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 was 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 (double-T cross-section) 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 the Dutch research organization 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 capacities 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 for the prefabricated element and C25 for the in-situ top-layer. In accordance with the Norwegian regulations the concrete grade refers to the characteristic cube strength. The actual concrete strength was determined in the laboratory on six cylinder samples taken from the prefabricated element and six cylinder samples taken from the in-situ concrete. The mean compressive cylinder strength was 77 MPa and 55 MPa, respectively.

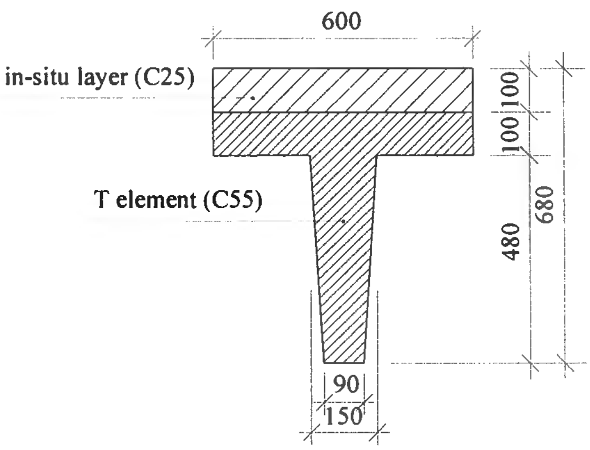

Figure 1 – Dimensions of the cross-section

The prefabricated element contains 54 ϕ4 St 1600/1800 prestressing wires, 40 in the tension zone and 14 distributed in the flange. A 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 ϕ8 mm stirrups with 300 mm spacing along the entire length.

Also the bond between the prefabricated element and the in-situ concrete layer was tested by simple pull-off tests to ensure that the shear stress over the contact surface would not exceed the failure stress. In both the simplified calculation and the finite element analysis perfect bond was assumed between the prefabricated element and the top-layer.

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.

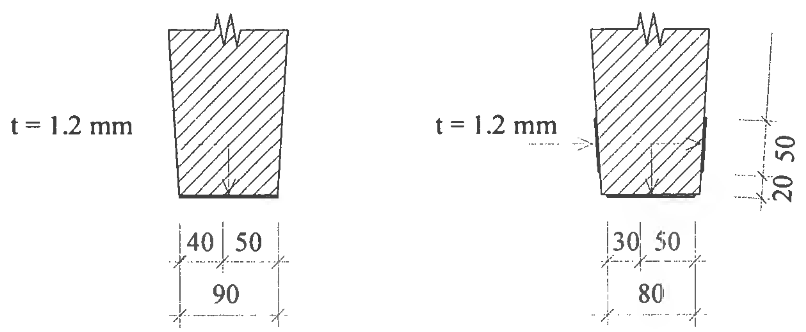

Figure 2 – Arrangement of the CFRP plates (beam no. 3 and no. 4)

The arrangement at the bottom is made up of two separate plates as SIKA's CFRP plates are manufactured in 50 mm wide stripes.

Figure 3 – Test no. 4

3 TEST ARRANGEMENT AND INSTRUMENTATION

The beams were loaded symmetrically with two concentrated point loads as it is illustrated in Figure 4. 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.

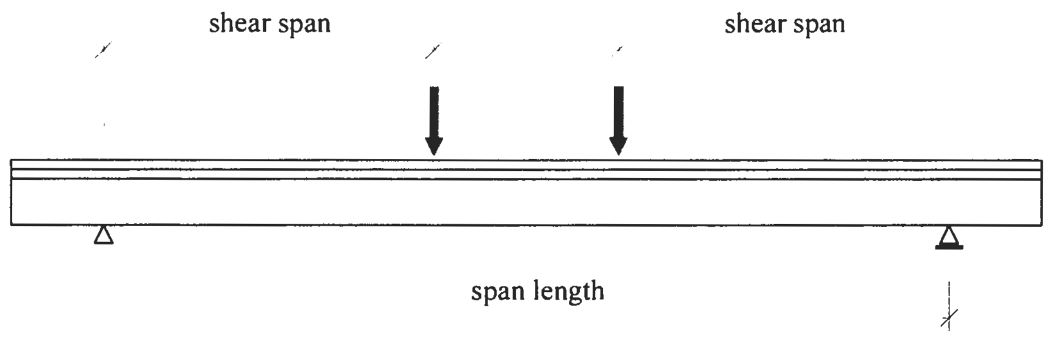

Figure 4 – Loading scheme (see Table 1 for actual values)

Table 1 – Test arrangements

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 capacities 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.

The Norwegian Standard does not contain specifications for CFRP. A perfectly linear stress-strain relationship was assumed when the moment capacity was calculated. As for the shear capacity, the longitudinal CFRP plates were not considered.

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 unity.

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 5, 6, 7 and 8 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 5).

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 6). At the last load level the beam was already severely damaged by wide flexural-shear cracks.

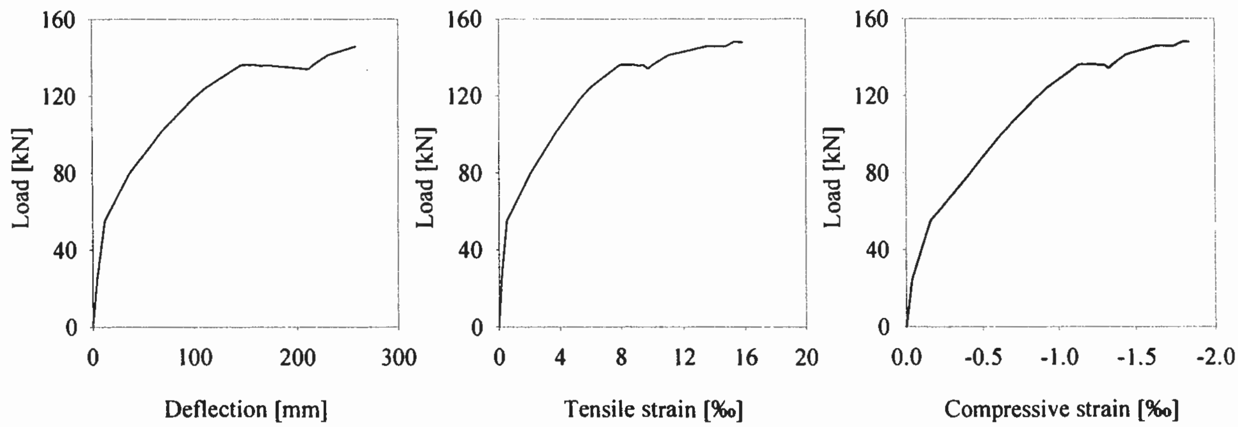

Figure 5 – Deflection and strain diagrams at mid-span at test no. 1

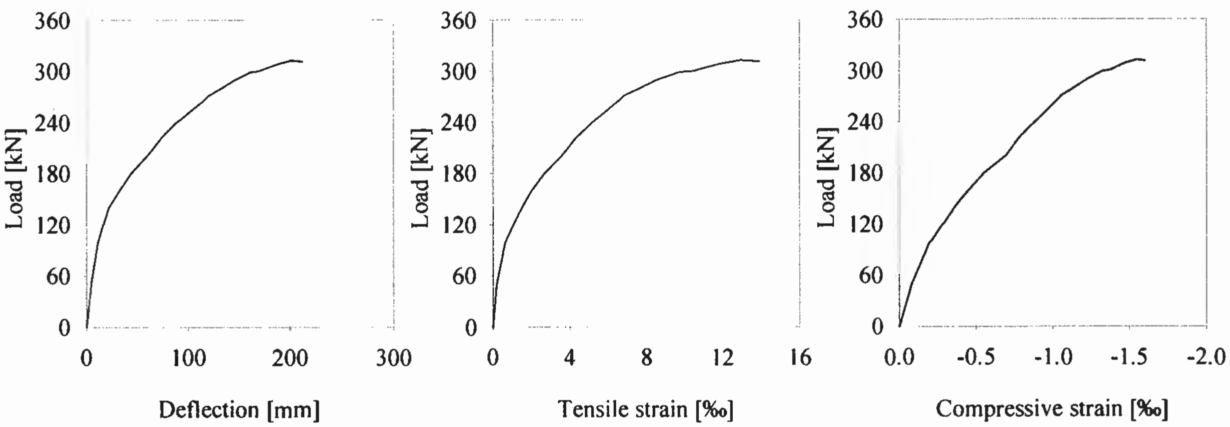

Figure 6 – Deflection and strain diagrams at mid-span at test no. 2

The failure at beam no. 3 occurred in such a 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 between the concrete and the CFRP plate, 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 7, 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 8, however suggested that also the maximum moment capacity was almost reached.

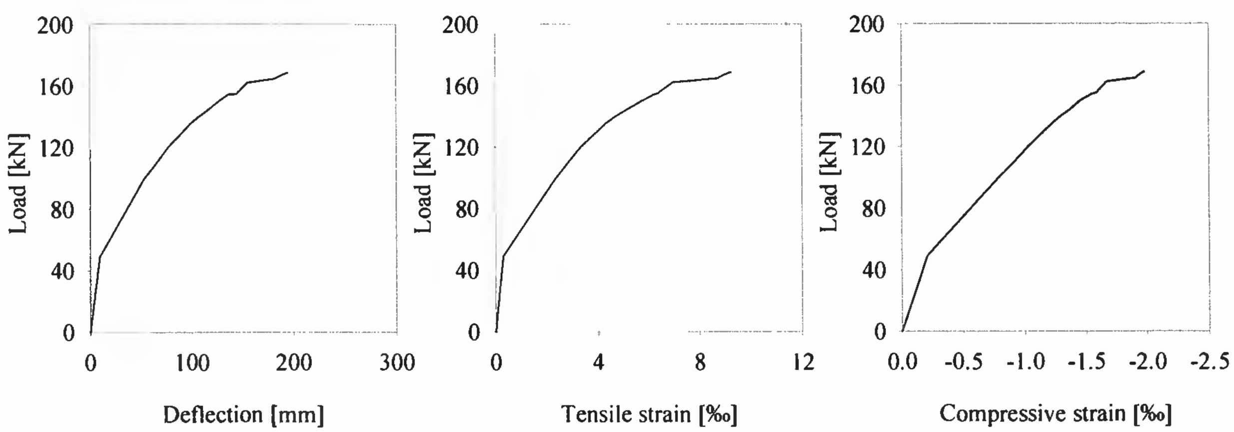

Figure 7 – Deflection and strain diagrams at mid-span at test no. 3

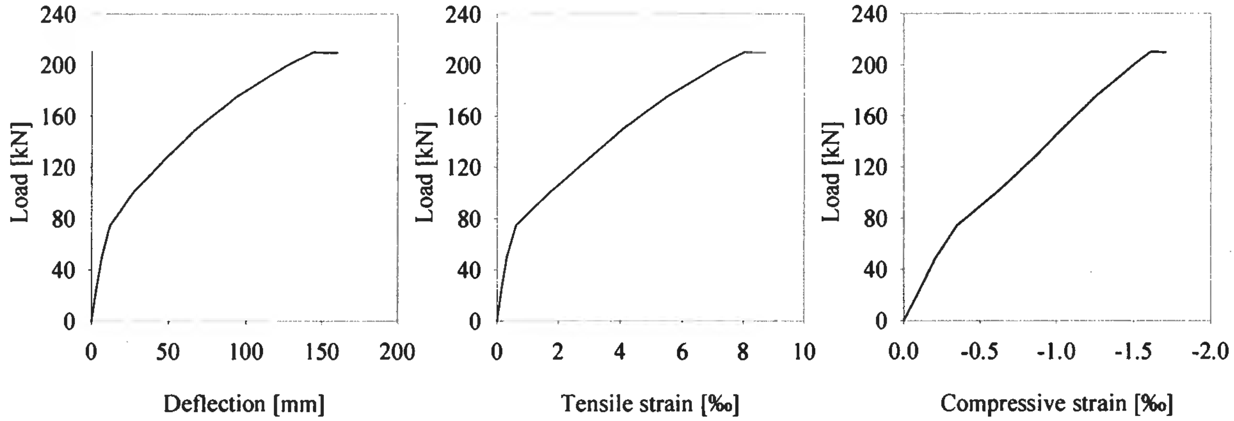

Figure 8 – Deflection and strain diagrams at mid-span at test no. 4

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, ε_2, but also of the coexisting principal tensile strain, ε_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 9).

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

where

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

and

$f_{2,max}$ peak stress value in compression in cracked concrete,
f_c uniaxial compressive strength,
ε_{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]:

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

where

n, k model parameters which may be estimated from

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

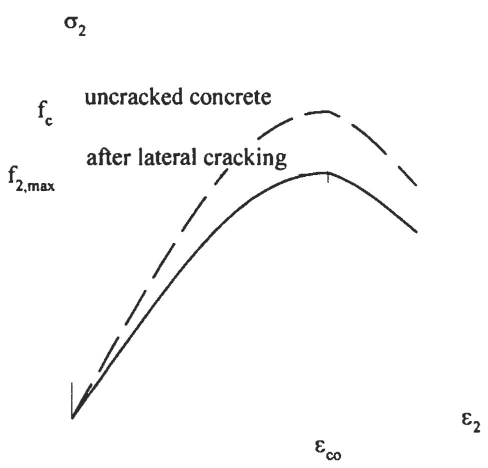

Figure 9 – Stress-strain curve for cracked concrete in compression

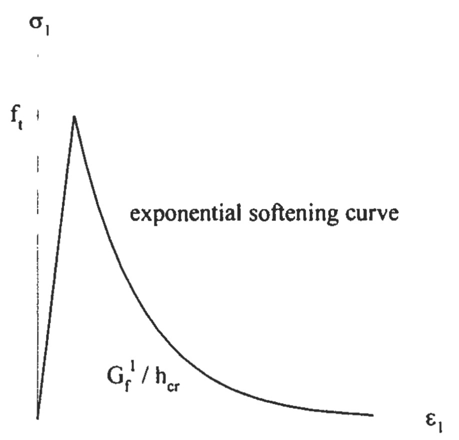

Figure 10 – Stress-strain curve in tension

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 10) 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.

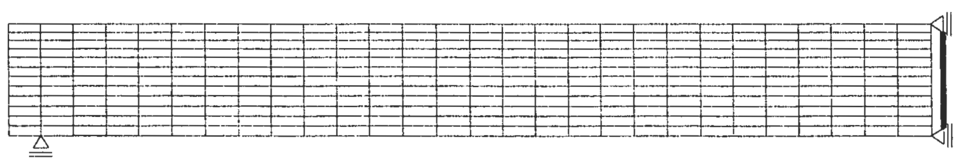

Figure 11 – Finite element mesh

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 12 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.

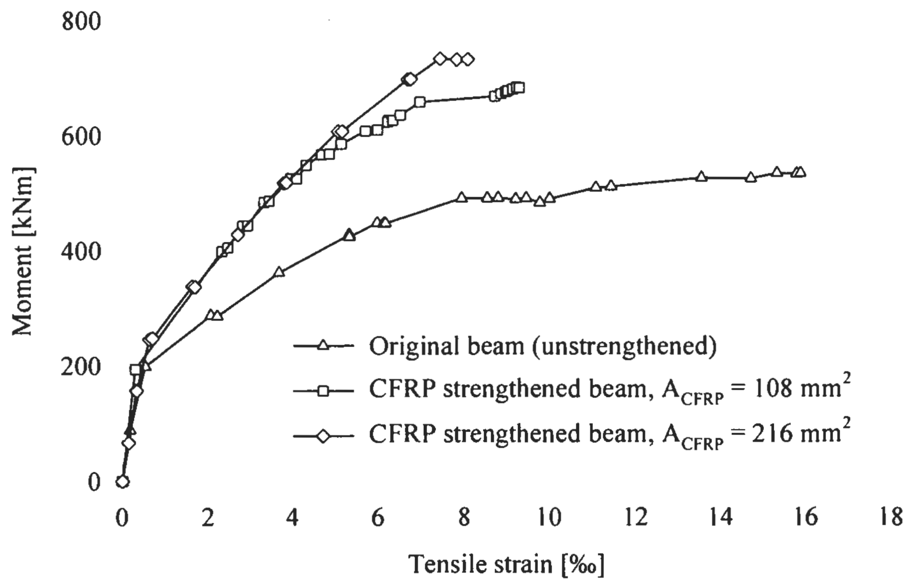

Figure 12 – Moment versus tensile strain at 50 mm from the bottom of the beam

7.2 Overview of the results

Figure 13 gives an overview of the calculated and measured moment and shear capacities while Table 3 and Table 4 show 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*

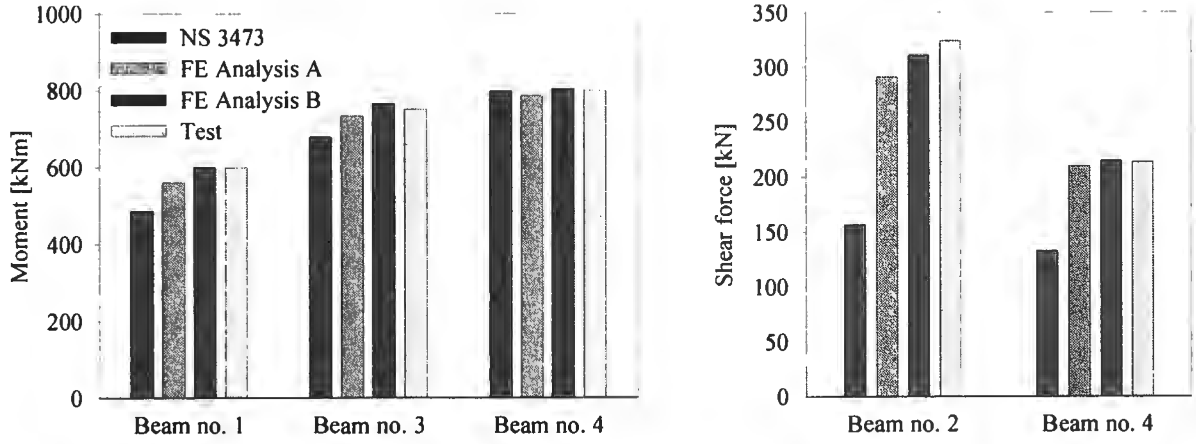

Figure 13 – Calculated and measured moment and shear capacity

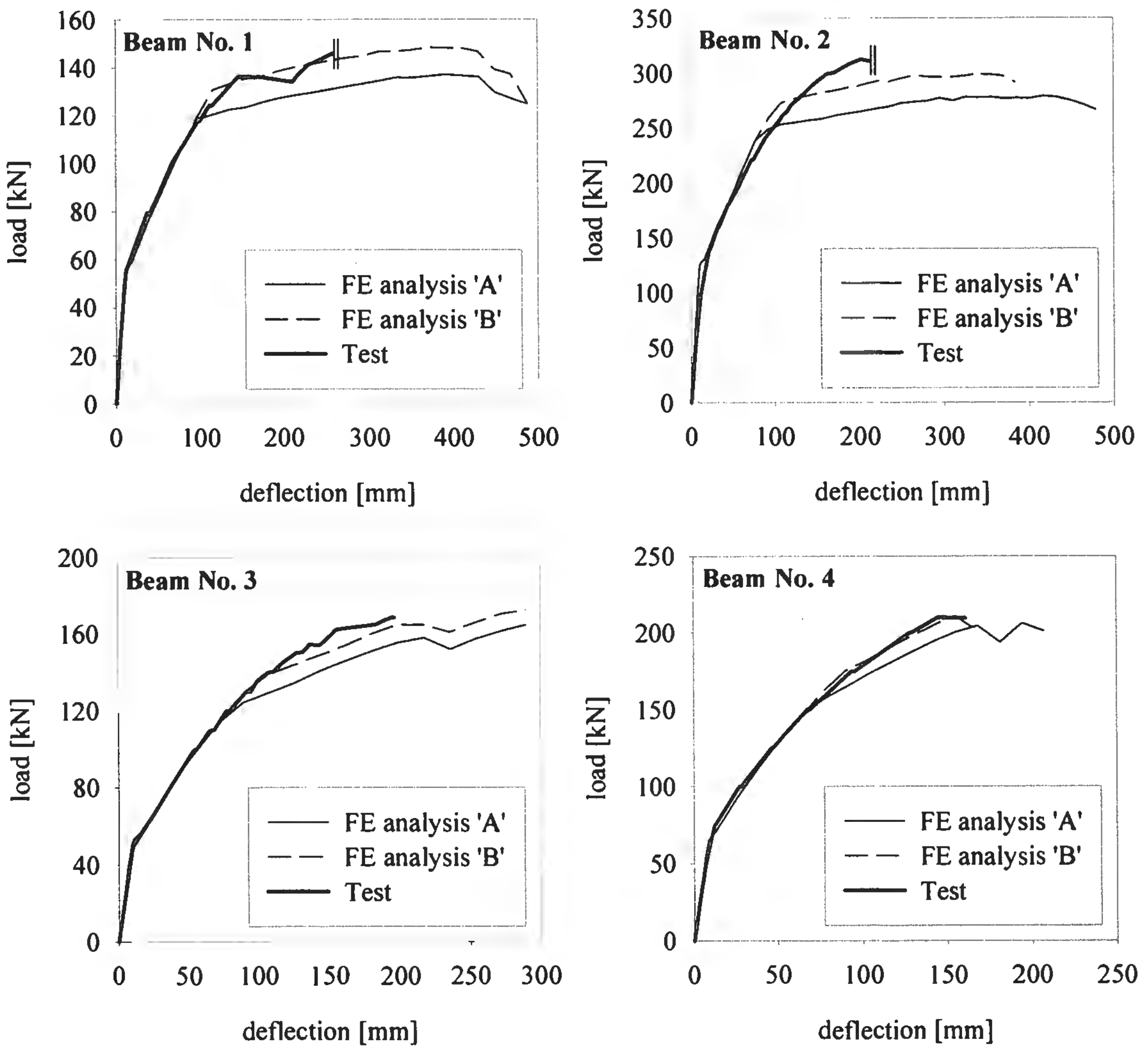

Figure 14 – Calculated and measured load-deflection curves at mid-span

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 the yield stress, $f_{p0.2}$ and the failure tensile strength, $f_{p,max}$ were applied. The load-deflection curves are shown in Figure 14.

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 advantages 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. On the other hand the possibility of brittle failure – as it was the case at test no. 3 – can be a concern.

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.

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