Abstract: A finite element (FE) model was calibrated using the data obtained from a full-scale test to failure of a 50 year old reinforced concrete (RC) railway bridge. The model was then used to assess the effectiveness of various strengthening schemes to increase the load-carrying capacity of the bridge. The bridge was a two-span continuous single-track trough bridge with a total length of 30 m, situated in Örnsköldsvik in northern Sweden. It was tested in situ as the bridge had been closed following the construction of a new section of the railway line. The test was planned to evaluate and calibrate models to predict the load-carrying capacity of the bridge and assess the strengthening schemes originally developed by the European research project called Sustainable bridges. The objective of the test was to investigate shear failure, rather than bending failure for which good calibrated models are already available. To that end, the bridge was strengthened in flexure before the test using near-surface mounted square section carbon fiber reinforced polymer (CFRP) bars. The ultimate failure mechanism turned into an interesting combination of bending, shear, torsion, and bond failures at an applied load of 11.7 MN (2,650 kips). A computer model was developed using specialized software to represent the response of the bridge during the test. It was calibrated using data from the test and then was used to calculate the actual capacity of the bridge in terms of train loading using the current Swedish load model which specifies a 330 kN (74 kips) axle weight. These calculations show that the unstrengthened bridge could sustain a load 4.7 times greater than the current load requirements (which is over six times the original design loading), whilst the strengthened bridge could sustain a load 6.5 times greater than currently required. Comparisons are also made with calculations using codes from Canada, Europe, and the United States. DOI: 10.1061/(ASCE)ST.1943-541X.0001116. This work is made available under the terms of the Creative Commons Attribution 4.0 International license, http://creativecommons.org/licenses/by/4.0/.

Author keywords: Bridge; Train load; Failure analysis; Ultimate load-carrying capacity; Shear; Near-surface mounted reinforcement (NSMR); Carbon fiber reinforced polymers (CFRP); Strengthening; Nonlinear finite element analysis (NLFEA); Structural safety and reliability.

Introduction

To meet future traffic demands, there is a constant need to make transport infrastructure more effective. For railway bridges, this can be achieved by increasing their load-carrying capacity to allow heavier and/or faster trains (where higher dynamic factors increase the static load effect on a bridge) and/or to increase their service life.

This paper describes the assessment of the load capacity (in terms of trains) using a calculation method that has been calibrated using a full-scale test to failure of a redundant concrete railway bridge in Örnsköldsvik (Övik) in northern Sweden. The method uses a nonlinear finite element analysis (NLFEA) with a detailed three-dimensional (3D) solid model using discrete reinforcement elements with both geometrical and material nonlinearities included. The model developed gave a detailed picture of deflections, stresses, and strains in the bridge with increasing train loads, which gave a better correlation between calculated results and the actual behavior of the bridge than the code methods currently in use. Whilst this is thought to be the first use of this method of assessment of the train load capacity of a bridge, the basics of the method are given in FIB (2008), and examples of advanced finite element modeling of large concrete structures are given in, e.g., Malm (2009), Schlune et al. (2009), Richard et al. (2010), and Bicanic et al. (2014).

Reports on full-scale tests to failure studying the behavior of bridges are rare. In this paper, some of them are discussed with failures of relevance to this study. A large-scale test (1:3) investigating the combined shear—torsion—bending of a curved concrete box-girder bridge in California was described by Scordelis et al. (1977, 1979). Failure was caused by concrete spall from a corner because of high shear and torsion stresses. Two full-scale tests of concrete bridges in Sweden were reported by Plos (1990, 1995) and...
Täljsten (1994). A slab frame bridge with a span of 21 m and a prestressed frame beam bridge with a span of 31 m were both tested to failure, which was because of shear and bending. For the slab beam, a brittle failure occurred when a new shear crack with a low slope emerged at a point load of 4.5 MN. For the prestressed bridge, the failure occurred when one of the beams punched through the end support wall at a point load of 8.45 MN. Neither of the failures were predicted by the available codes. In Switzerland, 89 bridges were studied during their demolition by Zwicky and Vogel (2000), and by Vogel and Bargähr (2006). Both sets of authors recommended improved methods for the assessment of real loads and for corrosion damage in prestressed concrete bridges. The tests were also simulated using solid elements by Pimentel et al. (2007) and using fiber elements capable of taking into account shear effects by Ferreira et al. (2012). In Switzerland, Fernandez Ruiz et al. (2007) carried out large-scale tests of the load-carrying capacity of box girder beams with thin webs including posttensioning tendons. The tendons decreased the load-carrying capacity of the compression struts necessary to transform the shear forces.

Details of the Bridge

The bridge studied was a continuous curved reinforced concrete (RC) trough bridge with two spans of 12 m each, designed to carry a single railway line, see Fig. 1. The bridge was designed and built in 1955 and was taken out of service in 2005 because of the building of a new high-speed railway, the Bothnia line. Before demolition, the bridge was loaded to failure to test its ultimate load-carrying capacity as part of the European research project sustainable bridges [Sustainable Bridges (SB) 2008]. The testing was carried out by applying loads on a beam perpendicular to the bridge (Fig. 2). The embankment south of the bridge was removed before the test. The loads were applied with jacks anchored in the ground beneath the bridge (Sustainable Bridges (SB)-7.3 2008). The bridge is skewed at an angle of 75° between its longitudinal and transverse directions. It is also curved in the horizontal plane with a radius of 300 m. Fig. 3 is of a longitudinal section showing the steel reinforcement.

To avoid a pure bending failure, for which good, calibrated models already exist, the bridge was strengthened before testing with bars of carbon fiber reinforced polymers (CFRP) (Täljsten et al. 2011). The two edge beams of the bridge were each fitted with nine Sto FRP Bar M10C with a length of 10 m and a rectangular cross-section of 10 × 10 mm. They were installed on 100-mm centers using the near-surface mounted reinforcement technique (NSMR) in presawn grooves, 15 × 15 mm, in the soffit of the bridge (Figs. 4 and 5) (Sustainable Bridges (SB)-6.3 2007).

Material Properties

The original concrete quality used in 1955 was Swedish K400, with a nominal compressive strength of 40 MPa (400 kp/cm² = 5,800 psi) measured on 200-mm cubes. This corresponds to a characteristic strength of 31 MPa (lower 5% percentile) and approximately to EC class C28/35. The steel reinforcement is mostly made from 16 and 25-mm diameter bars of quality Ks40 with a nominal yield strength of 400 MPa (58 ksi). Carbon fiber reinforcement Sto FRP Bar M10C has an $E_f = 260$ GPa and a mean tensile strength $f_f = 2,500$ MPa (363 ksi).

Separate mean concrete properties were determined by testing drilled core samples from each edge beam and the slab which showed that, by the date of the test in 2006, the concrete strength had increased to 68.5 MPa (9,935 psi) corresponding to EC class C55/67. This substantial increase is due the fact that the original cement was coarsely grinded and kept on hydrating and growing in strength after the 28 days when the initial strength was tested.

A concrete damage plasticity model was used in the finite element calculations for the bridge slab and mid columns. The model was chosen mostly because other users had obtained good results with it (FIB 2008). The following properties were assumed, based on the tested material properties: Young’s modulus of elasticity for the concrete $E_c = 25.4$ GPa; Poisson’s ratio $\nu = 0.167$; the dilatation angle $\beta = 35^\circ$; the flow potential eccentricity $\varepsilon = 0.1$; and the biaxial/uniaxial compression plastic strain ratio $f_{fs}/f_c = 1.16$ and the invariant stress ratio $\kappa = 0.666$ (Puurula 2012).

The steel reinforcement bars and the surrounding concrete were modeled together, increasing the nominal virtual stiffness of the steel up to the stress when the concrete cracks, see Fig. 6. This was done in accordance with the results of RILEM Committee 147-FMB “Fracture mechanics to Anchorage and Bond” (Elfgrén and Noghabai 2001, 2002). This procedure increased the stiffness of the calculated load-deflection diagram so that it better followed the curve from the test.

A summary of the material properties is presented in Table 1. Initial characteristic properties are given first based on the original drawings, followed by updated properties based on mean values of the tested samples taken from the bridge following the load test to failure.

Finite Element Model and Calibration with a Full-Scale Field Test

The Örnsköldsvik Bridge was modeled with successively improved models, starting with linear two-dimensional frame models and

---

Fig. 1. View of bridge in Örnsköldsvik in northern Sweden prior to testing (adapted from Sustainable Bridges (SB) 7.3 2008; image by Lennart Elfgrén)
ending with a nonlinear three-dimensional finite element model (Fig. 7) using Brigade (2011), which is based on Abaqus software. The calculation models were calibrated with results from the full-scale field test of the bridge (Puurula 2012). The boundary conditions and the nonlinear material properties during yielding of the steel reinforcement close to failure were deemed important parameters to calibrate.

The model had 1,650 separate structural parts (most of them discrete reinforcement bars), 152,460 elements, 164,003 nodes, and 511,317 variables. Solid elements in the concrete bridge were of type continuum, 3-dimensional, 8-node, reduced integration (C3D8R): 8-node linear brick, reduced integration, and hourglass control. Parameter studies were carried out with different element sizes; elements smaller than 150 mm did not improve the results. Discrete reinforcement bars were modeled as wires, type two-node linear 3D truss elements embedded in the concrete. The CFRP reinforcement bars in their grooves were modeled as perfectly bonded to the surrounding concrete. The steel beam used to introduce the load on the bridge was modeled using shell elements of type S4R: linear quadrilateral, four-node doubly curved shell, reduced integration, and hourglass control. The piles were modeled as springs, each inclined pile as a separate spring with a stiffness in the vertical direction of $k_v = 285 \text{ MN/m}$ and in the horizontal direction of $k_h = 71 \text{ MN/m}$. The earth pressure $\sigma_h$ on the East abutment was modeled as $\sigma_h = k_0 \cdot \gamma \cdot h$, where $k_0 = 0.34$ is a coefficient for the soil pressure, $\gamma = 20 \text{ kN/m}^3$ is the weight of the earth, and $h$ is the height from the Earth’s surface (m) (Puurula 2012). As discussed, the material modeling and the boundary conditions were important parameters in the calibration of the model. The contact between the steel beam and the concrete was modeled with a tie constraint, which does not allow the contact surfaces to move in relation to one another. Another issue was to calculate the overall load-deflection curve of the bridge. Here, it was essential to consider the effect of the surrounding concrete in Fig. 6.

**Fig. 2.** Elevation, plan, and section of the bridge showing the test loading arrangement using a steel beam placed in the middle of one of the two spans and pulled downwards (adapted from Sustainable Bridges (SB) 7.3 2008)
and the choice of damage parameters. Dilatation angles between $\beta = 10^\circ$ and $50^\circ$ gave similar results. Convergence problems were addressed using Riks method (FIB 2008).

The bridge was tested in July 2006. The load was applied using two jacks on top of a steel beam, which was pulled downwards (Figs. 1 and 2). The monitoring system consisted primarily of strain gauges that were spot-welded to the reinforcement and glued to the CFRP bars and the concrete, an optical laser displacement sensor and linear varying differential transducers (LVDTs) Sustainable Bridges (SB)-7.3 2008. The load-deflection curve from the final

---

**Fig. 3.** Longitudinal section of the bridge showing the layout of the main steel reinforcement and the outline strengthening scheme

**Fig. 4.** Cross section showing the principal dimensions of the bridge and reinforcement details together with the location of the near-surface mounted reinforcement (NSMR) with FRP Bar M10C on 100 mm centers

**Fig. 5.** Installation of the NSMR showing (a) sawing of grooves; (b) filling grooves with epoxy adhesive; (c) grooves following insertion of the CFRP reinforcement

---
test is given in Fig. 8, which also shows the calculated load-deflection curve and the effect of strengthening. It is shown that the two curves follow one another closely and that the bridge exhibits ductile behavior with a large deflection of the order of 0.1 m before failure. The calculated strains in the steel and CFRP reinforcement and in the concrete also correspond well to the measured values.

At the time of failure, high bond stresses between the concrete and the resin in the outermost groove initiated a bond failure after yielding of the bottom longitudinal steel reinforcement. The bond stresses were calculated to be 11.3 MPa with an alternative, refined model in which the CFRP reinforcement was embedded in epoxy (Puurula 2012). This is higher than the bond strength of 9.0 MPa for this type of bar (Nordin and Täljsten 2003). The bond failure lowered the available tensile force at the bottom and increased the inclination of the concrete compression struts which produced higher stresses in the stirrups, as fewer stirrups had to carry the load. These stresses, mostly caused by the vertical shear forces but also, to some extent, by the torsion moment from the loads transferred from the steel beam to the slab, ruptured the stirrups. The torsion moment originates primarily from the load of the steel beam. The edge beams twist outwards because they are nonsymmetrically supported on the bridge slab. The outsides of the edge beams deflect the most because of torsion, which explains why the final failure started in the outermost groove of the CFRP bars (Puurula et al. 2013). The ultimate load capacity was reached at an applied midspan load of 11.7 MN, see Fig. 9. The inclined failure crack had an inclination of approximately 35° with respect to the horizontal axis.

A preliminary description of the load test and the finite element calculations is given in Puurula et al. (2008) and in more detail in the Ph.D. theses of Sas (2011) and Puurula (2012) and in Sas (2012) and Puurula et al. (2013).

Comparison with Codes

The load capacity of the bridge was calculated using three major codes from the United States, Canada, and Europe, namely, ACI-318 [American Concrete Institute (ACI) 2011], CSA-A23.3 [Canadian Standards Association (CSA) 2004], and EC2 [European Committee for Standardization (CEN) 2004], respectively. The background to them is given in ASCE-ACI 445 (1998).

Table 1. Summary of Material Properties

| Material and Its Properties | Concrete | Steel | CFRP |
|-----------------------------|----------|-------|------|
| Material and Its Properties | $f_c$ | $f_t$ | $E_c$ | $G_F$ | $f_y = R_{eh}$ | $f_u = R_m$ | $E_s$ | $f_f$ | $E_f$ | $\varepsilon_{\text{uf}}$ |
| Initial characteristic properties | MPa | MPa | GPa | N/m | MPa | MPa | GPa | MPa | MPa | % |
| Based on drawings | 31 | 1.8 | 32 | — | $\Omega 16:410$ | $\Omega 16:500$ | 200 | 2,500 | 260 | $\approx 0.8$ |
| Mean properties based on tests (standard deviations are given in parenthesis) | 68.5 (8) | 2.2 (0.5) | 25.4 (1.7) | 154 (82) | $\Omega 25:390$ | $\Omega 25:500$ | $\Omega 16:738$ (2.4) | $\Omega 25:706$ (22.6) | $\Omega 25:192.1$ (23.3) | — | — | — |

Note: For the concrete: $f_c$ = compressive strength; $f_t$ = tensile strength; $E_c$ = modulus of elasticity; and $G_F$ = fracture energy. For the steel: $f_y = R_{eh}$ is the yield stress; $f_u = R_m$ is the ultimate stress; and $E_s$ = modulus of elasticity. For the CFRP: $f_f$ = tensile strength; $E_f$ = modulus of elasticity; and $\varepsilon_{\text{uf}}$ = failure strain.

Fig. 6. Steel stress-strain curve for 25-mm diameter bars with and without considering the effect of surrounding concrete (data from Elfgren and Noghabai 2001, 2002)

Fig. 7. (a) Model of bridge with nonlinear concrete in the bridge slab and midcolumns and linear concrete in the other parts; failure started in the east edge beam; (b) nonlinear discrete reinforcement is embedded in the concrete model of the bridge with a perfect bond to the concrete (Puurula 2012). This is higher than the bond strength of 9.0 MPa for this type of bar (Nordin and Täljsten 2003). The bond failure lowered the available tensile force at the bottom and increased the inclination of the concrete compression struts which produced higher stresses in the stirrups, as fewer stirrups had to carry the load. These stresses, mostly caused by the vertical shear forces but also, to some extent, by the torsion moment from the loads transferred from the steel beam to the slab, ruptured the stirrups. The torsion moment originates primarily from the load of the steel beam. The edge beams twist outwards because they are nonsymmetrically supported on the bridge slab. The outsides of the edge beams deflect the most because of torsion, which explains why the final failure started in the outermost groove of the CFRP bars (Puurula et al. 2013). The ultimate load capacity was reached at an applied midspan load of 11.7 MN, see Fig. 9. The inclined failure crack had an inclination of approximately 35° with respect to the horizontal axis.

A preliminary description of the load test and the finite element calculations is given in Puurula et al. (2008) and in more detail in the Ph.D. theses of Sas (2011) and Puurula (2012) and in Sas (2012) and Puurula et al. (2013).
The interaction between the shear force and the bending moment, as a function of a unit load $P$, is presented in Fig. 10. Because the dead load was already acting on the bridge, its effect [the hatched parts of the diagrams in Figs. 10(b and c)] has not been considered in the analysis. Calculations based on the initial characteristic concrete strength, $f_c = 31$ MPa, and the tested mean value, $f_c = 68$ MPa, are given in Sas (2011), Sas et al. (2011) and some of the results are summarized in Table 2.

The three codes predict the shear force capacity and the ultimate load capacity of the bridge in a conservative manner. The ratio between the predicted value $P_V$ and the test result $P_{Test}$ varies between 0.31 (EC2, with a concrete strut inclination of $\theta = 45^\circ$), 0.65 (CSA, $\theta = 38^\circ$), 0.66 (ACI, $\theta = 45^\circ$), and 0.78 (EC2 with minimum value $\theta = 22^\circ$). Because EC2 makes use of the variable angle truss model, both minimum and maximum capacities were estimated. They are shown in Table 2. The reason for the differences is the way the shear truss mechanism is applied. Codes ACI and CSA permit the use of the concrete’s contribution to the shear capacity, whereas in EC2, this is not permitted. Another factor that is responsible for some of the differences is that the codes differ in their treatment of the concrete compression strut inclination. In the ACI code, it is assumed to be fixed at 45°, which is quite a conservative assumption. The Canadian code is a simplified version of the modified compression field theory (MPCT), e.g., Collins and Mitchell (1991) and Bentz et al. (2006). Here, the inclination is determined iteratively from the cross-sectional equilibrium and is dependent on factors such as the crack spacing, concrete material properties and the average tensile and compressive strains over the cracked sections. In the simplified code definitions, the inclination depends primarily on the longitudinal strain $\varepsilon_{L}$ calculated at the middle of the cross-section. However, this definition has been calibrated using data obtained from beams with only steel reinforcement, whereas the bridge was also strengthened with CFRP reinforcement, a material that displays linear elasticity until failure. This might be one reason why the crack angle estimated by the CSA code does not correspond to the angle observed in the test.

The European code compensates for the omission of the concrete contribution to the shear capacity by adopting a crack angle that most closely matches the angle observed in the test. In addition, the truss model used in EC2 is a transparent geometrical method and the change in the tensile longitudinal strains because of the addition of the strengthening can be easily incorporated into the analysis to obtain cross-sectional equilibrium. As the longitudinal force in the tensile chord of the truss increases, the crack angle is reduced; therefore, the assumed crack has to bridge more stirrups to obtain equilibrium. In this way, EC2 predicts an increase of the shear force capacity after strengthening. However, disregarding the concrete contribution leads to conservative estimates of the shear capacity.

When the initial characteristic compressive strength of the concrete is used, the discrepancies between code predictions and test results are even larger (Table 2). This underlines the importance of using actual tested values in assessment of structures.

Fig. 8. Comparison of load-deflection curves for the cases with and without FRP strengthening; 1 = effect on stiffness, 2 = strengthening effect

![Fig. 8](image)

Fig. 9. Failure cracks with ruptured stirrups in the beams after the maximum load of 11.7 MN in the span close to the South abutment: (a) west beams; (b) east beams (images by Lennart Elfgren)
Fig. 10. Shear force and bending moment acting on the bridge under a unit load \( P = 1 \) MN and \( q = 58.7 \) kN/m (dead load): (a) geometry; (b) shear force \( V \); (c) bending moment \( M \); (d) forces in the longitudinal steel reinforcement because of bending and shear and shear: \( z \) denotes the internal lever arm.

### Assessment of Load-Carrying Capacity for a Train Load

A very simple first estimate of the load capacity of the bridge can be obtained by dividing the actual failure load, \( P = 11.7 \) MN, by the originally designed-for axle load of 250 kN. This gives the load capacity of the bridge as 11.7/0.25 = 46 axles. On a 12-m span, there is room for one locomotive with six axles, spaced at 1.6 m centers. This results in the approximation that the strengthened bridge may carry the equivalent of 46/6 = 7.9 locomotives.

However, by using the finite element model presented previously, now including the soil pressure at both ends of the bridge, it is possible to assess the capacity of the bridge more accurately. In the calculations using Brigade, the train load is increased successively and, at the same time, the uniformly distributed soil pressure is calculated as \( q_{h} \times 71 = 330 \) kN or approximately \( 4.71 \times 330/250 = 6.2 \) times the modern design loading, representing trains with an axle load of 330 kN or approximately 4.71 times the modern design loading, representing trains with an axle load of 330 kN.

In Fig. 12, vertical displacements are presented for the unstrengthened bridge at the train load level \( q = 304 \) kN/m². This load level corresponds to \( 304/64.5 = 4.71 \) times the modern design loading, representing trains with an axle load of 330 kN or approximately 4.71 times the modern design loading. The deflection in the middle of the bridge slab reached a value of 0.1028 m, when the maximum stress in the reinforcement in the slab were far beyond their yield limit (Fig. 13). Hence, according to the model, the failure of the original, unstrengthened bridge would start in the deck slab.

The influence of the two potential strengthening alternatives is illustrated in Figs. 14–18. Comparable load-deflection curves for the midpoint of the slab are given in Fig. 14. It is shown that strengthening the edge beams only contributes to a small stiffening of the slab. Strengthening of the slab, however, has a substantial effect on the slab stiffness. The strengthening of the slab also considerably increases the overall load-carrying capacity of the bridge. Failure deflections in the order of 0.1 m are large but are indicative of ductile behavior despite the introduction of nonductile CFRP reinforcement.
**Fig. 11.** Proposed strengthening of the slab with Sto FRP Bar M10C at 150 mm centers, parallel to the supports but not parallel to the steel reinforcement

**Fig. 12.** Modeled vertical displacements of the bridge slab for an unstrengthened bridge at the load level $q = 304 \text{kN/m}^2 = 4.71q_0$, where $q_0 = 64.5 \text{kN/m}^2$ is the mean live load for a train with an axle load of 330 kN. The maximum displacement in the midpoint of the slab is 0.1028 m

**Fig. 13.** Maximum tensile stress of 629 MPa, well above the yield stress of 440 MPa, is obtained for a load level of $q = 304 \text{kN/m}^2 = 4.71q_0$ for an unstrengthened bridge
The effect of the strengthening of the slab is further demonstrated in Fig. 15, in which load-stress curves are given for the transverse bottom slab steel reinforcement, showing that the CFRP strengthening of the slab decreases the steel tensile stress. In the case in which the slab is strengthened with transverse CFRP bars, much of the tensile force is taken by the carbon reinforcement especially after the steel reinforcement started to yield at a load level of $q = 230 \text{kN/m}^2$. The CFRP reinforcement reaches its ultimate failure stress at a load level of $q = 487 \text{kN/m}^2$ (or $7.54q_0$). After initial failure, the stress in the carbon bar decreases and the stresses in the steel reinforcement start to increase, now in the hardening part of the stress-strain curve (Fig. 16). Just strengthening the edge beams does not have any effect on the stresses in the tensile steel reinforcement in the slab.

Load-strain curves for the transverse bottom reinforcement in the middle of the slab for increasing train load are shown in Fig. 16. The steel reinforcement was located squarely in the slab to minimize the distance to the edge beams, whilst the CFRP reinforcement was placed slightly skewed, to be parallel with the supports, see Fig. 11. This is the reason why the steel reinforcement carries larger strains than the carbon reinforcement. After the failure of the carbon bars, the steel reinforcement takes over the load from the carbon bars and the strains in steel reinforcement increase rapidly for the same load level.

Plastic strains in the concrete at the failure load as seen from underneath are shown in Fig. 17 for the strengthened slab. It gives an indication of the crack pattern. In Fig. 18, load-strain curves are given for the longitudinal reinforcement in the centre of the East edge beam when the slab is strengthened transversely with CFRP bars. The steel tensile reinforcement in the edge beams begins to yield at the load level $q = 420 \text{kN/m}^2$.

In summary, the transverse carbon strengthening chosen was quite appropriate because the strengthened slab can sustain $q = 487 \text{kN/m}^2 = 7.54q_0$, and the steel reinforcement in the edge beams can sustain $q = 420 \text{kN/m}^2 = 6.5q_0$ before yielding starts. The rupture of the carbon at $\varepsilon_{uf} \approx 0.008$ is very brittle and is to be avoided. In this case, the yielding of the steel reinforcement in the edge beams leads to a ductile failure. This can, therefore, be predicted to be the ultimate limit of the bridge’s load-carrying capacity.

**Summary and Conclusions**

This paper has described how the load-carrying capacity of a reinforced concrete railway bridge can be calculated for different strengthening alternatives using a nonlinear three-dimensional finite element model with discrete reinforcement bars which was calibrated using a full-scale test to failure of a redundant 50 year old bridge. The strengthening of the bridge was successful as the carbon fiber bars increased both the stiffness of the bridge and its bending moment capacity.

The three-dimensional nonlinear calculation method with discrete reinforcement was used to model the bridge in an integrated way, including twist and deflections in all directions. The calculation model was first calibrated with test results from a full-scale loading to failure of a railway bridge in Örnsköldsvik in northern Sweden. The bridge behavior with increasing load and its ultimate load-carrying capacity were closely predicted using the calculation method described. The focus was on the assessment of the load-carrying capacity during nonlinear behavior of the materials rather...
than investigating other failure modes such as intermediate crack (IC) induced debonding, anchorage failure, peeling, or delamination of the carbon fiber reinforcement.

After adjustment and verification of the model, train loading was applied to the bridge using statistical mean values of actual material properties obtained from testing samples taken from the bridge after the conclusion of the load test. The load-carrying capacity of the bridge was then calculated for the bridge before and after strengthening with near-surface mounted reinforcement (NSMR) consisting of carbon fiber reinforced polymers (CFRP). The method can be used to ensure a ductile behavior after strengthening and to avoid brittle failure modes which may be caused by rupturing carbon fibers. Bridge sections for strengthening should be designed so that ductility is maintained by ensuring that the FRP bars do not rupture before extended yielding of the steel reinforcement.

The assessment of the different strengthening alternatives shows that the bridge, after strengthening the deck slab with CFRP bars, would have failed under a load 6.5 times the current maximum axle load of 330 kN, whilst the unstrengthened bridge would have failed under a load 4.7 times the axle load of 330 kN and approximately 6.2 times the originally designed-for axle load of 250 kN.

The calculations show that ordinary reinforced concrete bridges of the type studied are ductile and often have a reserve capacity, which may be utilized after a proper assessment procedure. This extra capacity is seldom obvious when a standard evaluation is run using nominal material properties and code models.

Therefore, to increase our knowledge and to use our existing bridges to their best potential, more bridges of different types should be tested to failure in a planned way, when they are going to be demolished. In this way we can (1) analyze the real behavior of typical bridges, (2) develop appropriate strengthening strategies, and (3) determine the real safety of our bridges—with and without strengthening.

Acknowledgments

The authors gratefully acknowledge support and contributions from our partners and collaborators in the Sustainable Bridges Project (www.sustainablebridges.net): the European Union 6th Framework Program; Banverket, Sweden; The Federal Institute for Materials Research and Testing (BAM), Germany; Chalmers University of Technology, Sweden; City University, U.K.; Cervenka Consulting, Czech Republic; COWI A/S, Denmark; Design Projektanvänkan, Sweden; Deutsche Bahn (DB), Germany; École Polytechnique Fédérale de Lausanne (EPFL); and Eidgenössische Materialprüfungsanstalt (EMPA), Switzerland; Luleå University of Technology and Lund University of Technology, Sweden; Finnish Rail Administration and Finnish Road Administration, Finland; Laboratoire Central des Pontes et Chaussées (LCPC), France; Network Rail, U.K.; Norut Technology, Norway; PKP Polish Railway Lines, Poland; Royal Institute of Technology (KTH), Sweden; RWTH, Germany; Skanska Sverige AB, Sweden; SNCF, France; STO Skandinavia AB, Swedish Geotechnical Institute, and Swedish Road Administration, Sweden; Universitat Politècnica de Catalunya, Spain; University of Minho, Portugal; University of Oulu, Finland; University of Salford, U.K.; Universität Stuttgart, Germany; Wrocław University of Technology, Poland and WSP, Finland. The following companies outside the Sustainable Bridges Project have also contributed: Botniabanan, Sweden; Denmark Technical University (DTU), Denmark; Nordisk Spännarmering, Sweden; Savonia University of Applied Sciences, Finland and Örnsköldsviks kommun, Sweden.
References

American Concrete Institute (ACI). (2011). “Building code requirements for structural concrete and commentary.” ACI-318, Farmington Hills, MI, 503.

ASCE and American Concrete Institute (ACI) 445 on Shear and Torsion. (1998). “Recent approaches to shear design of structural concrete.” J. Struct. Eng., 10.1061/(ASCE)0733-9445(1998)124:12(1375), 1375–1417.

Bentz, E. C., Vecchio, F. J., and Collins, M. R. (2006). “Simplified modified compression field theory for calculating shear strength of reinforced concrete elements.” ACI Struct. J., 103(4), 614–624.

Bicamie, N., Mang, H., Meschke, G., and De Borst, R., eds. (2014). Computational modelling of concrete structures, CRC Press/Balkema, Leiden, Netherlands, 1120.

Brigade. (2011). Brigade software. (http://www.scanscot.com/products/overview/) (Jun. 7, 2014).

BV Bridge. (2003). BVS 583.10 Broregler for nybyggnad – BV Bro, utgåva 9 [Rules for new bridges—version 9], Banverket, Borlänge, Sweden. (in Swedish).

Canadian Standards Association (CSA). (2004). “Brigade software.”

Collins, M. P., and Mitchell, D. (1991). “European Committee for Standardization (CEN).” (2004). “ASCE and American Concrete Institute (ACI) 445 on Shear and Torsion.”

Elfgren, L., and Noghabai, K. (2002). “Nordin, H. and Täljsten, B. (2003).”

Leiden, Netherlands, 1120.

Sustainable Bridges (SB). (2008). “Sustainable bridges—Assessment for future traffic demands and longer lives.” (www.sustainablebridges.net) (Jun. 7, 2014).

Sustainable Bridges (SB)-6.3. (2007). “Field Tests. Örnsköldsvik bridge—Full scale testing; Vitmossen—Strengthening of the subsoil; Frövi bridge—Strengthening and monitoring.” Deliverable D6.3. sustainable bridges—a project within EU FP6, B. Täljsten and A. Carolin, eds., Luleå Univ. of Technology, Luleå, Sweden.

Sustainable Bridges (SB)-7.3. (2008). “Field Test of a concrete bridge in Örnsköldsvik, Sweden.” Deliverable D 7.3, sustainable bridges, L. Elfgren, O. Enochsson, and H. Thun, eds., Luleå Univ. of Technology, Luleå, Sweden.

Schlune, H., Plos, M., and Gyllöft, K. (2009). “Improved bridge evaluation through finite element model updating using static and dynamic measurements.” Eng. Struct., 31(7), 1477–1485.

Scordelis, A. C., Elfgren, L. G., and Larsen, P. K. (1977). “Modelación numérica de estruturas porticadas de betão armado e pré-esforçado críticas ao esforço de transverso.” Mat. Struct., 35(6), 318–325.

European Committee for Standardization (CEN). (2004). “Eurocode 2: Design of concrete structures—Part 1-1: General rules and rules for buildings.” EN 1992-1-1, Brussels, Belgium, 225.

Federation internationale du béton (FIB; International Federation for Structural Concrete). (2008). “Practitioners’ guide to finite element modelling of reinforced concrete structures. state-of-art report.” Bullettin 45, Int. Federation for Concrete Structures, Lausanne, Switzerland, 344.

Fernandez Ruiz, M., Muttoni, A., and Hars, E. (2007). “Experimen- tial investigation on the load carrying capacity of thin webs including post-tensioning tendons.” Proc. Fiba Symp., Int. Federation of Structural Concrete, Lausanne, Switzerland, 483–490.

Ferreira, D., Buiran, J., and Mari, A. (2012). “Modelação numérica de estruturas porticadas de betão armado e pré-esforçado críticas ao esforço transverso e estatísticas de reforço [Numerical simulation of reinforced and prestressed frame structures critical to shear and strengthening proposals].” Proc. Encontro Nacional Betao Estrutural- BE2012, Faculdade de Engenharia da Universidade do Porto (FEUP), Porto, Portugal.

Malm, R. (2009). “Predicting shear type crack initiation and growth in concrete with non-linear finite element method.” Ph.D. thesis, Royal Institute of Technology, Stockholm, Sweden.

Nordin, H. and Täljsten, B. (2003). “Concrete beams strengthened with CFRP, a study of anchor lengths.” Proc. 10th Conf. on Structural Faults and Repair, Engineering Technical Press, Edinburgh, Scotland, 135.

Pimentel, P., Figueiras, J., and Bruhwiler, E. (2007). “Numerical modelling of prestressed beams for structural examination of existing bridges.” Congress on Numerical Methods in Engineering, J. César de Sá, R. Delgado, A. Rodríguez Ferran, J. Oliver, P. R. M. Lyra, and J. L. D. Alves, eds., APMTAC–Portuguese Association for Theoretical, Applied and Computational Mechanics, Lisbon, Portugal.

Plos, M. (1990). “Skjovrovskö i full skala på plattframbo i armerad betong [Full scale shear test of a reinforced concrete slab frame bridge].” Rep. 90, Chalmers Univ. of Technology, Div. of Concrete Structures, Gothenburg, Sweden, 45–72 (in Swedish).

Plos, M. (1995). “Application of fracture mechanics to concrete bridges. finite element analysis and experiments.” Ph.D. thesis, Chalmers Univ. of Technology, Division of Concrete Structures, Gothenburg, Sweden, 57–70.

Puurula, A. (2012). “Load-carrying capacity of a strengthened reinforced concrete bridge, Non-linear finite element modeling of a test to failure. Assessment of train load capacity of a two span railway trough bridge in Örnsköldsvik strengthened with bars of carbon fibre reinforced polymers (CFRP).” Doctoral thesis, Division of Structural Engineering, Luleå Univ. of Technology, Luleå, Sweden, 328.

Puurula, A., et al. (2008). “Full-scale test to failure of a strengthened reinforced concrete bridge. Calibration of assessment models for load-bearing capacities of existing bridges.” Nordic Concrete Research, Vol. 2008;2, Publ. 38, The Nordic Concrete Federation, Oslo, Norway, 121–132.

Puurula, A., et al. (2013). “Loading to failure and 3D nonlinear FE modelling of a strengthened RC bridge.” Struct. Infrastruct. Eng., 10.1080/15732479.2013.836546.

Richard, B., Epaillard, S., Cremona, C., Adelaide, L., and Elfgren, L. (2010). “Nonlinear finite element analysis of a 50 years old reinforced concrete trough bridge.” Eng. Struct., 32(12), 3899–3910.

Sas, G. (2011). “FRP shear strengthening of reinforced concrete beams.” Doctoral thesis, Luleå Univ. of Technology, Luleå, Sweden.

Sas, G., Blanksvärd, T., Enochsson, O., Täljsten, B., Puurula, A., and Elfgren, L. (2011). “Flexural-shear failure of a full scale tested RC bridge strengthened with NSM CFRP: Shear capacity analysis.” Nordic Concr. Res., 2(2011(44)), 189–206.

Sas, G., Blanksvärd, T., Enochsson, O., Täljsten, B., and Elfgren, L., (2012). “Photographic strain monitoring during full-scale failure testing of Örnsköldsvik bridge.” Struct. Health Monit., 11(4), 489–498.