# BETONGKONSTRUKSJONERS LIVSLØP

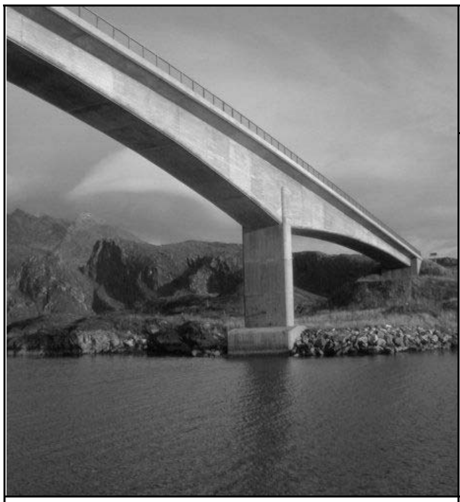
Et utviklingsprosjekt i samarbeid mellom offentlige byggherrer, industri og forskningsinstitutter

**Deltakere:**
Statens vegvesen (prosjektledelse), Forsvarsbygg, NORCEM A.S, Selmer Skanska AS, Sika Norge AS, Norges byggforskningsinstitutt, NTNU, SINTEF, NORUT Teknologi as

**Rapport nr. 12**

**Strengthening Prestressed Concrete Beams with Carbon Fiber Polymer Plates. Aktivitet DP2 B1**

Prosjektet er støttet av BA-programmet i Norges forskningsråd



# BETONGKONSTRUKSJONERS LIVSLØP

## Rapport nr. 12

## Strengthening Prestressed Concrete Beams with Carbon Fiber Polymer Plates

## Aktivitet DP2 B1

Utgiver:
Statens vegvesen, Vegdirektoratet
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INTERN RAPPORT NR.

| GRADERING | ANTALL SIDER OG BILAG |
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| 82-91228-17-5 | Desember 2006 | Finn Fluge og Bernt Jakobsen |

KORT SAMMENDRAG

Fire prefabrikerte, forspente T-bjelker er testet i laboratoriet ved NTNU. De fire T-bjelkene er fremstilt ved å dele to DT-elementer i to. DT-elementene kommer fra Isakveien bru som ble revet i 1999 etter 35 år i bruk.

To av bjelkene ble prøvet uten forsterkning for å bestemme referanse kapasiteter for moment- og skjærbrudd. Referansekapasitetene ble funnet å være henholdsvis 23% og 104% høyere enn beregnet i henhold til NS 3473:1998.

De andre bjelkene ble forsterket med langsgående karbonfiberplater (SIKA CarboDur S) med forskjellige arealer, den ene med $A_{CFRP}$ = 108 mm$^2$ og den andre med $A_{CFRP}$ = 216 mm$^2$. Dette ga en netto økning av momentkapasiteten på henholdsvis 28% og 37%.

Parallelt ble det utført elementanalyser av bjelkene. Ikke-lineær materialoppførsel ble modellert med en tøyningsbasert rissmodell med utgangspunkt i ”The Modified Compression Field Theory”. Med de valte inngangsverdier for fasthet til betong, karbonfiberplater og spennarmering underestimerte elementmodellene kapasiteten med opp til 11%.

Bruk av karbonfiber plater synes å være et enkelt og effektivt alternativ ved forsterkning av eksisterende betongkonstruksjoner.

| STIKKORD | NORSK | ENGLISH |
|---|---|---|
| | Betong | Concrete |
| | Karbonfiber plater | Carbon fiber plate |
| | Foroppspente element | Prestressed element |
| | Forsterkning | Strengthening |

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|---|---|---|
| **Rapport** | **Nr. 12** | Strengthening Prestressed Concrete Beams with Carbon Fiber Polymer Plates. |
| **Prosjekt** | | **Betongkonstruksjoners livsløp**<br>Et utviklingsprosjekt i samarbeid mellom offentlige byggherrer, industri og forskningsinstitutter. |
| **Aktivitet** | **DP2 B1** | Vedlikeholds- og oppgraderingsmetoder<br>Oppgraderingsmetoder<br>Prøving av karbonfiber-forsterkede bjelker, Isakveien bru. |
| **Deltagere** | | Statens vegvesen (prosjektledelse),<br>Forsvarsbygg,<br>NORCEM A.S,<br>Selmer-Skanska AS<br>Sika Norge AS<br>Norges byggforskningsinstitutt,<br>NTNU,<br>SINTEF,<br>NORUT Teknologi as |

Prosjektet er støttet av BA-programmet i
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**Emneord**
Betong
Karbonfiber plater
Føroppspente element
Forsterkning

**Key words**
Concrete
Carbon fiber plate
Prestressed element
Strengthening

**FORORD**

Fokus er i løpet av de senere årene flyttet fra bygging av nye konstruksjoner over mot forvaltning hvor det legges større vekt på problemstillinger knyttet til drift, vedlikehold og gjenbruk av eksisterende konstruksjoner.

Prosjektet "Betongkonstruksjoners livsløp" er knyttet opp mot denne typen utfordringer som en samlet bygg- og anleggsbransje står overfor. Kravene til bygg- og anleggskonstruksjoner er at de skal være funksjonelle og kostnadseffektive. Offentlige byggherrer forvalter og vedlikeholder et stort antall konstruksjoner som skal møte samfunnets krav til:

- sikkerhet
- kvalitet/økonomi
- miljø

Det ble de siste årene av 90-tallet lagt ned et betydelig arbeid i prosjektet "Bestandige betong-konstruksjoner". Av resultatene fra dette prosjektet og erfaringene fra prosjektet ”OFU Gimsøystraumen” fremgår det klart at beslutningen om å bygge bestandige betongkonstruksjoner må tas tidlig i planleggingsfasen og at det er behov for enkelt å kunne verifisere prosjekteringsforutsetningene.

”Betongkonstruksjoners livsløp” bygger videre på forannevnte prosjekter. Hovedvekten er lagt på klart formulerte forskningsoppgaver som dels konkretiserer eksisterende kunnskap og dels fyller hull i kunnskapsgrunnlaget. Aktivitetene er valgt innenfor en ramme som omfatter alle faser fra planlegging til riving og gjenbruk.

Prosjektets hovedmålsetning har vært:

**Kostnadseffektive og miljøgunstige betongkonstruksjoner**

med følgende delmål:

- Identifisere hovedparametre i levetidsmodellene og kalibrere dem mot felterfaringer
- System for vurdering av vedlikeholdstiltaks levetid
- System for instrumentell overvåkning av betongkonstruksjoners tilstandsutvikling
- Kunnskapsformidling gjennom normarbeid, kurs og internasjonale nettverk

Prosjektets sluttprodukter er:

- Grunnlag for veiledninger og regler for levetidsprosjektering
- Akseptkriterier for bedømmelse av betongkonstruksjoners bestandighet
- Datagrunnlag til bruk i standardiseringsarbeid og som inngangsdata til europeisk nettverksarbeid
- Kunnskap og kompetanse knyttet til sensorteknologi, måleteknikk, "intelligent" instrumentell overvåkning, katodisk beskyttelse etc., hvor industripartnerne gis mulighet til å utnytte resultatene kommersielt

Prosjektet har bestått av flere større og mindre aktiviteter gruppert i følgende delprosjekter:

- DP1. Levetidsprosjektering
  - A. Datainnsamling
  - B. Levetidsmodeller
- DP2. Vedlikeholds- og oppgraderingsmetoder
  - A. Vedlikeholdsmetoder
  - B. Oppgraderingsmetoder
  - C. Rustfri armering
- DP3. Måleteknikk

Aktivitetene i prosjektet er basert på enkeltforslag fra prosjektdeltakerne. Hvor aktivitetene hadde fellestrekk, kunne levere resultater til, eller benytte resultater fra andre aktiviteter ble dette identifisert ved oppstarten av prosjektet og nødvendig koordinering foretatt. Ellers er aktivitetene styrt meget selvstendig.

Prosjektet startet høsten 1999 og ble avsluttet høsten 2001. Prosjektet har vært støttet av BA-programmet i Norges forskningsråd med NOK 1 mill i hvert av årene 1999 og 2000.

I tillegg til støtten fra Norges forskningsråd har det vært ytet en betydelig egeninnsats fra deltakerne i form av personalinnsats og kjøp av FoU-tjenester. Prosjektkostnadene per 31-12-00 var NOK 7,25 mill, hvorav NOK 2,7 mill var benyttet til kjøp av FoU-tjenester fra forskningsinstitutter og NOK 0,5 mill fra konsulent. I år 2001 ble det kjøpt tjenester for NOK 1,7 mill som i sin helhet ble finansiert av prosjektdeltagerne. Samlede prosjektkostnader ved avslutningen av prosjektet er ca. NOK 9 mill.

Prosjektet har hatt følgende deltakere:

Statens vegvesen
Forsvarsbygg
NORCEM A.S
Selmer Skanska AS
NTNU
SINTEF
Sika Norge AS
Norges byggforskningsinstitutt
NORUT Teknologi as

I tillegg har prosjektet samarbeidet med Det Norske Veritas og ARMINOX, som alle har bidratt med egeninnsats.

Det er knyttet to dr. gradsstudenter til prosjektet.
Prosjektet mottok i juni 2000 et 3 års dr.grad stipendium. Stipendiat ble tilsatt 01-01-2001.

Prosjektet har vært ledet av Vegdirektoratet. Prosjektledelsen, som har bestått av Finn Fluge Vegteknisk avdeling, Vegdirektoratet og Bernt Jakobsen, Aadnesen a.s, har rapportert til en styringskomite som har bestått av representanter fra prosjektdeltakerne. Styringskomiteen har vært samlet to ganger årlig eller ved behov og har fastlagt mål og hovedstrategier.

## SUMMARY


The report describes laboratory tests and finite element analyses carried out on reinforced concrete beams strengthened with carbon fibre plates. Four prefabricated T-beams with 100 mm concrete pavement have been tested in the laboratory of the Department of Structural Engineering at the Norwegian University of Science and Technology. The beams were manufactured by cutting longitudinally two DT-elements taken from the Isakveien bridge, which was demolished after 35 years in service. Apart from some minor local crushing at the beam ends, the elements were in very good condition. Testing of the concrete strength on drilled cores gave mean cylinder strengths of 76.5 and 55.2 MPa for the structural concrete and the pavement layer, respectively. Corresponding strengths specified for the project were B600 (C55) and B300 (C25). The coring showed good bond in the joint.

The height of the T-beams including the pavement was H = 680 mm. The width and thickness of the beam flanges were B= 600 mm and t = 100 mm, respectively whereas the thickness of the web varied between 90 and 150 mm.

Forty tendon wires $\phi 4$ St 1600/1800 with a concrete cover of 30 mm were distributed over a height of 130 mm in the lower part of the web. In addition, 14 $\phi 4$ tendon wires with 30 mm concrete cover were installed in the lower part of the flange. The total prestressing force in one beam was $P_i$ = 706 kN. The shear reinforcement was $\phi 8$ mm c/c 300 mm.

Two simply supported beams were tested to determine the reference capacities with respect to bending moment and shear failure.  These reference capacities were 23% and 104% higher than those calculated according to the Norwegian design standard, NS 3473. For the beam subjected to bending moment failure, final failure with crushing of concrete did not appear. This was due to the geometry of the testing rig. The load-deflection curve shows however that the ultimate capacity of the beam was very nearly reached. For the other beam the shear failure occurred after onset of extensive diagonal cracking and large strains in the tensile reinforcement.

The two other beams were strengthened by longitudinal carbon fibre plates ( SIKA CarboDur S) with two different cross-sections, $A_{CFRP}$ = 108 mm$^2$ and $A_{CFRP}$ = 216 mm$^2$. The thickness of the carbon fibre plates was t = 1,2 mm. The mean tensile strength of the carbon fibres is described as 3050 N/mm$^2$, while the modulus of elasticity is 165000 N/mm$^2$. The carbon fibres were glued to the tension side of the beams. For both test set-ups the loading was arranged in order that bending moment failure should take place.

The increase of the capacities of the strengthened beams were, compared with the reference capacity 28% and 37 %, respectively. For the first beam the failure took place as rupture of the carbon fibre plate at the underside of the beam at a tension strain of about 10‰. The failure mode of the second beam was a typical shear failure. The load displacement curve approached a horizontal asymptote, indicating that also the maximum moment capacity was almost reached. This failure load is not directly comparable with the reference shear capacity due to different distances between the load and the supports.

The strengthening implied that the load level at the onset of the first cracking was increased by some 20%. Increased stiffness at the service load level (after cracking) was also significant; about 75% increase of the bending stiffness as determined by measured force-deflection and moment-strain relations. This increase is partly due to increased area of the tensile reinforcement, and partly due to the fact that the carbon fibre decreased the crack distances and thereby increased the “tension stiffening“ effect.

The beams have also been analysed by the FEM data program Diana. Non-linear material behaviour was represented by use of a crack model based on the “modified compression field theory”. The analyses were run with mean values for the strength of the concrete and the carbon fibres. For the steel reinforcement, characteristic strength was used as input values. The analyses underestimated the load carrying capacities of the beams by some 11%. If the strength of the prestressed reinforcement was increased by 10%, very good agreement between the test results and the results from the analyses was obtained.

Based on this investigation it seems that the use of carbon fibre plates may be a simple and effective method to strengthen existing concrete structures.

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| **Rapport nr.1:** | TITTEL: | Feltdata for kloridinitiert armeringskorrosjon. Sammenstilling og kvalitetsvurdering av tilgjengelige data. |
| | Aktivitet: | DP1 A1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Intern rapport nr. 2197. |
| | Forfattere: | Skjølsvold, O., Jacobsen, S., Lahus, O., Lindgård, J., Hynne, T. |
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| | ISBN | 82-91228-04-3 |
| | Sider: | 12+9+7 vedlegg + CD-ROM |
| | Dato: | Desember 2002 |

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| **Rapport nr. 2:** | TITTEL: | Laboratoriedata for kloridinitiert armeringskorrosjon. |
| | Aktivitet: | DP1 A1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A00732. |
| | Forfattere: | Hynne, T. og Lindgård, J. |
| | ISSN | 1502-2331 |
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| | Dato: | Januar 2003 |

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| **Rapport nr. 3:** | TITTEL: | Gimsøystraumen bru. Spesialinspeksjon 1992- kloridprofiler. Vurdering av kloridbelastning og diffusjonskoeffisient |
| | Aktivitet: | DP1 A1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Intern rapport nr. 2196. |
| | Forfattere: | Skjølsvold, O. |
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| **Rapport nr. 4:** | TITTEL: | Kloridinntrengning i ressursvennlig kvalitetsbetong. |
| | Aktivitet: | DP1 A2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORCEM rapport |
| | Forfattere: | Kjellsen, K.O. og Skjølsvold, O. |
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| **Rapport nr. 5:** | TITTEL: | Statistisk beregning av levetid for betongkonstruksjoner utsatt for kloridinntrengning. |
| | Aktivitet: | DP1 B1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01613. |
| | Forfattere: | Hynne, T., Leira, B.J., Carlsen, J.E. og Lahus, O. |
| | ISSN | 1502-2331 |
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| **Rapport nr. 6:** | TITTEL: | Dimensjoneringsformat for kloridbestandighet. |
| | Aktivitet: | DP1 B1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport STF22 A02601. |
| | Forfattere: | Leira, B.J. |
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| **Rapport nr. 7:** | TITTEL: | Pålitelighetsmetodikk ved bruk av FDV og levetidsberegninger. |
| | Aktivitet: | DP1 B2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Aas-Jakobsen. Rapp 6943-01. |
| | Forfattere: | Larsen, R.M. |
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| **Rapport nr. 8:** | TITTEL: | Effekt av reparasjon på levetid: Eksempelstudie fra Gimsøystraumen. |
| | Aktivitet: | DP1 B3 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01607. |
| | Forfattere: | Hynne, T. og Leira, B.J. |
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| **Rapport nr. 9:** | TITTEL: | Bestandighet og levetid av reparerte betongkonstruksjoner. |
| | Aktivitet: | DP2 A2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORUT Teknologi as rapport NTAS F2001-36. |
| | Forfattere: | Arntsen, B. |
| | ISSN | 1502-2331 |
| | ISBN | 82-91228-14-0 |
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| | Dato: | Oktober 2006 |

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| **Rapport nr. 10:** | TITTEL: | Restlevetid – Kai Sjursøya. |
| | Aktivitet: | DP2 A3 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Selmer Skanska AS, rapport nr. B 01-01. |
| | Forfattere: | Carlsen, J.E. |
| | ISSN | 1502-2331 |
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| **Rapport nr. 11:** | TITTEL: | Feltforsøk Sykkylven bru. |
| | Aktivitet: | DP2 A4 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Selmer Skanska AS, rapport nr. B 01-02 |
| | Forfattere: | Carlsen, J.E. |
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| **Rapport nr. 12:** | TITTEL: | Strengthening Prestressed Concrete Beams with Carbon Fiber Polymer Plates. |
| | Aktivitet: | DP2 B1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NTNU, Institutt for konstruksjonsteknikk. |
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| **Rapport nr. 13:** | TITTEL: | Forsterking av betongsøyler med karbonfibervev. |
| | Aktivitet: | DP2 B2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A00718. |
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| **Rapport nr. 14:** | TITTEL: | Forankringskapasitet av CFAP-bånd limt til betong. |
| | Aktivitet: | DP2 B2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01618. |
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| **Rapport nr. 15:** | TITTEL: | Nonlinear Finite Element Analysis of Deteriorated and Repaired RC Beams |
| | Aktivitet: | DP2 B3 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORUT Teknologi as rapport NTAS F2001-31. |
| | Forfattere: | Sand, B. |
| | ISSN | 1502-2331 |
| | ISBN | 82-91228-20-5 |
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| **Rapport nr. 16:** | TITTEL: | Styrkeberegning ved korrosjonsskader. |
| | Aktivitet: | DP2 B3 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01619. |
| | Forfattere: | Stemland, H. |
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| **Rapport nr. 17:** | TITTEL: | Korrosjonsegenskaper for rustfri armering. |
| | Aktivitet: | DP2 C1 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling.<br>NTNU, Institutt for konstruksjonsteknikk.<br>Rapport R-9-01. |
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| **Rapport nr. 18:** | TITTEL: | Heftforhold for rustfritt armeringsstål. |
| | Aktivitet: | DP2 C2 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling.<br>NTNU rapport. |
| | Forfattere: | Hofsøy, A., Sørensen, S.I. og Markeset, G. |
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| **Rapport nr. 19:** | TITTEL: | Service Life Design of Concrete Structures |
| | Aktivitet: | DP1 B4 |
| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. |
| | Forfattere: | Helland, S., Maage, M., Smeplass, S., Fluge, F. |
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| **Rapport nr. 20:** | TITTEL: | SLUTTRAPPORT |
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| | Utgiver: | Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. |
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**INNHOLDSFORTEGNELSE**

# 1. SAMMENDRAG


Karbonfiberplater kan være et attraktivt alternativ ved forsterkning av eksisterende betongkonstruksjoner. For å få økt innsikt i kapasitetsegenskapene til bærende elementer forsterket med karbonfiber komponenter, er det gjennomført forsøk og numeriske simuleringer for å klarlegge hvilken effekt en slik forsterkningen har.

Det ble foretatt laboratorieforsøk og elementmetodeanalyser av føroppspente bjelker forsterket med karbonfiberplater. Fire prefabrikkerte T-bjelker med 100 mm påstøp ble testet i laboratoriet ved Institutt for konstruksjonsteknikk, NTNU. Bjelkene var halve DT-elementer fra Isakveien bru ved Oslo som ble revet etter 35 års bruk. Bortsett fra litt knusning ved endeoppleggene var elementene i meget god stand. Fasthetsprøving på utborete kjerner ga midlere sylinderfastheter på 76,5 og 55,2 MPa for henholdsvis konstruksjonsbetongen og påstøpen. Tilsvarende var prosjektert betongkvalitet B600 (C55) og B300 (C25). Utboringen viste at det var god heft i støpefugen.

T-bjelkens totale høyde inkludert påstøp var H = 680 mm. Bredde av flensen var B = 600 mm, flensens tykkelse t = 100 mm, og stegets tykkelse varierte fra 90 til 150 mm.
Førti spenntråder Ø4 St1600/1800 med overdekning 30 mm var fordelt over en høyde på 130 mm i nederste del av steget. I tillegg var det lagt inn 14 Ø4 spenntråder med 30 mm overdekning i underkant av flensen. Total spennkraft i en bjelke var $P_i$ = 706 kN. Skjærarmeringen var Ø8 c/c 300 mm.

To av bjelkene ble prøvet fritt opplagt med tvillinglaster for bestemmelse av referansekapasiteter for moment- og skjærbrudd. Referansekapasiteten var 23% og 104 % høyere enn beregnet etter NS 3473. For bjelken med planlagt momentbrudd inntraff ikke endelig brudd ved knusning i betongtrykksonen. Dette skyldes prøveriggens geometri. Imidlertid viser last/forskyvnings-diagrammet at maksimalt lastnivå ble tilnærmet nådd. For den andre bjelken inntraff skjærbrudd etter utstrakt skrårissdannelse og store tøyninger i strekkarmeringen.

De øvrige to bjelkene ble forsterket med langsgående karbonfiberplater (SIKA CarboDur S) med to forskjellige arealer, $A_{CFRP}$ = 108 mm$^2$ og $A_{CFRP}$ = 216 mm$^2$. Karbonfiberplatenes tykkelse var 1,2 mm, gjennomsnittlige strekkfasthet var angitt til 3050 N/mm$^2$, mens E-modulen var 165000 N/mm$^2$. Karbonfiberplatene ble limt på strekksiden av T-bjelken, ved nedre del av steget. For begge prøveoppstillingene ble lastene plassert slik at momentbrudd skulle inntreffe.

Kapasitetsøkningen for de to forsterkede bjelkene var, sammenlignet med referansebjelken som fikk momentbrudd, henholdsvis 28% og 37%. For førstnevnte bjelke skjedde bruddet ved at karbonfiberplaten i underkant gikk til brudd ved ca. 10 $^{o}/_{oo}$ tøyning. Bruddet i den andre bjelken var et typisk skjærbrudd. Tøyningen i karbonfiberplaten var også her ca. 10 $^{o}/_{oo}$, og last/forskyvnings-diagrammets form og dets horisontale asymptote indikerer at den maksimale momentkapasiteten nær ble nådd. Bruddlasten kan ikke sammenlignes med den målte referansekapasiteten for skjær på grunn av forskjellig avstand mellom last og opplegg.

Forsterkningen førte til at lastnivået ved 1. riss økte med ca. 20 %. Stivhetsøkningen for lastnivå representative for bruksgrenselasten (etter opprissing) var også betydelige; anslagsvis 75% økning av bøyestivheten bestemt både ut fra målte last/forskyvnings- og moment-tøyningskurver. Økningen skyldes økt strekkarmeringsareal og at karbonfiberplatene bidrar til et tettere rissmønster og derved større ”tension-stiffening” effekt.

Bjelkene ble også analysert med elementmetodeprogrammet DIANA. Ikke-lineær material-oppførsel ble beskrevet ved hjelp av en rissmodell basert på modifisert trykkfelts teori (Vecchio og Collins). I utgangspunktet ble analysene gjennomført med middelfastheter for betongen og karbonfiberplatene, mens det for armeringen ble benyttet karakteristiske fastheter som inngangsverdi. Elementanalysene underestimerte derfor bjelkenes kapasitet med opptil 11 %. Dersom spennarmeringens fasthet øktes med 10 % ble det svært god overensstemmelse mellom analyser og forsøksresultat.

Basert på de gjennomførte undersøkelsene synes det som bruk av karbonfiberplater kan være en effektiv og enkel metode for forsterkning av eksisterende betongkonstruksjoner.

**Norwegian University of Science and Technology**
**NTNU**
Department of Structural Engineering

**REPORT**

| Tittel | Rapport nr. |
|---|---|
| **Strengthening prestressed concrete beams with Carbon Fiber Reinforced Polymer plates**<br>Experimental results and finite element analysis<br>DRAFT VERSION | Dato<br>2000-08-16 |
| Forfatter<br>Peter F. Takacs and Terje Kanstad | Sign. |
| ISBN Nr.<br>ISBN No. | Antall sider |

| Oppdragsgiver | Tilgjengelighet |
|---|---|
| Statens Vegvesen, Bru og Vegteknisk Avd.<br>SIKA Norge AS | |

Ekstrakt

Four prefabricated, prestressed T-beams with in-situ concrete top-layer were tested in the laboratory of Norwegian University of Science and Technology. The four T-beams were obtained by cutting half two DT-beams which were preserved when Isakveien Bridge was demolished in 1999. Isakveien Bridge was built in 1963. Two beams of the four were strengthened with longitudinal SIKA Carbon Fiber Reinforced Polymer (CFRP) plates.

Two beams were tested without strengthening to determine their flexural moment and shear capacity. Those were found 23 and 104 % higher than estimated according to the Norwegian Standard NS 3473:1998.

The other two beams were strengthened with longitudinal SIKA CarboDur S plates in two different quantities, $A_{CFRP} = 108\ mm^2$ and $A_{CFRP} = 216\ mm^2$. The excess net flexural moment capacity was 28 and 37 % respectively.

Simultaneously finite element analysis of the beams has been carried out. Non-linear material behavior was modeled by a total strain based crack model which was developed along the lines of the Modified Compression Field Theory. Finite element models have underestimated the capacity of the beams by up to 11 %.

Use of Carbon Fiber Reinforced Polymer plates has proved to be an effective and easy alternative to strengthen old concrete beams.

| Stikkord | Indexing terms |
|---|---|
| Material teknologi | Building technology |
| Betong | Concrete |
| Karbonfiber plater | Carbon fiber plate |
| Foroppspente element | Prestressed element |
| Forsterkning | Strengthening |

# CONTENT

## Norwegian summary (Norsk sammendrag)

Rapporten omhandler laboratorieforsøk og elementmetodeanalyser av føroppspente bjelker forsterket med karbonfiberplater. Fire prefabrikerte T-bjelker med påstøp er testet i laboratoriet ved Institutt for konstruksjonsteknikk, NTNU. Bjelkene er halve DT-element fra Isakveien bru ved Oslo som ble revet etter 35 år. Bortsett fra litt knusning ved endeoppleggene var elementene i meget god stand. Fashetsprøving på utborete sylindre (69.5 mm diameter) ga midlere sylinderfastheter på 76.5 og 55.2 $N/mm^2$ for henholdsvis konstruksjonsbetongen og påstøpen. Tilsvarende prosjektert betongkvalitet var B600 (C60) og B300 (C30). Utboringen viste at det var god heft i støpefugen.

To bjelker ble prøvd fritt opplagt med tvillinglaster for bestemmelse av referansekapasiteter for moment- og skjærbrudd. Referansekapasitetene er 23 og 104 % høyere enn beregnet etter NS 3473. For bjelken med planlagt momentbrudd inntraff ikke endelig brudd med knusning av betongtrykksonen på grunn av prøveriggens geometri. Imidlertid viser last-forskyvnings forløpet at maksimalt lastnivå allikevel ble tilnærmet oppnådd. For den andre bjelken inntraff skjærbrudd etter utstrakt skrårissdannelse og store tøyninger i strekkarmeringen.

De andre to bjelkene ble forsterket med langsgående karbonfiberplater (SIKA CarboDur S) med to forskjellige areal, $A_{CFRP}$=108 $mm^2$, og $A_{CFRP}$=216 $mm^2$. Materialets gjennomsnittlige strekkfasthet er angitt til 3050 $N/mm^2$, mens E-modulen er 165000 $N/mm^2$. For begge bjelkene ble lastene plassert slik at momentbrudd skulle inntreffe, og kapasitetsøkningen referert til referansebjelken for momentbrudd var henholdsvis 28 og 37 %. For førstnevnte bjelke skjedde bruddet ved at karbonfiberplata i underkant røk ved ca 10 ‰ tøyning. Bruddet i den andre bjelken var et typisk skjærbrudd. Tøyningen i karbonfiberplata var også nå ca 10 ‰, og lastforskyvnings-diagrammets horisontale asymtote indikerer at den maksimale momentkapasiteten også var nådd. Bruddlasten kan ikke sammenlignes med referansekapasiteten for skjær på grunn av forskjellig avstand mellom last og opplegg.

Forsterkningen førte til at lastnivået for 1.riss økte ca. 20 %. Økning i stivhet for lastnivå representative for bruksgrenselasten (etter opprissing) er også betydelig. Anslagsvis 75 % økning av bøyestivheten bestemt ved hjelp av målte tøyninger. Økningen skyldes økt strekkarmeringsareal og at karbonfiberplatene bidrar til et tettere rissmønster og derved større "tension-stiffening" effekt.

Bjelkene er også analysert med elementmetodeprogrammet Diana. Ikkelineær materialoppførsel er beskrevet ved hjelp av en rissmodell basert på modifisert trykkfelts teori. I utgangspunktet ble analysene gjennomført med middelfatsheter for betongen og karbonfiberplatene, mens det for armeringen ble benyttet karakteristiske verdier. Elementanalysene underestimerte da bjelkenes kapsitet med opptil 11 %. Dersom spennarmeringens fasthet ble økt med 10 % er det svært god overensstemmelse mellom analyser og forsøksresultat.

Bruk av karbonfiberplater synest å være en effektiv og enkel metode for forsterkning av eldre betongkonstruksjoner.

Prosjektet ble finansiert av følgende kilder:

Statens Vegvesen, Bruavdelingen og Vegteknisk avd. Gjennom NFR-prosjektet: Betongkonstruksjoners livsløp (1999).

SIKA Norge AS gjennom kontantstøtte og egeninnsats med forsterkning av bjelkene med SIKA CarboDur S.

NFR-prosjektet CMC (Computational Mechanics in Civil Engineering), Strategisk instituttprogram i regi av SINTEF.

Egeninnsats fra NTNU for gjennomføring av laboratorieforsøk og rapportering.

# 1 INTRODUCTION

Two prefabricated, prestressed DT elements with in-situ concrete top-layer were preserved for testing when Isakveien Bridge was demolished in 1999. Isakveien Bridge was built in 1963. Both DT elements were cut half for easier instrumentation of the tests and such way we have got four beams to test instead of the original two. That gave us further opportunities. Two beams of the four were strengthened with longitudinal SIKA Carbon Fiber Reinforced Polymer (CFRP) plates.

The prestressed concrete beams were tested in the laboratory of the Department of Structural Engineering at NTNU. The main objectives of the test series were to determine the ultimate bending and shear capacity of the original beams, their load-deflection and load-strain relationships and to investigate the effectiveness of using Carbon Fiber Reinforced Polymer plates as an alternative method of strengthening old concrete beams.

Finite element analyses of the beams have been being carried out simultaneously using the finite element program system DIANA. The numerical investigation was based on a total strain based crack model which was developed along the lines of the Modified Compression Field Theory originally proposed by Vecchio and Collins.

Note: Hereafter one beam is referred as one half of the original DT beam as it is shown in Figure 2.1. Respectively every quantity is given according to that.

The project has been financed by the following parties:

Norwegian Public Road Administration, Bridge Department and Road Technology Department, through the Norwegian Research Council project: Service-life of Concrete Structures (1999),

SIKA Norway with financial support and participation in the project with strengthening the beams with carbon fiber reinforced polymer SIKA CarboDur S plates,

Norwegian Research Council project CMC (Computational Mechanics in Civil-Engineering), strategic institute-program at SINTEF,

Norwegian University of Science and Technology with laboratory testing, documentation and reporting.

# 2 DESCRIPTION OF THE BEAMS

## 2.1 Geometry

The length of the beams is 11.20 meter. The cross-section is shown in Figure 2.1. Thickness of the in-situ top-layer is 10 cm according to the design specification. Actual measurement however shows that thickness is increasing gradually up to 13-14 cm along the last 3 meter at both ends.

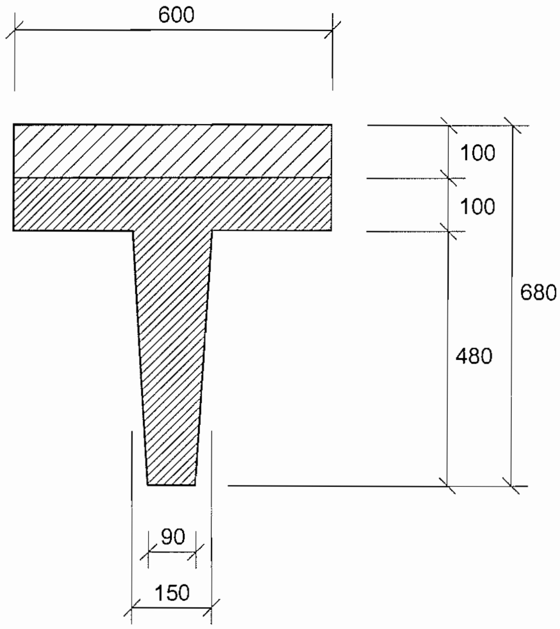


*Figure 2.1 Dimensions of the test beam (one half of the original DT element)*

## 2.2 Reinforcement

### 2.2.1 Prestressing steel

40 $\phi$4 St 1600/1800 prestressing wires are distributed over a height of 13 cm starting at 3 cm from the bottom of the web. Additionally 14 $\phi$4 wires are placed in the flange at 3 cm from the bottom of the flange. In each wire 12.6 kN initial prestressing force was applied.

Total initial prestressing force in one beam: $P_i = (40 + 14) \cdot 12.6 = 705.6\ kN$

### 2.2.2 Shear reinforcement

$\phi 8$ stirrups with 30 cm spacing along the entire length of the beam

## 2.3 Material properties

### 2.3.1 Concrete

Concrete grade is given in design specifications according to the old marking: B600 for the DT element and B300 for the in-situ concrete layer. To determine actual material parameters compression tests were carried out on 6 samples taken from the DT element and 6 samples taken from the in-situ layer.

The test samples were cylinders with diameter equal to 69.5 mm and variable height. Standard compression tests, however, should be carried out on standard 150/300 mm cylinders. To take into account the non-standard height/diameter ratio a correction factor was applied in accordance with the NS 3420 (Figure 2.2). Size effect due to different absolute dimensions was not taken into account.

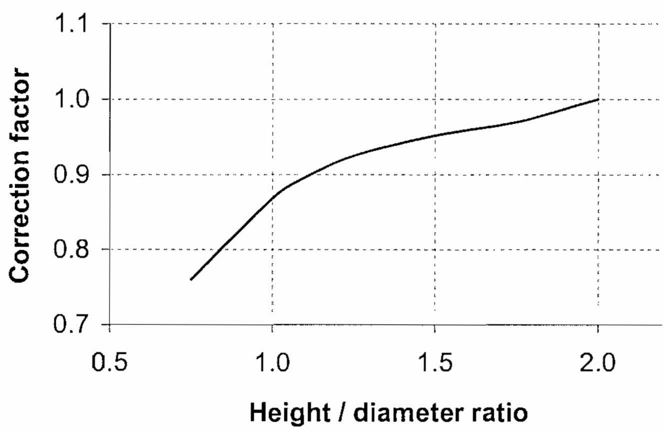


*Figure 2.2 Correction factor for cylinders with height/diameter ratio other than 2.0 (NS3420)*

Test results are shown in Table 2.1 and Table 2.2.

*Table 2.1 Compression test result / DT element*

| Sample | Diameter | Height | Weight | Force at failure | Stress at failure | Density | Height / Diameter | Correction factor | Strength |
|---|---|---|---|---|---|---|---|---|---|
| | *mm* | *mm* | *kg* | *kN* | *MPa* | *kg/m³* | - | - | *MPa* |
| 1 | 69.5 | 78 | 0.748 | 340 | 89.6 | 2528 | 1.12 | 0.901 | 80.7 |
| 2 | 69.5 | 72 | 0.690 | 333 | 87.8 | 2526 | 1.04 | 0.879 | 77.1 |
| 3 | 69.5 | 74 | 0.700 | 320 | 84.4 | 2493 | 1.06 | 0.887 | 74.8 |
| 4 | 69.5 | 67 | 0.642 | 349 | 92.0 | 2526 | 0.96 | 0.856 | 78.8 |
| 5 | 69.5 | 82 | 0.774 | 309 | 81.5 | 2488 | 1.18 | 0.913 | 74.4 |
| 6 | 69.5 | 82 | 0.769 | 305 | 80.4 | 2472 | 1.18 | 0.913 | 73.4 |
| **MEAN** | | | | | | **2506** | | | **76.5** |

*Table 2.2 Compression test result / in-situ concrete*

| Sample | Diameter | Height | Weight | Force at failure | Stress at failure | Density | Height / Diameter | Correction factor | Strength |
|---|---|---|---|---|---|---|---|---|---|
| | *mm* | *mm* | *kg* | *kN* | *MPa* | *kg/m³* | - | - | *MPa* |
| 1 | 69.5 | 77 | 0.727 | 239 | 63.0 | 2489 | 1.11 | 0.898 | 56.5 |
| 2 | 69.5 | 90 | 0.831 | 242 | 63.8 | 2434 | 1.29 | 0.931 | 59.4 |
| 3 | 69.5 | 97 | 0.875 | 181 | 47.7 | 2378 | 1.40 | 0.943 | 45.0 |
| 4 | 69.5 | 104 | 0.942 | 202 | 53.2 | 2388 | 1.50 | 0.952 | 50.7 |
| 5 | 69.5 | 80 | 0.750 | 237 | 62.5 | 2471 | 1.15 | 0.907 | 56.7 |
| 6 | 69.5 | 97 | 0.931 | 252 | 66.4 | 2530 | 1.40 | 0.943 | 62.6 |
| **MEAN** | | | | | | **2448** | | | **55.2** |

Characteristic value of the compression strength, $f_{ck}$, mean value of the tensile strength, $f_{ctm}$ and the fracture energy, $G_F$ were estimated according to the recommendations of the CEB-FIP Model Code 1990 [1]. See Table 2.3.

*Table 2.3 Concrete material parameters*

| | $f_{cm}$ | $f_{ck}$ | $f_{ctm}$ | $G_F$ |
|---|---|---|---|---|
| | *MPa* | *MPa* | *MPa* | *Nmm/mm²* |
| DT element | 77 | 69 | 5.1 | 0.125 |
| in-situ layer | 55 | 47 | 3.9 | 0.099 |

### 2.3.2 Prestressing wire

Quality of the prestressing wire is St 1600/1800.

$f_{p0.2} = 1600\ MPa$

$f_{p,max} = 1800\ MPa$

$E_p = 205\ GPa$

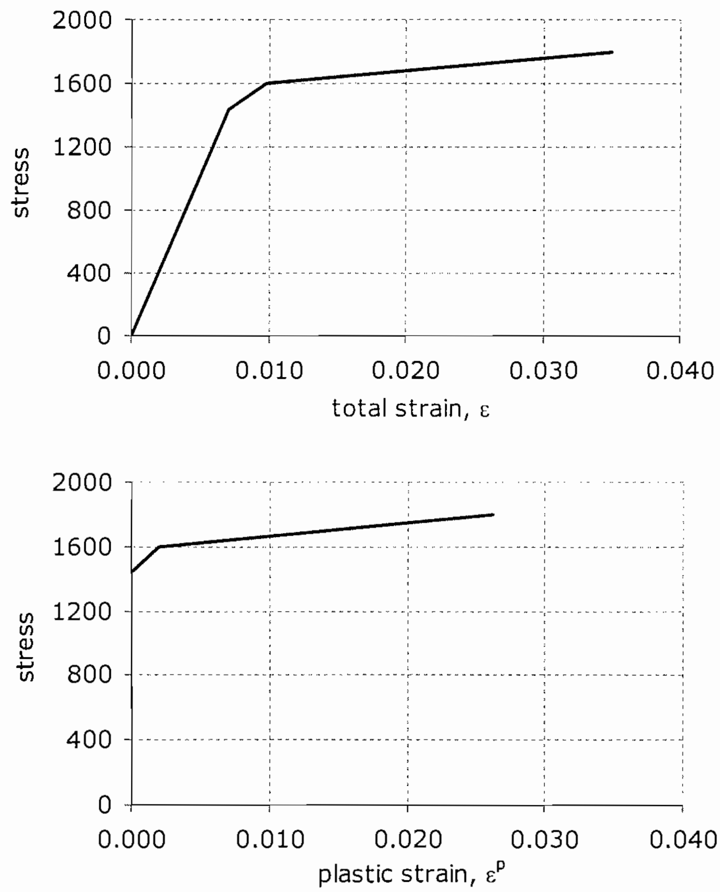


*Figure 2.3 Idealized stress – total strain / plastic strain diagrams*

## 2.4 Condition of the beams

The beams were in good condition without any serious damage which would have influenced their global structural behavior at the tests.

Although the bottom of the beams were slightly crushed at the ends, those were repaired with epoxy based resin. The supports were moved closer to each other, away from these crushed zones anyway.

# 3 STRENGTHENING OF THE BEAMS WITH CFRP PLATES

## 3.1 General

Beams no.3 and no.4 were strengthened with longitudinal Carbon Fiber Reinforced Polymer plates in two different quantities.

The CFRP plates used for strengthening the beams were SIKA CarboDur S plates. Its thickness and width are 1.2 mm and 50 mm respectively.

## 3.2 Geometrical arrangement

The total amount of CFRP is $A_{CFRP}$ = *108 mm*$^2$ and $A_{CFRP}$ = *216 mm*$^2$ for beams no.3 and no.4 respectively. See arrangements in detail in Figure 3.1 and Figure 3.2.

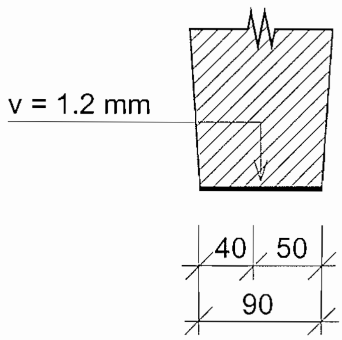


*Figure 3.1 Arrangement of the CRRP plates for beam no.3*

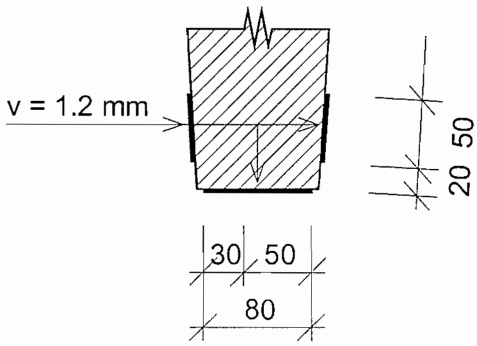


*Figure 3.2 Arrangement of the CRRP plates for beam no.4*

## 3.3 Material properties of the CFRP

Material properties of the SIKA CarboDur S plates:

$f_t = 2850\ MPa$

$f_{t,mean} = 3050\ MPa$

$E = 165\ GPa$

$\varepsilon_u > 1.7\ \%$

$\varepsilon_{u,mean} = 1.85\ \%$

# 4 TEST SETUP

## 4.1 Layout of the beams in the test rig

The four tests were carried out with different arrangements in accordance with the intended failure mode.

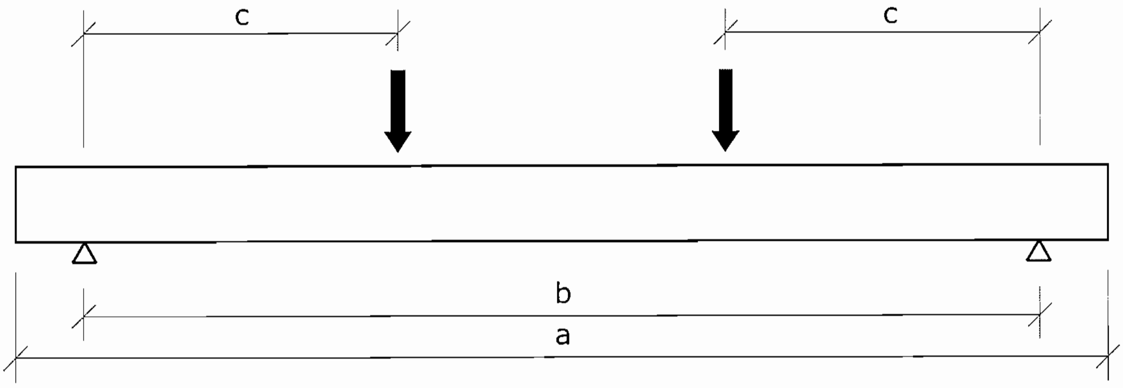


*Figure 4.1 Dimensions (see also Table 4.1)*

*Table 4.1 Dimensions*

| Test | | a | b | c |
|---|---|---|---|---|
| No. | Beam | Beam length [m] | Span length [m] | Arm of force [m] |
| 1 | Original beam (moment failure assumed) | 11.20 | 10.80 | 3.60 |
| 2 | Original beam (shear failure assumed) | 11.20 | 9.20 | 1.80 |
| 3 | Beam reinforced with CFRP plates (longitudinal, 108 $mm^2$) | 11.20 | 10.20 | 4.10 |
| 4 | Beam reinforced with CFRP plates (longitudinal, 216 $mm^2$) | 11.20 | 9.20 | 3.60 |

## 4.2 Measured variables

### 4.2.1 Displacements

Displacements were measured at midspan and 1.8 meter from midspan on both sides. In addition displacements were recorded at the supports in order to eliminate their influence from the deflections.

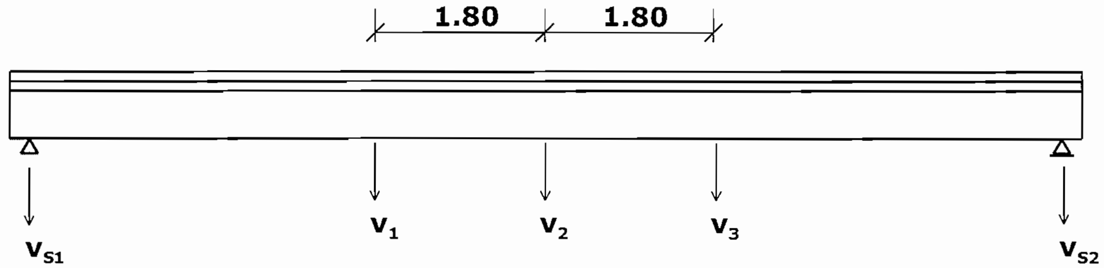


*Figure 4.2 Measured displacements*

### 4.2.2 Strains

Strain measurements were carried out at midspan in both the tension and the compression zones.

At the bottom (tension zone) strain was measured on both sides of the web at 50 mm from the bottom of the beam. Measurement was carried out by measuring the relative longitudinal displacement by LVDT on a basis length of 200 mm.

Strain measurement in the compression zone was carried out with strain gauges (PL60). The number and the position of the gauges varied from test to test. In each case at least one strain gauge was positioned at the top of the flange. The exact position of the gauges in each case are shown with the test results (Session 5)

### 4.2.3 Cracking

Average crack width and crack spacing were measured at certain load levels at tests no.1 and no.3.

# 5 MOMENT AND SHEAR CAPACITY OF THE BEAMS ACCORDING TO THE NORWEGIAN STANDARD

Flexural moment and shear capacity are calculated according to the Norwegian Standard NS 3473:1998 [2].

## 5.1 Moment capacity

### 5.1.1 General

The following assumptions were made in the calculation:

- cross-section remains plane after bending (Bernoulli-Navier theory)
- failure occurs when the strain in the extreme compression fibre reaches -2.5‰
- stress-strain diagrams are idealized as Figure 5.1 shows
- there is perfect bond between the concrete and the CFRP plates

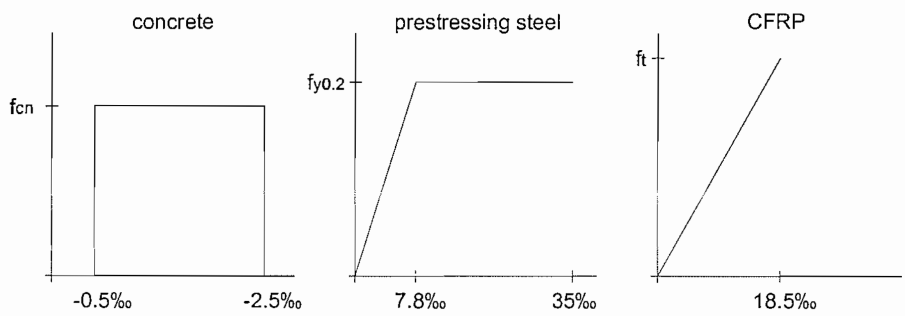


*Figure 5.1 Stress-strain diagrams*

### 5.1.2 Original beam (un-strengthened)

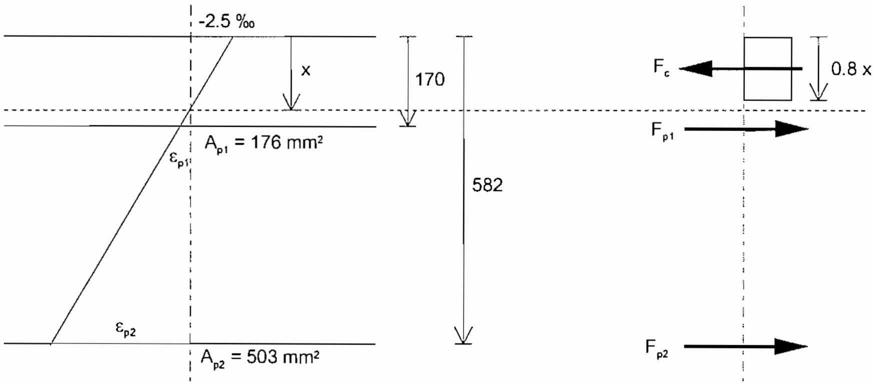


*Figure 5.2*

It was assumed that both layers of steel are yielding.

$$N = 0.8 \cdot x \cdot 600 \cdot f_{cn} + (176 + 503) \cdot f_{y0.2} = 0 \qquad x = 66.6\ mm$$

$$\varepsilon_{p1}^{total} = 0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p = 8.8\ ‰$$

$$\varepsilon_{p2}^{total} = 0.0025 \cdot \frac{582 - x}{x} + \varepsilon_p = 24.2\ ‰$$

$\varepsilon_p$ *strain from prestressing, 0.0048*

The assumptions were correct, both layers of steel are yielding.

$$M = -0.8 \cdot x \cdot 600 \cdot f_{cn} \cdot 0.6 \cdot x + 176 \cdot f_{y0.2} \cdot (170 - x) + 503 \cdot f_{y0.2} \cdot (582 - x) = 487\ kNm$$

### 5.1.3 Strengthened beams (beam no.3, $A_{CFRP}$ = 108 mm$^2$)

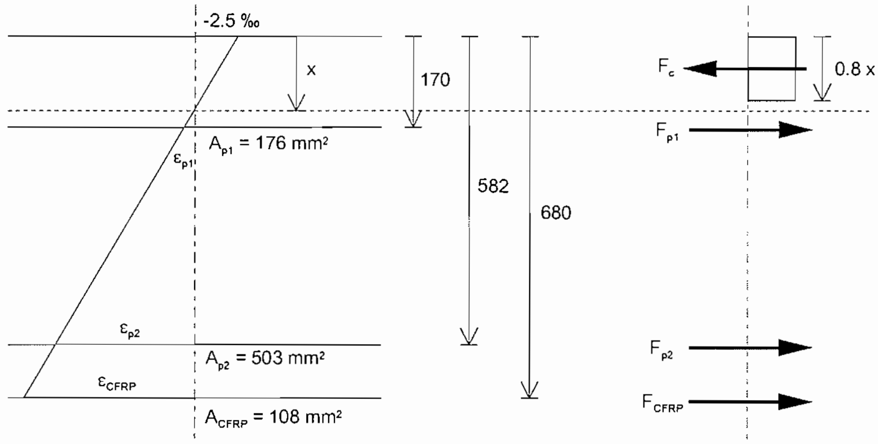


*Figure 5.3*

It was assumed that steels in the web are yielding while steels in the flange are in elastic state.

$$N = 0.8 \cdot x \cdot 600 \cdot f_{cn} + 503 \cdot f_{y0.2} + \left(0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p\right) \cdot E_p \cdot 176 +$$
$$+ 0.0025 \cdot \frac{680 - x}{x} \cdot E_{CFRP} \cdot 108 = 0$$

$x = 84.8\ mm$

$$\varepsilon_{p1}^{total} = 0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p = 7.4\ ‰$$

$$\varepsilon_{p2}^{total} = 0.0025 \cdot \frac{582 - x}{x} + \varepsilon_p = 19.5\ ‰$$

$$\varepsilon_{CFRP} = 0.0025 \cdot \frac{680 - x}{x} = 17.5\ ‰$$

$\varepsilon_p$ *strain from prestressing, 0.0048*

The assumptions were correct, steels in the web are yielding while steels in the flange are in elastic state.

$$M = -0.8 \cdot x \cdot 600 \cdot f_{cn} \cdot 0.6 \cdot x + 176 \cdot \left(0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p\right) \cdot E_p \cdot (170 - x) +$$
$$+ 503 \cdot f_{y0.2} \cdot (582 - x) + 108 \cdot 0.0025 \cdot \frac{680 - x}{x} \cdot E_{CFRP} \cdot (680 - x) = 679\ kNm$$

### 5.1.4 Strengthened beams (beam no.4, $A_{CFRP}$ = 216 $mm^2$)

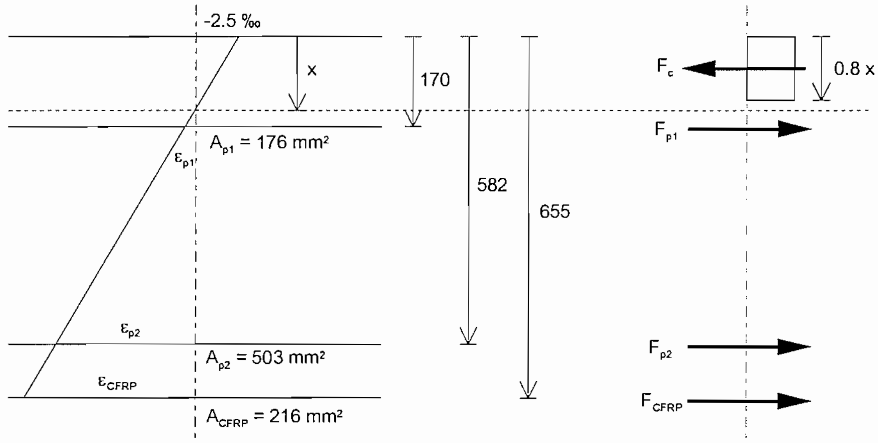


*Figure 5.4*

It was assumed that steels in the web are yielding while steels in the flange are in elastic state.

$$N = 0.8 \cdot x \cdot 600 \cdot f_{cn} + 503 \cdot f_{y0.2} + \left(0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p\right) \cdot E_p \cdot 176 +$$
$$+ 0.0025 \cdot \frac{655 - x}{x} \cdot E_{CFRP} \cdot 216 = 0$$

$x = 96.1\ mm$

$$\varepsilon_{p1}^{total} = 0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p = 6.8\ ‰$$

$$\varepsilon_{p2}^{total} = 0.0025 \cdot \frac{582 - x}{x} + \varepsilon_p = 17.5\ ‰$$

$$\varepsilon_{CFRP} = 0.0025 \cdot \frac{655 - x}{x} = 14.5\ ‰$$

$\varepsilon_p$ *strain from prestressing, 0.0048*

The assumptions were correct, steels in the web are yielding while steels in the flange are in elastic state.

$$M = -0.8 \cdot x \cdot 600 \cdot f_{cn} \cdot 0.6 \cdot x + 176 \cdot \left(0.0025 \cdot \frac{170 - x}{x} + \varepsilon_p\right) \cdot E_p \cdot (170 - x) +$$
$$+ 503 \cdot f_{y0.2} \cdot (582 - x) + 216 \cdot 0.0025 \cdot \frac{655 - x}{x} \cdot E_{CFRP} \cdot (655 - x) = 798\ kNm$$

## 5.2 Shear capacity

The calculation of the shear capacity was carried out according to the Norwegian Standard NS3473:1998 Section 12.3.2.

For beam no.3 and no.4 the effect of the longitudinal CFRP plates on shear capacity is neglected.

### 5.2.1 Concrete contribution with no axial force

$$V_{co} = 0.3\left(f_{td} + \frac{k_A\ A_s}{\gamma_c\ b_w\ d}\right) b_w\ d\ k_V \leq 0.6\ f_{td}\ b_w\ d\ k_V$$

$$V_{co} = 0.3\left(2.6 + \frac{100 \cdot 503}{1 \cdot 90 \cdot 550}\right) 90 \cdot 550 \cdot 1 \leq 0.6 \cdot 2.6 \cdot 90 \cdot 550 \cdot 1$$

$$V_{co} = 53.7\ kN \leq 91.3\ kN$$

### 5.2.2 Concrete contribution with axial compression

$$V_{cd} = V_{co} + 0.8\,\frac{N_f \cdot W_c}{M_f \cdot A_c}\,V_f \leq \left(f_{td}\ k_V - \frac{0.25\ N_f}{A_c}\right) b_w\ z_1$$

$$V_{cd} = V_{co} + 0.8 \cdot 705600 \cdot 71.98\,\frac{V_f}{M_f} \leq \left(2.6 \cdot 1 - \frac{0.25 \cdot 705600}{117600}\right) \cdot 90 \cdot 0.7 \cdot 550$$

The shear capacity of beams with axial compression varies along the beam axis. Here the shear capacity is calculated with respect to the cross-section under the concentrated point load in each test configuration. Shear capacity is increasing towards the support.

Furthermore, if the ultimate shear capacity is concerned, the existing shear force, $V_f$ in the cross-section under the point load is closely related to the total shear capacity, $V_d$ in the same cross-section (See equation above). Therefore an iterative solution was implemented in a spreadsheet. Eventually results can be verified if you compare $V_f$ in Table 5.1 and $V_d$ in Table 5.2.

*Table 5.1*

| | span lenght | arm of force | $M_f$ | $V_f$ | $\mathbf{V_{cd}}$ | $V_{upper\ limit}$ |
|---|---|---|---|---|---|---|
| | m | m | kNm | kN | **kN** | kN |
| Test no.1 | 10.8 | 3.6 | 399 | 134 | **67.4** | 124.5 |
| Test no.2 | 9.2 | 1.8 | 166 | 159 | **92.8** | 124.5 |
| Test no.3 | 10.2 | 4.1 | 473 | 131 | **65.0** | 124.5 |
| Test no.4 | 9.2 | 3.6 | 401 | 133 | **67.2** | 124.5 |

### 5.2.3 Contribution from shear reinforcement

$$V_{sd} = \frac{f_{sd}\ A_{sv}}{s}\ z\left(1 + \cot\alpha\right)\sin\alpha$$

$$V_{sd} = \frac{400 \cdot 100.5}{300}\ 0.9 \cdot 550 \cdot \left(1 + 0\right) \cdot 1$$

$$V_{sd} = 66.3\ kN$$

### 5.2.4 Total shear capacity

The total shear capacity is the sum of the concrete contribution (5.2.2) and the shear reinforcement contribution (5.2.3).

*Table 5.2 Shear capacity*

| | $V_{cd}$ | $V_{sd}$ | $\mathbf{V_d}$ |
|---|---|---|---|
| | kN | kN | **kN** |
| Test no.1 | 67.4 | 66.3 | **133.7** |
| Test no.2 | 92.8 | 66.3 | **159.1** |
| Test no.3 | 65.0 | 66.3 | **131.3** |
| Test no.4 | 67.2 | 66.3 | **133.5** |

## 5.3 Summary

Table 5.3 summarizes the moment and shear capacity in each test configuration. The shear capacity concerns the cross-section under the concentrated point load.

*Table 5.3 Moment and shear capacity according to NS 3473:1998*

| | $M^{total}$ | $V^{total}$ | $M^{net}$ | $V^{net}$ |
|---|---|---|---|---|
| | *kNm* | *kN* | *kNm* | *kN* |
| Beam no.1 | 487 | 134 | 422 | 126 |
| Beam no.2 | 487 | 159 | 442 | 147 |
| Beam no.3 | 679 | 131 | 622 | 127 |
| Beam no.4 | 798 | 133 | 753 | 129 |

The net capacity is the total capacity minus the moment/shear due to selfweight.

# 6 TEST RESULTS

## 6.1 Beam no.1 (original beam for moment failure)

### 6.1.1 General

The first beam was tested to determine the actual flexural moment capacity of the original beam. Actual failure was not reached due to the limitations of the test rig and the large deflections of the beam. The almost horizontal asymptote of the load-deflection curve, however, suggests that the ultimate capacity was reached. Load-strain curves also make clear that failure would happen due to the crushing of the concrete in the compression zone. The strain at the extreme compression fibre, $\varepsilon_3$ was -1.8 ‰ at the last stage.

A horizontal plateau can be seen in each load-deflection/strain diagram at load level 136 kN. At that load level the hydraulic jacks reached their ultimate capacity of movement, so steel plates must have been placed under the jacks in order to go further with the test. The procedure caused about an hour long delay.

For notations of the measured variables see Figure 4.2 and Figure 6.1.

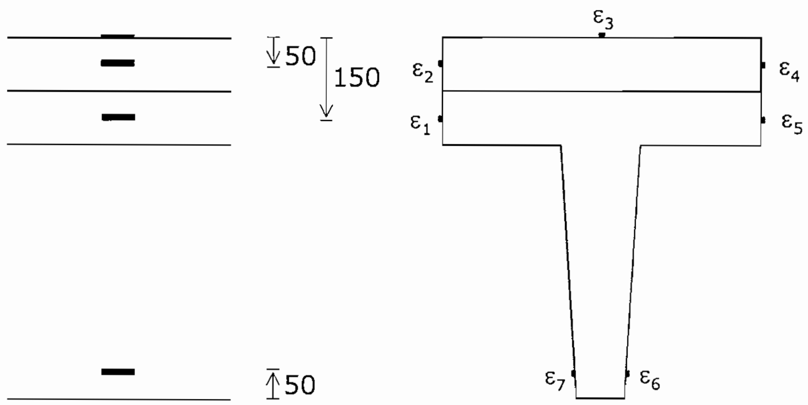


*Figure 6.1 Position and notation of the strain measurements at midspan*

## 6.1.2 Deflection and strain diagrams

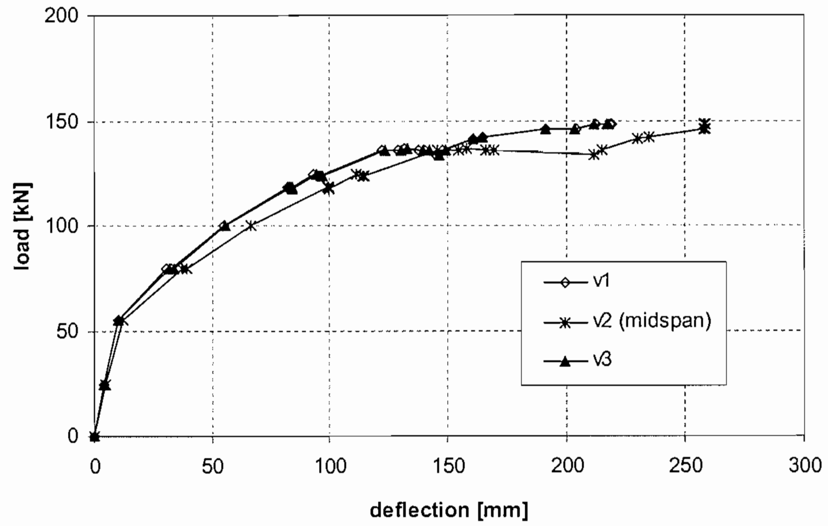


*Figure 6.2 Deflections*

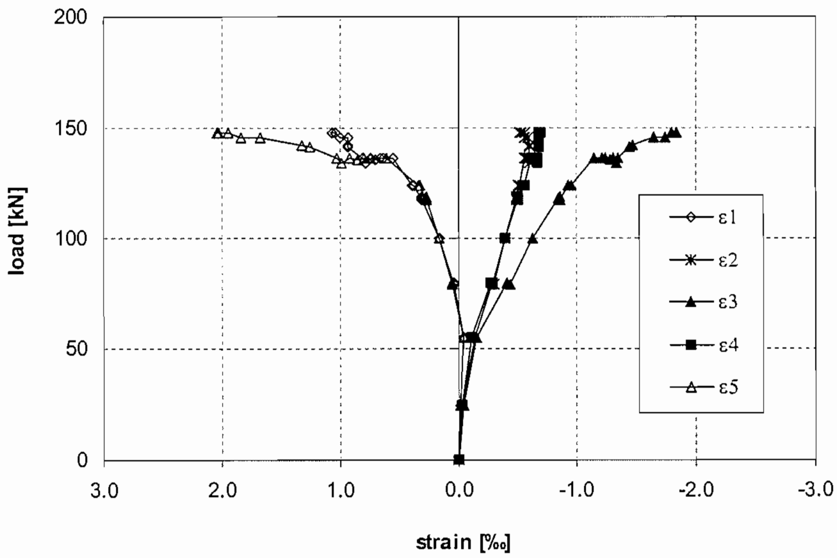


*Figure 6.3 Strains*

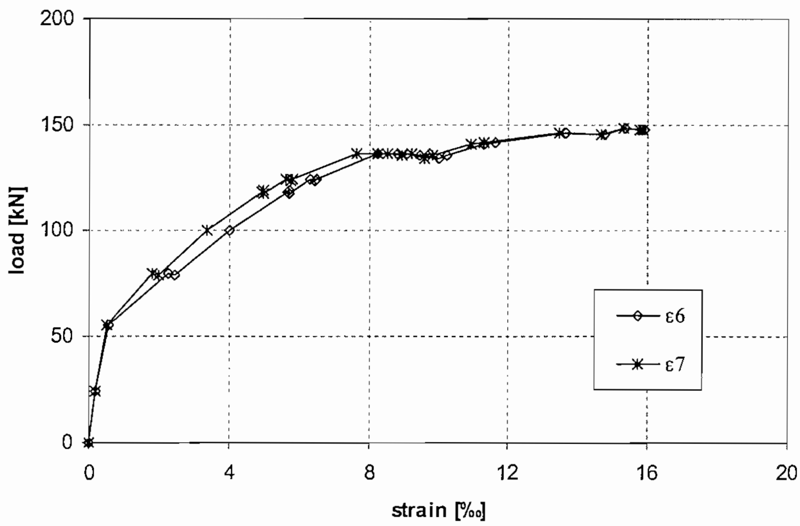


*Figure 6.4 Strains*

### 6.1.3 Cracking

The table below summarizes the crack observations.

*Table 6.1 Crack width and spacing*

| Load level (kN) | Average crack width (mm)[1] | Number of cracks in 2 meters[2] |
|---|---|---|
| 55 | first crack appears | |
| 60 | not available | 2 |
| 80 | 0.15 | 17 |
| 100 | 0.20 | n.a. |

[1] between the point loads
[2] the central two meters

## 6.2 Beam no.2 (original beam for shear failure)

### 6.2.1 General

The intended failure mode for the second beam was shear failure. Point loads were positioned 1.80 meter from the supports which is about 2.8 times the effective height.

For notations of the measured variables see Figure 4.2 and Figure 6.5.

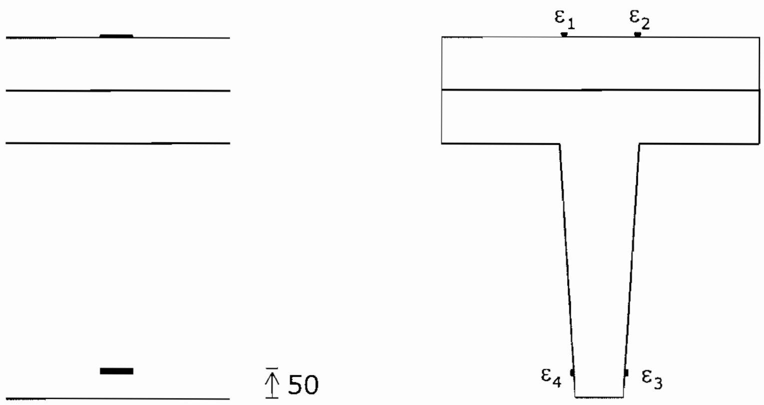


*Figure 6.5 Position and notation of the strain measurements at midspan*

The failure of the beam was a combined flexural-shear failure. Although it happened on an unexpected way. When the hydraulic jacks reached their ultimate capacity of movement (~150mm), the beam was heavily cracked and the load-deflection diagrams suggested that failure could happen by any further loading. In order to go further with the test, it was agreed to place steel plates under the jacks. To perform this operation the jacks must have been unloaded and temporary steel bars were placed between the test rig and the beam to keep the beam loaded. The steel bars were placed approx. 80 cm from the jacks in the direction of the midspan. As the jacks were unloaded the steel bars gradually took over the load. The failure happened during this operation and can be explained by the combination of two reasons:

a) Flexural-shear cracking was already severe and well-developed towards the jacks. When the steel bars took over the load at about 80 cm from the position of the jacks, the diagonal compression field was not able to adapt to the new direction due to the system of severe cracks.

b) The shear capacity of prestressed beams are decreasing as the distance from the support is increasing.

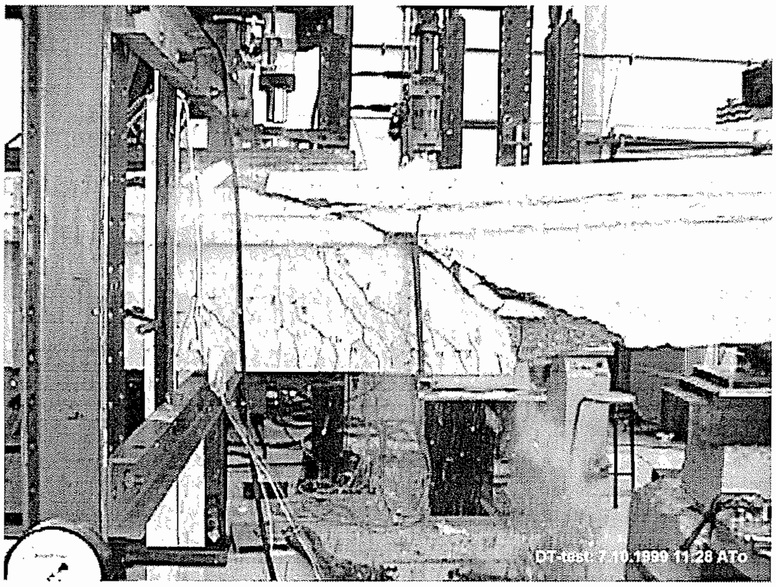


*Figure 6.6 The beam exactly when failure occurs*

### 6.2.2 Deflection and strain diagrams

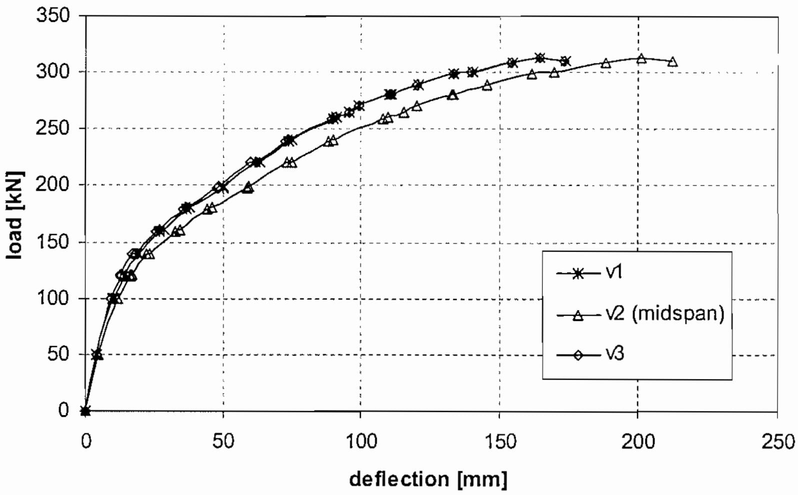


*Figure 6.7 Deflections*

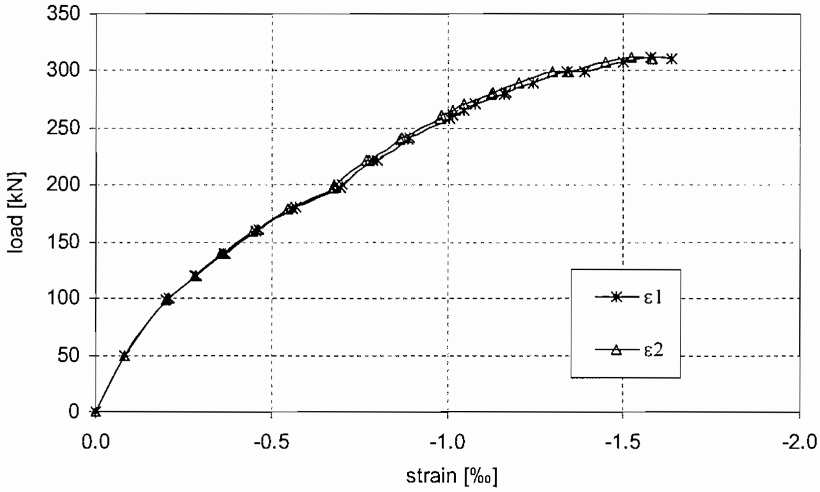


*Figure 6.8 Strains*

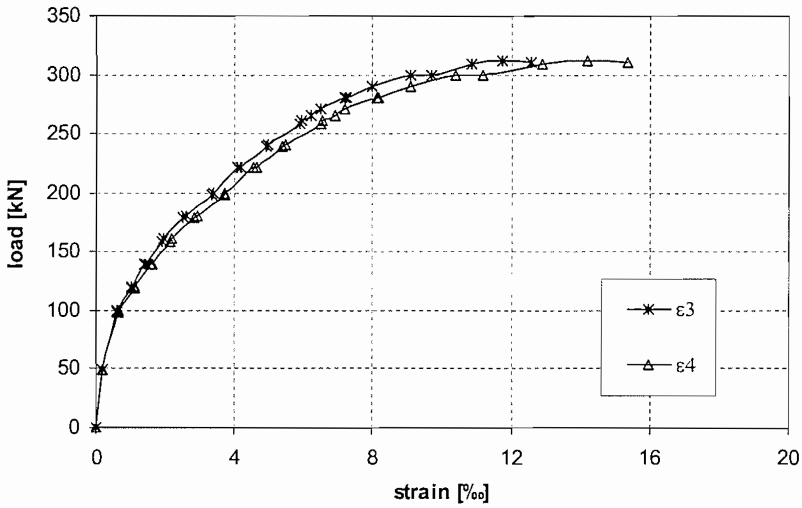


*Figure 6.9 Strains*

## 6.3 Beam no.3 (reinforced with longitudinal CFRP plates, 108 mm$^2$)

### 6.3.1 General

The failure occurred in the CFRP plate such way that the plate split apart along a horizontal surface starting from a wide flexural-shear crack.

For notations of the measured variables see Figure 4.2 and Figure 6.10.

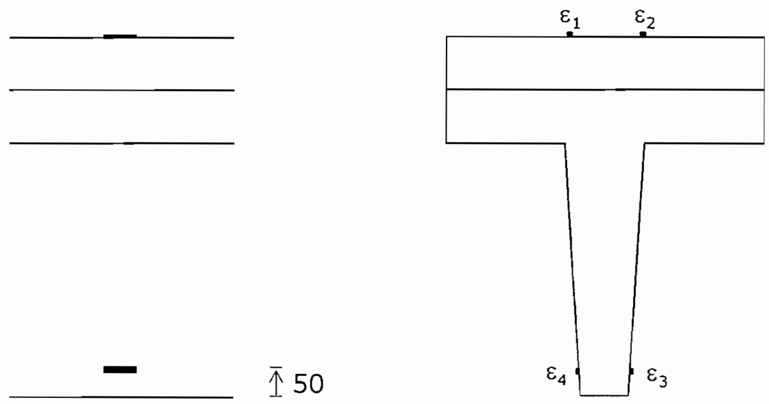


*Figure 6.10 Position and notation of the strain measurements at midspan*

### 6.3.2 Deflection and strain diagrams

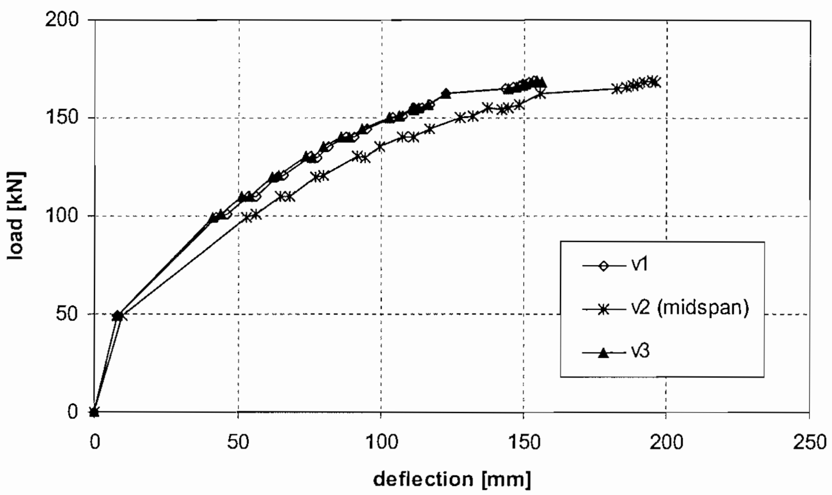


*Figure 6.11 Deflections*

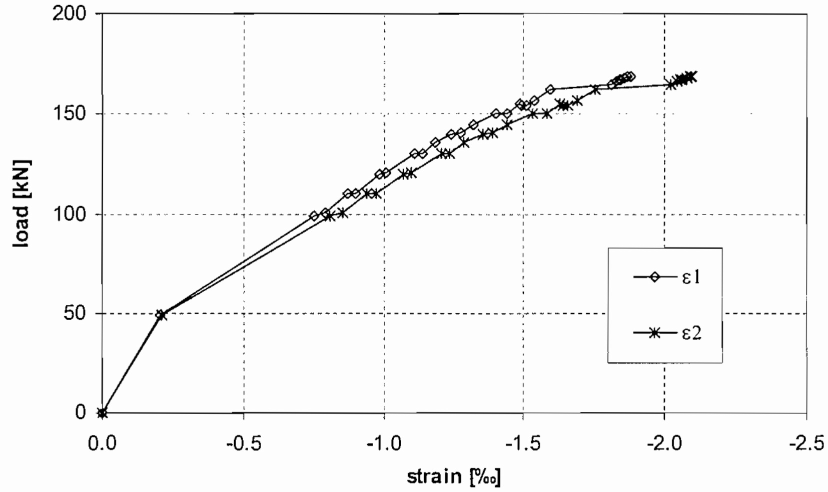


*Figure 6.12 Strains*

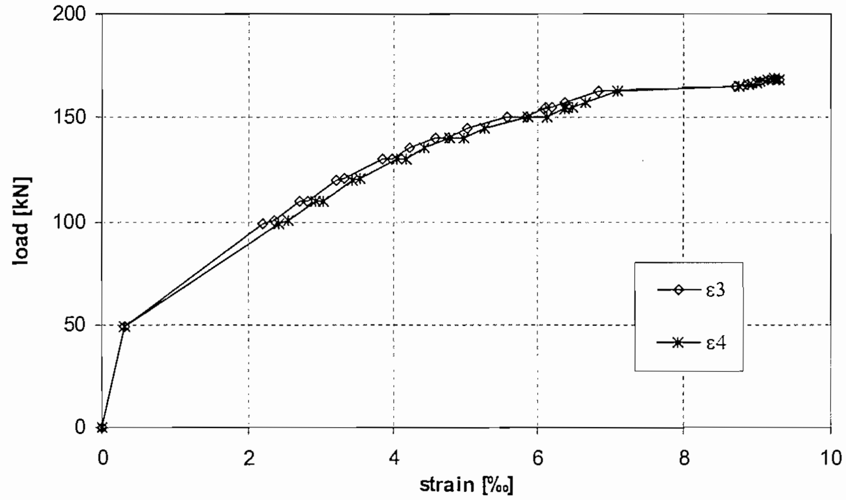


*Figure 6.13 Strains*

### 6.3.3 Cracking

We have found that the average crack width is smaller along the height of the prestressing wires ('tension zone' in Figure 6.14) than in the web above that zone ('web'). That shows that the dense and fine reinforcement is effectively controlling cracking. It was also experienced that the crack width was considerably reduced at the bottom of the beam, right above the CFRP plate.

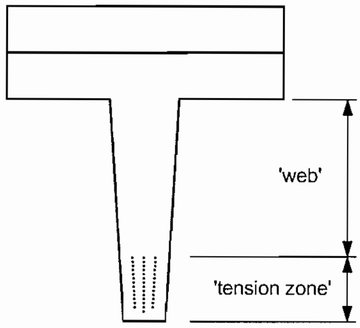


*Figure 6.14*

The table below summarizes the crack observations.

*Table 6.2 Crack width and spacing*

| Load level (kN) | Crack width (mm)[1] | | | Number of cracks in 2 meters[2] |
|---|---|---|---|---|
| | Tension zone | Web | Shear crack | |
| 60 | 0.10 – 0.15 | 0.20 – 0.30 | | n.a. |
| 80 | 0.10 – 0.20 | 0.10 – 0.50 | | 19 |
| 100 | 0.10 – 0.30 | 0.10 – 0.60 | | 26 |
| 120 | 0.10 – 0.50 | 0.10 – 0.70 | up to 0.90 | 31 |
| 140 | 0.10 – 0.50 | 0.10 – 0.80 | up to 1.30 | n.a. |
| 160 | up to 0.70 | up to 1.20 | up to 2.00 | n.a. |

[1] between the point loads
[2] the central two meters

## 6.4 Beam no.4 (reinforced with longitudinal CFRP plates, 216 mm$^2$)

### 6.4.1 General

The failure was a typical shear failure.

For notations of the measured variables see Figure 4.2 and Figure 6.15.

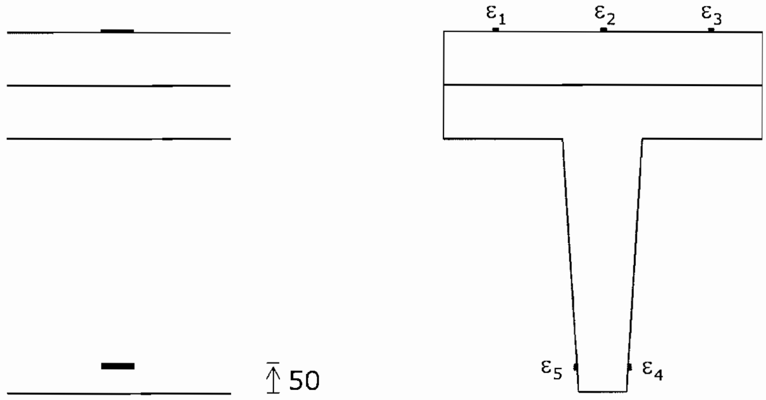


*Figure 6.15 Position and notation of the strain measurements at midspan*

### 6.4.2 Deflection and strain diagrams

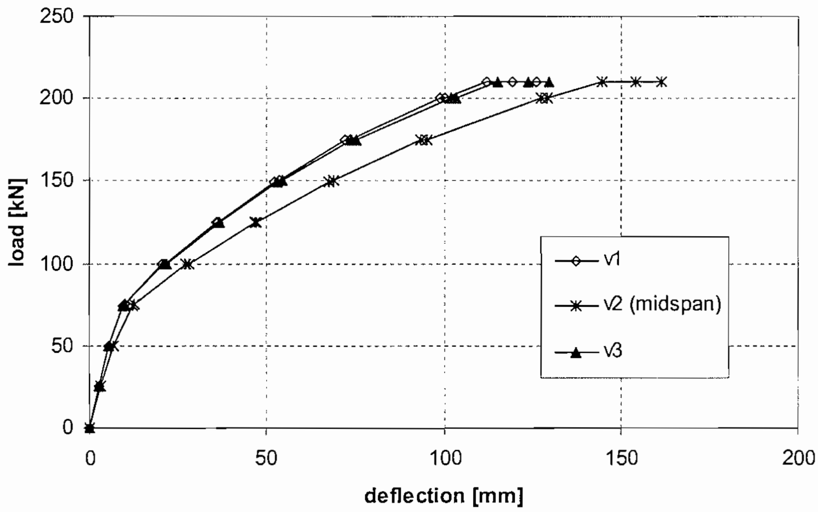


*Figure 6.16 Deflections*

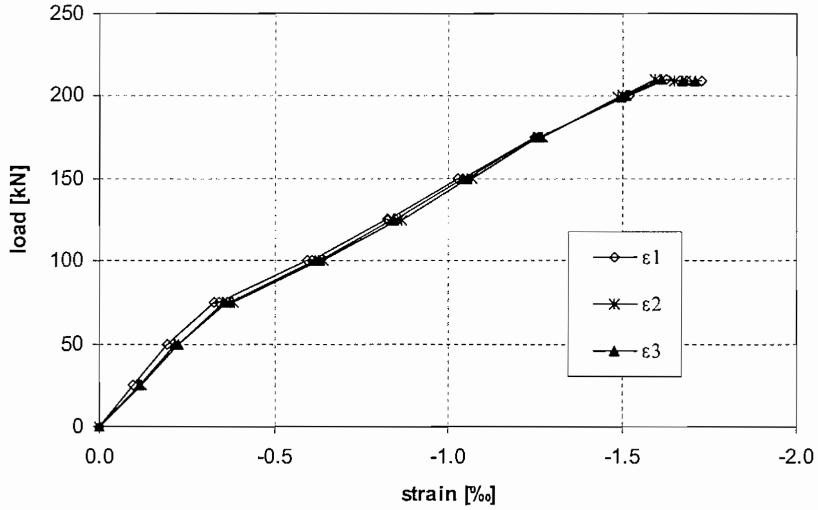


*Figure 6.17 Strains*

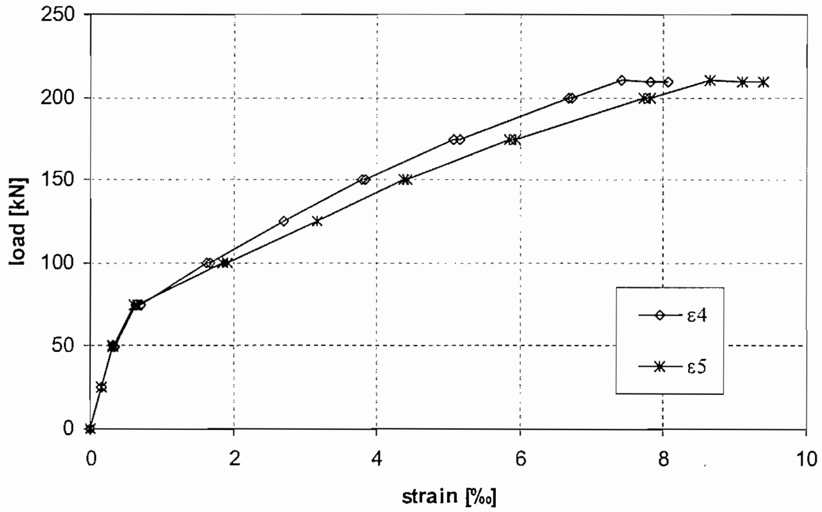


*Figure 6.18 Strains*

# 7 FINITE ELEMENT ANALYSIS

Finite element analyses of the beams have been carried out simultaneously with the tests using the general purpose finite element program system DIANA. Some aspects of the geometrical and material modeling is summarized here. A more comprehensive description of the theoretical background can be found in the referred literature.

## 7.1 Geometry model

Due to the low width-height ratio of the slab, uniform stress distribution can be assumed in the direction perpendicular to the plane of the finite element mesh and so a two dimensional plane stress idealization is justified.

Since the structure and loading scheme are symmetrical, it was sufficient to model only the left half of the beam. I used eight-node plane stress elements with 3*2 Gauß integration scheme.

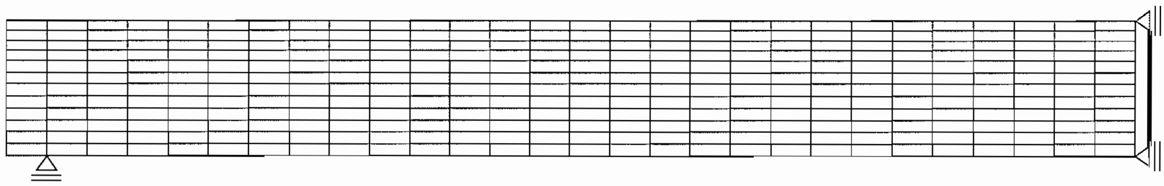

*Figure 7.1 Finite element mesh*

## 7.2 Material model

### 7.2.1 Material model for concrete

A total strain based crack model [3] was used for the analysis which was developed along the lines of the Modified Compression Field Theory originally proposed by Vecchio and Collins [4]. The total strain based constitutive model describes the stress as a function of the strain. In my analyses the stress-strain relationships were evaluated in the principal directions of the strain vector which is known as the rotating crack concept.

In cracked concrete the principal compressive stress is a function not only of the principal compressive strain, $\varepsilon_2$, but also of the coexisting principal tensile strain, $\varepsilon_1$.

$$f_{2,max} = f_c \frac{1}{0.8 + 170\varepsilon_1} \leq f_c$$

where

$f_{2,max}$ compressive strength when the lateral principal tensile strain is $\varepsilon_1$,
$f_c$ uniaxial compressive strength.

To describe the compressive stress – compressive strain relationship the Thorenfeldt curve [3, 5] was chosen:

$$\sigma_2 = -f_{2,max} \frac{\varepsilon_2}{\varepsilon_p} n \left(1 - n + \left(\frac{\varepsilon_2}{\varepsilon_p}\right)^{nk}\right)^{-1}$$

where

$\varepsilon_p$ compressive strain at the maximum compressive stress, -0.0022,
$n,k$ model parameters which can be estimated from

$$n = 0.8 + \frac{f_c}{17} \qquad k = \begin{cases} 1 & \text{if} \quad 0 > \varepsilon_2 > \varepsilon_p \\ 0.67 + \frac{f_c}{62} & \text{if} \quad \varepsilon_2 \leq \varepsilon_p \end{cases}$$

In the tensile regime a constant stress cut-off with exponential softening branch was used.

See Section 2.3.1 for material parameters.

### 7.2.2 Material model for the prestressing steel

Elasto-plastic material model with strain hardening was used for the prestressing wires. Hardening parameters was set according to the idealized stress-strain curve illustrated in Figure 2.3.

Relaxation was estimated according to the recommendations of the CEB-FIP Model Code 1990: 3.2 percent of the initial prestressing force after 1000 hours and 6.2 percent after 30 years.

### 7.2.3 Material model for CFRP

Linear elastic material model was used for the Carbon Fiber Reinforced Polymer. See Section 3.3 for material parameters.

## 7.3 Analysis results

Since the objective of the numerical investigation was to the simulate the real structural behavior of the beams, mean values of the material parameters were used instead of the characteristic values. These parameters were available for the concrete and the Carbon Fiber Reinforced Polymer, but not for the prestressing steel. Therefore two analyses were preformed in each case: A) material properties of the prestressing steel are taken by its known characteristic values and B) material properties of the prestressing steel are taken by estimated mean values which are assumed to be 10 percent higher than its characteristic values concerning the tensile strength and yield stress. Such an arbitrarily chosen value may be questionable but it is the author's opinion that it is reasonable and the curves can be easily adjusted for other values by scaling.

Figure 7.2-5 shows the analysis results (continuous line for analysis A, dashed line for analysis B).

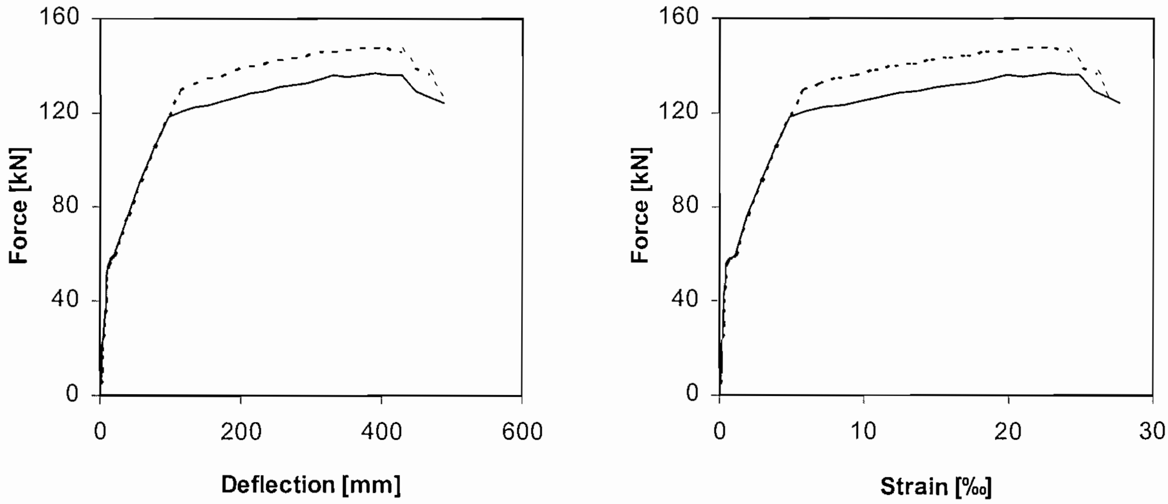


*Figure 7.2 Beam no.1 - Deflection and strain at the extreme tensile fibre at midspan*

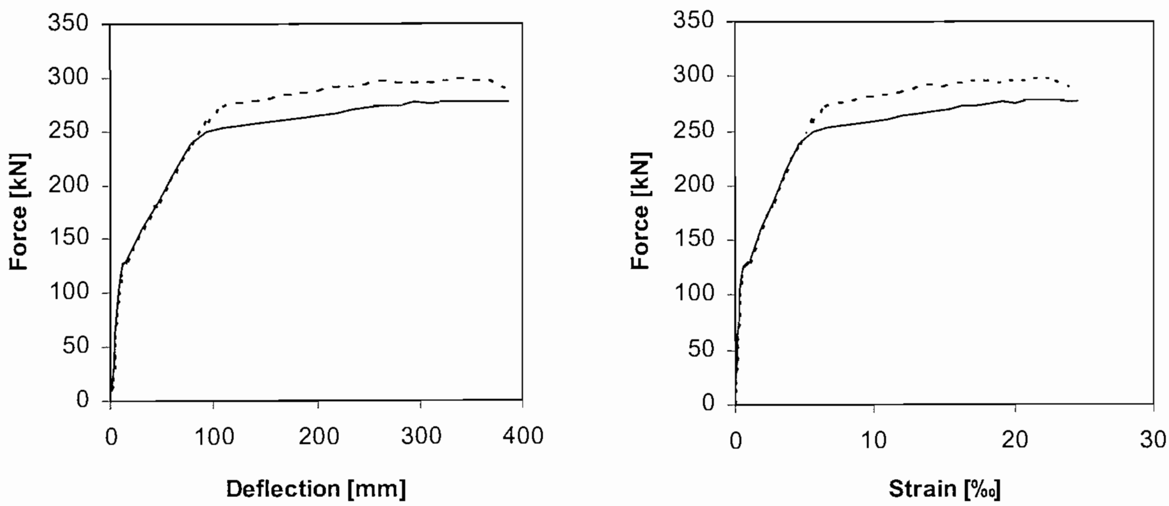


*Figure 7.3 Beam no.2 - Deflection and strain at the extreme tensile fibre at midspan*

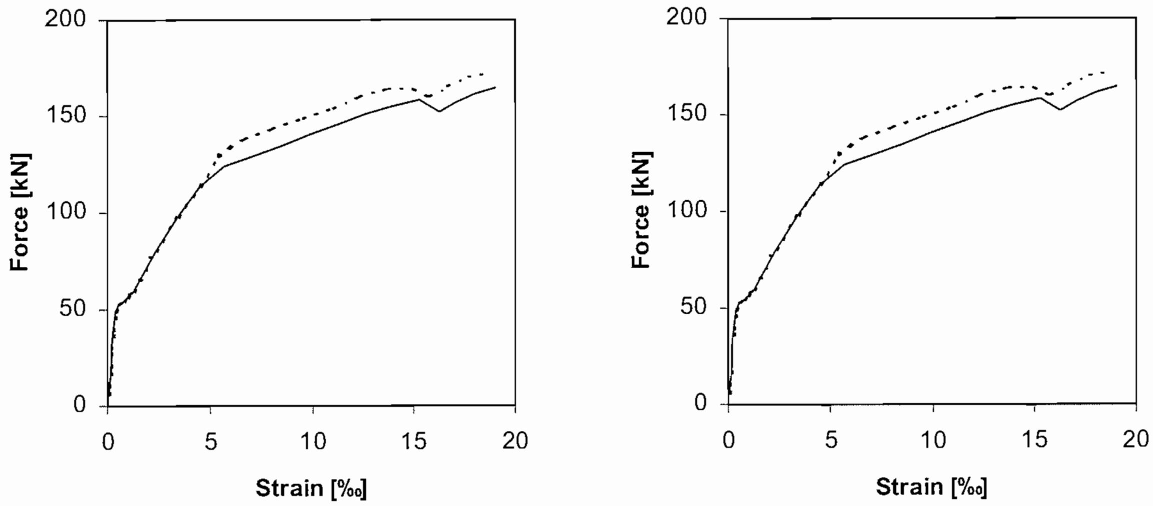


*Figure 7.4 Beam no.3 - Deflection and strain at the extreme tensile fibre at midspan*

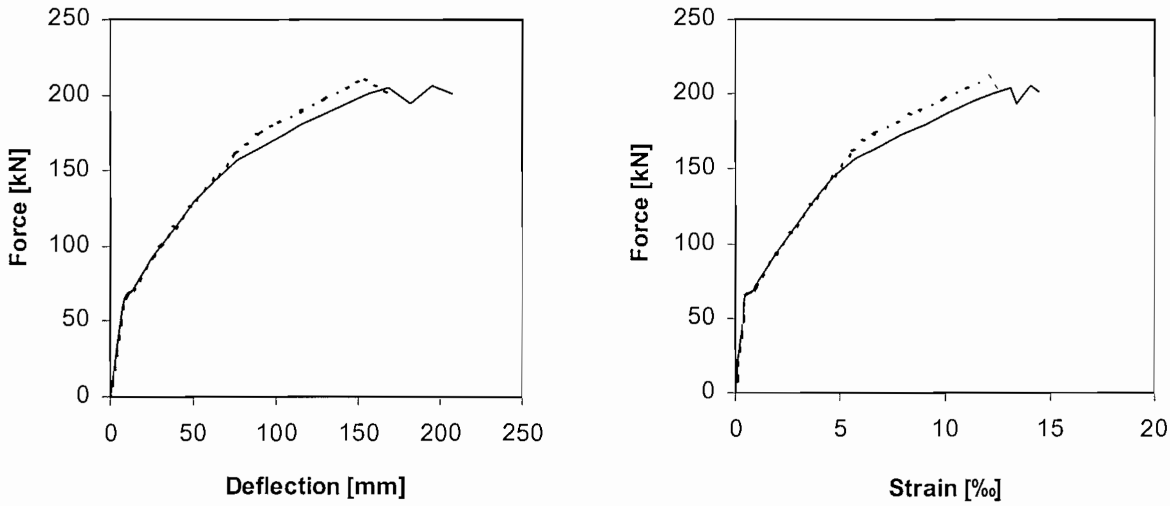


*Figure 7.5 Beam no.4 - Deflection and strain at the extreme tensile fibre at midspan*

# 8 EVALUATION OF THE RESULTS

## 8.1 Efficiency of strengthening with CFRP (test results)

Strengthening the beams with longitudinal Carbon Fiber Reinforced Polymer plates in quantities of 108 mm$^2$ and 216 mm$^2$ resulted in 28 and 37 percent increase in the net flexural moment capacity respectively. Figure 8.1 shows the moment - strain relations for beam no.1, 3 and 4 with respect to the strain in the tension zone (5 cm from the bottom) at midspan. It is clear from the diagram that further strengthening would not be effective.

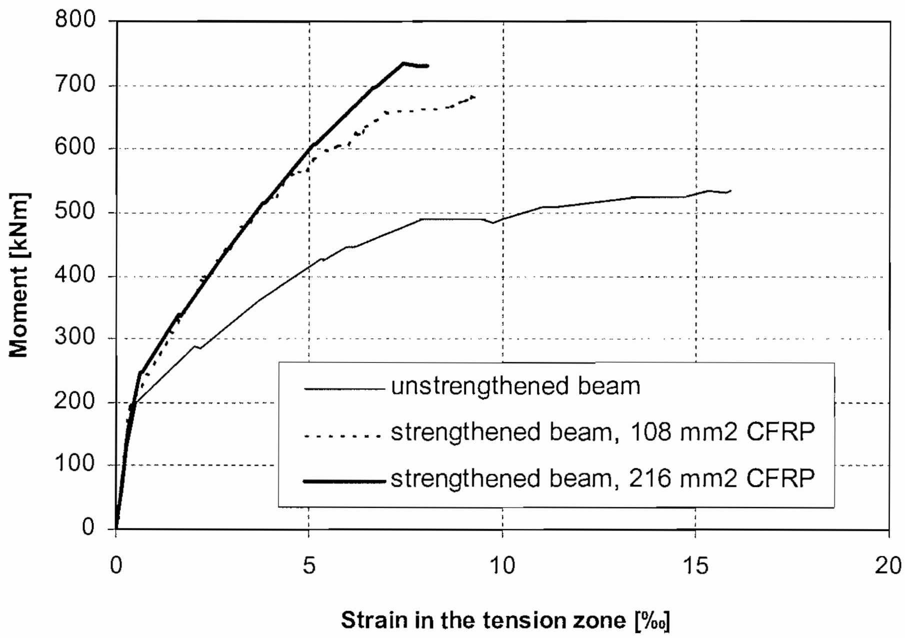


*Figure 8.1 Test results at beam no.1, no.3 and no.4*

***Note:** Due to the different span length in the three cases and so the different moment from the selfweight, the initial stress and strain state were slightly different. This small difference, however, was eliminated by adjusting the second and third curves such way that they were moved horizontally toward the first curve until its starting point (zero curvature) coincided with the starting point of the first curve. In practical terms this means that a small part of the point loads – in case of the beam no.3 and no.4 – were considered as pre-loading in order to achieve an almost equivalent stress and strain state at midspan as at beam no.1.*

## 8.2 Measurement vs. NS 3473

Table 8.1 compares the calculated and the measured ultimate capacity of the beams. At the test results only the ones which were relevant to the failure mode are presented and the corresponding calculated values are highlighted.

*Table 8.1 Total moment and shear capacity*

| | NS 3473 | | test results | | difference | |
|---|---|---|---|---|---|---|
| | M | V | M | V | M | V |
| | *kNm* | *kN* | *kNm* | *kN* | *%* | *%* |
| Beam no.1 | **487** | 134 | 597 | | 23 | |
| Beam no.2 | 487 | **159** | | 324 | | 104 |
| Beam no.3 | **679** | 131 | 750 | | 10 | |
| Beam no.4 | **798** | **133** | 801* | 214 | 0 | 61 |

*The failure was a shear failure but load-deflection and load-strain diagrams indicated that the maximum moment capacity was reached.

## 8.3 Measurement vs. finite element analysis

Test and analysis results are compared in Figure 8.2-5.

***Note:*** *In case of beam no.1 and no.2 the force – deflection curves belonging to testing are terminated without failure. See Section 5 for details.*

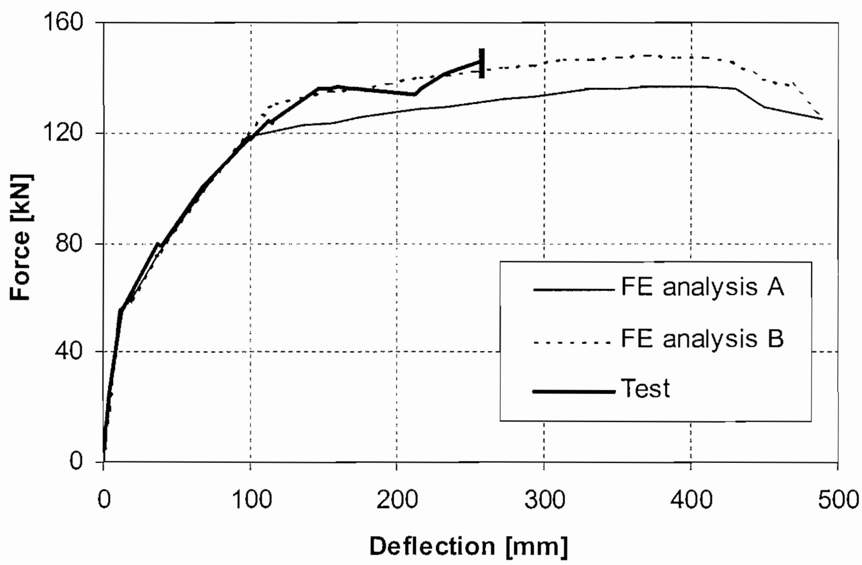


*Figure 8.2 Beam no.1 - Deflection at midspan*

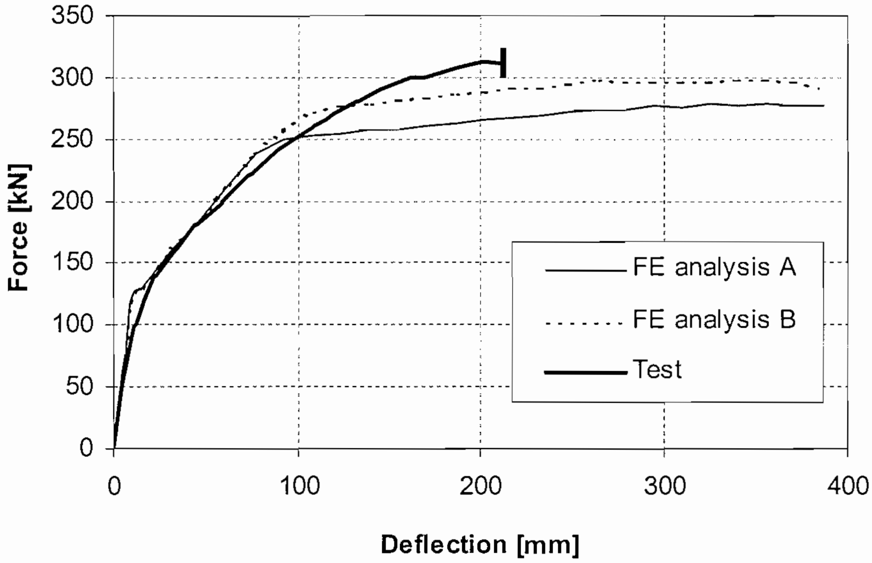


*Figure 8.3 Beam no.2 - Deflection at midspan*

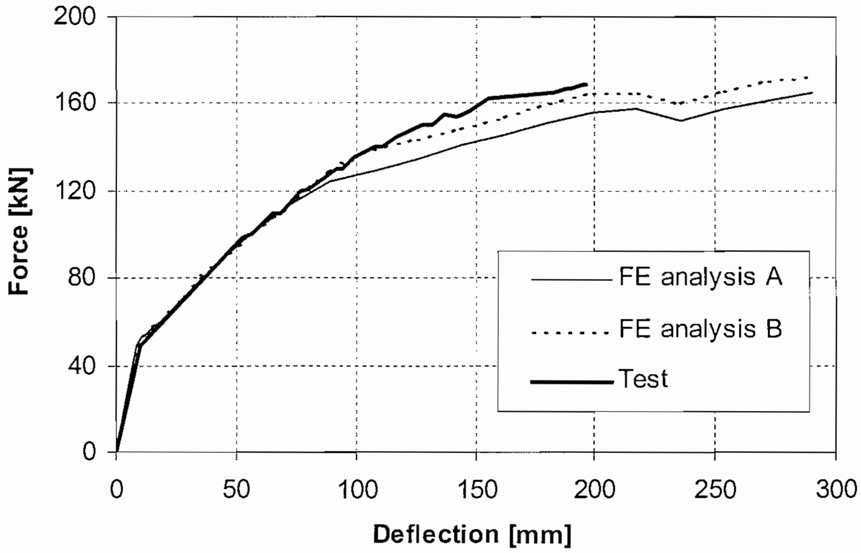


*Figure 8.4 Beam no.3 - Deflection at midspan*

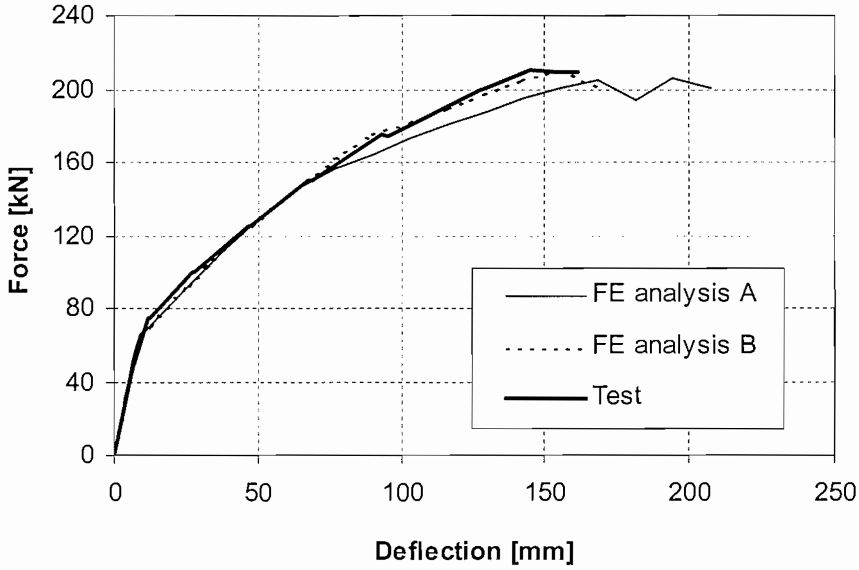


*Figure 8.5 Beam no.4 - Deflection at midspan*

# 9 CONCLUSIONS

Strengthening the beams with longitudinal Carbon Fiber Reinforced Polymer plates in quantities of 108 $mm^2$ and 216 $mm^2$ resulted in 28 and 37 percent increase respectively in net flexural moment capacity.

The ultimate flexural moment and shear capacity of the original (un-strengthened) beams have proved to be 23 and 104 % higher at testing than those were estimated according to the Norwegian Standard NS 3473.

Strengthening old concrete beams with Carbon Fiber Reinforced Polymer plates has proved to be an easy and effective solution.

The total strain based crack model which is based on the Modified Compression Field Theory has proved to be very well suited for modeling cracking of concrete with the smeared cracking approach. The model was found numerically stable which is often not the case with other cracking models.

The finite element models have underestimated the capacity of the beams by up to 11 %.

## References


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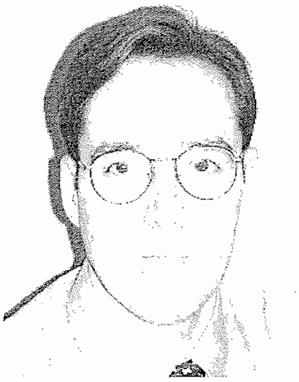
# Strengthening Prestressed Concrete Beams with Carbon Fiber Reinforced Polymer Plates



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**Abstract**

Application of Carbon Fiber Reinforced Polymer plates for strengthening concrete beams has been investigated. The investigation consisted of full scale testing on four prestressed concrete T elements and finite element modeling using an advanced smeared crack model based on the Modified Compression Field Theory. The technology has proved effective; substantial excess capacity were reached with a relatively low-cost solution. Finite element models have given fairly accurate prediction with respect to the flexural moment and shear capacity.

**Keywords**: building technology, prestressed concrete, full scale test, strengthening, Carbon Fiber Reinforced Polymer, finite element analysis

## 1 INTRODUCTION

Strengthening old concrete structural elements with Carbon Fiber Reinforced Polymer (CFRP) plates has become a popular alternative to traditional techniques such as installing steel plates or external prestressing cables. They are relatively cheap, easy to handle and work with, effective and not subjected to corrosion.

When Isakveien Bridge in Oslo was demolished after 35 years in service two prefabricated prestressed concrete DT elements with in-situ concrete top-layer were preserved for testing. The two DT elements were cut half lengthwise, so eventually four T elements were obtained. Two beams of the four were tested to determine their actual flexural moment and shear capacity for reference. The other two beams were strengthened with longitudinal Carbon Fiber Reinforced Polymer plates in two different quantities. The objective with the test series was to investigate the technology for future application mainly in the field of strengthening bridges similar to Isakveien Bridge.

The investigation has been carried out at the Norwegian University of Science and Technology, Department of Structural Engineering [1]. The clients were the Norwegian Public Road Administration (NPRA) and Sika Norway, the latter providing the necessary technology and material for the CFRP strengthening.

As part of the investigation finite element analysis has been carried out with the general purpose finite element program system DIANA. The numerical modeling was based on a total strain based crack model which was developed at TNO Building and Construction Research [2] along the lines of the Modified Compression Field Theory originally proposed by Vecchio and Collins [3]. The objective with the finite element analysis was to investigate and to test this crack model and to provide reference data for the tests.

The flexural moment and shear capacity were also calculated according to the Norwegian Standard, NS 3473 to obtain preliminary reference data for both the tests and the finite element modeling with respect to the ultimate capacity.

## 2 DESCRIPTION OF BEAMS

### 2.1 Original T elements

The beams were 11.20 meter long with a cross-section shown in Figure 1. The design concrete grade was C55 (B600) for the prefabricated element and C25 (B300) for the in-situ top-layer. The actual concrete strength was determined in the laboratory on six samples taken from the prefabricated element and six samples taken from the in-situ concrete. The mean compressive cylinder strength was 77 MPa and 55 MPa respectively.

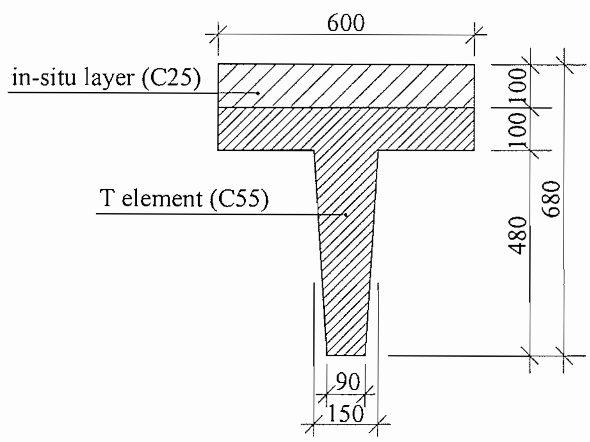


*Figure 1 – Dimensions of the cross-section*

The prefabricated element contains 54 $\phi 4$ St 1600/1800 prestressing wires, 40 in the tension zone and 14 distributed in the flange. 12.6 kN initial prestressing force was applied in each wire which means 680 kN axial compression force in total with an eccentricity of 93 mm with respect to the cross-section area of the prefabricated element only. The web contains $\phi 8$ stirrups with 300 mm spacing along the entire length.

Also the bond between the prefabricated element and the in-situ concrete layer was tested to ensure that the shear stress over the contact surface will not exceed the failure stress.

### 2.2 Strengthening the T elements with Carbon Fiber Reinforced Polymer plates

Two beams (beam no.3 and no.4) were strengthened with longitudinal Carbon Fiber Reinforced Polymer plates in two different quantities, 108 mm$^2$ and 216 mm$^2$. The detailed arrangements are shown in Figure 2. The type of the CFRP plates were SIKA CarboDur S with the following material properties: modulus of elasticity, $E = 165000$ MPa, mean tensile failure strength, $f_{tm} = 3050$ MPa and strain at failure, $\varepsilon_u = 1.85$ ‰. The plates were attached to the concrete element with Sikadur epoxy-based adhesive.

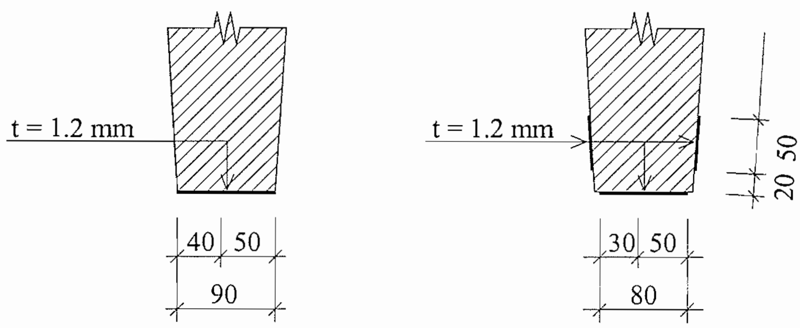


*Figure 2 – Arrangement of the CFRP plates (beam no.3 and no.4)*

## 3 TEST ARRANGEMENT AND INSTRUMENTATION

The beams were loaded symmetrically with two concentrated point loads. The span length and the shear span varied from test to test (Table 1) in accordance with the intended failure mode which was flexural moment failure for beam no.1 and no.3 and shear failure for beam no.2 and no.4.

*Table 1 – Arrangement of the tests*

| *Test* | *Type* | *Span length [m]* | *Shear span [m]* |
|---|---|---|---|
| *No.1* | *Original beam / assumed moment failure* | *10.80* | *3.60* |
| *No.2* | *Original beam / assumed shear failure* | *9.20* | *1.80* |
| *No.3* | *CFRP strengthened, 108 mm$^2$* | *10.20* | *4.10* |
| *No.4* | *CFRP strengthened, 216 mm$^2$* | *9.20* | *3.60* |

The beams were instrumented with strain gauges (PL60) in the compression zone and linear variable displacement transducers (LVDTs) on a basis length of 200 mm in the tension zone. Deflection measurements were carried out at mid-span and 1.8 meter from mid-span on both sides.

## 4 CAPACITY ACCORDING TO THE NORWEGIAN STANDARD

The flexural moment and shear capacity were determined according to the Norwegian Standard (NS 3473:1998, Concrete structures – Design rules) [4] under a common set of assumptions. In addition perfect bond was assumed between the CFRP plate and the concrete. It should be, however, mentioned that this assumption may be an oversimplification under certain circumstances and the problem requires further investigation in general.

Table 2 shows the calculated moment and shear capacities where the net values are obtained by subtracting the moments and shear forces resulting from the selfweight. The calculation was carried out with the material and load coefficients being set to one.

*Table 2 – Moment and shear capacity according to NS 3473*

| *Test* | *Moment total kNm* | *Moment net kNm* | *Shear total kN* | *Shear net kN* |
|---|---|---|---|---|
| *No.1* | *487* | *422* | *133* | *125* |
| *No.2* | *487* | *442* | *157* | *145* |
| *No.3* | *679* | *622* | *131* | *127* |
| *No.4* | *798* | *753* | *133* | *129* |

## 5 TEST RESULTS

The measured variables in Figure 3, 4, 5 and 7 are deflection at mid-span, tensile strain at 50 mm from the bottom of the T element and compressive strain at the extreme compression fiber respectively.

Although test no.1 was terminated without failure due to the large deflection and the limitations of the test rig, it was clear from the structural responses that the failure would have occurred as it was assumed, i.e. flexural moment failure due to the crushing of the concrete in the compression zone. The load-deformation diagrams also suggested that the maximum moment capacity was reached (Figure 3).

Test no.2 which was intended to provide the reference shear capacity did not reach failure either over the normal course of loading. The structural response, however, suggested that the maximal load level was reached and failure would have occurred by any further loading (Figure 4). At the last load level the beam was already severely damaged by wide flexural-shear cracks.

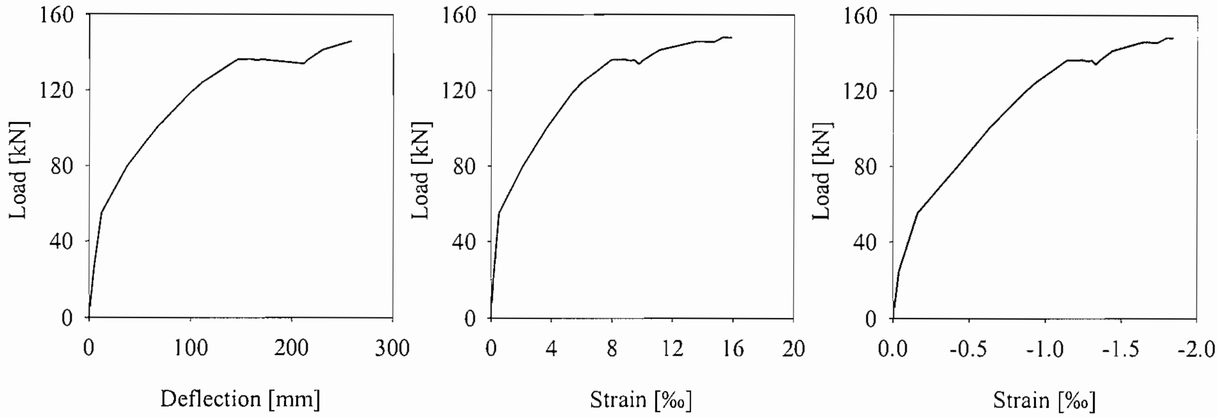


*Figure 3 – Deflection and strain diagrams at test no.1*

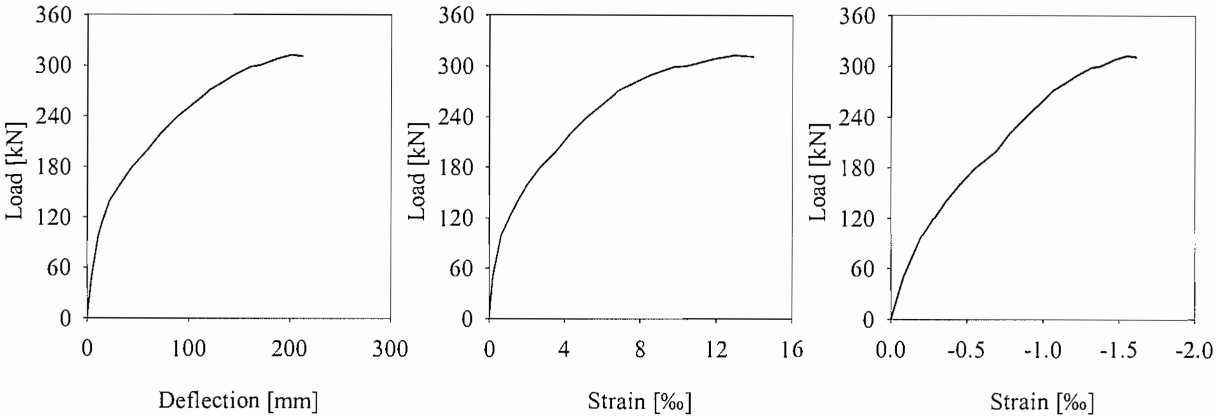


*Figure 4 – Deflection and strain diagrams at test no.2*

The failure at beam no.3 occurred such way that the CFRP plate split apart from the concrete along a horizontal surface starting from a wide flexural-shear crack. This type of failure may be a concern for this technology. It may be explained with the combination of high shear stresses, high normal stress concentration in the plate at the crack opening and coexisting transverse shear stresses due to vertical movement at the crack opening. Although the failure happened in a state with relatively large compressive and tensile strains, as shown in Figure 5, the failure itself was very brittle having a rather explosive character.

The failure at beam no.4 was a typical shear failure. The structural responses, as shown in Figure 6, however suggested that also the maximum moment capacity was almost reached.

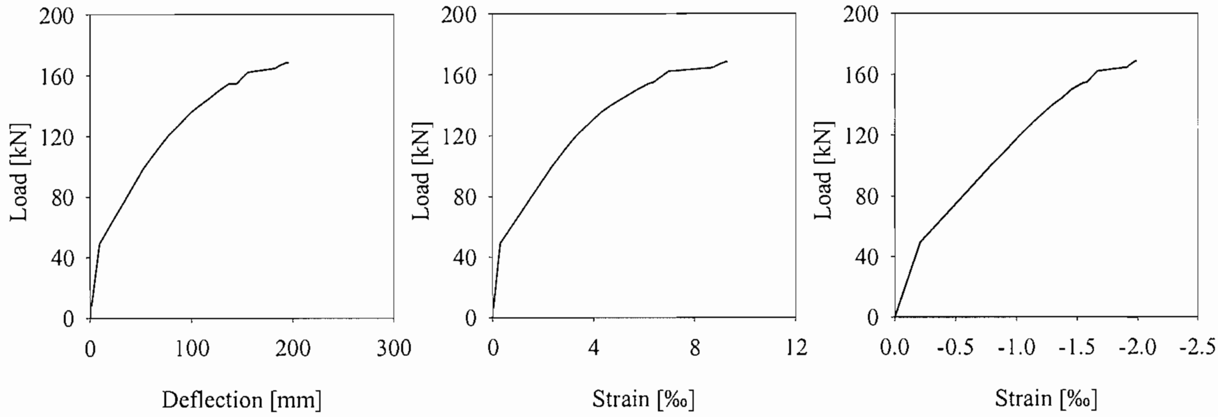


*Figure 5 – Deflection and strain diagrams at test no.3*

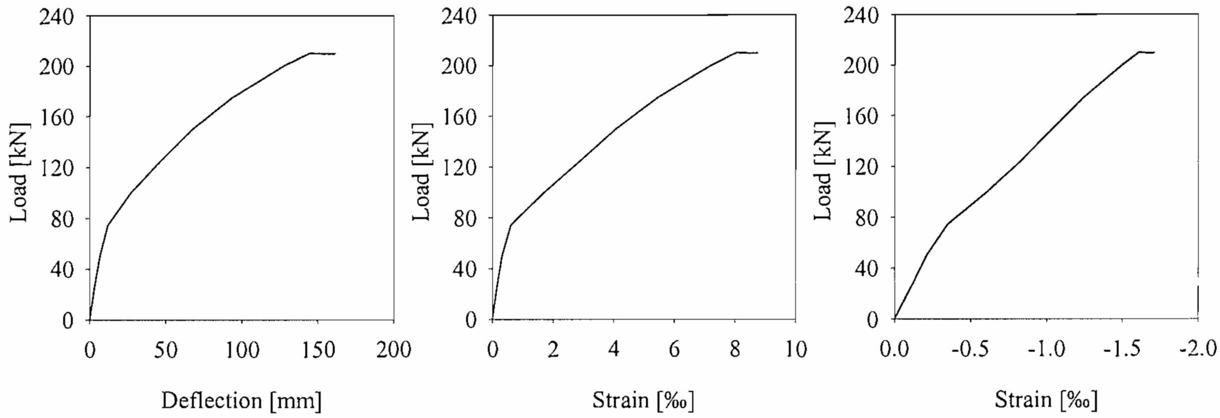


*Figure 6 – Deflection and strain diagrams at test no.4*

# 6 FINITE ELEMENT MODELING

## 6.1 General

The finite element analysis was carried out with the general purpose finite element program system DIANA. DIANA is based on the displacement method and highly suitable for modeling concrete structures due to the wide range of concrete material models and advanced numerical tools. The governing non-linear phenomena in the ultimate limit state are cracking and crushing of the concrete and plasticity of the prestressing steel. For concrete cracking several approaches exist within the smeared crack concept. In our analysis a total strain based crack model was used which was developed along the lines of the Modified Compression Field Theory at TNO Building and Construction Research [2, 3].

The constitutive model describes the tensile and compressive behavior with one stress-strain relationship where the stress is directly related to the total strain. This concept is similar to hypo-elasticity where the loading and unloading are following the same stress-strain path. In the DIANA implementation, however, the behavior in unloading is modeled with secant unloading.

In our analysis the rotating crack concept was chosen where the stress-strain relationships are evaluated in the principal directions of the strain vector.

Since the objective of the numerical investigation was to the simulate the real structural behavior of the beams, material parameters were taken by their mean values instead of the characteristic values. These parameters were available for the concrete and the Carbon Fiber Reinforced Polymer, but not for the prestressing steel. Therefore it was decided that two analyses should be performed for each case. In analysis 'A' material properties of the prestressing steel were taken by its known characteristic values while in analysis 'B' those were taken by roughly estimated mean values which were assumed to be 10 percent higher than the characteristic values with respect to the yield stress, $f_{p0.2}$ and the failure tensile strength, $f_{p,max}$. Such an arbitrarily chosen value may be questionable but it is the authors' opinion that it is reasonable and the results can be easily adjusted for other values by scaling.

## 6.2 Concrete in compression

In cracked concrete the principal compressive stress is a function not only of the principal compressive strain, $\varepsilon_2$, but also of the coexisting principal tensile strain, $\varepsilon_1$. If the concrete is cracked in the lateral direction, the compressive stresses are reduced. The reduction is introduced through reducing the peak stress value according to Equation 1 [5]. It is assumed that the stress-strain curve is fully determined with its peak stress value and the corresponding strain value for a given base curve (Figure 7).

$$f_{2,max} = f_c \cdot \frac{1}{1+K_c} \leq f_c \tag{1}$$

where

$$K_c = 0.27 \cdot \left( -\frac{\varepsilon_1}{\varepsilon_{co}} - 0.37 \right) \tag{2}$$

and

$f_{2,max}$ peak stress value in compression in cracked concrete,
$f_c$ uniaxial compressive strength,
$\varepsilon_{co}$ compressive strain at maximum compressive stress (may be taken as -0.002).

As a base curve in compression, the serpentine curve was chosen. The serpentine curve was first used on concrete by Popovics in 1973, and later adapted to high strength concrete by Thorenfeldt et al. [6]:

$$\sigma_2 = -f_{2,max} \frac{\varepsilon_2}{\varepsilon_{co}} n \left( n - 1 + \left( \frac{\varepsilon_2}{\varepsilon_{co}} \right)^{nk} \right)^{-1} \tag{3}$$

where

$n, k$ model parameters which may be estimated from

$$n = 0.8 + \frac{f_c}{17} \qquad k = \begin{cases} 1 & \text{if} \quad 0 > \varepsilon_2 > \varepsilon_p \\ 0.67 + \frac{f_c}{62} & \text{if} \quad \varepsilon_2 \le \varepsilon_p \end{cases} \qquad (4, 5)$$

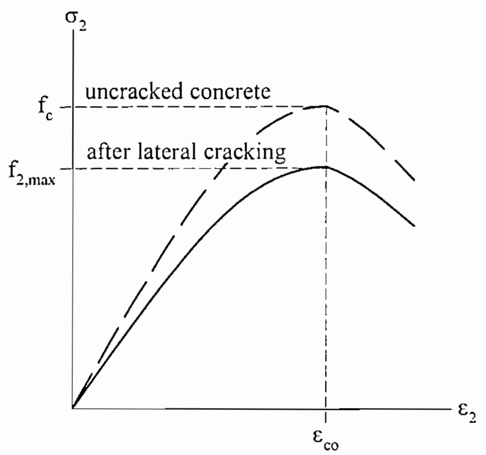


*Figure 7 – Stress-strain curve for cracked concrete in compression*

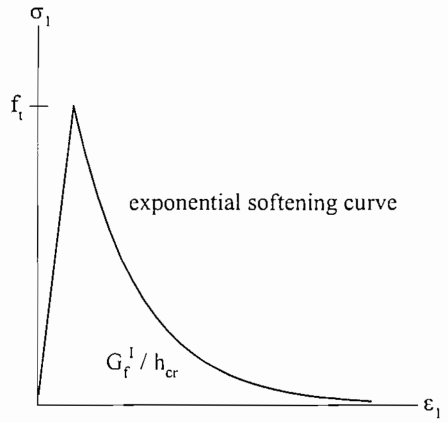


*Figure 8 – Stress-strain curve in tension*

## 6.3 Concrete in tension

The advancement in the Modified Compression Field Theory is that the tensile stresses in the post-cracking state are not neglected. The principal tensile stress is evidently zero at the crack openings but not zero between the cracks. Accounting for their contribution to the load carrying capacity brings the results closer to the real structural behavior thus furnishing a less conservative model for cracked concrete.

The tensile stresses in the post-cracking state are taken into account with an average value which is directly related to the average principal tensile strain. The shape of the softening branch of the stress-strain curve (Figure 8) was chosen to be exponential where the parameters are determined from the tensile strength, $f_t$, the fracture energy, $G_f^I$, and the estimated numerical crack bandwidth, $h_{cr}$.

## 6.4 Finite element model

A two dimensional finite element mesh consisting of plane stress elements was used under the assumption that the stress distribution in the flange is uniform in the horizontal direction of the cross-section. Such an idealization was justified due to the low width-height ratio of the flange. Since the structure and loading scheme are symmetrical, it was sufficient to model only the left half of the beam. Eight-node plane stress elements with 3×2 Gauß integration scheme were used.

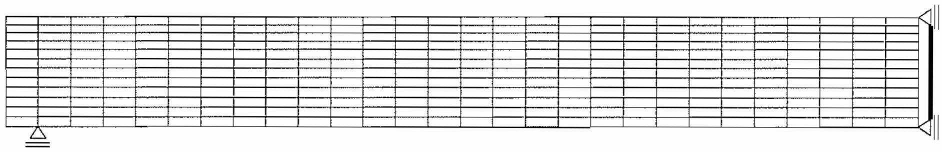

*Figure 9 – Finite element mesh*

### 6.5 Non-linear phased analysis

Phased analysis is an ideal tool to simulate the structural behavior where certain structural elements are added to the already existing ones in a latter stage when the existing structure is already subjected to loading. Obviously a newly added element is stress-free at the moment of installation under normal circumstances. In our case this concerns the in-situ concrete layer and most importantly the CFRP plate. In phased structural analysis the entire analysis is divided into separate calculation phases where each phase is a complete finite element calculation and represents a certain stage in the structural history. The output from previous phases in existing elements contributes automatically to the initial conditions in the subsequent calculation phase. Newly activated elements are evidently free from stresses and strains at the moment of activation unless initial conditions are specified.

The non-linear solution technique was a combined incremental-iterative procedure. The incremental part was the spherical path arc-length method while the iterative part was the linear stiffness method [2]. This solution technique has worked well with the smeared crack model and has proved stable.

## 7 EVALUATION OF THE RESULTS

### 7.1 Efficiency of the CFRP strengthening

Strengthening the beams with longitudinal Carbon Fiber Reinforced Polymer plates in quantities of 108 $mm^2$ and 216 $mm^2$ resulted in 28 and 37 percent increase in the net flexural moment capacity respectively. Figure 10 shows the moment-strain curves for beams no.1, 3 and 4 with respect to the tensile strain measured by the LVDTs at 50 mm from the bottom at mid-span.

Under the current configuration, further strengthening with respect to the bending moment would not make any significant change in the overall capacity of the beams without strengthening them for shear as well.

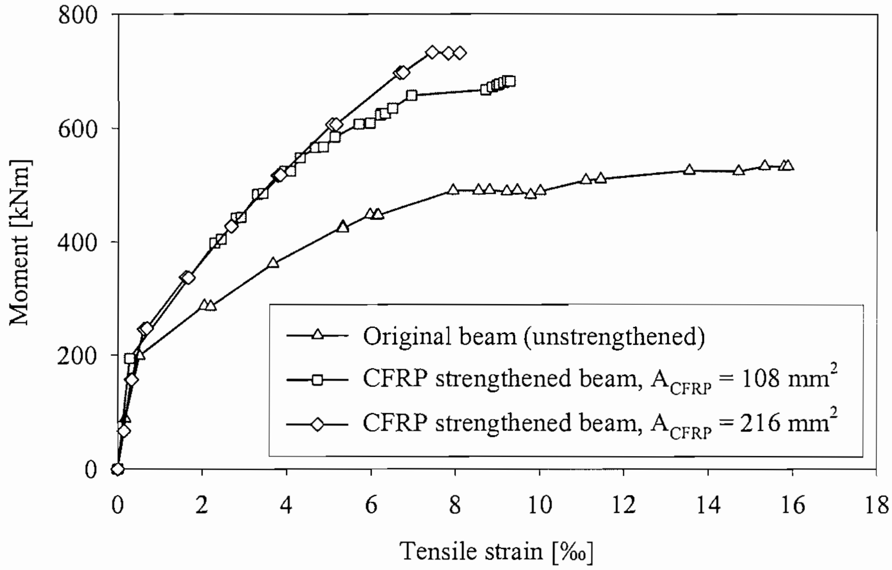


*Figure 10 – Moment versus tensile strain at 50 mm from the bottom of the beam*

## 7.2 Overview of the results

Figure 11 gives an overview of the calculated and measured moment and shear capacities while Table 3 and Table 4 shows the differences in percentage as compared to the values calculated according to the Norwegian Standard.

Finite element analyses have given a fairly accurate prediction with respect to the maximum flexural moment and maximum shear force. Their advantage over simplified design methods is obvious, particularly in the case of shear design where the conservative nature of the simplified shear design method is evident.

*Table 3 – Differences in percentage in the moment capacity as compared to NS 3473*

| *Test* | *Finite Element Analysis* | | *Test* |
|---|---|---|---|
| | *A* | *B* | |
| *No.1* | *15.0* | *23.4* | *23.2* |
| *No.3* | *8.1* | *12.7* | *10.6* |
| *No.4* | *-1.5* | *0.8* | *0.3* |

*Table 4 – Differences in percentage in the shear capacity as compared to NS 3473*

| *Test* | *Finite Element Analysis* | | *Test* |
|---|---|---|---|
| | *A* | *B* | |
| *No.2* | *85.4* | *98.1* | *106.4* |
| *No.4* | *57.9* | *61.7* | *60.9* |

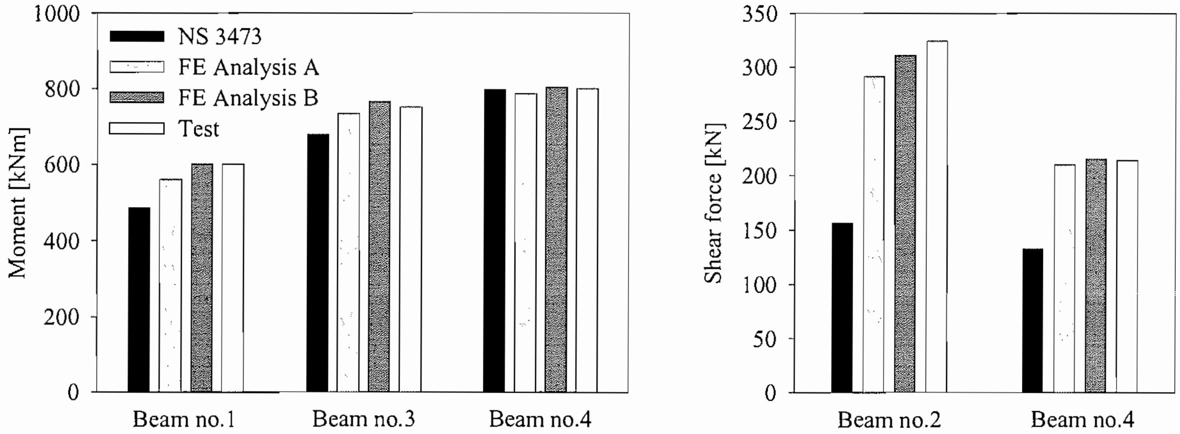


*Figure 11 – Calculated and measured moment and shear capacity*

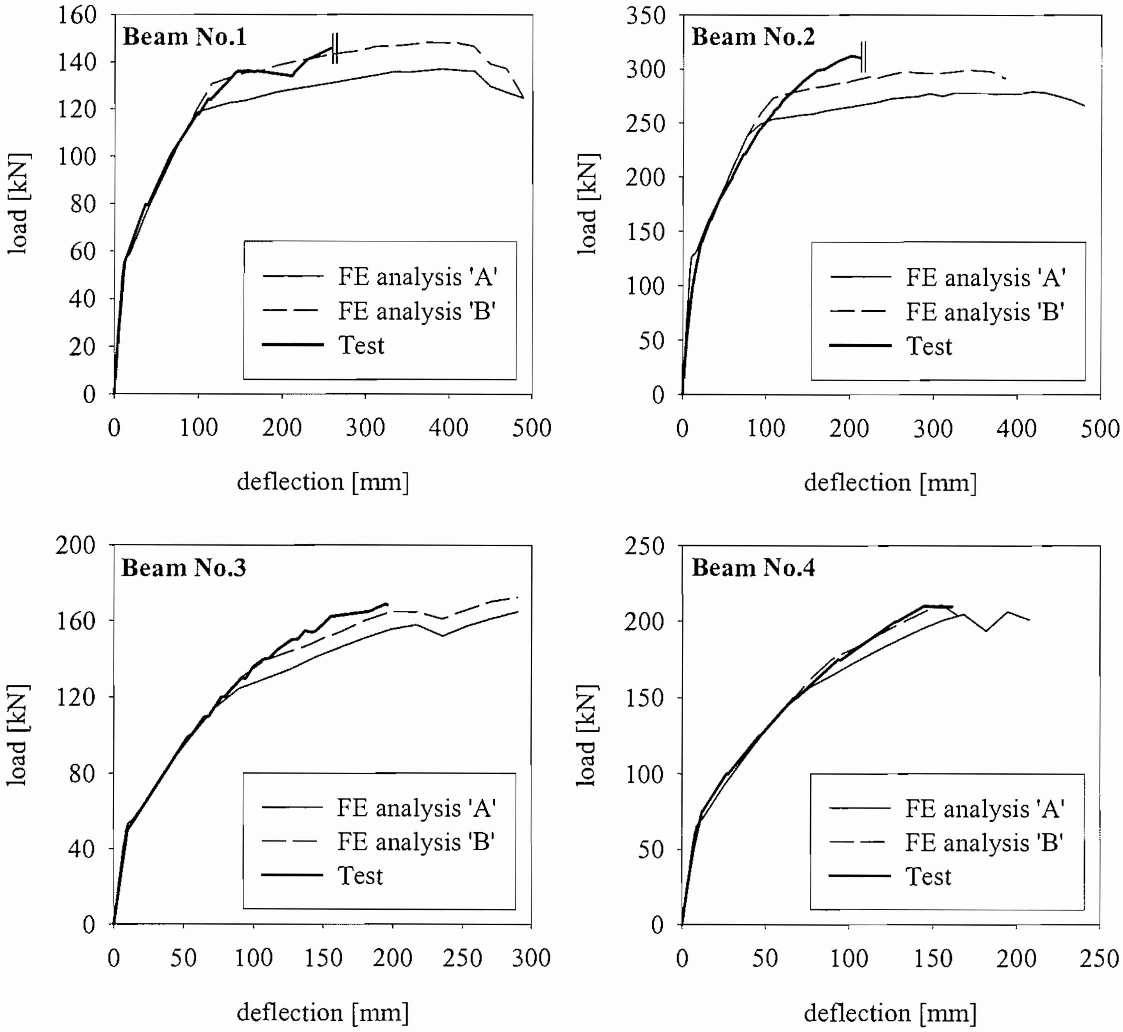


*Figure 12 – Calculated and measured load-deflection curves at mid-span*

### 7.3 Finite element analysis versus test

The finite element analyses underestimated the moment capacity by up to 7 percent and the shear capacity by up 10 percent. The same values are -2 percent (over-estimated) and 4 percent respectively when the assumed mean values of yield stress, $f_{p0.2}$ and the failure tensile strength, $f_{p,max}$ were applied. The load-deflection curves are shown in Figure 12.

## 8 CONCLUSIONS

Strengthening the beams with Carbon Fiber Reinforced Polymer plates has proved effective. The amount of 108 mm$^2$ and 216 mm$^2$ longitudinal SIKA CarboDur S plates resulted in 28 and 37 percent increase in the net flexural moment capacity respectively. The technology has several advantage over traditional technologies: the plates are easy to install and work with, the CFRP is not subjected to corrosion and the solution is cost-effective.

The ultimate flexural moment and shear capacity of the original (un-strengthened) beams were 23 and 106 percent higher at testing than calculated according to the Norwegian Standard, NS 3473.

The total strain based crack model has proved to be very well suited for modeling concrete cracking with the smeared crack approach. Reasonable agreement with test results was reached with respect to both the moment and the shear capacities. The model was found numerically stable.

**Acknowledgement**

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