Opening - Welcome
International
DIANA Users Meeting

Ane de Boer
DIANA Users Association &
Ministry of Infrastructure, The Netherlands

Why Gothenburg-Sweden after

London-UK, Nijmegen–NL, Essen-Germany, Porto-Portugal, Trondheim-Norway, Delft-NL, Brescia-Italy







- **DIANA Users in Sweden**
- **Local arrangements: Lundgren and Hanjari**

International DIANA Users Meeting
Chalmers University of Technology
Gothenburg, Sweden

Gothenburg well-known city

Volvo's hometown & Factories
Ferry link to Danmark
Chalmers University of Technology
Sport events like: Indoor European Championship Athletics
Speed skating track

International DIANA Users Meeting
Chalmers University of Technology
Gothenburg, Sweden

Activities Users Association in cooperation with TNO DIANA BV

•Development meetings:
Since 1984, the foundation year of the Users Association
-General meetings at the Users offices in The Netherlands,
including Technical meetings in the Netherlands twice a year since 2000
-International Users Meeting since London 2004
- Users Association open for International Members since 2005,
so join us, a registration form at the registration desk of this Meeting!
-Activities:
In cooperation with CUR-NL: Concrete Mechanics Applications 2003 [CD],
Advanced FE analyses (CUR2003-1;NL) & Design in 3D(2008-1;NL)
•Wish Lists: International User Wish List since User Meeting 2005
•DIANA User Advisory Board: short, mid and longterm wishes
•Set-up of a User Database of articles, presentations around DIANA
NLFEA Guideline: Girders published 18 April 2013!!

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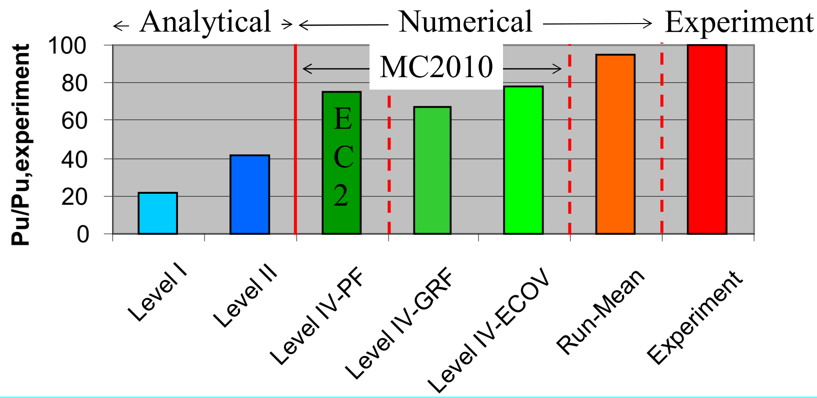
NLFEA Guidelines / Safety Formats EC2-MC2010

Safety Formats Slabexperiments Stevinlab; Framcos8 - Belletti etc.



Henrik Schlune: Chalmers PhD Thesis Alternative Safety Formats

International DIANA Users Meeting
Chalmers University of Technology
Gothenburg, Sweden

Location of this meeting

- **Hosting this meeting:**
  Chalmers University of Technology
  Concrete Structures, Division of Structural Engineering
  Chalmers Teknik Park

International DIANA Users Meeting
Chalmers University of Technology
Gothenburg, Sweden

Social Event

17:00 City / Harbour guided Tour
start: Teknik Park
finish: Tvåkanten

19:30 Dinner Restaurang Tvåkanten

Opportunity meeting TNO DIANA BV people

- Wijtze Pieter Kikstra, Scientific Employee
- Gerd-Jan Schreppers, General Director

Opportunity meeting DIANA Users Association people

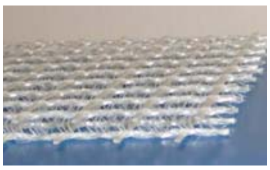
- Nynke Vollema
- Henco Burggraaf
- Ane de Boer

Opportunity to meet other DIANA Users !!

International DIANA Users Meeting
Chalmers University of Technology
Gothenburg, Sweden

Research at Concrete Structures

- Existing structures
  - Use of FEM in design and assessment of bridges
  - Structural effects of reinforcement corrosion
  - Multi-scale modelling of chloride ingress
- New reinforcement types
  - Fibre reinforced concrete
  - Textile reinforced concrete
- Civil defence shelters
  - Concrete structures subjected to blast and fragment impacts



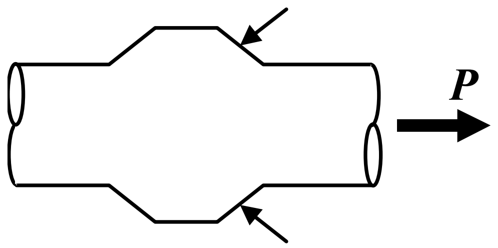
User-supplied subroutines developed for use in Diana

Karin Lundgren, Kamyab Zandi Hanjari
Chalmers University of Technology

Main user-supplied subroutines developed

- Frictional bond model, for 3D-analyses
- Corrosion, including swelling and transport of rust through cracks, for 3D-analyses

Bond: Interaction between concrete and reinforcement



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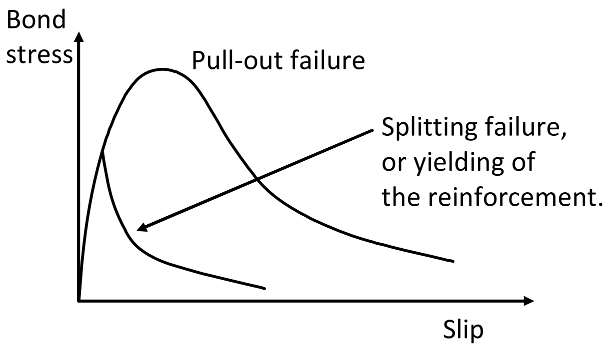
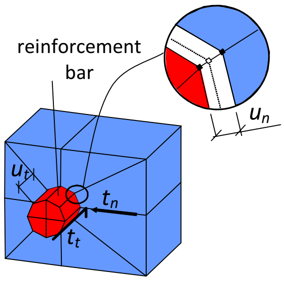
Bond-slip



Three-dimensional analyses



t_n = normal stress

t_t = bond stress

u_t = slip

u_n = relative normal displacement

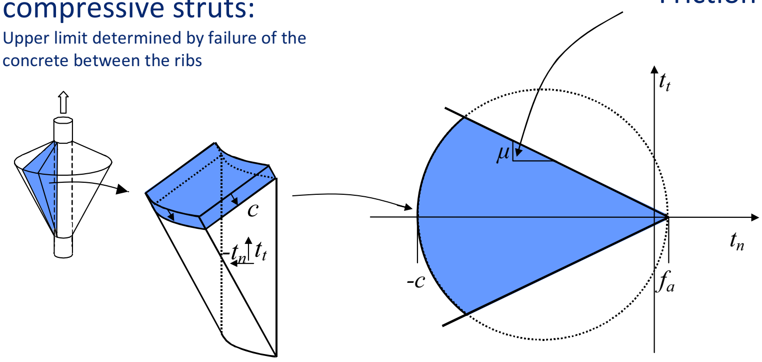
in the layer

Bond model

- Stress in the inclined compressive struts:

  Upper limit determined by failure of the concrete between the ribs

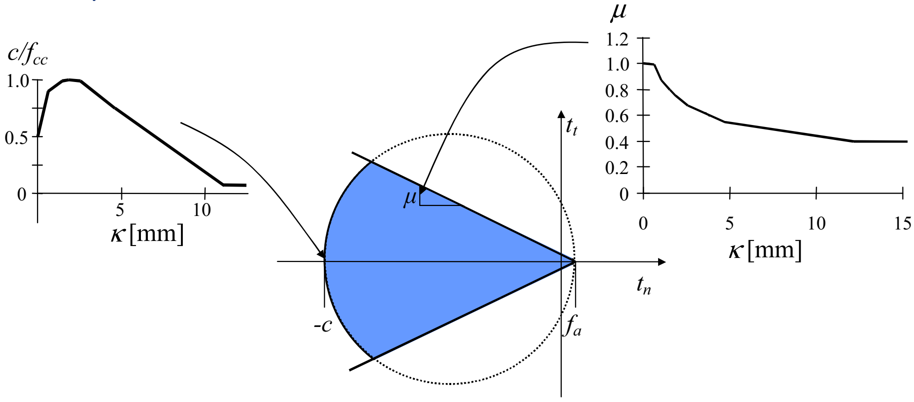
- Friction



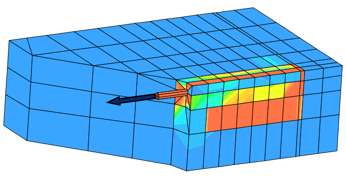
Most important input needed

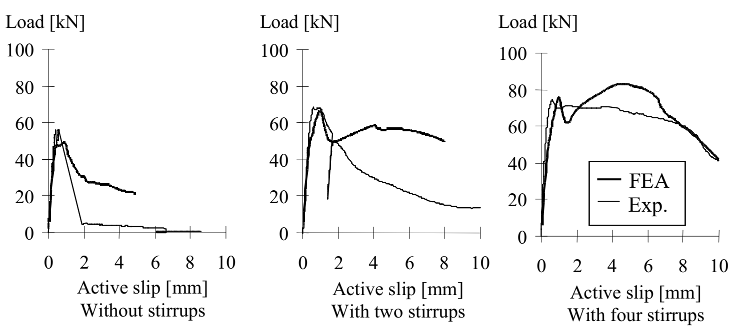
- Stress in the inclined compressive struts

- Friction

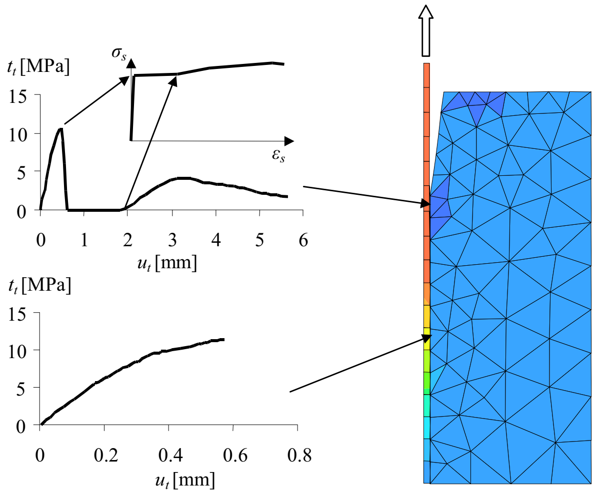


Splitting failure

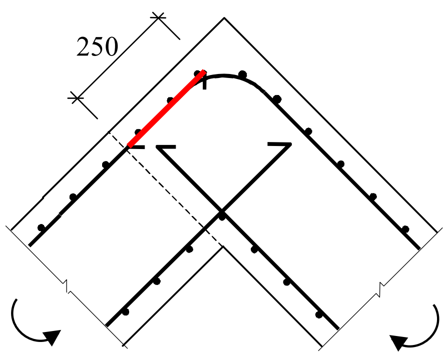




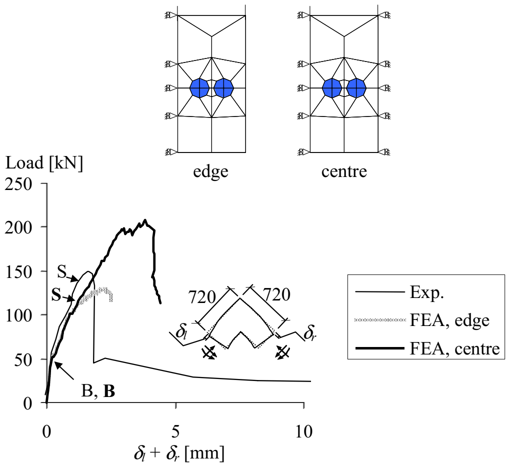
Yielding of the reinforcement

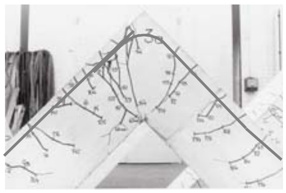


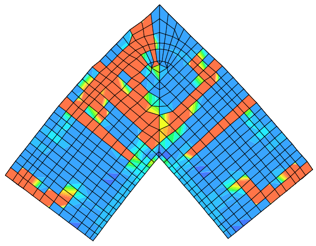
Lapped reinforcement splice in frame corner



3D analyses of lapped reinforcement splice







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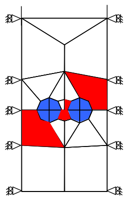
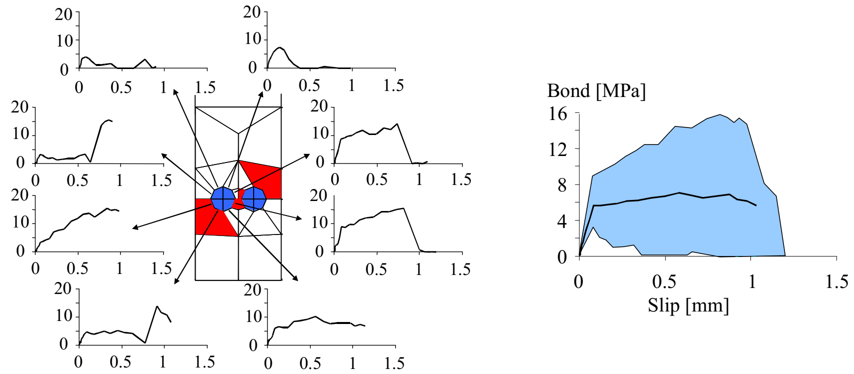
Bond - slip in centre slice(1)

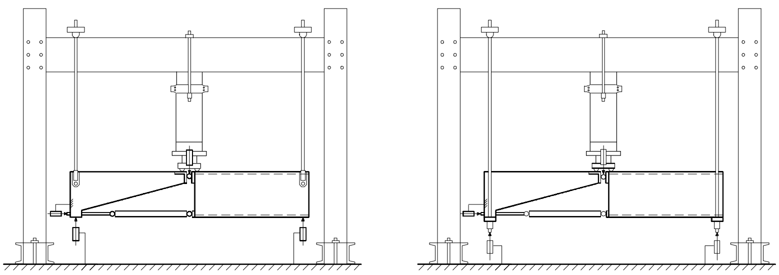


Bond - slip in centre slice(2)



Anchorage with different support conditions

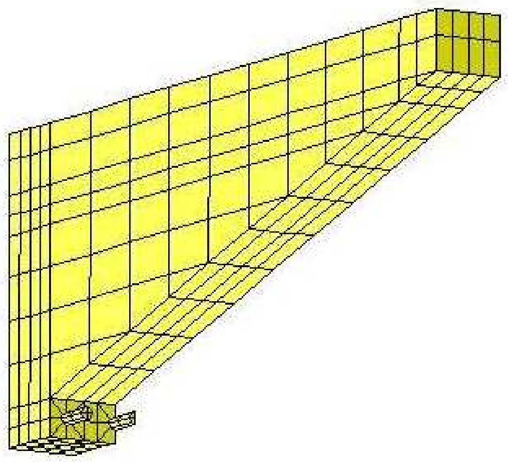
Test setups



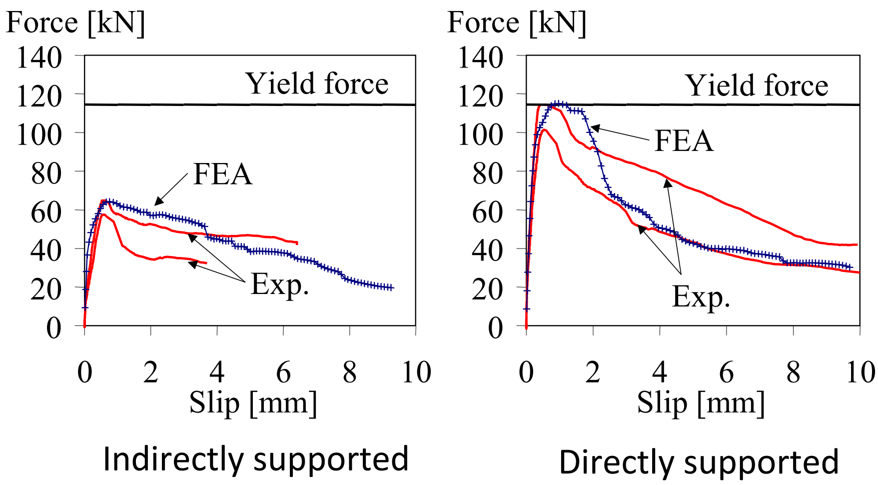
Indirectly supported

Directly supported

3D analyses, using the bond model



High strength concrete, two reinforcement bars



Bond model, features

- With the same input parameters, various bond-slip curves are obtained, depending on for example:
  - splitting of the concrete
  - yielding of the reinforcement
  - outer pressure
- Can be used in detailed analyses of anchorage regions, and thereby increase the understanding of the bond mechanism.

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Corrosion of reinforcement

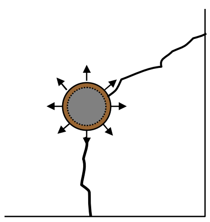
Area of reinforcement decreases

Increasing volume

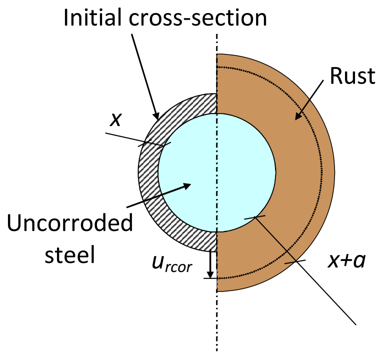
Splitting forces

Reduction of concrete cross-section

Bond is affected

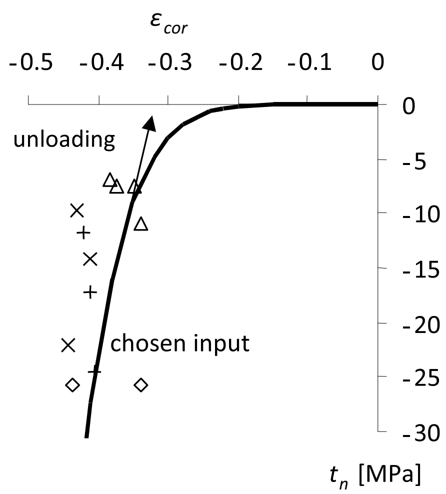


Modelling of corrosion layer



Strain in the rust depends on corrosion penetration *x*, expansion factor v_{rs}, and deformation u_{rcor}

Mechanical behaviour of the rust



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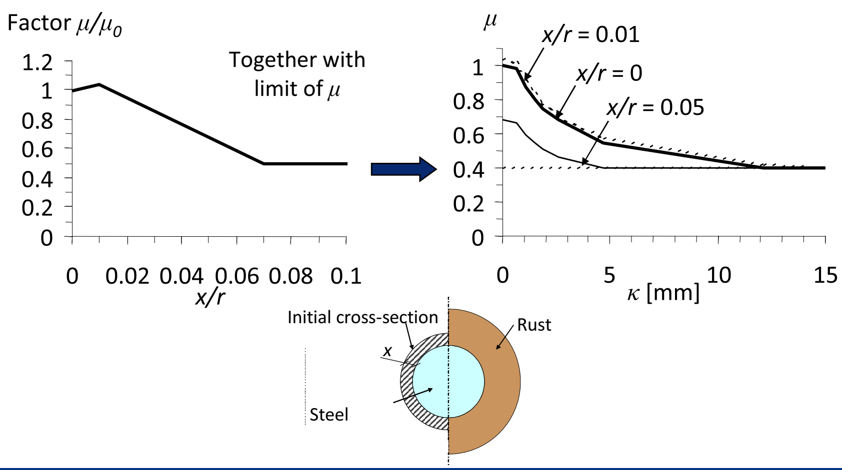
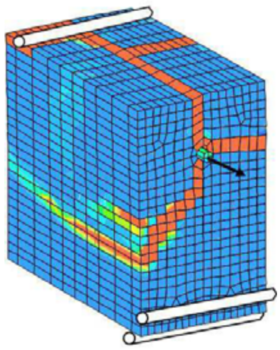
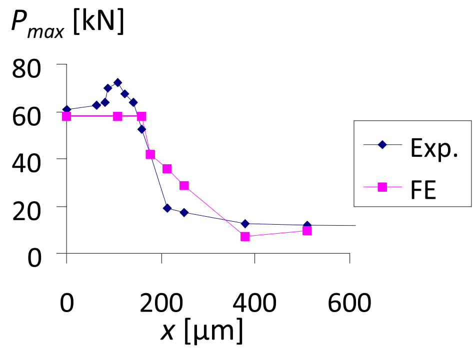
Corrosion affects the friction

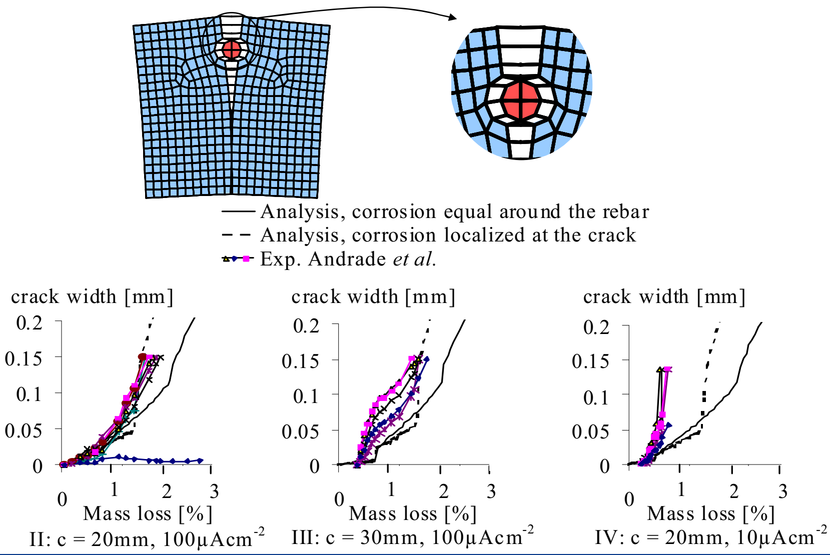


Analyses of pull-out tests of Almusallam

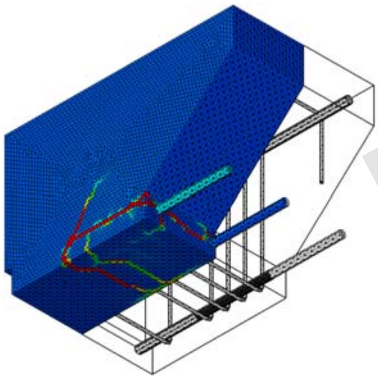


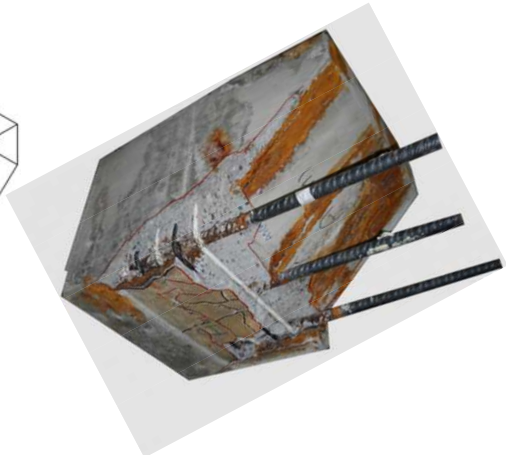


Analyses of corrosion cracking tests



Corrosion model further developed in PhD project of Kamyab Zandi Hanjari





Corrosion products through cracks

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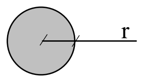
Extension of corrosion model(1)



Original corrosion model

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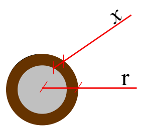
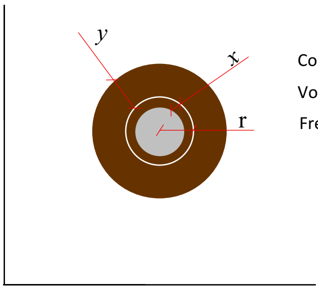
Extension of corrosion model(2)



Corrosion penetration: *x*

Original corrosion model

Extension of corrosion model(3)



Corrosion penetration: x

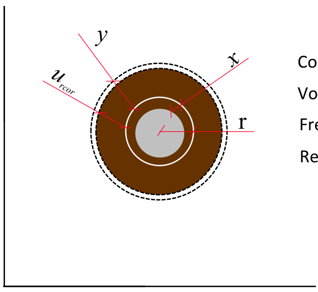
Volume rust / volume steel: v_{rs}

Free increase of the radius: y

$$y = -r + \sqrt{r^2 + (v_{rs} - 1)\cdot(2rx - x^2)}$$

Original corrosion model

Extension of corrosion model(4)



Corrosion penetration: x

Volume rust / volume steel: v_{rs}

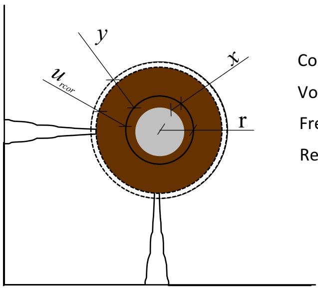
Free increase of the radius: y

Real increase of the radius: u_{rcor}

$$y = -r + \sqrt{r^2 + (v_{rs} - 1)\cdot(2rx - x^2)}$$

Original corrosion model

Extension of corrosion model(5)



Corrosion penetration: x

Volume rust / volume steel: v_{rs}

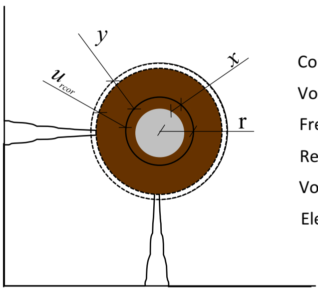
Free increase of the radius: y

Real increase of the radius: u_{rcor}

$$y = -r + \sqrt{r^2 + (v_{rs} - 1)\cdot(2rx - x^2)}$$

Original corrosion model

Extension of corrosion model(6)



Corrosion penetration: x

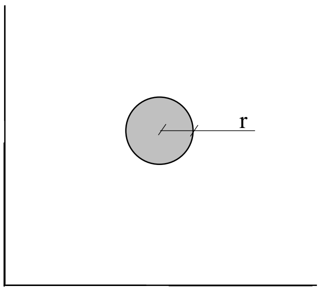
Volume rust / volume steel: v_{rs}

Free increase of the radius: y

Real increase of the radius: u_{rcor}

Volume flow of rust: V

Element size: e

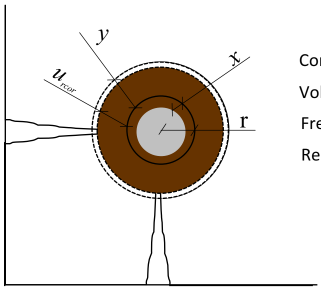


$$y = -r + \sqrt{r^2 + (v_{rs} - 1)\cdot(2rx - x^2)}$$

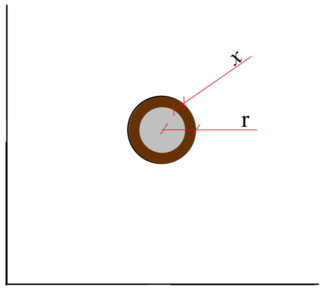
Original corrosion model

Extended corrosion model

Extension of corrosion model(7)



Corrosion penetration: x
Volume rust / volume steel: v_{rs}
Free increase of the radius: y
Real increase of the radius: u_{rcor}

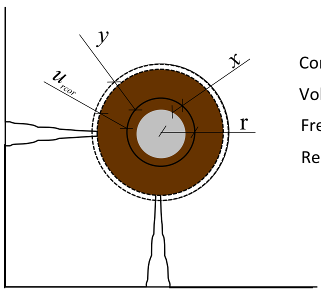


$$y=-r+\sqrt{r^2+(v_{rs}-1)\cdot(2rx-x^2)}$$

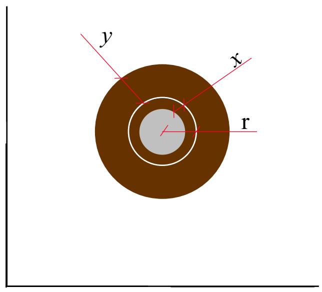
Original corrosion model

Extended corrosion model

Extension of corrosion model(8)



Corrosion penetration: x
Volume rust / volume steel: v_{rs}
Free increase of the radius: y
Real increase of the radius: u_{rcor}

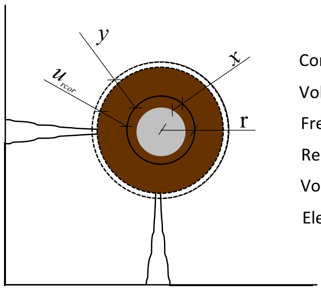


$$y=-r+\sqrt{r^2+(v_{rs}-1)\cdot(2rx-x^2)}$$

Original corrosion model

Extended corrosion model

Extension of corrosion model(9)



Corrosion penetration: *x*

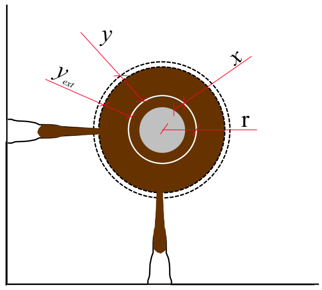
Volume rust / volume steel: v_{rs}

Free increase of the radius: *y*

Real increase of the radius: u_{rcor}

Volume flow of rust: *V*

Element size: *e*



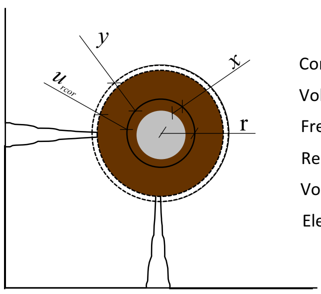
$$y = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)\cdot(2rx - x^2)}$$

Original corrosion model

$$y_{ext} = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)(2rx - x^2) - \frac{V}{\pi \cdot e}}$$

Extended corrosion model

Extension of corrosion model(10)



Corrosion penetration: *x*

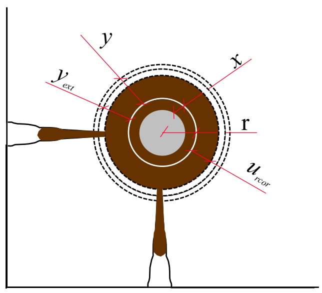
Volume rust / volume steel: v_{rs}

Free increase of the radius: *y*

Real increase of the radius: u_{rcor}

Volume flow of rust: *V*

Element size: *e*



$$y = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)\cdot(2rx - x^2)}$$

Original corrosion model

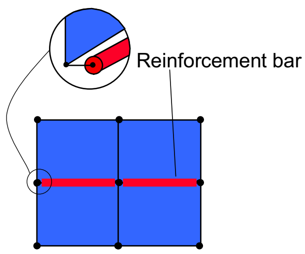
$$y_{ext} = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)(2rx - x^2) - \frac{V}{\pi \cdot e}}$$

Extended corrosion model

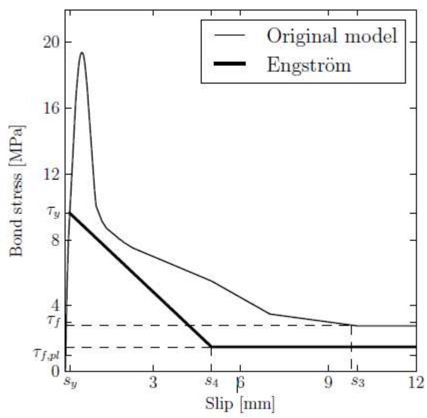
Corrosion model, features

- Corrosion is modelled in time steps
- Describes the swelling effect of corrosion
- Mechanical properties of corrosion products are included
- Can be combined with the bond model; thus describing the effect of corrosion on bond

Simple model for reduction of bond stress at yielding



- In analyses where bond-slip is given as input
- At reinforcement yielding: Bond deviates from given bond-slip curve



Overview of user-supplied subroutines developed

- Frictional bond model, for 3D-analyses
- Corrosion, including swelling and transport of rust through cracks, for 3D-analyses
- Bond loss at yielding, simple bond-slip model
- Varying with element age:
  - Concrete strength
  - Bond strength

Finally..

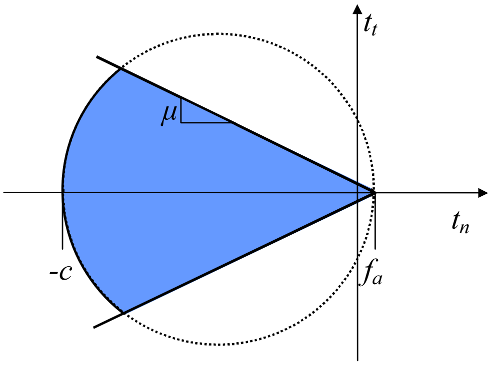
- From the manual:

  "The user-supplied interface model should be coded with great care. The routine should perform the intended function without influencing other parts of DIANA. See Volume Analysis Procedures for the precautionary measures to be taken when applying user-supplied subroutines. Before using the user-supplied interface model in production analyses, it shall be developed and tested on a single-element example to verify the accuracy of its constitutive behavior. "

  This can not be stressed too much!

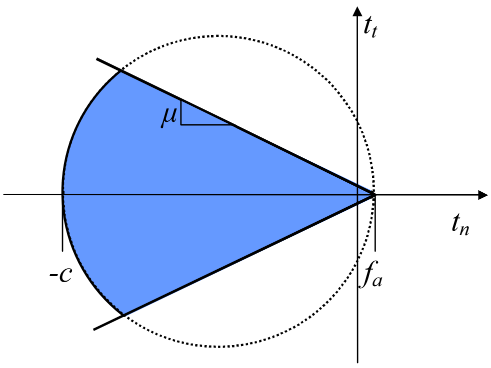
Bond model

- Friction
- Ability to cause normal stress at slip
- Upper limit determined by failure of the concrete between the ribs
- Adhesion



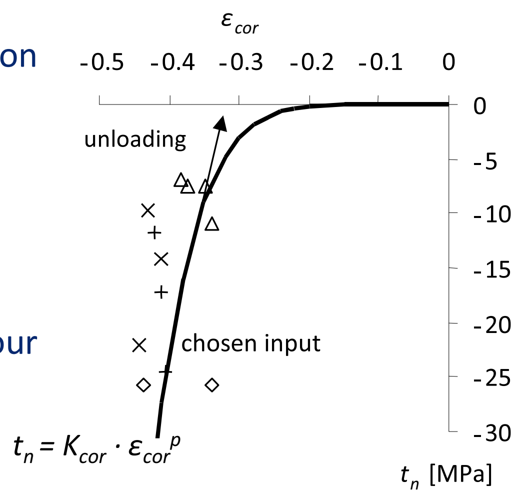
Input needed for bond model

- Linear stiffnesses D11 and D22
- $\mu(\kappa)$, $c(\kappa)$, $f_a(\kappa)$, η
- Damaged region, μ_{d0}, η_{d0}



Input needed for corrosion model

- Corrosion penetration as function of time, *x(t)*
- Volume of the rust relative to the uncorroded steel, *v*
- Mechanical behaviour of the rust: K_{cor}, p



Fibre reinforced concrete in dapped-end beams

8th International DIANA Users Meeting

25-26 April 2013

Gothenburg - Sweden

NTNU
Norwegian University of Science and Technology

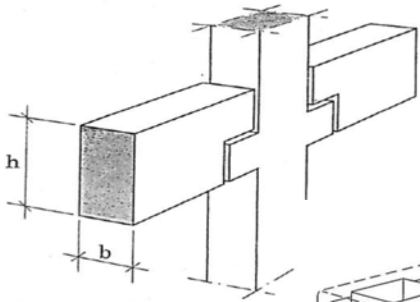
Elena V. Sarmiento
Giedrius Zirgulis
Max A.N. Hendriks
Mette R. Geiker
Terje Kanstad

Introduction > Overview

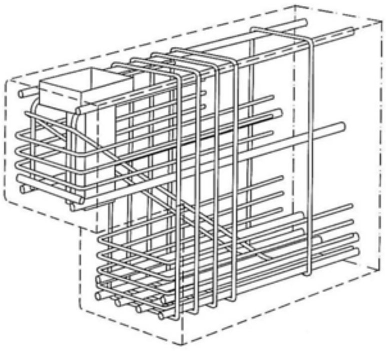
SINTEF NTNU sfi

Introduction
Overview
Motivation

Introduction > Motivation

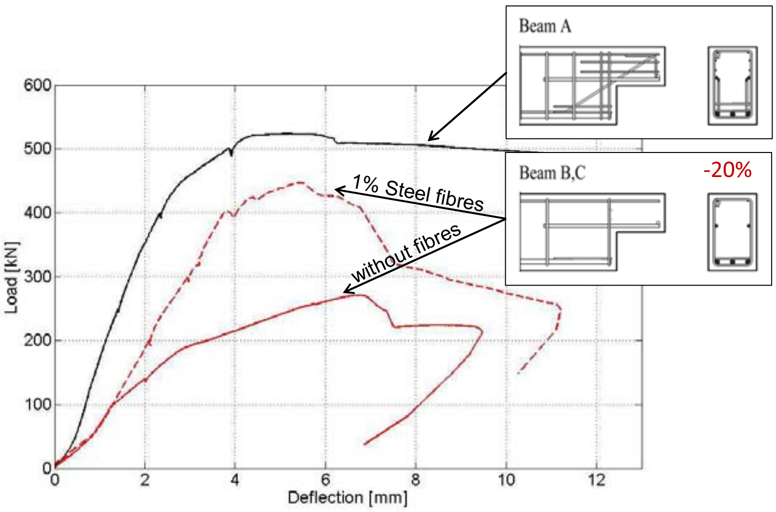


Application for FRC



Introduction
Overview
Motivation
Dapped-end beams project
Previous experience

Dapped-end beams project > Previous experience(1)



Introduction
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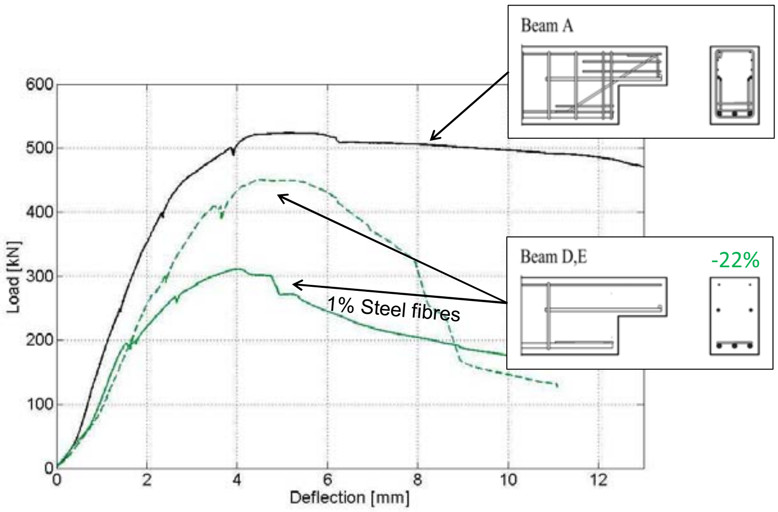
Previous experience

Dapped-end beams project > Previous experience(2)



SINTEF
NTNU
sfi

Introduction
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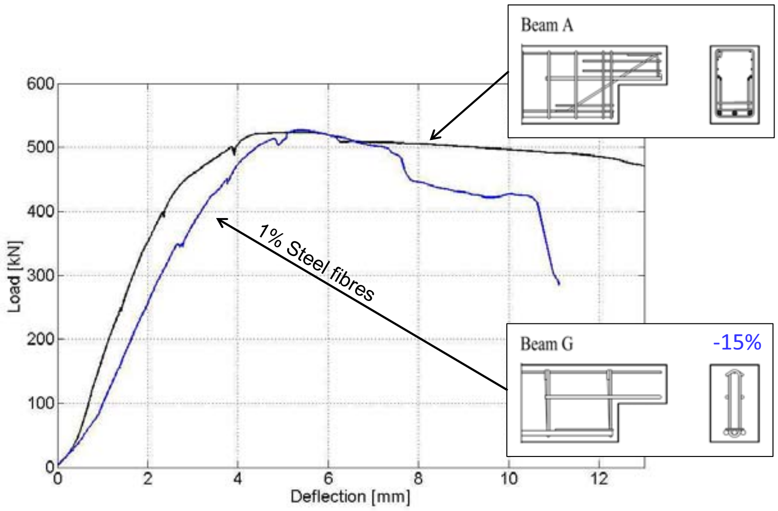
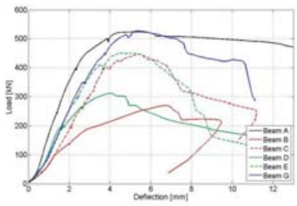
Dapped-end beams project
Previous experience

Dapped-end beams project > Previous experience(3)



Dapped-end beams project > Limitations(1)

- Optimization of the fibre reinforcement (type and content)
- Optimization of the ordinary reinforcement layout



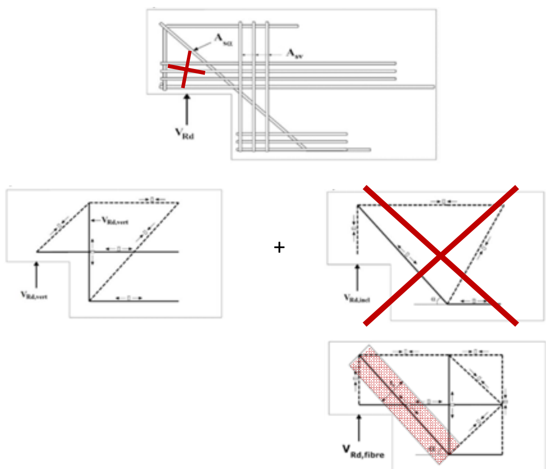
Limited cases of study

Finite Element Analysis

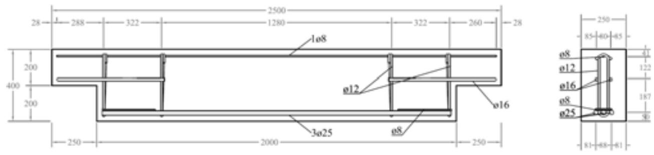
Dapped-end beams project > Limitations(2)

But in addition...

- **Design methods** for D-regions **for Fibre reinforced concrete**

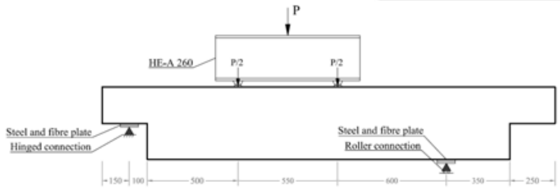


Finite Element Analysis > Study case(1)



Fresh concrete properites	
t_{500}	3 s
Slump	600 mm

Fibre properties	
Name	Bekaert Dramix RC-65/60-BN
Material	Steel
Length	60
Diameter	0,9
Surface	Rounded, plain, hook-ended
Volume content	1%



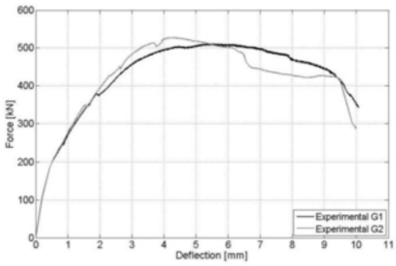
Fibre reinforced concrete in dapped-end beams E.V. Sarmiento et al.

Introduction
- Overview
- Motivation

Dapped-end beams project
- Previous experience
- Limitations

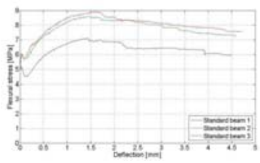
FEA
- Study case

Finite Element Analysis > Study case



Standard 3P bending test (*EN14651*)

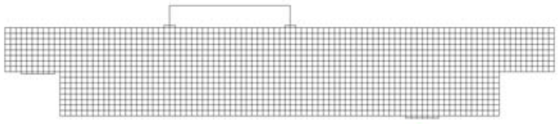
Beam	$f_{ct,L}$ [kN]	$f_{R,1}$ [kN]	$f_{R,2}$ [kN]	$f_{R,3}$ [kN]	$f_{R,4}$ [kN]
7	5,19	5,87	6,56	6,39	6,35
8	5,91	7,22	8,53	8,20	7,62
9	6,05	7,36	8,84	8,30	7,93
Average	**5,72**	**6,82**	**8,11**	**7,63**	**7,30**



Compressive test

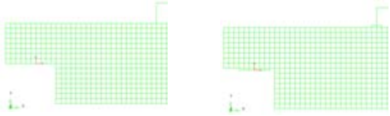
Cube	$f_{cm,cube}$[MPa]	f_{cm}[MPa]
1	74,2	
2	79,2	
3	78,9	
Average	**77,4**	**67,5**

Finite Element Analysis > Model description



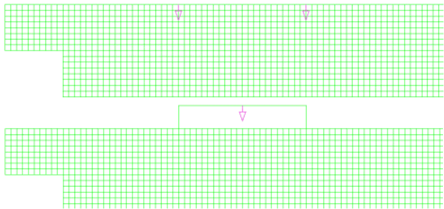
Supports and loading

- Simple supports
- Interface elements



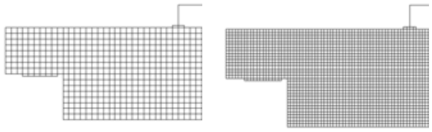
Loading control

- Force load
- Displacement load



Mesh size

- Element size 25x25
- Element size12.5x12.5



Finite Element Analysis > Materials / geometries(1)

Fibre reinforced concrete

Elements

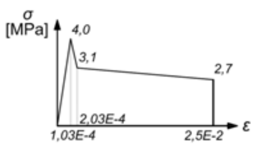
▪Eight-node quadrilateral plane stress elements (CQ16M)

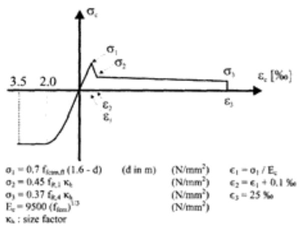
Crack modeling

▪Semeared crack approach with Rotating crack model

Concrete in tension

▪Multi-linear tension softening curve





RILEM Design methods for steel fibre reinforced concrete. σ-ε-design method

Concrete in compresion

▪Ideal

Finite Element Analysis > Materials / geometries(2)

Reinforcement

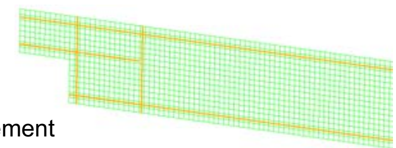
Strain hardening model

- Yield stress: 566 Mpa
- Modulus of elasticity: 200 000 MPa
- Ultimate strain and stress at ultimate strain: 7.5‰, 589 MPa

Finite Element Analysis > Materials / geometries(3)

Bond properties

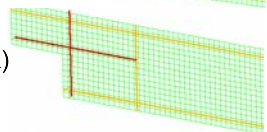
- All Embedded reinforcement

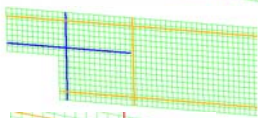


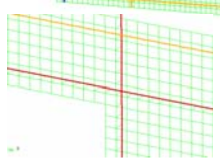
- Bond-slip + Embedded reinforcement

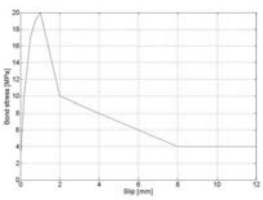
  Option 1
  - Reinf with slip: Truss elements (CL6TR)
  - Line interface elements (CL12I)

  Option 2 - «BAR in Plane stress»
  - Reinf with slip: Truss elements (CL6TR)
  - Line-plane interface elements (CQ22IF)









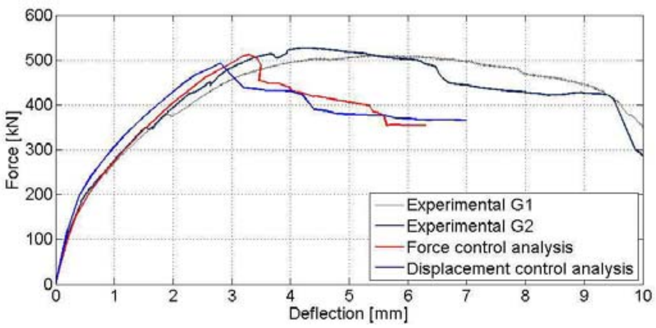
David Fall et al. *Non-linear finite element analysis of steel fibre reinforced beams with conventional reinforcement.* BEFIB2013

Finite Element Analysis > Results and discussion

- Control method
- Step size
- Convergence norm
- Mesh size
- Concrete strain limit
- Bond properties

Finite Element Analysis > Results and discussion

Control method

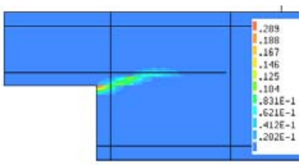


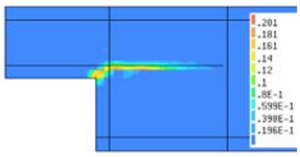
Finite Element Analysis > Results and discussion

Control method

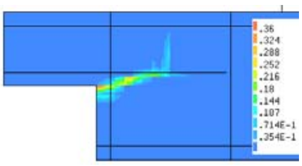
Displacement control

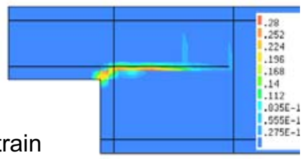
Force control





Principal strain
4mm deflection

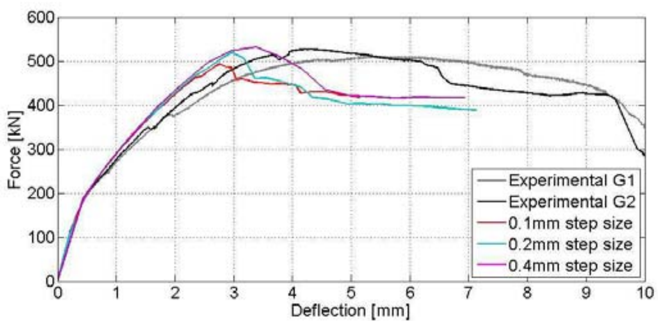




Principal strain
6mm deflection

Finite Element Analysis > Results and discussion

Step size

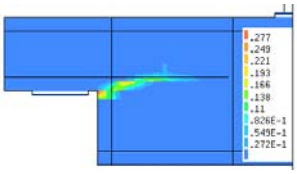


Finite Element Analysis > Results and discussion

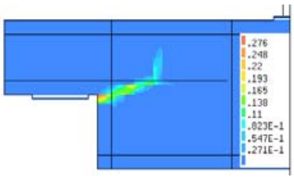
Step size

Principal strain
5mm deflection

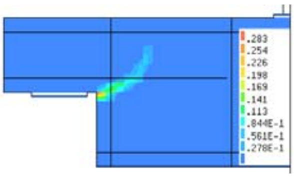
0.1mm step size



0.2mm step size

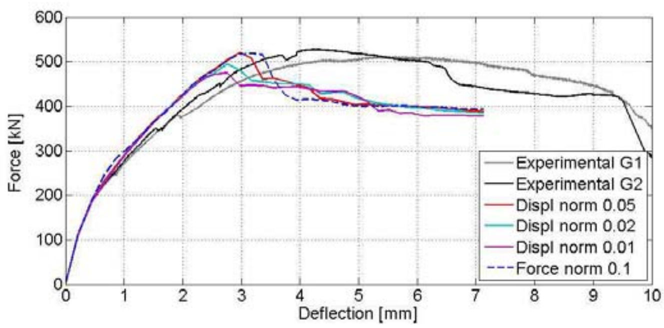


0.5mm step size



Finite Element Analysis > Results and discussion

Convergence norm



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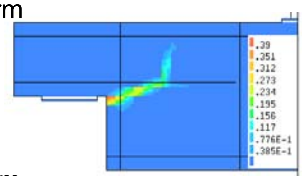
Finite Element Analysis > Results and discussion

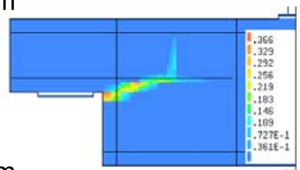
Convergence norm

Principal strain
6.8mm deflection

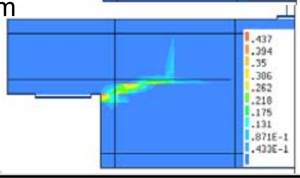
Displ norm
0.05



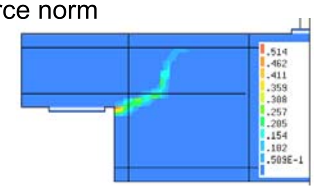
Displ norm
0.02



Displ norm
0.01



Force norm
0.1



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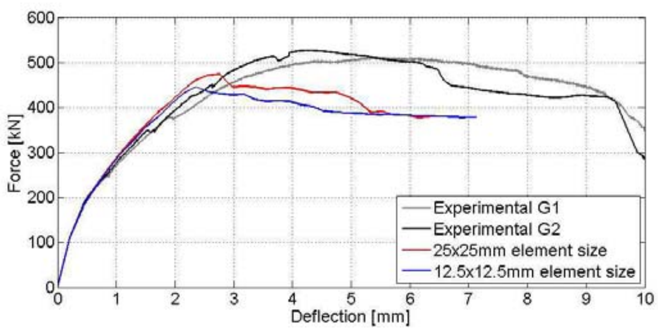
Finite Element Analysis > Results and discussion

Mesh size

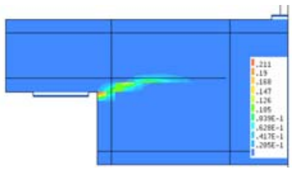
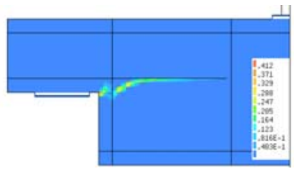


Finite Element Analysis > Results and discussion

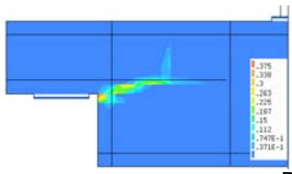
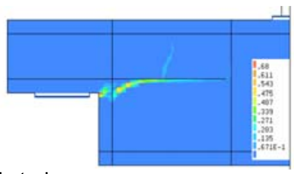
Mesh size

25x25mm el. size

12.5x12.5 el. size

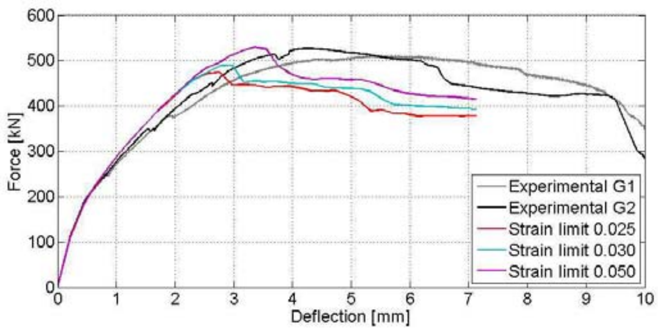
Principal strain
4mm deflection

Principal strain
6mm deflection

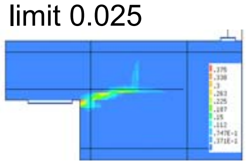
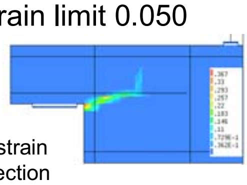
Finite Element Analysis > Results and discussion

Concrete strain limit



Strain limit 0.025

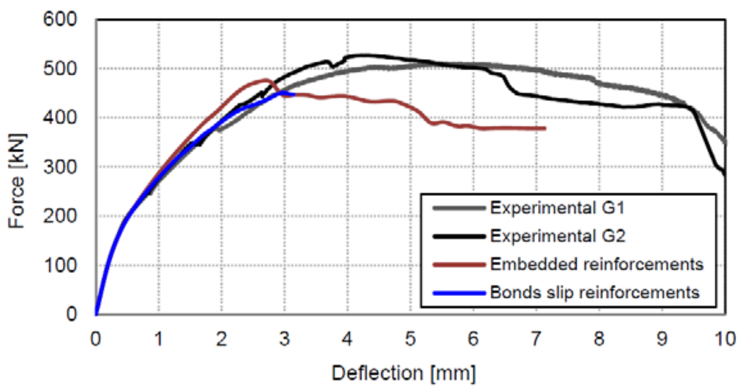
Strain limit 0.050

Principal strain
6mm deflection

Finite Element Analysis > Results and discussion

Bond properties



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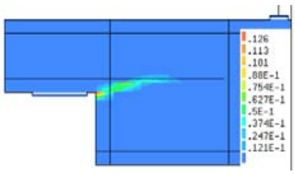
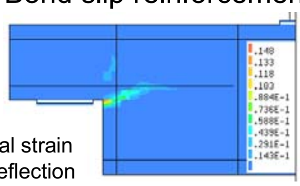
Bond properties

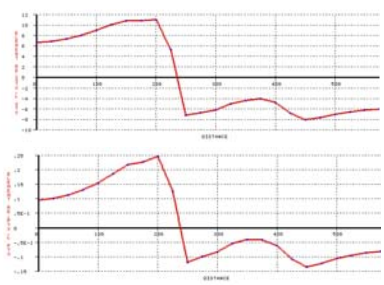
Embedded reinforcements



Bond slip reinforcements



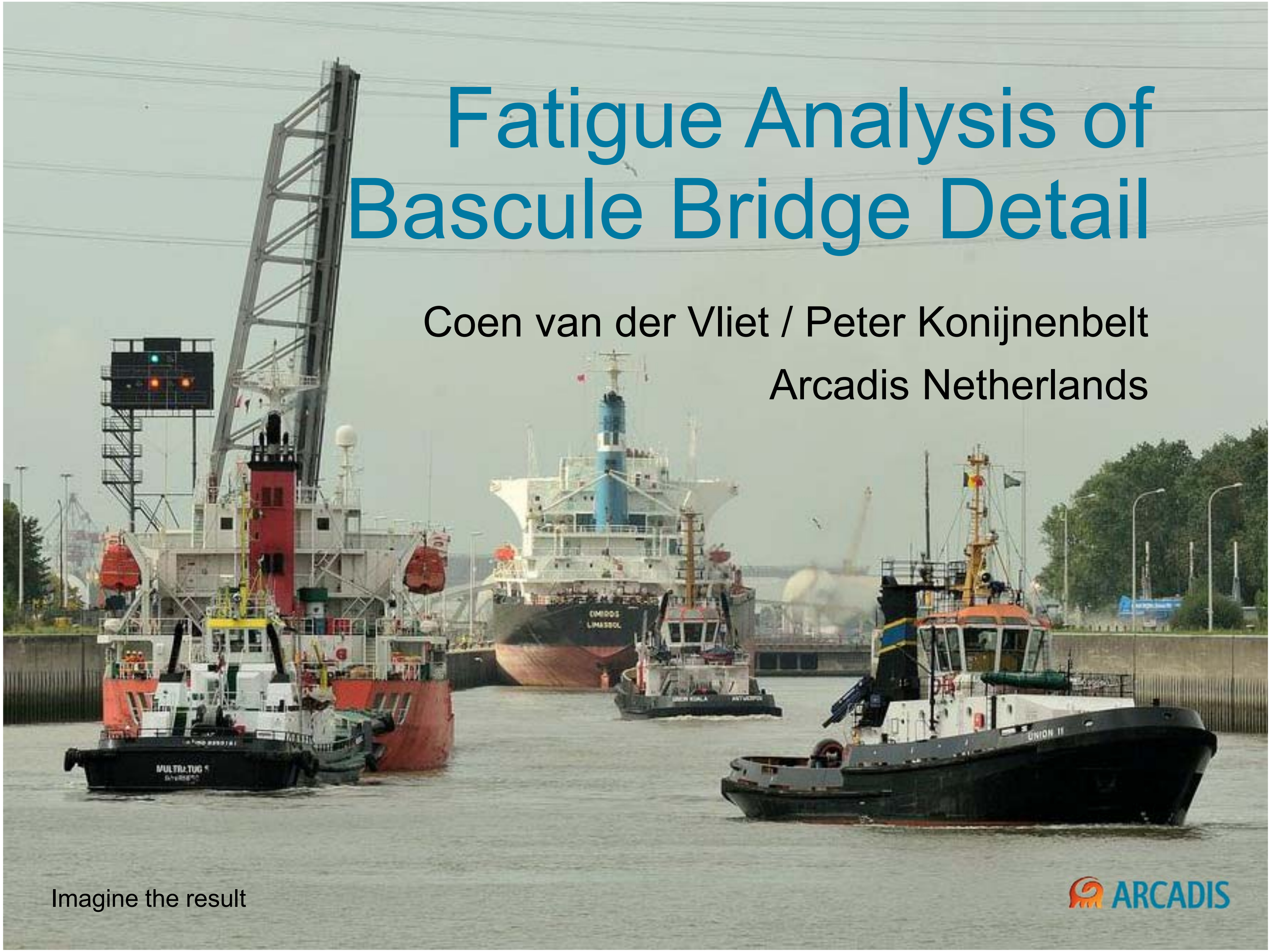
Principal strain
3mm deflection



Summary

- In general, the maximum load is under a certain limit well described
- The model underestimates the loads at large deflections
- The crack pattern is poorly described
- The bond slip properties have to be improved
- Dependency on the mess alignment
- Other material models for FRC need to be evaluated

Thank you for your attention



Fatigue Analysis of Bascule Bridge Detail

Coen van der Vliet / Peter Konijnenbelt
Arcadis Netherlands

Imagine the result

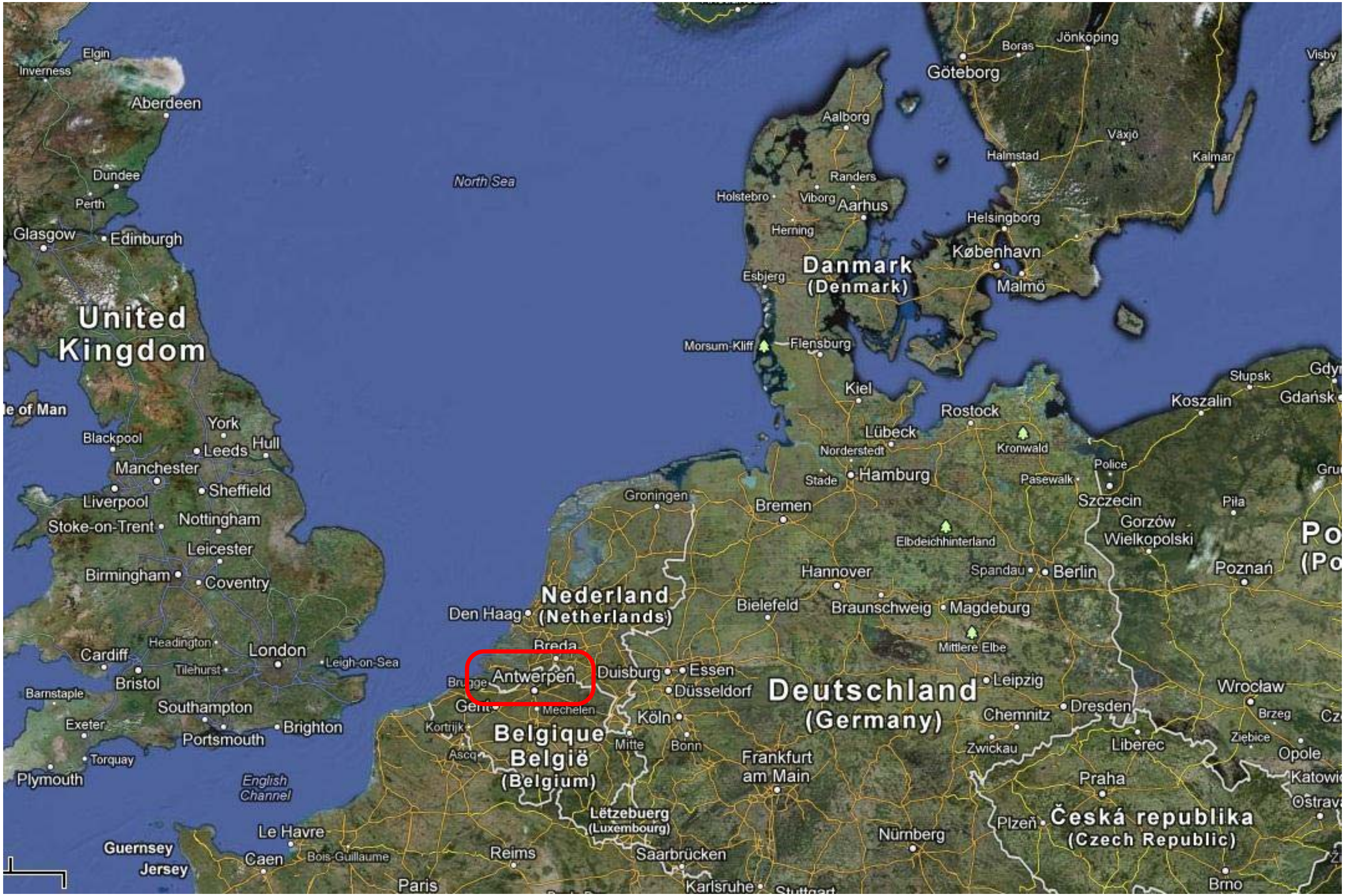
ARCADIS

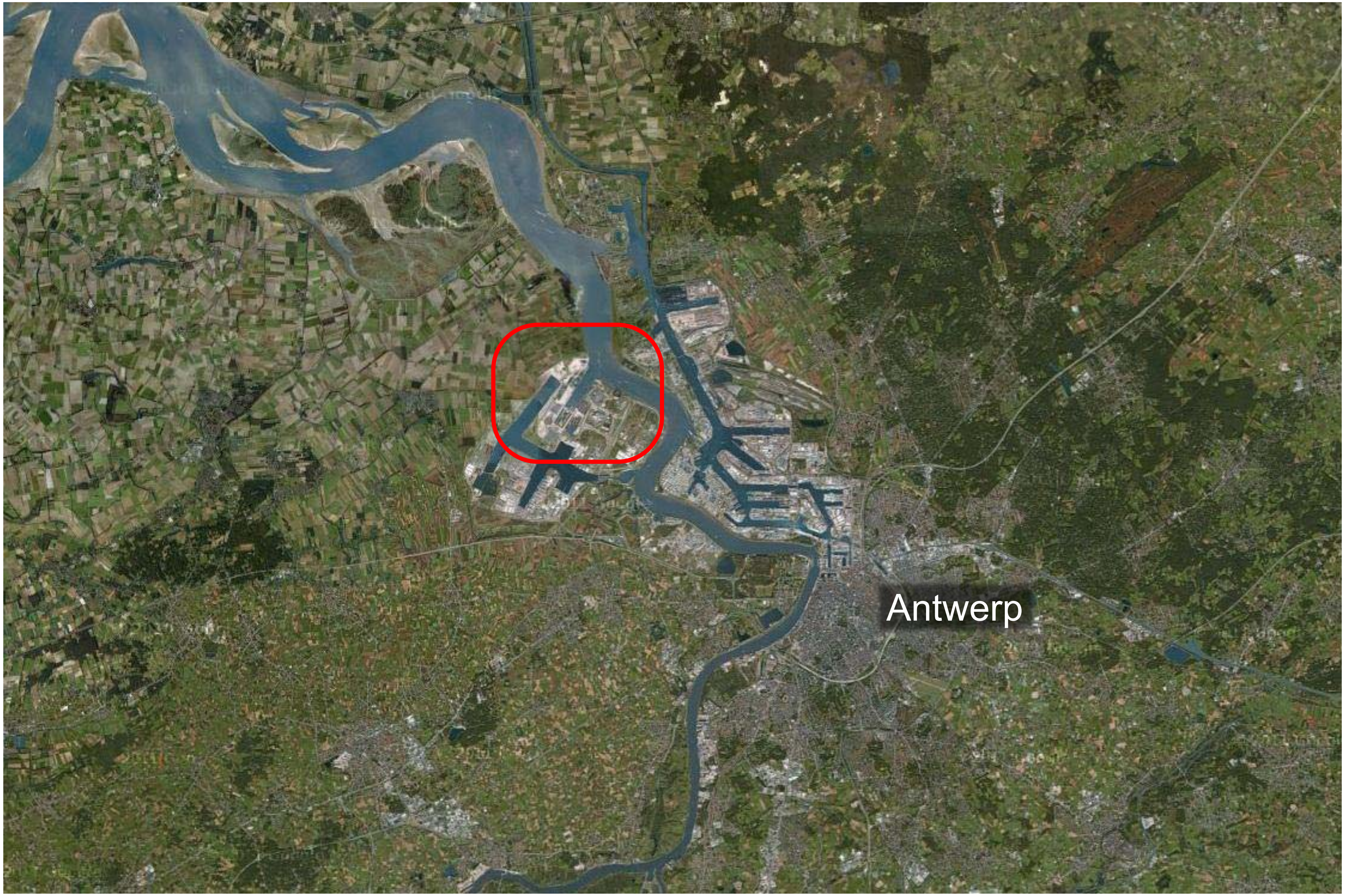
Contents

- Project
- Fatigue
- Approach
- FE Models & Results
- Fatigue Verification
- Related Topics

Project

ARCADIS


United Kingdom
North Sea
Danmark (Denmark)
Nederland (Netherlands)
Deutschland (Germany)
Belgique België (Belgium)
Lëtzebuerg (Luxembourg)
Česká republika (Czech Republic)
English Channel
Antwerpen
Den Haag
Breda
Mechelen
Gent
Brugge
London
Göteborg
København
Malmö
Hamburg
Berlin
Bremen
Rostock
Kiel
Lübeck
Hannover
Köln
Düsseldorf
Duisburg
Essen
Leipzig
Dresden
Praha
Frankfurt am Main
Nürnberg
Paris
Reims
Le Havre
Caen


Antwerp


Deurganckdok
Deurganckdok Sluice
Waasland Harbour

Deurganckdoksluis

Port of Antwerp

Length:	500 m
Width:	68 m
Bridge Span:	70 m

Construction

Start	2011
Delivery	2016

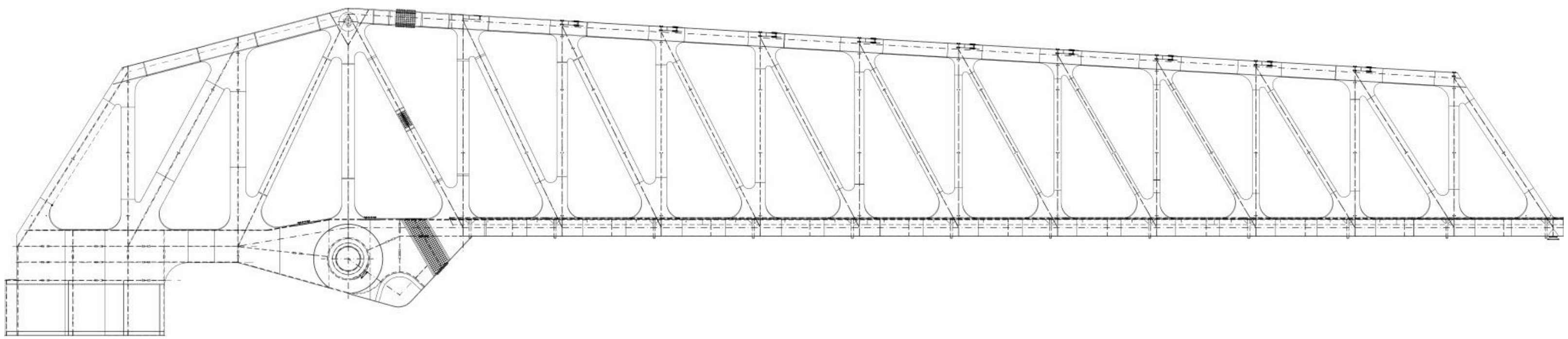
Parties involved:

Client: Belgian Dep. Of Mobility and Public Works

Contractor: THV Waaslandsluis

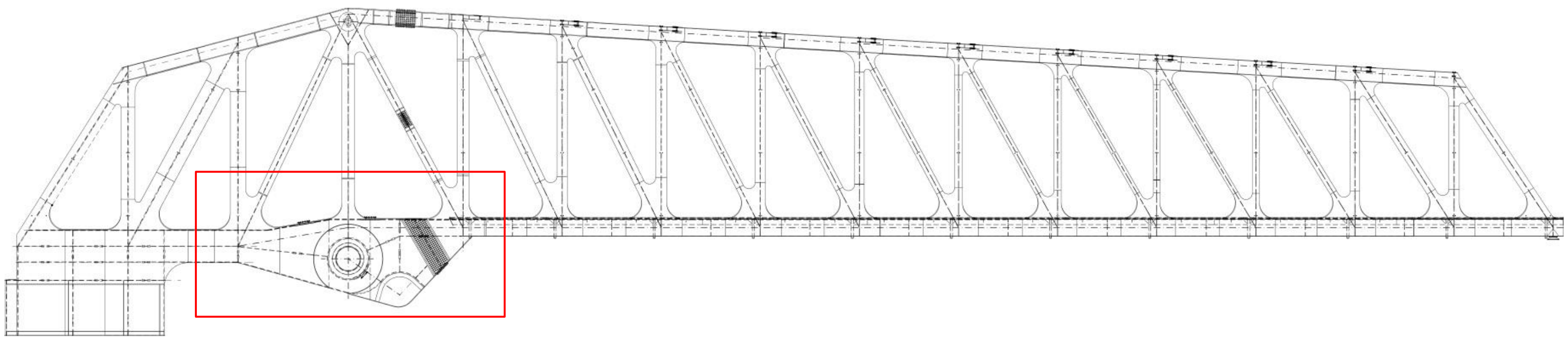
ARCADIS

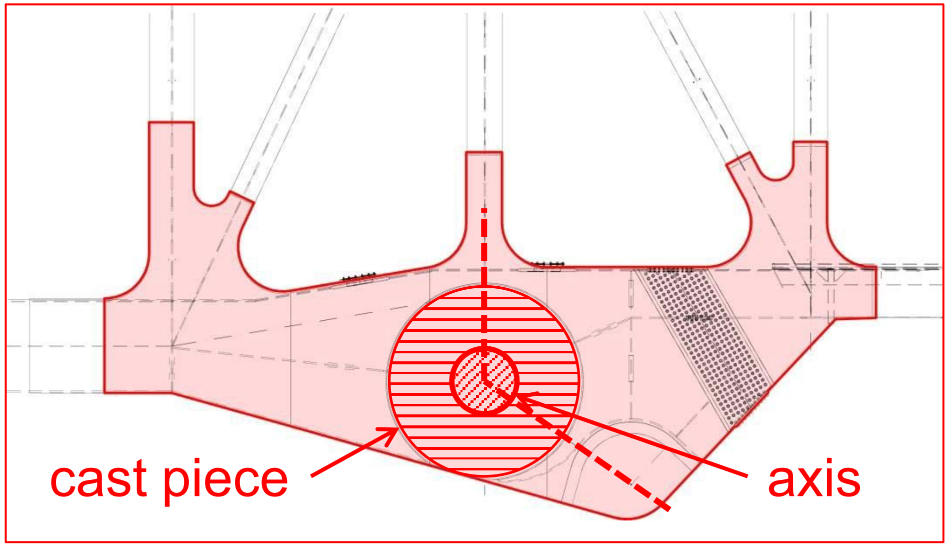
Bascule Bridges



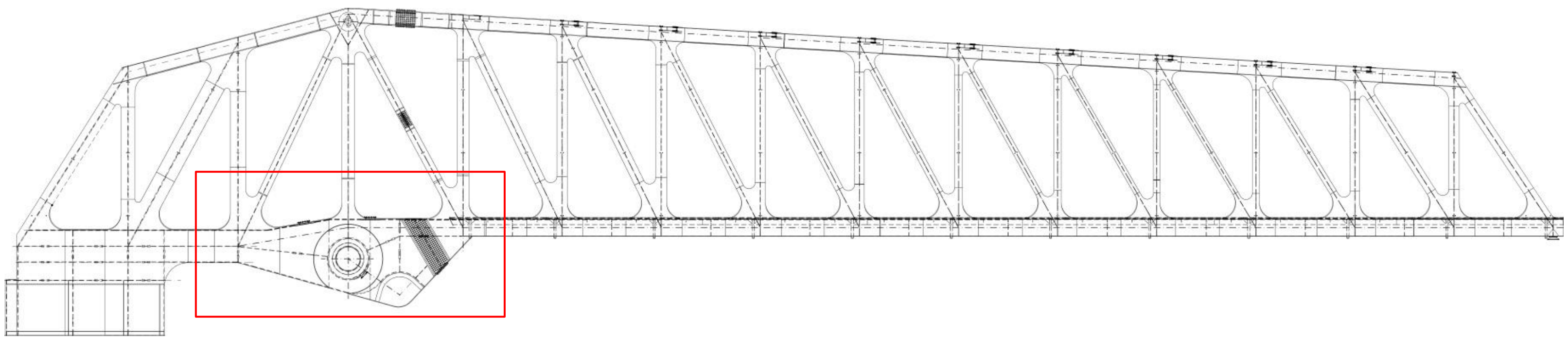
ARCADIS

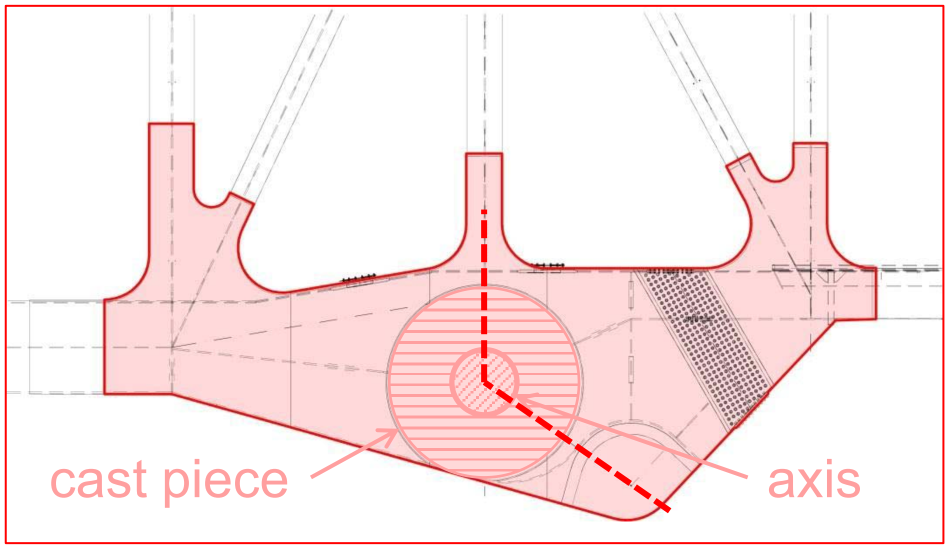
Bascule Bridges

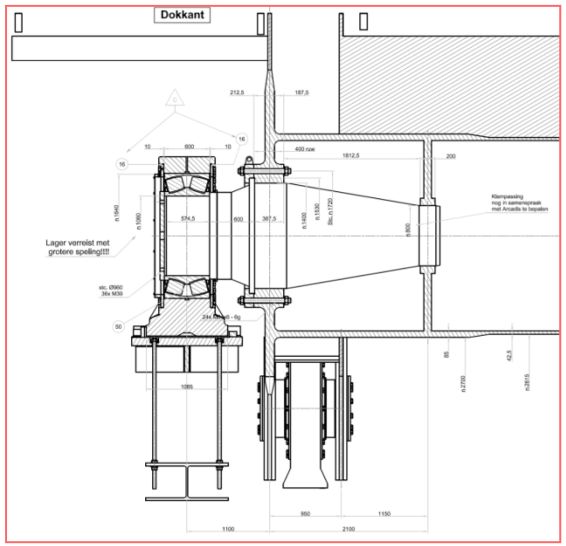




Bascule Bridges





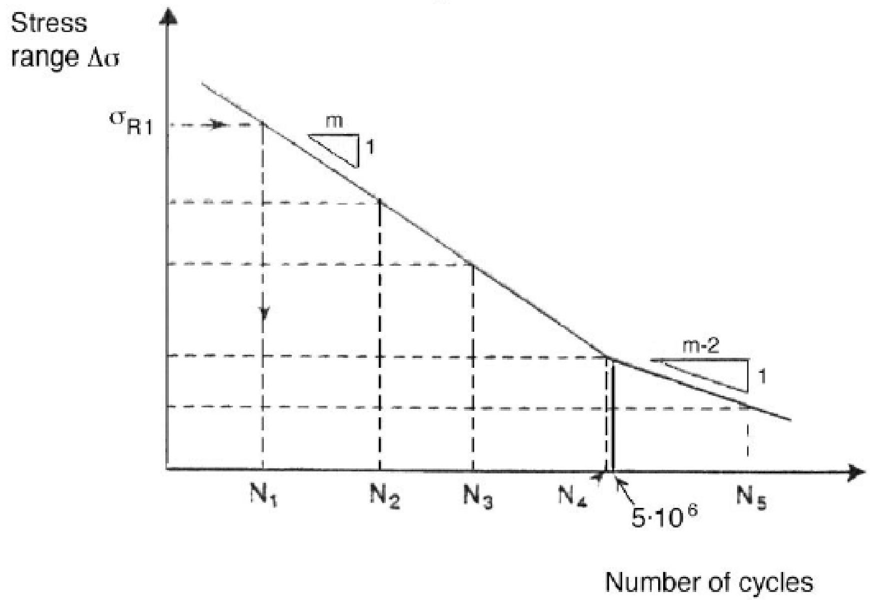


Fatigue

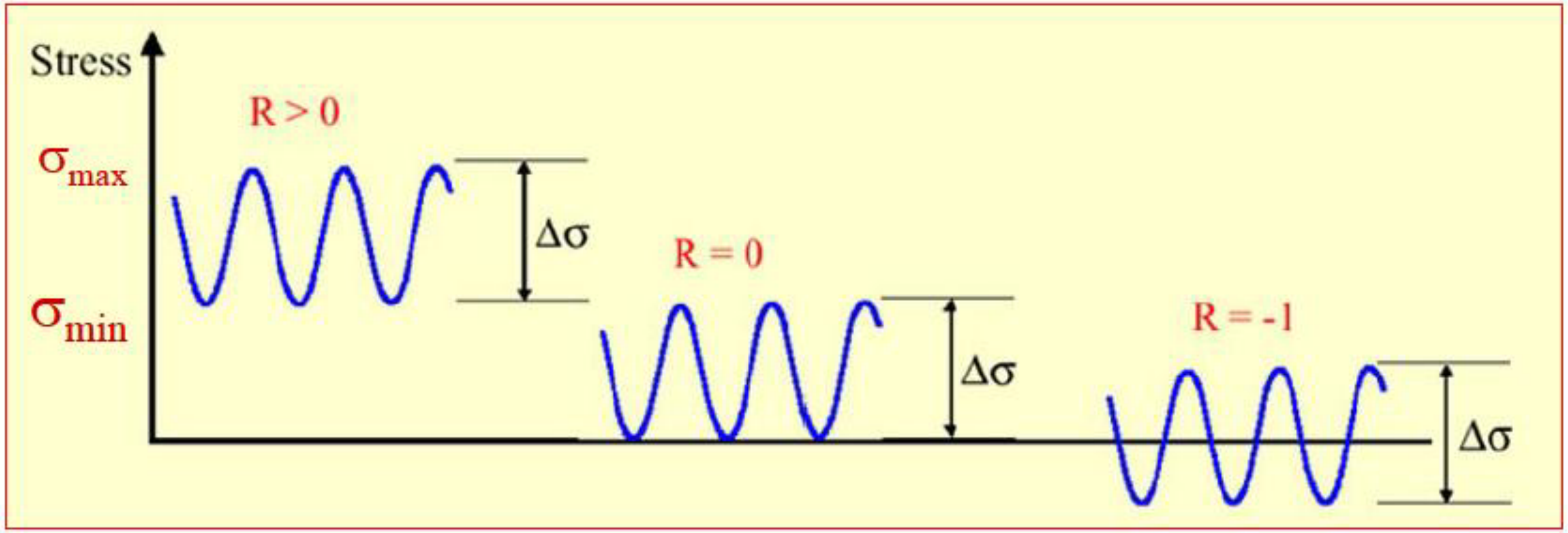
ARCADIS

Fatigue – general principle

- Cyclic loading reduces strength
- Strength reduction related to log(N)
- Miner: $\Sigma(n_i/N_i) \leq 1$
- Stress peaks at notches etc.



Fatigue – general principle

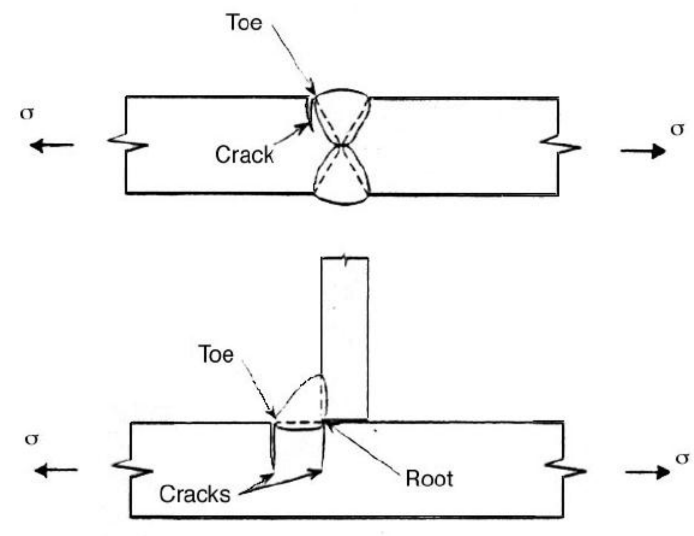


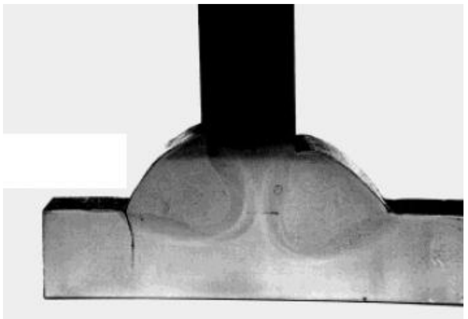
$$\Delta\sigma = \sigma_{max.} - \sigma_{min.}$$

$$R = \sigma_{min} / \sigma_{max.}$$

Fatigue - welds

- Welds: local disturbances of stress distribution
- Stress trajectories and concentrations
- Stress peak depends on weld detail

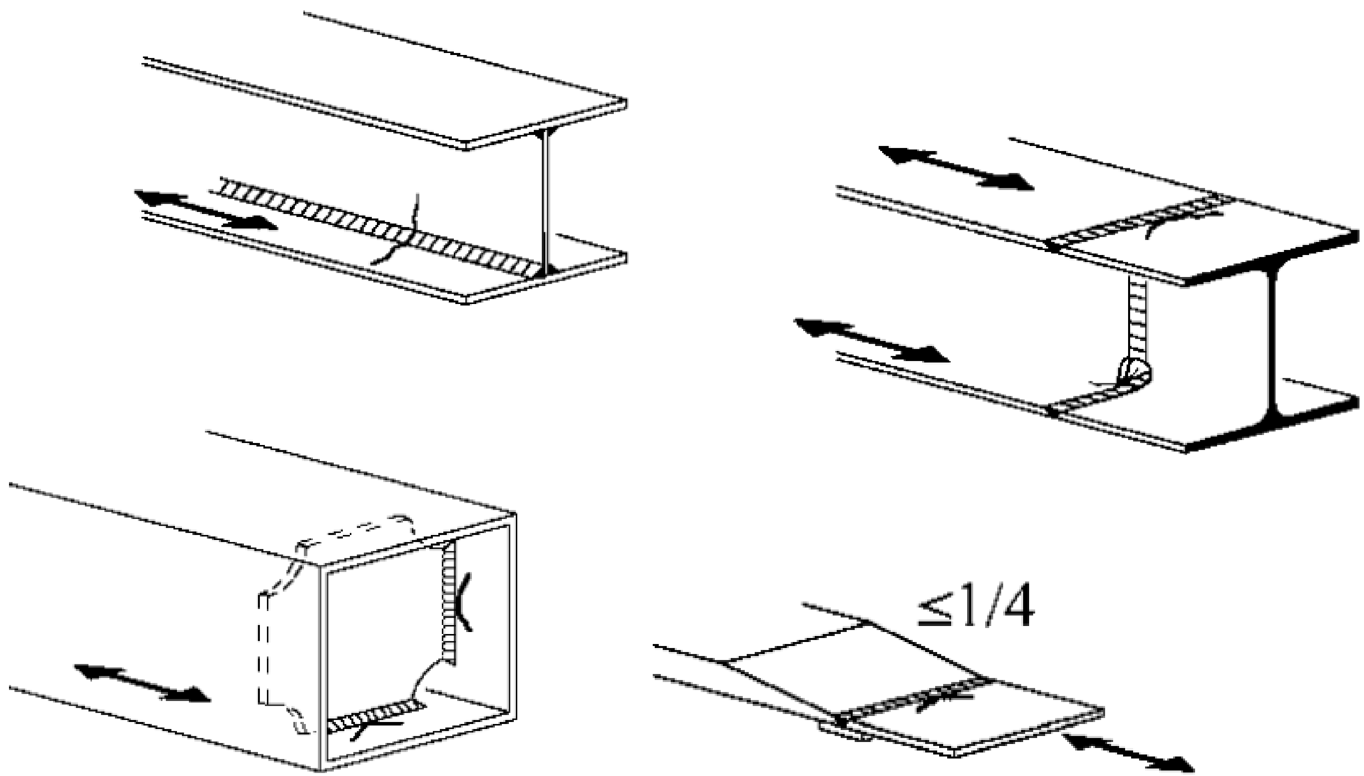




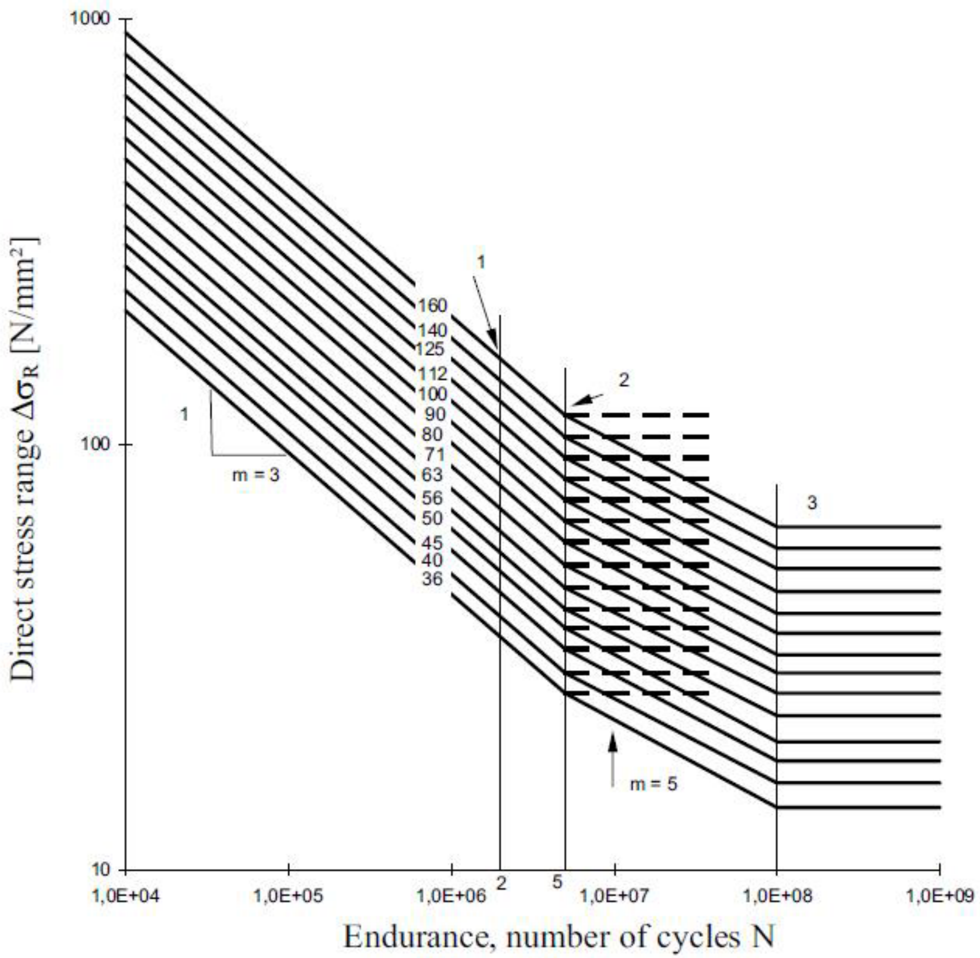
Fatigue – welds

- Residual stresses equal yield strength
- $\sigma_{max} = f_y$, $\sigma_{min} = f_y - \Delta\sigma$
- Stress level not important

Fatigue – weld details



Fatigue – detail cat. strength



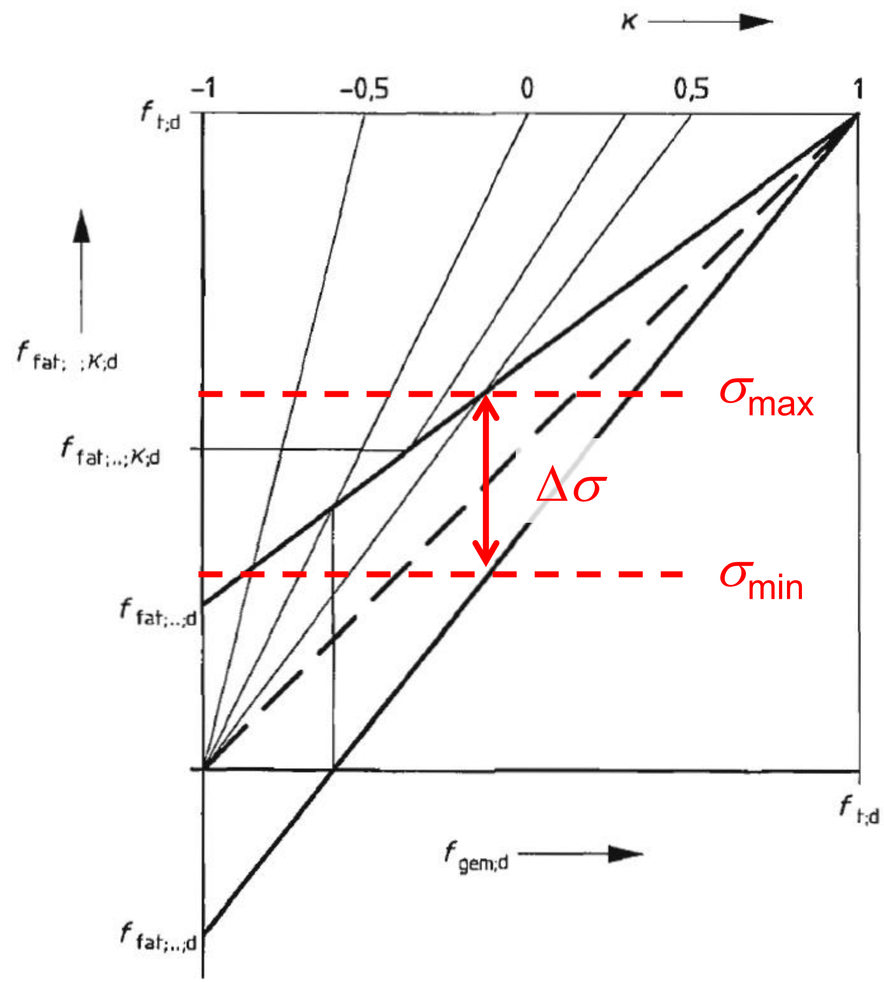
1 Detail category $\Delta\sigma_C$

2 Constant amplitude fatigue limit $\Delta\sigma_D$

3 Cut-off limit $\Delta\sigma_L$

Fatigue – base material

- Stress relieved: no residual stresses
- Stress level does matter
- Fatigue strength depends on
  - stress level ($\kappa = \sigma_{min}/\sigma_{max}$)
  - component (bending, tension compression, shear)

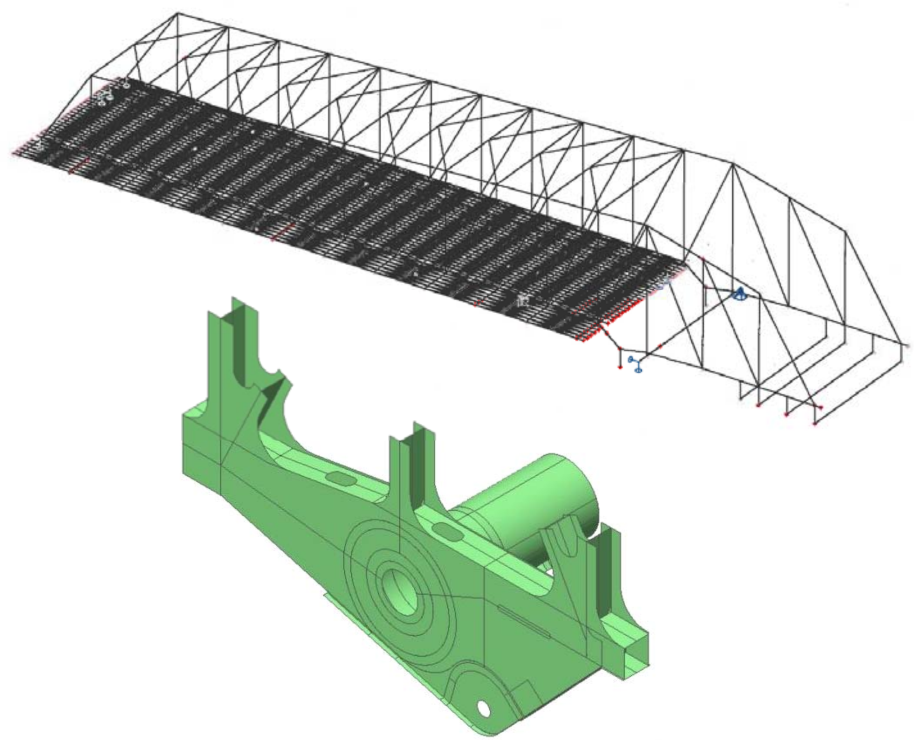


Approach

ARCADIS

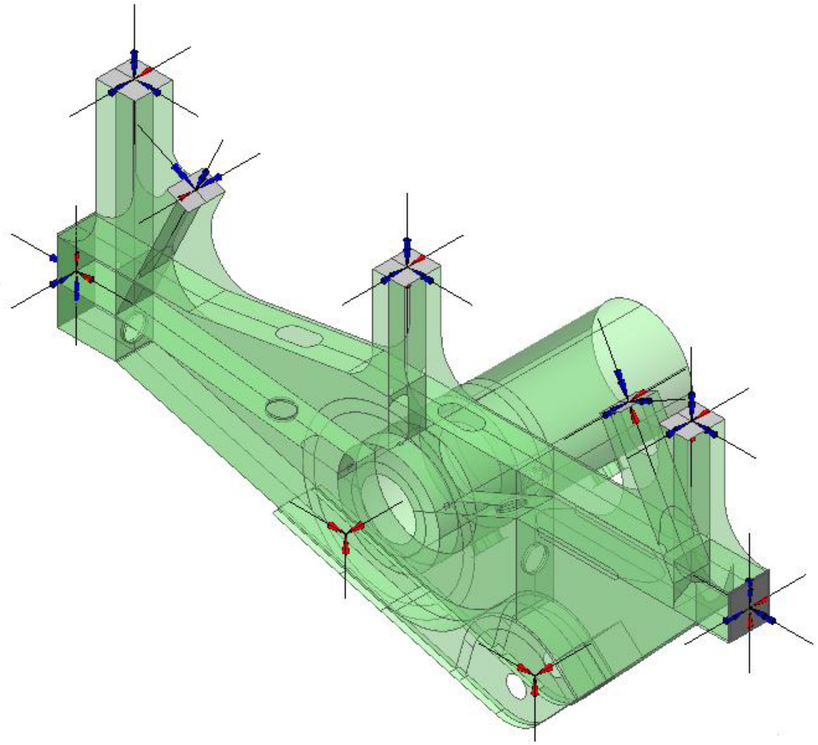
Step 1: global → local

- Translate global structural behaviour to local behaviour around pivot point
- Balanced load case on local model
- Supports to take unbalance



Step 1: global → local

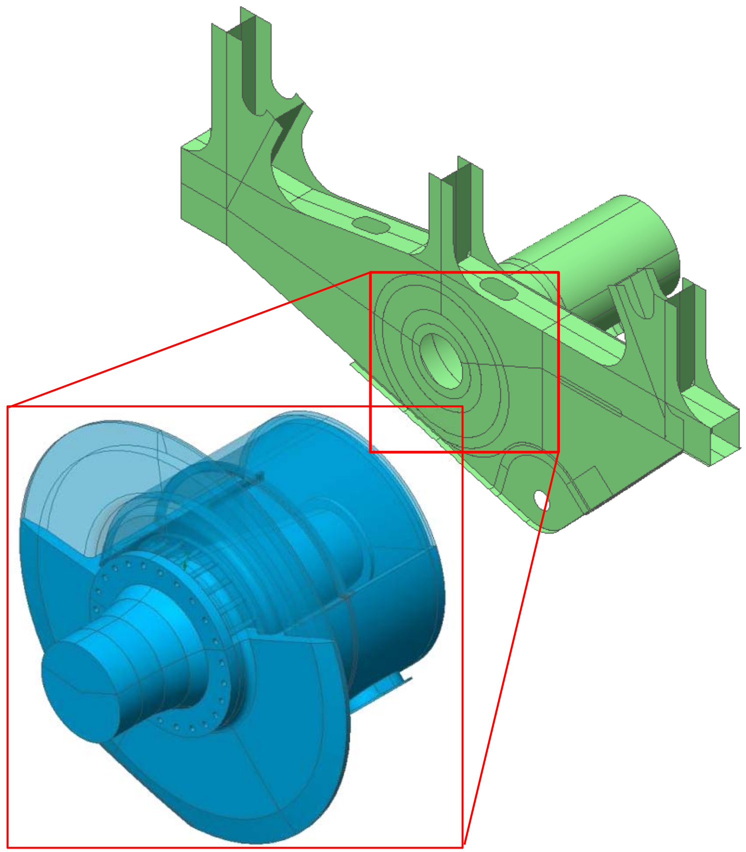
- Unit loads at model boundaries
- Member forces from global model → load factors in load combinations
- Fatigue verification of welds around pivot point



ARCADIS

Step 2: local → detail

- Translate local structural behaviour to correct stress state in detail model of cast piece
- Supports to take unbalance



Step 2: local → detail

- Stresses at sections in local model → loads in detail model
- Fatigue verification at fillets

Finite Element Models & Results

ARCADIS

Model 1: shells

- Purpose vs model specs
  - Provide proper boundary conditions for detail model
    - Geometry: global shape sufficient
    - Mesh: only fine elements in cast piece area
  - Enable detailed fatigue verification of welds around cast piece
    - Geometry: accurate, up to man and mouse holes
    - Mesh: fine elements at every critical spot

Model 1: shells

primary scope…

- Purpose vs model specs
  - Provide proper boundary conditions for detail model
    - Geometry: global shape sufficient
    - Mesh: only fine elements in cast piece area
  - Enable detailed fatigue verification of welds around cast piece
    - Geometry: accurate, up to man and mouse holes
    - Mesh: fine elements at every critical spot

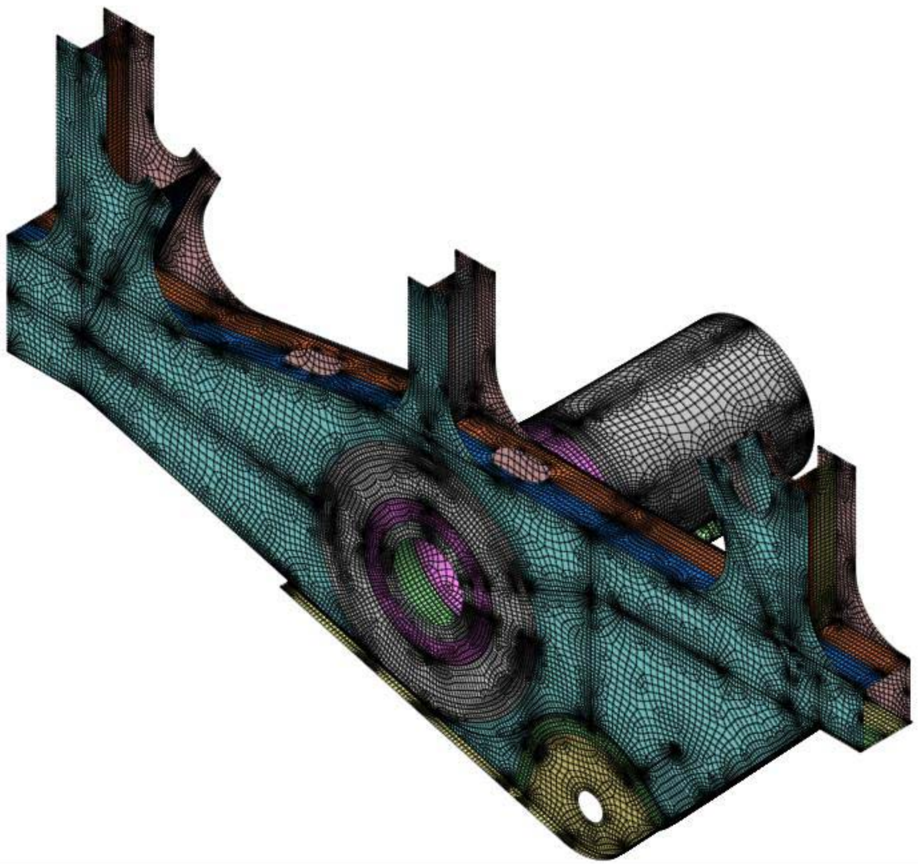
ARCADIS

Model 1: shells

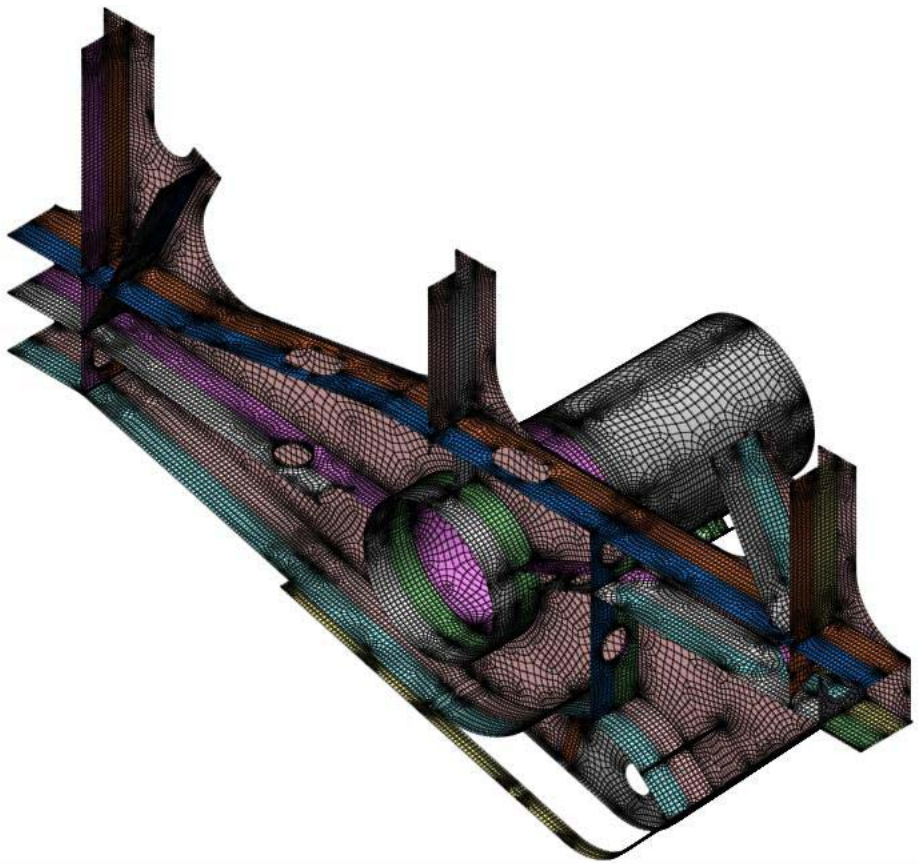
- Element size and grading optimum between
  - model size
  - peak stress accuracy

- Result
  - symmetrical linear grading 25 – 75 for every edge
  - ‘refinement’ factor 2

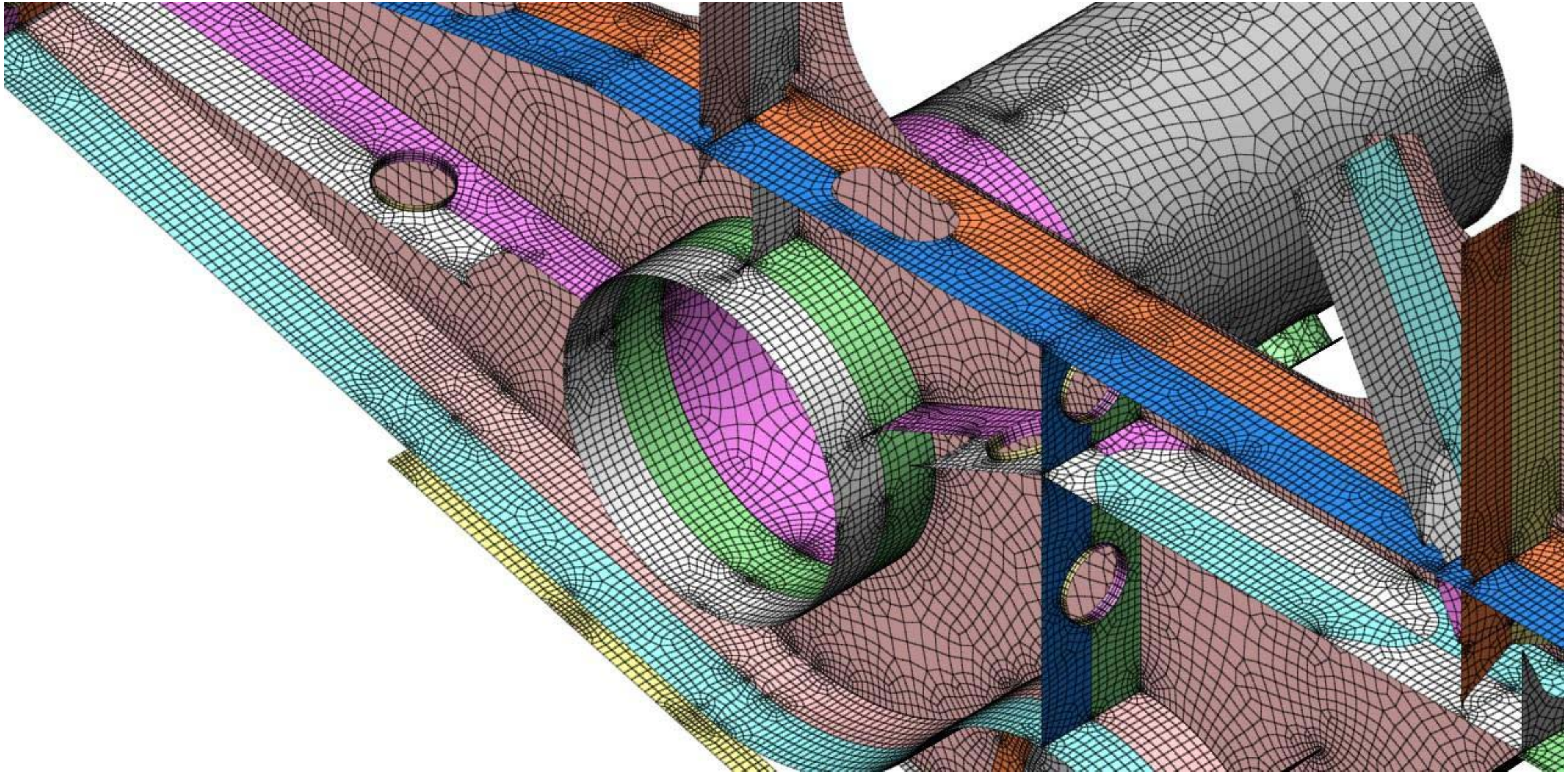
Model 1: shells



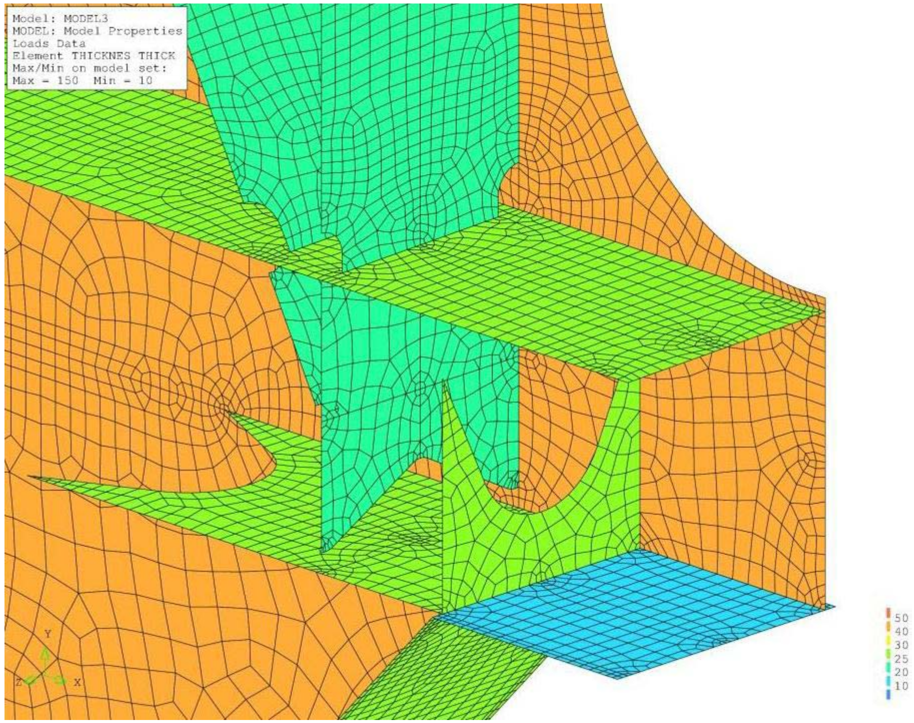
Model 1: shells

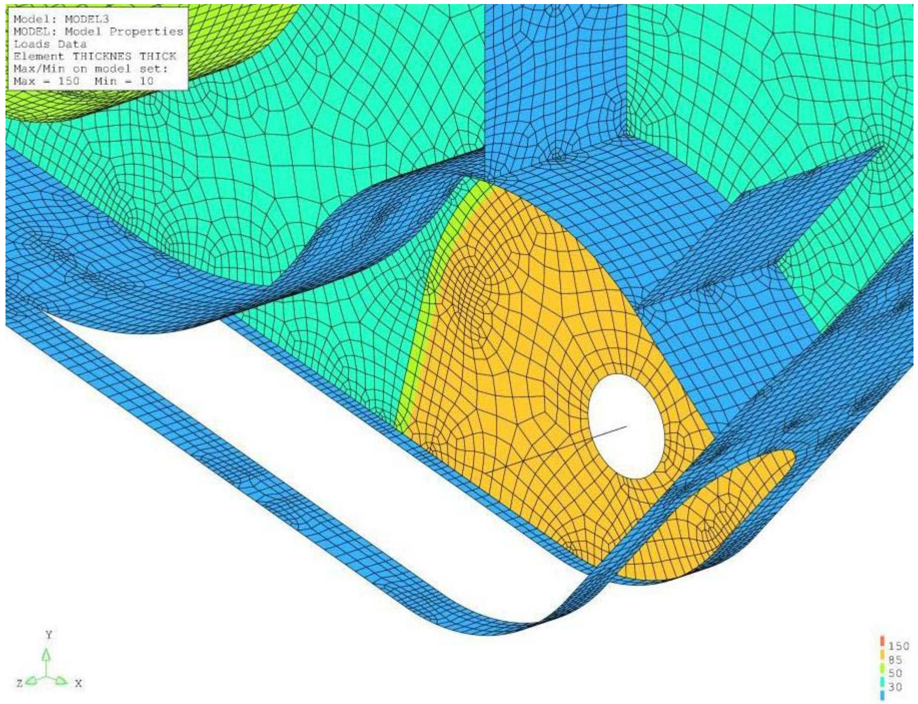


Model 1: shells

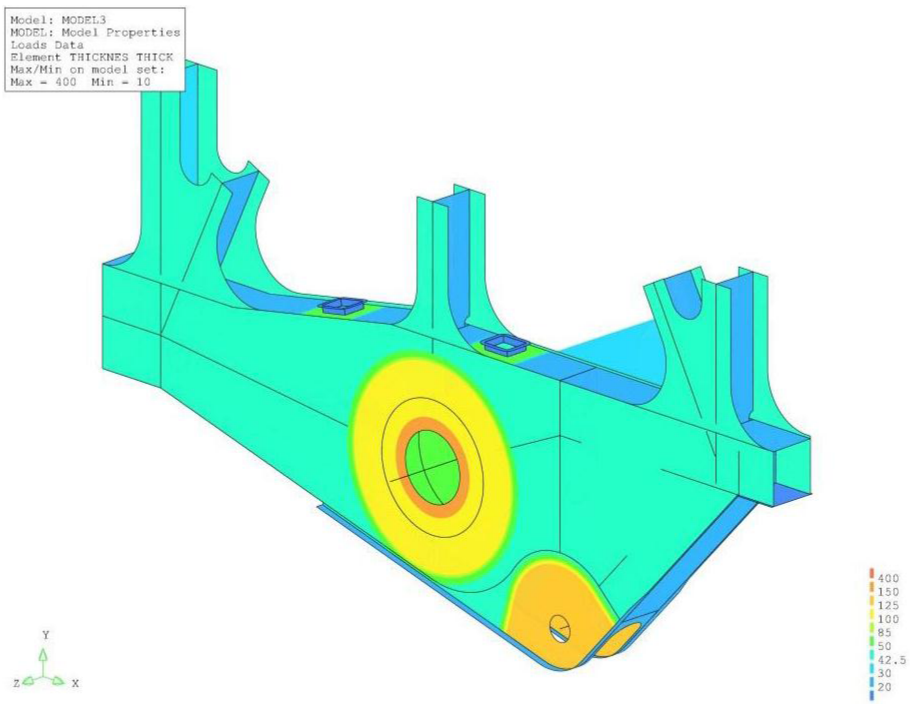


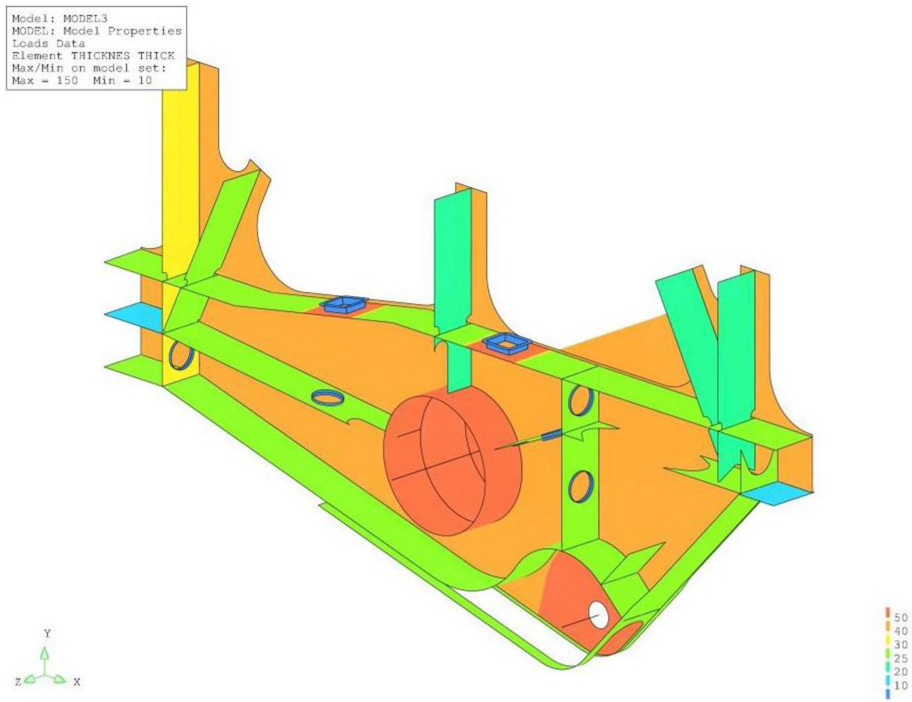
Model 1: shells





Model 1: shells



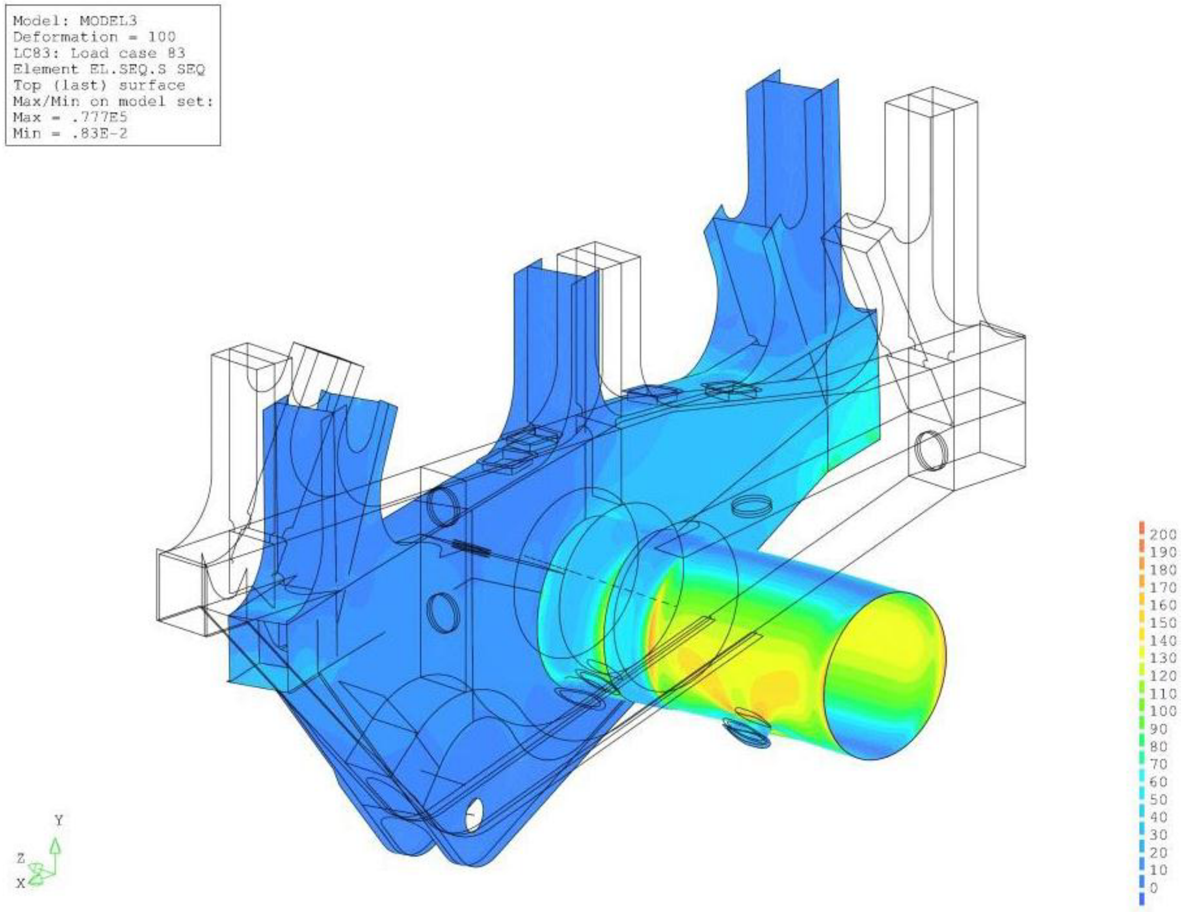


Model 1: shells

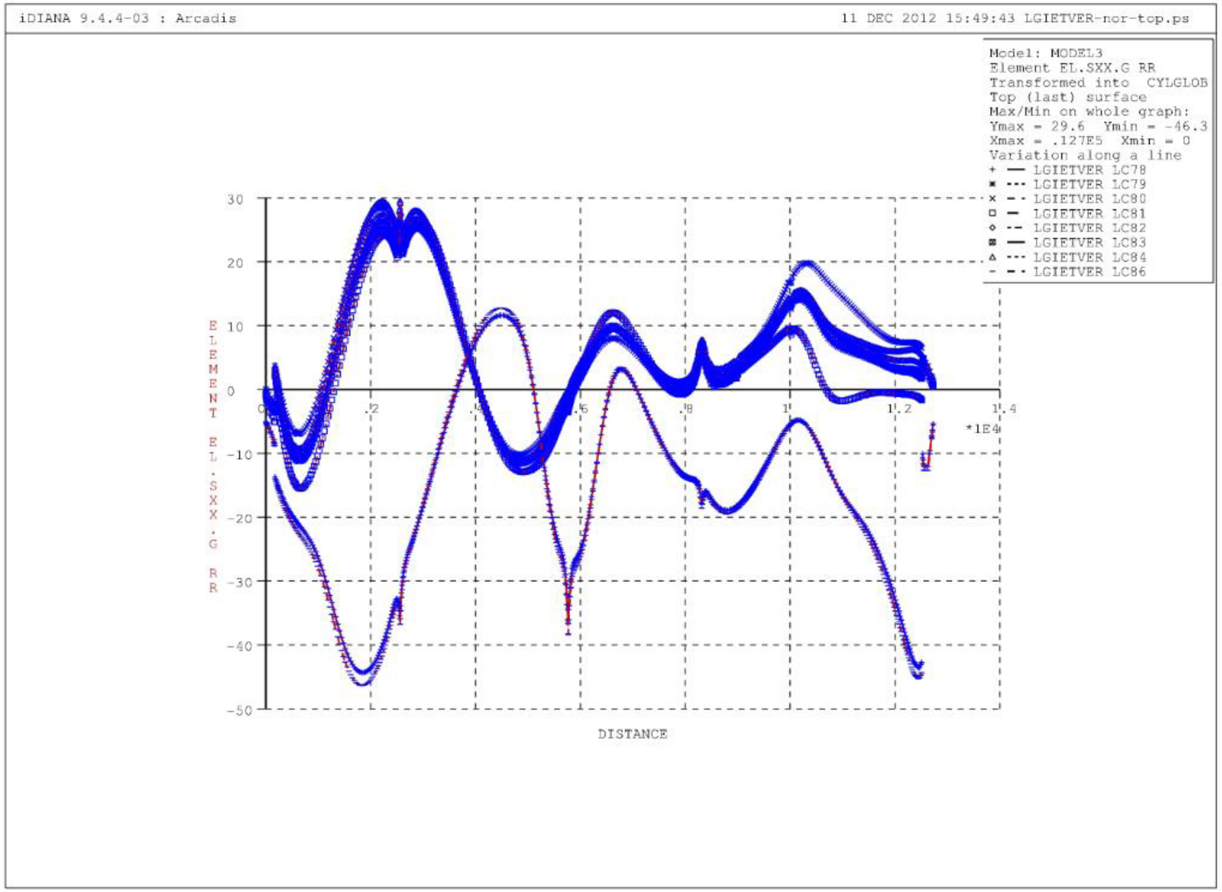
- Model verification
  - SOL for inidividual (unity) load cases
  - Deformations compared to Scia model

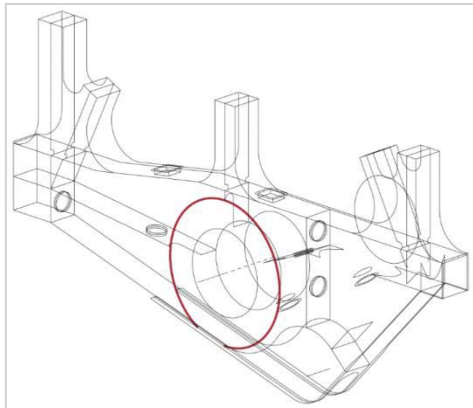
- Results
  - First impression based on Von Mises stresses and principal stresses
  - Fatigue verification based on stress range along section

Model 1: shells

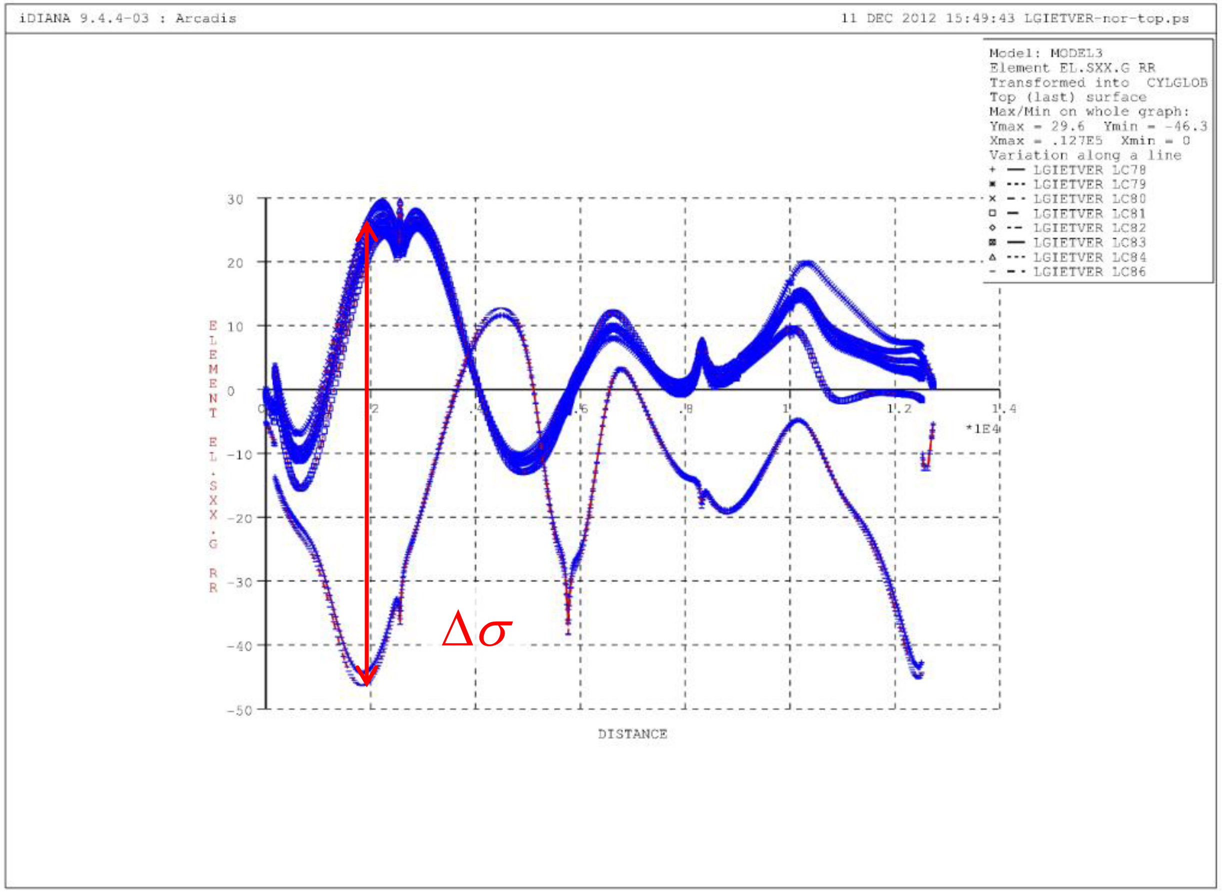


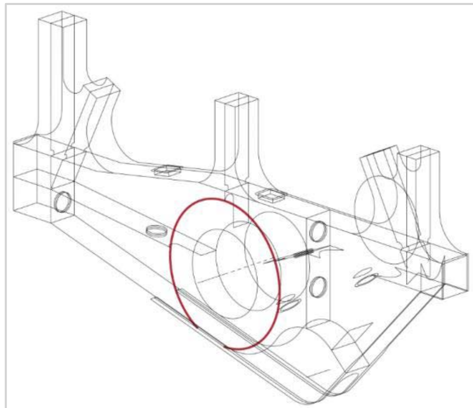
Model 1: shells



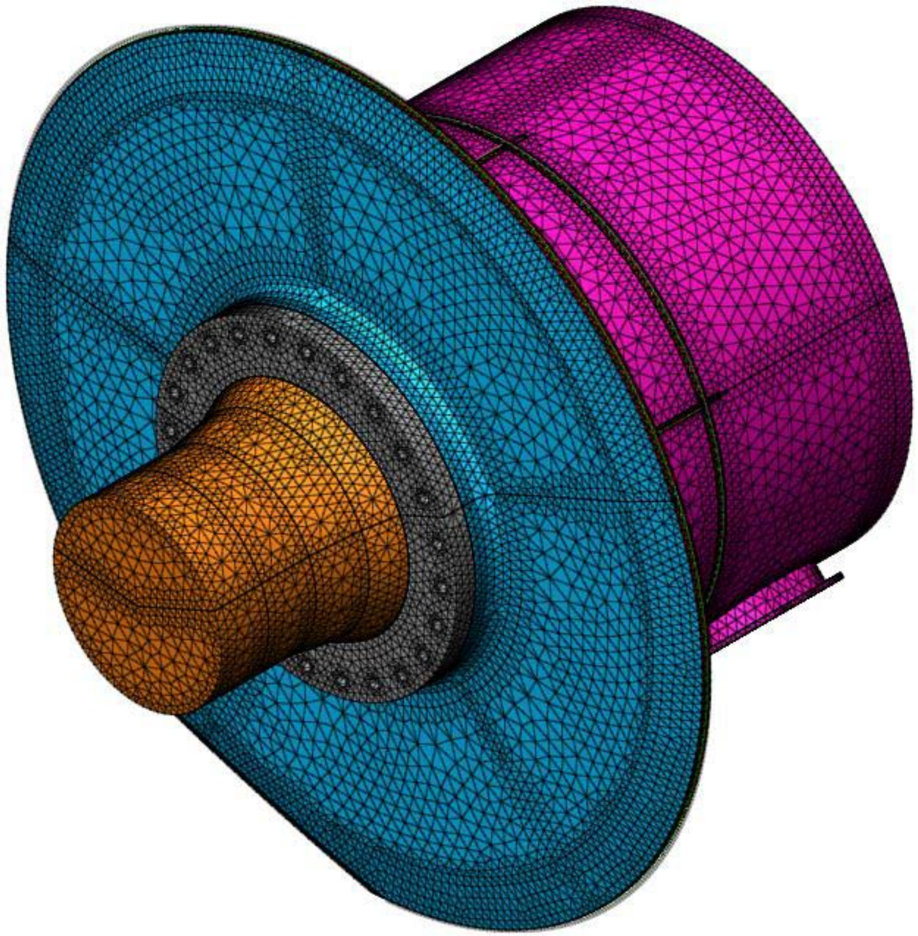


Model 1: shells

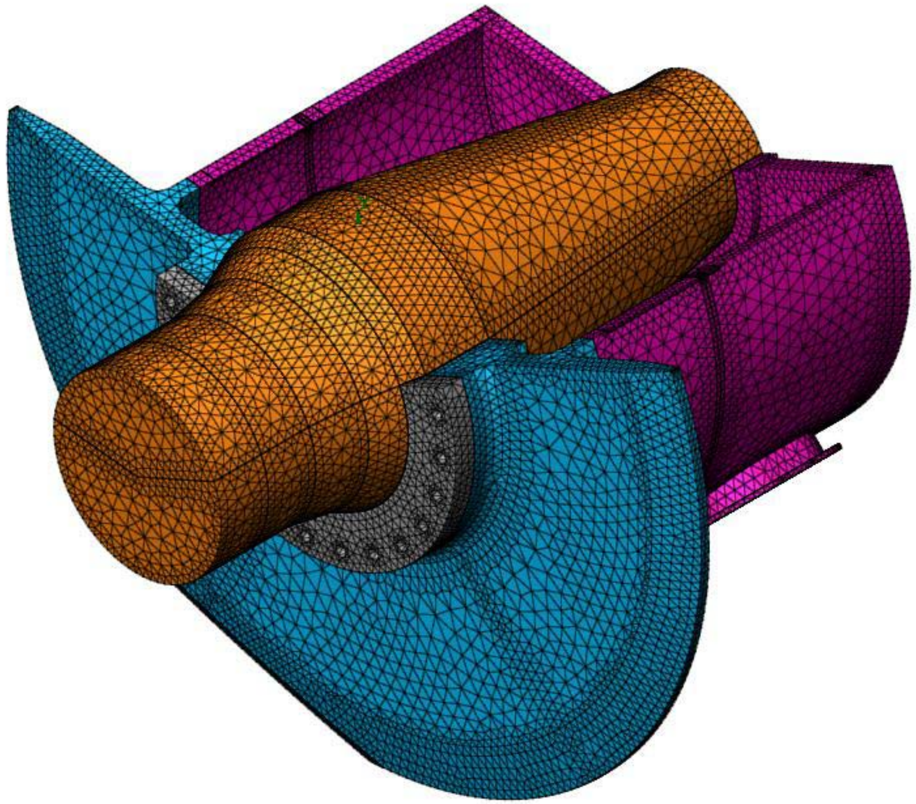




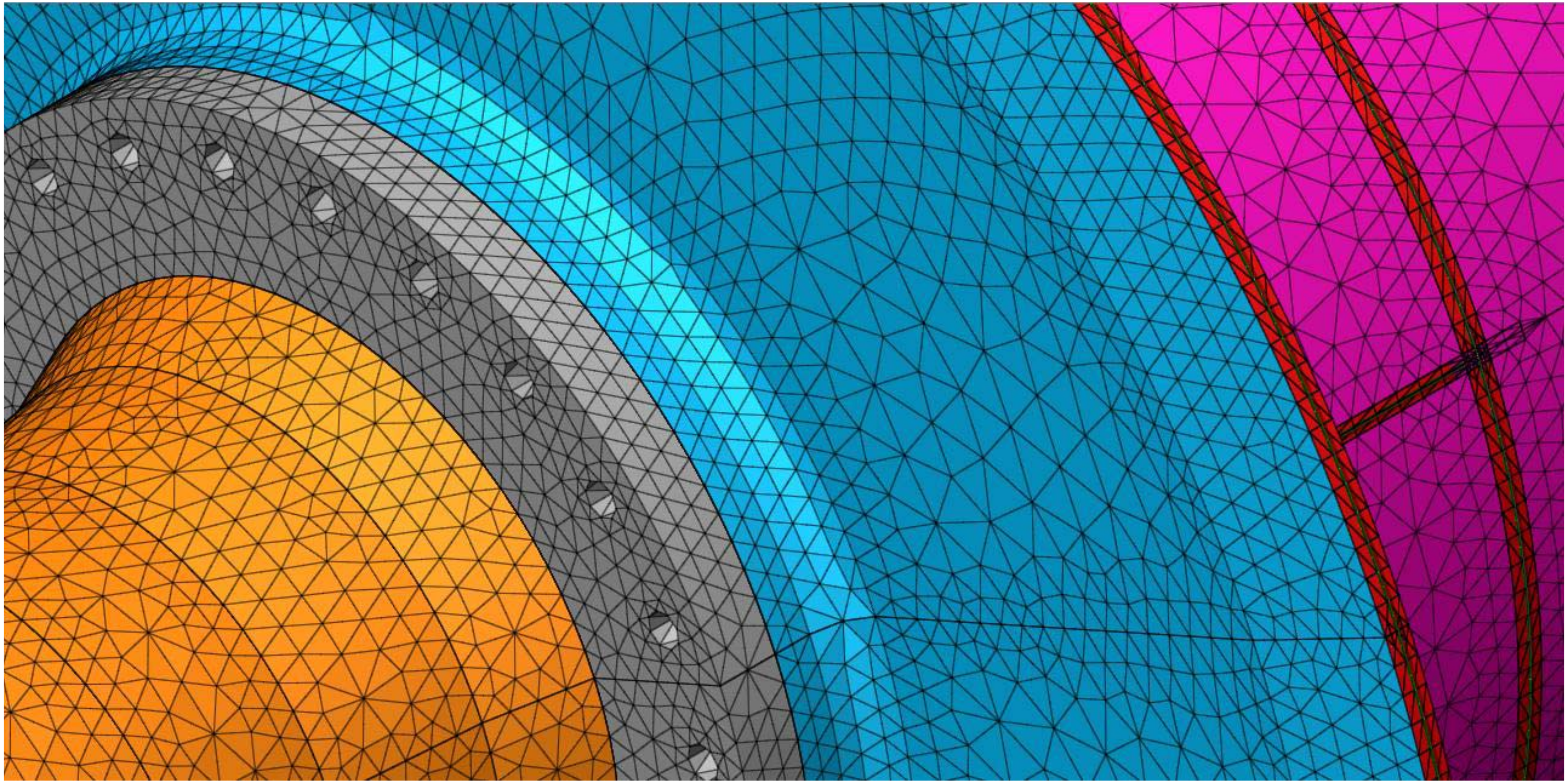
Model 2: solids



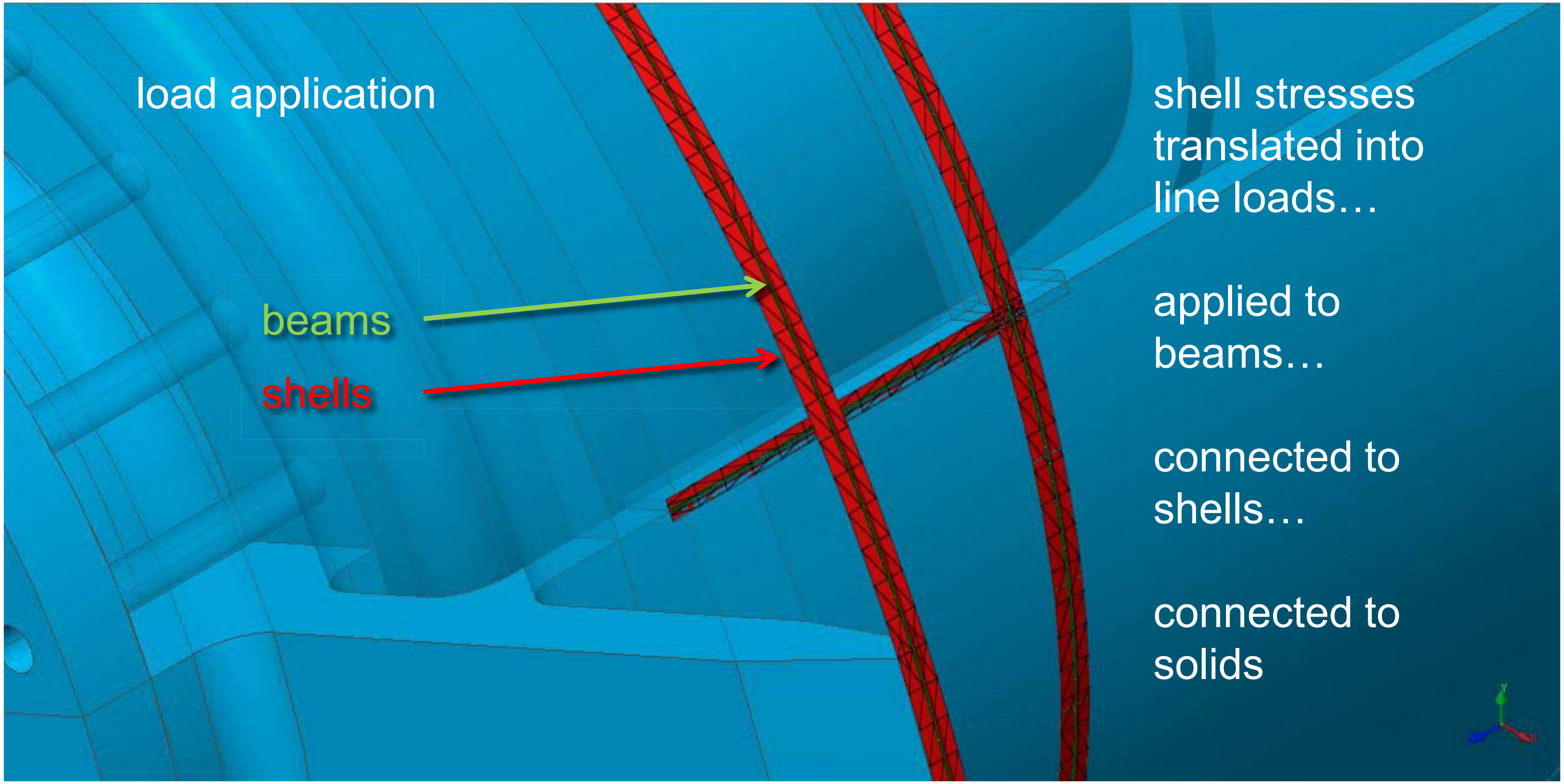
Model 2: solids



Model 2: solids



Model 2: solids



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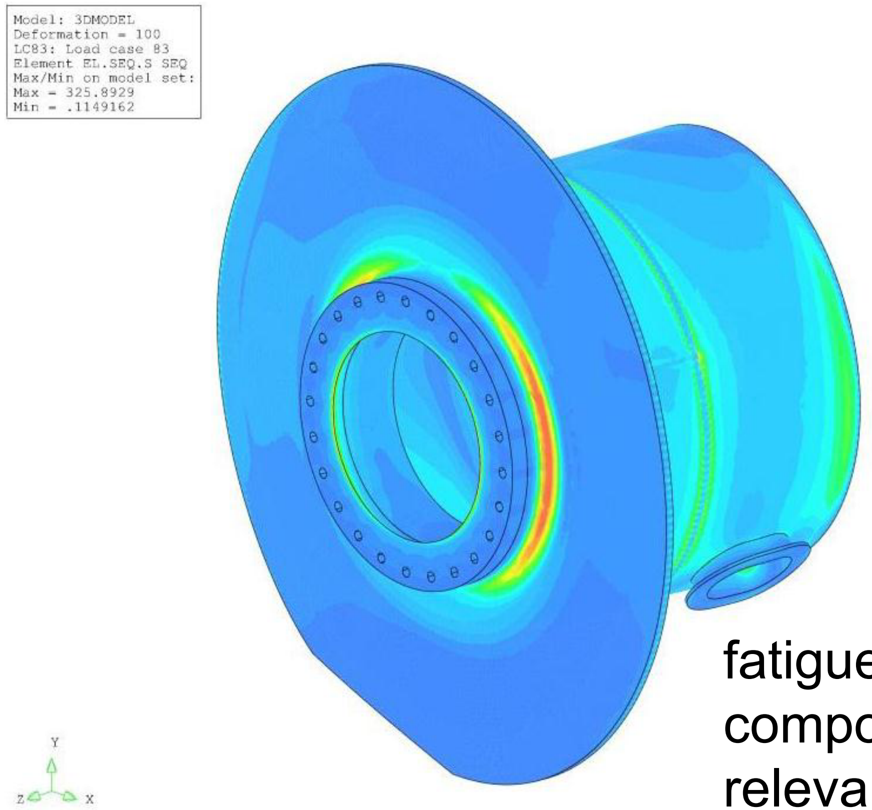
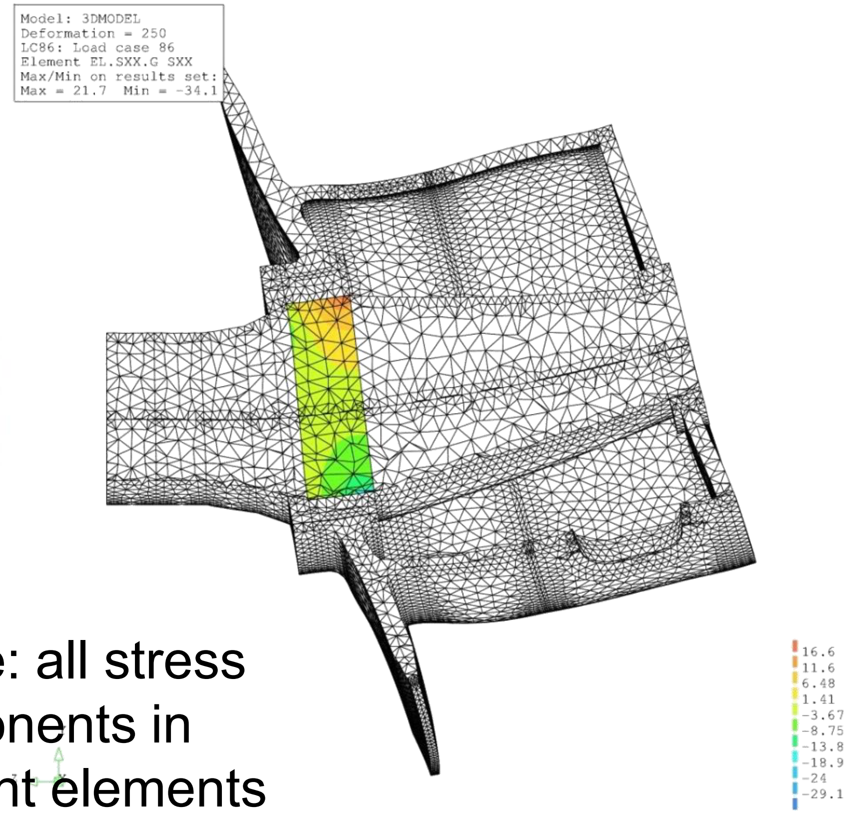
Model 2: solids





fatigue: all stress components in relevant elements

Fatigue Verification

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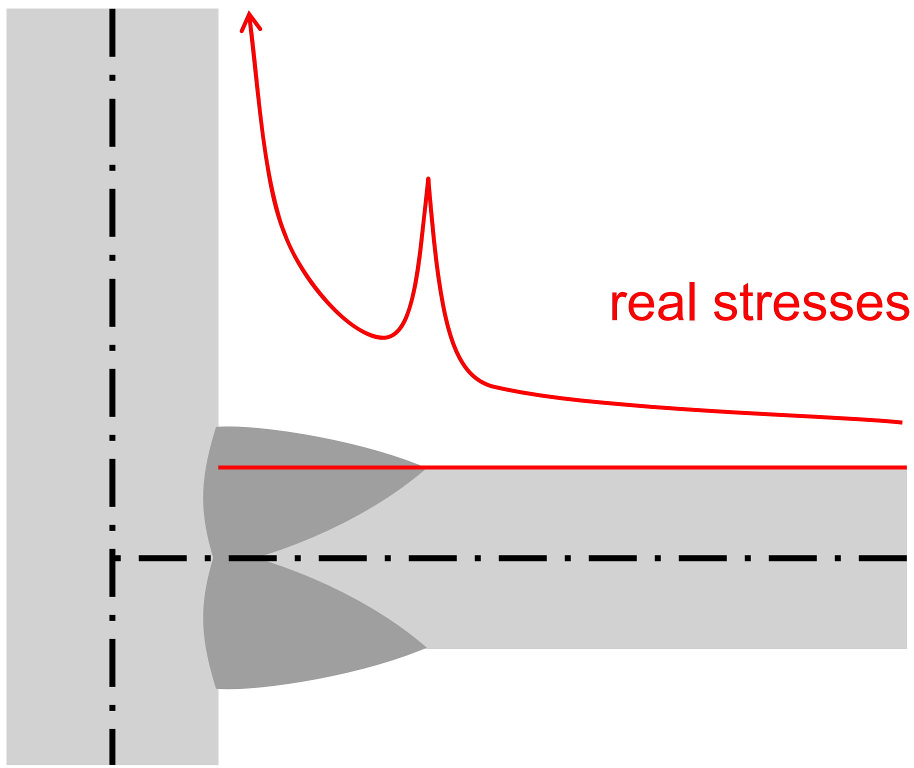
Fatigue verification

- Hot spot stress
  - realistic geometry (3D volume)
  - extrapolation of stress towards weld toe

- Nominal stress
  - ‘Bernouilli – models’
  - $\sigma = n / t + 6\, m / t^2$

Shell model vs reality



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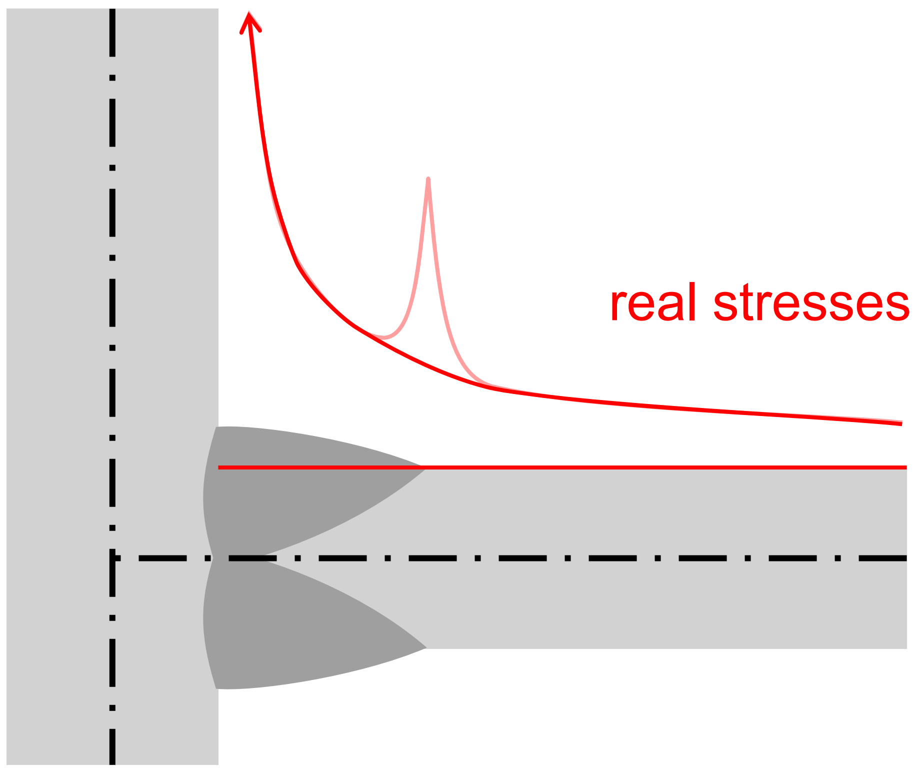
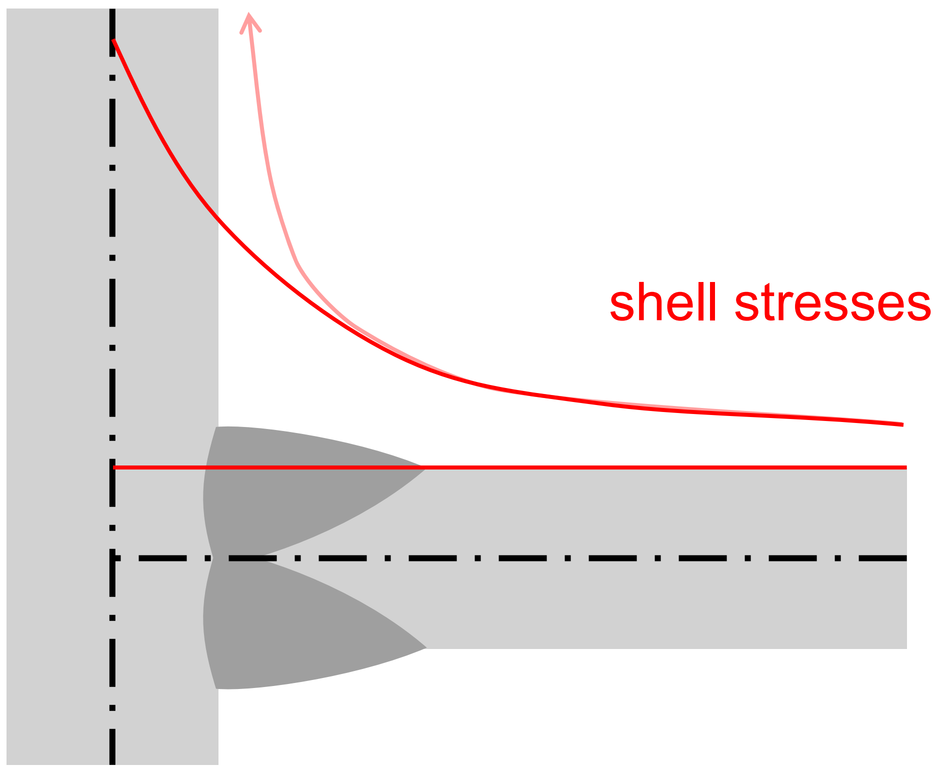
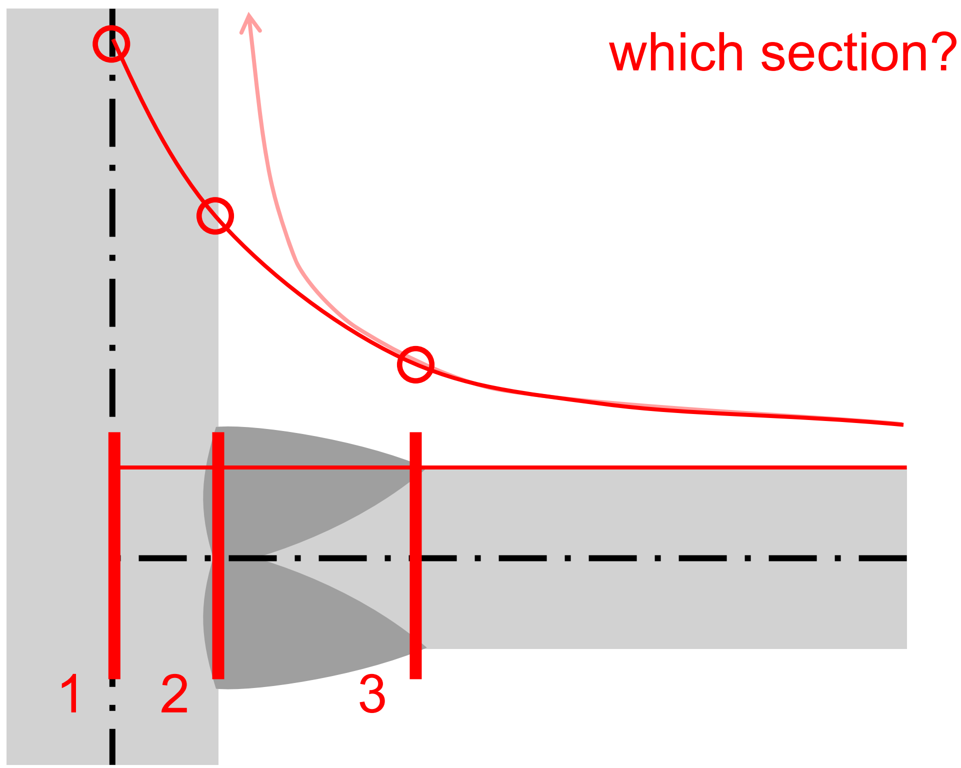
Shell model vs reality



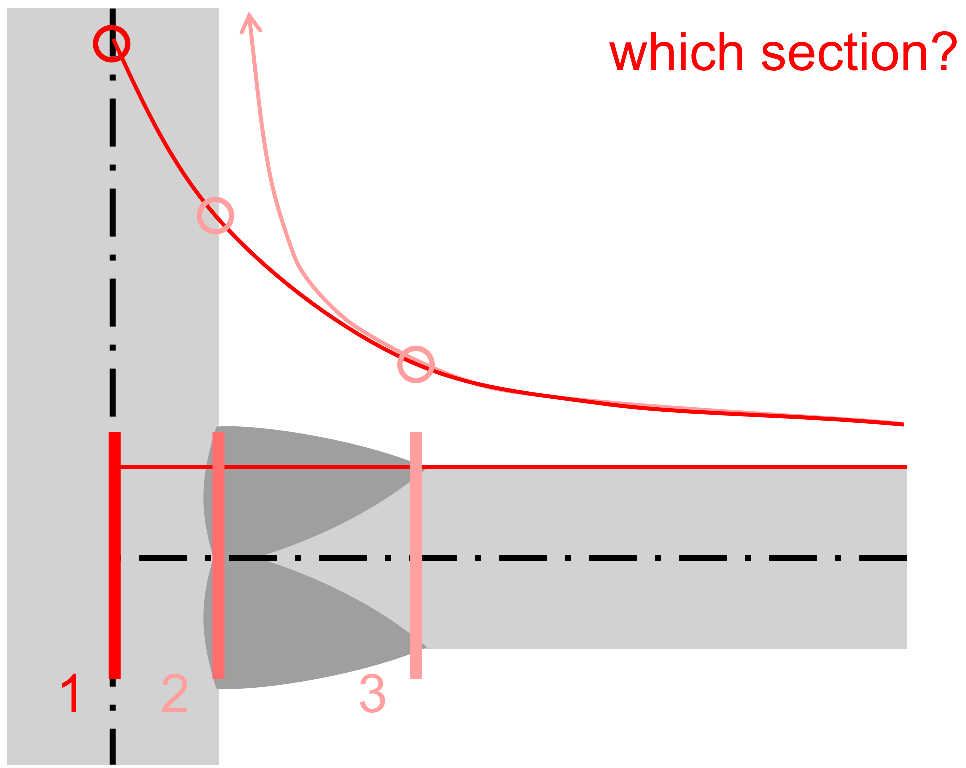
Shell model vs reality



Shell model vs reality

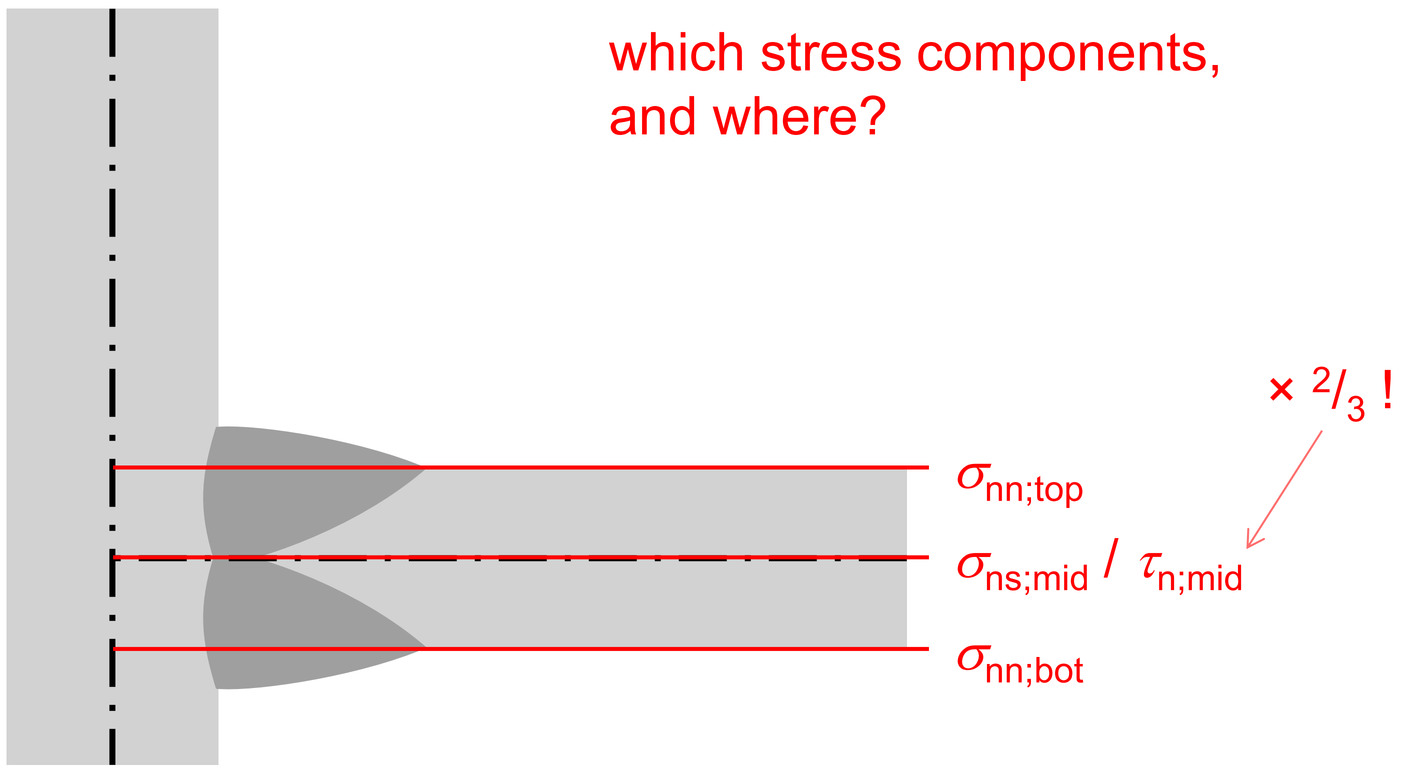


Shell model vs reality

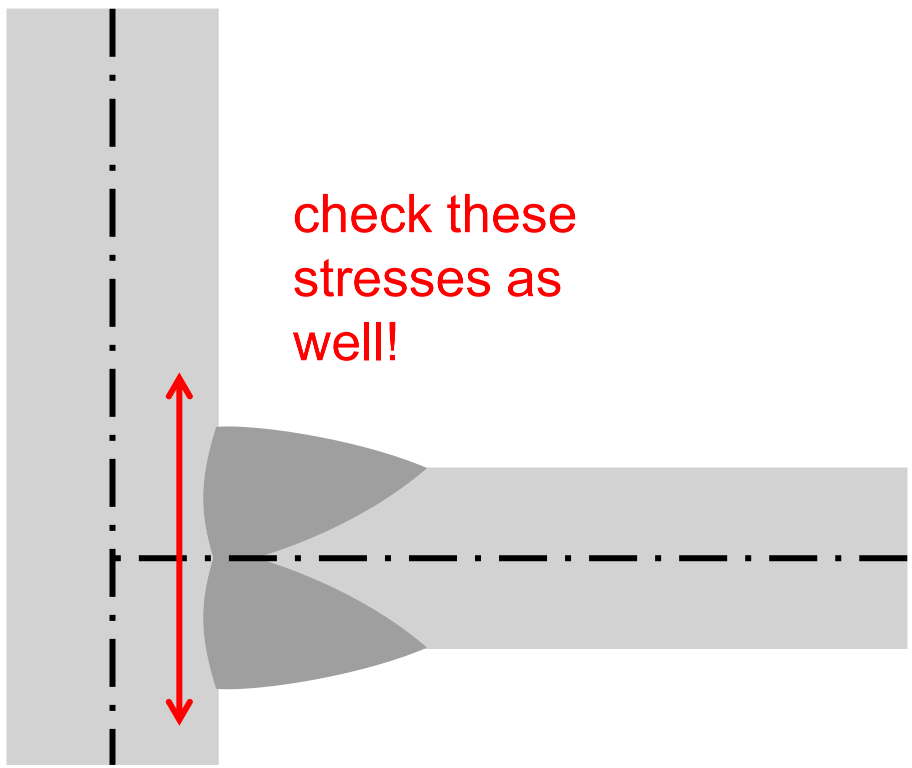


Shell model vs reality

which stress components,
and where?

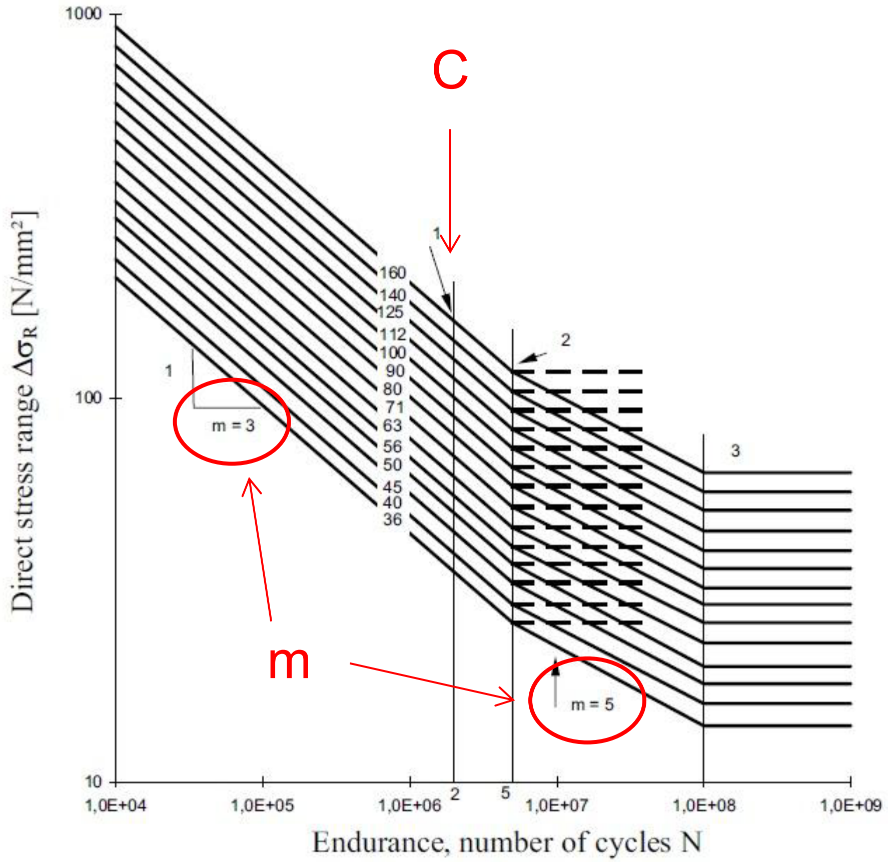


Shell model vs reality



Fatigue strength

- $N = C \times \Delta\sigma^{-m}$
- Opening and closing i.c.w. wind governing
- n = 310 000 cycles
- $n / N < 1$



Fatigue design

- $n / N > 1 \rightarrow$ design modification
- check consequences: update model
- iterative process

Wishful thinking

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This would be nice…

- automated determination of stress range $\Delta\sigma$
- cycle counter based on successive loadcases
- results in shifted section (e.g. from center line to weld toe)
- automated fatigue verification
  - base material
    - σ_{max} and σ_{min}
    - Strength pars f_{fat} and m
    - Result: n / N
  - along welds…

This would be nice…

- automated fatigue verification
  - along welds
    - weld modelled as ‘shell-interface’ (already present in DIANA)
    - detail category as material parameter
    - stress components normal and tangential to weld
    - result: n / N

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Imagine the result

8th International
DIANA Users Meeting

25-26 April 2013

Chalmers University of Technology
Gothenburg - Sweden

Study of the behaviour of reinforced concrete slabs subjected to concentrated loads near supports

B. BELLETTI*, **C. DAMONI***, **M.A.N. HENDRIKS #**

* University of Parma, Italy

#Norwegian University of Science and Technology, Norway

+ Delft University of Technology, The Netherlands

NTNU Norwegian University of Science and Technology

TUDelft Delft University of Technology

OUTLINES

Background and aim of the research

Cases studies analyzed: experimental program

Finite element model

Evaluation of the carrying capacity of the RC slabs analyzed:

- ***One way shear resistance***
  (Model Code 2010)
- ***Punching resistance***
  (Eurocode2, Regan's formulation, CSCT)
- ***Bending resistance***
  (Yield line theory)
- ***Nonlinear finite element analyses***
  -Sensitivity study
  -Levels of Approximation Model Code 2010

Final remarks

Background and aim of the research

The **Dutch Ministry of Infrastructure and the Environment** initiated a project to r**e-evaluate** the **carrying capacity** of **existing bridges** and **viaducts**

Why?

- increased traffic in the last years
- reallocation of emercency lanes to traffic lanes
- need for additional traffic lanes

For a certain amount of Dutch bridges and other infrastructures the safety verifications are not satisfied if the usual analytical procedures are adopted

↓

The Dutch Ministry of Infrastructure and the Environment proposed to make a **structural assessment** of **existing structures** through the use of **nonlinear finite element analyses**

↓

Concern about the (shear) bearing capacity of Dutch bridges

Bridges in highways	1515
Bridges in main roads	930
Other bridges	711
Tunnels	544
Aquaducts	8
Ecoducts	7
Alltogether	3715

Joost Walraven – Salò, 25 October 2010.

Background and aim of the research

Final release of a document containing **guidelines for nonlinear finite element (NLFE) analyses of reinforced and prestressed concrete elements**

Rijkswaterstaat
Ministerie van Infrastructuur en Milieu

Rijkswaterstaat Technisch Document (RTD)

Guidelines for
Nonlinear Finite Element Analysis
of Concrete Structures

Doc.nr.: RTD 1016:2012
Versie: 1.0
Status: Definitief
Datum: 2012

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CONTENTS

Background and aim of the research

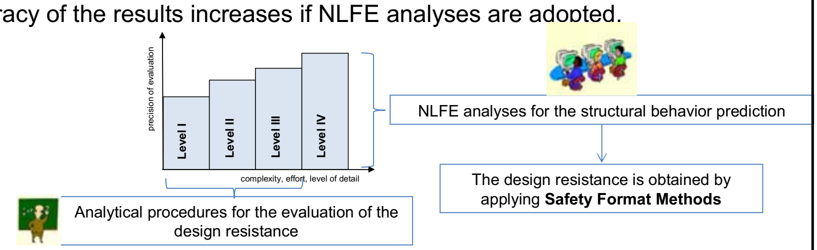
NLFEA represent a powerful instrument for structural assessment

The results of NLFEA strongly depend on the modelling choices
→ big **scatter in the results** for the same structure analyzed by several analysts

Guidelines for NLFEA to be followed by all users in order to obtain reliable and safe results is of big importance

- The project well matches with the philosphy of the **Model Code 2010**:

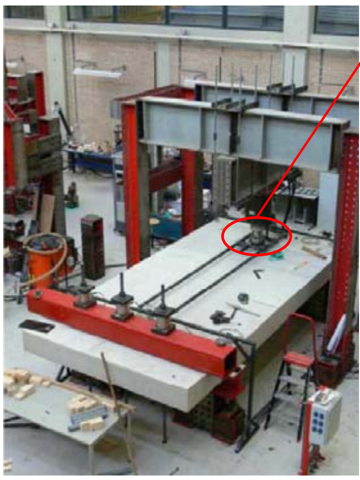
  Carrying capacity of concrete elements evaluated with different **Levels of Approximation** adopting analytical calculations and NLFE analyses: the accuracy of the results increases if NLFE analyses are adopted.

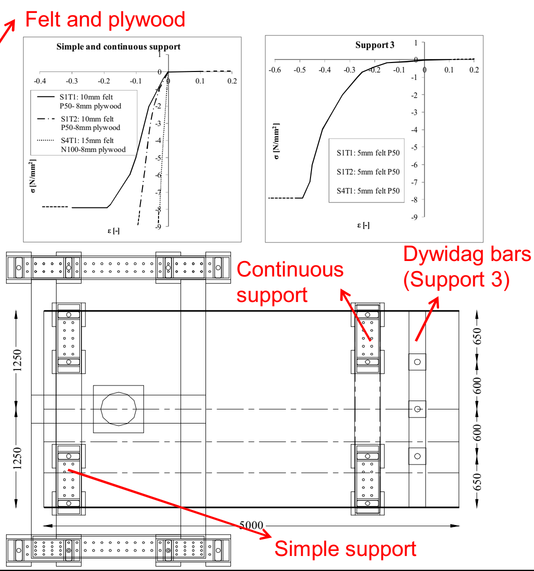


- The carrying capacity of 3 RC slabs tested at TuDelft has been evaluated with:
  -NLFEA (following the Dutch guidelines)
  -analytical calculations

Cases studies analyzed: experimental program

18 slabs tested *2500 x 5000 x 300 mm* → 3 chosen as cases studies: **S1T1**, **S1T2**, **S4T1**



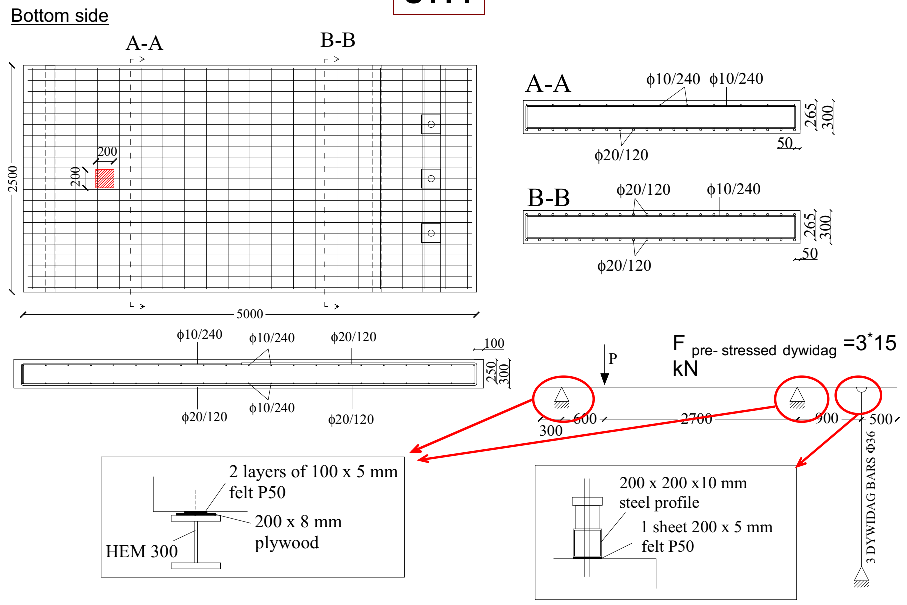


Lantsoght, E., 2012. Shear tests of Reinforced Concrete Slabs Experimental data of Undamaged Slabs 06-01-2012. Technical Report, Stevin laboratory, Delft University of Technology.

TU Delft

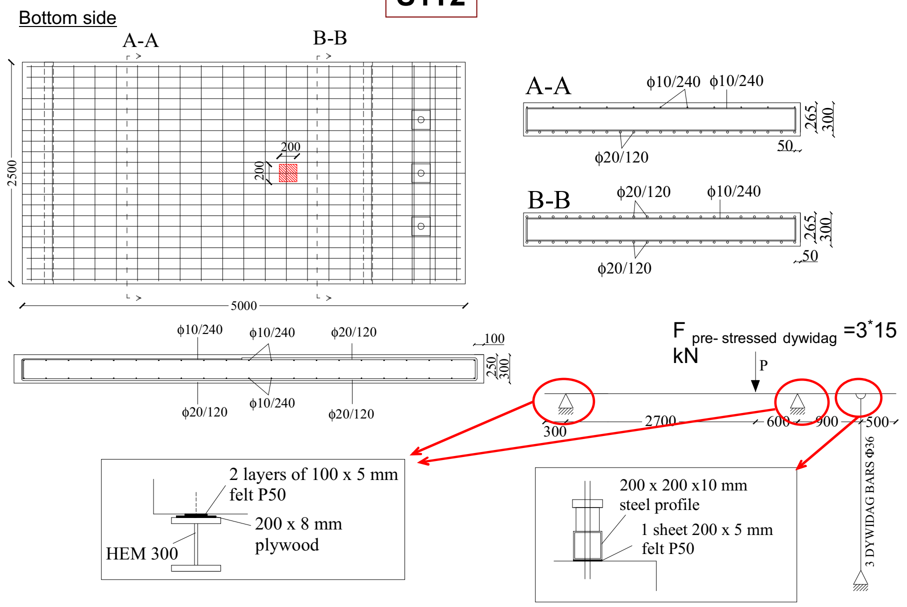
Cases studies analyzed: experimental program

S1T1



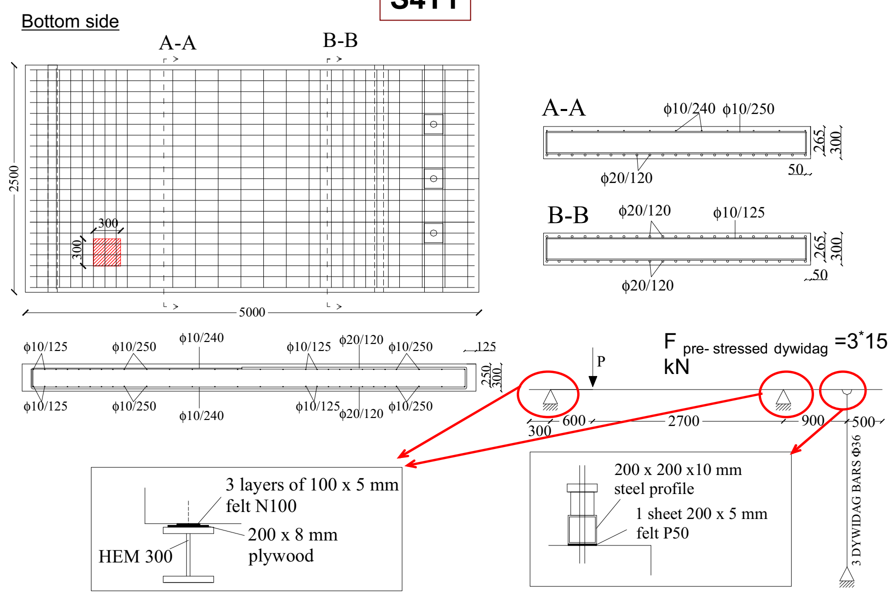
Cases studies analyzed: experimental program

S1T2



Cases studies analyzed: experimental program

S4T1



Cases studies analyzed: experimental program

CRACK PATTERN AT FAILURE, FAILURE MODE AND ULTIMATE LOAD

S1T1		S1T2		S4T1	
E_c:	30910 Mpa	E_c:	30910 Mpa	E_c:	34930 Mpa
f_c:	29.7 Mpa	f_c:	29.7 Mpa	f_c:	42.9 Mpa
f_t:	2.8 MPa	f_t:	2.8 MPa	f_t:	3.8 MPa
Failure:	one-way shear	Failure:	one-way shear	Failure:	one-way shear
$P_{u,exp}$:	954 kN	$P_{u,exp}$:	1023 kN	$P_{u,exp}$:	1160 kN

Evaluation of the carrying capacity of the slabs

BEFORE STARTING WITH THE MODELLING IT IS IMPORTANT TO «FEEL», THANK TO CIVIL ENGINEERING KNOWLEDGE, WHICH SHOULD BE THE STRUCTURAL BEHAVIOUR

Analytical calculation

One-way shear resistance → Model Code 2010 Level I- II Approximation

Punching resistance →
- Eurocode 2
- Regan's formulation
- Critical Shear Crack Theory (Muttoni)

Bending resistance → Yield line method

Numerical procedure

NLFE analyses → **Sensitivity study** → Model Code 2010 Level IV Approximation

-**shell / brick** model
-**parametric study** on sensitive parameters of the crack model

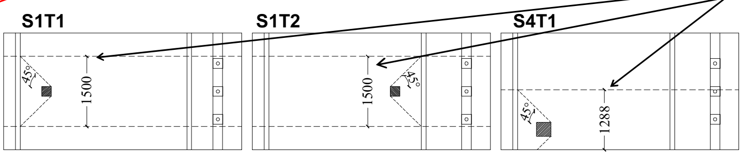
Safety format methods for NLFE analyses
in order to reduce the peak load values to obtain the **DESIGN SHEAR RESISTANCE**

Evaluation of the carrying capacity of the slabs

Analytical calculation: One-way shear resistance

Model Code 2010

RC slabs treated as **beams without shear reinforcement** with an **effective width b_{eff}**



Level I and **II Approximation** provided by **Model Code 2010**

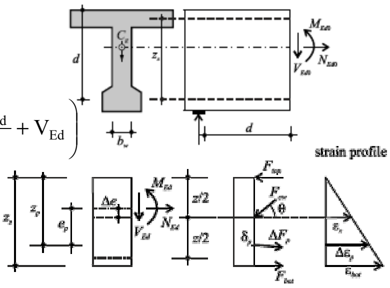
Design shear resistance

$$V_{Rd,c} = k_v \frac{\sqrt{f_{ck}}}{\gamma_c} z b_{eff}$$

$$\begin{cases} k_{v,\text{Level I}} = \dfrac{180}{1000 + 1.25z} \\ k_{v,\text{Level II}} = \dfrac{0.4}{1 + 1500\varepsilon_x} \cdot \dfrac{1300}{1000 + k_{dg} z} \end{cases}$$

$$\varepsilon_x = \frac{1}{2E_s A_s}\left(\frac{M_{Ed}}{z_1} + V_{Ed}\right)$$

$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75$$



Evaluation of the carrying capacity of the slabs

Analytical calculation: Punching resistance

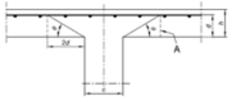
Eurocode 2

$$V_{Rdc} = v_{Rdc} \cdot u \cdot d_{eff}$$

$$v_{Rdc} = C_{Rd,c} k (100 \cdot \rho \cdot f_{ck})^{1/3}$$

$$C_{Rd,c} = 0.18 / \gamma_c$$

$$k = 1 + \sqrt{\frac{200}{d_{eff}}} \qquad \rho = \sqrt{\rho_l \cdot \rho_t}$$



Regan

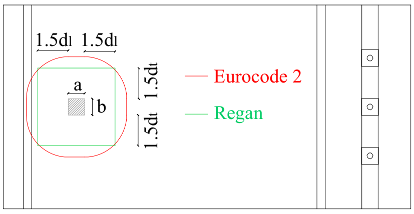
$$P_R = P_{R1} + P_{R2}$$

$$\begin{cases} P_{R1} = \xi_{sl} \cdot v_{cl} \cdot u_2 \cdot d_l + 2 \cdot \xi_{st} \cdot v_{ct} \cdot u_l \cdot d_t \\ P_{R2} = \dfrac{2d_l}{a_v} \cdot \xi_{sl} \cdot v_{cl} \cdot u_2 \cdot d_l \end{cases}$$

$$\xi_{sl} = \sqrt[4]{\frac{500}{d_l}} \qquad v_{cl} = 0.27 \cdot \sqrt[3]{100 \rho_l f_{ck}}$$

$$\xi_{st} = \sqrt[4]{\frac{500}{d_t}} \qquad v_{ct} = 0.27 \cdot \sqrt[3]{100 \rho_t f_{ck}}$$

control perimeter



$$u = 2 \cdot a + 2 \cdot b + 4\pi d_{eff}$$

$$u_l = 1.5 d_l + a + 1.5 d_l$$

$$u_2 = 1.5 d_t + b + 1.5 d_t$$

Evaluation of the carrying capacity of the slabs

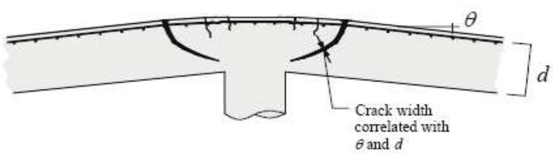
Analytical calculation: Punching resistance

S1T1

Critical Shear Crack Theory (CSCT)

Muttoni A. (2008) "Punching Shear Strength of Reinforced Concrete Slabs without Transverse Reinforcement," *ACI Structural Journal*, V. 105, No. 4, 440-450.

rotation θ of the slab chosen as control parameter since the opening of the critical shear crack reduces the strength of the inclined concrete compression strut that carries shear and eventually leads to the punching shear failure



Failure criterion

$$\frac{\tau_R}{\tau_c} = \frac{V_R}{\tau_c \cdot u \cdot d} = \frac{1}{0.4 + 0.125 \cdot \theta \cdot d \cdot k_{dg}}$$

MAIN STEPS FOLLOWED:

1. An linear elastic (LE) finite element analysis (mesh with shell elements) carried out to determine the maximum shear force $v_{max,el}$ and the control perimeter u;
2. a NLFE analysis (mesh with shell elements) carried out to determine the rotation θ;
3. from the intersection between the results of NLFE analysis and the failure criterion the failure load is determined.

Evaluation of the carrying capacity of the slabs

S1T1

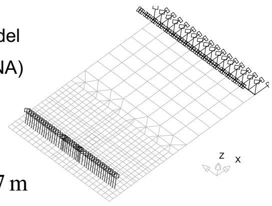
Analytical calculation: Punching resistance

CSCT

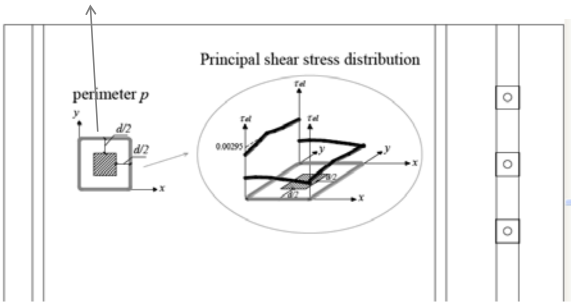
1. Calculation of maximum shear force $v_{max,el}$ and control perimeter *u*

2D finite element model

(Software used: DIANA)



$$d = \sqrt{d_x \cdot d_y} = 0.257\,m$$



Principal shear stress detected from **LE analysis**:

$$v_{max,el} = 2950 \cdot 0.257 = 759.3\,N/m$$

Control perimeter u:

$$u = \frac{Q}{v_{max,el}} = \frac{1000}{759.3} = 1.32\,m$$

Applied load in LE analysis

Evaluation of the carrying capacity of the slabs

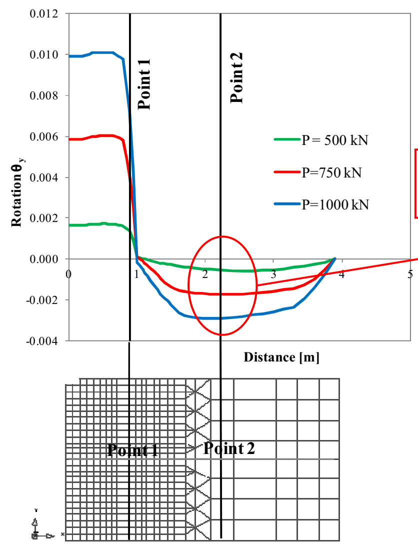
S1T1

Analytical calculation: Punching resistance

CSCT

NLFLE analysis

2. Evaluation of rotation θ



Rotation θ = **difference** between the rotations of the slab at two points: **Point 1** and **Point 2.**

Point 1: located at the centroid of the applied load.

Point 2: chosen so that the maximal relative rotation is obtained .

is not the same at each load step

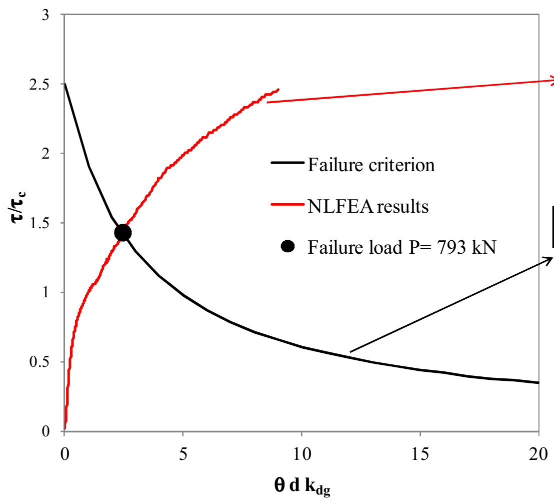
⇩

Arbitaly chosen θ_{max} at P=750 kN

Evaluation of the carrying capacity of the slabs

S1T1 | **Analytical calculation: Punching resistance** | **CSCT**

3. Determination of failure load



NLFE analysis

$$\tau / \tau_c = P / (u \cdot d \cdot 0.3 \cdot \sqrt{f_c}) = P / (1320 \cdot 257 \cdot 1.63)$$

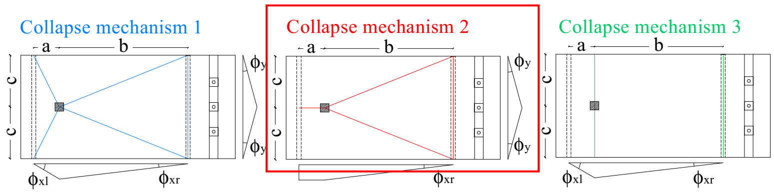
Failure criterion

$$\tau_R / \tau_c = 1 / (0.4 + 0.125 \cdot \theta \cdot d \cdot k_{dg})$$

$P_{failure}$=793 kN

Evaluation of the carrying capacity of the slabs

S1T1 | **Analytical calculation: Bending resistance** | **Yield line method**



Collapse mechanism 1

$$L_{Vi} = m_x^+ \cdot 2c \cdot \phi_{xl} + m_x^+ \cdot 2c \cdot \phi_{xr} + m_x^- \cdot 2c \cdot \phi_{xr} + 2m_y \cdot a \cdot \phi_y + 2m_y \cdot b \cdot \phi_y = L_{ve} = P_{c1}\delta \longrightarrow P_{c1}=2198.22\text{kN}$$

Collapse mechanism 2

$$L_{Vi} = m_x^+ \cdot 2c \cdot \phi_{xr} + m_x^- \cdot 2c \cdot \phi_{xr} + 2m_y \cdot a \cdot \phi_y + 2m_y \cdot b \cdot \phi_y = L_{ve} = P_{c2}\delta \longrightarrow \mathbf{P_{c2}=790.7\ kN}$$

Collapse mechanism 3

$$L_{Vi} = m_x^+ \cdot 2c \cdot \phi_{xl} + m_x^+ \cdot 2c \cdot \phi_{xr} + m_x^- \cdot 2c \cdot \phi_{xr} = L_{ve} = P_{c3}\delta \longrightarrow P_{c3}=1970.5\text{ kN}$$

Yield pattern of collapse mechanism 2 in agreement with experimental crack patetrn

Different collapse mechanisms considered, according to the kinematic theorem of limit analysis

Evaluation of the carrying capacity of the slabs

Analytical results

Measured mechanical properties used in calculations (γ_c=1, γ_s=1)

Ultimate load P values [kN]

	Experiments	Analytical calculations					
	One-way shear failure	One-way shear		Punching			Bending
		Level I	Level II	Regan	EC2	CSCT	Yield line
S1T1	954.0	307.3	536.5	708.2	698.1	793	790.7
S1T2	1023.0	242.4	734.1	708.2	698.1	-	-
S4T1	1160.0	344.6	559.4	769	716.4	-	-

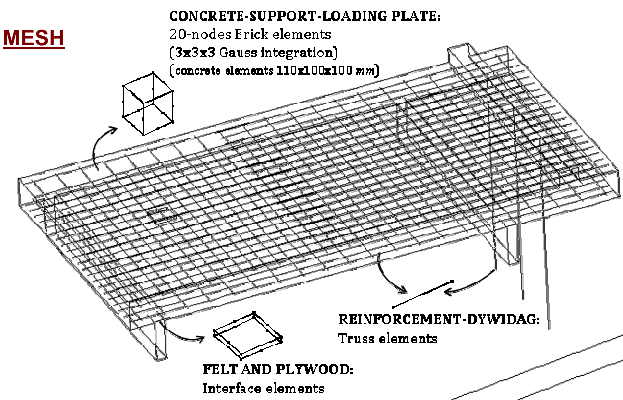
Minimum load value detected **analytically**: **one-way shear**

Some remarks: it has been assumed a direct correspondance between minimum strength (analytically obtained) and actual failure mode.

Finite element model

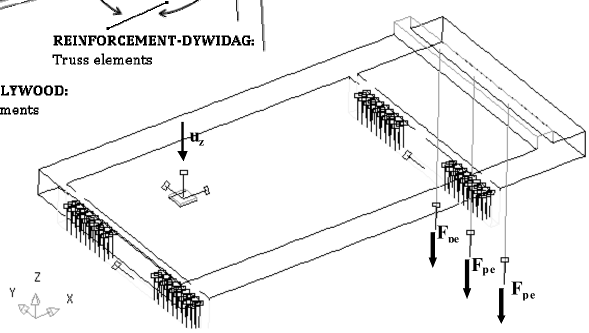
Model with brick elements

MESH



CONVERGENCE CRITERION:
displacement control
Newton Raphson based on force (10^{-2}) and energy (10^{-3})

BOUNDARY CONDITIONS



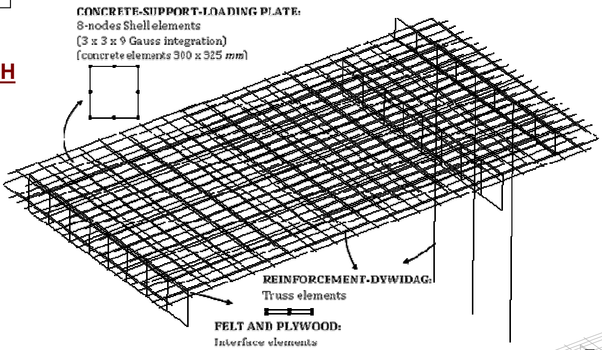
Software used: DIANA 9.4

Finite element model

S1T1

Model with shell elements

MESH

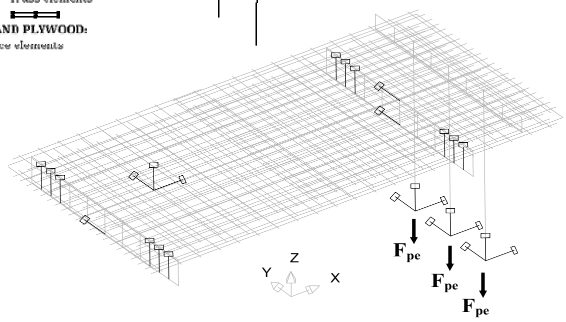


CONVERGENCE CRITERION:

displacement control

Newton Raphson based on force (10^{-2}) and energy (10^{-3})

BOUNDARY CONDITIONS



Software used: DIANA 9.4

Crack model and constitutive laws

Total strain crack models

Constitutive laws

Rotating

$$ {}^{t+\Delta t}_{i+1}\tilde{\varepsilon}_{ns} = \begin{bmatrix} \frac{1-\nu}{(1+\nu)(1-2\nu)} & \frac{\nu}{(1+\nu)(1-2\nu)} \\ \frac{\nu}{(1+\nu)(1-2\nu)} & \frac{1-\nu}{(1+\nu)(1-2\nu)} \end{bmatrix} {}^{t+\Delta t}_{i+1}\varepsilon_{ns} $$

Fixed

$$ D = \begin{bmatrix} \frac{E_n}{1-\nu_{ns}\nu_{sn}} & \frac{\nu_{sn}E_n}{1-\nu_{ns}\nu_{sn}} & 0 \\ \frac{\nu_{ns}E_s}{1-\nu_{ns}\nu_{sn}} & \frac{E_s}{1-\nu_{ns}\nu_{sn}} & 0 \\ 0 & 0 & G_{ns} \end{bmatrix} $$

$$ G_{ns} = \beta G $$

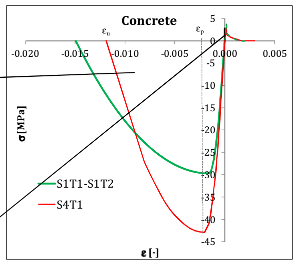
ν_{ns}, ν_{sn} **=Poisson's ratio (variable)**

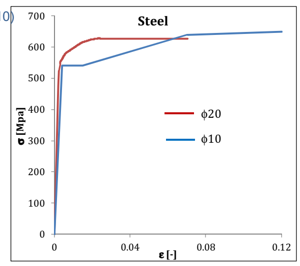
β **=shear retention factor (variable)** (linearly decreasing from 1 to 0 as a function of *E* and n

G_c=250G_f

(Nakamura, H., Higai, T., 2001. Compressive Fracture Energy and Fracture Zone Length of Concrete. Modeling of inelastic behaviour of RC structures under seismic loads, P.Benson Shing Tada-aki Tanabe (eds))

G_f=73$f_c^{0.18}$ (Model Code 2010)

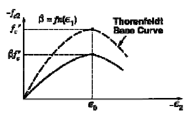




Multi-axial stress state:

Reduction of f_c due to lateral cracking-

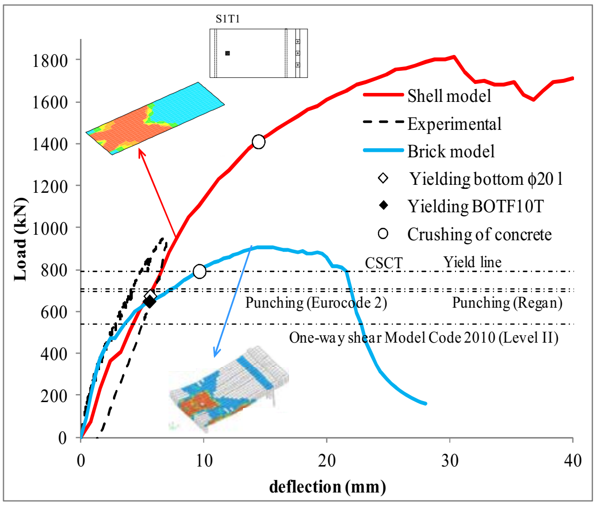
(Vecchio, F.J., Collins, M.P.: Compression response of cracked reinforced concrete. Journal Structural Engineering, 1993, 119, 3590˚3610)



For 3D modelling with brick elements total strain crack models implemented in standard software are not powerful as in 2D modelling!

Evaluation of the carrying capacity of the slabs

S1T1 | **Nonlinear finite element analyses** | **2D model**



As expected the **shell model** is not able to properly describe the shear failure of the slab and substantially **overestimates** the capacity of the slab

⬇

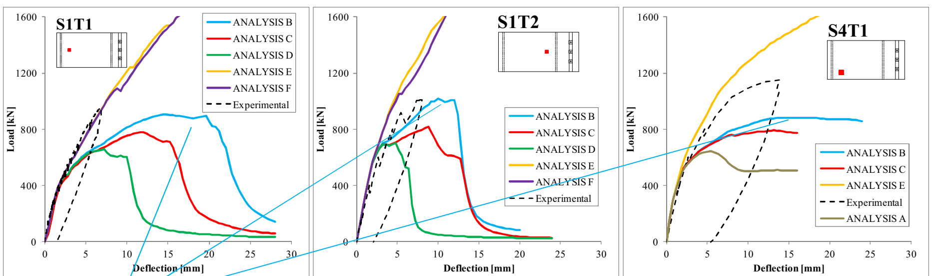
The other slabs (S1T2 and S4T1) have been analyzed only with 3D models

Evaluation of the carrying capacity of the slabs

Nonlinear finite element analyses | **Model with brick elements**

Parametric study: combined the main sensitive parameters of the crack model

	ν	$f_{c,red}/f_c$	G_f	G_c	crack model	β
Analysis A	0.15	1	MC2010	$250G_{fMC2010}$	rotating	/
Analysis B	variable	1	MC2010	$250G_{fMC2010}$	rotating	/
Analysis C	variable	0.6	MC2010	$250G_{fMC2010}$	rotating	/
Analysis D	variable	0.6	MC2010	$120G_{fMC2010}$	rotating	/
Analysis E	variable	0.6	MC2010	$250G_{fMC2010}$	fixed	variable
Analysis F	variable	0.6	MC2010	$120G_{fMC2010}$	fixed	variable

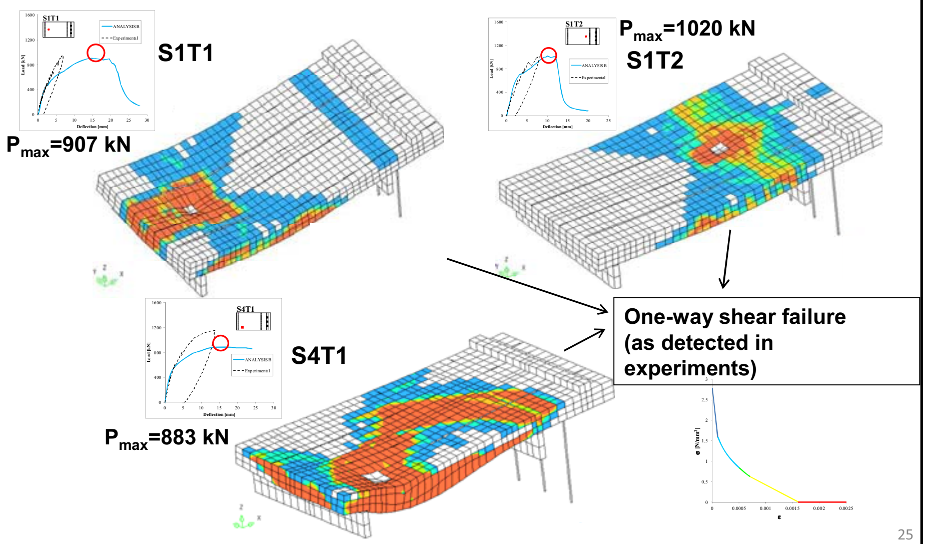


ANALYSIS B =analysis that better fit with experiments for all slabs
Chosen as reference for the application of safety format methods

Evaluation of the carrying capacity of the slabs

Nonlinear finite element analyses

ANALYSIS B: crack pattern at failure



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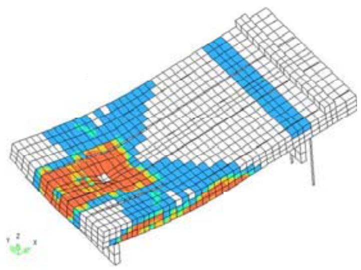
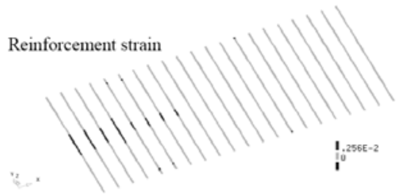
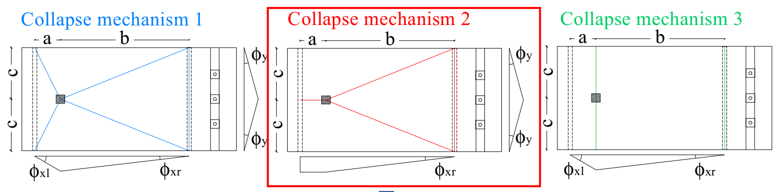
Critical assessment of the results

S1T1



Strain values in the ϕ10 transversal bars at the bottom of the slab at peak load





The failure load strongly depends on boundary condition modelling (e.g. interface elements stiffness, etc.). For real cases it could be quite difficult to have these data.

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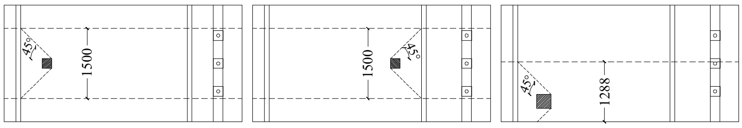
Effective widths assumptions

Effective width b_{eff} used for the shear strength calculatiuon.



Effective width as a resul from NLFEA, as used in the analytical calculations and as observed in the experiments

	NLFE	Analytical	Experimental
S1T1	1695	1500	1800
S1T2	-	1500	1700
S4T1	1250	1288	1300

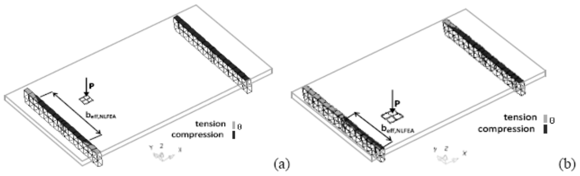


Fig. 17. Effective width b_{eff} determined from NLFE analyses for slabs S1T1 (a) and S4T1 (b).

Evaluation of the carrying capacity of the slabs

Overall results

Measured mechanical properties used in calculations (γ_c=1, γ_s=1)

Ultimate load P values [kN]

	Experiments	Analytical calculations						NLFE analyses
	One-way shear failure	One-way shear		Punching			Bending	ANALYSIS B (One-way shear failure)
		Level I	Level II	Regan	EC2	CSCT	Yield line	
S1T1	954.0	307.3	536.5	708.2	698.1	793	790.7	906.1
S1T2	1023.0	242.4	734.1	708.2	698.1	-	-	1020.1
S4T1	1160.0	344.6	559.4	769	716.4	-	-	883.1

Minimum load value detected **analytically**: **one-way shear**

The failure mode detected analytically and from NLFE analyses is consistent with the experimental failure mode

Design shear resistance: Levels of Approximation | **Model Code 2010**

Evaluated the **DESIGN SHEAR RESISTANCE** of the slabs applying the **Levels of Approximation**:

- **Analytically: Level I-II Approximation** → Design mechanical properties used in calculations (γ_c=1.5, γ_s=1.15)
- **From NLFE analysis: Level IV Approximation**

3 SAFETY FORMAT METHODS proposed by MC2010 to reduce the peak load obtained from NLFEA in order to determine the **design shear resistance**

1. **Partial Factor Method (PF)**
2. **Global Resistance Factor Method (GRF)**
3. **Estimation of Coefficient of Variation Method (ECOV)**

PF **Design mechanical properties** as input data in NLFEA

$$R_d = R(f_d, \ldots)$$

Peak load obtained from NLFEA

GRF **Mean mechanical properties** as input data in NLFEA

$$R_d = \frac{R(f_m, \ldots)}{\gamma_R \gamma_{Rd}} = \frac{R(f_m, \ldots)}{1.2 \cdot 1.06} = \frac{R(f_m, \ldots)}{1.27}$$

Peak load obtained from NLFEA

ECOV **Mean mechanical properties +Characteristic mechanical properties** as input data in NLFEA

Peak loads obtained from NLFEA

$$R_m = R(f_m, \ldots), R_k = R(f_k, \ldots);$$

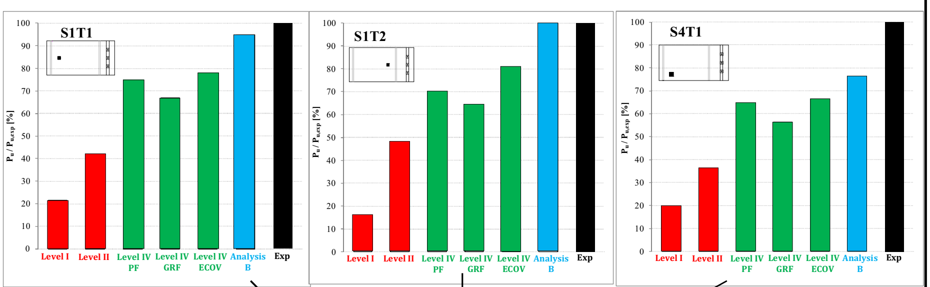
$$R_d = \frac{R_m}{\gamma_R \gamma_{Rd}}$$

$$\gamma_R = \exp(\alpha_R \beta V_R) = \exp\left(0.8 \cdot 3.8 \cdot \left[\frac{1}{1.65} \ln(R_m / R_k)\right]\right)$$

Evaluation of the carrying capacity of the slabs

Design shear resistance: Levels of Approximation: overall results P_u/P_{exp} [%] | **Model Code 2010**

Level I- II **Level IV (PF- GRF- ECOV)** **Analysis B (no safety format)** **Experimental (100%)**



Good agreement with Model Code 2010 philosophy: by increasing the Level of Approximation the design resistance increases

From **Level II** to **Level IV:** Increase ranging from 69% to 87

Conclusions

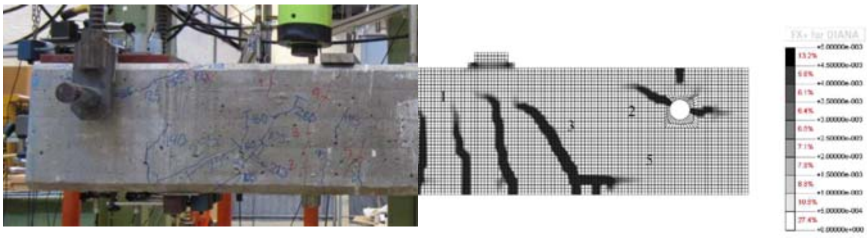
- **Three slabs** subjected to **concentrated loads near supports** have been analyzed. The **carrying capacity** of each slab has been determined with **NLFE analyses** and with **analytical procedures.**
- For all slabs the lowest resistance value calculated analytically is the **one-way shear** resistance, in **accordance to experimental test** observations.
  However, S1T1 and S1T2 also showed a **fully developed flexural cracking pattern** at failure. For S1T1 the critical **collapse mechanism** calculated with the **yield line method** is in **agreement** with the **crack pattern** observed in **experimental test** at **failure**.
- The **results** of **NLFEA** strongly **depend** on the **modeling choices** made by the analyst, especially with regards of relatively simple crack models.
  The availability of **guidelines** on how to properly perform NLFE analyses is of big help for analysts.
- The **results** obtained **well fit** with the philosophy of the **Model Code 2010**: by **increasing** the **approximation level** the **shear resistance** of the slabs **increases**.
- Once the finite element model is properly validated, the structural assessment carried out with multiple approximation levels can be of benefit for intervention plans on existing structures (e.g. maintenance, repair, demolition etc.)

The authors acknowledge the Dutch Ministry of Transport, Public Works and Water Management for supporting this research.

THANK YOU
FOR YOUR ATTENTION

MODELLING OF BOND BEHAVIOUR OF NATURALLY CORRODED REINFORCEMENT IN CONCRETE STRUCTURAL MEMBERS

Experimental and Numerical Study



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Tómas Guðmundsson
Karin Lundgren

Outline

- Laboratory work
  - Test setup
  - Test specimens
  - Testing
  - Analyze results
- Non linear finite element modeling
- Comparison of FE results and tests
- Conclusions

Test specimens

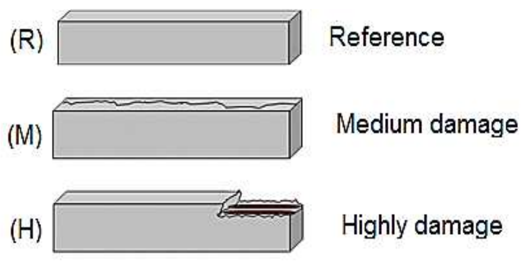
- Work done by contractors
  - Specimens cut off bridge
  - Guarding rail and slab cut off
- Preparation on site
  - Correct lengths
- Preparation in Laboratory
  - Drilled for supports
  - Strengthened
  - Documentation of cracks

Specimen Classifications

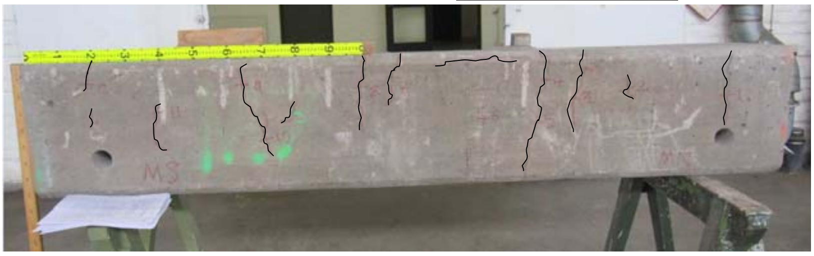
•**R**eference specimens: No defects

•**M**edium damaged specimens: surface cracks

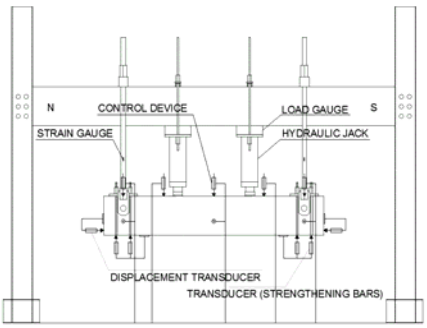
•**H**ighly damaged specimens: spalling of the concrete cover



Test specimens

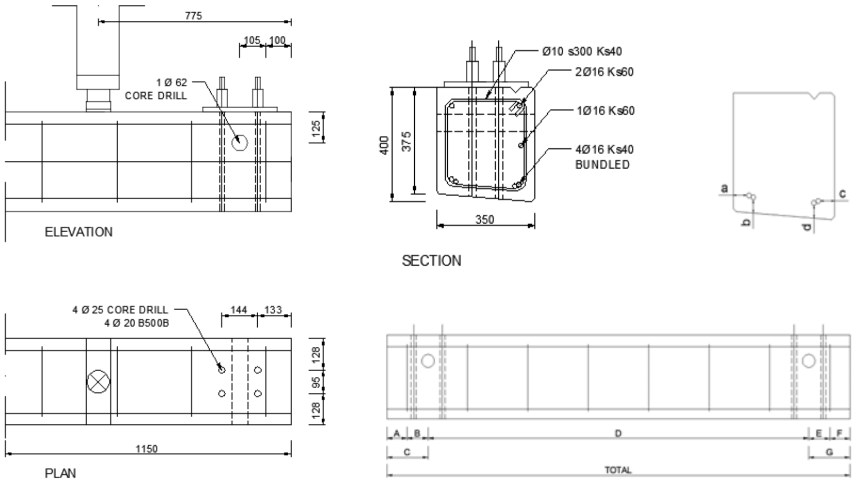


Test setup



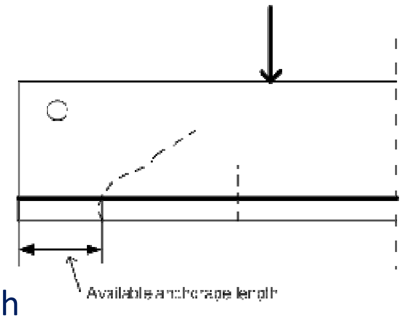
- Indirectly supported four-point bending test
- 8+14 specimens

Test specimens



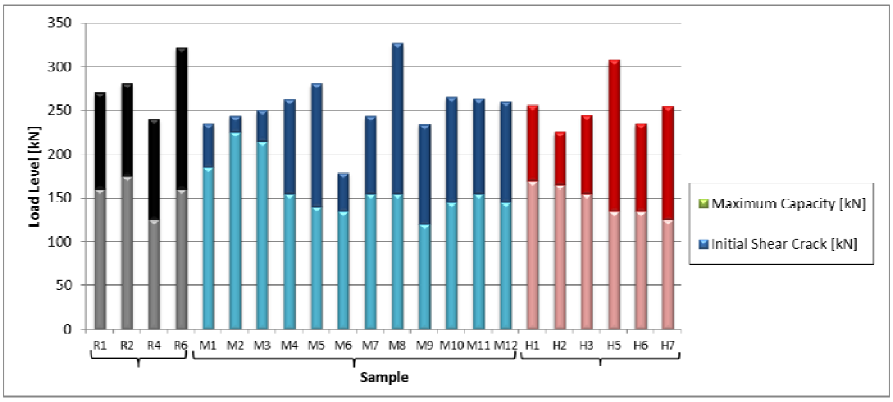
Testing

- 8+14 beams tested
- Deformation control
- Key parameters
  - Ultimate load
  - Free end slip
  - Crack pattern
  - Available anchorage length



CHALMERS

All test samples: behaviours in the first and the second round



Initial shear cracking occurs in a lower level before the anchorage failure
10% higher capacity in Reference Samples

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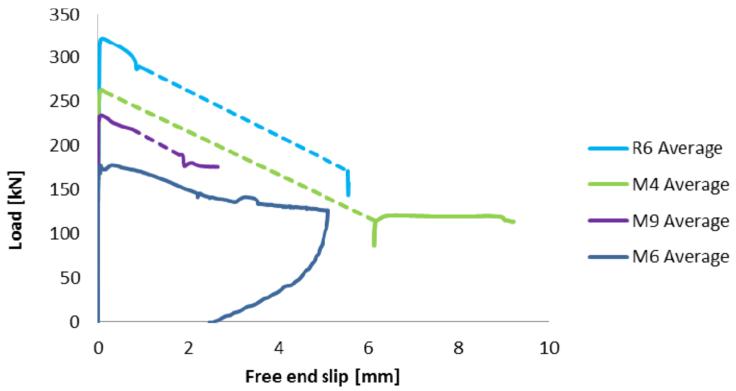
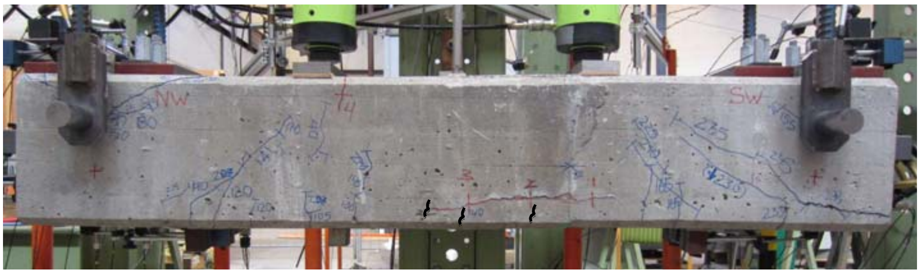
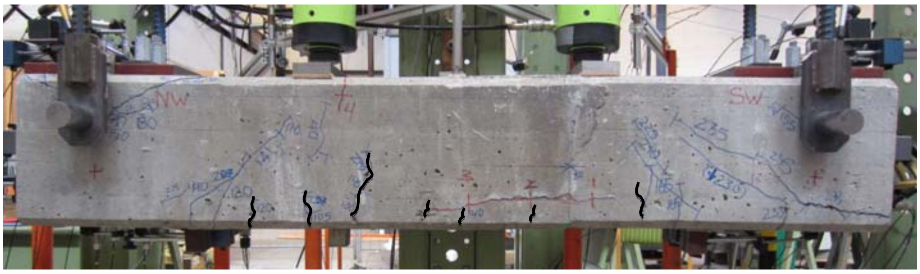
Test results – Force / Free end slip



Crack pattern after testing(1)





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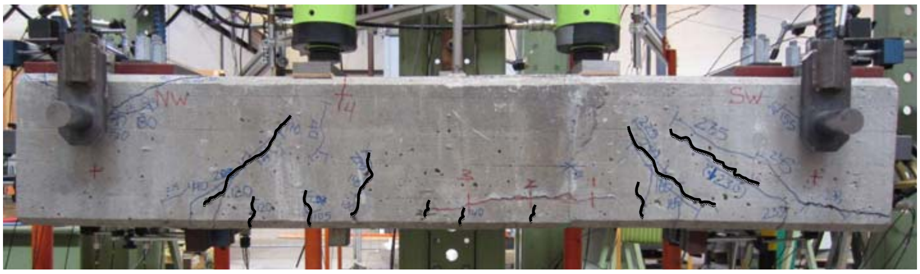
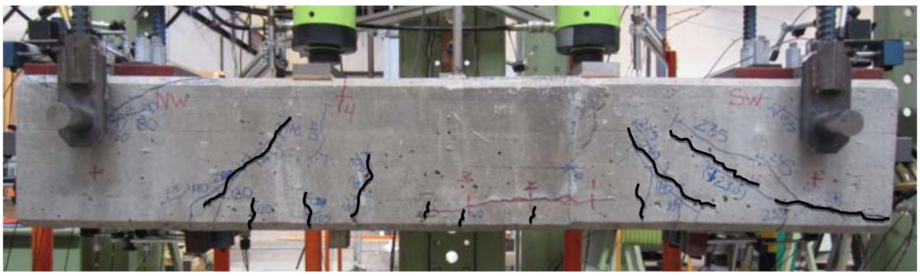
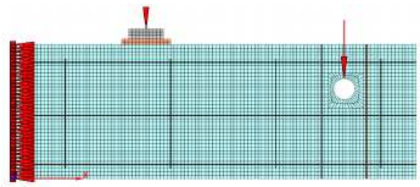
Crack pattern after testing(2)

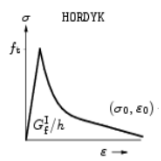


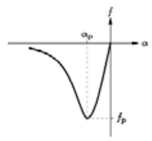


FE model

- Steel
  - Elastio plastic material model
  - Truss elements L2TRU
- Concrete
  - Constituive material model based on non-linear fracture mechanics
  - Smeared rotated crack model based on total strain
  - Tension behavior, Hordijk curve
  - Compression behavior, Thorenfeldt curve
  - Four node quadrilaterial plane stress elements Q8MEM
- Interface layer
  - Truss elements L8IF

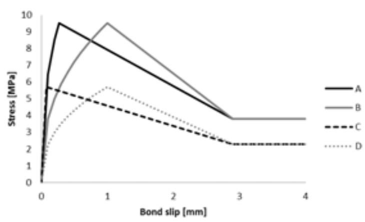


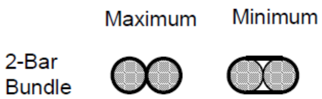




FE model – Material Parameters

- Steel
  - Old test data
- Concrete
  - Compression strength
  - Tensile strength
  - Fracture energy
- Bond slip – FIB Model Code
  - Max. Bond strength
  - Shape of curve
  - Parameters of bars





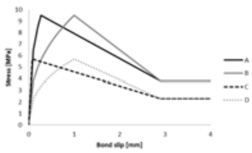
Material Parameters(1)

- Concrete

Analysis:	f_{cm} [MPa]	f_{ctm} [MPa]	E_{cm} [GPa]	G_f [N/mm]	Bond curve
Reference (III)	48	3.5	35	0.147	A
Reference (IV)	48	3.5	35	0.147	A
Reference (VII)	48	2.7	35	0.1	A
Damaged (XII)	48	2.7	35	0.1	C
Damaged (XIII)	48	2.7	35	0.1	D

FIB 2010 $$f_{ctm} = 0.3 \cdot f_{ck}^{\frac{2}{3}} \; for\ concrete\ grades \leq C50$$

Frost Damage Effects, Hanjari 2010 $$f_{ctm} = 0.027 \cdot (f_{cc}^{d})^{1.2}$$



FIB 2010 $$G_f = 73 \cdot (f_{cm})^{0.18}$$

Material Parameters(2)

- Steel

	f_{sy} [MPa]	f_{su} [MPa]	ε_{sy} [%₀]	ε_{su} [%₀]	Es [GPa]
KS60	693	907	0.312	1.25	222
KS40	468	638	0.227	1.14	206
Strenghtening	500	550	0.243	1.05	205

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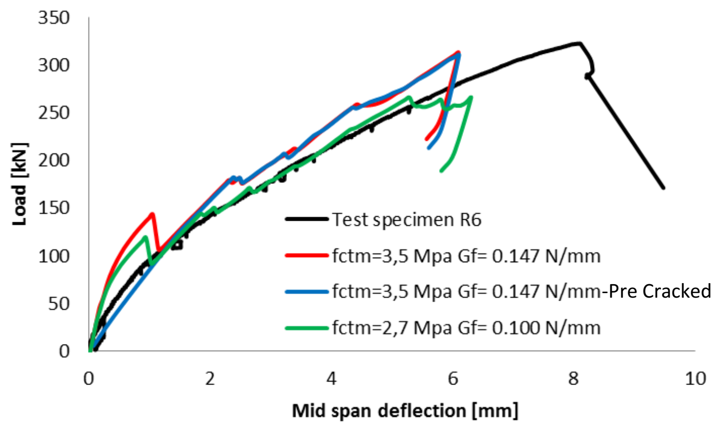
Comparison – Reference Force displacement



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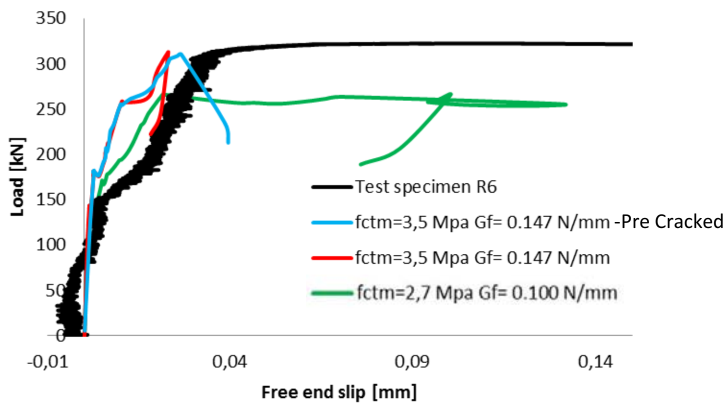
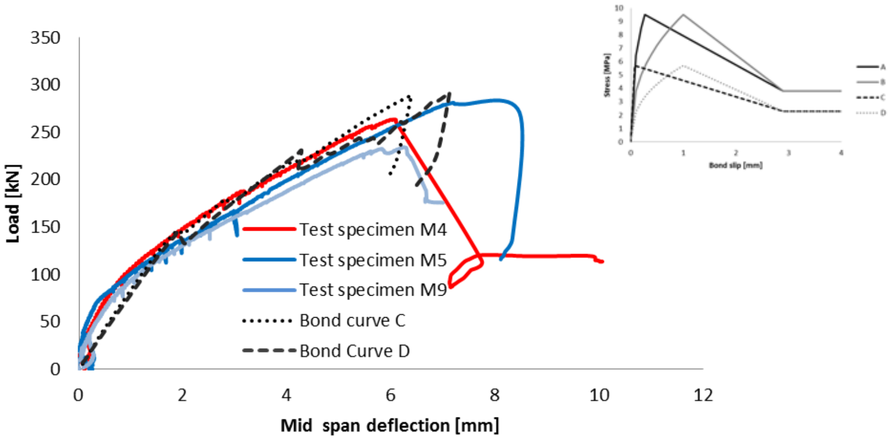
Comparison – Reference Free end slip



Comparison – Medium damaged Force displacement



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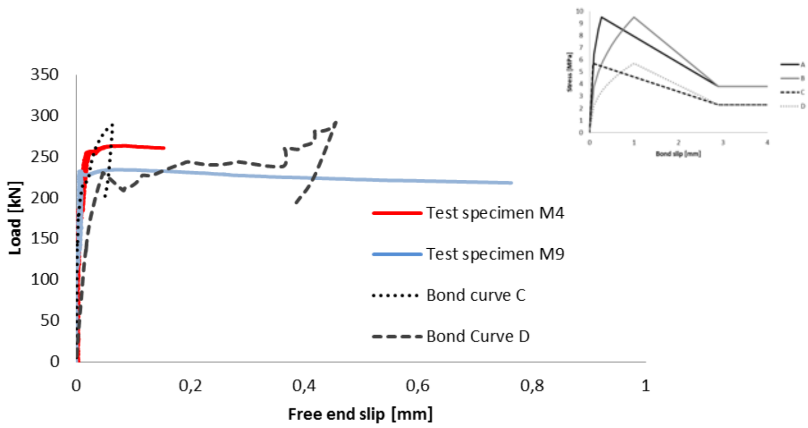
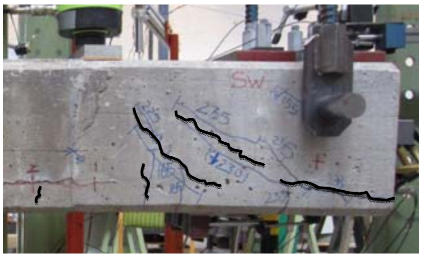
Comparison – Medium damaged Free end slip



Comparison of the Crack Pattern



Conclusions

- Higher degree of corrosion leads to decrease of maximum load
- Longer available anchorage length leads to higher maximum load
- Maximum load is not connected to first shear or flexural crack

Conclusions

- 2D analysis sufficient regarding:
  - Maximum load
  - Maximum deflection
- 2D analysis not sufficient regarding:
  - Free end slip
  - Crack pattern
- Adjusting material parameters gives better correlation

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The use of DIANA for non-linear FE analysis in the advanced concrete structures course at Chalmers

Mario Plos

Division of Structural Engineering
Chalmers University of Technolog
Sweden

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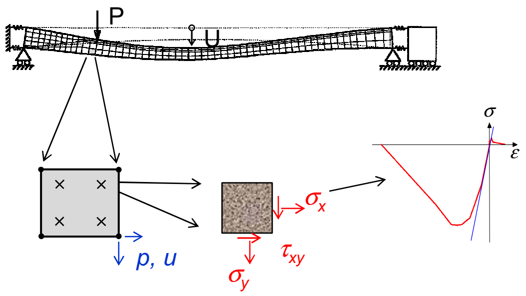
Non-linear FEA of concrete structures

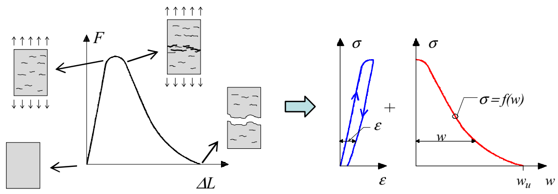
- FEM - Finite element method

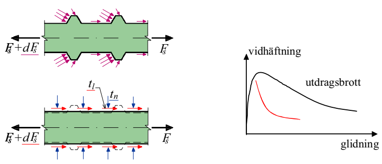


– Long tradition at Chalmers
– Today: engineering practice

Non-linear FEA of concrete structures

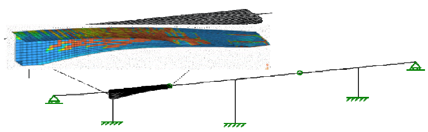
- FEM - Finite element method
- Fracture mechanics and bond modelling

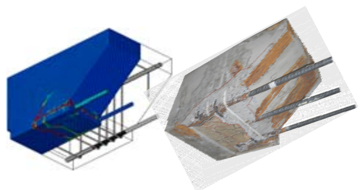


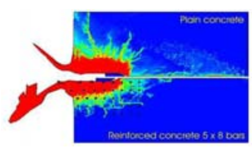


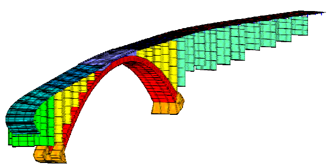
Non-linear FEA of concrete structures

- FEM - Finite element method
- Fracture mechanics and bond modelling
- Non-linear analysis of concrete structures









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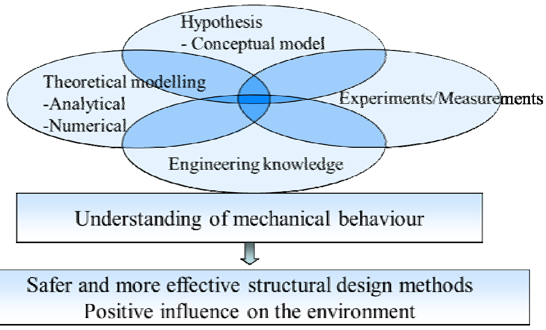
Non-linear FEA of concrete structures

- FEM - Finite element method
- Fracture mechanics and bond modelling
- Non-linear analysis of concrete structures
- Important part of our research methodology



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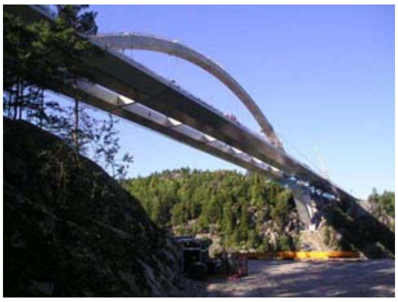
Master programme

Structural Engineering and Building Technology



Master programme

Structural Engineering and Building Technology

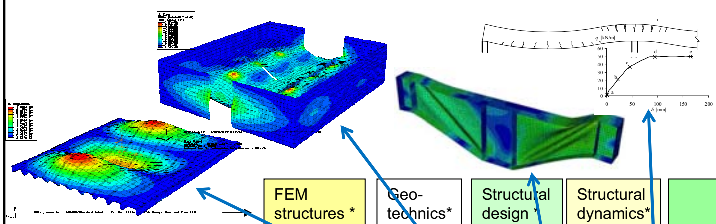
From learning outcomes:

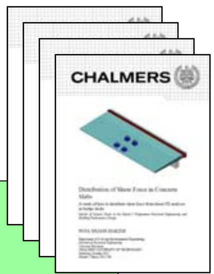
- Understanding of structural behaviour
- Ability to do structural idealisation and to choose appropriate models
- Ability to work methodically with modern tools

Master programme

Structural Engineering and Building Technology

- FEM in education:





FEM structures *
Geo-technics*
Structural design *
Structural dynamics*
Structural systems
FEM basics*
Timber engg.
Structural concrete
Steel structure
Concrete structures
Material performance
Heat, moisture engineering
Indoor climate and HVAC
Building physics
Building technology and services engineering
Sound & vibration*
Build. acoust & commun noice*
Sustainable building - competition*
Master's Thesis

The course *Concrete Structures*

- Final course in master programme
- Preparing for:
  - Long-lasting career as practising Structural Engineer
  - Master thesis and research career
- Contents:
  - Continuous beams and columns
  - Finite element analysis of concrete structures
  - Restraint cracking and time-dependent deformations

The course *Concrete Structures*

- Finite element analysis of concrete structures:
- Lectures and group exercises
- FEM project in DIANA

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The course *Concrete Structures*

Learning outcome

- Use of FEA in different stages
(from conceptual design to assessment)
- To choose suitable element types, BC:s and loading
- Previous course: To design reinforcement from linear analysis
- Basic theory and experience in nonlinear analyses
  - Nonlinear concrete material, localisation
  - Modelling of reinforcement, how different choices affect the results
  - Understanding and interpretation of analysis results

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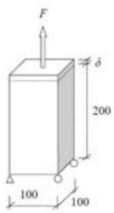
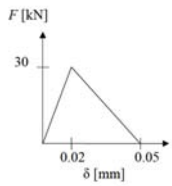
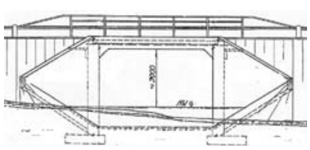
The course *Concrete Structures*

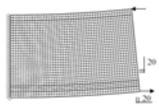
Lectures and exercises

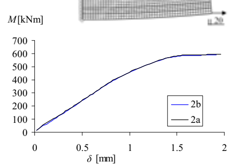
- Concrete cracking and concrete modeling
- Reinforcement–concrete interaction
- Modeling on various levels
- Non-linear FE analysis
- Material response in 3D
- FEA in practice

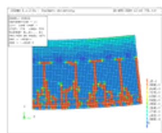


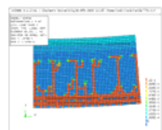








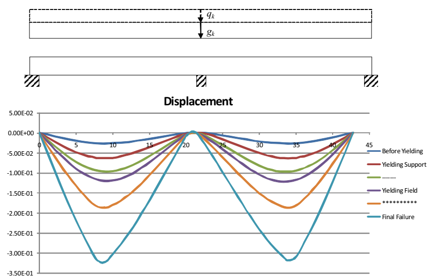


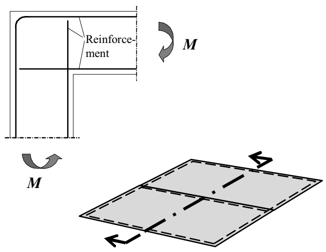


FEM project in DIANA and FX+

Group work (~4 students)

- Analysis of continuous beam
- Group specific investigation

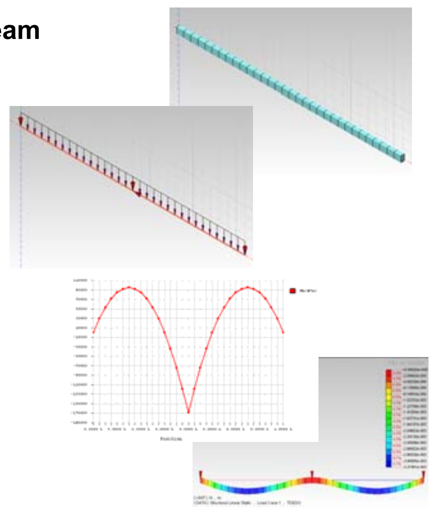




FEM project in DIANA and FX+

Part 1 – Analysis of continuous beam

- Leaflet to follow
- Well defined analysis
  - Beam elements
  - Embedded reinforcement
  - Total Strain Crack Model with rotating cracks
- Focus on:
  - "Understand what I do"
  - Verification
  - Interpretation of results



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FEM project in DIANA and FX+

Part 2 – Group specific investigation

- Independent work
- Open problem
  - Find relevant information
  - Comprehend
  - Model and analysis on their own
- Focus on:
  - Learn independently
  - Teach student colleagues
  - Learn from each other

Temperature effects
Loading
Frame joints
Material models
Shrinkage
Safety evaluation
Boundary conditions
Creep
Prestress
Slab modelling

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FEM project in DIANA and FX+

Our experience

- Using Diana and FX+ works in general
- More user-friendly pre- and postprocessor desirable
  - improved integration: FX+, MeshEdit & Diana solver
- Great preparation for master thesis!
- The analysis of continuous beam:
  - Fits well within the course
  - Results hard to comprehend
  - Not full benefit of FE posibility
  - Continuum analysis would fit better in the MP progression

FEM project in DIANA and FX+

Discussion

- How can we develop the project?
- How can we improve our education regarding FEA of concrete structures?

Thank you!

FEM analysis of reinforced concrete decks with compressive membrane action

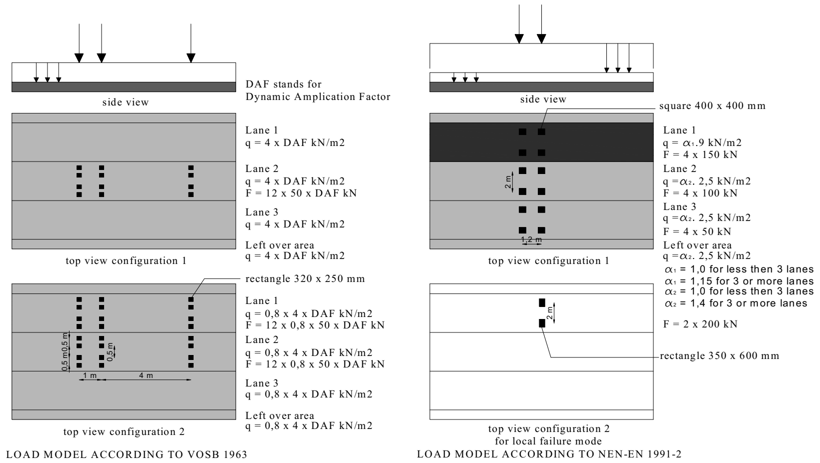
Cor van der Veen Delft University of Technology

Gert Jan Bakker (Witteveen + Bos)

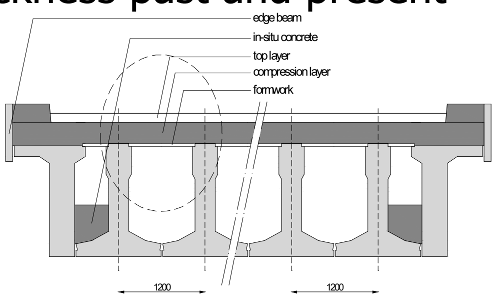
Overview

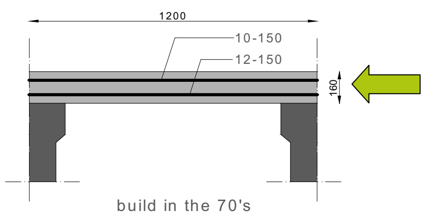
- Introduction
- Problem definition
- compressive membrane action
- Overview of finite element models
- Finite element models including compressive membrane action
- Practical example
- Conclusions & recommendations

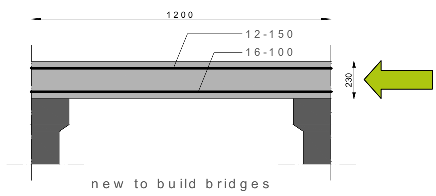
Traffic loading past and present (EC)



Bridge Deck thickness past and present

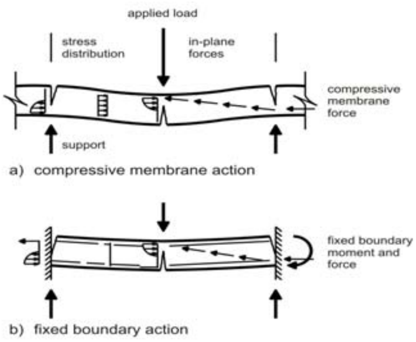




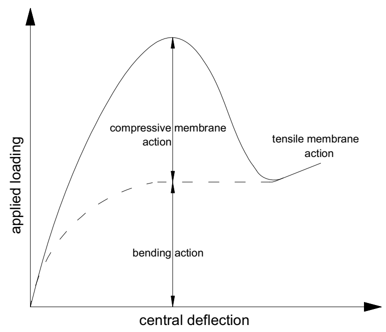


Compressive membrane action CMA allowed in REINFORCED Decks:

Canadian Highway Bridge Design Code
UK Highway Agency Standard BD 81/02
New Zealand



strength increase by compressive membrane action

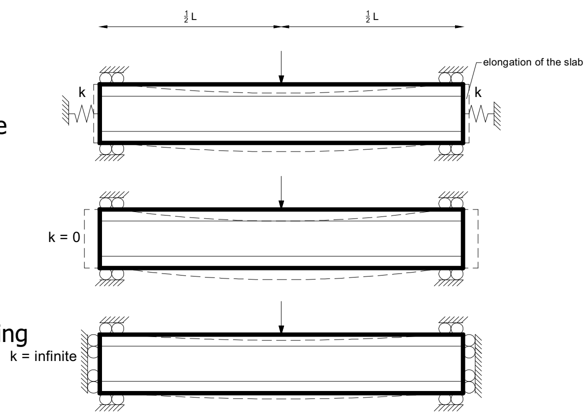


Partly horizontal restrained model

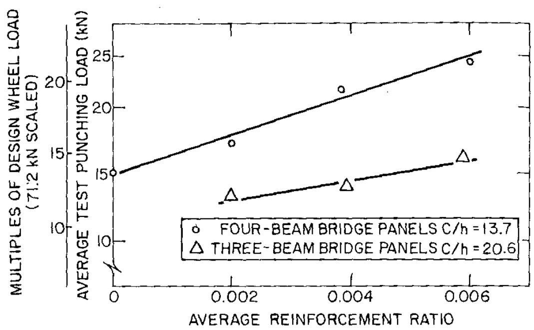
- Partly restrained
- (effect reinforced transverse beams, 50%)

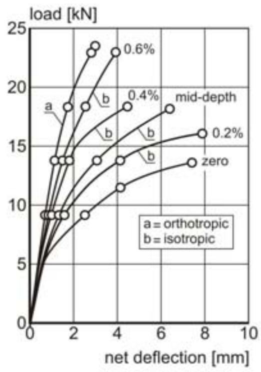
- No restrain (free)

- Fully restrained
- (effect transverse prestressing and beams, 90%)



Variation of Punching load with reinforcement ratio (scale 1:8) Batchelor et al

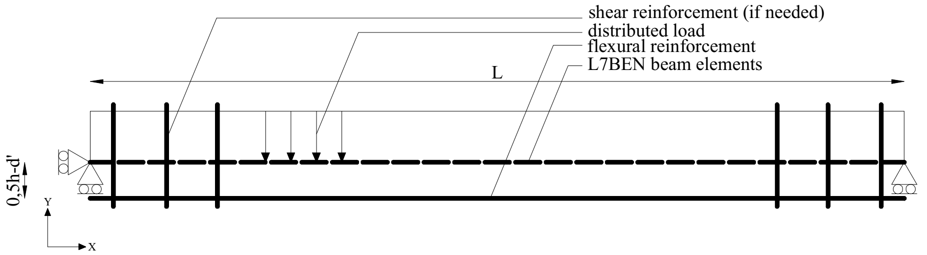




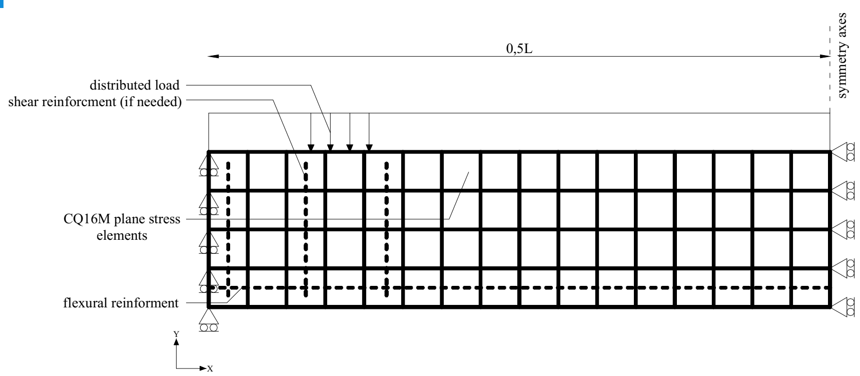
Overview of finite element models

- Linear elastic behavior
  - Comparison models
- Non linear behavior
  - The model must be a non linear model
  - Cracking of the concrete is important
  - Plasticity of steel and concrete
  - Softening concrete

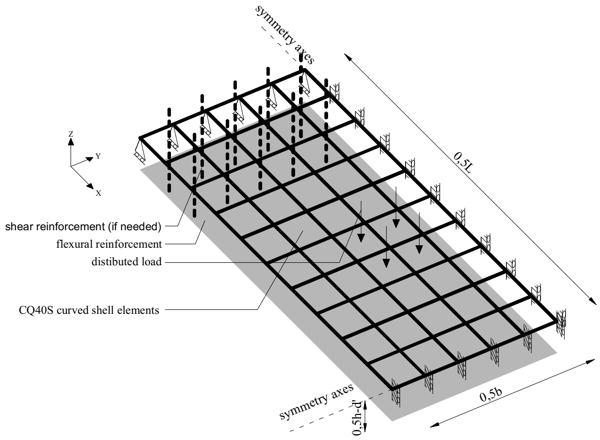
2D beam model (L7BEN)



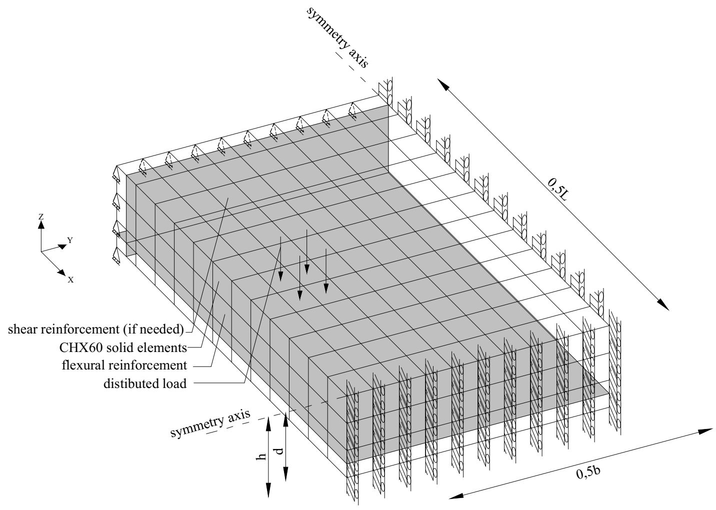
2D plane stress model (CQ16M)



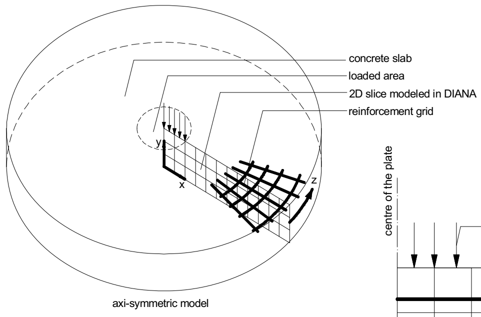
3D curved shell model (CQ40S)

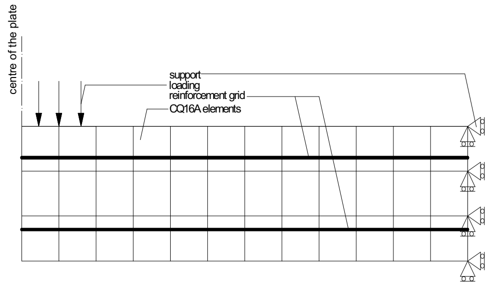


3D solids model (CHX60)



Axi symmetric model (CQ16A)





Comparison of the models

	input	graphical output	realistic model	punch behaviour	calculation time
2D beam model	++	--	+	irrelevant	++
2D plane stress model	+	+	+	irrelevant	+
3D curved shell model	o	-	+	--	o
3D solids model	-	++	++	o	--
axi-symmetric model	+	+	-*	++	+

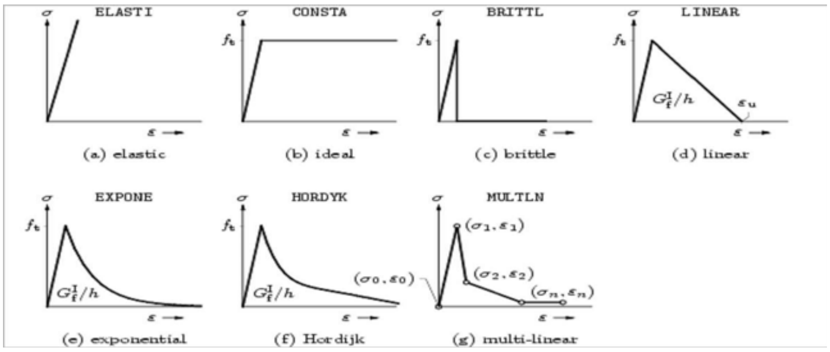
* : rectangular slabs cannot be modelled with this model

For bending failure the 2D plane stress model is the best option

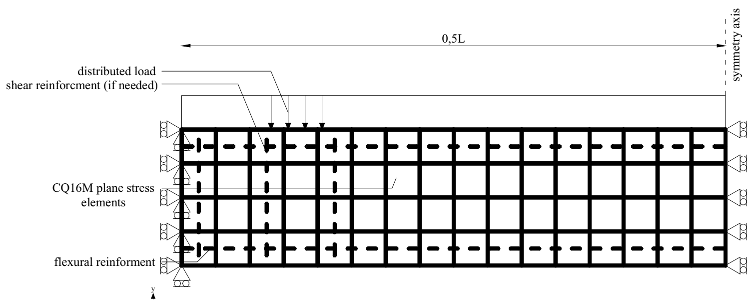
For a combination of punching shear and bending failure the axi symmetric model is the best option

Material properties

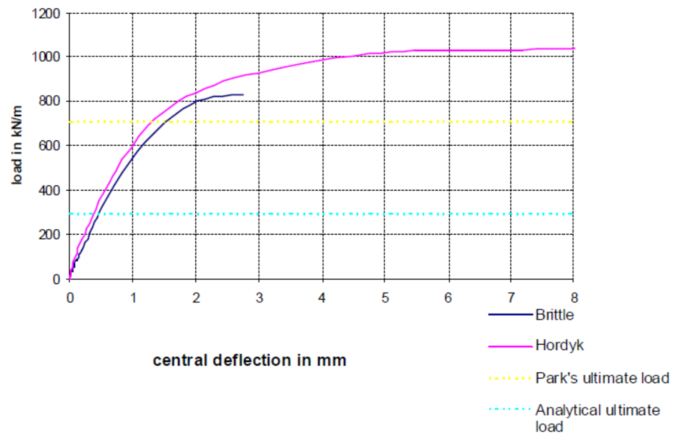
- Total strain rotating crack model
- Brittle and Hordijk Tension softening
- Ideal plastic in compression
- Ideal plastic for reinforcement



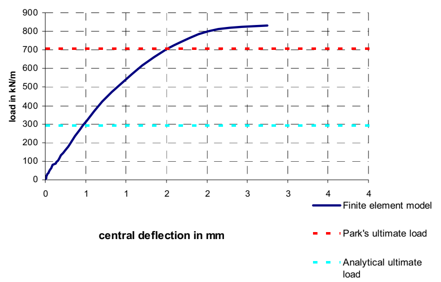
Bending failure including CMA
FE Model



Results FE model flexural failure with CMA
Results affected by softening choice



Results FE model flexural failure with CMA



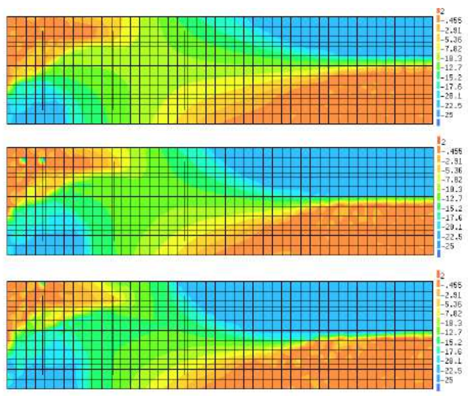
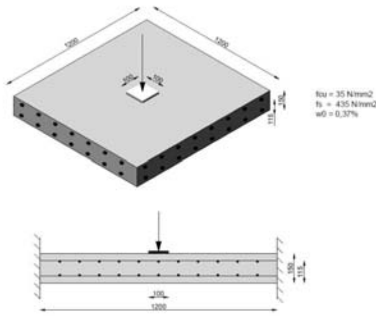
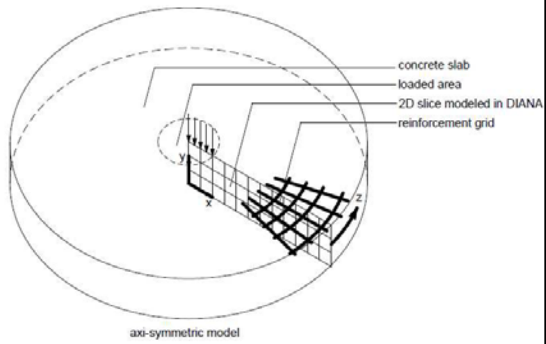


Figure 6.4: σ_{xx} over load steps 23 to 28, from 924 to 997 kN/m

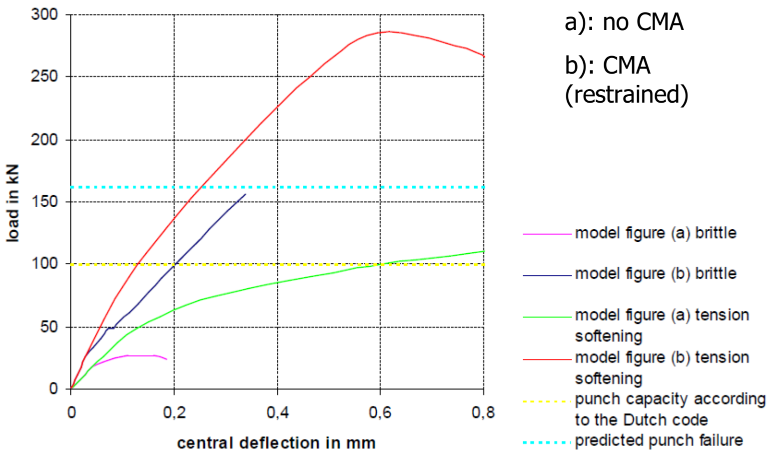
Increase of compressive zone

Wheel load on concrete deck





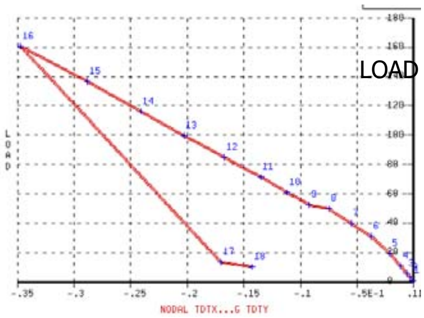
Axi symmetric model different softening



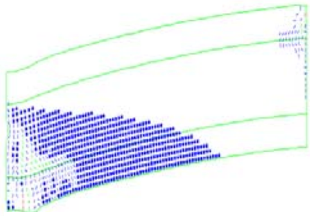
a): no CMA

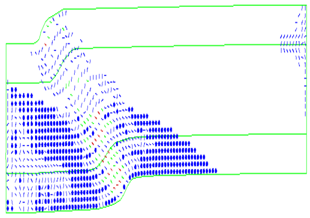
b): CMA (restrained)

Axi symmetric model (punching shear)



deflection

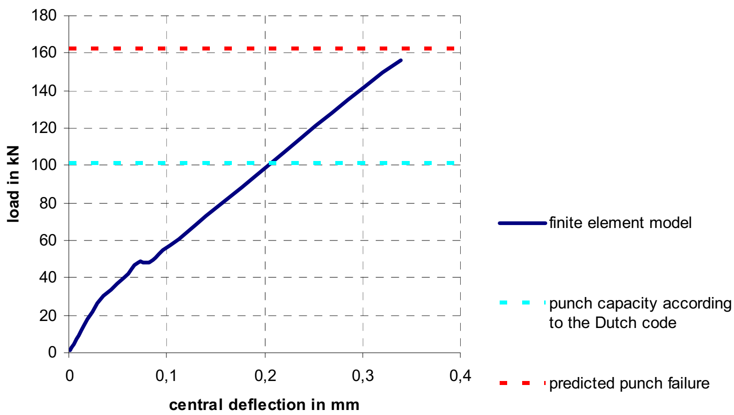




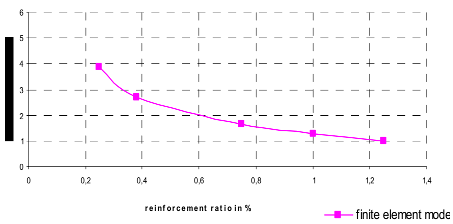
Clamped (restrained model)

Brittle behaviour, force controlled

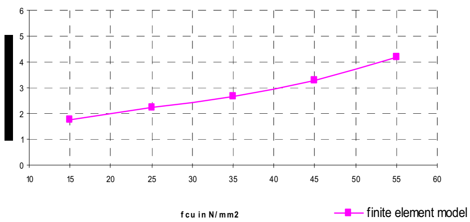
Predictions Axi symmetric model



Enhancement ratio (CMA/without CMA)

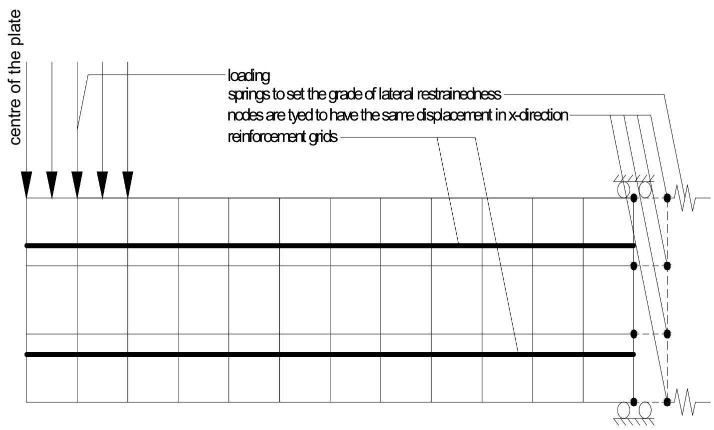


Reinforcement ratio (%)

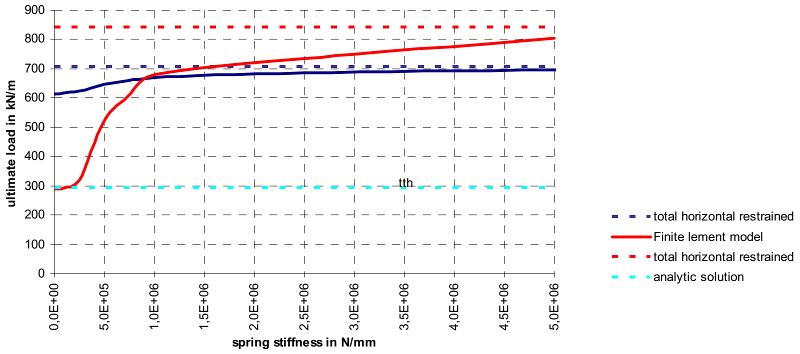


Concrete compressive strength (MPa)

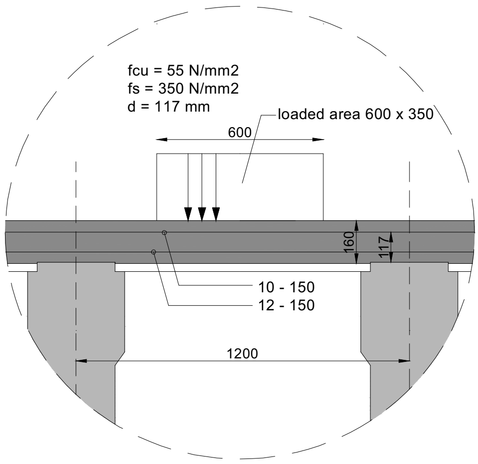
Effect boundary condition (restrained factor)



Effect restrained factor on ultimate load



Practical example



Traditional calculation (no CMA)

- Bending 223,5 kN
- Punching shear failure 477,6 kN

The wheel load capacity prescribed by the Dutch code is 1,35 x 200 = 270,0 kN

The code requirement is not met

CMA included

• Bending	597,4 kN	2,71
• Finite element model	600,0 kN	2,72
• Punching shear failure	1225,4 kN	5,56

Note: punching shear is normally always governing, but loaded area is relatively large compared to span

Conclusions

- If a clamped concrete slab is lateral restrained, compressive membrane action enhances the bearing capacity
- A non linear finite element model can predict the bearing capacity including compressive membrane action
- Partly horizontal restrained structures can also be modeled by adjusting the boundary conditions
- Using finite element models it can be shown that the decks of ZIP girder bridges commonly found in the Netherlands can withstand the wheel loads prescribed by the latest Eurocodes
- Choice of the softening curve greatly affect the ultimate load; calibration of parameters necessary

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Non-linear finite element analysis of steel fibre reinforced concrete in combination with conventional reinforcement

David Fall and Karin Lundgren
Chalmers University of Technology

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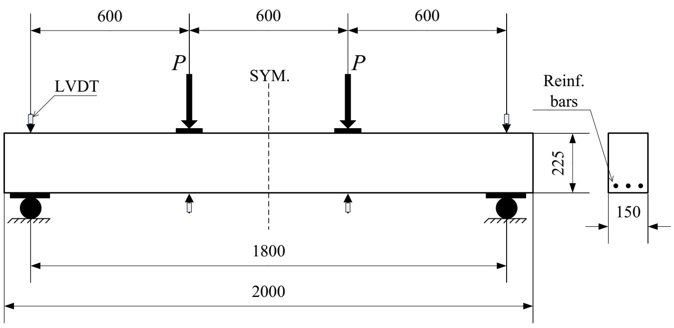
Modeling of SFRC-beams

Four point bending:



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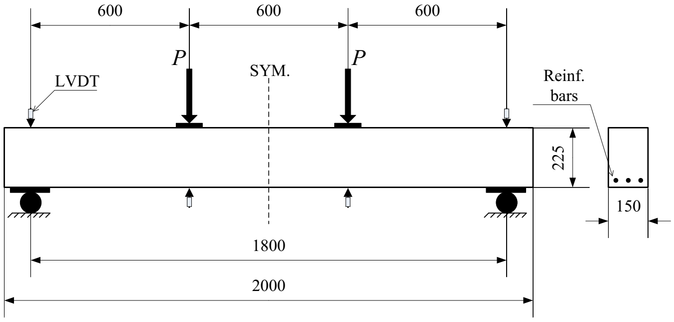
Modeling of SFRC-beams

Four point bending:



Fibre content, $V_f = 0.00\%$, 0.25% and 0.50%.

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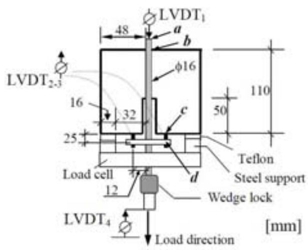
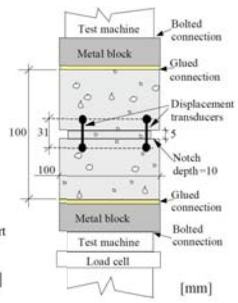
Modeling of SFRC-beams

- Reinforcement properties from tension test
- Bonding properties from pull-out test
- Concrete properties in tension from uniaxial test

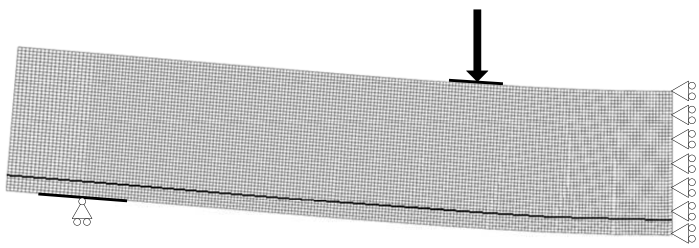




Modeling of SFRC-beams

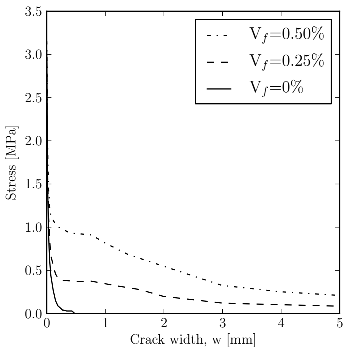
- Two dimensional plain-stress model
- Deformation controlled phased analysis
- Reinforcement represented by truss elements
- Interface elements
- Smeared crack approach
- Rotating cracks

Modeling of SFRC-beams

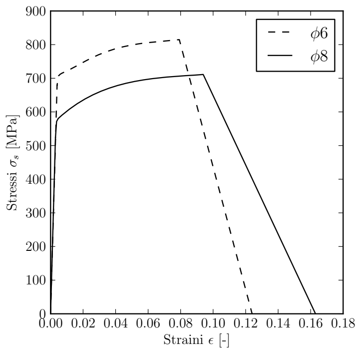


Material input

Concrete in tension

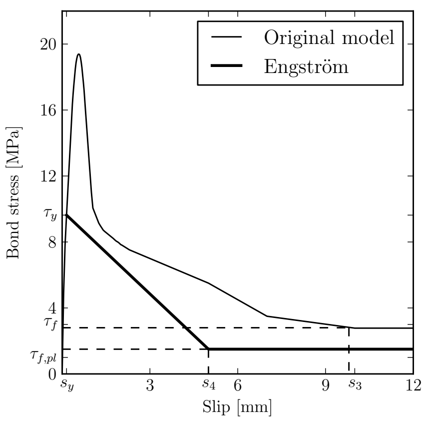


Reinforcement



Material input

Bond stress vs. slip



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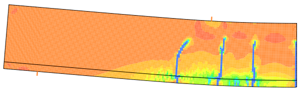
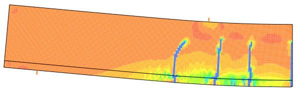
Material input

Prior yielding:

Original



Modified



$V_f = 0.50\%$, applied deformation 2.7 mm (prior yield.)

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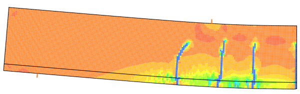
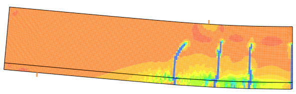
Material input

Prior yielding:

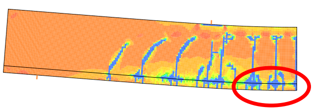
Original

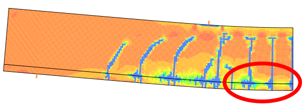
Modified





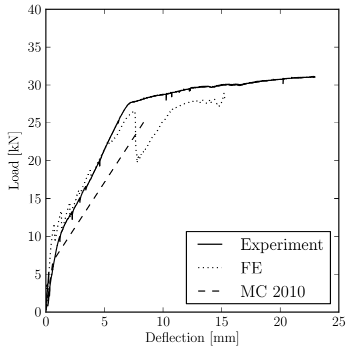
After yielding:



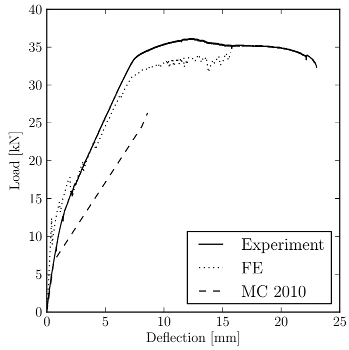


$V_f = 0.50\%$, applied deformation 2.7 mm (prior yield.) and 14 mm (after yield.)

Results: General behavior



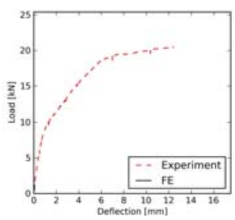
V_f=0%

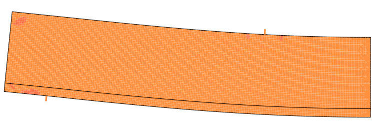


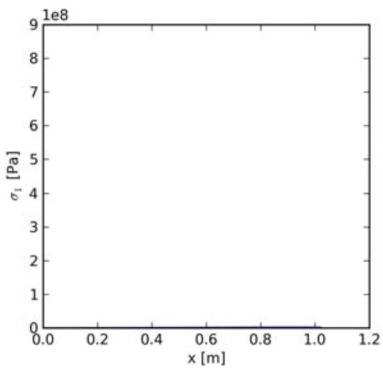
V_f=0.50%

Load - deflection (mid-span)

Results: V_f=0.25%







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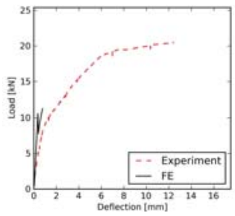
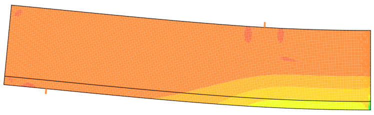
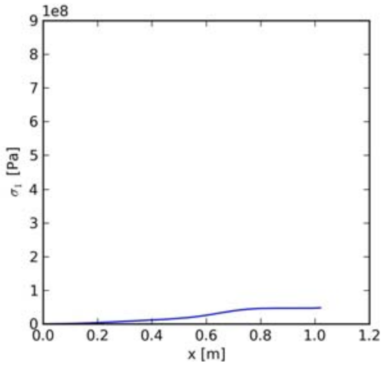
TailorCrete

Results: V_f=0.25%







CHALMERS

TailorCrete

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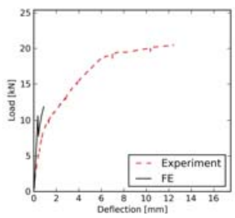

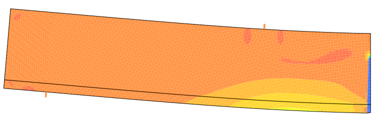

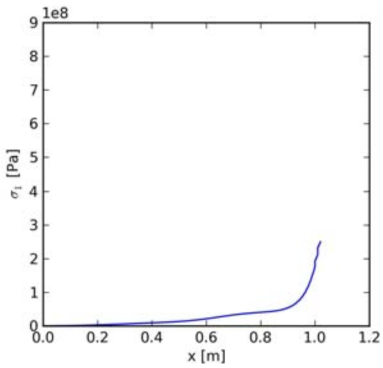

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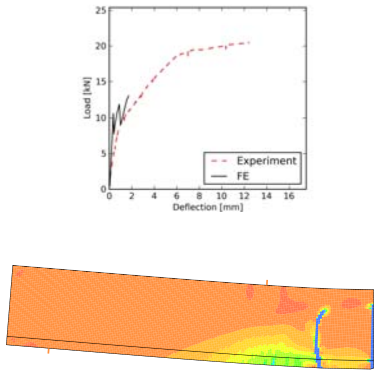

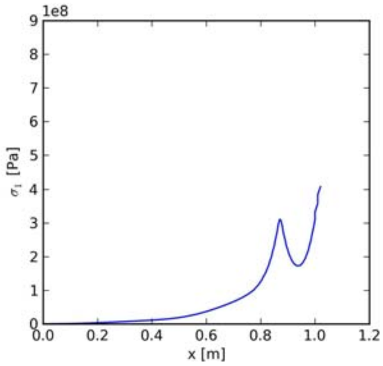

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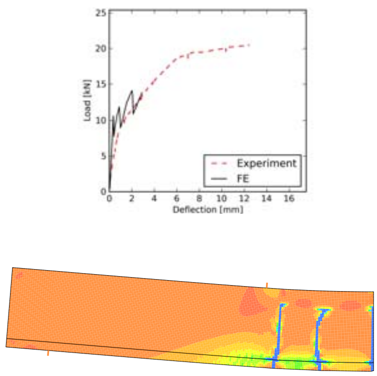

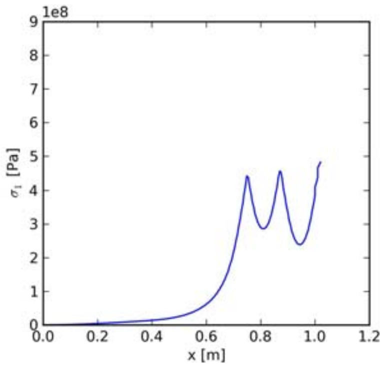

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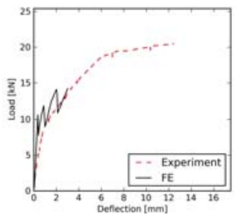

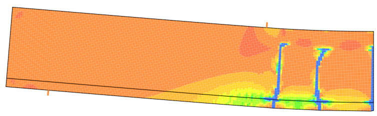

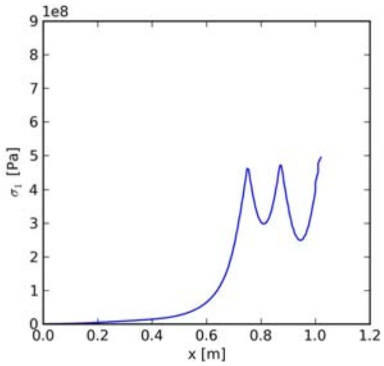

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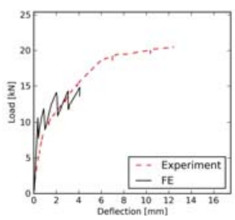

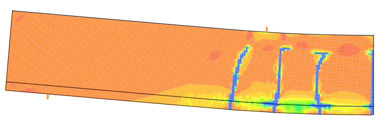

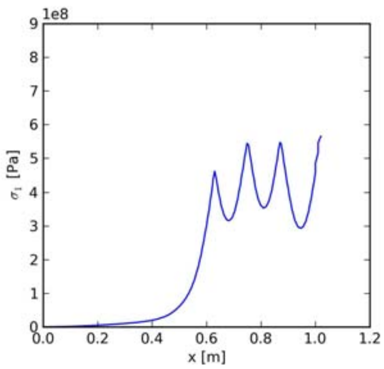

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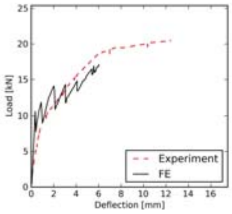

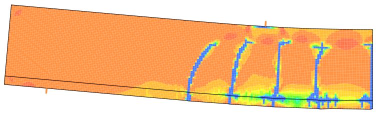

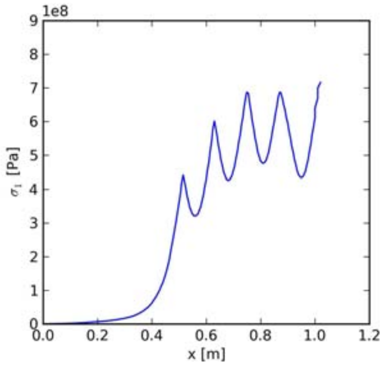

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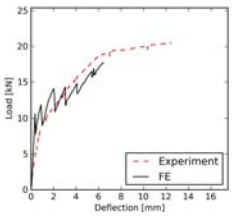

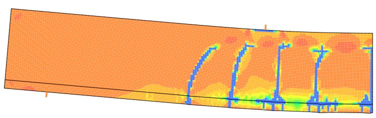

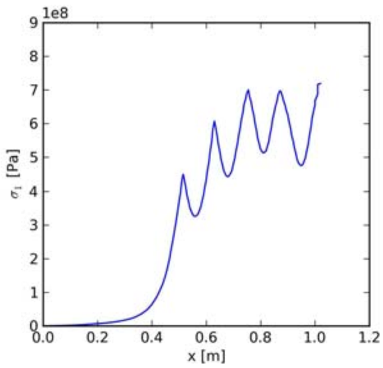

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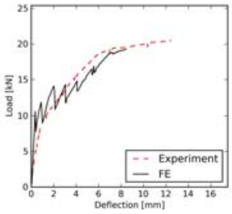

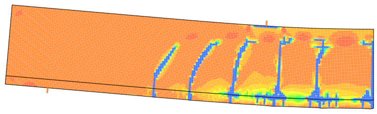

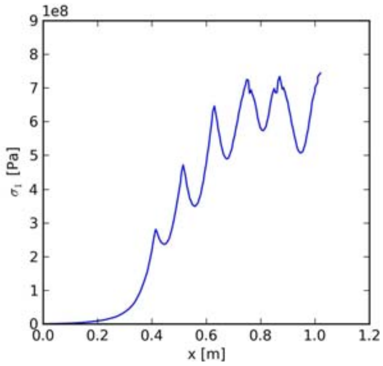

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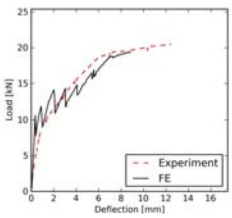

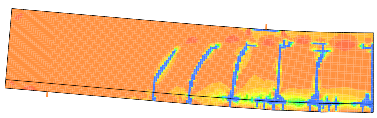

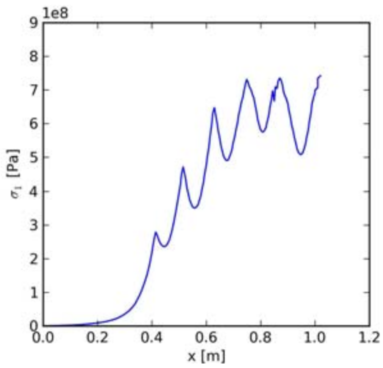

Conclusions

- The effect of SFRC, seen in experiments, can be estimated with FE-analysis.

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Conclusions

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- Crack patterns agree when comparing experiments and FE-analyses.

Conclusions DIANA Users Meeting 2013 21/27

Conclusions

- The effect of SFRC, seen in experiments, can be estimated with FE-analysis.
- Crack patterns agree when comparing experiments and FE-analyses.
- Utilizing a bond-stress model where the bond stress is reduced post yielding, resulted in more localized crack patterns.

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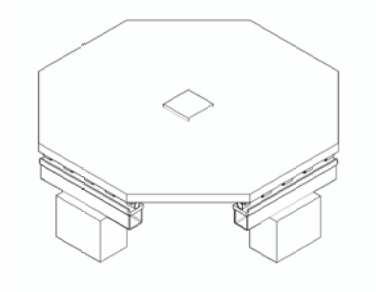
TailorCrete

Conclusions

- The effect of SFRC, seen in experiments, can be estimated with FE-analysis.
- Crack patterns agree when comparing experiments and FE-analyses.
- Utilizing a bond-stress model where the bond stress is reduced post yielding, resulted in more localized crack patterns.
- MC2010 underestimates the beneficial effects from steel fibres for these beams. The underestimation increases with increased fibre content.

Further work



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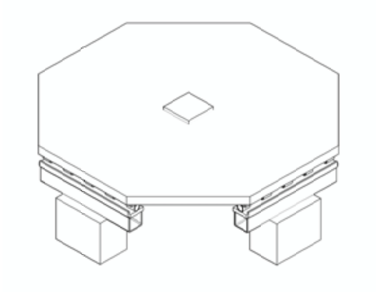
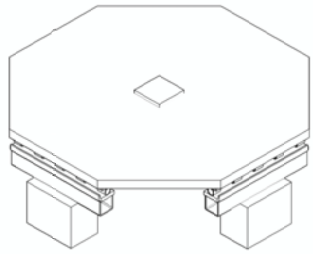
Further work



- Span length, $l = 2.2$m.
- Thickness, $t = 80$mm.
- Deflection measured in 28 points on top surface
- Strain gauges at supports for measuring reaction forces
- Unsymmetric reinforcement

Further work



- Span length, $l = 2.2$m.
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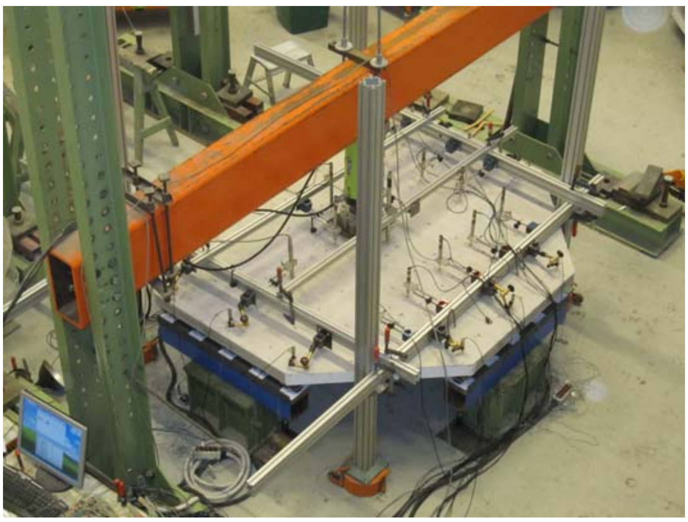
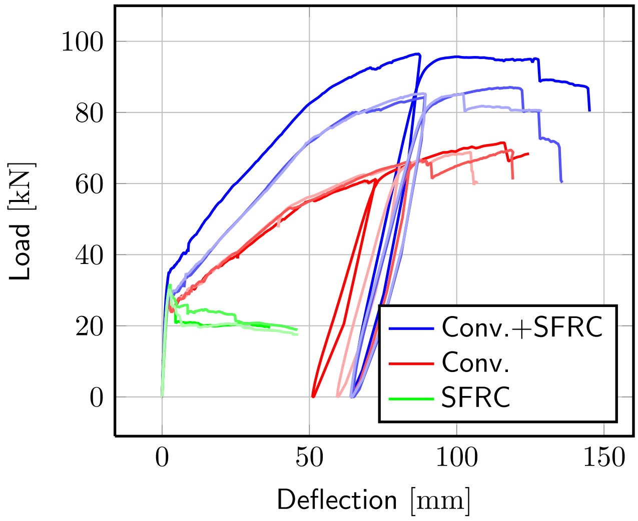
- Deflection measured in 28 points on top surface
- Strain gauges at supports for measuring reaction forces
- Unsymmetric reinforcement

	#	Reinforcement bars	Steel fibres	Textile
Type CR	3	$\phi 6$	-	-
Type CFR	3	$\phi 6$	0.5%	-
Type FR	3	-	0.5%	-
Type BTRC	3	-	-	Basalt
Type GTRC	3	-	-	AR-Glass

Further work



Further work



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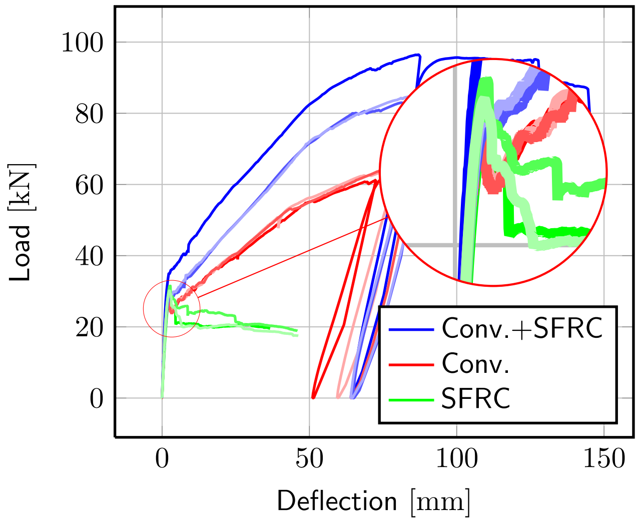
Further work



Further work DIANA Users Meeting 2013 24/27

Further work: Material testing

- Compressive test
- RILEM-beams
- Uniaxial tension test
- Tension test of reinforcement bars

Further work: Modelling

Pre-test models	Jiangpeng Shu and Kamyab Zandi
Detailed non-linear solid modelling	Jiangpeng Shu
Modelling of textile reinforced concrete	Natalie Williams Portal and Kamyab Zandi
Non-linear shell modelling	Master Thesis
Practitioner's approach	Master Thesis
Modelling of RILEM-beams	David Fall

Project partners

DANISH TECHNOLOGICAL INSTITUTE

BEKAERT

D-P DESIGNTOPRODUCTION

DRAGADOS

el aley promoción inmobiliaria

ETH Eidgenössische Technische Hochschule Zürich Swiss Federal Institute of Technology Zurich

GIBOTECH moving technology

GRACE Construction Products

SUPERPOOL

unicon CEMENTIR HOLDING

University of Southern Denmark

david.fall@chalmers.se

Structural appraisal existing box girders

8th International DIANA Users Meeting, Gothenburg, April 25-26, 2013 Henco Burggraaf, TNO, The Netherlands

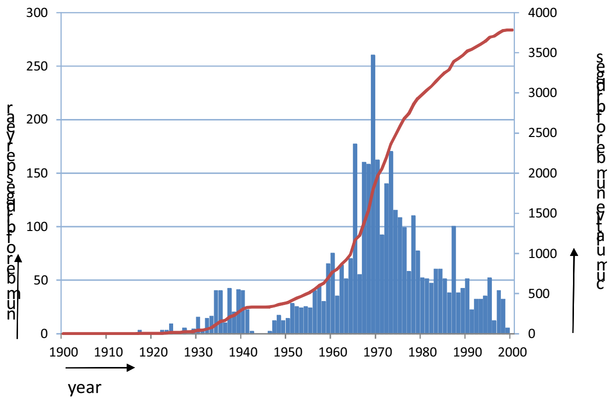
Contents

Introduction

- 2007: recognition of risk that concrete structures, designed before 1974, may have insufficient shear capacity
- Dutch building codes:
  - VB ’74: reduction of shear strength due to higher steel qualities
  - VBC 1990 + VBC 1995: further reduction of shear strength
- Lack of shear capacity may give a brittle failure mechanism, without warning

Introduction

- Bridges in the Netherlands



Introduction

- Assessment of each concrete structure would require too much time
- How can the Dutch Ministry of Infrastructure and Environment guarantee the structural safety of their concrete structures?

Approach

- Corporation between Dutch Ministry of Infrastructure and Environment, TU Delft, TNO and engineering companies
- Risk controlled and phased approach
  - Quick Scans and qualitative classification
  - Advanced analyses
- Fundamental approach
  - Looking for evidence of hidden reserves
  - Research outside the codes and on the edges of available knowledge

Risk controlled and phased approach

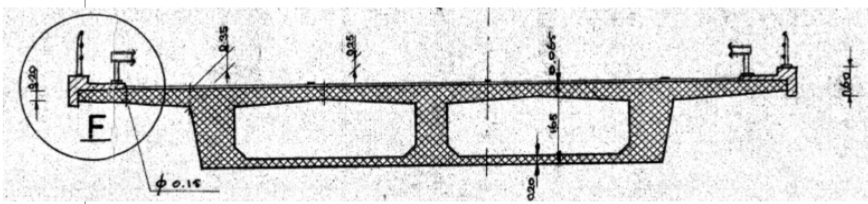
- Use of Quick Scans on basis of conservative assumptions
  - Risk ranking
  - (Advanced) analysis of structures with high risk
  - (Advanced) Analysis of structures with low risk
- Safety measures if shear capacity is not sufficient
  - Two structures up to now

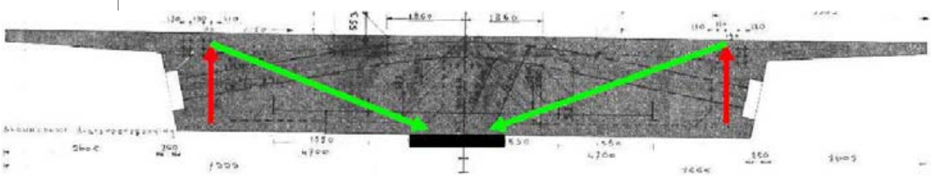
Risk controlled and phased approach

- Use of Quick Scans on basis of conservative assumptions
  - Risk ranking
  - (Advanced) analysis of structures with high risk
  - (Advanced) Analysis of structures with low risk
- Safety measures if shear capacity is not sufficient
  - Two structures up to now

Box girder bridges

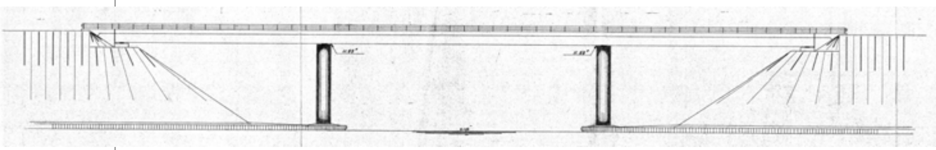
- Critical due to absence of suspension reinforcement

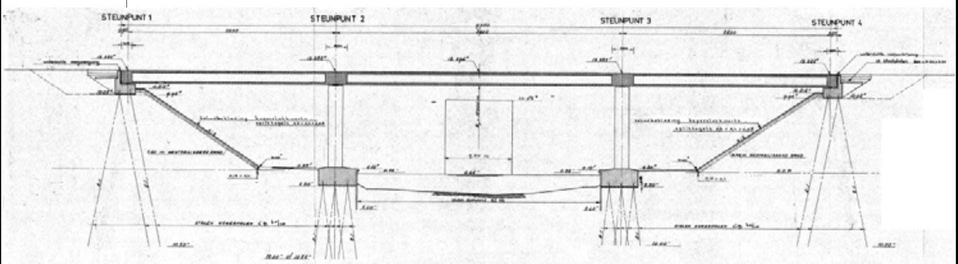




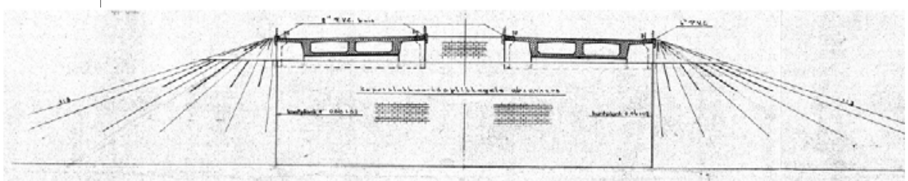
Box girder bridge KW 10H-101

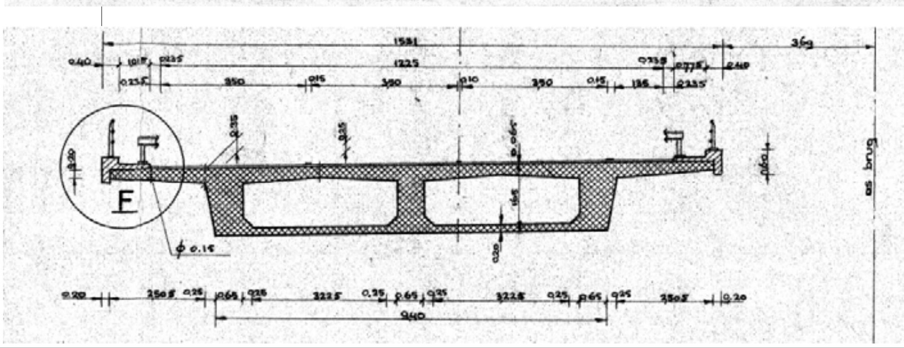
Box girder bridge KW 10H-101





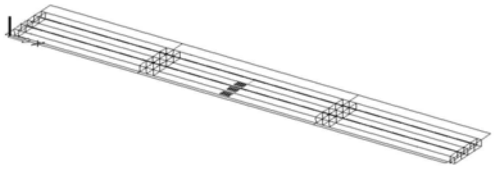
Box girder bridge KW 10H-101





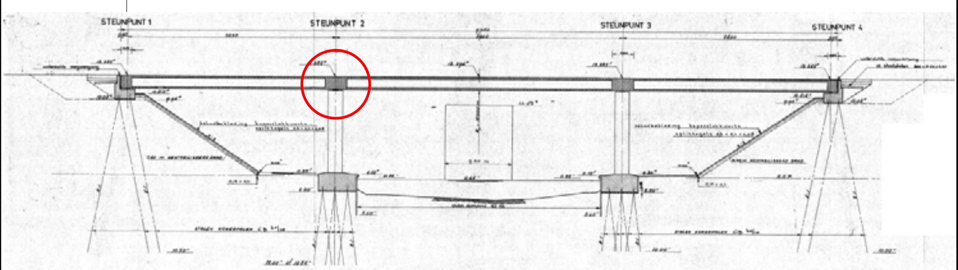
Linear 2.5D analysis

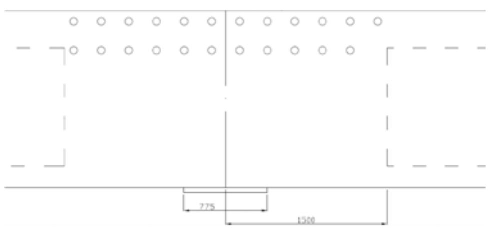
- Model (Scia Engineer)



- Conclusion
  - Shear capacity of mid wall, side walls and crossbeams is sufficient
- Recommendation
  - 3D stress analysis of connection between walls and crossbeams

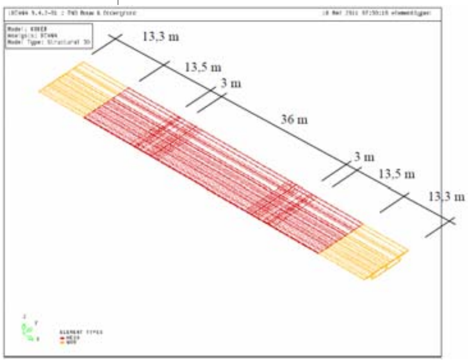
Connection walls and crossbeams

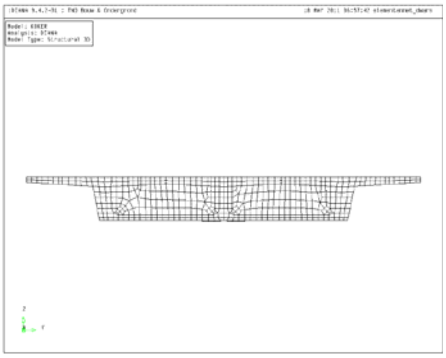




Linear 3D analysis

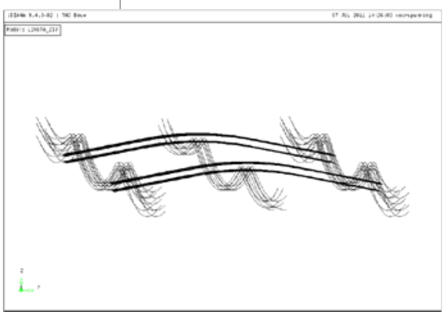
- Geometry and mesh
  - CHX60 & CQ40S

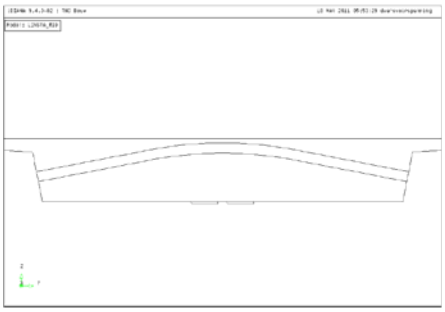




Linear 3D analysis

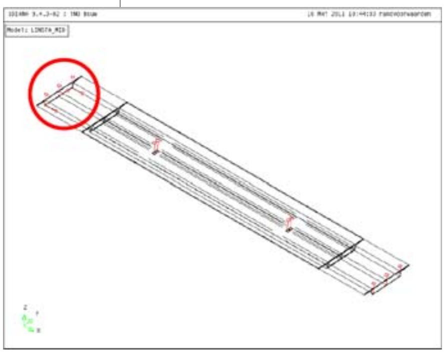
- Prestressing tendons

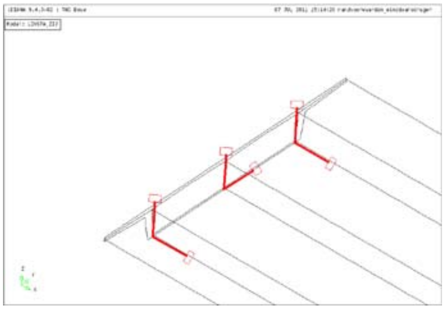




Linear 3D analysis

› Boundary conditions





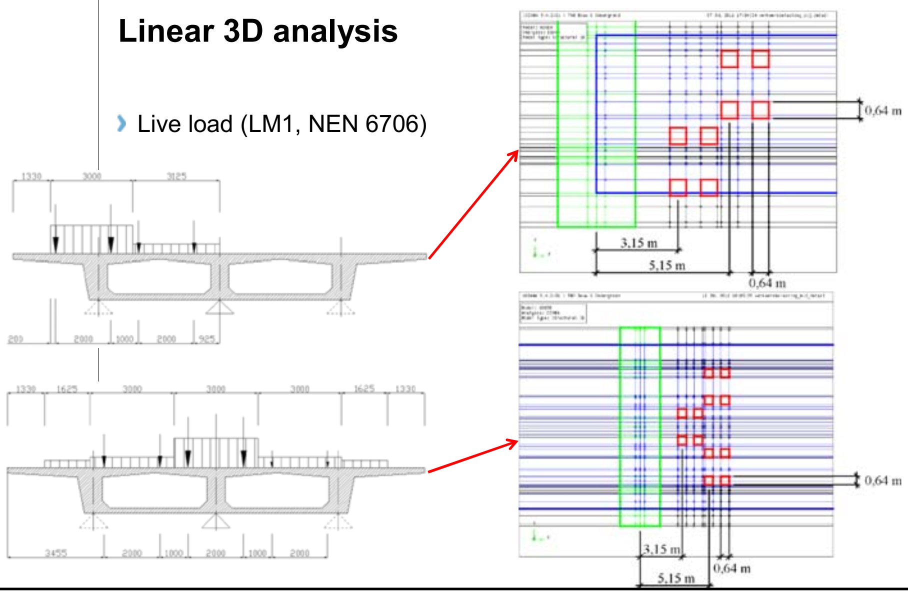
Linear 3D analysis

› Loads

- Permanent load (self weight, asphalt)
- Prestress load
- Live load (LM1, NEN 6706)

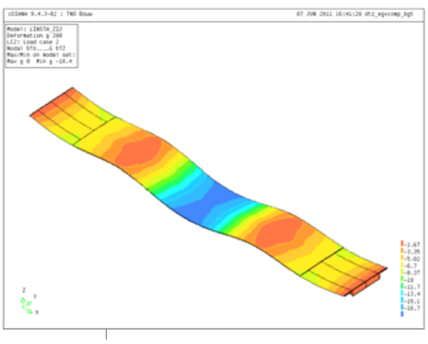
Linear 3D analysis

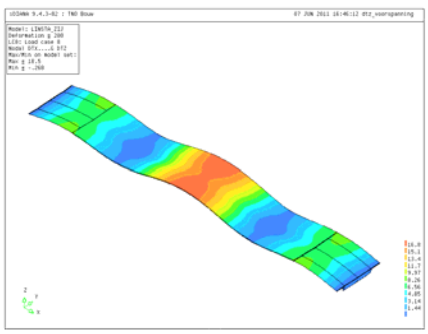
› Live load (LM1, NEN 6706)



Linear 3D analysis

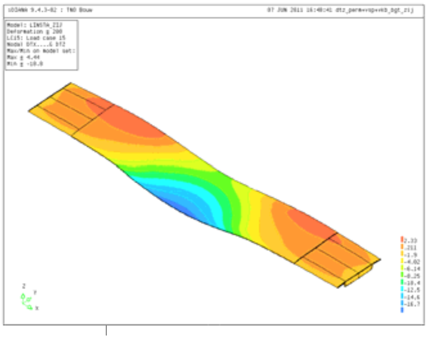
› Displacements permanent load (left) and prestress load (right)

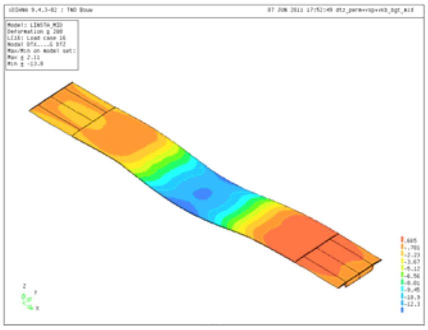




Linear 3D analysis

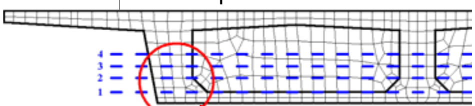
- Displacements SLS_{side} (left) and SLS_{mid} (right)



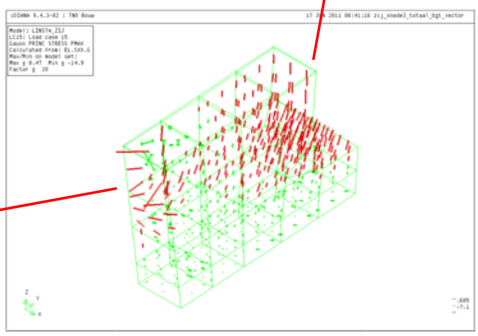


Linear 3D analysis

- Principle stress connection wall and crossbeam



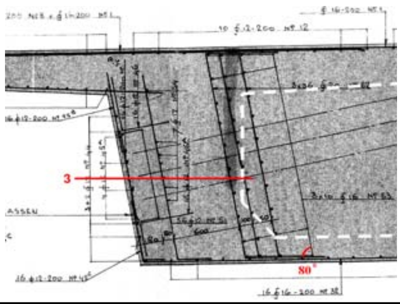
max. 3.68 MPa



local effects due to
transverse prestress

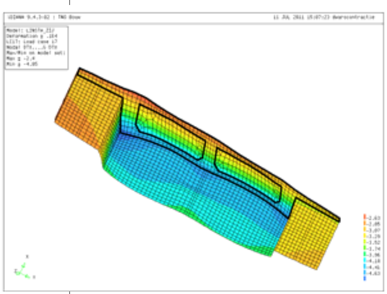
Linear 3D analysis

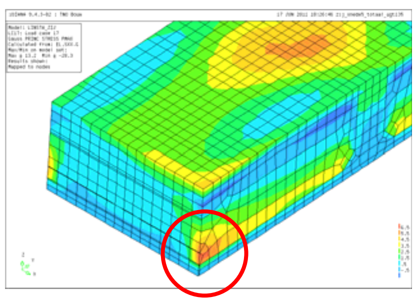
- Unity checks principle stress connection wall and crossbeam
  - Connection side wall and cross beam: U.C. = 3.68/2.04 = 1.80 > 1 → ☹
  - Connection mid wall and cross beam: U.C. = 1.81/2.04 = 0.89 < 1 → ☺
- Considering splitting reinforcement near side wall
  - Area where tensile strength is exceeded:
    $A = 0.875 \cdot 200^2 = 35000$ mm^2
  - Tensile force:
    $F = (35000 \cdot (3.68+2.04)/2) \cdot 10^{-3} = 100.1$ kN
  - Required reinforcement cross section:
    $F = (100100/330)/\sin 80°$
    $= 308$ mm$^2 < 2 \cdot 314$ mm^2 → ☺



Linear 3D analysis

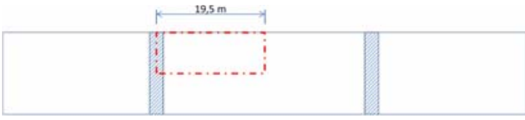
- Increased principle tensile stress at surface just above bottom slab
- Insufficient reinforcement to withstand the increased tensile stress
- Study the effects with a non-linear 3D analysis

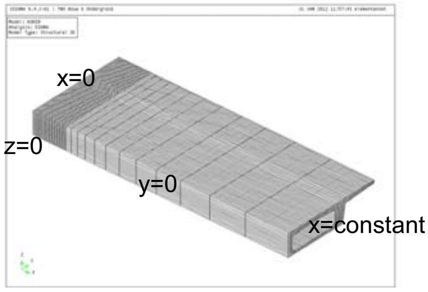




Non-linear 3D analysis

- Model (DIANA)
  - Box girder: 20702x CHX60
  - Support: 18x CQ48I



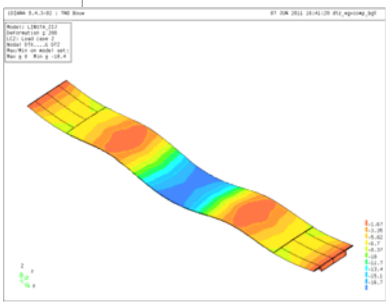


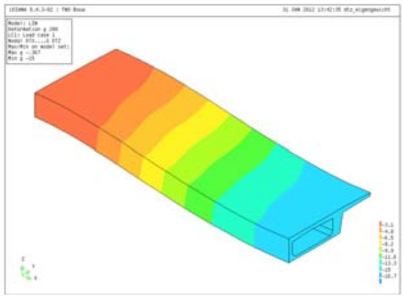
Non-linear 3D analysis

- Material properties concrete
  - Hordijk softening (tension), parabolic softening (compression)
  - Parameters based on EN 1992-1-1 and MC 1990 with $f_{ck,cube}$ = 42,2 N/mm^2
- Material properties reinforcement
  - Von Mises with f_y = 330 N/mm^2

Non-linear 3D analysis

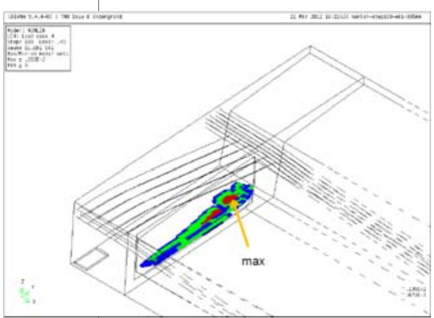
- Comparison 3D models
  - Non-linear 3D model stiffer
  - Non-linear 3D model conservative with respect to principle tensile stresses just above bottom slab

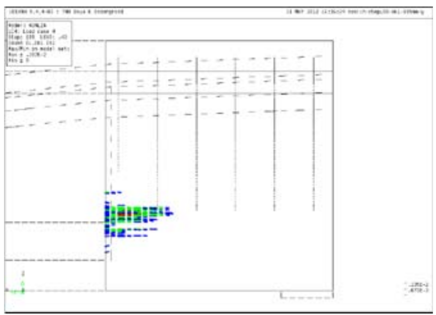




Non-linear 3D analysis

- Cracking SLS
  - Maximum crack width (0.2 mm) smaller than allowed crack width XC3 (0.3 mm)

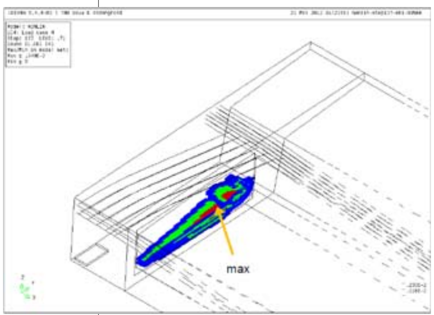


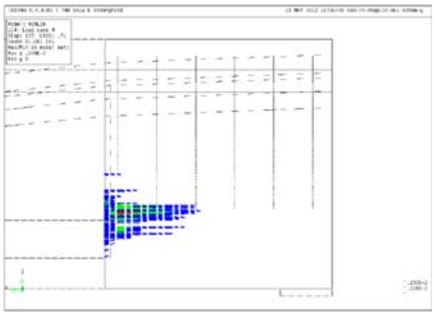


Non-linear 3D analysis

› Cracking ULS

- No cracking over full thickness of crossbeam

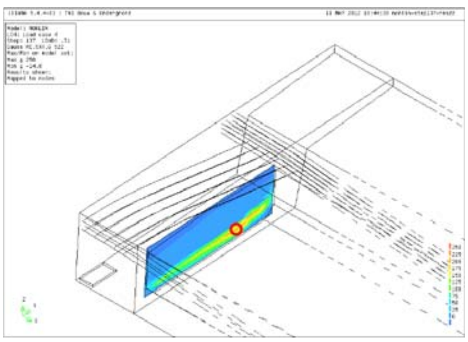




Non-linear 3D analysis

› Reinforcement stresses ULS

- Maximum reinforcement stress (258 MPa) smaller than yield stress (330 MPa)



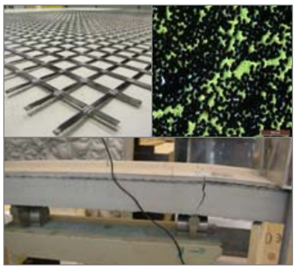
Conclusions

- Linear 2.5D analysis
  - Shear capacity of mid wall, side walls and crossbeams OK
- Linear 3D analysis
  - Connection between walls and crossbeams OK
- Non-linear 3D analysis
  - Crossbeam just above bottom slab OK

End of presentation

Thank you for your attention

Numerical Modelling of Textile Reinforced Concrete (TRC)



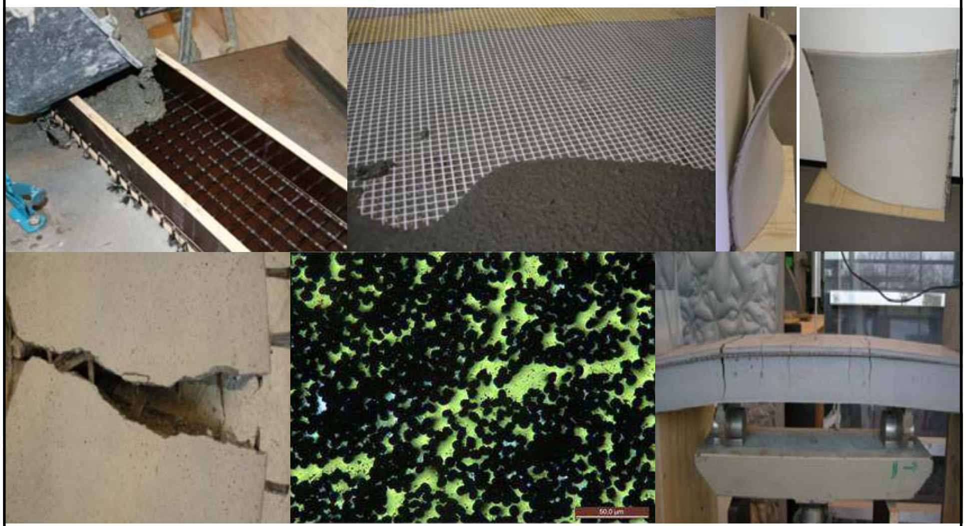
8th International DIANA Users Meeting 2013
Gothenburg, April 25-26 2013

NATALIE WILLIAMS PORTAL

Overview

- Explore modelling methods for TRC
- Two studies:
  - 1) TRC Beam
  - 2) Two-way TRC slab
- Validation of Numerical analyses

Textile Reinforced Concrete (TRC)

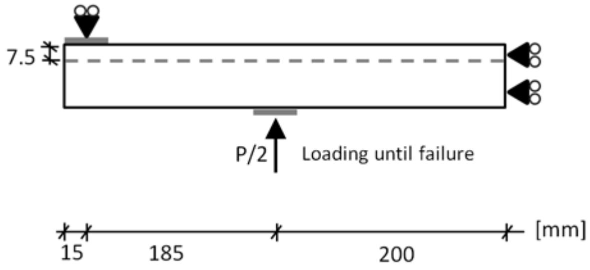


STUDY 1

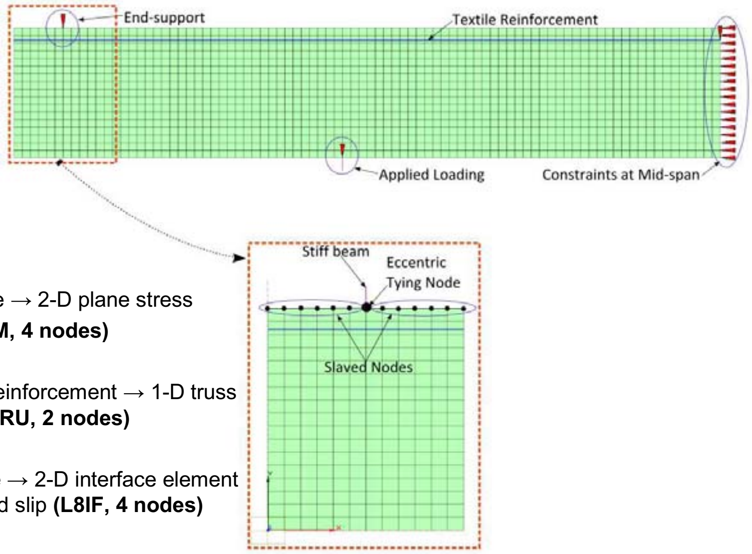
General Model Features – Study 1

- 2-D non-linear model of thin beam (8 cm) under bending stress
- Displacement control

- Idealized support and boundary conditions:



Element Selection

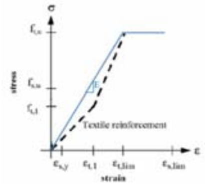


- Concrete → 2-D plane stress **(Q8MEM, 4 nodes)**
- Textile reinforcement → 1-D truss bar **(L2TRU, 2 nodes)**
- Interface → 2-D interface element with bond slip **(L8IF, 4 nodes)**

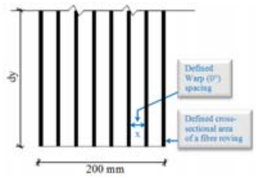
Material Characterization

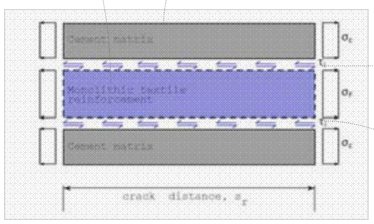
Total strain based crack model with rotating crack:
→Modified Thorenfeldt curve
→Hordijk tension softening model
→Crack band width = element size

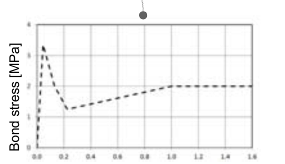
Simplified bi-linear stress-strain
→Ideal-plastic after ultimate strength



Monolithic reinforcement



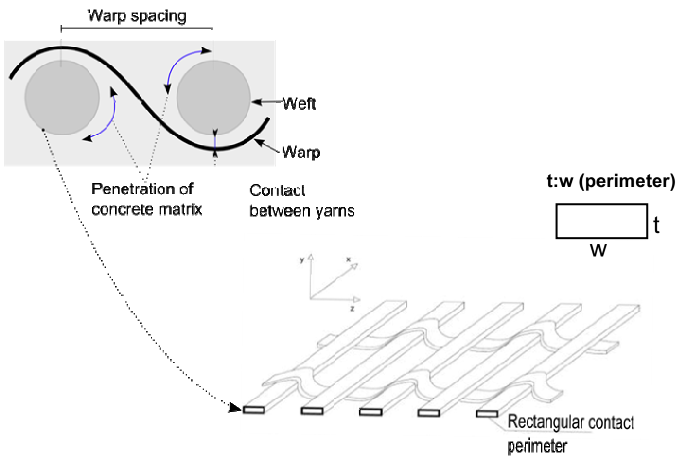




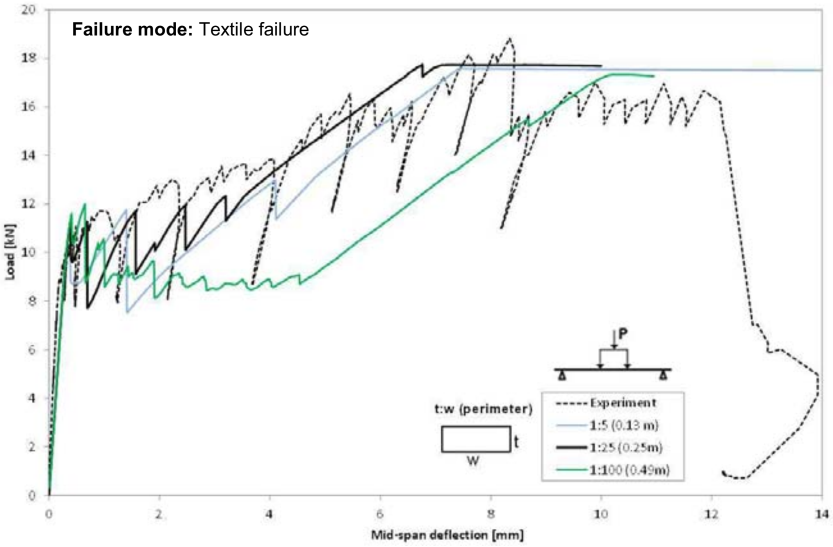
(Schladitz et al., 2011)

Bond Interface

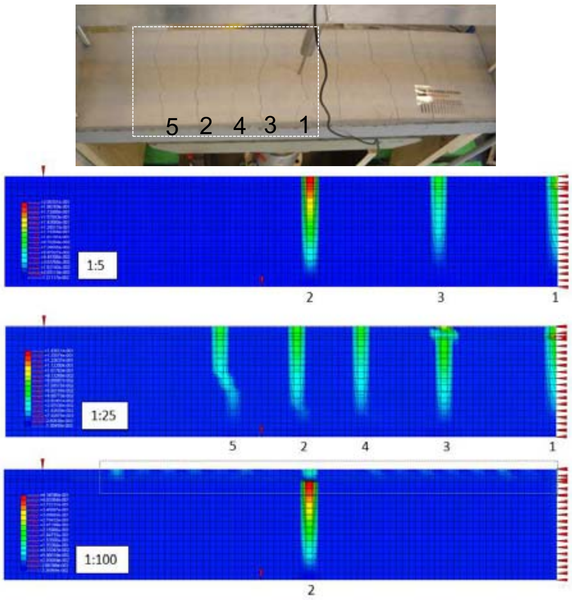
- Contact perimeter = sensitive parameter in model
- Vary thickness:width (t:w) of *rectangular* contact perimeter



Load versus mid-span deflection



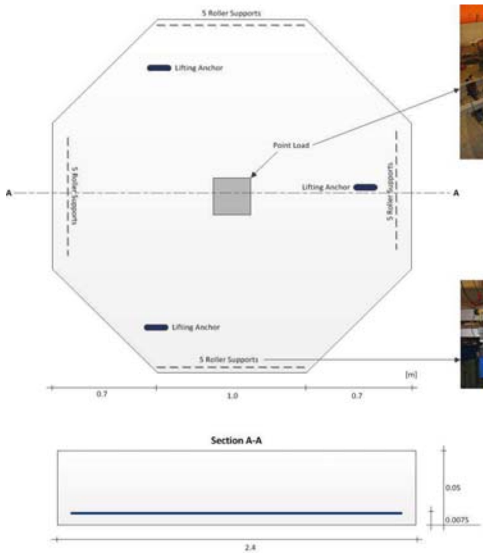
Crack pattern vs. Simulation

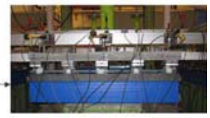


Brief Summary – Study 1

- **Sensitive modelling parameter** = Contact perimeter
  - Affects crack pattern and stiffness in cracked region
- Reasonably good agreement between calibrated numerical results **(1:25)** and experimental results
- Model only depicts **textile failure** and not actual failure **delamination**

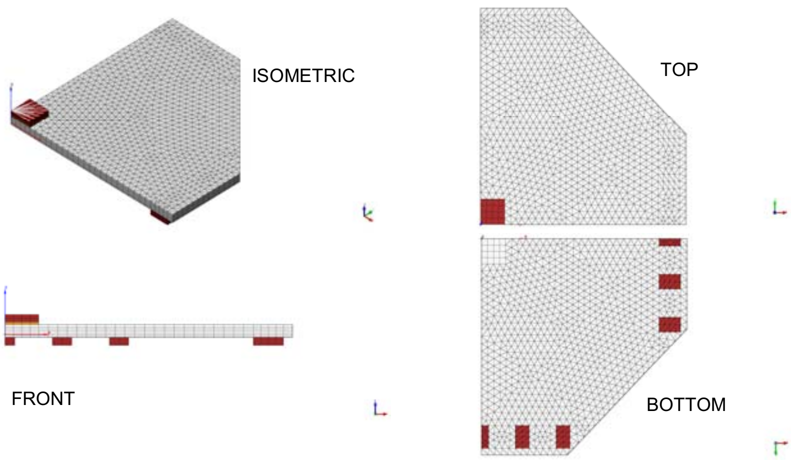
Study 2 – Two-way Slab



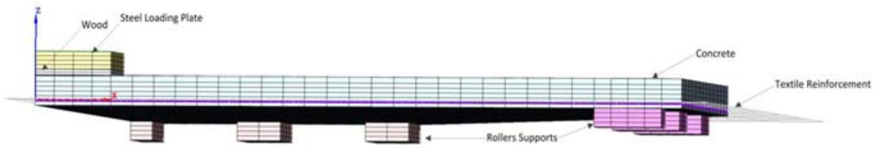


General Features – Two-way Slab

- 3-D non-linear model of two-way slab (5 cm) under point load
- Displacement control
- Idealized support and boundary conditions:



Element Selection



- Concrete, Wood and Steel Loading Plate → 3-D solid elements **(HX24L, TP18L, 8 & 6 nodes)**
- Textile reinforcement → Reinforcement Grid in Solid **(nodes according to embedded element)**
- Interface between supports and loading → 3-D interface elements **(Q24IF, T18IF, 8 & 6 nodes)**

Material Characterization

Reinforcement Grid:

- Contribute to element stiffness/nodal force

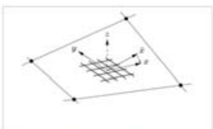


Figure 14.24: Axes of reinforcement grid

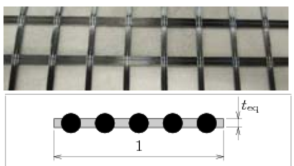


Figure 14.25: Equivalent thickness of reinforcement grid

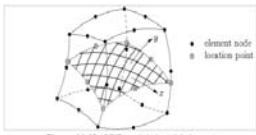
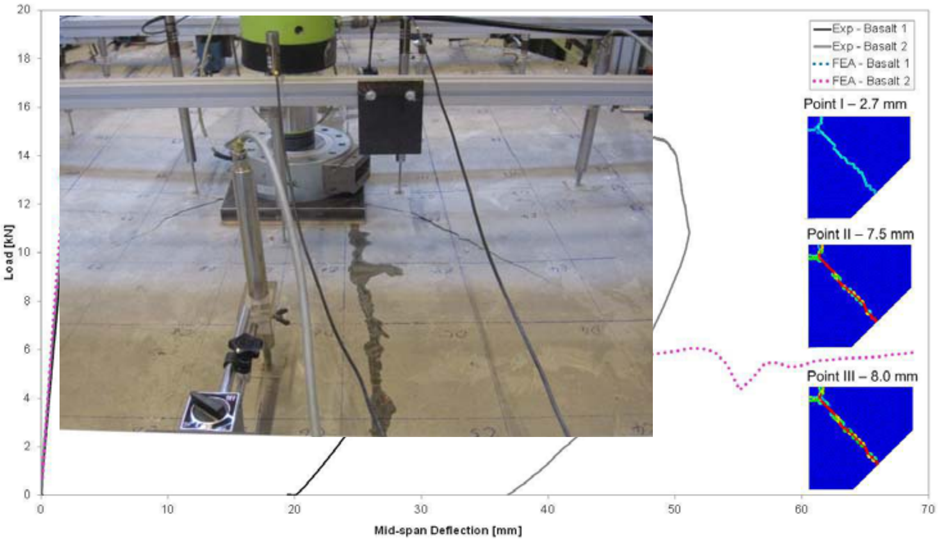


Figure 14.43: GRID particle in solid element

Concrete:

- Similar to Study 1
- Multi-linear tensile curve

Load versus mid-span deflection



Discussion

- STUDY 1 – How to improve model?:
  - Weaken concrete elements surrounding textile reinforcement
  - 3-D modelling
  - Bi-linear stress-strain relationship for textile reinforcement
- STUDY 2 – How to improve model?:
  - Possible bond-slip in reinforcement grid and solid elements?
  - Bi-linear stress-strain relationship for textile reinforcement

THANK YOU FOR YOUR KIND ATTENTION!

Acknowledgments:

•Danish Technological Institute (DTI)
•European Community's Seventh Framework Programme under grant agreement NMP2-LA-2009-228663 (TailorCrete)
•FORMAS – Swedish Research Council (Homes for Tomorrow)

Crackfree Concrete Structures"

Anja B. E. Klausen, PhD candidate
Gunrid Kjellmark, MSc, researcher
Professor Terje Kanstad

NTNU, SINTEF & COIN

Objectives & Project particpants

Main project activities

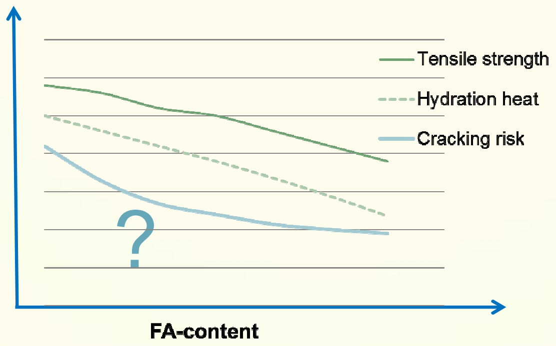
- Experimental work
- Calculation methods and computer programs
- Crack risk assessment of real structures
- Long term research topics

NTNU SINTEF

To establish (and maintain) a methodology for crack assessment of concrete structures

Activities ranging from materials testing to FE Analysis and crack-development-mapping in large concrete structures

Crack risk vs FA content



Active industrial partners:

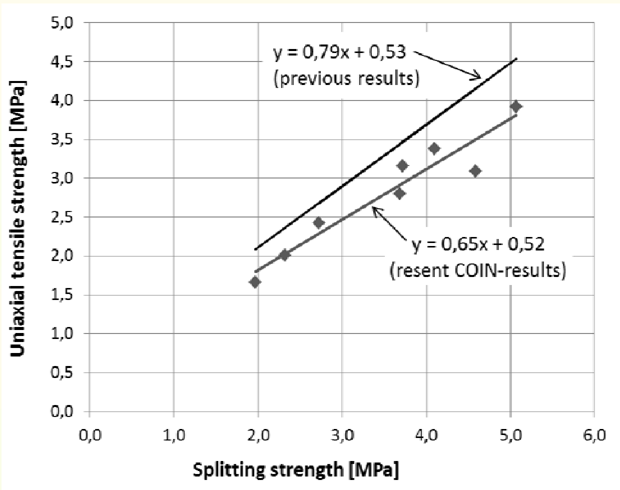
Norcem (Knut Kjellsen (project manager)
Norwegian public roads authorities (Øyvind Bjøntegaard)
Skanska, Norway (Sverre Smeplass)
Unicon, Mapei and Veidekke

Experimental work comprises:

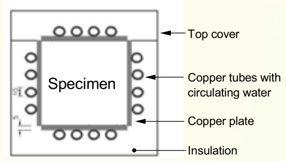
- Strength development vs maturity (ft, Ec, fc)
- Semi adiabatic heat development
- Autogenous shrinkage (for up to 6 months)
- Drying shrinkage
- Thermal dilation
- Creep in compression and tension
- TSTM (Temperature Stress Testing Machine) – which simulates the stress development in real concrete structures under realistic temperature history and degree of restraint
- Field observations

- **Problems with the shrinkage, thermal dilation and TSTM-equipment. Large resources have been spent.**

Uniaxial vs splitting tensile strength for FA concretes

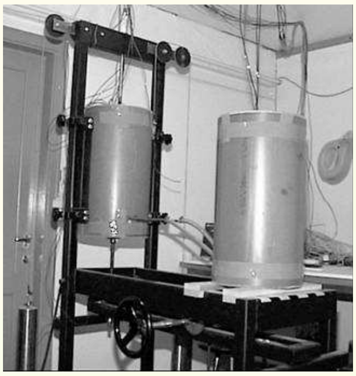


TSTM & Dilation rig

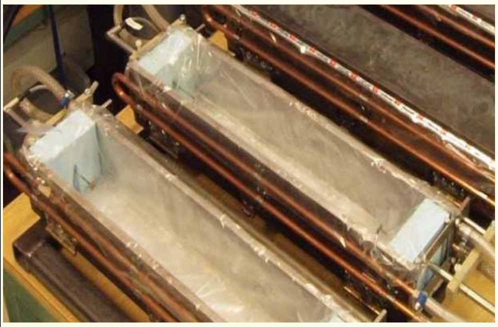


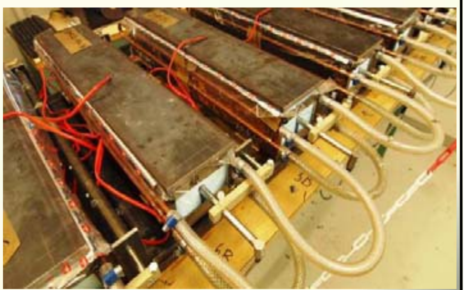
The situation in the structure's critical section can be tested with the right material, realistic temperature history and the right degree of restraint

Compressive and tensile creep



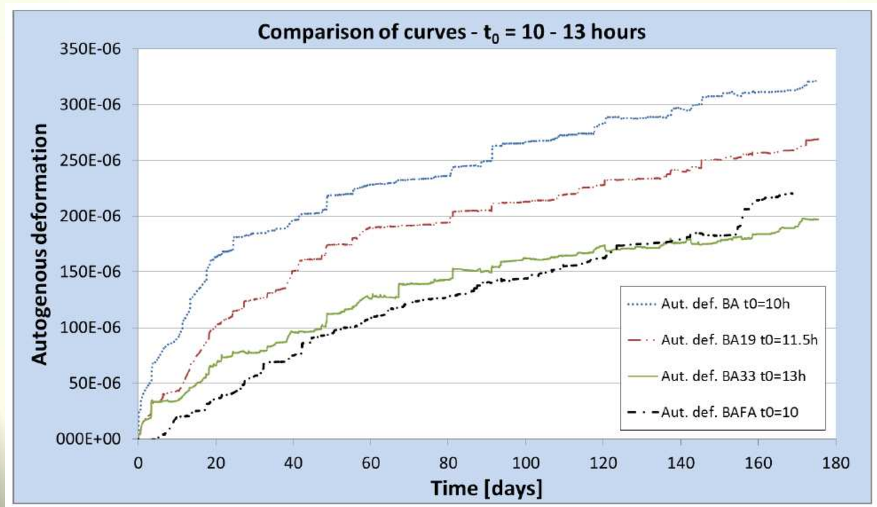
7 free deformation rigs with temperature control





Longterm autogenous shrinkage

Concrete with different amount of FA as cement replacement

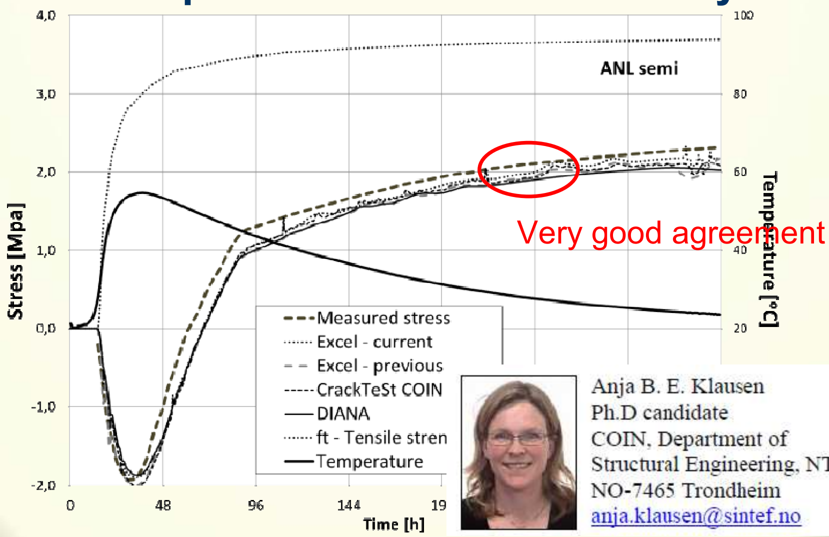


Calculation methods

Based on linear visco-elastic theory for aging materials

- In house Excel or visual basic programs
- **CrackTestCOIN** – Special purpose program based on the Swedish program ConTestPro (TULuleå)
- **Diana** - General purpose FE Program (Delft, NL)

Example: TSTM results vs theory



Anja B. E. Klausen
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COIN, Department of Structural Engineering, NTNU
NO-7465 Trondheim
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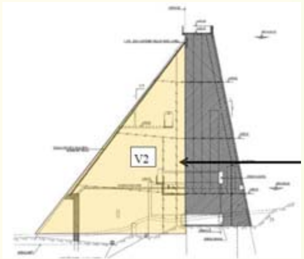
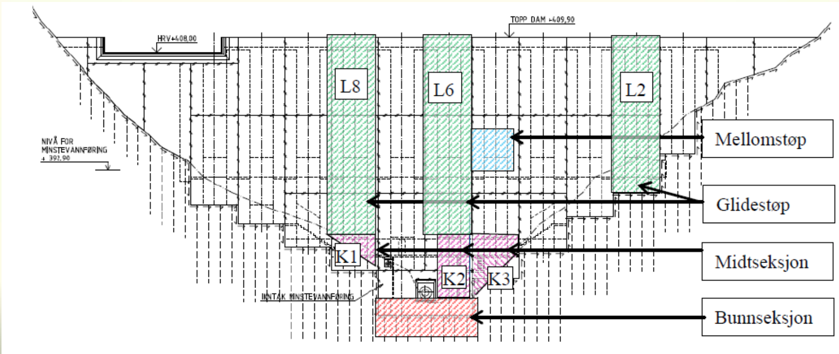
Project where the researchers presently are involved:

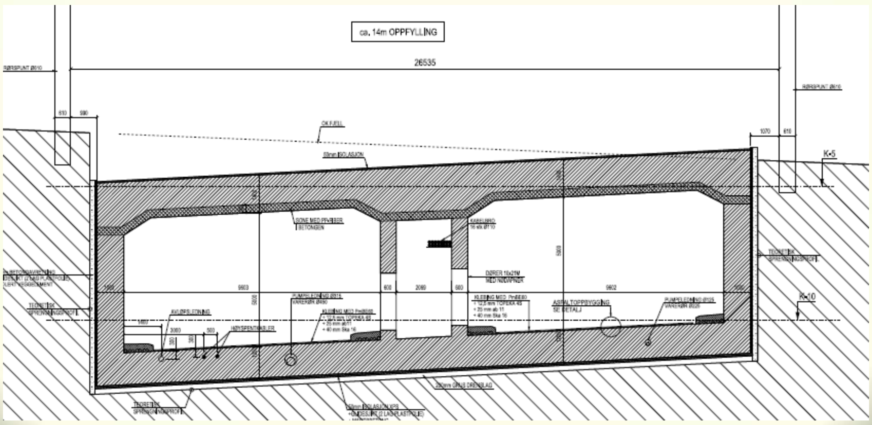
Dam NN





Projects where the researchers presently are involved(II):

The Møllenberg tunnel (culvert) Trondheim



Project where the researchers will be involved(III):

Edge beams in concrete bridges

Longterm research questions (Ia)

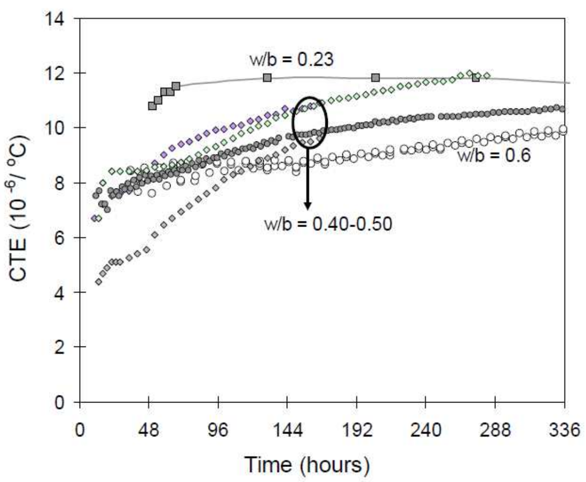
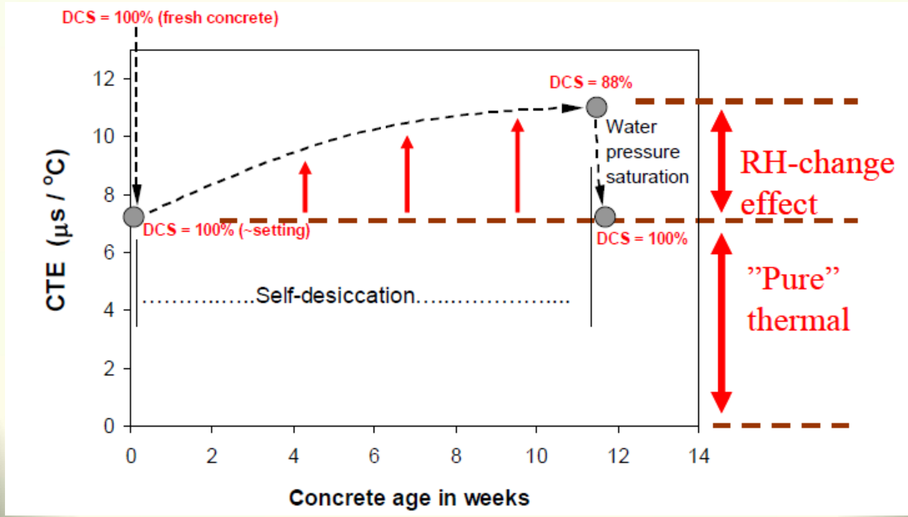


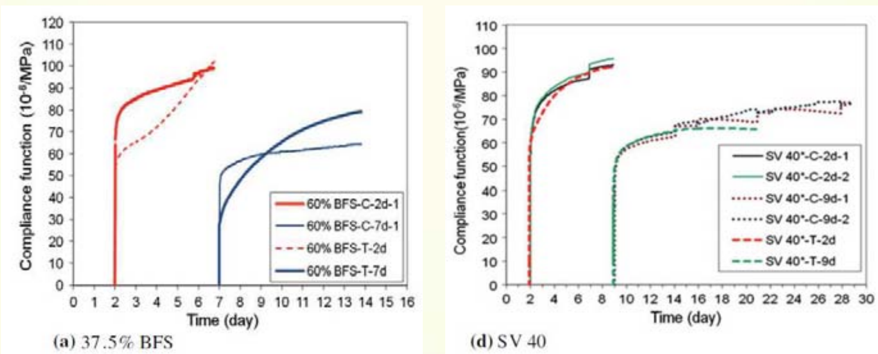
Fig. 17 Laboratory measurements of the coefficient of thermal expansion (α_T = CTE) over time for different concretes with variable water-to-binder (w/b) ratio. [19]

Longterm research questions (Ib)



Longterm research questions (II):

Tensile vs compressive creep



COIN

Thank you for your attention!

Finite element modeling of a two-way slab with different approaches

Jiangpeng Shu, M. Plos, K Lundgren, K Zandi Hanjari

Division of Structural Engineering,
Chalmers University of Technology,
Gothenburg, Sweden.

2013-04-26

Background and objective

- Project:
  Load carrying capacity of existing bridge deck slabs
- Overall aim:
  To develop improved methods for assessment with non-linear and linear FE analysis, as well as analytical methods
- Objective of this study:
  To study and verify modeling methods for non-linear analysis of two-way slabs

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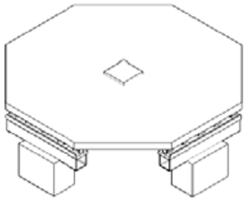
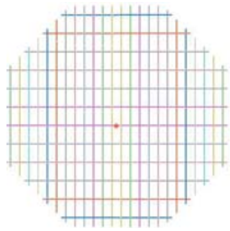
Test

Test set-up



- Span length, $l = 2.2$m.
- Thickness, $t = 80$mm.
- Deflection measured in 28 points on top surface
- Strain gauges at supports for measuring reaction forces
- Unsymmetric reinforcement



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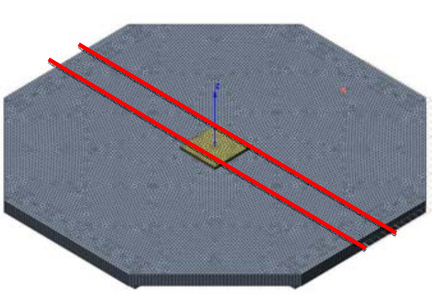
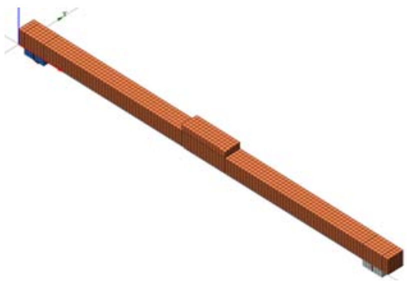
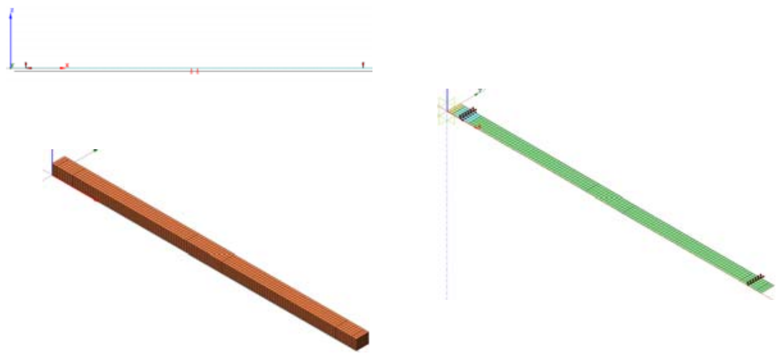
Finite element, beam strip analysis





- Take a beam strip from the slab
- Dimension: 98mm*196mm*2400mm
- With one reinforcement inside

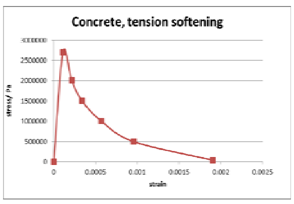
Model

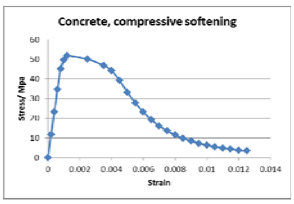


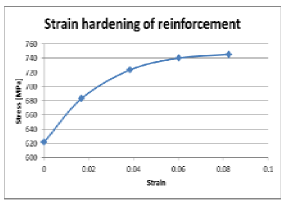
- Three different models for beam strip: beam (L7BEN), shell (Q20SH) and solid (HX24L) element models

Material model

Concrete		Reinforcement	
Young's modulus	24.6 GPa	Young's modulus	187 GPa
Poison's ratio	0.15	Poison's ratio	0.3
Crack band width	Element size	Yield strength	620 MPa
Tensile strength	2.6 MPa		
Compressive strength	51.7 MPa		







- Concrete: Smeared cracking, total strain based rotating crack model
- Reinforcement: Embedded reinforcement bar with Von Mises plasticity model, Yield strength 620 MPa

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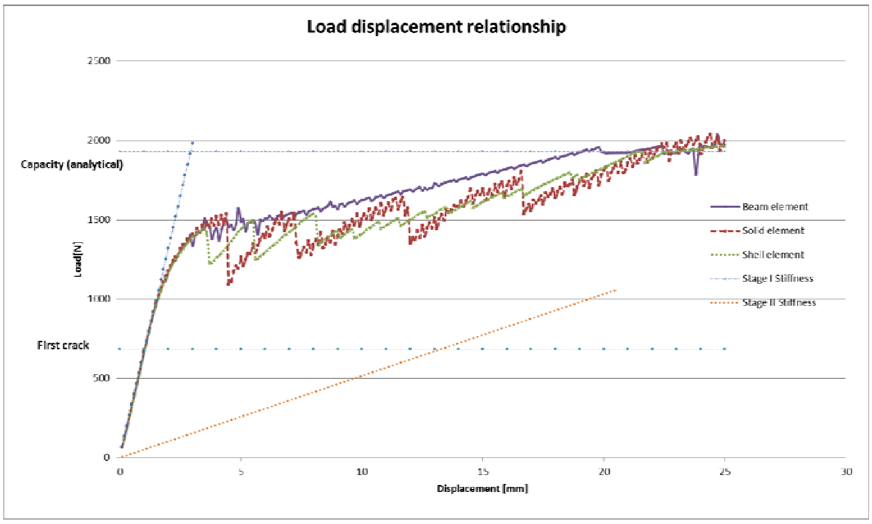
Result of beam strip analysis



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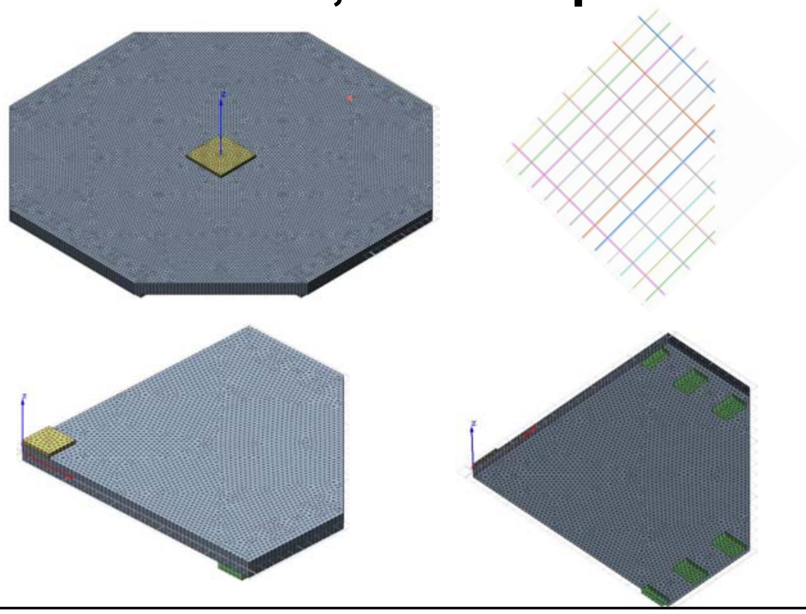
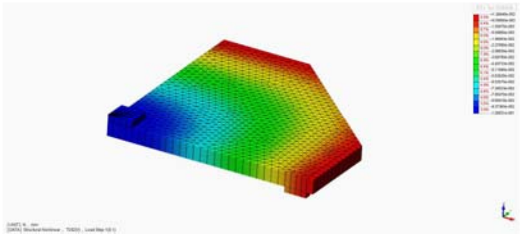
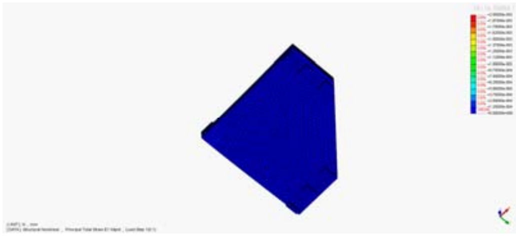
Slab model, with steel plates

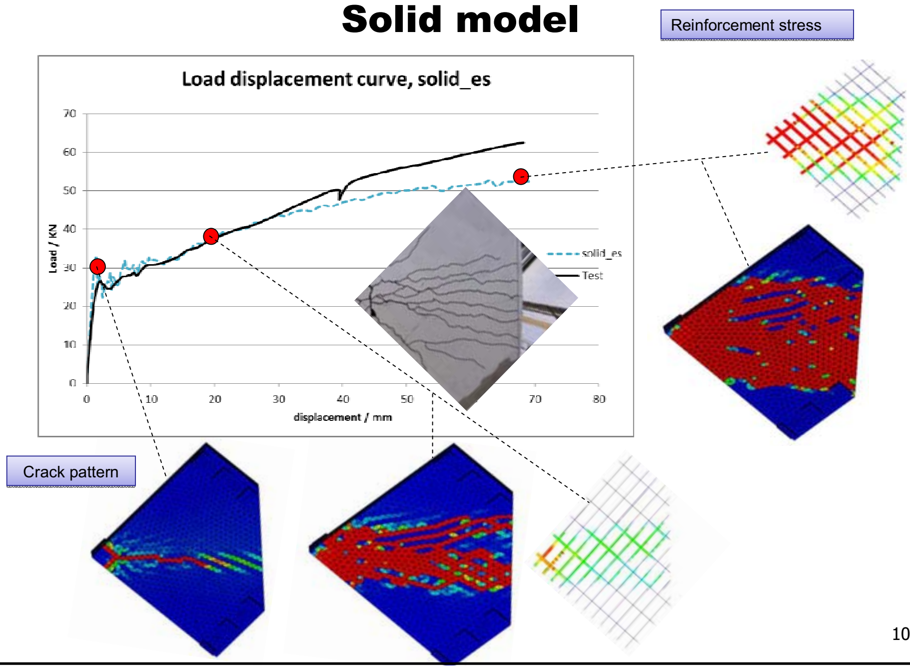


Solid model





Solid model



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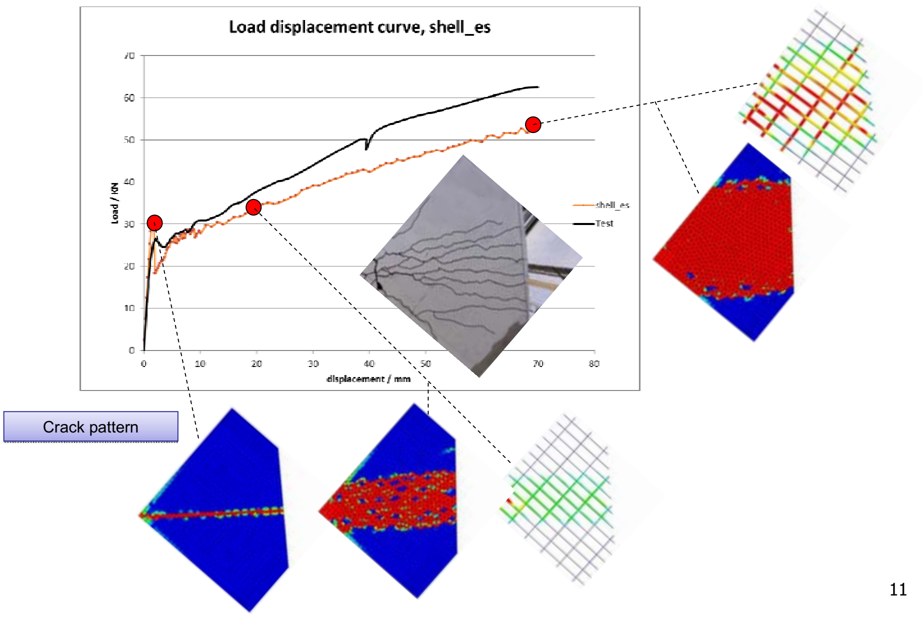
Shell model

Reinforcement stress



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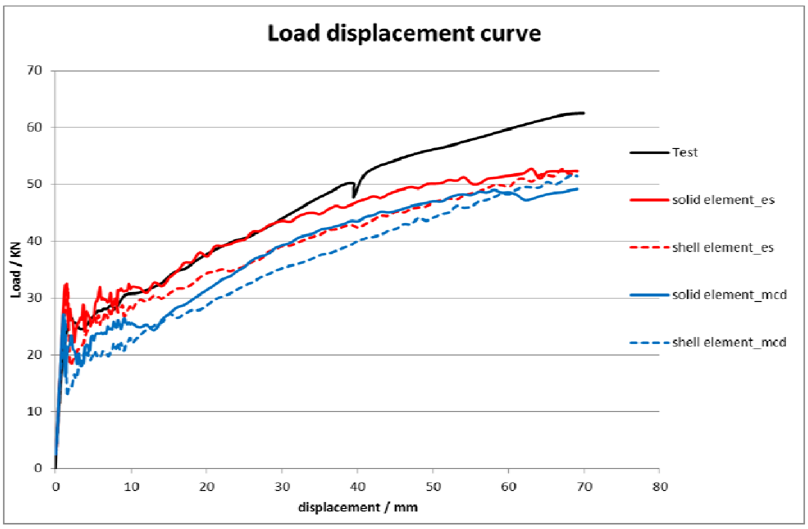
Result of slab test analysis



- _es: crack band width = element size
- _mcd: crack band width = mean crack distance

Conclusions

- The analysis of the two-way slab with both solid and shell elements gave results consistence with that in the experiments.
- Element type, analysis parameters can influence results largely, e.g. the crack band width
- By using shell element, the calculation time can be decreased by a 50%
- Study is still in progress
  - All aspects of the response not yet studied
  - Still differences in response between FE model and test
  - Alternative models for crack and element type are to be used

Modelling of tensile membrane action at very large displacements by use of DIANA

Dirk Gouverneur

Magnel Laboratory for Concrete Research – Department of Structural Engineering

8th International DIANA Users Meeting, Gothenburg 26/04/2013

Outline

1. Introduction
   - Robustness
   - Membrane action
   - Belgian case study
2. Research project
   - Test set-up
   - Loading procedure
   - Results
3. DIANA model
   - Mesh
   - Results
4. Conclusion

Robustness

"No disproportionate collapse after initial damage"

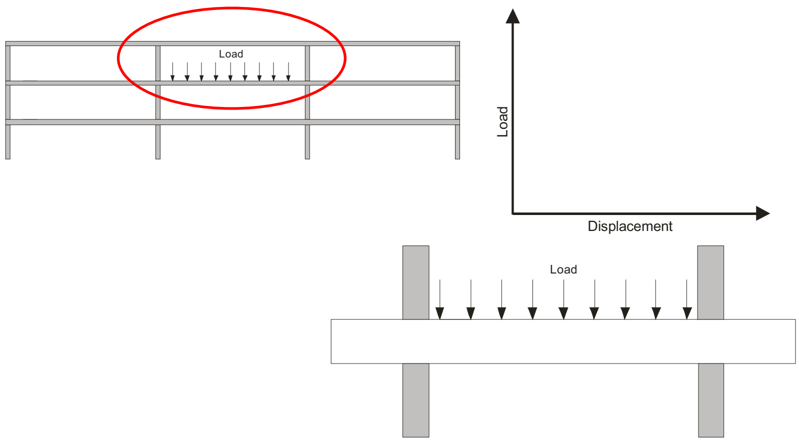
•Goal:

- Protection of occupants

•Tools:

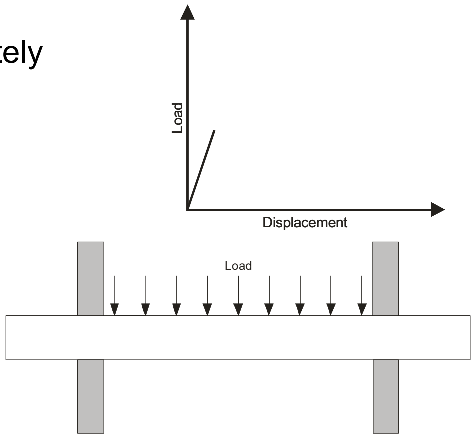
- Alternate load paths
- Key elements
- Compartmentation

Membrane action



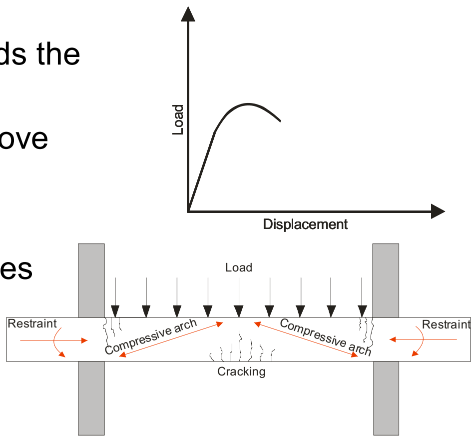
Membrane action

- Linear-elastic behaviour
- Neutral axis is approximately at mid-depth



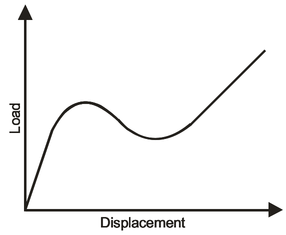
Compressive membrane action

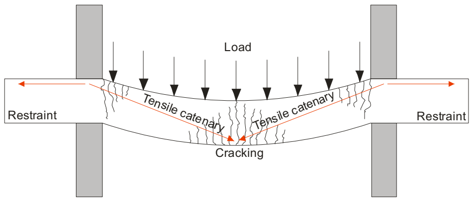
- Small displacements
- Neutral axis moves towards the compressive zone
- Edges of the slab try to move outwards
- Sufficient lateral restraint
- in-plane compressive forces
- Compressive arch



Tensile membrane action

- Large displacements
- Catenary actions occur
- Advanced state of cracking
- Reinforcement acts as a tensile net





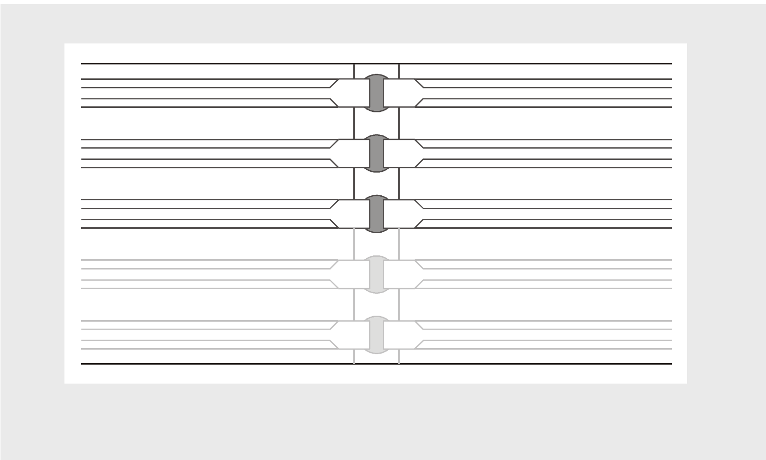
Belgian case study

- Truck impact on a pillar of Wemmel bridge on the ring road of Brussels (July 2007)

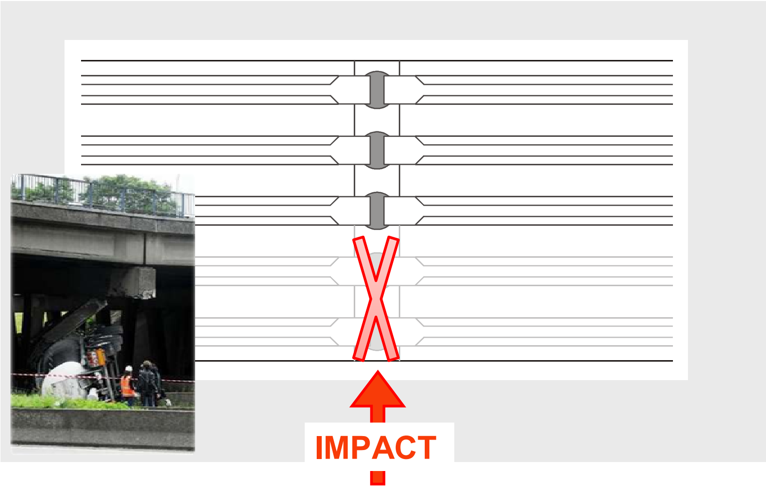
Belgian case study

- Truck impact on a pillar of Wemmel bridge on the ring road of Brussels (July 2007)
- 2 pillars and head beam destroyed
- 2 pillars damaged
- 25 cm displacement of the deck
- **No progressive collapse** (due to membrane action of the bridge deck)
- Repair: the deck was jacked up, new pillars were installed and the deck is being repaired

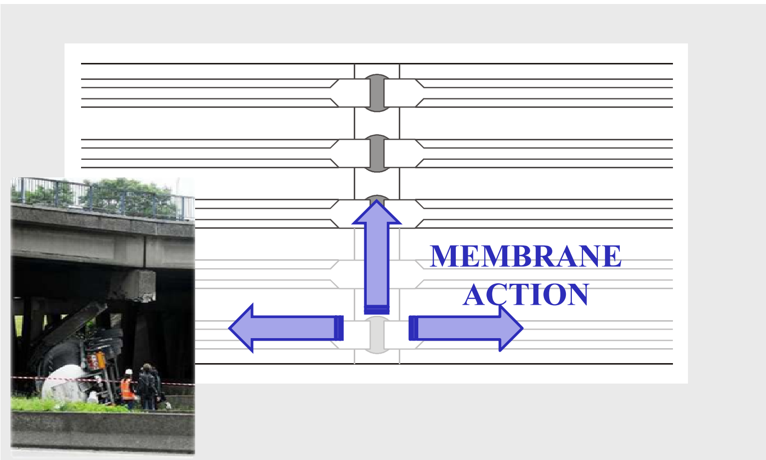
Belgian case study(1)



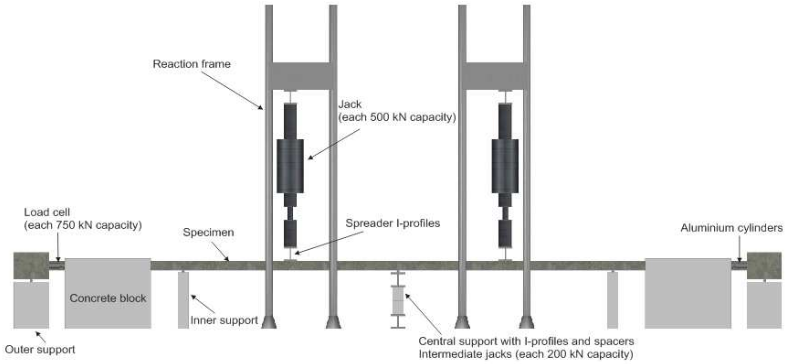
Belgian case study(2)



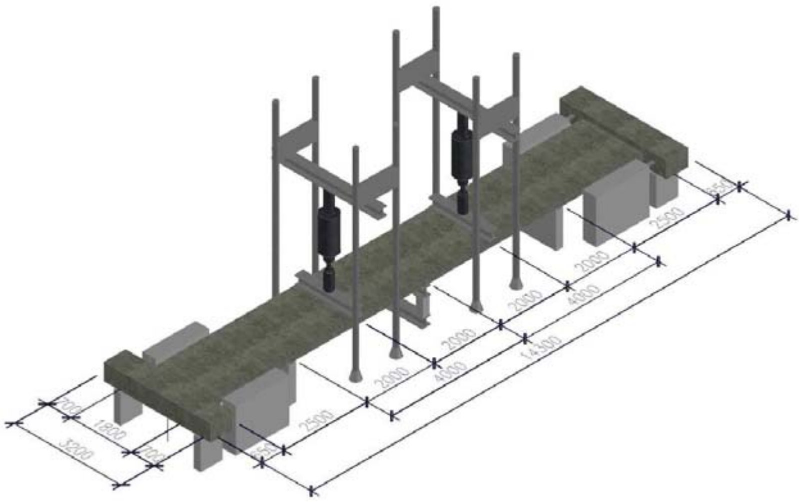
Belgian case study(3)



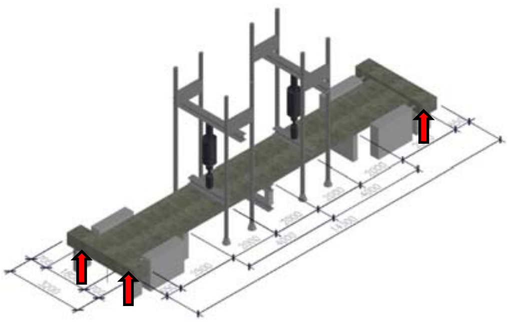
Test set-up



Test set-up

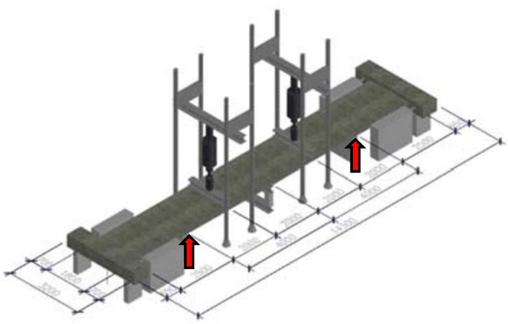


Test set-up

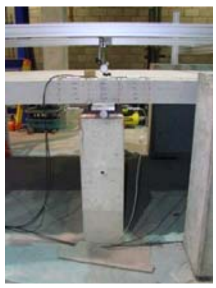


Vertical supports
- Edge beams

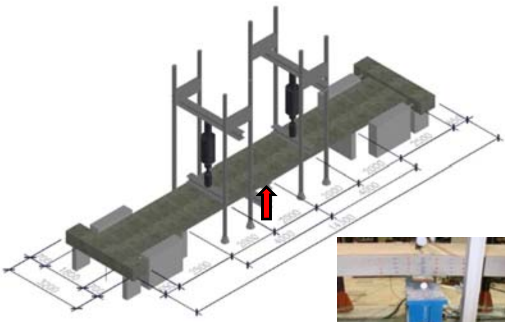
Test set-up



Vertical supports
- Edge beams
- Inner supports

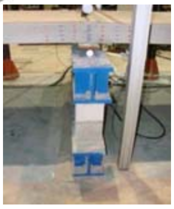


Test set-up

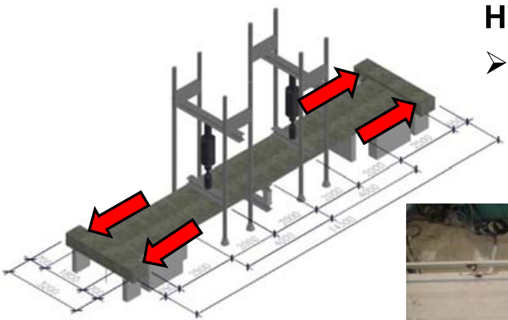


Vertical supports

- Edge beams
- Inner supports
- Central support (simulated failure)

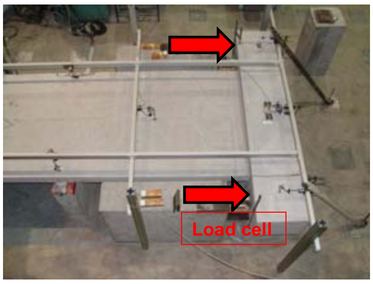


Test set-up

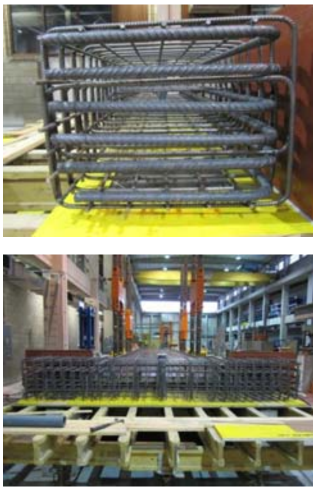


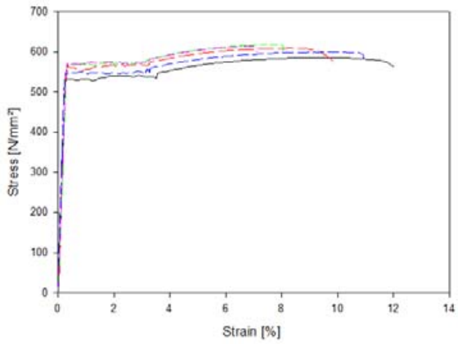
Horizontal supports

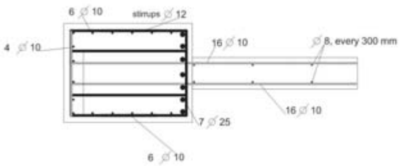
- Restraining concrete blocks



Reinforcement



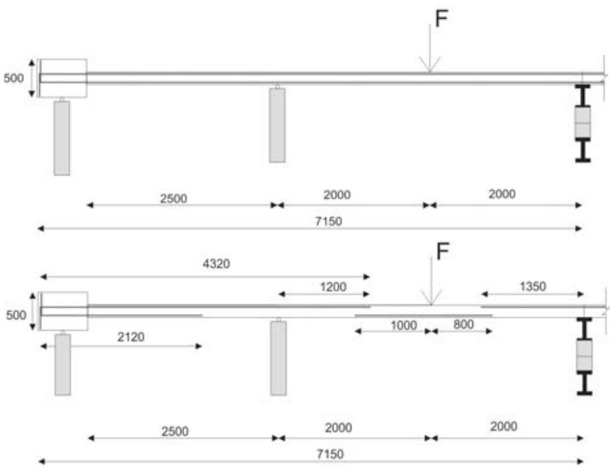




Reinforcement arrangement

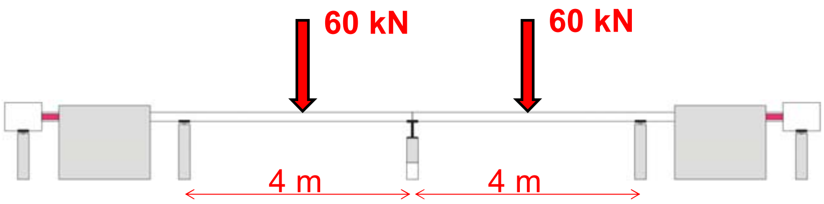
Slab 1
Continuous reinforcement

Slab 2
Curtailed reinforcement



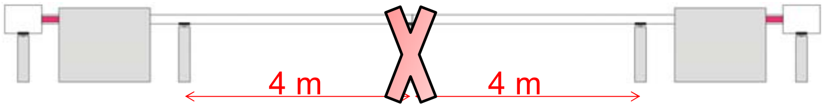
Loading procedure

- PHASE 1: Loading of the slab with service load up to 60 kN/jack and unloading



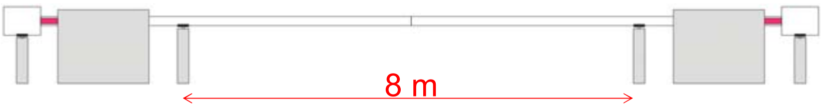
Loading procedure

- PHASE 1: Loading of the slab with service load up to 60 kN/jack and unloading
- PHASE 2: Removal of central support



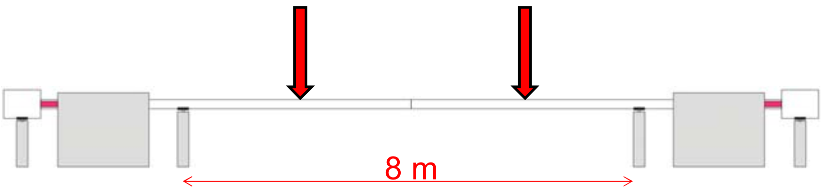
Loading procedure

- <u>PHASE 1:</u> Loading of the slab with service load up to 60 kN/jack and unloading
- <u>PHASE 2:</u> Removal of central support

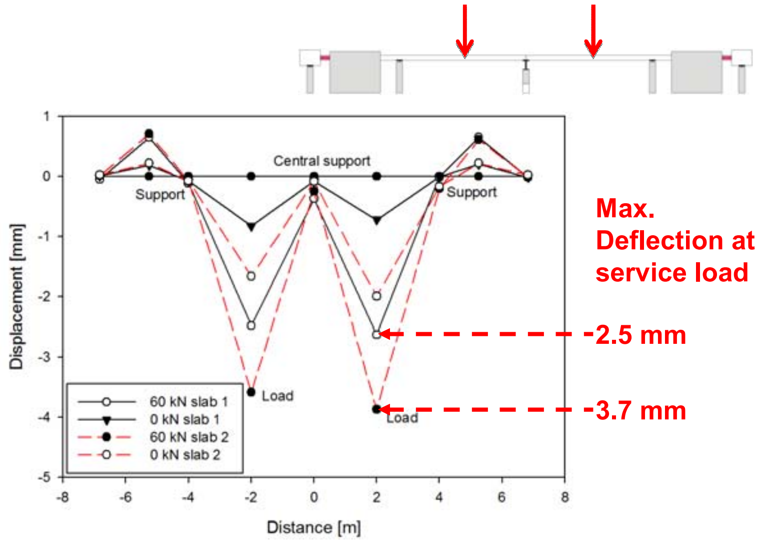


Loading procedure

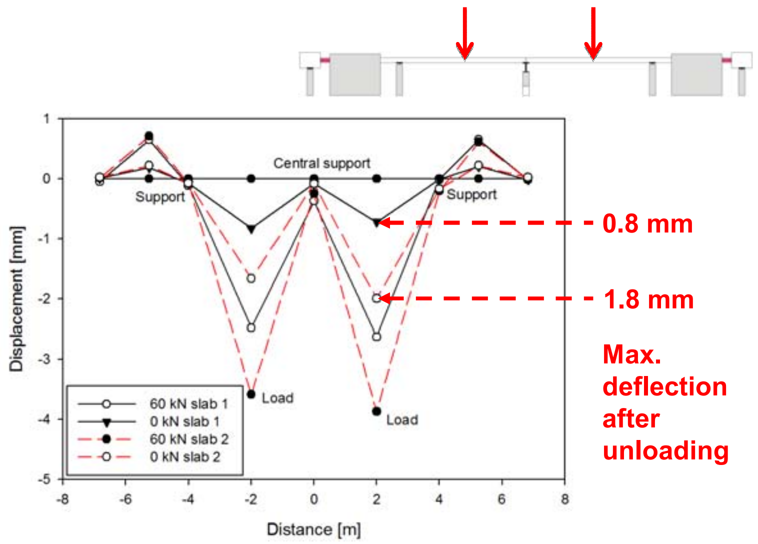
- <u>PHASE 1:</u> Loading of the slab with service load up to 60 kN/jack and unloading
- <u>PHASE 2:</u> Removal of central support
- <u>PHASE 3:</u> Displacement controlled loading to failure



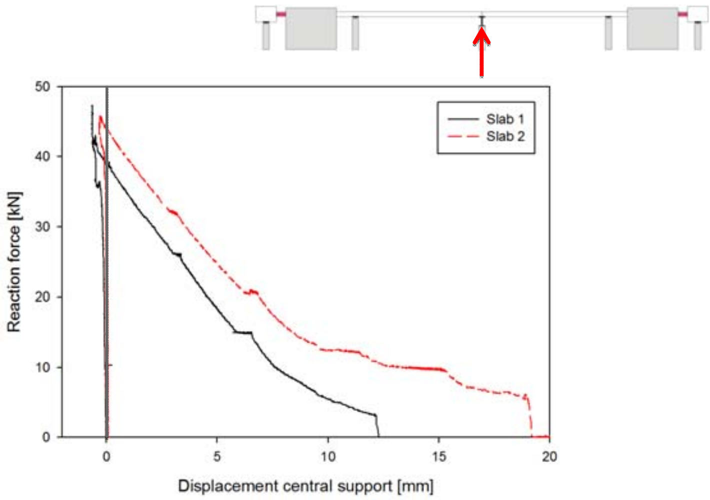
Results Phase 1



Results Phase 1



Results Phase 2a



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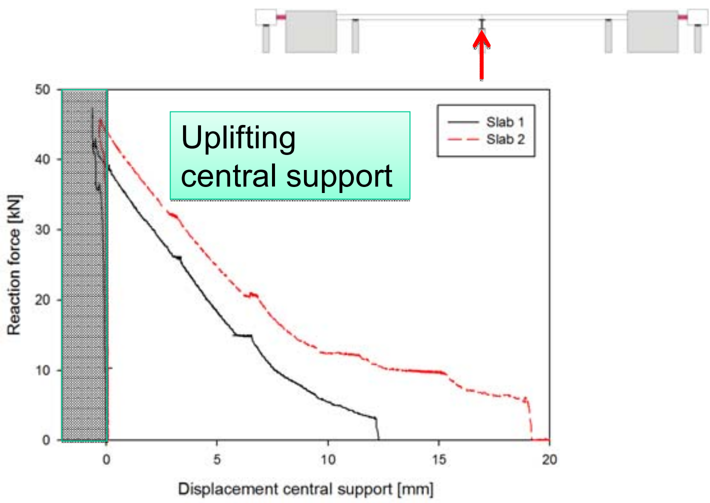
Results Phase 2b



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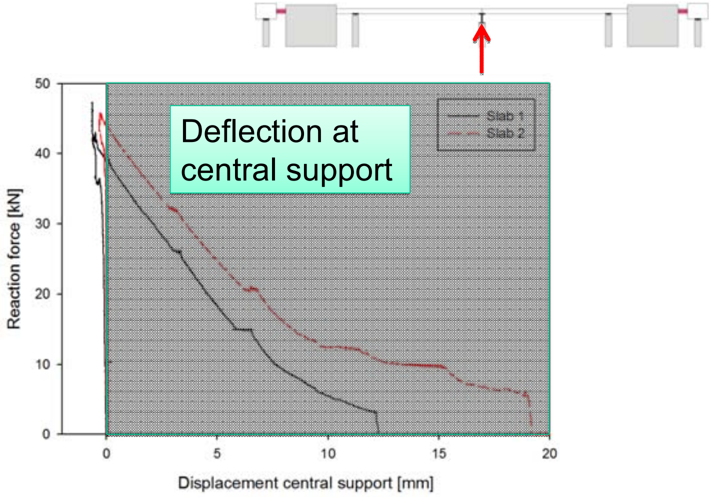
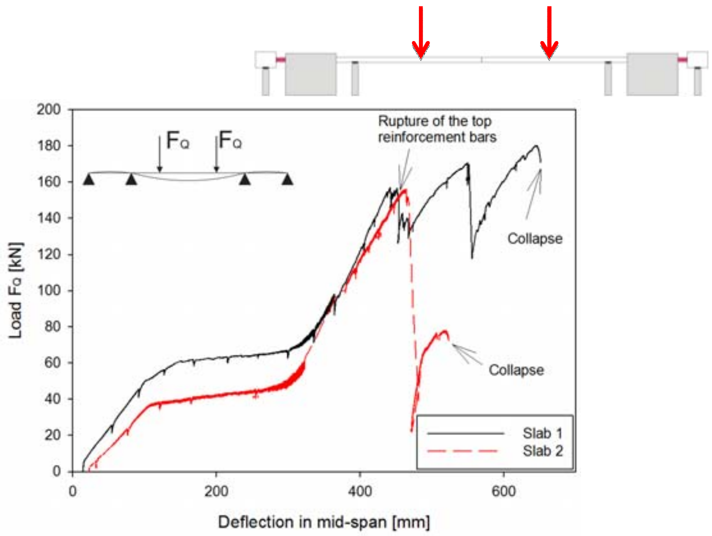
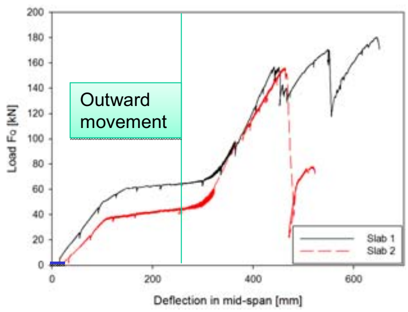
Results Phase 2c



Results Phase 3

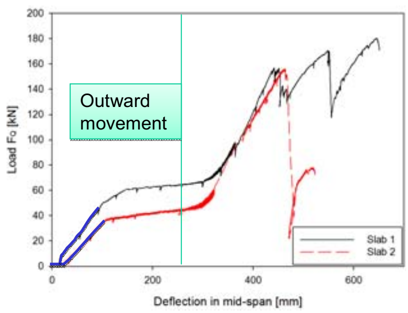


Results Phase 3



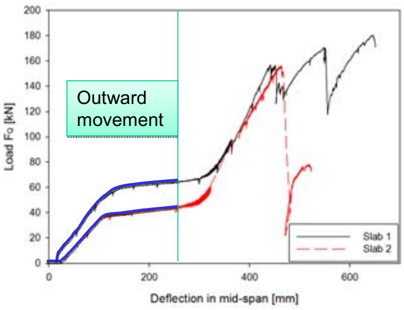
- Initial deflection due to Phase 1 & 2

Results Phase 3



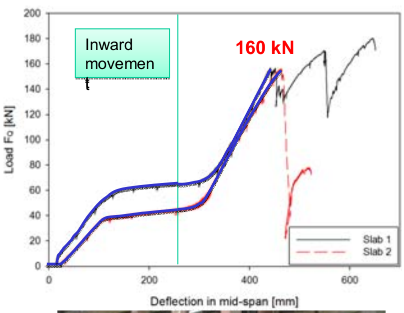
- Initial deflection
- Linear behaviour

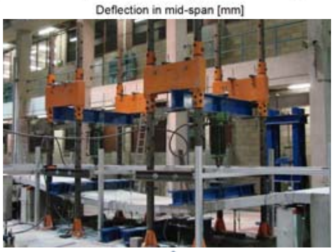
Results Phase 3



- Initial deflection
- Linear behaviour
- Nonlinear behaviour
  - Yielding of the top reinforcement bars
  - Development of plastic hinges over the supports

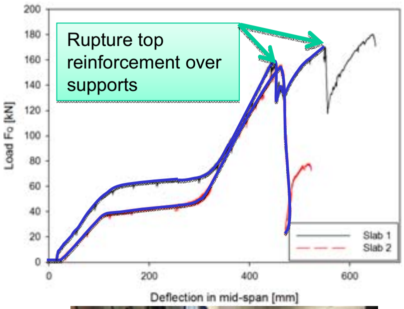
Results Phase 3

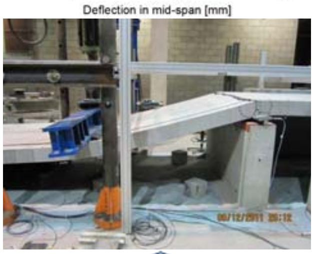




- Initial deflection
- Linear behaviour
- Nonlinear behaviour
- Increase of the load-displacement curve due to tensile membrane action

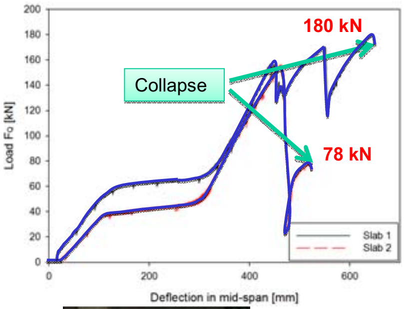
Results Phase 3





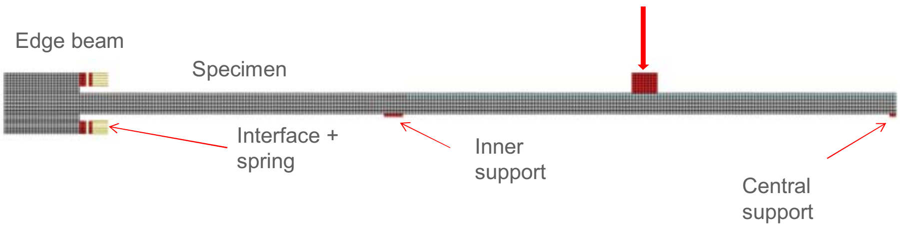
- Initial deflection
- Linear behaviour
- Nonlinear behaviour
- Increase of the load-displacement curve
- Rupture of the top reinforcement over supports

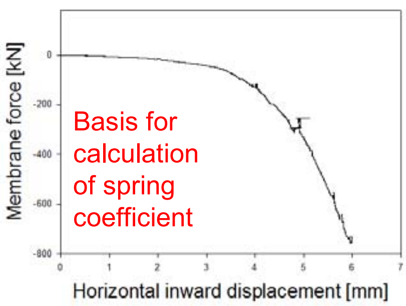
Results Phase 3



- Initial deflection
- Linear behaviour
- Nonlinear behaviour
- Increase of the load-displacement curve
- Rupture of the top reinforcement over supports
- Collapse of the specimens

FE-Modelling



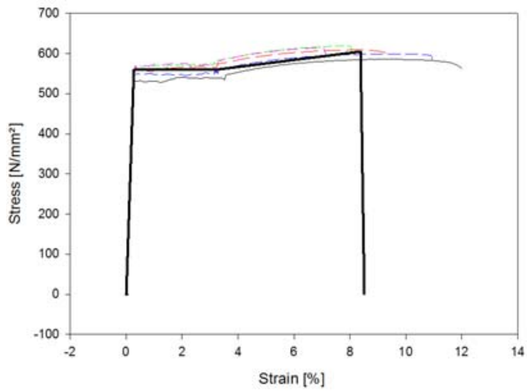


- **Phase 1**
  - Self weight
  - Loading until service load
  - Unloading
- **Phase 2**
  - Removal of central support
  - Self weight
  - Displacement contolled loading

Materials

Steel

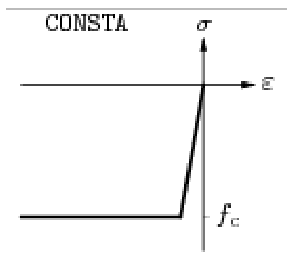
NAME "Truss"
YOUNG 2.05000E+005
POISON 3.00000E-001
YIELD VMISES
HARDEN STRAIN
HARDIA

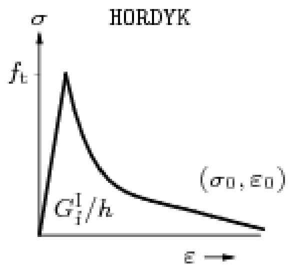


Materials

Concrete

NAME "Concrete"
YOUNG 3.00E+4
POISON 0.15
TOTCRK FIXED
TENCRV HORDYK
GF1 0.09
COMCRV CONSTA
COMSTR 35
TENSTR 2.8
BETA 0.01

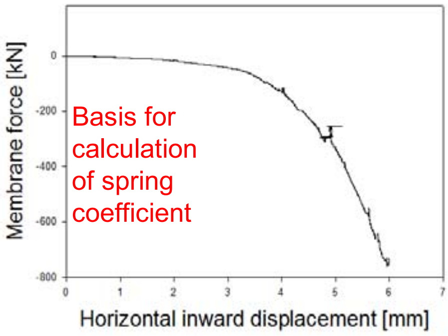




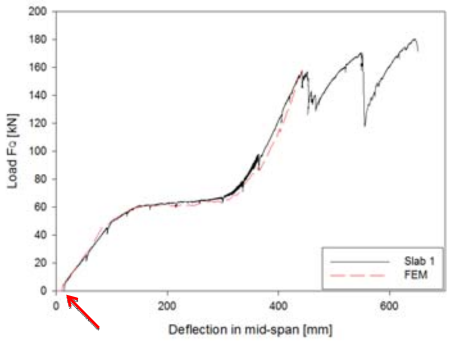
Materials

Spring

NAME "Spring"
SPRING 5757
FDUX



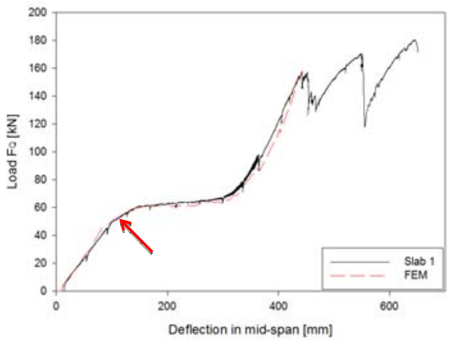
Results FEM, Slab 1



Displacement

Crack development

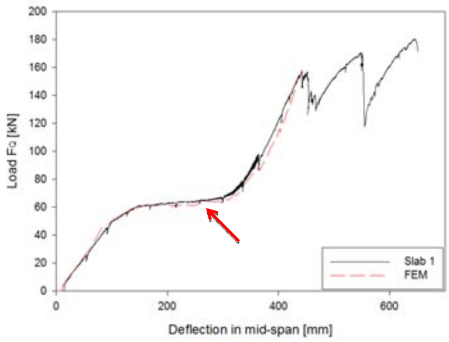
Results FEM, Slab 1



Displacement

Crack development

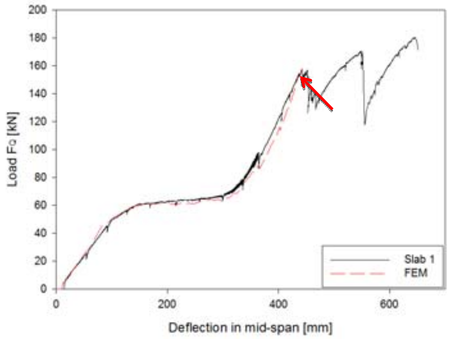
Results FEM, Slab 1



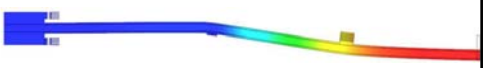
Displacement

Crack development

Results FEM, Slab 1

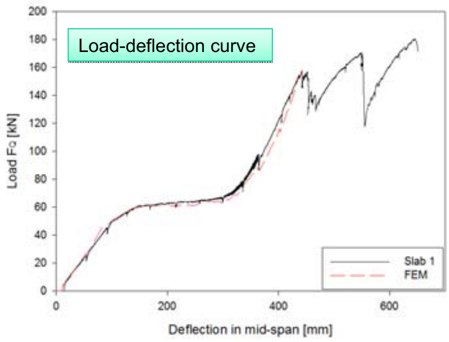


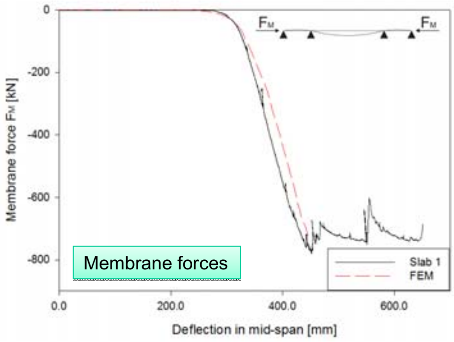
Displacement



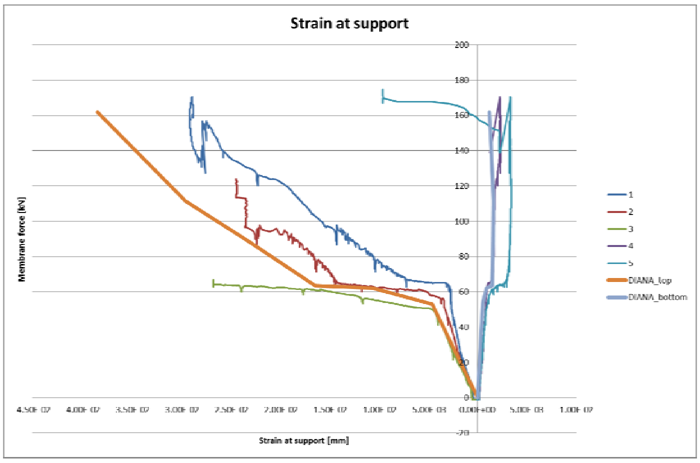
Crack development

Results FEM, Slab 1

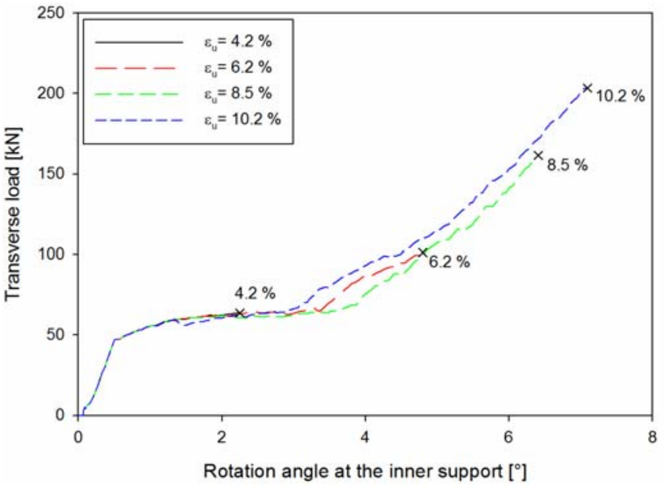




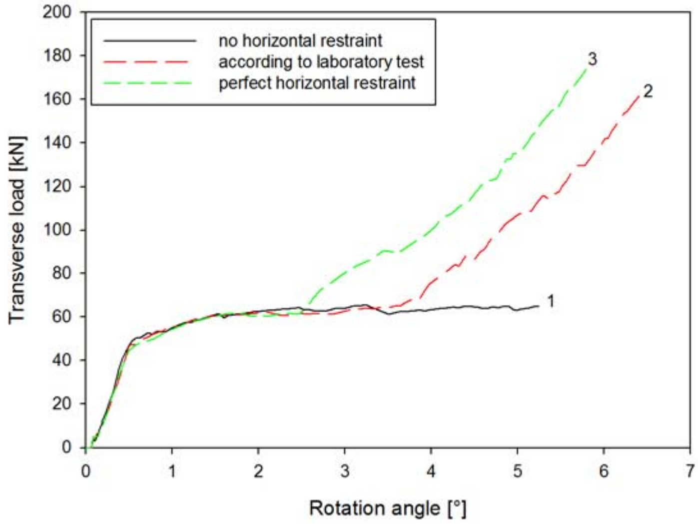
Strain over support, Slab 1



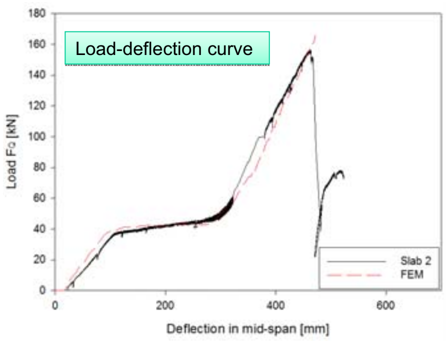
Results FEM, Slab 1

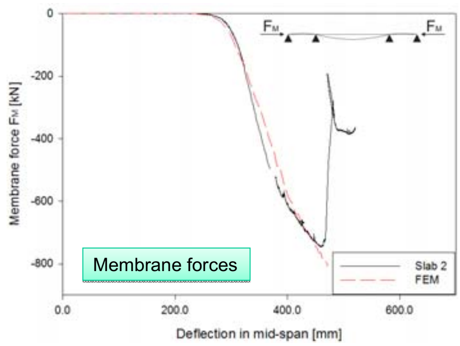


Results FEM, Slab 1

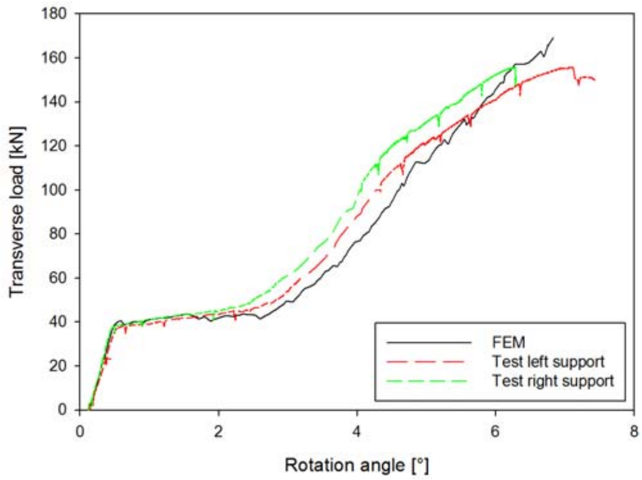


Results FEM, Slab 2





Results FEM, Slab 2

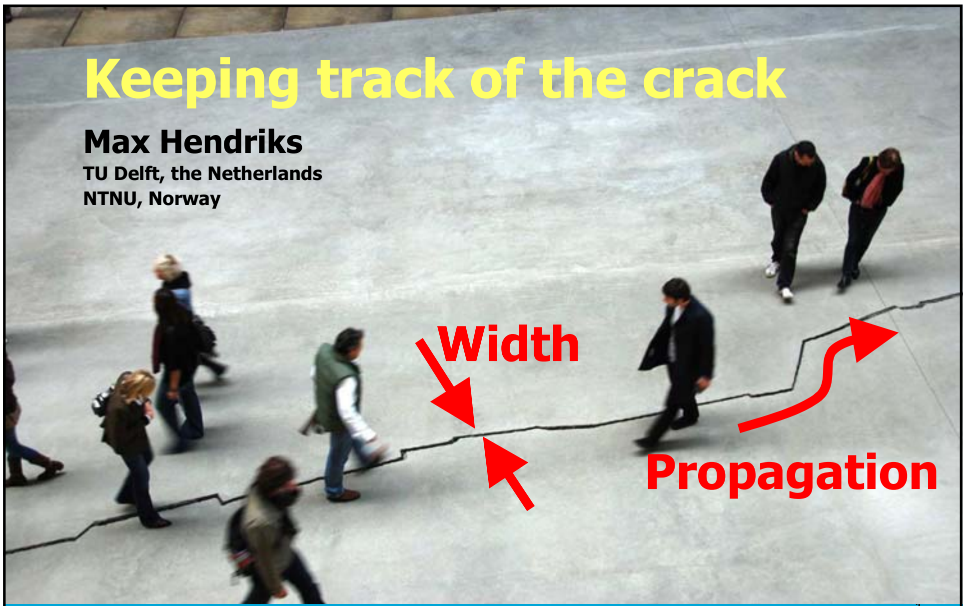


Conclusion

- Load transfer process from an **elastic bending mechanism** to a **catenary mechanism controlled by tension**
- Significant increase in the load carrying capacity due to **tensile membrane forces**
- An non-linear **DIANA-Model** was developed
- The load-displacement behaviour could be modeled taking the effect of **tensile membrane behaviour** into account

Keeping track of the crack

Max Hendriks
TU Delft, the Netherlands
NTNU, Norway



1

Presenting work of the ongoing PhD project of Arthur Slobbe at the TU Delft

TU Delft
Delft University of Technology

Topics

- Relevance of G_f / ℓ
  - Example: A beam failing in shear
- Crack width
- Crack propagation

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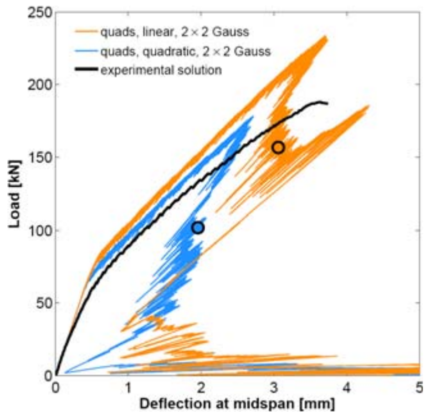
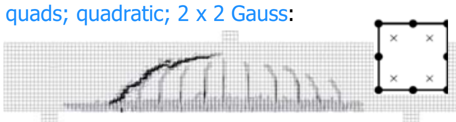
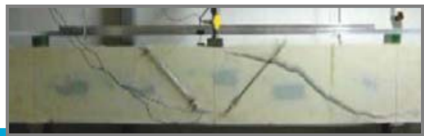
Beam failing in Shear



quads; linear; 2 x 2 Gauss:

quads; quadratic; 2 x 2 Gauss:





* **Sarkhosh, R., den Uijl, J.A., Braam, C.R., Walraven, J.C.** (2010)

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Beam failing in Shear

- (High) sensitivity for
  - Fracture energy G_f
  - Assumed crack bandwidth ℓ

$$(G_f / \ell)$$

  - Element type
  - Element alignment
  - Element integration scheme
  - …

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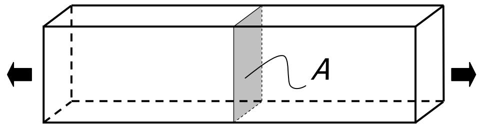
Fracture energy G_f

- *fib* Model Code 1990 versus *fib* Model Code 2010



- A = 1 m^2 and G_f = 100 J/m^2 gives 100 J
- ≈ a weight of 50 kg at a height of 20 cm.

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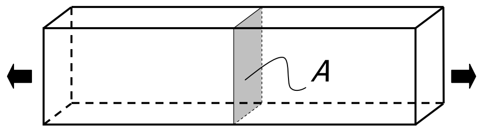
Fracture energy G_f

- *fib* Model Code 1990 versus *fib* Model Code 2010



- A = 1 m^2 and G_f = 100 J/m^2 gives   100 J
- ≈ a weight of 50 kg at a height of 20 cm.

- 1 can of beer = 340 ml = 150 kcal =
  150 x 4.186 x 1000 = 600,000 J

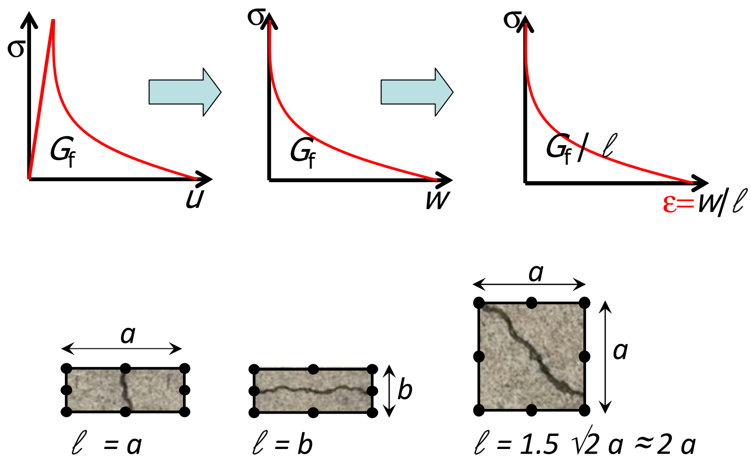
Conclusion for G_f

Although small in size G_f can play a very important role.

Especially for:

- Unreinforced concrete
- Failure in unreinforced regions

Smeared cracking: G_f / ℓ



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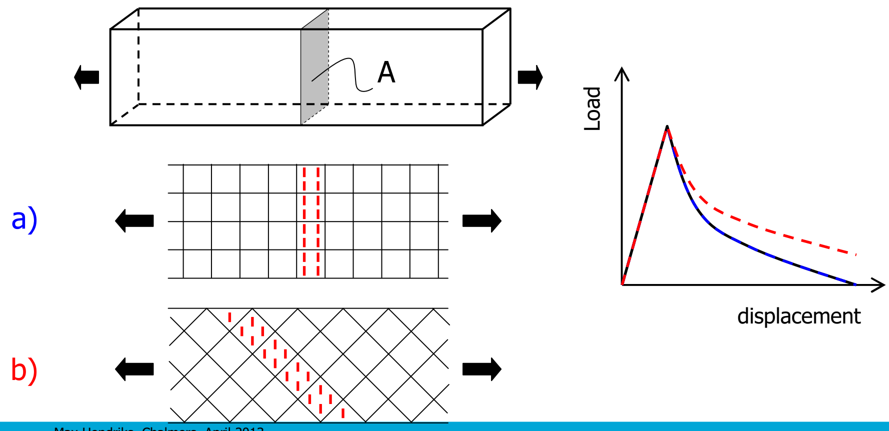
Motivation

Directional mesh bias



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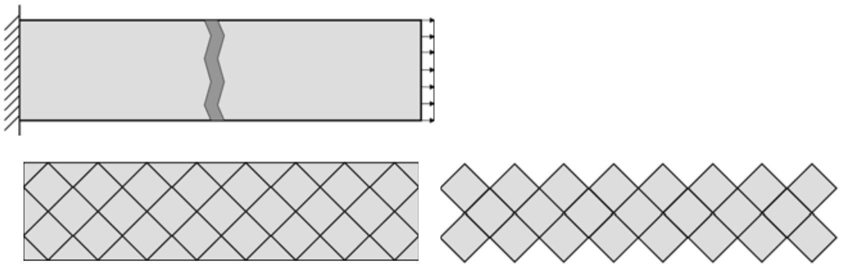
Numerical test with periodic boundary conditions

Requirements for testing procedure



We need a test that enables:
- to adopt different mesh alignments
- to study cracking or strain localization in uniform meshes
- to study cracking or strain localization without boundary disturbances

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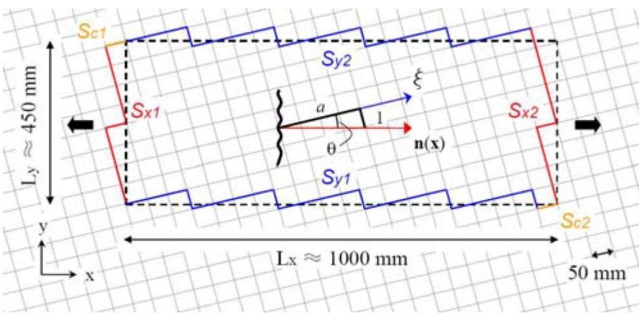
Numerical test with periodic boundary conditions

General concept



$$\boxed{d_i(S_{j1}) = d_i(S_{j2}) - \Delta_{ji}}$$

$$d_x(S_{x1}) = d_x(S_{x2}) - \Delta_{xx}$$
$$d_x(S_{y1}) = d_x(S_{y2}) - \Delta_{yx}$$
$$d_y(S_{x1}) = d_y(S_{x2}) - \Delta_{xy}$$
$$d_y(S_{y1}) = d_y(S_{y2}) - \Delta_{yy}$$

$$d_x(S_{c1}) = d_x(S_{c2}) - \Delta_{xx} + \Delta_{yx}$$
$$d_y(S_{c1}) = d_y(S_{c2}) + \Delta_{yy} - \Delta_{xy}$$

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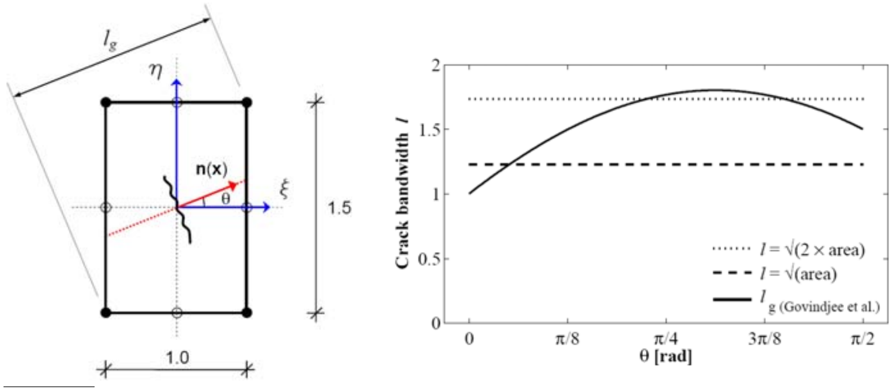
Systematic assessment crack band model

Crack bandwidth according to Govindjee et al.*



* **Govindjee, S., Kay, J.C., Simo, J.C.** (1995)
Anisotropic modeling and numerical simulation of brittle damage in concrete
** **Oliver, J.** (1989)
A consistent characteristic length for smeared cracking models

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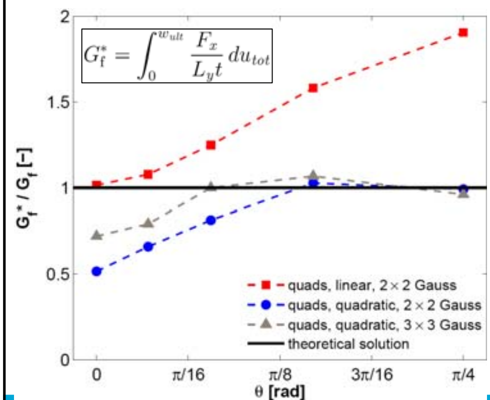
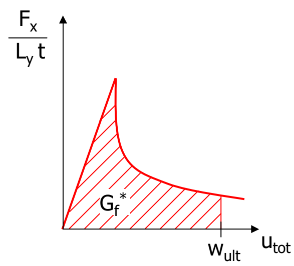
Systematic assessment crack band model

Test series – results with l_g





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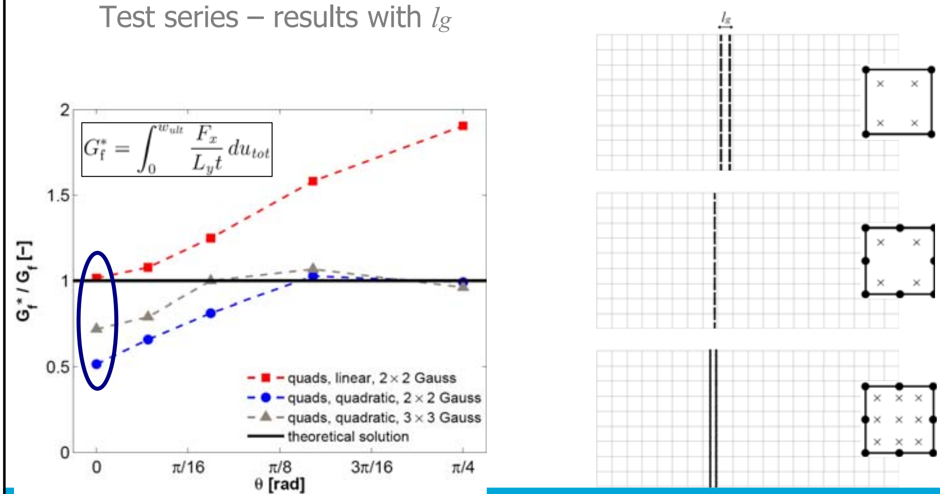
Systematic assessment crack band model

Test series – results with l_g



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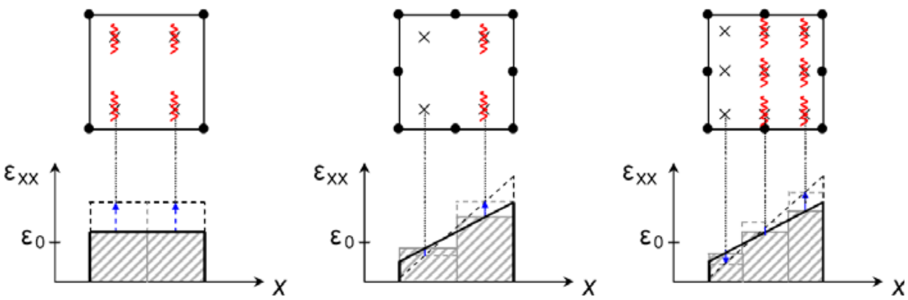
Systematic assessment crack band model

Test series – results with l_g



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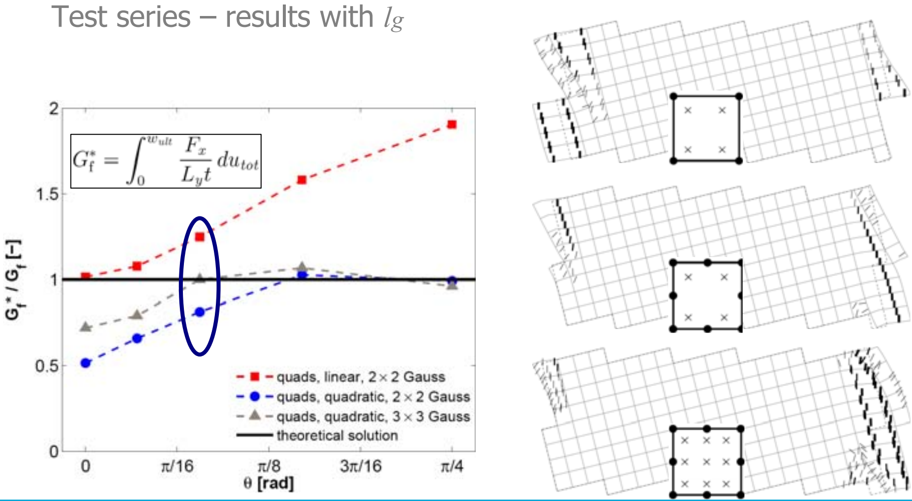
Systematic assessment crack band model

Test series – results with l_g



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New crack bandwidth formulation

Derivation l_p

$$l_p = \alpha \cdot \gamma \cdot l_g$$

$$\frac{G_f^*}{G_f} = \boxed{\alpha} \cdot \gamma$$

α - values:

Types	quadrilateral 2 x 2 Gauss	quadrilateral 3 x 3 Gauss
linear	1	-
quadratic	$1/2$	$13/18$

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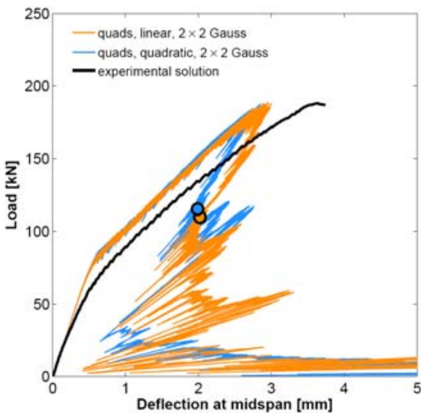
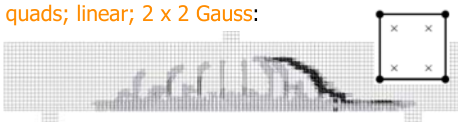
New crack bandwidth formulation

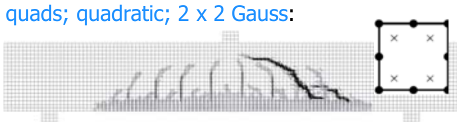
Validation on benchmark – results with l_p



quads; linear; 2 x 2 Gauss:



quads; quadratic; 2 x 2 Gauss:

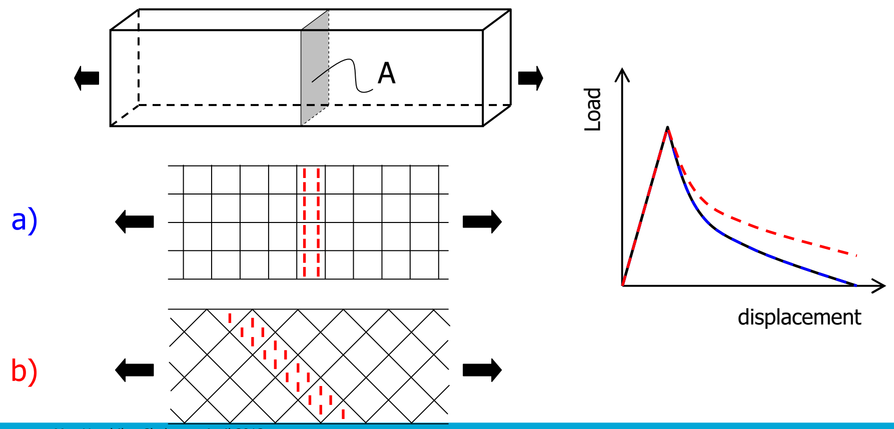


Conclusions for crack bandwidth formulation

- Numerical test for systematic assessment of directional mesh bias.
- New crack bandwidth formulation that takes into account:

  1. Linear versus quadratic elements
  2. Integration scheme
  3. Alignment factor

Mesh objectivity
Mesh-induced directional bias



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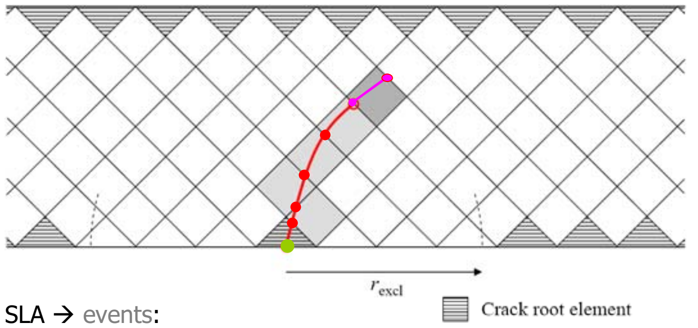
Crack-tracking technique
General concept



SLA → events:

1. Crack path initiation
2. Crack path localization
3. Crack path propagation

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(Local) Crack-tracking technique

Motivation

- Possible remedy for observed mesh-induced directional bias.
- Advanced models (E-FEM, X-FEM) use it, and even need it!

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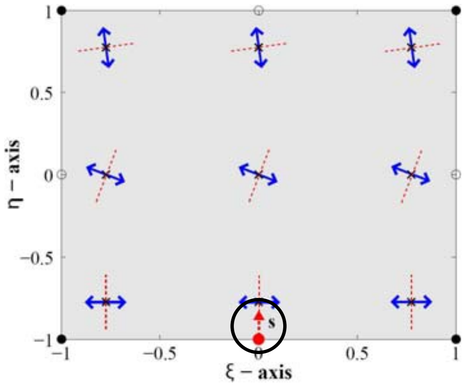
Proposed crack propagation algorithm

σ_1 - state in crack root element



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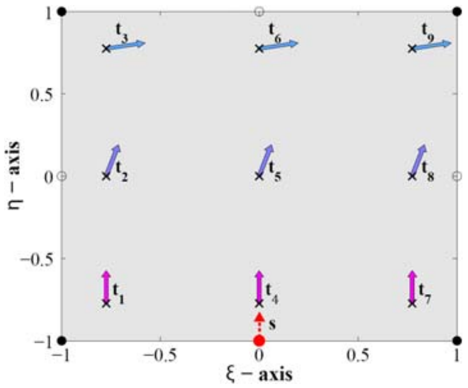
Proposed crack propagation algorithm

Local crack propagation vectors $\mathbf{t}_j$



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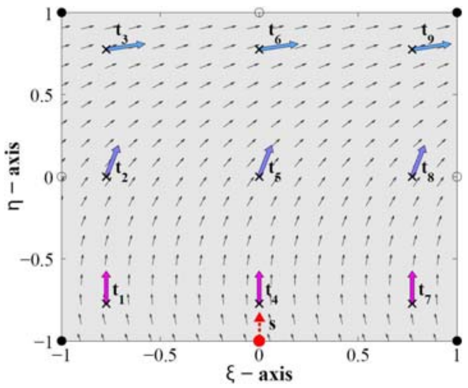
Proposed crack propagation algorithm

Crack propagation vector field $\mathbf{T}(\xi,\eta)$



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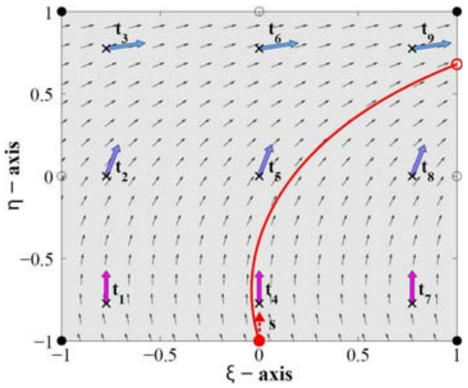
Proposed crack propagation algorithm

Crack path line through crack root element



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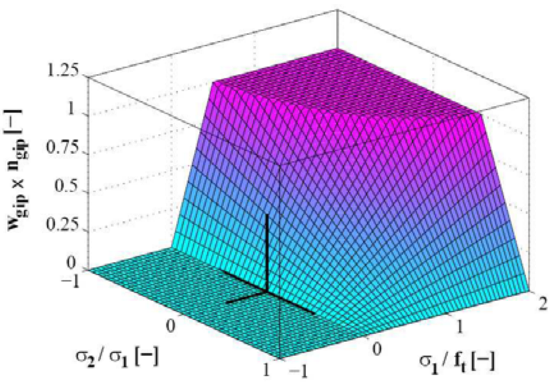
Proposed crack propagation algorithm

Weight factors in integration points



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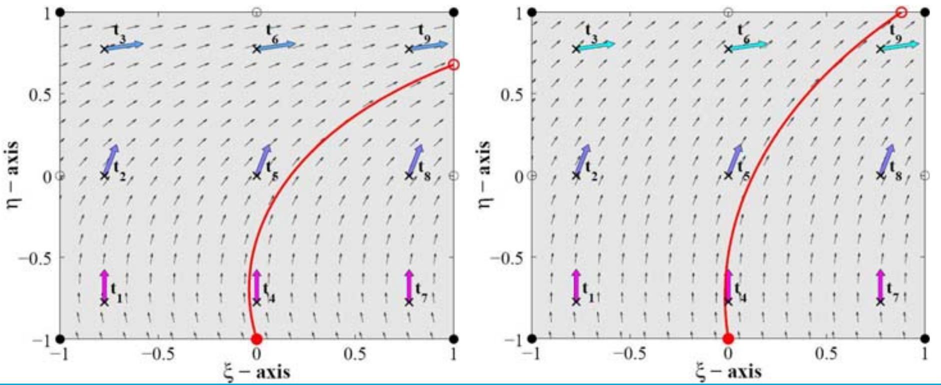
Proposed crack propagation algorithm
Influence principal stress state



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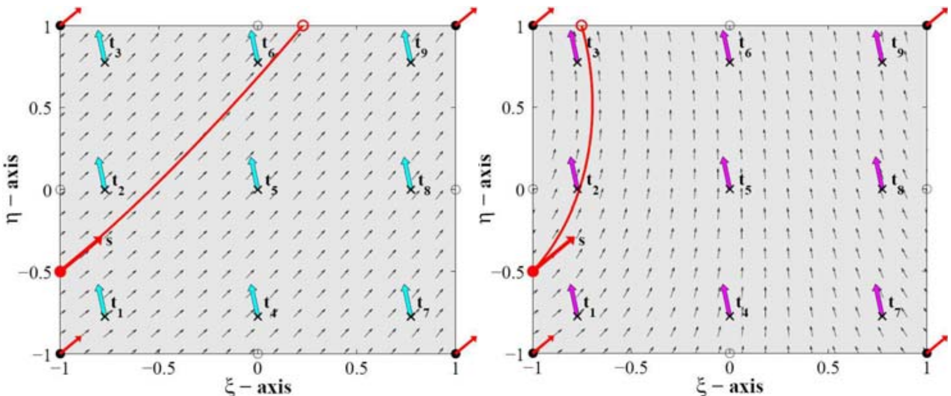
Proposed crack propagation algorithm
Crack path line through crack tip element



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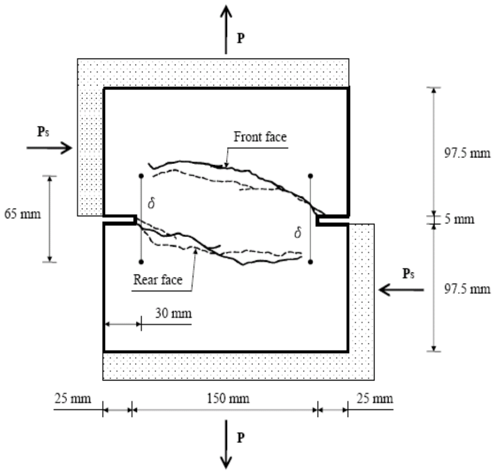
Validation CPA on fracture tests

Double-edge-notched specimen



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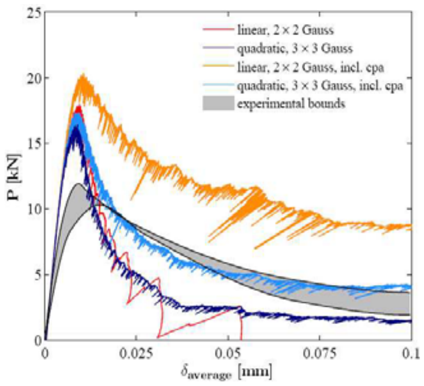
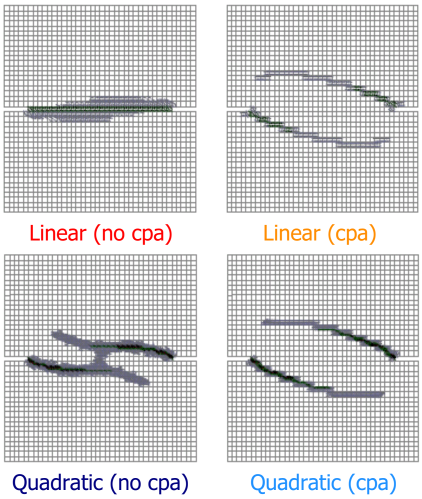
Validation CPA on fracture tests

Double-edge-notched specimen





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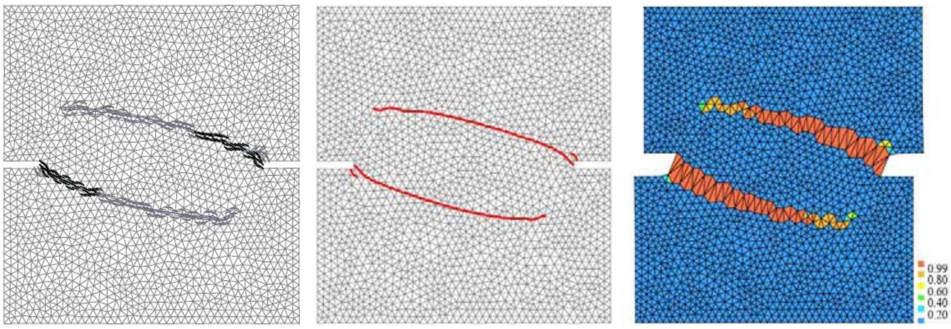
TU Delft

Validation CPA on fracture tests

Double-edge-notched specimen



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Conclusions for crack propagation algorithm

- can improve simulations of localized damage in plain concrete specimens;
- especially when higher order quadratic elements are used;
- is less capable to analyze failure in RC structures.

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tack

Max Hendriks, Chalmers, April 2013

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Anchorage of corroded bars

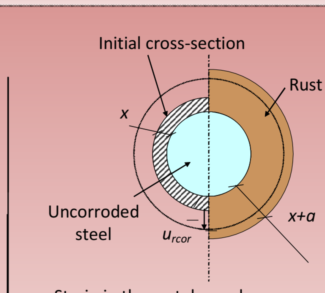
from Cover Cracking to Cover Spalling

Kamyab Zandi Hanjari
Chalmers University of Technology
CBI - Swedish Cement and Concrete Research Institute

DIANA Users Meeting 25-26 April 2013

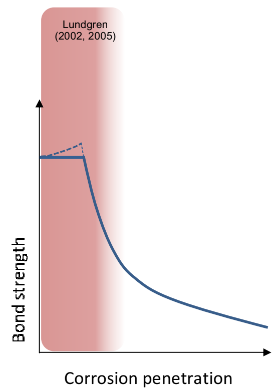
Development since 2002(1)

Lundgren (2002, 2005)



Strain in the rust depends on corrosion penetration x, expansion factor v_{rs}, and deformation u_{rcor}

$$y = -r + \sqrt{r^2 + (v_{rs} - 1)\cdot(2rx - x^2)}$$



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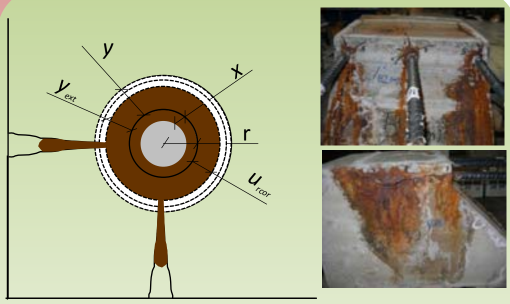
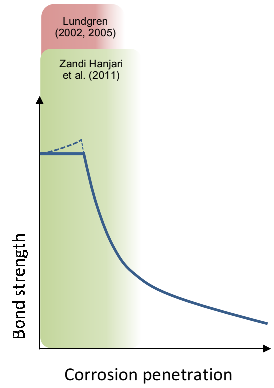
Development since 2002 (2)

Lundgren (2002, 2005)

Zandi Hanjari et al. (2011)



$$y_{ext} = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)(2rx - x^2)} - \frac{V}{\pi \cdot e}$$



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Development since 2002 (3)

Lundgren (2002, 2005)

Zandi Hanjari et al. (2011)

Zandi Hanjari (2013)

Diana (corrosion phase)
- Impose corrosion-time step
- Tabulate strains for all step

Script
- Calculated crack width, w_{cr}, after each step

if w_{cr} < 2mm

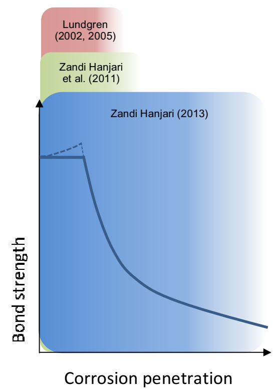
if w_{cr} > 2mm

Script
- Tabulate out strains & disp.
- Find the crack planes
- Find spalled cover
- Remove the spalled elements
- Set-up a new FE model
- Req. corrosion level reached?

No

Yes

Diana
- pull-out phase



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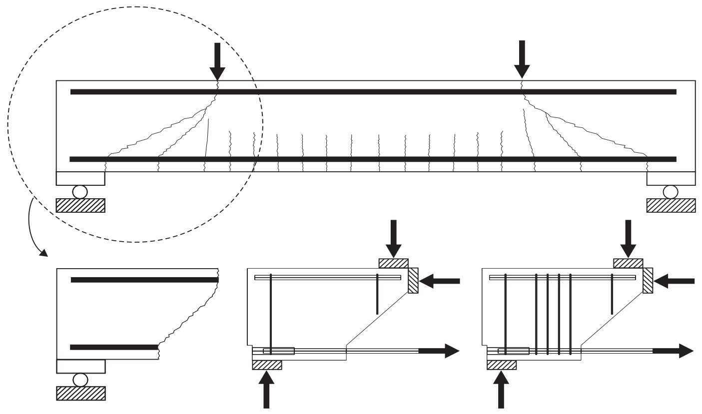
Eccentric pull-out specimens



Beam-end   Specimen type A   Specimen types B and C

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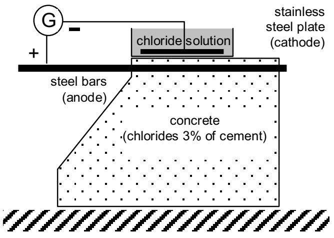
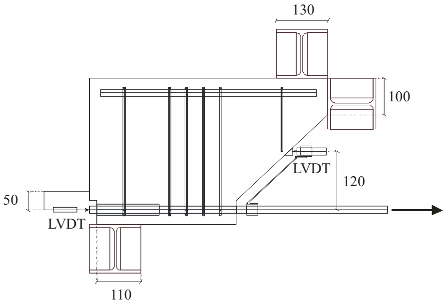
Eccentric pull-out tests



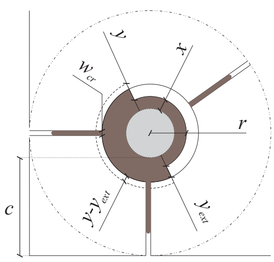
Artificial corrosion

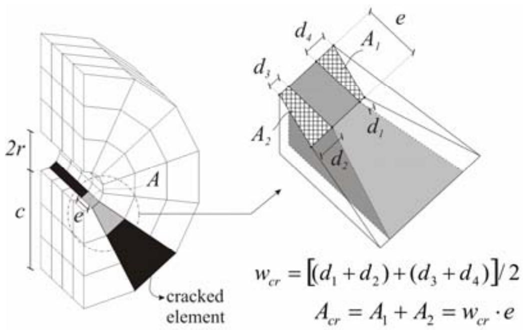


Pull-out test

Rust flow through cracks

Extension of corrosion model

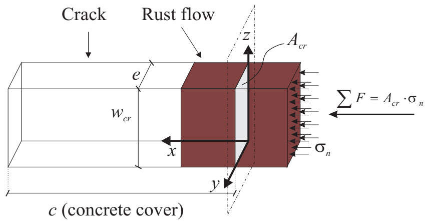


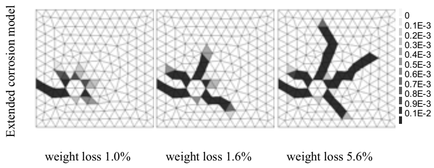


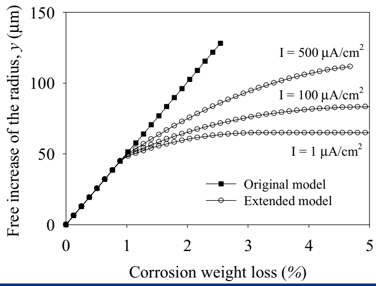
$$y_{ext} = -r + \sqrt{r^2 + (\upsilon_{rs} - 1)(2rx - x^2) - \frac{V}{\pi \cdot e}}$$

One-dimensional flow of rust

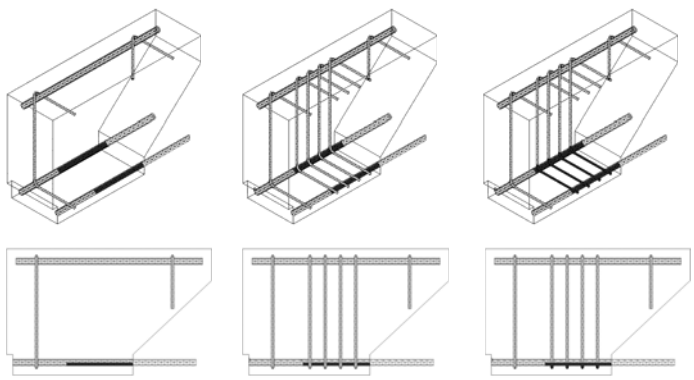
- One-dimensional plug flow model
- Constant velocity of corrosion products across the crack
- Lagrangian formulation
- Ideal flow

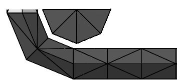






3D non-linear structural analysis





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