	>50%
α_6	1	1,15	1,4	1,5

Note: Intermediate values may be determined by interpolation.

$l_{0,min}$ $\geq$ max{0,3 α_6 $l_{b,rqd}$; 15ϕ; 200}

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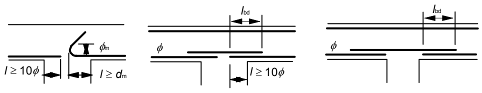
Anchorage of Bottom Reinforcement at Intermediate Supports

(9.2.1.5)

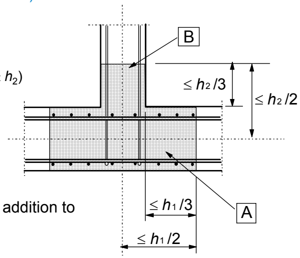


- Anchorage length, l, $\geq 10\phi$ for straight bars
  $\geq \phi_m$ for hooks and bends with $\phi \geq 16$mm
  $\geq 2\phi_m$ for hooks and bends with $\phi < 16$mm
- Continuity through the support may be required for robustness (Job specification)

Supporting Reinforcement at 'Indirect' Supports

(9.2.5)

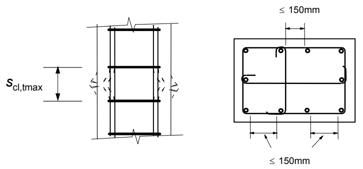
A	supporting beam with height h_1
B	supported beam with height h_2 ($h_1 \geq h_2$)



- The supporting reinforcement is in addition to that required for other reasons
- The supporting links may be placed in a zone beyond the intersection of beams

Columns (2)

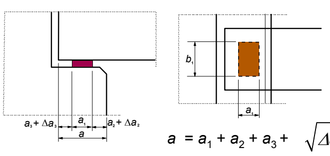
(9.5.3)



- $s_{cl,tmax}$ = 20 × ϕ_{min}; b; 400mm
- $s_{cl,tmax}$ should be reduced by a factor 0,6:
  - in sections within h above or below a beam or slab
  - near lapped joints where ϕ > 14. A minimum of 3 bars is rqd. in lap length

Additional rules for precast concrete

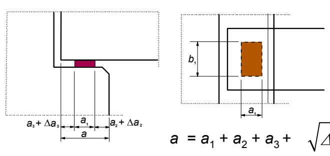
Bearing definitions (10.9.5)



$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^{\,2} + \Delta a_3^{\,2}}$$

a_1 net bearing length = F_{Ed} / (b_1 f_{Rd}), but $\geq$ min. value
  F_{Ed} design value of support reaction
  b_1 net bearing width
  f_{Rd} design value of bearing strength
a_2 distance assumed ineffective beyond outer end of supporting member
a_3 similar distance for supported member
Δa_2 allowance for tolerances for the distance between supporting members
Δa_3 = l_n/2500, l_n is length of member

Bearing definitions (10.9.5)

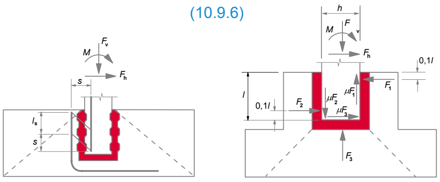


$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^{\,2} + \Delta a_3^{\,2}}$$

Minimum value of a_1 in mm

Relative bearing stress, σ_{Ed}/f_{cd}	$\leq$ 0,15	0,15 - 0,4	> 0,4
Line supports (floors, roofs)	25	30	40
Ribbed floors and purlins	55	70	80
Concentrated supports (beams)	90	110	140

Pocket foundations
(10.9.6)

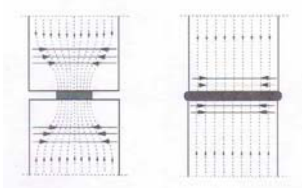


$l \leq s + l_s$ $\qquad$ $l \leq 1.2\,h$

Special attention should be paid to:

- shear resistance of column ends
- detailing of reinforcement for F_1 in top of pocket walls
- punching resistance of the footing slab under the column force

Connections transmitting compressive forces



Concentrated bearing $\qquad$ Soft bearing

For soft bearings, in the absence of a more accurate analysis, the reinforcement may be taken as:

$A_s = 0{,}25\ (t/h)\ F_{ed}/f_{yd}$

Where:
t = padding thickness
h = dimension of padding in direction of reinforcement
F_{ed} = design compressive force on connection

Lightweight aggregate concrete

Prof.dr.ir. J.C. Walraven

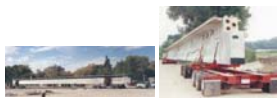
02 February 2008

Group Concrete Structures

Lightweight concrete structures in the USA

Nappa bridge
California 1977

Oronado bridge San Diego



52 m prestressed concrete beams, Lafayette USA

Rilem Standard test

Raftsundet Bridge, Norway

Antioch Bridge california

Qualification of lightweight aggregate concrete (LWAC)

Lightweight aggregate concrete is a concrete having a closed structure and an oven dry density of not more than 2200 kg/m³ consisting of or containing a proportion of artificial or natural lightweight aggregates having a density of less than 2000 kg/m³

Lightweight concrete density classification

Density classification

Density class		1,0	1,2	1,4	1,6	1,8	2,0
Oven dry density (kg/m³)		801-1000	1001-1200	1201-1400	1401-1600	1601-1800	1801-2000
Density (kg/m³)	Plain concrete	1050	1250	1450	1650	1850	2050
	Reinforced concrete	1150	1350	1550	1750	1950	2150

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Conversion factors for mechanical properties

The material properties of lightweight concrete are related to the corresponding properties of normal concrete. The following conversion factors are used:

η_E conversion factor for the calculation of the modulus of elasticity
η_1 coefficient for the determination of the tensile strength
η_2 coefficient for the determination of the creep coefficient
η_3 coefficient for the determination of the drying shrinkage
ρ oven-dry density of lightweight aggregate concrete in kg/m³

Antioch Bridge, California, 1977

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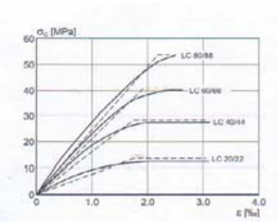
Design stress strain relations for LWAC

The design stress strain relations for LWAC differ in two respects from those for NDC.



- The advisory value for the strength is lower than for NDC (sustained loading factor 0,85 in stead of 1,0)
- The ultimate strain $\varepsilon_{l,cu}$ is reduced with a factor η_1=0,40+0,60ρ/2200

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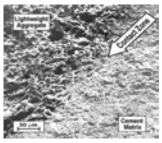
Shrinkage of LWAC

The drying shrinkage values for lightweight concrete (concrete class ≥ LC20/25) can be obtained by multiplying the values for normal density concrete for NDC with a factor η_3=1,2

The values for autogenous shrinkage of NDC represent a lower limit for those of LWAC, where no supply of water from the aggregate to the drying microstructure is possible. If water-saturated, or even partially saturated lightweight concrete is used, the autogenous shrinkage values will considerably be reduced (water stored in LWAC particles is extracted from aggregate particles into matrix, reducing the effect of self-dessication



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Shear capacity of LWAC members

The shear resistance of members without shear reinforcement is calculated by:

$$V_{lRd,ct} = \{(0{,}15/\gamma_c)\eta_1 k(100\rho_l f_{lck})^{1/3} + 0{,}15\sigma_{cp}]b_w d$$

where the factor η_1=0,40+0,60ρ/2200 is the only difference with the relation for NDC

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Punching shear resistance

Like in the case for shear of LWAC members, also the punching shear resistance of LWAC slab is obtained using the reduction factor η_1 = 0,4 + 0,6ρ/2200. the punching shear resistance of a lightweight concrete slab follows from:

$$V_{Rd,c} = (C_{lRd,c} k\eta_1 (100\rho_l f_{lck})^{1/3} + 0{,}08\sigma_{cp} \geq \eta_1 v_{l\min} + 0{,}08\sigma_{cp}$$

where $C_{lRd,c}$ = 0,15/γ_c

(in stead of the 0,18/γ_c for NDC)

TUDelft

Plain and lightly reinforced concrete

Prof.dr.ir. J.C. Walraven

02 February 2008

Group Concrete Structures

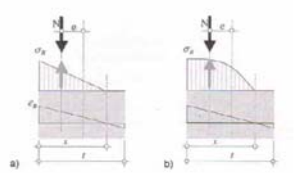
Field of application

Members for which the effect of dynamic action may be ignored

- Members mainly subjected to compression other than due to prestressing, e.g. walls, columns, arches, vaults and tunnels
- Strip and pad footings for foundations
- Retaining walls
- Piles whose diameter is ≥ 600mm and where $N_{ed}/A_c \leq 0{,}3f_{ck}$

Additional design assumptions

12.3.1 Due to the less ductile properties of plain concrete, the design values should be reduced. The advisory reduction factor is 0,8



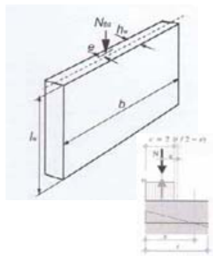
ULS: design resistance to bending and axial failure

The axial resistance N_{Rd}, of a rectangular cross-section with a uniaxial eccentricity e, in the direction of h_w, may be taken as:

$N_{Rd}=\eta f_{cd}\, bh(1\text{-}2e/h_w)$

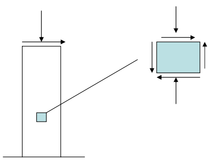
where

ηf_{cd} is the design compressive strength belonging to the block shaped stress-strain relation



Shear

12.6.3 (1): "In plain concrete members account may be taken of the concrete tensile strength in the ultimate limit state for shear, provided that either by calculation or by experience brittle failure can be excluded and adequate resistance can be ensured"



Using Mohr's circle it should be demonstrated that in nowhere in the structure the principal concrete tensile stress of the concrete exceeds the design tensile strength f_{ctk}

Simplified design method for walls and columns

In the absence of a more rigorous approach, the design resistance in terms of axial force slender wall or column in plain concrete may be calculated as follows:

$N_{Rd}=b \cdot h_w \cdot f_{cd} \cdot \phi$

where

N_{Rd} is the axial resistance
b is the overall width of the cross-section
h_w is the overall depth of the cross-section
ϕ is a factor taking account eccentricity, including second order effects

$\phi = 1{,}14 \cdot (1\text{-}2e_{tot}/h_w) - 0{,}02\ l_0/h_w \leq (1 - 2e_{tot}/h_w)$

EN1992-2

G. Mancini
Politecnico di Torino

EN 1992-2

EUROCODE 2 – Design of concrete structures
Concrete bridges: design and detailing rules

Approved by CEN on 25 April 2005
Published on October 2005
Supersedes ENV 1992-2:1996

Prof. Ing. Giuseppe Mancini
Politecnico di Torino

- EN 1992-2 contains principles and application rules for the design of bridges in addition to those stated in EN 1992-1-1
- Scope: basis for design of bridges in plain/reinforced/prestressed concrete made with normal/light weight aggregates

Section 3 ⇒ MATERIALS

- Recommended values for C_{min} and C_{max}

  C30/37 (Durability) C70/85 (Ductility)

- α_{cc} coefficient for long term effects and unfavourable effects resulting from the way the load is applied

  Recommended value: 0.85 → high stress values during construction

- Recommended classes for reinforcement: "B" and "C"

  (Ductility reduction with corrosion / Ductility for bending and shear mechanisms)

Section 4 ⇒ Durability and cover to reinforcement

- XC3 class recommended for surface protected by waterproofing

- When de-icing salt is used

  Exposed concrete surfaces within (6 m) of the carriage way and supports under expansion joints: directly affected by de-icing salt

  Recommended classes for surfaces directly affectd by de-icing salt: XD3 – XF2 – XF4, with covers given in tables 4.4N and 4.5N for XD classes

- Bare concrete decks without waterproofing or surfacing should be classified as abrasion class XM2

- When concrete surface is subject to abrasion by ice or solid transportation in running water → increase the cover by 10 mm, min

Section 5 ⇒ Structural analysis

- Geometrical imperfections

Piers

$$\vartheta_l = \vartheta_0\, \alpha_n$$

1/200 (recom.) $2\sqrt{l} \le 1$

Arches

Shape of imperfections based on the shape of first horizontal and vertical buckling mode, idealised by a sinusoidal profile having amplitude

$$a = \vartheta_l \frac{l}{2}$$

(l = half wavelength)

- Linear elastic analysis with limited redistributions

Limitation of δ due to uncertaintes on size effect and bending-shear interaction

$\delta \geq 0.85$ (recommended value)

- Plastic analysis

Restrictions due to uncertaintes on size effect and bending-shear interaction:

$$\frac{x_u}{d} \leq \begin{cases} 0.15 \text{ for concrete strength classes } \leq \text{C50/60} \\ 0.10 \text{ for concrete strength classes } \geq \text{C55/67} \end{cases}$$

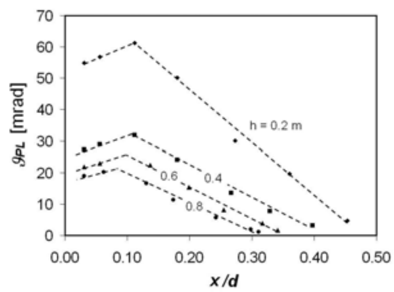
- Rotation capacity

Restrictions due to uncertaintes on size effect and bending-shear interaction:

in plastic hinges

$$\frac{x_u}{d} \leq \begin{cases} 0.30 \text{ for concrete strength classes } \leq \text{C50/60} \\ 0.23 \text{ for concrete strength classes } \geq \text{C55/67} \end{cases}$$

Numerical rotation capacity



- Nonlinear analysis $\Rightarrow$ Safety format

Reinforcing steel — Mean values: $1.1\ f_{yk}$, $1.1\ k\ f_{yk}$

Prestressing steel — Mean values: $1.1\ f_{pk}$

Concrete — Sargin modified mean values: $\gamma_{cf}\ f_{ck}$

$\gamma_{cf} = 1.1\ \gamma_s / \gamma_c$

- Design format
  - Incremental analysis from SLS, so to reach $\gamma_G\ G_k + \gamma_Q\ Q$ in the same step
  - Continuation of incremental procedure up to the peak strength of the structure, in corrispondance of ultimate load q_{ud}
  - Evaluation of structural strength by use of a global safety factor γ_0

$$R\left(\frac{q_{ud}}{\gamma_0}\right)$$

- Verification of one of the following inequalities

$$\gamma_{Rd} E\left(\gamma_G G + \gamma_Q Q\right) \le R\left(\frac{q_{ud}}{\gamma_O}\right)$$

$$E\left(\gamma_G G + \gamma_Q Q\right) \le R\left(\frac{q_{ud}}{\gamma_{Rd} \cdot \gamma_O}\right)$$

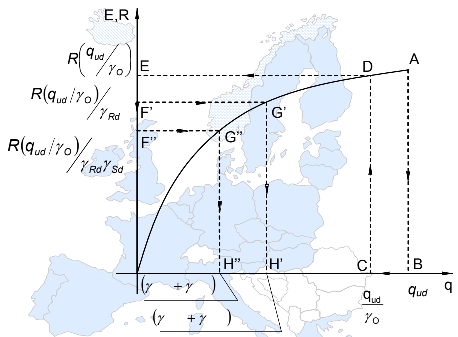
(i.e.) $R\left(\frac{q_{ud}}{\gamma_{O'}}\right)$

$$\gamma_{Rd}\gamma_{Sd} E\left(\gamma_g G + \gamma_q Q\right) \le R\left(\frac{q_{ud}}{\gamma_O}\right)$$

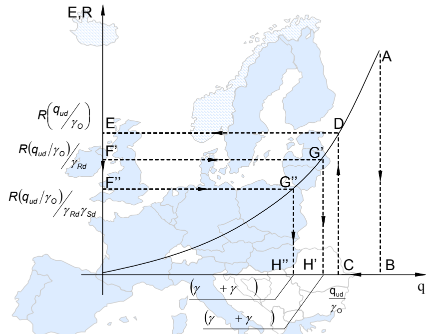
With
- γ_{Rd} = 1.06 partial factor for model uncertainties (resistence side)
- γ_{Sd} = 1.15 partial factor for model uncertainties (actions side)
- γ_0 = 1.20 structural safety factor

If γ_{Rd} = 1.00 then $\gamma_{0'}$ = 1.27 is the structural safety factor

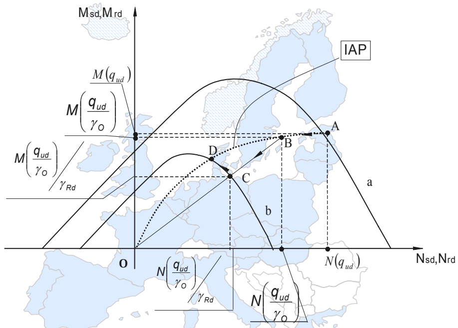
- Safety format *Application for scalar combination of internal actions and underproportional structural behaviour*



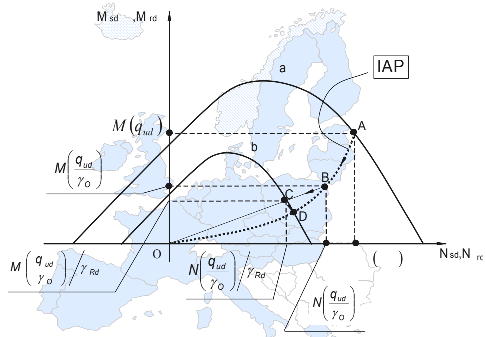
- Safety format *Application for scalar combination of internal actions and overproportional structural behaviour*



- Safety format *Application for vectorial combination of internal actions and underproportional structural behaviour*



- Safety format *Application for vectorial combination of internal actions and overproportional structural behaviour*



For vectorial combination and $\gamma_{Rd} = \gamma_{Sd} = 1.00$ the safety check is satisfied if:

$$M_{ED} \le M_{Rd}\left(\frac{q_{ud}}{\gamma_{0'}}\right)$$

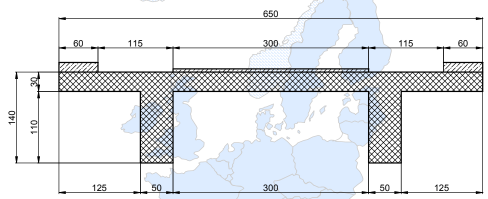
and

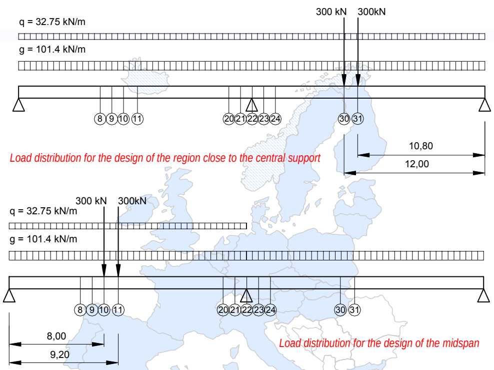
$$N_{ED} \le N_{Rd}\left(\frac{q_{ud}}{\gamma_{0'}}\right)$$

Example 1:

- Two spans R. C. bridge (l = 20 + 20 m)
- Advance shoring (20+5 m / 15 m)
- Dead load at t_0 = 28 days and t_1 = 90 days
- ξ (28, 90, ∞) = 0.51

N. L. analyses at t_1 (no redistribution due to creep)
t_∞ (full redistribution due to creep)





Incremental loading process

- Application of self weight in different statical schemes with γ_G = 1
- Modification of internal actions by creep by means of ξ function (γ_G = 1) only for t = t_∞
- Application of other permanent actions (γ_G = 1) on the final statical scheme
- Application of live loads with γ_G = 1
- Starting of incremental process so that γ_G = 1.4 and γ_Q = 1.5 is reached in the same step
- Continuation of incremental process up to attainment of peak load (Critical region: central support section)

Safety format : γ_{GI}

Load case (bending moments)	Time	γ_{Gu}	γ_{Qu}	γ_{Gu}/γ_{GI}	γ_{Qu}/γ_{GI}	$M(\gamma_{GI})^*$ [kNm*10³]	Gain
Maximum negative (X)	t_1	2.03	2.175	1.60	1.71	-12.2	14%
Maximum negative (Y)	∞	2.03	2.175	1.60	1.71	-12.2	14%
Maximum positive (W)	t_1	1.97	2.11	1.55	1.66	-10.4	11%
Maximum positive (Z)	∞	1.92	1.07	1.51	1.62	-10.1	7.8%

Where:

$$^*M(\gamma_{GI}) = M\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{GI}}\right)$$

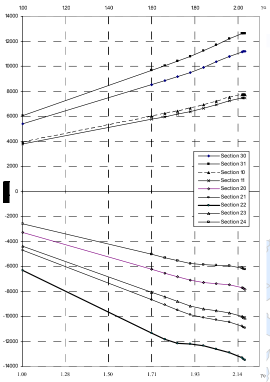
Safety format : γ_{gl}

Load case (bending moments)	Time	γ_{Gu}/γ_{gl}	$M(\gamma_{gl})^*$ [kNm*10³]	$M(\gamma_{gl})/\gamma_{Rd}$ [kNm*10³]	$\gamma_G(M(\gamma_{gl})/\gamma_{Rd})$	Gain Γ
Maximum negative (X)	t_1	1.69	-12.0	-11.3	1.60	14%
Maximum negative (Y)	∞	1.69	-12.2	-11.5	1.53	7.1%
Maximum positive (W)	t_1	1.64	-9.96	-9.40	1.55	11%
Maximum positive (Z)	∞	1.60	-10.6	-10.0	1.51	7.8%

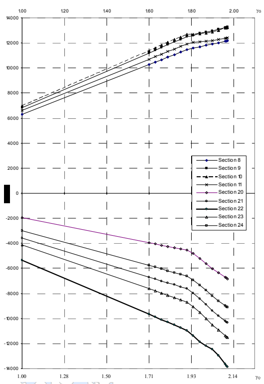
Where:

$$^*M(\gamma_{gl}) = M\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{gl}}\right)$$

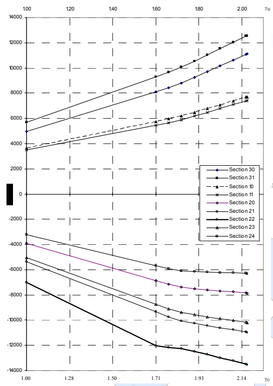
- Gain = $\dfrac{\gamma_{Gu}-1.4}{1.4} = \dfrac{\gamma_{Qu}-1.5}{1.5}$
- Critical section : number 22
- Reduction of gain by application of model uncertaintes only in case Y due to the increase of negative bending moment by creep and consequent translation of N.L. behaviour



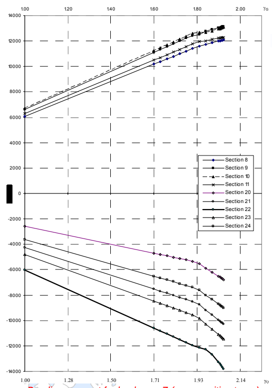
Bending moment for load case X (max. negative t = t_1)



Bending moment for load case W (max. positive t = t_1)



Bending moment for load case Y (max. negative t = ∞)

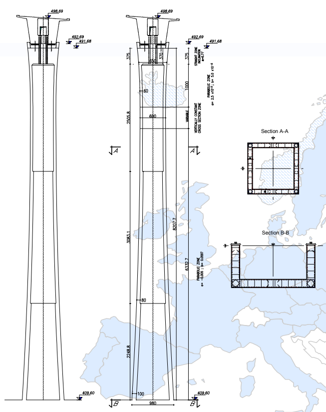


Bending moment for load case Z (max. positive t = ∞)

Example 2: Set of slender piers with variable section

- Depth: 82 / 87 / 92 / 97 m
- Unforeseen eccentricity: 5/1000 x depth
- $\gamma_G = \gamma_Q = 1.5$ (for semplification)
- Critical section at 53.30 m from plinth in which both thickness and reinforcement undergo a change

Safety format applied to that section



Pier geometry and reinforcement arrangement

Safety format : γ_{Gl}

Pier depth [m]	$\frac{\gamma_{Gu}}{\gamma_{Gl}}$	Critical section $N(\gamma_{Gl})^*$ [kN×10³]	Critical section $M(\gamma_{Gl})^{**}$ [kNm×10³]	Top section *N Safety* [kNm×10³]	Top section *M Safety* [kNm×10³]	$\gamma_{\sigma}(E(\gamma_{gl})/\gamma_{Rd})$	Gain Γ
82	2.45	151	242	134	74.9	2.42	62%
87	2.15	137	243	119	66.5	2.15	43%
92	1.85	122	233	103	57.6	1.86	24%
97	1.58	103	218	86	48.1	1.56	4%

Where:

$$^*N(\gamma_{Gl}) = N\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{Gl}}\right) = N\left(\frac{\gamma_{Gu}\cdot (G+Q)}{\gamma_{Gl}}\right)$$

$$^{**}M(\gamma_{Gl}) = M\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{Gl}}\right) = M\left(\frac{\gamma_{Gu}\cdot (G+Q)}{\gamma_{Gl}}\right)$$

Safety format : γ_{gl}

Pier depth [m]	$\frac{\gamma_{Gu}}{\gamma_{gl}}$	Critical section $N(\gamma_{gl})^*$ [kN×10³]	$M(\gamma_{gl})^{**}$ [kNm×10³]	$N(\gamma_{gl})/\gamma_{Rd}$ [kNm×10³]	$M(\gamma_{gl})/\gamma_{Rd}$ [kNm×10³]	*N Safety* [kNm×10³]	*M Safety* [kNm×10³]
82	2.59	159	260	150	245	152	243
87	2.28	146	262	138	247	139	246
92	1.96	129	251	122	237	123	236
97	1.67	110	234	104	221	104	220

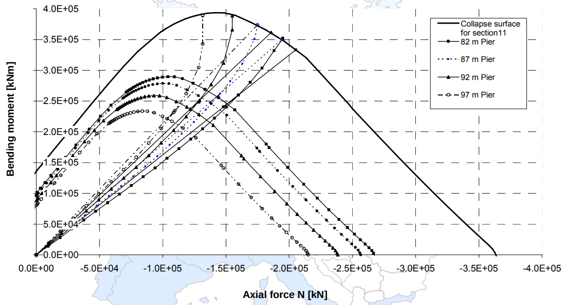
Where:

$$^*N(\gamma_{gl}) = N\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{gl}}\right) = N\left(\frac{\gamma_{Gu}\cdot (G+Q)}{\gamma_{gl}}\right)$$

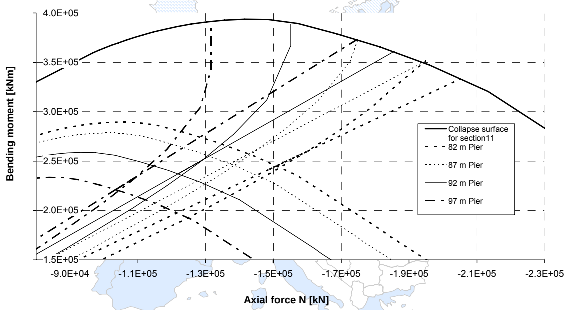
$$^{**}M(\gamma_{gl}) = M\left(\frac{\gamma_{Gu}\cdot G + \gamma_{Qu}\cdot Q}{\gamma_{gl}}\right) = M\left(\frac{\gamma_{Gu}\cdot (G+Q)}{\gamma_{gl}}\right)$$

Pier depth [m]	Top section *N Safety* [kNm*10³]	Top section *M Safety* [kNm*10³]	$\gamma_{\sigma}(E(\gamma_{gl})/\gamma_{Rd})$	Gain Γ
82	135	75.4	2.44	63%
87	121	67.6	2.19	46%
92	104	58.1	1.88	25%
97	87	48.6	1.57	5%

Safety format : γ_{gl}



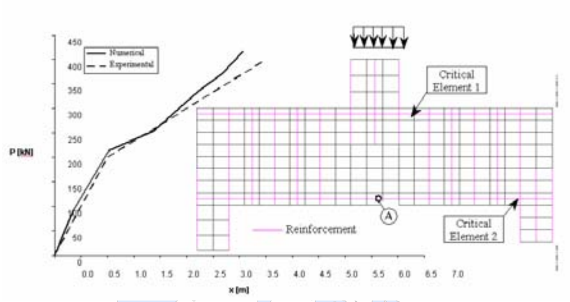
Safety format : γ_{gl} - enlargement of the most interesting region of the omothetic curves

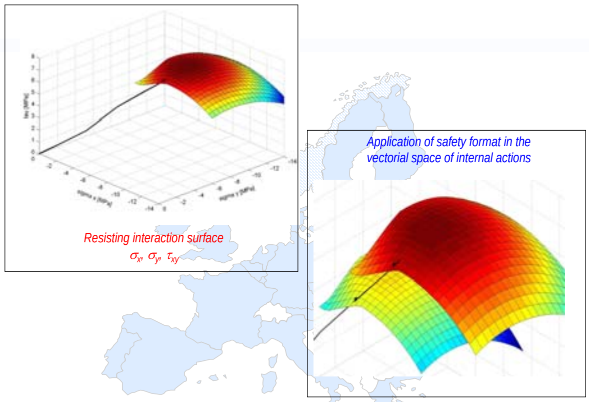


Example 3: Continous deep beam experimentally tested (Rogowsky, Mac Gregor, Ong)

- Adina N.L. code
- Concrete strength criterion by Carbone, Giordano, Mancini
- Peak load reached at the crushing of second element (model unable to reach the equilibrium for further load increments)

R/C deep beam: FE half mesh (right) load-displacement curve of point A (left)





Set of external and internal actions

- $q_{ud} = 404.0\,\text{kN}$, $\sigma_x = -8.47\,\text{MPa}$, $\sigma_y = -5.77\,\text{MPa}$, $\tau = 6.99\,\text{MPa}$
- $q_{max} = 319.5\,\text{kN}$, $\sigma_x = -6.82\,\text{MPa}$, $\sigma_y = -4.75\,\text{MPa}$, $\tau = 5.69\,\text{MPa}$

Behaviour with limited non linearity

Very limited effect of model uncertaintes

Section 6 ⇒ **Ultimate limit state (ULS)**

- Robustness criteria for prestressed structures

3 different approaches

a) Verification of load capacity with a reduced area of prestressing

- Evaluation of bending moment in frequent combination of actions: M_{freq}
- Reduction of prestressing up the reaching of f_{ctm} at the extreme tensed fibre, in presence of M_{freq}
- Evaluation of resisting bending moment M_{Rd} with reduced prestressing and check that:

$M_{Rd} > M_{freq}$

Redistributions can be applied

Material partial safety factors as for accidental combinations

b) Verification with nil residual prestressing

- Provide a minimum reinforcement so that

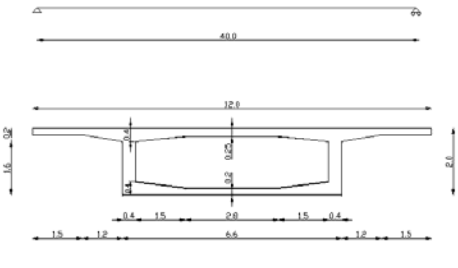
$$A_{s,\min} = \frac{M_{rep}}{z_s\, f_{yk}} \left(- \frac{A_p \cdot \Delta\sigma_p}{f_{yk}} \right)$$

$\Delta\sigma_p < 0.4\, f_{ptk}$ and 500 MPa

where M_{rep} is the cracking bending moment evaluated with f_{ctx} (f_{ctm} recommended)

c) Estabilish an appropriate inspection regime (External tendons!)

Simple supported span



Transverse section

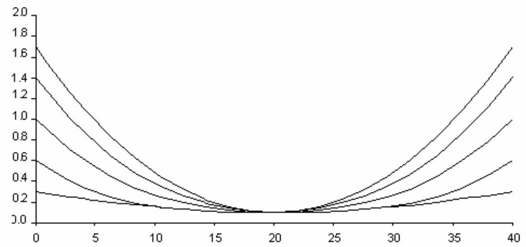
Materials:

- Concrete — f_{ck} = 30.0 Mpa; f_{ctm} = 2.0 Mpa; f_{cd} = 20 Mpa (γ_m=1.5)
- Ordinary reinforcement — f_{syk} = 430 Mpa; f_{syd} = 374 Mpa (γ_m=1.15)
- Prestressing steel — f_{ptk} = 1800 Mpa

Geometrical parameters:

A=6.055 m^2 J=3.59 m^4 y_G=1.21 m

Tendon layout



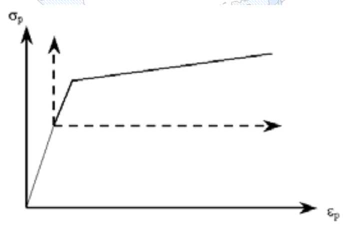
Brittle failure

1. Reduction of prestressing up to reaching of f_{ctm} at the extreme tensed fibre in presence of M_{freq}

Active tendons	0	1	2	3	4	5	6	7	8
N_{prec} [kN]	0.00	-2852.28	-5704.56	-8556.84	-11409.12	-14261.40	-17113.68	-19965.96	-22818.24
M_{prec} [kN m]	0.00	-3166.03	-6332.06	-9498.09	-12664.12	-15830.15	-18996.18	-22162.22	-25328.25
M_{dec} [kN m]	5933.88	10497.53	15061.17	19624.82	24188.46	28752.11	33315.75	37879.39	42443.04
σ_{inf} [MPa]	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
M_{freq} [kN m]	51933.5	48767.47	45601.438	42435.41	39269.38	36103.35	32937.32	29771.28	26605.25

In such condition add ordinary reinforcement so that $M_{Rd} \geq M_{freq}$, with γ_C = 1.3 and γ_S = 1.0

Take care of preelongation for the contribution of tendons to the evaluation of M_{Rd}



Such condition is reached for an addition of 1ϕ14 / 150 mm in the bottom slab and 1ϕ12 / 150 mm in the webs and top slab

2. Provide a minimum reinforcement evaluated as

$$A_{s,\min} = \frac{M_{rep}}{z_s \, f_{yk}}$$

M_{rep} = cracking moment evaluated with f_{ctm} and nil prestressing

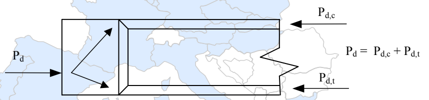
z_s = lever arm at USL = 1.62 m

The required ordinary reinforcement results 1ϕ12 / 150 mm in the bottom slab

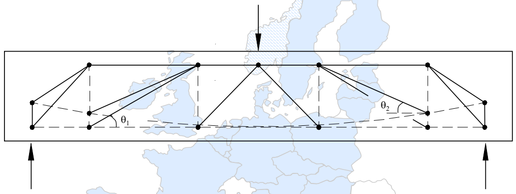
- Shear design of precast prestressed beams

- High level of prestress → $\sigma_{cp}/f_{cd} > 0.5$
- Thin webs
- End blocks
- Redundancy in compressed and tensed chords

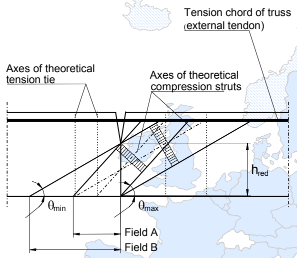
Web verification only for compression field due to shear ($\alpha_{cw} = 1$)



- Superimposition of different truss models



- Bending–shear behaviour of segmental precast bridges with external prestressing (only)



$$h_{red} = \frac{V_{Ed}}{b_w\,\nu\,f_{cd}}(\cot\theta + \tan\theta)$$

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{h_{red}\,f_{ywd}\,\cot\theta}$$

$h_{red,min} = 0.5\ h$ (recommended value)

Field A : arrangement of stirrups with θ_{max} (cot θ = 1.0)
Field B : arrangement of stirrups with θ_{min} (cot θ = 2.5)

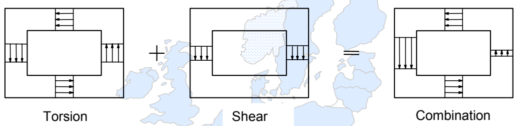
- Shear and transverse bending interaction

Web of box girder

Semplified procedure

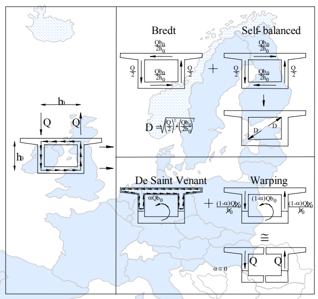
When $\frac{V_{Ed}}{V_{Rd,\max}} < 0.20$ or $\frac{M_{Ed}}{M_{Rd,\max}} < 0.10$ The interaction can be disregarded

- Combination of shear and torsion for box sections



Each wall should be designed separately

- Bending–shear-torsion behaviour of segmental precast bridges with external prestressing (only)



Design the shear keys so that circulatory torsion can be maintained !

- Fatigue

- Verification of concrete under compression or shear

Traffic data — S-N curves — Load models

National authorities

- λ values semplified approach (Annex NN, from ENV 1992-2)

- Application of Miner rule

$$\sum_{i=1}^{m} \frac{n_i}{N_i} \le 1$$

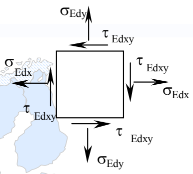
$N_i \Rightarrow$
- Given by national authorities (S-N curves)
- $N_i = 10\exp\left(14 \cdot \frac{1-E_{cd,\max,i}}{\sqrt{1-R_i}}\right)$

where: $R_i = \frac{E_{cd,\min,i}}{E_{cd,\max,i}}$; $E_{cd,\min,i} = \frac{\sigma_{cd,\min,i}}{f_{cd,\,fat}}$; $E_{cd,\max,i} = \frac{\sigma_{cd,\max,i}}{f_{cd,\,fat}}$

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd}\left(1 - \frac{f_{ck}}{250}\right)$$

K_1 =0.85 (Recommended value)

- Membrane elements



- Compressive stress field strength defined as a function of principal stresses
- If both principal stresses are comprensive

$$\sigma_{cd\,\max} = 0.85 f_{cd} \frac{1+3{,}80\alpha}{(1+\alpha)^2}$$

is the ratio between the two principal stresses ($\alpha \le 1$)

- Where a plastic analysis has been carried out with $\theta = \theta_{el}$ and at least one principal stress is in tension and no reinforcement yields

$$\sigma_{cd\,\max} = f_{cd}\left[0{,}85 - \frac{\sigma_s}{f_{yd}}(0{,}85-\nu)\right]$$

σ_s is the maximum tensile stress value in the reinforcement

- Where a plastic analysis is carried out with yielding of any reinforcement

$$\sigma_{cd\,\max} = \nu\, f_{cd}\left(1 - 0{,}032\left|\theta - \theta_{el}\right|\right)$$

θ is the angle to the X axis of plastic compression field at ULS (principal compressive stress)

θ_{el} is the inclination to the X axis of principal compressive stress in the elastic analysis

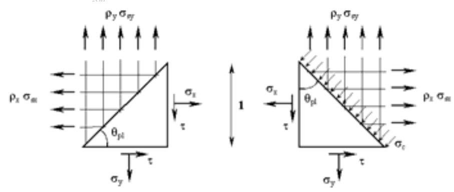
$|\theta - \theta_{el}| \le 15$ degrees

Model by Carbone, Giordano, Mancini

Assumption: strength of concrete subjected to biaxial stresses is correlated to the angular deviation between angle ϑ_{el} which identifies the principal compressive stresses in incipient cracking and angle ϑ_u which identifies the inclination of compression stress field in concrete at ULS

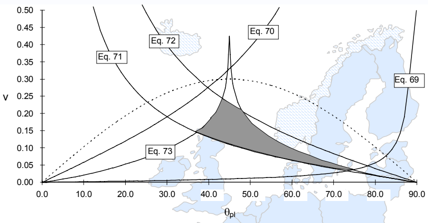
With increasing $\Delta\vartheta$ concrete damage increases progressively and strength is reduced accordingly

Plastic equilibrium condition



$$\sigma_x + \tau\cot\vartheta_{pl} - \sigma_{sx}\rho_x = 0$$
$$\tau + \sigma_x\cot\vartheta_{pl} - \sigma_{sy}\rho_y\cot\vartheta_{pl} = 0$$
$$\tau\tan\vartheta_{pl} - \sigma_x + \sigma_{sx}\rho_x - \sigma_c = 0$$
$$\tau - \sigma_y\tan\vartheta_{pl} + \sigma_{sy}\rho_y\tan\vartheta_{pl} - \sigma_c\tan\vartheta_{pl} = 0$$

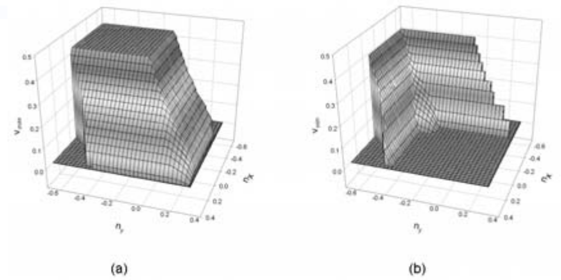
Graphical solution of inequalities system



$\omega_x = 0.16$
$\omega_y = 0.06$
$n_x = n_y = -0.17$
$\vartheta_{el} = 45°$

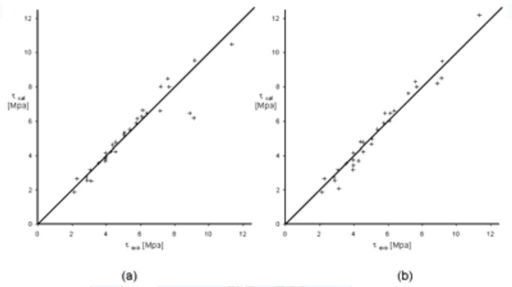
$$\frac{\tau}{|f_c'|}\left(\tan\vartheta_{pl} + \cot\vartheta_{pl}\right) - \left[0.55 - 0.12\ln\left|\vartheta_{pl} - \vartheta_{el}\right|\right] = 0$$

$$v \ge -(\omega_x + n_x)\tan\vartheta_{pl} \quad (69)$$
$$v \le (\omega_x - n_x)\tan\vartheta_{pl} \quad (70)$$
$$v \ge (-\omega_y + n_y)\cot\vartheta_{pl} \quad (71)$$
$$v \le (\omega_y - n_y)\cot\vartheta_{pl} \quad (72)$$
$$v \le \nu\sin\vartheta_{pl}\cos\vartheta_{pl} \quad (73)$$



(a) (b)

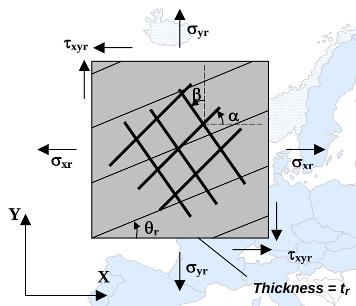
Resisting domain for v_{max} (a) and v_{min} (b) with ϑ_{el}=45°, ω_x=ω_y=0.3



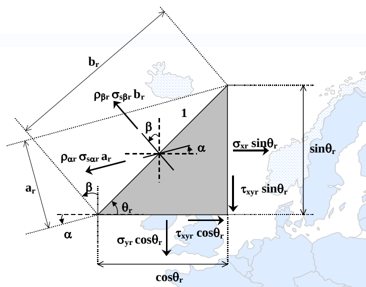
(a) (b)

Experimental versus calculated panel strenght by Marti and Kaufmann (a) and by Carbone, Giordano and Mancini (b)

Skew reinforcement



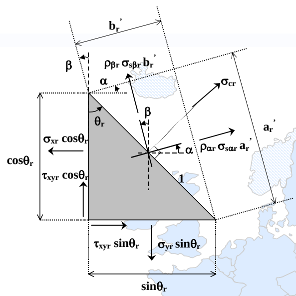
Plates conventions



Equilibrium of the section parallel to the compression field

$$\rho_{\alpha r}\sigma_{s\alpha r} = \frac{\sigma_{xr}\sin\theta_r\cos\beta - \sigma_{yr}\cos\theta_r\sin\beta + \tau_{xyr}\cos(\theta_r + \beta)}{\sin(\theta_r - \alpha)\cos(\alpha - \beta)}$$

$$\rho_{\beta r}\sigma_{s\beta r} = \frac{\sigma_{xr}\sin\theta_r\sin\alpha + \sigma_{yr}\cos\theta_r\cos\alpha + \tau_{xyr}\sin(\theta_r + \alpha)}{\cos(\theta_r - \beta)\cos(\alpha - \beta)}$$



Equilibrium of the section orthogonal to the compression field

$$-\sigma_{xr}\cos\theta_r + \tau_{xyr}\sin\theta_r + \rho_{\alpha r}\sigma_{s\alpha r}a'_r\cos\alpha - \rho_{\beta r}\sigma_{s\beta r}b'_r\sin\beta + \sigma_{cr}\cos\theta_r = 0$$
$$-\sigma_{yr}\sin\theta_r + \tau_{xyr}\cos\theta_r + \rho_{\alpha r}\sigma_{s\alpha r}a'_r\sin\alpha - \rho_{\beta r}\sigma_{s\beta r}b'_r\cos\beta + \sigma_{cr}\sin\theta_r = 0$$

Use of genetic algorithms (Genecop III) for the optimization of reinforcement and concrete verification

Objective: minimization of global reinforcement

Stability: find correct results also if the starting point is very far from the actual solution

Section 7 ⇒ Serviceability limit state (SLS)

- Compressive stresses limited to $k_1 f_{ck}$ with exposure classes XD, XF, XS (Microcracking)

$k_1 = 0.6$ (recommended value)

$k_1 = 0.66$ in confined concrete (recommemded value)

- Crack control

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,3[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XD3 XS1, XS2, XS3		Decompression

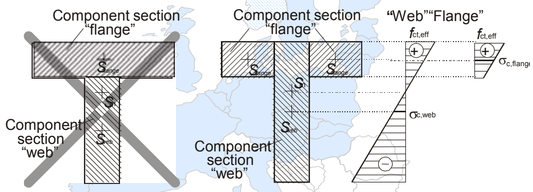
Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

Decompression requires that concrete is in compression within a distance of 100 mm (recommended value) from bondend tendons

- Minimum reinforcement areas

Clarification about T and Box beams



- Control of shear cracks within the webs

Concrete tensile strength f_{ctb} is:

$$f_{ctb} = \left(1 - 0,8\frac{\sigma_3}{f_{ck}}\right) f_{ctk,0.05}$$

σ_3 is the larger compressive principal stress ($\sigma_3 > 0$ and $\sigma_3 < 0.6\, f_{ck}$)

- The larger tensile principal stress σ_1 is compared with f_{ctb}

$\frac{\sigma_1}{f_{ctb}}$ < 1 ⇒ minimum longitudinal reinforcement

$\frac{\sigma_1}{f_{ctb}}$ ≥ 1 ⇒ crack width controlled or calculated considering the skewness of reinforcement

Section 8 ⇒ Detailing of reinforcement and prestressing tendons

- Couplers for prestressing tendons
- In the same section maximum 67% of coupled tendons
- For more than 50% of coupled tendons:

Continous minimum reinforcement

or

Residual stress > 3 MPa in characteristic combination

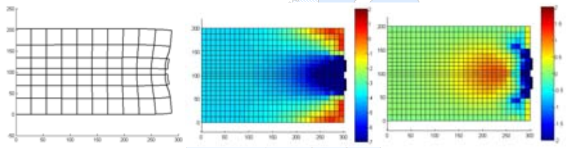
- Minimum distance of sections in which couplers are used

Construction depth h	Distance a
≤ 1,5 m	1,5 m
1,5 m < h < 3,0 m	a = h
≥ 3,0 m	3,0 m

- For tendons anchored at a construction joint a minimum residual compressive stress of 3 MPa is required under the frequent combination of actions, otherwise reinforcement should be provided to carter for the local tension behind the anchor

Baricentric prestressing, two coupled tendons over two

t = 14 gg

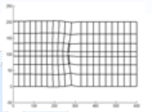


deformation σ_x σ_y

Baricentric prestressing

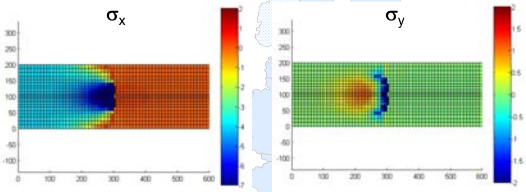
two coupled tendons over two

t = 41 gg



deformation

σ_x σ_y



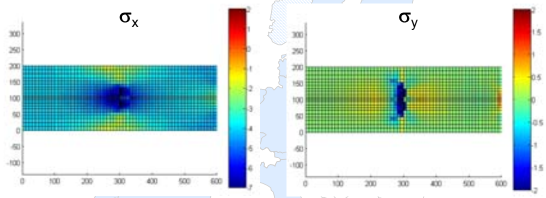
Baricentric prestressing

two coupled tendons over two

t = 42 gg

deformation

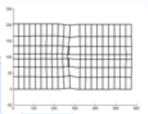
σ_x σ_y



Baricentric prestressing

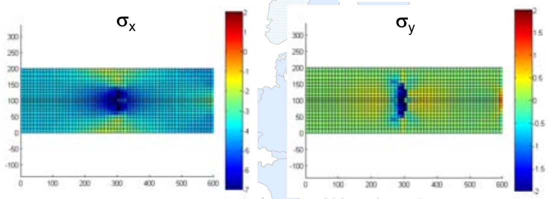
two coupled tendons over two

t = 70 gg



deformation

σ_x σ_y



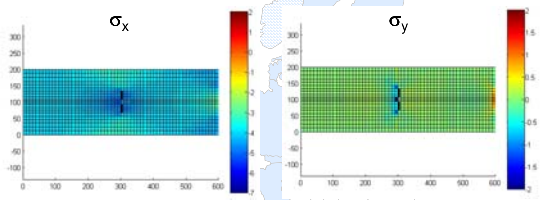
Baricentric prestressing

two coupled tendons over two

t = ∞

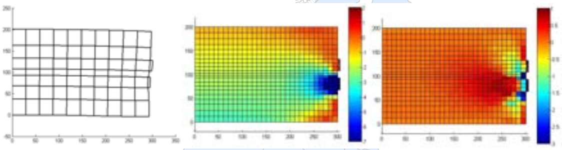
deformation

σ_x σ_y



Baricentric prestressing, one coupled tendon over two

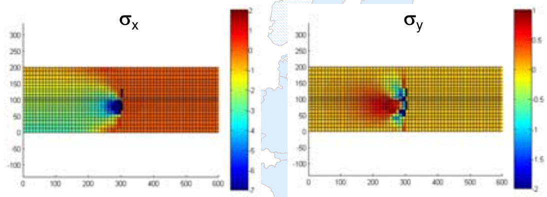
t = 14 gg



Baricentric prestressing

one coupled tendons over two

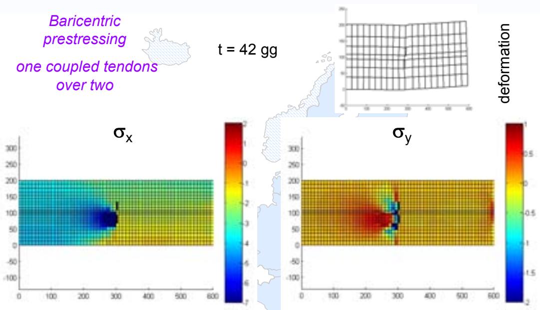
t = 41 gg



Baricentric prestressing

one coupled tendons over two

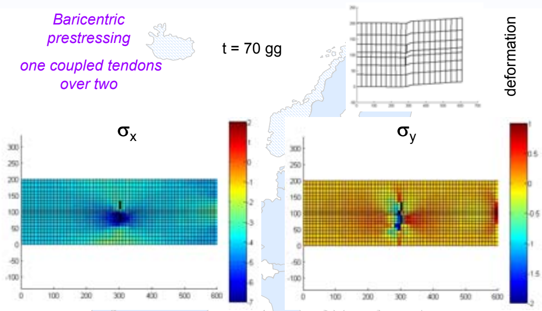
t = 42 gg



Baricentric prestressing

one coupled tendons over two

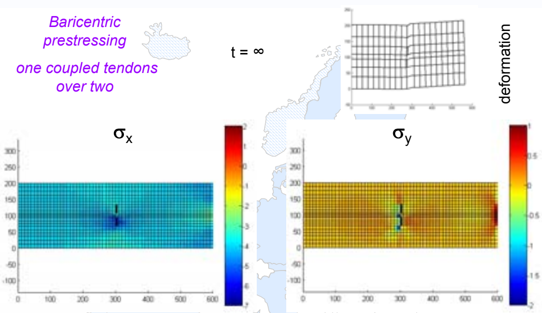
t = 70 gg



Baricentric prestressing

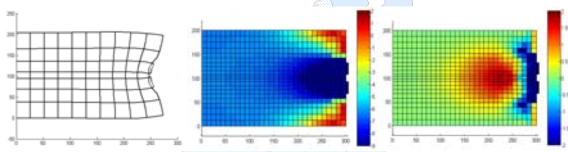
one coupled tendons over two

t = ∞



Baricentric prestressing, two anchored tendons over two

t = 14 gg

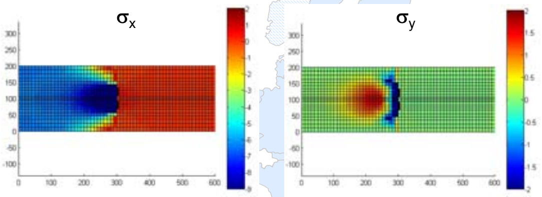


Baricentric prestressing

two anchored tendons over two

t = 41 gg

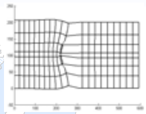
deformation



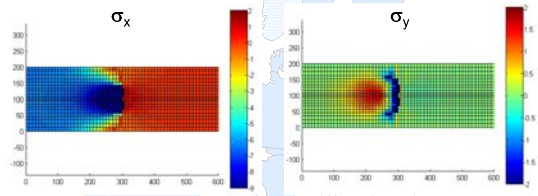
Baricentric prestressing

two anchored tendons over two

t = 70 gg



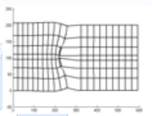
deformation



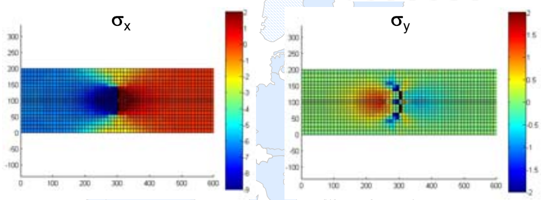
Baricentric prestressing

two anchored tendons over two

t = ∞



deformation

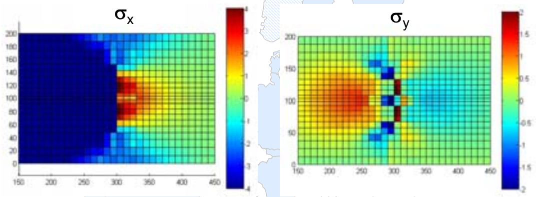


Baricentric prestressing

two anchored tendons over two

t = ∞

Zoomed areas near anchorages



Section 113 ⇒ Design for the execution stages

Take account of construction procedure

- *Construction stages*
- *Redistribution by creep in the section*
- *Redistribution by creep for variation of statical scheme*

- Actions during execution → Cross reference to EN1991-1-6

- Statical equilibrium of cantilever bridge → unbalanced wind pressure of 200 N/m² (recommended value)
- For cantilever construction { *Fall of formwork* / *Fall of one segment* }
- For incremental launching → Imposed deformations!
- In case in SLS decompression is required, tensile stresses less then f_{ctm} (recommended value) are permitted during the construction in quasi-permanent combination of actions

Annex B ⇒ Creep and shrinkage strain

- HPC, class R cement, strength ≥ 50/60 MPa with or without silica fume
- Thick members → kinetic of basic creep and drying creep is different
- Distiction between
  - ***Autogenous shrinkage:*** *related to process of hydratation*
  - ***Drying shrinkage:*** *related to humidity exchanges*
- Specific formulae for SFC (content > 5% of cement by weight)

- Autogenous shrinkage

- For t < 28 days $f_{ctm}(t) / f_{ck}$ is the main variable

$$\frac{f_{cm}(t)}{f_{ck}} < 0.1 \qquad \varepsilon_{ca}(t, f_{ck}) = 0$$

$$\frac{f_{cm}(t)}{f_{ck}} \geq 0.1 \qquad \varepsilon_{ca}(t, f_{ck}) = (f_{ck} - 20)\left(2.2\frac{f_{cm}(t)}{f_{ck}} - 0.2\right)10^{-6}$$

- For t ≥ 28 days

$$\varepsilon_{ca}(t, f_{ck}) = (f_{ck} - 20)\left[2.8 - 1.1\exp(-t/96)\right]10^{-6}$$

97% of total autogenous shrinkage occurs within 3 mounths

- Drying shrinkage (RH ≤ 80%)

$$\varepsilon_{cd}(t, t_s, f_{ck}, h_0, RH) = \frac{\mathrm{K}(f_{ck})\left[72\exp(-0.046 f_{ck}) + 75 - RH\right](t - t_s)10^{-6}}{(t - t_s) + \beta_{cd}\, h_0^{\,2}}$$

with: $K(f_{ck}) = 18$ if $f_{ck} \leq 55$ MPa

$K(f_{ck}) = 30 - 0.21 f_{ck}$ if $f_{ck} > 55$ MPa

$$\beta_{cd} = \begin{cases} 0.007 & \textit{for silica-fume concrete} \\ 0.021 & \textit{for non silica-fume concrete} \end{cases}$$

- Creep

$$\varepsilon_{cc}(t, t_0) = \frac{\sigma(t_0)}{E_{c28}}\left[\Phi_b(t, t_0) + \Phi_d(t, t_0)\right]$$

Basic creep — Drying creep

- Basic creep

$$\Phi_b\left(t, t_0, f_{ck}, f_{cm}(t_0)\right) = \phi_{b0}\frac{\sqrt{t - t_0}}{\left[\sqrt{t - t_0} + \beta_{bc}\right]}$$

with:

$$\phi_{b0} = \begin{cases} \dfrac{3.6}{f_{cm}(t_0)^{0,37}} & \text{for silica-fume concrete} \\ 1.4 & \text{for non silica-fume concrete} \end{cases}$$

$$\beta_{bc} = \begin{cases} 0.37\exp\left(2.8\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for silica-fume concrete} \\ 0.4\exp\left(3.1\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for non silica-fume concrete} \end{cases}$$

- Drying creep

$$\Phi_d(t, t_s, t_0, f_{ck}, RH, h_0) = \phi_{d0}\left[\varepsilon_{cd}(t, t_s) - \varepsilon_{cd}(t_0, t_s)\right]$$

with:

$$\phi_{d0} = \begin{cases} 1000 & \text{for silica-fume concrete} \\ 3200 & \text{for non silica-fume concrete} \end{cases}$$

- Experimental identification procedure

At least 6 months

- Long term delayed strain estimation

Formulae | Experimental determination

- Safety factor for long term extrapolation γ_{lt}

t (age of concrete for estimating the delayed strains)	γ_{lt}
t < 1 year	1
t = 5 years	1,07
t = 10 years	1,1
t = 50 years	1,17
t = 100 years	1,20
t = 300 years	1,25

Annex J ⇒ Detailing rules for particular situations

Bearing zones of bridges

- Consideration of brittleness of HSC with a factor to be applied to f_{cd}

$$\frac{0,46.f_{ck}^{2/3}}{1+0,1.f_{ck}}\cdot f_{cd} \quad (\leq 1)$$

- Edge sliding

$A_S \cdot f_{yd} \geq F_{Rdu} / 2$

$\vartheta = 30°$

- Anchorage zones of postensioned members

Bursting and spalling in anchorage zones controlled by reinforcement evaluated in relation to the primary regularisation prism

$$\frac{P_{max}}{c \cdot c'} \leq 0,6 \cdot f_{ck}(t)$$

where c,c' are the dimensions of the associate rectangle

↓

similar to anchorage plate

$$\left.\begin{matrix} c/a \\ c'/a' \end{matrix}\right\} \leq 1,25\sqrt{\frac{c \cdot c'}{a \cdot a'}}$$

being a,a' the dimensions of smallest rectangle including anchorage plate

Primary regularisation prism represents the volume in which the stresses reduce from very high values to acceptable values under uniaxial compression

The depth of the prism is 1.2 max(c,c')

Reinforcement for bursting and spalling (distributed in each direction within the prism)

$$A_S = 0,15\frac{P_{max}}{f_{yd}}\gamma_{P,unf} \quad \text{(with } \gamma_{P,unf} = 1.20\text{)}$$

Surface reinforcement at the loaded face

$$A_{surf} \geq 0,03\frac{P_{max}}{f_{yd}}\gamma_{P,unf} \quad \text{(in each direction)}$$

Annex KK ⇒ Structural effects of time dependent behaviour of concrete

Assumptions

- *Creep and shrinkage independent of each other*
- *Average values for creep and shrinkage within the section*
- *Validity of principle of superposition (Mc-Henry)*

Type of analysis	Comment and typical application
General and incremental step-by-step method	These are general methods and are applicable to all structures. Particularly useful for verification at intermediate stages of construction in structures in which properties vary along the length (e.g.) cantilever construction.
Methods based on the theorems of linear viscoelasticity	Applicable to homogeneous structures with rigid restraints.
The ageing coefficient method	This method will be useful when only the long -term distribution of forces and stresses are required. Applicable to bridges with composite sections (precast beams and in-situ concrete slabs).
Simplified ageing coefficient method	Applicable to structures that undergo changes in support conditions (e.g.) span-to- span or free cantilever construction.

- General method

$$\varepsilon_c(t) = \frac{\sigma_0}{E_c(t_0)} + \varphi(t,t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\varphi(t,t_i)}{E_c(28)}\right)\Delta\sigma(t_i) + \varepsilon_{cs}(t,t_s)$$

A step by step analysis is required

- Incremental method

- At the time t of application of σ the creep strain $\varepsilon_{cc}(t)$, the potential creep strain $\varepsilon_{\infty cc}(t)$ and the creep rate are derived from the whole load history

- The potential creep strain at time t is:

$$\frac{d\varepsilon_{\infty cc}(t)}{dt} = \frac{d\sigma}{dt}\frac{\varphi(\infty,t)}{E_{c28}}$$

- $t \Rightarrow t_e$

under constant stress from t_e the same $\varepsilon_{cc}(t)$ and $\varepsilon_{\infty cc}(t)$ are obtained

$$\varepsilon_{\infty cc}(t)\cdot\beta_c(t,t_e) = \varepsilon_{cc}(t)$$

- Creep rate at time t may be evaluated using the creep curve for t_e

$$\frac{d\varepsilon_{cc}(t)}{dt} = \varepsilon_{\infty cc}(t)\frac{\partial\beta_c(t,t_e)}{\partial t}$$

- For unloading procedures

$|\varepsilon_{cc}(t)| > |\varepsilon_{\infty cc}(t)|$

and t_e accounts for the sign change

$$\varepsilon_{ccMax}(t) - \varepsilon_{cc}(t) = \left(\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)\right)\cdot\beta_c(t,t_e)$$

$$\frac{d\left(\varepsilon_{ccMax}(t) - \varepsilon_{cc}(t)\right)}{dt} = \left(\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)\right)\cdot\frac{\partial\beta_c(t,t_e)}{\partial t}$$

where $\varepsilon_{ccMax}(t)$ is the last extreme creep strain reached before t

- Application of theorems of linear viscoelasticity

- $J(t,t_0)$ an $R(t,t_0)$ fully characterize the dependent properties of concrete
- Structures homogeneous, elastic, with rigid restraints
- Direct actions effect

$$S(t) = S_{el}(t)$$

$$D(t) = E_C\int_0^t J(t,\tau)\,dD_{el}(\tau)$$

- Indirect action effect

$$D(t) = D_{el}(t)$$

$$S(t) = \frac{1}{E_C}\int_0^t R(t,\tau)\,dS_{el}(\tau)$$

- Structure subjected to imposed constant loads whose initial statical scheme (1) is modified into the final scheme (2) by introduction of additional restraints at time $t_1 \geq t_0$

$$S_2(t) = S_{el,1} + \xi(t,t_0,t_1)\Delta S_{el,1}$$

$$\xi(t,t_0,t_1) = \int_{t_1}^{t} R(t,\tau)\,dJ(\tau,t_0)$$

$$\xi(t,t_0,t_0^+) = 1 - \frac{R(t,t_0)}{E_C(t_0)}$$

- When additional restraints are introduced at different times $t_i \geq t_0$, the stress variation by effect of restrain j introduced at t_j is indipendent of the history of restraints added at $t_i < t_j$

$$S_{j+1} = S_{el,1} + \sum_{i=1}^{j} \xi\left(t, t_0, t_i\right) \Delta S_{el,i}$$

- Ageing coefficient method

Integration in a single step and correction by means of χ ($\chi \cong 0.8$)

$$\int_{\tau=t_0}^{t} \left[\frac{E_c(28)}{E_c(\tau)} + \varphi_{28}(t,\tau) \right] d\sigma(\tau) = \left[\frac{E_c(28)}{E_c(t_0)} + \chi(t,t_0)\varphi_{28}(t,t_0) \right] \Delta\sigma_{t_0 \to t}$$

- Simplified formulae

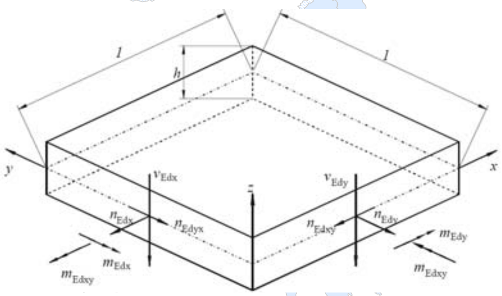
$$S_{\infty} = S_0 + \left(S_1 - S_0\right) \frac{\varphi(\infty, t_0) - \varphi(t_1, t_0)}{1 + \chi\varphi(\infty, t_1)} \frac{E_c(t_1)}{E_c(t_0)}$$

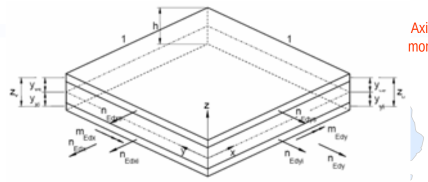
where: S_0 and S_1 refer respectively to construction and final statical scheme

t_1 is the age at the restraints variation

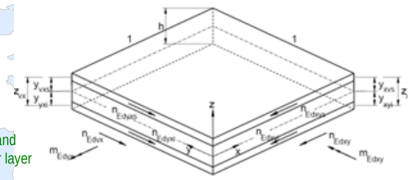
Annex LL ⇒ Concrete shell elements

A powerfull tool to design 2D elements





Axial actions and bending moments in the outer layer



Membrane shear actions and twisting moments in the outer layer

RANTIVA BRIDGE

Sandwich model: Numerical example

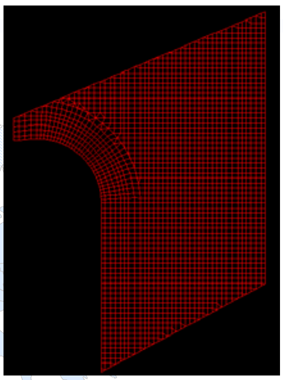
Mesh

2215 shell elements

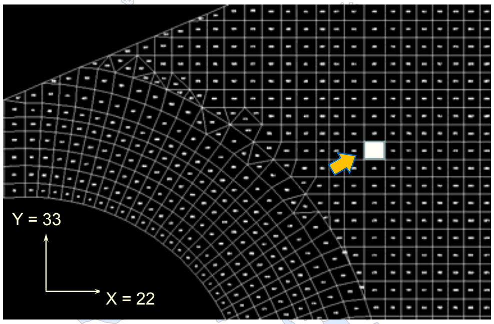
2285 nodes

6 D.o.F. per node

13710 D.o.F. in total



Element chosen: n°682



Symbols, conventions and general data

$\alpha = 0 \Rightarrow$ transverse reinforcement, A_{sx}, direction 22

$\beta = 0 \Rightarrow$ longitudinal reinforcement, A_{sy}, direction 33

Concrete properties f_{cd} = 20.75 MPa

f_{ctm} = 3.16 MPa

f_{ctd} = 1.38 MPa

Steel properties f_{yd} = 373.9 MPa

Dimensioning of α reinforcement (transverse) in the inferior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Max M33	277	-5134	-230	616	1121	-476	95	-212

Load combination that maximizes this reinforcement

H sez. (m)	Layers thicknesses tsup (m)	tinf (m)
1.0000	0.23	0.18

Increment of internal actions due to shear (for the single layer) nsd22 (KN/m)	nsd33 (KN/m)	nsd23 (KN/m)
0	0	0

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22 (KN/m)	**nsd33** (KN/m)	**nsd23** (KN/m)	case (-)	θ (°)	νf_{cd} (N/mm²)	$\sigma_c(f)$ (N/mm²)	$n_{R1(x)}$ (kN/m)	$n_{R2(y)}$ (kN/m)	**A_s(x)nec** (cm²/m)	**A_s(y)nec** (cm²/m)
-633	-4070	481	no.	65.0	17.6	17.6	0.0	0.0	15.7	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Lower layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22 (KN/m)	**nsd33** (KN/m)	**nsd23** (KN/m)	case (-)	θ (°)	νf_{cd} (N/mm²)	$\sigma_c(f)$ (N/mm²)	$n_{R1(x)}$ (kN/m)	$n_{R2(y)}$ (kN/m)	**A_s(x)nec** (cm²/m)	**A_s(y)nec** (cm²/m)
909	-1064	-711	yes	23.1	11.1	11.1	1212.0	607.2	31.3	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Dimensioning of β reinforcement (longitudinal) in the inferior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Max M22	261	-5134	-219	657	1014	-464	79	-197

Load combination that maximizes this reinforcement

	Layers thicknesses	
H sez. (m)	tsup (m)	tinf (m)
1.0000	0.23	0.19

Increment of internal actions due to shear (for the single layer)		
nsd22 (KN/m)	nsd33 (KN/m)	nsd23 (KN/m)
0	0	0

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	nsd33	nsd23	case	θ	νf_{cd}	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	A_s(x)nec	A_s(y)nec
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
-695	-3904	474	no	45.0	17.6	17.6	0.0	0.0	15.7	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Lower layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	nsd33	nsd23	case	θ	νf_{cd}	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	A_s(x)nec	A_s(y)nec
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
956	-1229	-693	yes	20.6	11.1	11.1	1216.7	611.5	31.3	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Dimensioning of
α reinforcement (transverse)
in the superior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Max M22	261	-5134	-219	657	1014	-464	79	-197

Load combination that maximizes this reinforcement

	Layers thicknesses	
H sez. (m)	tsup (m)	tinf (m)
1.0000	0.23	0.19

Increment of internal actions due to shear (for the single layer)		
nsd22 (KN/m)	nsd33 (KN/m)	nsd23 (KN/m)
0	0	0

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	nsd33	nsd23	case	θ	νf_{cd}	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	A_s(x)nec	A_s(y)nec
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
-695	-3904	474	no	45.0	17.6	17.6	0.0	0.0	15.7	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Lower layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	nsd33	nsd23	case	θ	ν fcd	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	A_s(x)nec	A_s(y)nec
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
956	-1229	-693	yes	20.6	11.1	11.1	1216.7	611.5	31.3	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Dimensioning of
β reinforcement (longitudinal)
in the superior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Max M22	261	-5134	-219	657	1014	-464	79	-197

Load combination that maximizes this reinforcement

	Layers thicknesses	
H sez.	**tsup**	**tinf**
(m)	(m)	(m)
1.0000	0.23	0.19

Increment of internal actions due to shear (for the single layer)		
nsd22	**nsd33**	**nsd23**
(KN/m)	(KN/m)	(KN/m)
0	0	0

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
-695	-3904	474	no	45.0	17.6	17.6	0.0	0.0	15.7	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

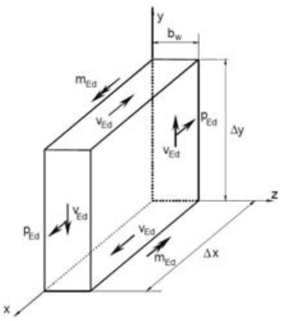
Lower layer verification

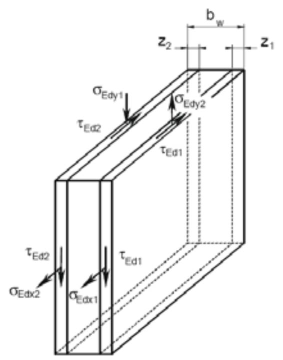
Internal actions on the layer			Cracked ?	Concrete parameters			Actions in reinforcement at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	$\sigma_c(f)$	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
956	-1229	-693	yes	20.6	11.1	11.1	1216.7	611.5	31.3	15.7

Minimum reinforcement ϕ 20/20 = 15.7 cm²/m

Annex MM ⇒ Shear and transverse bending

Webs of box girder bridges

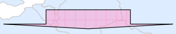




Modified sandwich model

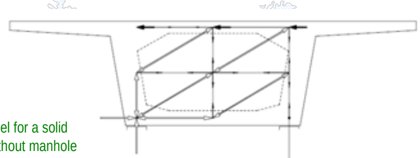
Annex NN ⇒ Damage equivalent stresses for fatigue verification

Unchanged with respect to ENV 1992-2

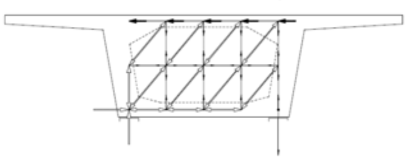


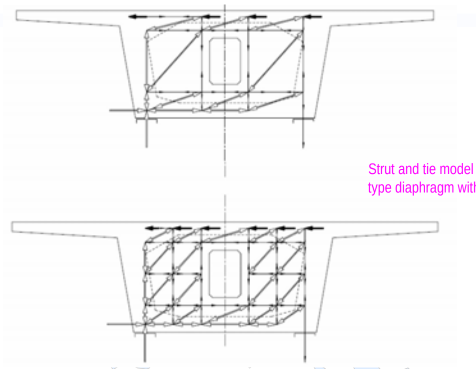
To be used only for simple cases

Annex OO ⇒ Typical bridge discontinuity regions



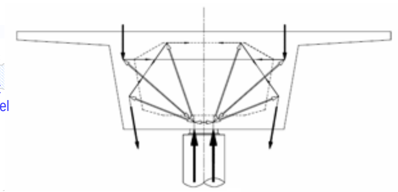
Strut and tie model for a solid type diaphragm without manhole

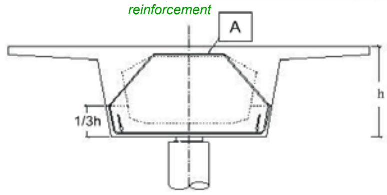




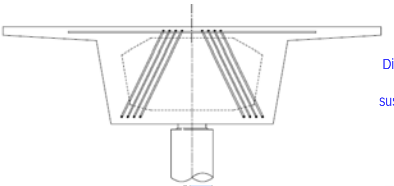
Strut and tie model for a solid type diaphragm with manhole

Diaphragms with indirect support. Strut and tie model



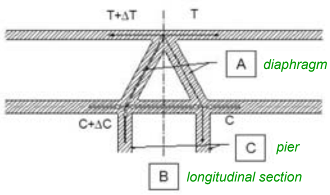


Diaphragms with indirect support. Anchorage of the suspension reinforcement

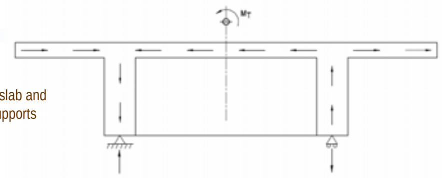


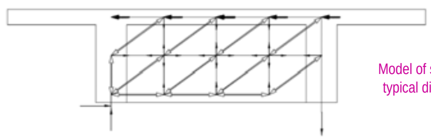
Diaphragms with indirect support. Links as suspension reinforcement

Diaphragm in monolithic joint with double diaphragm: Equivalent system of struts and ties.



Torsion in the deck slab and reactions in the supports





Model of struts and ties for a typical diaphragm of a slab

EN 1992-2 ⇒ A new design code to help in conceiving more and more enhanced concrete bridges

Thank you for the kind attention

EN1992-3

T. Jones
Arup

Eurocode 2 – Design of Concrete Structures – Part 3 : Liquid retaining and containment structures

Dr Tony Jones
Arup

Introduction

Project Team

- Convenor Prof. Andrew Beeby, UK.
- Andrea Benedetti,Italy
- Prof K van Breugel, Netherlands
- Dr Dieter Pichler, Austria
- Dr Karl-Heinz Reineck, Germany
- M Grenier, France.

Task to Convert part 4 of the ENV to part 3 of EN1992.

Why do we have part 3?

- Very few specific items
- Some manipulation of part 1 equations
- Some other rules that more correctly belong in part 1.
- Aim for next version to be included in part 1

Scope of EN1992-3

- Additional rules for …… the containment of liquids or granular solids
- Only for those parts that directly support the stored materials
- Stored materials at -40°C to +200°C
- “clauses covering liquid tightness may also be relevant to other types of structure”

Excludes

- Storage of materials at very high or low temperatures.
- Storage of materials leakage of which would constitute a major health risk.
- Pressurised vessels
- Floating structures
- Large dams
- Gas tightness

Changes to Part 1

- Background to why changes to part 1 are required
- Some background to their basis.

Basic Design Variables

Special design situations

- Operating conditions
- Explosions
- Temperature of Stored materials
- Testing

- Reference to EN1991-4 for Actions

Materials

Concrete

- Effect of temperature on Material Properties (including creep) – Annex K
- Thermal Coefficient of Expansion – warning on variability.

Reinforcement

- Reference to EN1992-1-2 for temperatures >100°C

Durability

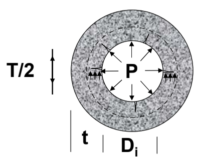
Abrasion due to:

- Mechanical Attack
- Physical Attack
- Chemical Attack

Analysis

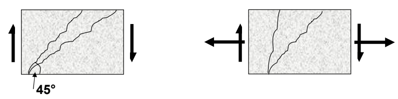
- Consideration of temperature effects (gradients)
- Consideration of internal pressures
  - Solids at the surface
  - Liquids at the centre line

$T=P(D_i+t)$



Ultimate Limit State

Shear under tension – Cot θ conservatively limited to 1.0



Note EN1992-1 limit of Cot θ for tension flanges = 1.25 – could have been a general rule

Design for dust explosion

Basic guidance given in EN 1991-4 and EN 1991-1-7

But TG thought that more helpful information should be provided:

- Venting and protection of surroundings
- Actions considered acidental
- Combination with other actions (part filled bins)
- Need for specialist assistance

Serviceability

Table 7.105 — Classification of tightness

Tightness Class	Requirements for leakage
0	Some degree of leakage acceptable, or leakage of liquids irrelevant.
1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable.
2	Leakage to be minimal. Appearance not to be impaired by staining.
3	No leakage permitted

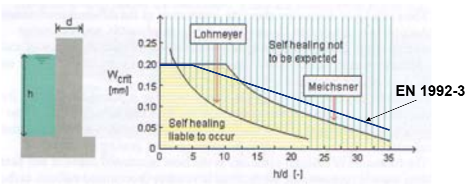
Tightness Class 0. — the provisions in 7.3.1 of EN 1992-1-1 may be adopted.

Tightness Class 1. — any cracks which can be expected to pass through the full thickness of the section should be limited to w_{k1}. The provisions in 7.3.1 of EN 1992-1-1 apply where the full thickness of the section is not cracked and where the conditions in (112) and (113) below are fulfilled.

Tightness Class 2. — cracks which may be expected to pass through the full thickness of the section should generally be avoided unless appropriate measures (e.g. liners or water bars) have been incorporated.

Tightness Class 3. — generally, special measures (e.g. liners or prestress) will be required to ensure watertightness.

Tightness Class 1 – Through Cracking



Original Diagram from Walraven

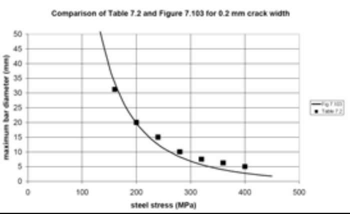
– Cracks may be expected to heal when range of strain under service conditions is less than 150 x 10^{-6}

Tightness Class 2

Minimum depth of compression zone (or section that remains in compression lesser of 50mm or 0.2h under quasi permanent loads

Control of cracking without direct calculation

Revised figures for maximum bar spacing/bar stress given – These are as Part 1 except for Tension rather than Flexure



[Note: This also means the bar diameter modification (exp 7.7N) is modified slightly]

Calculation of crack width – refer to Annexes L and M

Minimising cracking due to restraint

- Limit temperature rise
- Reduce restraints
- Use concrete with low thermal expansion
- Use concrete with high tensile strain capacity
- Apply prestress.

Detailing

Guidance on:
- Postensioning of circular tanks
- Minimum wall thicknesses in prestressed tanks
- Temperature effects on unbonded tendons
- Opening moments in the corners of tanks
- Provision of movement joints.

Annexes (all informative)

Annex K – Effect of temperature on the properties of concrete
Annex L – Calculation of strians and stresses in concrete sections subjected to restrained imposed deformations
Annex M – Calculation of crack widths due to restraint of imposed deformations
Annex N – Provision of movement joints

Annex K – Effect of temperature in the properties of concrete.

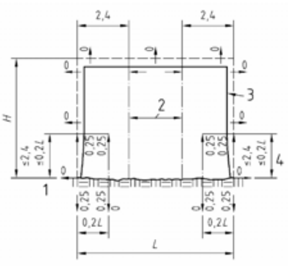
- Material enhancements given for sub zero temperatures – not always conservative to ignore.
- For elevated temperatures reference to fire part, to avoid duplication.
- Methods presented to calculate increased creep (and transitional thermal strain) and reduced elastic modulus.

Annex L – Calculation of the strains and stresses in concrete sections subjected to restrained imposed deformations

Actual strain $\varepsilon_{az} = (1-R_{ax})\varepsilon_{i_{av}}$

Stress in concrete $\sigma_z = E_{c,eff}\,(\varepsilon_{i_{av}} - \varepsilon_{az})$

Restraint Factor



(a) Wall on base

Annex M – Calculation of crack widths due to restraint of imposed deformations

Two case considered:

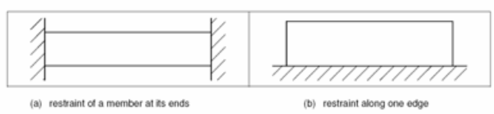
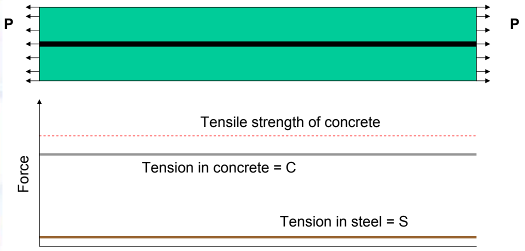


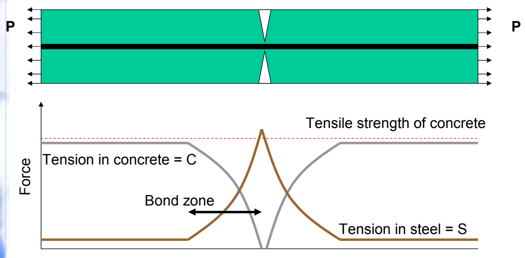
Figure M.1 — Types of restraint to walls

End Restraint



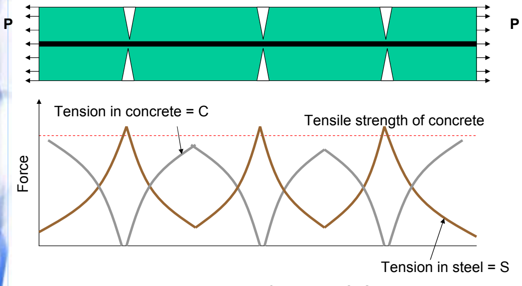
At any point along the element force = P = C+S

End Restraint



At any point along the element force = P = C+S

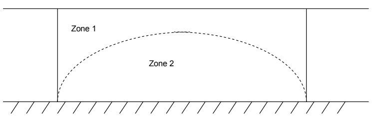
End Restraint



At any point along the element force = P = C+S

$\varepsilon_{sm}-\varepsilon_{cm}=0,5\alpha_e k_c k f_{ct,eff}(1+1/(\alpha_e\rho))/E_s$

Edge Restraint



Zone 1 end restraint
Zone 2 edge restraint

Edge Restraint

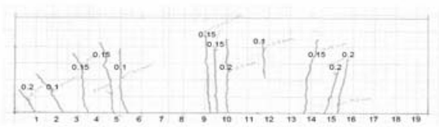


Figure 2 *Crack pattern observed in a 19.5m long wall*

From Bamforth

$\varepsilon_{sm} - \varepsilon_{cm} = R_{ax}\,\varepsilon_{free}$

Annex N – Provision of Movement Joints

Table N.1 — Design of joints for the control of cracking

Option	Method of control	Movement joint spacing	Reinforcement
(a)	continuous – full restraint	Generally no joints, though some widely spaced joints may be desirable where a substantial imposed deformation (temperature or shrinkage) is expected.	Reinforcement in accordance with Chapters 6 and 7.3
(b)	Close movement joints – minimum restraint	Complete joints at greater of 5 m or 1.5 times wall height	Reinforcement in accordance with Chapter 6 but not less than minimum given in 9.6.2 to 9.6.4.

National Choices

- Definition of w_{k1} (crack width limit for tightness class 1 structures)
- X_{min} depth of section to remain in compression for tightness class 2 structures
- κ maximum duct size related to wall thikness
- t_1 and t_2 minimum wall thicknesses for class 0 and class 1 or 2 structures respectively.

Summary

- Relatively short document
- Most of what is in the main code could be handled in Part 1
- There is useful information in the Annexes which are all informative to allow local interpretation as appropriate.