Failure tests on concrete bridges: Have we learnt the lesson?

Niklas Bagge[1]*, Cosmin Popescu[1,2], Lennart Elfgren[1]

[1] *Department of Civil, Environmental and Natural Resources Engineering, Luleå University of Technology, 971 87 Luleå, Sweden.*

[2] *Norut Northern Research Institute, Rombaksveien E6-47, 8504 Narvik, Norway.*

Abstract

Full-scale failure tests of bridges play an important role in the better understanding of the behaviour of bridges and in the development of assessment methods. Such experiments are challenging and often expansive and, thus, are rare. This paper provides a review of failure tests on concrete bridges with a focus on the learning outcomes from the tests. In total, 40 failure tests have been identified for 28 bridges. Various types of bridges have been studied, involving bridge decks composed of slabs, girders and combinations thereof. There are examples of both reinforced concrete (RC) and prestressed concrete (PC) decks. In general, the tests indicated conservative estimates of the load-carrying capacity using theoretical calculations based on methods traditionally used for design and assessment. It can also be concluded that almost a third of the experiments resulted in unexpected failures, mainly shear instead of flexure. It also seems that inaccurate representation of geometry, boundary conditions and materials often explains the differences between the theoretical and the tested capacity.

Key words: Assessment, bridges, carbon fibre reinforced polymers, fatigue, flexure, full-scale failure test, load-carrying capacity, prestressed concrete, punching, reinforced concrete, shear.

1 Introduction

In order to provide reliable and appropriate performance assessments of existing bridges, a proper understanding of the structure is crucial. The common approach used by engineers to determine the load-carrying capacity is based on linear elastic structural analysis with verification of the action effects against local resistance models. Improved structural analysis can be carried out using plastic or nonlinear methods, but the latter method is rarely used in practice. However, the question remains: how precise are the available methods for assessment and can we rely on them? At service-load level, a bridge can be investigated to calibrate the assessment methods; this field has been extensively studied (Brownjohn et al. 2003; fib 2003; Goulet & Smith 2013; Jang & Smyth 2017; Sanayei et al. 2012; Živanović et al. 2007). Due to its nature, the approaches for analysis at ultimate-load level cannot be calibrated in the same way for a bridge and the structural behaviour proven for service loads is, because of nonlinearities, not necessarily representative at higher load levels.

Methods for determining the resistances of concrete structures, and consequently the load-carrying capacity, are often based on small-scale laboratory experiments so the conditions are not necessarily representative of those for actual bridges. Usually, laboratory experiments are carried out at an element level, while the entire bridge structure needs to be taken into account using structural analyses, including the interaction between different elements. Moreover, real-world structures involve a higher degree of uncertainties compared to elements investigated in a controlled environment and this has to be accounted for when assessing existing bridges. In order to understand the structural behaviour of bridges better and to improve methods for determination of their capacity, large-scale laboratory experiments have been carried out under more realistic conditions. Some examples of studies include those by Bouwkamp et al. (1974), Scordelis et al. (1977), Roschke and Pruski (2000), Vaz Rodrigues et al. (2008), Nilimaa et al. (2012) and

* Corresponding author. Tel.: +46 (0)920 491 264
E-mail address: niklas.bagge@ltu.se (N. Bagge)

Amir et al. (2016). Moreover, tests have been carried out on structural elements taken from bridges, for instance, by Shenoy and Frantz (1991), Labia et al. (1997), Eder et al. (2005), Harries (2009), Cook (2010), Rogers et al. (2012), Vill et al. (2012) and Barr et al. (2014). However, even these studies involve simplification of the reality. Thus, destructive in-situ testing plays a very important role in the understanding of the structural behaviour of bridges and the development of reliable methods for assessment of them. Specifically, the load-carrying capacity and the failure mechanism are of significance in order to ensure the requested structural safety.

Full-scale testing of bridges involves a range of challenges and, therefore, such tests are relatively rare. Such tests are expensive, and present organisational and safety issues, difficulties in terms of loading and measurements as well as sometimes limited accessibility to the structural elements under investigation. With this in mind, it is important to take advantage of the experience obtained from each particular bridge study. The aim of this review paper is to highlight the full-scale failure tests of concrete bridges carried out to date and evaluate the lessons learnt from these studies. This is clearly an issue, with old tests tending to be forgotten and new ones carried out without knowledge of the previous ones. Here, bridge failure test refers to testing until the ultimate load-carrying capacity of the structure is reached and, preferably, the failure mechanism can be determined. Thus, this paper is highly relevant for bridge managers, practising engineers as well as researchers.

2 Overview of full-scale tests

A literature review yielded 30 concrete bridges that have been tested to failure since the first known test in Great Britain in 1952. All these bridges are listed in Table 1. Some of them involved several failure tests. For several projects, the documentation is limited and difficult to access and, thus, additional failure tests not identified in the review may exist. There were also issues associated with publications or results in foreign languages. Bridges made from reinforced concrete (RC) and prestressed concrete (PC) have been investigated. Various types of bridges have been tested and, in this paper, they are classified as either girder or slab bridges, based on the primary load-carrying system of the deck. The group of girder bridges includes bridges with the deck composed of a trough, a box-girder with cantilevers or the most common construction, that of several solid beams (either precast or cast-in-place) connected to a slab at the top. Both the girder and slab bridges involve frame structures. Only road bridges have been tested to failure, except for the South Bank bridge and the Örnsköldsvik Bridge, which were a pedestrian and railway bridge, respectively. In order to provide an improved understanding of the actual structural systems of bridges, the literature review focused on studies of bridges which had no major modifications to their structure. However, due to unanticipated experimental results leading to significant learning outcomes, some bridges have been included that were cut longitudinally to make it possible to load the structure to failure. In addition to the bridges listed in Table 1, a failure test of the De Vecht Bridge was carried out in the Netherlands in 2016, but no reports are available yet.

Figure 1 shows the bridge studies by country, age and type of bridge element tested against number of tests or number of bridges. The United States has an extensive stock of existing bridges with many in need of repair, strengthening or reconstruction (Department of Transport 1990). It can also be seen that smaller countries such as Sweden and Korea have put some effort into this type of research. Notably, shear-related issues were the focus in the majority of the tests in Sweden, representing nearly half of the studies with predicted shear failures. Moreover, a review of the literature shows that a range of other types of bridges (e.g. steel and composite bridges) have been extensively tested in both the United States and Sweden.

Basically, two main reasons for decommissioning a bridge have been identified, these being severe damage and deterioration, and reconstruction due to, for instance, realignment of the road/railway or upgrading of the connected infrastructure. However, one case has been identified where the bridge was built specifically for experimental investigation and, in two cases, the cause of removal of the bridge is not fully clear. In Figure 1(b), the ages of the bridge have been shown with the background to why the bridge was taken out of service. When evaluating the tests, this background plays an important role since time-dependent changes of bridge characteristics may have an impact on the outcomes. From the data available, the majority of bridges were in good condition at the time of the experimental investigation and this can be expected to lead to a bridge evaluation involving less uncertainties. Severe structural damage or deterioration can be present after a service life of about 40 years and the majority of the tests of older

Table 1. Concrete bridges tested to failure (sorted by test year).

Bridge	Country	Construction year	Test year	Bridge type	Material	Length [m]	No. spans	Failure mode [a)]
South Bank	GBR	1948	1952	Girder	PC	86.56	4	F
Glatt 1	CHE	1955	1960	Slab[c)]	PC	38.00	3	F
Tingstad	SWE	1956	1967	Girder	RC	31.30	2	F
Boiling Fork Creek	USA	1963	1970	Girder	PC	80.47	4	P + S
Elk River	USA	1938	1970	Girder	RC	N/A	N/A	F
Casselton	USA	1962	1972	Slab	RC	19.80	3	F
Glatt 2	CHE	1954	1975	Slab[c)]	PC	12.50[b)]	1	S + F
Brønsholm	DEN	1956	1977	Slab	PC	47.74	2	S
Pennsylvania State Univ.	USA	1977	1981	Girder	PC	36.88	1	F
Lethbridge	CAN	1950	1984	Girder	RC	42.70	3	F
Hanpyung[d)]	KOR	1920	1987	Slab	RC	71.48	6	F
Ansan[d)]	KOR	1949	1987	Girder	RC	48.00	4	F
Stora Höga 1	SWE	1980	1989	Slab[c)]	RC	21.00[b)]	1	S
Stora Höga 2	SWE	1980	1989	Girder[c)]	PC	31.00[b)]	1	S
Batavia	USA	1953	1991	Slab	RC	32.23	3	P + S
Niobrara River	USA	1938	1992	Slab	RC	42.40	5	P
Thurloxton Underpass	GBR	1970	1993	Slab	RC	4.27[b)]	1	F
Smedstua	NOR	1989	1996	Slab	RC	38.30	3	F + S
Dongcheon[d)]	KOR	1949	1997	Girder	RC	132.00	11	F
Barr Creek	AUS	1939	1998	Slab	RC	16.46	4	P
Lennox Brach	USA	1932	1999	Slab	RC	23.77	3	F
Bangjimoon[d)]	KOR	1956	1999	Girder	RC	55.20	6	F
Seoul-Pusen	KOR	1971	2000	Girder	PC	360.00	12	F
Baandee	AUS	1969	2002	Slab	RC	37.64	5	P + S
Xin Xing Tang	CHN	1995	2005	Girder	PC	114.00	3	F
Xi Cheng River	CHN	N/A	2005	Girder	RC	120.00	6	F
Örnsköldsvik	SWE	1955	2006	Girder[c)]	RC	24.10	2	S
Nanping	CHN	1964	2007	Girder	RC	39.90	3	F
Paint Creek	USA	1967	2010	Girder	RC	43.84	3	F
Kiruna	SWE	1959	2014	Girder	PC	121.50	5	P + S

[a)] flexural failure (F), punching failure (P) or shear failure (S),
[b)] free span,
[c)] frame structure,
[d)] see Section 3.16

bridges were on structures that were damaged in some way.

In Figure 1(c), the type of tested and failed structural part is shown. Most commonly, RC elements have been tested and particularly RC slabs. The types of bridge tested are strongly dependent on the bridges available and slabs are fundamental for most of the bridge decks. Consequently, the reason for slab tests dominating the tests undertaken is not directly coupled to a lack of knowledge about these structural elements. On the contrary, a lack of full-scale studies of PC bridges can be identified.

Apart from the bridges in Table 1 and Figure 1, numerous other bridges have been tested without the ultimate load-carrying capacity being reached:

- Ruytenschildt Bridge: A 45.0 m long road bridge with five continuous spans and a deck composed of a RC slab (Lantsoght et al. 2016).
- Zuojiabao Bridge: A 120 m long road bridge with six equally long simply supported spans and a deck composed of T-shaped RC girders (Zhang et al. 2011a).
- East Meng Jiang Nu Bridge: A 144 m long road bridge with nine equally long simply supported spans and a deck composed of PC hollow-core slabs (Wang et al. 2011).
- Särevere Bridge: A 39.7 m long road bridge with three simply supported spans and the deck composed of T-shaped RC girders (Maanteemet 2015).
- Vienna Bridge: A 44.6 m long road bridge with one simply supported span and a deck composed of PC box girder segments (Pukl et al. 2002).
- Lautajokk Bridge: A 7.2 m long railway bridge with one simply supported span and a deck composed of a RC trough (Paulsson et al. 1996).

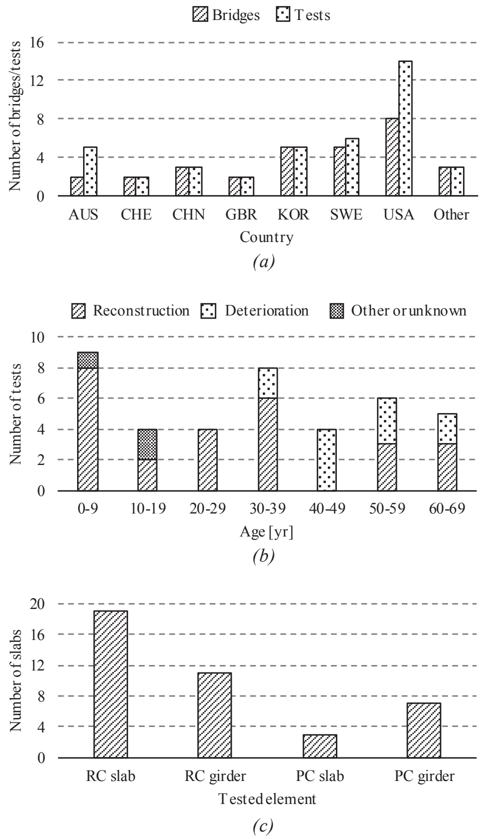

Figure 1. General data about failure tests: (a) country of test, (b) age of bridge at test and reason for test, and (c) type of element investigated.

Interestingly, the Ruytenschildt Bridge was tested twice in order to investigate the shear capacity. In both tests, the capacity of the loading system was insufficient, although increased capacity was used for the second attempt. After 29 years in service, the Lautajokk Bridge was subjected to cyclic loading at service-load level, in an attempt to induce a fatigue-related failure. However, after six million cycles, there was no failure and only minor visible concrete cracks were observed. Thereafter, the load applied to the bridge was increased without failure due to limited capacity of the loading system. The bridge was tested in a laboratory with boundary conditions similar to those existing when the bridge was in service. All these bridges consistently had considerably higher capacity than predicted by the approaches usually used for bridge assessment.

The bridges listed in Table 1 are described in the next two sections, categorised by cause of failure. Each subsection, corresponding to one test, is titled with bridge name, country in which the bridge was located, year of testing and the main source publication. In some cases, no bridge name was given in the original publication(s) reporting the test. In these cases, the closest city or village name has been used. The four bridges in Table 1 marked [d)] were investigated for the same reasons, tested in a similar way and reported together, yielding approximately the same conclusions. At the same time, the results from these tests and associated analyses are only sparsely described and mainly in a foreign language. Therefore, the outcomes from these studies are briefly presented together in Section 3.16.

3 Flexural failure modes

3.1 South Bank Bridge – Great Britain – 1952 – N/A (1951, 1952)

The South Bank Bridge was a PC girder pedestrian bridge with four continuous spans of lengths 23.17 m, 17.98 m, 23.17 m, and 16.46 m with a cantilever of 5.79 m. The deck was supported on columns and monolithically cast-in-place, consisting of a PC girder (1 219 × 559 mm^2) with trapezoidally-shaped RC cantilever slabs (1 200 × 76 – 292 mm^2) on either side at the top. A Freyssinet post-tensioning system was used and, during construction, the prestress losses were measured, indicating the prestress forces in the middle of the cables to be reduced to half of those at the end anchorage due to friction losses. The bridge was built for the Festival of Britain in London and was to be demolished thereafter.

The bridge was loaded with weights distributed on the deck in the first span. Using such weights, it was possible to compare the surface load assumed at design with the actual capacity. A crane was used to gradually add weights to the structure and, after large deformation, a flexural failure occurred with almost simultaneous concrete crushing at the loaded midspan and the adjacent intermediate support. Although the span collapsed (see Figure 2), the prestressed cables did not rupture; however, the bond between the steel and surrounding grout failed. The bridge was designed to carry an imposed surface load of 4.79 kN/m^2 (100 lb/ft^2) but the ultimate load in the test was 2.5 times higher. Thus, it can be concluded that the design understated the load-carrying capacity. Based on the information presented in N/A (1952), it can also be concluded that the causes of the "hidden" capacity have not been investigated. However, a part of the difference can be explained by the intended safety margin. The difference could also be associated with more favourable material properties and structural behaviour than assumed during the design.

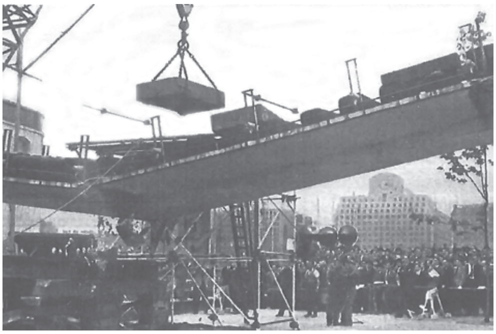

Figure 2. The South Bank Bridge after collapse (N/A 1952).

3.2 Glatt Bridge 1 – Switzerland – 1960 – Rösli et al. (1963)

The Glatt Bridge 1 was a skew (13.8°) PC frame slab bridge with two inclined intermediate supports 23.00 m apart. The distance to the abutments was 7.50 m (see Figure 3). The 8.89 m wide deck was monolithically cast-in-place with two lanes for road traffic on the interior slab (5 500 × 450 mm^2) and a pedestrian lane on cantilevers on either side. The cantilevers consisted of a thin slab with curbs at the edges. A post-tensioning BBRV system was used to prestress the bridge longitudinally using 19 cables in the slab and two cables in each curb. Construction of a new highway to Zürich Airport meant that the five-year old bridge had to be demolished and, thus, it was decided to investigate the bridge with regard to both fatigue and static capacity.

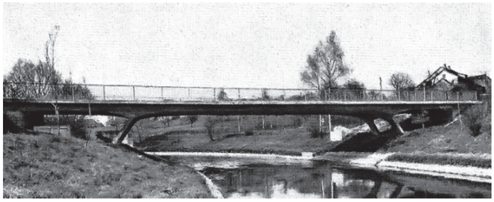

Figure 3. The Glatt Bridge 1 (Rösli et al. 1963).

The test was carried out using four jacks on top of the bridge anchored in the ground. The bridge was loaded symmetrically around the bridge centre with distances between the loading points of 7.66 m longitudinally and 3.38 m transversally. First, the bridge was loaded with two million cycles at service-load level (i.e. 880 kN), producing almost no change in elastic behaviour and without any permanent deformations. The midspan deflection was ± 5.7 mm and the corresponding stress amplitude of the prestressing cables was ± 8.1 MPa. The tests were continued in stages up to 2.8 times the service-load level with a deflection of 42 mm (i.e. about 1/500th of the bridge span). In total, 6.6 million cycles were applied. The maximum stress amplitude was ± 38.2 MPa which was quite small compared to the prestress level of about 950 MPa. A fatigue failure occurred in the cantilever at the connection to the curb. The load-carrying capacity of the bridge was not reached when this happened and the prestress in the main slab remained operational. The subsequent static test resulted in a flexural failure: at 4 700 kN, the slab failed in one of the end spans (see Figure 4), the load dropped, and the test proceeded until a final flexural failure of the slab in the middle of the interior span.

When tested, the bridge behaved as predicted. In the fatigue tests, the structure was heavily overloaded, yet the failure was only local with the residual load-carrying capacity well above the required value. In the static test, the bridge behaved in a ductile manner with extensive concrete cracking and deflection and, thus, there was warning of the failure.

Figure 4. The Glatt Bridge 1 after flexural failure in the end-span (Rösli et al. 1963).

3.3 Tingstad Bridge – Sweden – 1967 – Bådholm et al. (1967a, b)

The Tingstad Bridge was a curved, continuous, two-span PC bridge with span lengths 15.90 m and 15.40 m (see Figure 5) The deck was unsymmetrical: one part, consisting of four girders (1 000 × 600 mm^2) connected with a slab at the top, carried two lanes of road traffic, and the other part, consisting of one such girder with a cantilever slab in the top, carried a pedestrian lane. The total width of the deck was 10.40 m including curbs and the centre-to-centre distance of the girders was 2.0 m. At the supports (abutments and piers), the slab thickness (160 mm) was gradually increased to the height of the girder. The bridge was monolithically cast-in-place and was reinforced using the Dywidag prestressing system. Since the bridge did not fulfil the free height requirements when upgrading the highway, it had to be replaced. A full-scale failure test was designed to evaluate the bridge's flexural and torsional stiffness and, ultimately, the flexural capacity at the intermediate support.

Imposed displacements at the abutment were used to induce hogging at the intermediate support. The abutment was removed to make space for displacements downwards and two hydraulic jacks were installed to control the loading. Moreover, weights were placed in the midspan to avoid failure in sections not designed for hogging moments. In order to investigate the torsional stiffness, the imposed displacement was produced alternately by the two hydraulic jacks.

After extensive crack formation at the intermediate support, followed by concrete crushing at the place where the slab thins, a flexural failure occurred for a moment of 13 200 kNm (see Figure 6). The theoretically determined flexural capacity was 84.7 % of the tested peak load and thus, relatively consistent with the experimental result. However, from this study, it is proposed that further investigation of existing uncertainties takes place to enable improvement of the assessment. For instance, the material properties and positions of prestressed reinforcement are parameters that should be determined.

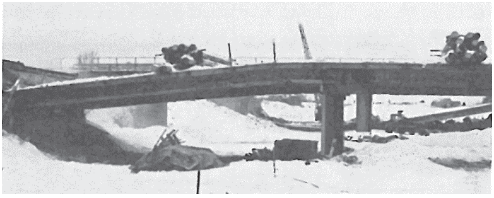

Figure 5. The Tingstad Bridge during loading using imposed displacement at the abutment (Bådholm et al. 1967a).

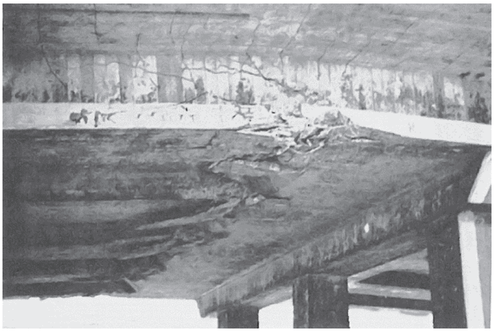

Figure 6. The Tingstad Bridge after failure of the girder at the intermediate support (Bådholm et al. 1967a).

3.4 Elk River Bridge – USA – 1970 – Burdette and Goodpasture (1971, 1972, 1973)

The Elk River Bridge was a skew (30º) multi-span (number of spans not reported) RC girder bridge carrying two lanes of traffic, with span lengths of 15.24 m (see Figure 7). The deck, composed of four girders (457 × 1 130 mm^2) with a centre-to-centre distance of 2.08 m connected with a slab (8 331 × 260 mm^2) at the top, was monolithically cast-in-place and simply supported on abutments and piers. After the area where the bridge was located was inundated, a new highway was built on higher ground. Consequently, several quite new bridges, in good condition, became redundant and available for failure tests. For improved understanding of the failure mechanisms and to provide information about the examination of design methods for bridges, four bridges were loaded to failure, two of which were concrete bridges. The results from testing the second bridge in the experimental programme are described in Section 4.1.

Figure 7. The Elk River Bridge (Burdette & Goodpasture 1973).

In order to simulate loading of two trucks passing the bridge side-by-side, using the rear axles of the design vehicle H20 (AASHO 1969), eight concentrated loads were applied to the bridge in the most adverse position for the midspan moment. In each wheel position, a hydraulic jack was used to apply the load to the bridge deck slab. Rods, through the structure and anchored in the bedrock, supported the jacks and those strains were measured to obtain the applied load. The bridge was loaded incrementally until the peak load of 7 280 kN whereafter the load dropped due to increased deflections and the test was stopped. A flexural failure took place and it was similar in magnitude to the theoretically assessed capacity (92.7 % of the tested load-carrying capacity). Visual monitoring during the test indicated that the first crack appeared at 40.0 % of the peak load. The difference between experimental and theoretical capacities can partly be explained by the use of reinforcing steel yield strength rather than tensile strength in the calculations. Based on the reported results of this test, it can be concluded that the bridge behaved in a ductile manner and failure was as predicted. Moreover, it shows that the flexural capacity can be accurately predicted for single-span bridges based on standard methods.

3.5 Casselton Bridge – USA – 1972 – Jorgenson and Larson (1976)

The Casselton Bridge was a skew (25º), continuous three-span RC slab bridge with two traffic lanes in the middle and a pedestrian lane on each side. The lengths of the end-spans were 6.10 m, and the interior span was 7.60 m. The deck, supported on piers and abutments, consisted of a monolithically cast-in-place slab (9 750 × 305 mm^2) with curbs (920 × 254 mm^2) keyed in on either side. The bridge design used values for the concrete compressive strength of 21 MPa (47 MPa) and 276 MPa (366 MPa) for the reinforcing steel yield strength, with tested mean values from the experimental programme specified in brackets. Due to realignment of the highway, the bridge was decommissioned and tested to failure 10 years after construction. The aims of the tests were to compare stresses occurring in the structure with corresponding design values and also to evaluate theoretical and experimental loads causing residual deformations and failure of the bridge, respectively.

For loads testing linear bridge response, a test setup with a steel frame and a hydraulic jack, supported by rods connected to the bridge piers, was used to represent a vehicle with four wheels (200 × 200 mm^2) with 1.8 m transverse and 1.2 m longitudinal spacings. The simulated vehicle overloading was applied centrically to the central span and increased up to 293 kN. For increased loading, the test setup was modified and a line load was applied to the middle of the central span through a transversally-aligned load distribution beam. The loading was increased up to 3670 kN, at which point the capacity of the loading system was reached. Therefore, two additional loading frames and hydraulic jacks were installed parallel in their unloaded state to the initial system, in order to continue the test up to a flexural failure at 4 226 kN.

In the linear stress range of the bridge, evaluation of the strains measured indicated considerably lower stresses than theoretically determined, although the calculations were carried out based on different assumptions regarding the interaction between the slab and the curbs. However, another possible explanation was the poor weather (low temperature) prevalent during installation of the gauges, leading to unreliable measurements. Using load-deflection curves from a set of loading cycles, residual deformations were observed at 39.5 % of the peak load, compared with 40.7 % according to calculations based on the assumption of residual deformations obtained when reaching the yield strength of the reinforcing steel (curbs excluded). With regard to the flexure capacity, assuming formation of yield hinges at both midspans and the intermediate supports and curbs included in the cross-section, the theoretical load-carrying capacity resulted in an overestimation of 5.8 %. Thus, from this full-scale test, it can be learnt that the flexural capacity can be relatively accurately predicted and linear analysis with redistribution of internal forces can be a useful method for statically indeterminate structures. Moreover, it shows the advantage of in-situ testing of material parameters for assessment of existing bridges; for this bridge, such tests revealed a considerably higher strength of both concrete and reinforcing steel.

3.6 Pennsylvania State University Bridge – USA – 1981 – McClure and West (1984)

The Pennsylvania State University Bridge was a curved, simply supported, single-span PC box-girder bridge with a length of 36.88 m (see Figure 8). Two girders with a centre-to-centre distance of 5.73 m carried one road lane each and they consisted of 17 precast RC box-shaped elements (2 743 × 1 676 mm^2) connected with shear dowels and post-tensioned bars and tendons. At the top were cantilever slabs (11 468 × 254 mm^2) with a 533 mm wide cast-in-place curb at each edge. Segmental bridges were of particular interest and, thus, this bridge was built as a part of a test track at Pennsylvania State University, specifically for experimental investigation aimed at improving the understanding of structural behaviour. In 1978, the bridge behaviour was studied under service loads following the American design standard (AASHTO 1973) and, three years later, one of the girders was loaded to failure.

The investigation at service-load level was based on trucks, similar to the design vehicles HS20-44 (AASHTO 1973), passing over the bridge (McClure & West 1980). Destructive testing was accomplished by two loading systems, centred around the midspan, with loading points above the webs of the girder in order to obtain a flexural failure. Each system consisted of one hydraulic jack on either side of the girder, connected to a bedrock anchor at the base and a hinge connection to a transversally located load distribution beam at the top (see Figure 8). The test was carried out with daily load increments controlled by changing the pressure in the jacks.

For trucks crossing, the bridge remained in the elastic range. The first crack was observed during the failure test in the bottom surface of the girder when the load level was 39.8 % of the load-carrying capacity. Thereafter, further flexural cracking took place and, at a load of 3 896 kN, loud sounds were heard, followed by an opening between two girder segments. At the peak load of 4203 kN, a series of loud sounds occurred, and the load dropped while the deflections drastically increased. An attempt to proceed with the loading resulted in increased opening between the segments, and concrete crushing and spalling in the bridge deck slab. After the test, rupture of all the strands was observed.

The two tests were simulated using finite element (FE) models and, in the failure test, nonlinear material responses were taken into account. Consistently, the test revealed a stiffer behaviour in terms of strains and deflections than given by the FE analyses. However, no capacity estimates were presented by McClure and West (1984). The review of the results of this project indicated that a rather limited amount of theoretical evaluation had been carried out (at least as described in the literature). This is surprising, given the unique nature of the project, that included construction of a bridge purely for experimental studies. For instance, the analyses indicated underestimated stiffness but the reason was not fully identified. A better understanding of how this kind of bridge behaves and how it can fail is important, but to provide a foundation for improved assessments in the future it is necessary to thoroughly investigate and clarify existing uncertainties.

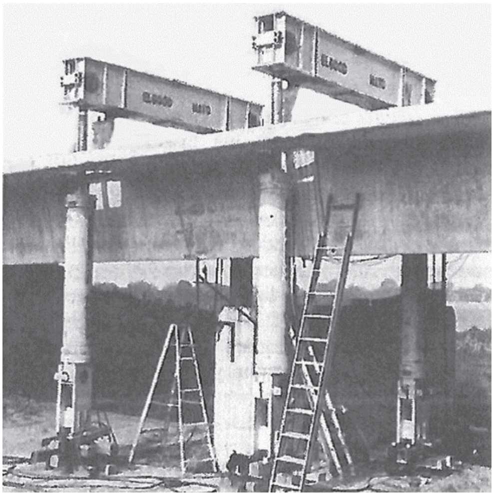

Figure 8. The loading of Pennsylvania State University Bridge (McClure & West 1984).

3.7 Lethbridge Bridge – Canada – 1984 – Scanlon and Mikhailovsky (1987)

The Lethbridge Bridge was a continuous three-span RC girder bridge for two-lane traffic, where the lengths of the end-spans and interior spans were 12.20 m and 18.30 m, respectively (see Figure 9). The cast-in-place deck, supported on piers and abutments, consisted of five girders (432 × 762 – 1 524 mm^2) with parabolic varying height and a centre-to-centre distance of 1.83 m. They were connected with a slab (8 640 × 254 mm^2) at the top. Interestingly, the slab thickness was 43 % thicker than specified in drawings. Due to the upgrading of the highway and, thus, replacement of the bridge, it became available for destructive testing, as a part of a research project aiming to improve assessment of the load-carrying capacity of bridges.

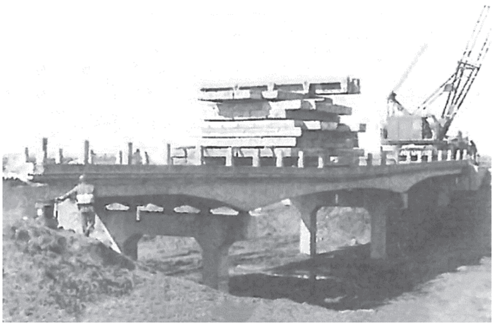

Figure 9. The Lethbridge Bridge illustrating loading procedure for testing (Scanlon & Mikhailovsky 1987).

A test setup was designed to investigate the flexural capacity in the interior span. First, the bearings at the abutments were replaced with hydraulic jacks to enable measurement of the reaction forces. Thereafter, the failure test was carried out in two steps: (1) incremental loading with weighed precast concrete elements in the middle of the interior span to a load level corresponding to about 60 % of the capacity (see Figure 9) and (2) loading to failure with the hydraulic jacks at abutments (upward) to produce additional flexural moment in the interior span. During the test, formation of flexural cracks was observed and, at the peak load and corresponding to a midspan moment of 11.4 kNm in the interior span, crushing of the curbs took place, followed by a drop in load. The bridge behaved in a ductile manner without structural collapse and considerable residual deformation occurred, indicating yielding of the reinforcing steel. Comparison with the predicted load-carrying capacity showed the calculated values to be 52.4 % and 95.0 % of the tested capacity, based on reinforcing steel yield and tensile strength, respectively. By ignoring the curbs and assuming the bridge geometry to be as specified in the drawings, 39.6 % of the tested capacity was obtained. Consequently, it can be seen from this study that: (1) flexural failures of statically determinate structures can be precisely modelled and predicted, based on standard equations for the flexural strength and (2) the existing structure should be inspected prior to assessment of the load-carrying capacity in order to reduce the number of uncertainties. Apart from representative material parameters, the importance of determining the geometrical characteristics was apparent in this study, with up to 14 % increase in capacity when using in-situ geometry rather than dimensions from the drawings. Moreover, the curbs were designed to contribute up to 19 % of the load-carrying capacity, which also indicates the significance of an accurate representation of the structure in the analysis.

3.8 Niobrara River Bridge – USA – 1992 – Azizinamini et al. (1994a, b); Shekar et al. (1993)

The Niobrara Bridge was a two-lane, five-span RC slab bridge, composed of three continuous interior spans (9.525 m, 11.430 m and 9.525 m) and two simply supported end-spans (lengths: 6.096 m). The bridge is shown in Figure 10. The deck, composed of a slab (7 976 × 394 mm^2 at spans 1 and 5, 7 976 × 432 mm^2 at spans 2 – 4) was monolithically cast-in-place and was supported on integral abutments and intermediate piers. At the edges of the slab, there were curbs (330 × 508 mm^2) on top. From drawings, the concrete compressive strength was 21 MPa and the reinforcing steel yield strength was 227 MPa. However, during the experimental investigation, a programme of material tests was carried out, indicating a mean value for the concrete strength of 22 MPa with an appreciable scatter (14 – 38 MPa), and a yield strength of 345 MPa. Moreover, the material evaluation revealed severe deterioration of the concrete, primarily due to freeze-thaw cycles, inadequately air-entrained concrete and alkali-silica reactions.

The bridge was assessed to be in poor condition and to have insufficient load-carrying capacity, so was decommissioned in 1972, after 38 years of service. However, the bridge was tested again in 1992, most probably implying that further degradation of the bridge had occurred, which could be of significance for the experimental outcome. The bridge was a part of a broader study including another six similar RC slab bridges, continuous with three spans, aimed at an experimental investigation of the structural behaviour and identification of parameters that are important for the load-carrying capacity. In the study, the effects of span length, skewness, deck slab widening and bridge condition were tested by simulating truck crossings following a predefined schedule at service-load levels.

The Niobrara River Bridge was further investigated with two failure tests: (1) at the simply supported end-spans and (2) at the outer spans of the continuous part. Based on a bridge design vehicle HS20 (AASHTO 1989) consisting of three axles, the test setup was designed with respect to the position producing the highest flexural moment. Consequently, the simply supported span was subjected to one axle per lane at the midspan, whilst the continuous span was subjected to two axles per lane with the central axle positioned 3.0 m from the abutment. To achieve the loading, hydraulic jacks were used for each wheel with a steel frame transferring the forces to the adjacent piers. With this test setup, it was possible to simulate truck loading in either lane, both lanes or along the longitudinal centre line of the bridge.

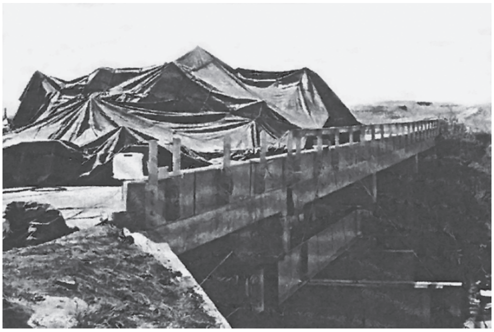

Figure 10. The Niobrara Bridge (Shekar et al. 1993).

In the test of the continuous span, a linear load-deflection response was observed, without damage associated to the loading, up to the deflection limit specified at the serviceability limit state (i.e. deflection of 1/800th span length). The load at the deflection limit was 51.8 % of the peak load, and the first crack was observed at 71.4 %. Further loading resulted in yielding at the point where the load was applied closest to the abutment, followed by cracking and yielding at the adjacent intermediate support. Consequently, failure started and the test was stopped with a sequence of decreasing loads with a peak load of 3 987 kN. From the test, it was concluded that there was ductile behaviour with the load-carrying capacity being more than 10 times higher than given at the initial bridge assessment. The test of the simply supported span and its shear-related failure is further described in Section 4.7.

3D linear FE analysis of the bridge revealed consistent results in term of deflections in the test in the linear range. However, to calculate the load-carrying capacity, moment-curvature analysis using an assumption of yield lines was used and, based on in-situ tested material properties, the failure was expected at 96.6 % of the experimentally derived capacity. From this test, it can be seen that:

- The material properties and the idealization of the behaviour are of importance in bridge assessment and should be determined from the specific bridge investigated.
- Different types of analysis for determination of the load-carrying capacity can result in appreciably different outcomes and, thus, this aspect should be carefully treated during assessment. For instance, plastic analysis or linear analysis with redistribution of internal forces are useful methods to avoid undesired understated results.
- Non-structural elements (e.g. curbs) may appreciably contribute to the load-carrying capacity.

3.9 Thurloxton Underpass – United Kingdom – 1993 – Cullington et al. (1996); Daly (1994)

The Thurloxton Underpass was a 4.27 m long single-span RC slab bridge with the slab (18 000 × 305 mm^2) simply supported on retaining walls. Re-decking was due and a test was carried out for evaluation of the shear capacity in connection with revised code requirements. The bridge load-carrying capacity was assessed to shear capacity which had been reduced due to the anchorage length of the longitudinal tensile reinforcement not following the code requirements described in the British standard (Department of Transport 1990).

The slab was cut longitudinally in an approximately 1 m wide strip and loaded 950 mm from the abutment which would lead to shear failure according to the code. A steel beam was used to distribute the load along the strip width and two hydraulic jacks with ground anchors were used to produce the load. Due to the nature of the footing of the retaining wall and the loading system used, it was not possible to apply the load closer to the support. Contrary to expectations, no shear cracks were initiated during the test. Instead, flexure was shown to be critical and the slab behaved in a ductile manner, with concrete crushing in the top. The peak load was 440 kN and the test was stopped after a significantly increased deflection for a slightly reduced load. Thus, the actual load-carrying capacity was 7 % higher than the predicted shear capacity, assuming fully anchored reinforcement, and about 30 % higher than the flexural capacity.

When the bridge deck was removed, some restraint was present at the support; however, it was concluded to have had a minor influence on the capacity. Moreover, a comparative laboratory study was carried out on full-scale slabs and, with a reduced shear span, shear failures occurred for loads well above the predicted capacities. Thus, the assessment standard was concluded to be conservative, even though the longitudinal reinforcement was assumed to be fully anchored.

3.10 Lennox Branch Bridge – USA – 1999 – Alkhrdaji et al. (1999, 2000)

The Lennox Branch Bridge was a skew (15º) two-lane simply supported three-span RC slab bridge with equal spans lengths of 7.96 m (see Figure 11). The slab (7 620 × 470 mm^2) was monolithically cast-in-place, supported on piers and abutments, and with curbs on top at either side (457 × 254 mm^2). The bridge was considered to be in good condition after an inspection. It was decommissioned due to construction of a new highway. In order to demonstrate and evaluate two strengthening methods using carbon fibre reinforcing polymers (CFRP), a series of tests were carried out. All of the spans were tested to failure; two of them were strengthened to improve the flexural capacity, the first span with near surface mounted (NSM) CFRP rods at the base of the slab and the second span with externally bonded sheets.

Figure 11. The Lennox Branch Bridge (Alkhrdaji et al. 1999).

The failure tests were carried out by loading each span centrically using four hydraulic jacks transversally aligned 1.2 m apart. Under the bridge deck slab, longitudinal reaction beams between the adjacent supports were used to carry the forces transferred by rods from each jack. The test was quasi-static, meaning loading was cyclic, gradually increasing until failure was reached. All the spans failed in flexure with load-carrying capacities of 2 652 kN, 2 412 kN and 2 060 kN. For the strengthened spans, a quasi-ductile structural behaviour was observed due to yielding of the reinforcing steel and, to some extent, slippage of the CFRP sheets. In the final test without strengthening, a less stiff post-yield behaviour was revealed with the peak load being reached after relatively large deformations. In the first test, a sudden failure took place when the CRFP rods ruptured and, in the second test, failure was more gradual with a combination of delamination and sheets failing in various locations. In the third test, a typical flexural failure occurred with concrete crushing occurring after steel yielding. The purpose of the strengthening was to improve the capacity by 30 %. However, this goal was not achieved using CFRP sheets, because the type of failure mechanism involved debonding, suggesting that the full strength of the sheets was not fully used.

Theoretical analyses of the test resulted in significant underestimations of the load-carrying capacity with 76 %, 91 % and 73 % of the tested peak loads. In-situ tested material parameters were used and studies indicated only small errors associated to the skewness of the slab and the strain-hardening of the reinforcing steel. Updating the material parameters from values assumed when designing the experiment to those derived from in-situ tests yielded a 45 – 53 % increase in the load-carrying capacity. Eventual rotational restraints at the supports would lead to more favourable estimates, although it was not possible to determine the degree of restraint from the test. In the studies, the analyses were mainly based on hand calculations and no detailed nonlinear analysis was carried out to understand more fully the cause of the conservative result.

3.11 Seoul-Pusan Highway Bridge – South Korea – 2000 – Oh et al. (2002)

This bridge, located on the highway between Seoul and Pusan, South Korea, carried two lanes of traffic and was a PC girder bridge with twelve 30.0 m equally long simply supported spans (see Figure 12). The girders consisted of I-shaped precast elements (200 – 560 × 1 890) with seven prestressed wires each and a centre-to-centre distance of 2.09 m between the girders. On the top, a slab (8 100 × 180 mm^2) was cast-in-place monolithically with a strip separating the two traffic lanes and 750 mm wide curbs on either side. However, the degrees of composite action of the slab and the girders were unknown. After upgrading the highway to carry increased amounts of traffic, the bridge was used for destructive testing. This bridge was of particular interest since it was one of the most common types of bridges in Korea. An imprecise capacity assessment was expected, so it was considered beneficial to investigate the structural behaviour and load-carrying capacity through a failure test.

Figure 12. The Seoul-Pusan Highway Bridge (Oh et al. 2002).

The bridge was investigated at both the service-load and ultimate-load levels, using a loading configuration based on a tandem vehicle with 2.0 m between the wheels transversally and 4.2 m longitudinally. The loading was applied centrically in the end span and centrically over two girders and, thus, punching of the slab was avoided. Each load was produced by two hydraulic jacks coupled to strands anchored in the bedrock. The jacks loaded a steel beam, which further transferred the load to the girder.

Preloading the bridge caused the first vertical crack to form in the midspan region, at 7.3 % of the load-carrying capacity and thereafter both flexural and shear cracking took place adjacent to the loading. The subsequent failure test resulted in the concrete midstrip crushing and detaching at 50.0 % of the peak load, producing reduced stiffness and, at 92.3 %, the adjacent curb was crushed as well. This can be seen are an indication of the ultimate failure that was induced by crushing of the slab. The load-carrying capacity was 4 312 kN. As the load-carrying capacity was about ten times higher than the design load, it was concluded that the safety margin was well above the requirements. In the test, diaphragms contributed to transverse distribution of loads and this effect should be taken into account for more precise assessments of the load-carrying capacity. No theoretical analyses were described by Oh et al. (2002).

3.12 Xin Xing Tang Bridge– China – 2005 – Jiaquan et al. (2006)

The Xin Xing Tang Bridge was a continuous three-span PC box girder bridge with span lengths of 32 m, 50 m and 32 m (see Figure 13). It was a single box girder with one cell, cantilever flanges, constant height and inclined webs. Inspection of the bridge showed severe concrete cracking of the base plate and the web at one quarter and three quarter length of the interior span. Due to degradation and extension of the highway between Shanghai and Nanjing, it was necessary to replace some bridges. Two common bridge types were selected in order to investigate damage evolution, the correlation between damage and dynamic characteristics and ultimately the load-carrying capacity. Apart from the Xin Xing Tang Bridge, the Xi Cheng River Bridge (described in Section 3.13) was also included in the project.

Figure 13. The Xin Xing Tang Bridge (Wenping 2006).

The bridge was tested on its main span (i.e. the interior span). Initially, the girder was loaded with weights (i.e. piles of reinforcement) placed around the midspan. When the bridge was about to fail, water tanks were filled and used to increase the weight to the ultimate load. During the failure tests, extensive flexural cracking took place and for a load of 17 460 kN, the non-prestressed reinforcement at the midpoint of the span ruptured and the concrete crushed at the top of the girder. At this point, the deflection of the girder was about 290 mm. Due to the large deformations, the bridge deck was lifted from the supports at the abutments and the structural behaviour changed. Acting in a similar way to a simply supported beam, the bridge was further loaded until the load-carrying capacity of 18 320 kN was reached, producing a deflection of 353 mm. Thereafter, the test was stopped without any structural collapse and consequently, it can be concluded from the test that the Xin Xing Tang Bridge suffered a ductile failure. Further details about the test and theoretical evaluation can be found in Wenping (2006).

3.13 Xi Cheng River Bridge – China – 2005 – Jiaquan et al. (2006)

The Xi Cheng River Bridge was a simply supported RC girder bridge with six spans of length 20 m (see Figure 14). The deck was 13.0 m wide in total, and consisted of six T-shaped girders connected with diaphragms and to the slab at the top. The girders were supported on abutments. The objectives of the investigation of the Xi Cheng River Bridge were the same as for the bridge summarized in Section 3.12.

Figure 14. The Xi Cheng River Bridge (Wenping 2006).

The bridge was loaded with weights made of reinforcements hoops, placed around the midspan. In the test, the outermost girder failed at 10 070 kN, caused by concrete crushing at the top of the slab and rupture of the reinforcement in the midspan. The bridge was further loaded and the entire deck collapsed at 11 120 kN. However, during the loading, extensive concrete cracking took place and failure was imminent. In Wenping (2006), additional information about the test was presented.

3.14 Nanping Bridge – China – 2007 – Zhang et al. (2009, 2011b, 2013)

The Nanping Bridge had three equally long spans (13.30 m) and was a skew (25º) RC beam bridge carrying road traffic in two lanes (see Figure 15). The deck was simply supported on piers or abutments in each span, and it was composed of six parallel precast inverted U-shaped channel elements (1 000 × 340 mm^2) bolted together. The edge beams also had 200 mm wide curbs. Thus, a system of 340 mm wide girders and a 300 mm thick slab was created. An inspection of the bridge revealed severe deterioration, involving considerable carbonation of the concrete along with reinforcing steel corrosion (12.1 %), which led to concrete spalling and extensive cracking of the beams. As it needed rebuilding, the 43 year-old bridge became redundant and available for destructive testing. The primary aim of the experimental study was to investigate the bridge behaviour at service-load levels (including overloading) as well as to determine the ultimate capacity and failure mechanism.

Figure 15. The Nanping Bridge (Zhang et al. 2011b).

At service-load level, a load configuration corresponding to the design vehicle specified by the Chinese design code (Ministry of Transport of the People's Republic of China 1989) was investigated, with the load placed in the most adverse position with regard to flexural moment (i.e. in the midspan). The load from a tandem with axles 1.4 m apart and two other such tandems were simulated. A reaction frame, composed of a system of steel beams, was anchored in a steel box filled with gravel below the bridge and one hydraulic jack was used at each position to represent the wheels. In the subsequent failure, the loading system was rearranged with a load distribution beam positioned transversally on the bridge in the midspan with two hydraulic jacks producing the load.

Loading the bridge with hydraulic jacks at the service-load level showed a structural behaviour consistent with loading using actual trucks. The bridge behaviour was linear elastic to a load level corresponding to 36.2 % of the peak load (i.e. 4 tandems), causing significant damage to the bridge and, thus, a nonlinear behaviour was observed at 54.3 % of the peak load (i.e. 6 tandems). Moreover, the permitted deflection at SLS (*L*/600), according to the design code (Ministry of Transport of the People's Republic of China 1989), was reached at 68.8 % of the peak load (i.e. 7.6 tandems). After simulating overloading by vehicles, the bridge was incrementally loaded to failure. After extensive yielding of the reinforcing steel (yield strength reached at 92.3 % of the peak load) and concrete crushing of the top flange, the peak load was obtained at approximately 2 600 kN, compared with a predicted load-carrying capacity of 2 400 kN. The maximum deflection measured was 240 mm, however, with a high variation between the different elements due to the bolted connections not being able to transfer the forces that occurred in the test. In addition to the predominant flexural failure, severe inclined cracking was observed in the web of the beams adjacent to the support. Based on this study, which was mostly experimental, the following conclusions can be drawn:

- Overloading of vehicles can be precisely simulated using a loading system consisting of hydraulic jacks.
- The load-carrying capacity with regard to flexural failure can be precisely predicted. At the same time, design codes for bridges can be conservative, with regard to the requirements at both SLS and ULS. For instance, the load-carrying capacity corresponded to the weight of 13 design vehicles.
- A ductile failure occurred. However, unexpected shear-related cracking also took place, which resulted in a sudden collapse of the structure, aided by the lack of shear reinforcement.

3.15 Paint Creek Bridge – 2010 – USA – Stillings (2012)

The Paint Creek Bridge was a skew (15º) PC girder bridge with two lanes for road traffic in three 14.58 m long spans (see Figure 16). In total, nine simply supported precast box girders (914 × 533 mm^2), placed side-by-side, formed the bridge deck. No material parameters were documented and only the concrete compressive strength (about 70 MPa) was determined from material testing. The structural condition of the bridge was, based on visual inspections, rated as being poor with occurrence of concrete spalling on the extreme web girders and corroded shear reinforcement observed (see Figure 16). Consequently, the bridge was scheduled for replacement. The behaviour of individual box girders is usually well understood, however, how the system of adjacent girders acts as a whole when subjected to ultimate loads was less well researched. Thus, tests were planned with the objectives to investigate the response of the system under ultimate loads and to determine the deterioration's effect on the ultimate capacity.

Figure 16. The Paint Creek Bridge (Stillings 2012).

Static tests were carried out on each span, although the results were presented for only the first two consecutive spans. The test on the first span was conducted on bare girders (concrete overlay removed) with some girders intentionally damaged by cutting some of the prestressing strands. These deficiencies were intended to mimic corroded strands, a typical form of deterioration found in reality for this type of bridge. The results were compared with those from the second span, where the test was conducted in the as-built, unmodified condition. Three hydraulic jacks were placed symmetrically along the bridge width and produced displacement control loading centrically in each span. Longitudinally-aligned reaction beams transferred the forces to the adjacent supports.

In the tests, moderate crushing of the top flange was noted at load-carrying capacities of 2 671 kN and 2 686 kN for the first and second span, respectively. The bridge behaved in a ductile manner and no collapse occurred. Predictions based on the modified compression field theory (MCFT), using Response-2000 software (Bentz 2000), were consistent with the tested capacities, with approximately 8 % underestimation for the first span and 1 % overestimation for the second span.

Comparing the failure loads between the two spans, there was only a small difference in capacity although there were structural differences (damage induced by cutting prestressing strands and with/without overlay). However, the structural differences impacted the stiffness of the structure. Overall, the stiffness was greater than predicted, presumably due to partial fixity of the girders' supports. According to Stillings (2012), deviations between as-built and as-designed specifications together with a variation of the elastic modulus (in the range of ± 20 %) cannot be solely responsible for the difference in bridge stiffness. However, it can be noted that information regarding the residual prestressing forces was not objectively obtained for this particular bridge and certain values were assumed (i.e. loss of 16 %). Therefore, it is not clear whether the prestressing forces were correctly taken into consideration in the analysis.

3.16 Hanpyung, Ansan, Dongcheon and Bangjimoon Bridge – South Korea – 1987, 1987, 1997 and 1999 – Byun et al. (1989); Song et al. (2002); You et al. (1998)

Table 2 summarizes the geometric characteristics of four RC bridges in South Korea tested to failure in order to understand further the structural behaviour of these bridge types. The specified spans lengths of Bridges 1 and 2 should be considered as approximate, since inconsistencies in reported information have been identified. The deck of Bridge 1 was composed of a slab and Bridges 2 – 4 were composed of multiple girders connected with a slab at the top. All of the bridges were monolithically cast-in-place and supported on piers and abutments. They were taken out of service with varying degree of deterioration. For instance, the piers of Bridge 3 were settled, the curbs were severely damaged, but only small cracks and minor concrete spalling were observed on the rest of the deck.

The bridges were tested at service-load and ultimate-load levels. Only Bridge 4 was tested to failure under unmodified conditions using eccentric loading to two adjacent girders on one side of the deck. The deck of Bridge 1 was cut transversally to obtain simply supported conditions and, for Bridges 2 and 3, the decks were cut longitudinally in order to apply centric loading to two girders and one girder, respectively. Moreover, the curbs and railings were removed from Bridges 3 and 4 in order to reduce the uncertainties in the analysis of the test. In all tests, the loads were cyclically applied in the midspans using hydraulic jacks with a slight variation in the loading system. For Bridge 1, a reaction beam was aligned longitudinally under the deck to transfer the forces to the piers; for the other bridges, bedrock anchors were used.

It was concluded that all the bridges failed due to a ductile flexural failure. However, the failure mechanism was not further described for any of the tests. The ratios of tested and calculated capacity were 3.0, 0.72, 1.7 and 4.2, indicating a high degree of uncertainty in the evaluation of the test using design codes. Uncertainties associated with geometry and material properties were mentioned as possible sources of the differences between theoretical and experimental results. However, no further investigations have been reported with regard to these aspects, although this would be useful in order to improve understanding. Bridges 3 and 4 were also analysed more thoroughly using nonlinear FE analysis (Song et al. 2002). This study showed that it was possible to model the behaviour of these bridges precisely up to the load level of the experimental failure. Notably, restraints at the supports were to be introduced in order to obtain consistent results.

4 Shear-related failure modes

4.1 Boiling Fork Creek Bridge – USA – 1970 – Burdette and Goodpasture (1971, 1972, 1973, 1974)

The Boiling Fork Creek Bridge was a skew (20º) PC girder bridge for two-lane traffic (see Figure 17). It had four equally long (20.12 m) simply supported spans. Four precast PC I-shaped girders (118 – 559 × 1 143 mm^2) with a centre-to-centre distance

Table 2. Geometric characteristics of the Hanpyung Bridge, Ansan Bridge, Dongcheon Bridge and Bangjimoon Bridge.

Item	1	2	3	4
Bridge name	Hanpyung	Ansan	Dongcheon	Bangjimoon
Span lengths	6 × 6.0 m	4 × 12.0 m	11 × 12.0 m	6 × 9.2
Support conditions	Continuous	Simply supported	Simply supported	Simply supported
Slab dimension	4 000 × 260 mm^2	6 830 × 260 mm^2	5 050 × 250 mm^2	6 500 × 260 mm^2
Curb dimension	-	360 × 490 mm^2	250 × 270 mm^2	N/A
Girder dimension	-	450 × 1 070 mm^2	450 × 1 030 mm^2	350 × 890 mm^2
Number of girders	-	5	3	4
Girder distance (c/c)	-	1.40 m	1.80 m	1.60 m

of 2 692 mm (varies) were supported on piers and abutments. A RC slab (8 530 × 178 mm^2) with curbs at the edges (991 × 178 mm^2) was cast-in-place on top of the girders with interaction through stirrups across the interface between the elements. This bridge was a part of a research project that included four bridge failure tests. The background of the project and the test procedure for these bridges are described in Section 3.4.

The bridge was tested to failure in three of its spans. In the first tests, the bridge deck slab failed due to local punching failures. This failure mode was not intentional and, thus, it was not extensively reported, although it may be of interest. In the first test carried out in the end span, cracking in the slab-girder interface and diagonal cracking in the girder adjacent to the abutment was observed at 4 226 kN. It was concluded that a shear failure of the girder was imminent, but a sudden punching failure occurred with a load of around 4 350 kN. For the second test in the interior span, the load was located in a way to obtain higher shear in the girders. The first crack occurred at 4 003 kN but even here a local, unexpected failure took place at a load of 5 151 kN. In the third test, carried out in the other end span, a horizontal concrete pad was cast on top of the slab in order to avoid a local failure. The structural response was similar to the previous test with the stirrups sheared between the precast interior girders and the slab at 83.3 % of the peak load, followed by moment redistribution to the girder and concrete crushing in the top. Diagonal cracks were formed in the web and a brittle shear failure took place in the two interior girders at a load of 4 620 kN (see Figure 18). Theoretical analysis showed a load-carrying capacity that was 110 % of the tested one, meaning an understated predicted value. However, the evaluation was limited to the flexural capacity and assumed composite action between the different structural elements.

From the studies of the Boiling Fork Creek Bridge, the following can be learnt:

- Punching has to be taken into account for concentrated loading in order to avoid unexpected, brittle failures. However, in this test, the punching failure that occurred remained local.
- The interaction between different structural parts was critical in this test and needs to be carefully treated. Ultimately, this was one of the explanations for the brittle shear failure of the girders in this test.
- Although the bridge test resulted in a shear failure, only estimated flexure-related capacity was reported. Thus, it is concluded that the shear capacity was not completely taken into account, which led to unexpected and understated results.

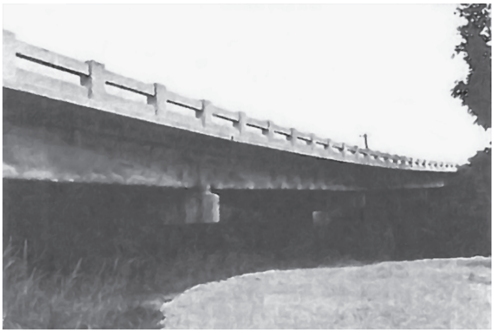

Figure 17. The Boiling Fork Creek Bridge (Burdette & Goodpasture 1974).

Figure 18. The Boiling Fork Creek Bridge after shear failure of the PC girder (Burdette & Goodpasture 1973).

4.2 Glatt Bridge 2 – Switzerland – 1975 – Weder (1977)

The Glatt Bridge 2 was a single-span portal frame PC slab bridge with a free span length of 12.5 m (see Figure 19). The bridge deck was, in total, 21.0 m wide and carried two lanes of road traffic on an interior slab (16 000 × 353 mm^2). Slender cantilever slabs with curbs (250 × 300 mm^2) at the edge carried the pedestrian lanes. A Freyssinet post-tensioning system was used to prestress the deck longitudinally using 62 cables, and transversally using 15 cables. The bridge was monolithically cast-in-place with concrete having a compressive strength of 87.5 MPa (CoV: 14 %), which was about 25 MPa higher than assumed at design. Due to building of a new highway, the bridge was taken out of service and an experimental programme was designed to investigate its condition and load-carrying capacity.

Figure 19. The Glatt Bridge 2 (Weder 1977).

Inspection revealed the bridge to be in good condition with, for instance, very little corrosion on the prestressing cables. The residual prestress forces were determined by removing 18 cables, which produced losses of 0 to 30 % in relation to the initial prestress level. Also, fatigue tests were carried out on the strands. For a constant stress amplitude of around 240 MPa, four strands failed after less than two million cycles, while eight strands withstood additional cycles.

The load-carrying capacity of the bridge was predicted to be about 20 000 kN but, due to difficulties in producing such a high load, it was decided to cut the slab longitudinally in two 0.90 wide strips. For one of the strips, the built-in end supports were kept and for the other, the ends were cut free to obtain a simply supported strip. The loads were applied using eight pairwise jacks that were supported by four transversally aligned reaction beams anchored in the slab outside the strips. Incremental loading was carried out up to a load of 448 kN when a sudden shear-related failure occurred adjacent to the abutment for the strip which retained its supports. A crack was formed between the support at the bottom towards the top of the slab, a chunk of concrete spalled in the compression zone at the bottom and the strip dropped. It was found that the weakness in the section was caused by a longitudinal cable that was misplaced and had entered the strip at an angle. However, the bridge was unloaded, then reloaded using a new loading sequence. This time, the strip resisted a slightly lower load and flexural failure occurred in the midspan by concrete crushing at the top of the slab. Due to the unexpected behaviour of the slab, the theoretically determined load-carrying capacity of 487 kN was never reached, although the tests showed there were double the predicted action effects of permanent loads and live loads. Thus, it can be concluded from the outcomes of this test that the modification had a crucial impact on the bridge behaviour and testing the bridge without modifications would have led to transverse redistribution of internal forces and a different failure mode. In addition, it can be concluded that the bridge exhibited a generally ductile behaviour and, taking into account the residual load-carrying capacity, the bridge was robust. The test of the second strip proceeded as expected, with a flexural failure occurring at 399.5 kN, a load which the theoretical capacity overestimated by 1.3 %.

4.3 Brønsholm Bridge – Denmark – 1977 – Pedersen et al. (1980); Vejdirektoratets broafdelningen et al. (1979)

The Brønsholm Bridge was a skew (58º), continuous two-span PC slab bridge with equal span length of 20.39 m and additional 3.48 m cantilevers at the bridge ends (see Figure 20). The deck slab was 18 340 mm wide and, on average, 430 mm thick and was supported on rows of six columns each. It carried traffic loads on two road lanes and two bicycle and pedestrian lanes. Using a system from Freyssinet, the monolithically cast-in-place slab was prestressed longitudinally; this was one of the first Danish bridges to use this prestressing technique. At the time of construction, concrete samples were tested giving an average value of the compressive strength of 39.3 MPa, compared with 46.0 MPa from samples taken from the bridge 21 years later. A corresponding comparison of the prestressed reinforcing steel gave 1350 MPa and 1450 MPa for the yield strength, and 1600 MPa and 1594 MPa for the tensile strength. Inspection of the bridge revealed no signs of deteriorations and, at demolition, the quality of the tendon grouting was shown to be satisfactory with no corrosion of the prestressed reinforcement. The bridge had to be demolished due to realignment of the highway.

It was decided to investigate the decommissioned bridge since only a few bridge failure tests had been reported and the conditions assumed in bridge design and assessment needed to be verified, for instance, with regard to the degree of redistribution of internal forces at the ultimate limit state. Moreover, the degree of deterioration of the bridge and the residual prestressing force were of particular interest. Before the tests, the deck was cut longitudinally into a 2.8 m strip on the loaded span because of budgetary constraints. The intention of the change to the structural system was to simplify the boundary conditions and reduce the influence of skewed supports, thus facilitating the measurements and evaluation. Introducing a strip also resulted in eccentrically loaded columns at the support and, thus, a temporary support was created using a steel beam along the centreline in order to subject the slab to flexure and shear only.

Figure 20. The Brønsholm Bridge after collapse (Pedersen et al. 1980).

The loading was carried out using two transversally aligned hydraulic jacks, 1.43 m apart, 10.49 m from the end support. A transverse steel beam with rods anchored in the bedrock supported the loading system. At the end support, a hydraulic jack was installed connected to the temporary column which provided data about the reaction forces. Using incremental loading, with intermittent data recording, the slab was loaded up to the theoretical level for flexural cracking. Thereafter, the bridge was cyclically unloaded and reloaded until no additional cracks were formed and the existing cracks stopped growing (3 cycles), followed by an increasing load until failure. The flexural capacity was reached at the midspan, indicated by yielding of the reinforcing steel and concrete crushing, and redistribution of internal forces took place. However, at 1 529 kN, the slab failed in a brittle shear failure at the load application point with a lack of residual capacity leading to the collapse. The part of the slab located towards the end support fell to the ground after an additional flexural and shear failure between the loading and the intermediate support (see Figure 20).

Based on a plastic analysis, and assuming yield hinges at both the midspan and the intermediate support, a load-carrying capacity 2.6 % higher than the tested capacity was calculated. However, this result should be considered in the light of the reduced rotational capacity associated with the changed boundary condition (i.e. the slab strip in the loaded span and full bridge width slab at the intermediate support) and the absence of stirrups in the slab. Despite the outcome of the bridge test, no shear capacity analyses were reported.

Based on the findings from testing the Brønsholm Bridge, the following can be learnt:

- The flexural capacity can be accurately predicted. Plastic analysis can be beneficial to improve the load-carrying capacity in comparison to a linear elastic analysis. However, the shear strength has to be carefully treated, taking into account the actual internal forces after redistribution and also the flexure and shear interaction. This particular test showed the consequences of a lack of knowledge of shear forces causing the structure to collapse.
- The test was carried out after modifications to the boundary conditions of the slab, removing the ability to distribute loads transversally. Thus, a more robust structure, less prone to dramatic collapse, can be expected for an unmodified deck. Consequently, it is of importance to perform tests under realistic conditions in order to draw further conclusions about the overall behaviour.

4.4 Stora Höga Bridge 1 – Sweden – 1989 – Plos (1990, 1995); Plos et al. (1990)

The Stora Höga Bridge 1 was a single-span RC portal frame bridge with a free span length of 21.0 m between supporting walls (see Figure 21). The deck carried traffic in two lanes and consisted of a monolithically cast-in-place slab (4 500 × 650 – 1150 mm^2) with a parabolically-varying thickness and curbs (350 × 395 mm^2) on the sides. The highway underneath the bridge was upgraded and, thus, several bridges built in 1980 were scheduled for demolition, for instance, the bridges described in this section and in Section 4.5. The primary aim of the two bridge tests was to investigate the accuracy of models for calculating the shear resistance, at that time under discussion for implementation in the forthcoming European standard. In addition, the tests were to be used for evaluation of improved methods to determine the load-carrying capacity based on FE analysis. Moreover, a strengthening system using steel plates longitudinally glued to the bottom side of the slab was to be investigated on the Stora Höga Bridge 1. The purpose of the strengthening was to improve the flexural strength and produce a shear-related failure.

The loading of the bridge was carried out using four hydraulic jacks, supported by rods through the slab and anchored in the bedrock. The loads were transferred to the bridge through two transversally located load distribution beams. The loads were applied at four locations in line (two for each load distribution beam), 4.00 m from the supporting wall. In order to record measurements and to document the formation of cracks, the bridge was loaded in intervals of 400 kN up to the peak load of 4 600 kN. Thereafter, the load decreased to 4 500 kN when the slab failed in shear between the supporting wall at the bottom and the load application at the top (see Figure 22). The failure was brittle and a drastic load drop occurred. At 52.2 % of the peak load, an increased crack width was noticed for some flexural cracks already identified before the test. Moreover, cracks were observed at the support, in the top of the slab, at 56.5 % of the peak load, and the first shear crack was formed adjacent to the load application point at 87.0 % of peak load.

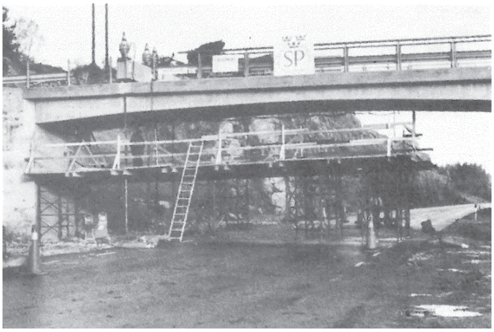

Figure 21. The Stora Höga Bridge 1 (Plos & Gylltoft 1995).

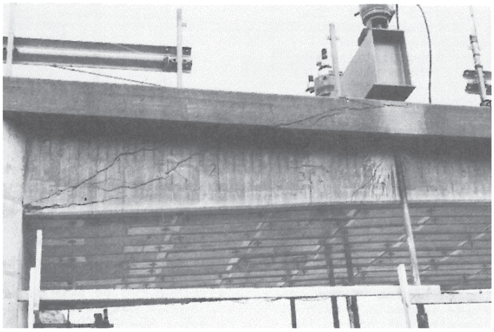

Figure 22. The Stora Höga Bridge 1 after shear failure between the applied load and supporting wall (Plos & Gylltoft 1995)

In comparison to the test, the theoretically determined load-carrying capacity was about 2.1 times higher when based on in-situ tested concrete parameters and the shear model in the Swedish standard (BBK 79 1987). The prediction using the code was deemed to be consistent with the test within the normal scatter of results from small-scale experiments and, thus, the scale effects are reasonably accounted for by the theoretical models. However, a possible explanation for the conservative result, along with the fact that the shear resistance model is not very precise, could be the influence of strengthening on the shear behaviour and, consequently, the shear capacity. Analyses of the test also indicated that strengthening the bridge was successful, increasing flexural resistance about 2.8 times, producing a load-carrying capacity of 6 000 kN for the actual test setup (Täljsten 1994).

The bridge test was simulated using nonlinear FE analyses both with the discrete and smeared crack concepts (Plos & Gylltoft 1995). Using the smeared crack concept, the crack formation was consistent with the test, the slab deflections were appreciably overestimated and, due to numerical problems, the failure mode was not predicted correctly. The modelling of boundary conditions and the material properties were some explanations for the differences in the observed and modelled structural behaviour and, thus, these aspects should be carefully treated in the model. Despite this, nonlinear FE simulations can be regarded as giving a better understanding of structural behaviour, and more precisely modelling than is achievable using analytical models.

4.5 Stora Höga Bridge 2 – Sweden – 1989 – Kjellgren and Bergström (1990); Plos (1990, 1995); Plos et al. (1990)

The Stora Höga Bridge 2 was a single-span PC portal frame bridge with a free span of 31.0 m (see Figure 24). It was a two-lane road bridge and was included in the project further described in Section 4.4. Two parallel girders (1 660 × 1 160 – 1 460 mm^2) with a centre-to-centre distance of 2 480 mm were prestressed, using a BBRV post-tensioning system. The girders were connected at the top with a RC slab (8 420 × 360 mm^2) with curbs (300 × 388 mm^2) at the edges. The bridge was monolithically cast-in-place with vertical supporting walls.

In order to examine the same objectives and using a similar loading procedure as the non-prestressed portal frame bridge (see Section 4.4), this bridge was loaded to failure. In order to reduce the load needed to reach failure, the bridge was cut longitudinally approximately along the centreline of the deck. Three simply supported load distribution beams were positioned parallel to the bridge with an intermediate distance of 750 mm longitudinally, resulting in loading 4.75 m from the edge of supporting wall.

In the test, extensive flexural cracks were formed in the region adjacent to the loading point, in addition to a few flexural-shear cracks. At a load 30 % higher than the predicted load-carrying capacity according to the shear model described in the Swedish standard (BBK 79 1987), a sudden, unexpected failure occurred at the support. The load was 8 500 kN and the lower part of the girder punched into the wall, followed by a drop in the load to 6 000 kN. As the boundary conditions and redistribution of internal forces had then changed, the loading increased to 6 300 kN (5 % increase), resulting in an almost doubled deflection under the load, coupled with combined shear failure and concrete crushing in the slab (see Figure 24). For the new support conditions, the failure took place for a considerably lower load than predicted using the shear and flexural resistance models in the Swedish standard (BBK 79 1987). Thus, this bridge test provides the following learning outcomes:

- In order to model the structural behaviour accurately and to avoid unexpected failure, the interaction to the surrounding elements (and possible changes) should be taken into account when assessing a specific structural element. In this study, the capacity of the supporting walls was not verified prior to the test, resulting in an unexpected failure mode.
- The presence of interaction between flexure and shear should be taken into account when determining the load-carrying capacity, otherwise predictions of that capacity may be unsafe.

Figure 23. The Stora Höga Bridge 2 (Plos & Gylltoft 1995).

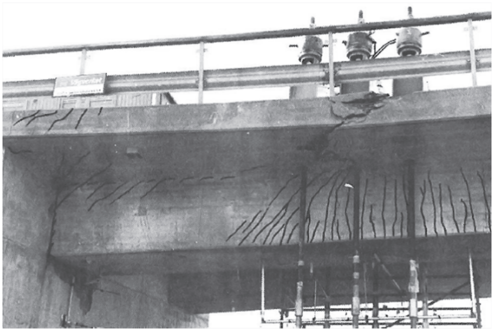

Figure 24. The Stora Höga Bridge 2 after shear failure in the supporting wall and deck slab (Kjellgren & Bergström 1990)

4.6 Batavia Bridge – USA – 1991 – Aktan et al. (1992); Miller et al. (1994)

The Batavia Bridge was a skew (30º) continuous three-span RC slab bridge carrying two lanes of traffic (see Figure 25). The length of the end-spans was 9.754 m and the interior span was 12.19 m. The deck was a 11 125 mm wide slab, supported on intermediate piers and keyed to abutments at the ends. At an age of 38 years, the bridge was to be demolished as a result of its poor structural condition. Prior to testing and the removal of asphalt overlay, non-destructive tests were carried out in order to explore their effectiveness at damage detection, with results, outside the scope of this paper, reported elsewhere (Aktan et al. 1992). When the concrete slab was exposed after removal of the overlay, it was found that the concrete deck was in a relatively good condition over the traffic lane but severely deteriorated in adjacent regions. The deterioration was associated with material degradation due to freeze-thaw cycling, alkali-silica reaction and corrosion.

Figure 25. The Batavia Bridge (Aktan et al. 1992).

The testing was aimed at gaining insight into the bridge behaviour and subsequently using the information to verify existing assessment methods. In order to replicate the front tandem of the design vehicle HS20-44 (AASHTO 1989) passing on one lane only, two 1525 × 690 × 760 mm RC blocks were cast-in-place on top of the slab to distribute the load from hydraulic jacks. The load was statically applied in the end span, 3.9 m away from the abutment, by pulling through rock anchors grouted into the bedrock. The critical location of the loading was determined using FE analysis.

The test was evaluated based on measurements made at different critical points observed during testing. According to Miller et al. (1994), these critical points, called limit states, were related to: (1) changes in rotational response, (2) changes in load path and (3) failure initiation. The first limit state occurred between 1 000 kN and 1 300 kN. Changes in rotational stiffness of the slab at the intersection with the abutment and pier cap were observed at the lower and upper limit of the previously mentioned range, respectively. Up to 2 200 kN, a change in the maximum deflection path was observed from “perpendicular to the traffic lane, to parallel to the skew” (Miller et al. 1994). The last limit state begun with failure initiation starting immediately after 2 200 kN and ended with a combined shear and punching failure of the concrete block at 3 200 kN. The failure started in the deteriorated region of the slab and propagated towards the healthy part. Measurements of reinforcing steel strains were used as indicators of imminent failure when, for example, large jumps were registered. However, yielding only occurred close to the failure load in regions under the loading blocks and over the piers.

Analytical calculations were carried out in an attempt to evaluate the adequacy of the calculation methods. It was found that nonlinear FE analysis was able to improve both the evaluation of load effects and supply resistance over the more simplistic methods such as effective-strip and linear FE analysis, respectively. The nonlinear FE analysis predicted the flexural yielding well, implying about 25 % higher loads than predicted by the American standard (AASHTO 1989). The bridge’s actual failure was assessed against the resistance model for shear and punching described by AASHTO (1989); it overestimated the capacity by more than 65%.

The following can be learnt from this study:

- Existing bridge rating methods were not able to “save” the bridge from demolition, although the test revealed a load-carrying capacity equivalent to 22 HS20-44 design vehicles.
- Accounting for real boundary conditions is of great importance when determining the actual bridge response.
- The prediction of the punching-shear failure mode according to American standard (AASHTO 1989) was found to be unreliable. This is because the deterioration of the slab had not been appropriately incorporated into resistance models, and thus a higher load was expected. In such cases, a 30° angle (as shown in the test) of the failure plane should have been assumed in the calculation, reportedly reducing the punching shear capacity by almost 50% (Miller et al. 1994).

4.7 Niobrara River Bridge – USA – 1992 – Azizinamini et al. (1994a, b); Shekar et al. (1993)

The geometry of the Niobrara Bridge and the test procedure have already been described in Section 3.8, as a part of a broader experimental programme. In the failure test of the simply supported span of the bridge, no apparent damage was observed at a load level of 78.0 % of the peak load, implying a maximal deflection equal to the deformation limit at the serviceability limit state (i.e. deflection of 1/800th span length). However, strain measurements of the reinforcing steel located underneath the load indicated a tendency for nonlinearity although there was no significant yield plateau. Under further loading, cracks were formed, the longitudinal reinforcing steel yielded at the bottom of the slab and the stiffness was reduced with greatly increased deformations. Although most of the flexural capacity was being used, and the load was more than four times the load-carrying capacity according to the initial assessment, a sudden failure occurred at 2 225 kN with punching from the outer load application towards the curb. Due to yielding of the longitudinal reinforcement in the experiment, a moment-curvature analysis assuming a yield line was carried out, giving a load-carrying capacity of 1 930 kN.

This study provides important learning outcomes. The flexural failures can be predicted relatively accurately, but other more brittle failure modes, such as punching, should be considered with caution due to the lack of warning. In this case, no analysis of the shear or punching capacity was carried out (or at least not reported) and an unexpected failure took place. Thus, the experiment provides rare, but unused, information that could be used for evaluation of assessment methods.

4.8 Smedstua Bridge – Norway – 1996 – Statens Vegvesen (1998)

The Smedstua Bridge was a continuous three-span RC slab bridge supported on piers and abutments with span lengths of 11.00 m, 16.30 m and 11.00 m (see Figure 26). The deck consisted of an approximately 6.0 m wide slab with a slender cantilever slab on each side (2.0 m and 3.0 m wide), with curb at the ends. On one side, there were two traffic lanes and on the other side there was a pedestrian lane. Due to widening of the highway running under the bridge, it was necessary to extend the distance between the piers. However, due to economic reasons, it was decided to replace the bridge. Before demolishing it, several tests were carried out to investigate the bridge's behaviour at service-load and ultimate-load levels.

Figure 26. The Smedstua Bridge (Statens Vegvesen 1998).

To investigate the bridge at ultimate-load level, a 6.9 m wide and 12.5 long container, centrically positioned in the traffic lane in the interior span, was gradually filled with gravel. At the same time, counterweights were placed on the deck at the abutments in order to avoid lifting. First, the container was completely filled up, providing a force of 8 140 kN. Since the load-carrying capacity was not reached, the counterweights were stepwise removed at the end-supports. After the first counterweight was removed, a small lift occurred and a few seconds after removal of the second counterweight, the bridge deck slab collapsed (see Figure 27). The failure was concluded to be as a result of a combination of flexure and shear failure and it took place about 4.0 m from the intermediate support. Although the final failure was dramatic, the bridge behaved in a ductile manner with concrete cracking and yielding of the longitudinal reinforcement.

At failure, the load was about 4.9 times higher than the traffic load assumed at bridge design and thus it can be concluded that there was a considerable safety margin. The theoretical evaluation of the bridge using the Norwegian design standard (NBR 1992) indicated the shear to be critical with an estimated capacity of 43 % of the tested peak load. Based on the yield strength of the reinforcing steel, the flexural capacity was predicted to be reached at 89 % of the tested capacity, compared with considerable yielding observed at 94 %. Thus, it can be concluded that the shear capacity was very conservatively predicted. Moreover, the flexural capacity was underestimated, but this can be explained by the ability of the structure to redistribute the internal forces from highly stressed regions to those less stressed. The loading procedure, that of having a container distributing the load on the bridge, would also have some impact on the outcome of the test not accounted for in the analyses.

Figure 27. The Smedstua Bridge at collapse (Statens Vegvesen 1998).

4.9 Barr Creek Bridge – Australia – 1998 – Haritos et al. (2000)

The Barr Creek Bridge was a two-lane continuous four-span RC slab bridge with the end-spans and interior spans having lengths of 3.66 m and 4.57 m, respectively (see Figure 28). The deck was 6 720 mm wide (excluding curbs) slab and monolithically cast-in-place continuously over piers with end-abutments providing rotational restraint. The structural condition of the bridge was rated as being good, however spalling of the concrete cover due to accelerated corrosion was reported on the columns in the splash zone of the creek. Therefore, the bridge was taken out of service in 1971 and, prior to demolition, was wired with instrumentation and tested at the serviceability and ultimate-load limit states, with the aim of improving analytical methods for bridge rating and assessment. Both dynamic and static tests were carried out.

Figure 28. The Barr Creek Bridge (Haritos et al. 2000).

The static tests were based on a test setup using the tandem vehicle T44 (Austroads 1996b). The loads were produced by hydraulic jacks on top of the bridge and, due to poor soil conditions, a steel frame attached to the piers was used for the reactions. The static test was divided into the following loading schemes: (1 and 2) single-lane loading in the middle of the end-span (span 1) and the adjacent interior span, to about 250 kN including a dynamic factor of 1.3; (3 and 4) as previous loading but with the end support changed into a hinged connection by cutting the top reinforcement; (5) simultaneous single-lane loading in span 1 and span 3 to initiate failure and (6) double-lane loading in span 4 to failure.

FE analyses were carried out at serviceability-load level and compared with results obtained from static load tests. There was good agreement although internal spans were more flexible than initially believed, mainly because of their poor condition. Prior to the test, the deck overlay was stripped to expose the concrete surface, which revealed concrete spalling at the intermediate supports. No differences were observed between load cases with and without the introduction of saw cuts, indicating a rotational free connection at the support rather than a fixed one. The same conclusion was drawn based on analyses of the dynamic test. In the final test, a combined punching and shear failure took place with the load-carrying capacity being 3 210 kN. In order to determine the load-carrying capacity theoretically, the load configurations in tests 5 and 6 were analysed based on beam analysis with plastic redistribution as well as yield-line analysis. In all cases, the analytical results were conservative compared with test results; these underestimates (e.g. 12 % for the yield-line analysis) were reportedly due to the influence of membrane effects. At the same time, from the report by Haritos et al. (2000), it can be concluded that the tests had not been investigated using models for punching and shear resistance.

4.10 Baandee Bridge – Australia – 2002 – Pressley et al. (2004)

The Baandee Bridge was a continuous five-span RC slab bridge carrying road traffic in two lanes with spans of lengths 7.39 m and 7.62 m for the end-spans and interior spans, respectively (see Figure 29). The slab (8 530 × 305 mm^2) was cast-in-place on piers and had curbs (610 × 305 mm^2) on either side. Due to realignment of the highway, two bridges of this type were redundant and, with the aim of developing more accurate methods to assess the load-carrying capacity of slab bridges, one of the bridges was destructively tested. However, the other bridge was used for testing material parameters which were assumed to be identical to the tested bridge.

Figure 29. The Baandee Bridge (Pressley et al. 2004).

Based on the experience that the flexural and punching capacity are often critical when assessing the capacity of this type of bridge, the experimental programme was designed to investigate these failure modes. The formation of cracks was studied at service-load levels for comparison to requirements at SLS. For comparison to the procedure for bridge assessment, the test setups were based on the design vehicle T44, composed of a tandem with the axles 1.2 m apart and a wheel centre-to-centre distance of 1.8 m (Austroads 1996b). Two flexural and punching tests were carried out: the flexural tests with loading in the middle of either end spans and the punching tests with loading centrically around a pier at the intermediate supports (axle distance increased to 1.8 m). The load was produced by hydraulic jacks at each loading point with ground anchors thread through the structure.

First, the punching tests were carried out. The first test resulted in a sudden failure at 3 540 kN, when the pier between the load application punched through the slab. The second test was stopped at 3 460 kN due to a functionality problem associated with one of the hydraulic jacks. In the two tests, similar cracking was observed. This was a considerably higher capacity than the 1 200 kN predicted by the Australian standard (Austroads 1996a).

In the flexural tests, the structural behaviour of the bridge was more ductile, with formation of flexural cracks and yielding of the reinforcing steel adjacent to the loading, and subsequent formation of flexural cracks at the intermediate support. However, after a period of almost constant loading with increased deflection, a sudden, unexpected shear failure took place. The critical shear crack was transversally aligned between the load and the intermediate support with a change in direction towards the abutment beside the loading point. In the first test, in which one design vehicle was simulated, the failure load was 2 260 kN and in the second test, with two parallel design vehicles, the failure load was 3 540 kN. Structural analyses with various levels of complexity were carried out prior to the test. Expressed as a percentage of the experimental load-carrying capacities for the two tests, the following results were obtained: (1) 42 % and 37 % using linear elastic analysis, (2) 96 % and 81 % using plastic analysis assuming the formation of plastic hinges at the critical section in the span and at the intermediate support and (3) 101 % and 89 % using nonlinear FE analysis. In these analyses, the shear capacity was not taken into account and the calculated capacities associated with shear for the two testes were later calculated to be 115 % and 96 % of the peak load (Candy et al. 2004).

From the four tests, the following can be learnt:

- Models to determine the punching capacity can be very conservative. However, a local analysis with consideration of only one pier can be improved by taking into account the actual distribution of action effects more precisely.
- Both plastic and nonlinear structural analysis are able to predict the flexural capacity precisely and the latter can also capture the load-deflection response of the structure.
- In the absence of shear reinforcement, structural failures due to shear are usually brittle and should be carefully treated with the actual action effects. This study also showed a case where the design code produced unconservative predictions.

4.11 Örnsköldsvik Bridge – Sweden – 2006 – Puurula et al. (2015); SB (2008)

The Örnsköldsvik Bridge was a skew (16.8º) single-track railway bridge with a structure consisting of a portal frame with a RC trough in two continuous spans with free distances of 11.92 m and 12.18 m. (see Figure 30). The trough was composed of two girders (1 000 × 1 100 mm^2) with curbs (623 × 350 mm^2) at the top and a slab (2 900 × 350 mm^2) between the girders at the bottom. The bridge deck was monolithically cast-in-place with vertical supporting walls and the in-situ tested concrete compressive strength (68.5 MPa) was appreciably higher than that used for the design 51 years earlier (it had increased by about 37.5 MPa). Upgrading and realignment of the railway lane was planned so the bridge became redundant. As a part of the project Sustainable Bridges (SB 2007), the bridge was extensively investigated to calibrate methods for condition assessment (i.e. non-destructive methods) and methods for analysis of the structural behaviour and load-carrying capacity. In addition, a strengthening technique, using NSM CFRP rods installed at the base of the trough, was tested and evaluated. Strengthening the bridge was intended to change the failure mode from an expected flexural failure to a shear failure.

The bridge was statically loaded in the midspan on top of the trough girders with two hydraulic jacks and a transverse load distribution beam (see Figure 30). Reaction forces from the load produced by the jacks were taken by cables anchored in the bedrock. In loading the bridge to failure, a generally ductile structural behaviour was revealed and the yielding of the longitudinal reinforcing steel was measured. After severe cracking and intermediate crack-induced debonding of the CFRP rods, leading to force redistribution, one of the girders failed in combined flexure and shear at a load of 11 700 kN and 96 mm deflection. At failure, several stirrups crossing the critical shear crack ruptured (see Figure 31).

Figure 30. The Örnsköldsvik Bridge (SB 2008).

Figure 31. The Örnsköldsvik Bridge after failure of the trough girder (Image by Lennart Elfgren).

A comparative study between different design codes with regard to the shear capacity was carried out. Based on in-situ tested material parameters, the American (ACI 318 2011), Canadian (CSA A23.3 2004) and European (SS-EN 1992-1-1 2005) standards predicted load-carrying capacities of

66 %, 65 % and 78 %, respectively, of the peak load in the full-scale experiment. Using information about the material from the bridge design, a 7 to 13 % lower capacity was given. For refined structural assessment, the bridge test was simulated using nonlinear FE analyses with various levels of approximation, ranging from a simplified 1D model (Ferreira et al. 2015) to a detailed 3D model (Puurula et al. 2015). The most complex 3D model, with the concrete modelled with continuum elements and the reinforcement using embedded discrete bars, was able to simulate the structural behaviour precisely, including the failure and the peak load. The FE model also indicated significantly increased stiffness and load-carrying capacity due to strengthening.

The study of the Örnsköldsvik Bridge provides the following lessons:

- The bridge behaved as expected but the design codes appear to be conservative with regard to the shear capacity. The actual capacity corresponded to 6.5 times the permitted axle load for the bridge, meaning a margin of safety well above the requirements.
- Nonlinear FE analysis is a powerful method to improve the accuracy of assessment of the capacity. However, the simulation of the bridge was sensitive to a range of modelling choices (e.g. material parameters and boundary conditions) and, for this bridge, information from the experiments was used to calibrate the FE model. Such thorough knowledge is not available from regular bridge assessments and, thus, the information collected can be useful for future assessments.
- Strengthening with NSM CFRP rods improved the performance of the bridge and could be used to upgrade the load-carrying capacity of bridges. At the same time, such strengthening leads to stiffening of the structure and this test illustrates the importance of taking the deformation capacity into account in order to avoid unexpected, sudden failures.

4.12 Kiruna Bridge – Sweden – 2014 – Bagge et al. (2014, 2017a, b)

The Kiruna Bridge was a continuous five-span PC girder bridge with road and pedestrian traffic in two lanes (see Figure 32). The span lengths were 18.00 m, 20.50 m, 29.35 m, 27.15 m and 26.50 m. The deck, supported on abutments and piers, was composed of three PC girders (410 – 650 × 1 923 mm^2) monolithically cast-in-place with a RC slab (4 900 × 220 – 300 mm^2) at the top with curbs (300 × 300 mm^2) on each side. The girders had a centre-to-centre distance of 5.41 m and they were reinforced with a BBRV post-tensioning system. Inspection indicated the bridge to be in good condition with existing damage limited to some vertical cracks in the girders adjacent to the intermediate support, assumed to be associated with curtailment of reinforcing steel bars in the bottom of the girders. Due to settlement of the ground, caused by surrounding underground mine workings, the bridge was monitored over a period of eight years before it was taken out of service. Over that time, no additional damage occurred. The bridge was set to be demolished because of relocation of the city, so a test programme was designed to improve methods of assessment of existing bridges. The study included evaluation of methods to determine residual prestress forces in tendons, the usability and impact of two types of strengthening systems using CFRP, and the structural behaviour and load-carrying capacity of the bridge girders and the deck slab. The failure test of the slab is described in Section 4.13.

Figure 32. The Kiruna Bridge (Image by Niklas Bagge).

The loading system consisted of two transversally aligned load distribution beams, supported on the girders in the middle of the shortest interior span, and two hydraulic jacks, with wires anchored in the bedrock, loading each load distribution beam. The bridge was investigated using a preloading schedule, during which flexural cracks occurred in the girders, both adjacent to the loading point and at the intermediate supports. Diagonal cracks were formed at the loading point. In the failure test, the load was applied symmetrically to the bridge up to a load of 12 000 kN, followed by modified loading to obtain failures in one of the outer girders and then the interior girder. For the two failures, the loads were 13 400 kN and 12 700 kN, respectively, (see Figure 33). The interior girder showed a considerable residual capacity after failure. Both girders behaved in a ductile manner, with extensive concrete cracking and yielding of the reinforcing steel. However, a brittle shear failure took place in the final phase and stirrups crossing a diagonal crack of the girder ruptured on either side of the loading point; at this juncture, the loading plate punched through the slab.

Figure 33. The Kiruna Bridge after shear failure of the girders (Image by Niklas Bagge).

Theoretical calculations of the load-carrying capacity indicated that the flexural capacity was reached at 67.9 % of the tested load-carrying capacity obtained when the first failure occurred. The calculations were based on linear elastic structural analysis, resistance models according to European standard (SS-EN 1992-1-1 2005) and in-situ testing of material parameters. Assuming redistribution of internal forces, shear failure occurred at 77.6 % of the tested value, but adjacent to the support instead of the loading point. Simulations of the test based on nonlinear FE analysis were more precise, with the tested failure mode predicted to occur at a loading of less than 1 % more than the tested value. However, the FE analysis was sensitive to assumed boundary conditions and material parameters.

From the study of the Kiruna Bridge girders, the following can be learnt:

- A high degree of redistribution of internal forces, in relation to the linear elastic assumption, took place during the test. This should be taken into account when assessing statically indeterminate bridges to utilize the flexural capacity of the structure fully.
- The resistance models used to determine the shear capacity are conservative and do not necessary reflect the actual behaviour and failure mode.
- Nonlinear FE analysis can be used to improve assessment and is able to predict the load-carrying capacity precisely. Since inaccurate inputs to the simulations may lead to overstated capacities, existing uncertainties should be treated carefully. For instance, bridge-specific material parameters should be used.

4.13 Kiruna Bridge – Sweden – 2014 – Bagge et al. (2014); Shu et al. (2017)

The Kiruna Bridge (see Figure 32), previously described in Section 4.12, was studied with a focus on the girders, including loading two of three girders to failure. Thereafter, the bridge deck slab adjacent to the still-functioning, but pre-cracked, girder was investigated. There was no damage to the slab found during visual inspection of the structure between the tests. The slab was loaded in the middle of the previously tested span, using loading plates (600 × 350 mm^2) with a centre-to-centre distance of 2.0 m in the bridge’s longitudinal direction. To produce the load, one hydraulic jack, connected to wire anchors in the bedrock, was used and the load was distributed to the plates using a longitudinally aligned beam.

In the test, considerable deflection of the girders occurred and, at a load of 3 320 kN, the slab suddenly, without any warning, failed with a combined failure mode of shear and punching (see Figure 34). The failure was initiated between one of the loading plates and the girder, and propagated on either side of the plate, thus, producing a U-shaped failure surface. After the test, the bridge deck slab was analysed based on methods at varying levels of approximation. First, analytical resistance models were used as described by the European standard (SS-EN 1992-1-1 2005) and, ultimately, nonlinear FE analysis was used. The analytical models resulted in a capacity of 44.6 % and 45.5 % of the tested capacity for one-way shear and punching, respectively. The nonlinear FE analysis precisely assessed the capacity with an underestimation of 1.5 % and correctly predicted the failure mode of the test. In the theoretical evaluation, a study of the arching action and distribution of shear stresses was also made. From these tests, the following can be learnt:

- Design models for shear and punching can be imprecise, in this study producing a highly underestimated capacity. Here, the impact of arching action for loads applied close to supports is one possible reason.
- In contrast to the local resistance models in the European standard (SS-EN 1992-1-1 2005), nonlinear FE analyses were able to predict the complex failure that occurred in the test accurately. However, the assumed boundary conditions in the FE model play an important role

and the full structural response of the bridge should preferably be simulated in the analysis.

– Due to the outcomes from the design standards and nonlinear FE analysis, a successively improved structural assessment was recommended in order to understand the structure better and to utilize its capacity.

Figure 34. The Kiruna Bridge at shear and punching failure of the deck slab, view from below (Image by Niklas Bagge).

5 Discussion of learning outcomes

Out of 30 bridges, in total 40 tests were identified and the types of failure summarized in Figure 35. In many cases, the shear-related failures were a combination of shear and punching and, in this summary, these are grouped together as the predominant failure mode was not always stated in the original source of the tests. There is a clear difference between expected failure mode in the tests and the actual outcome. The differences between expected and actual failure modes may inaccurately lead to the conclusion that the models for assessing the shear capacity are unconservative, especially for bridge deck slabs. In fact, the unexpected shear failure can, in many cases, be associated with inadequate attention to shear-related issues. In a few full-scale tests, only the flexural capacity has been reported and discussed in available papers and reports and, for these cases, it has been assumed that a flexural failure was expected although it is not explicitly stated. About 28 % of the failures can be regarded as unexpected in terms of the failure mode.

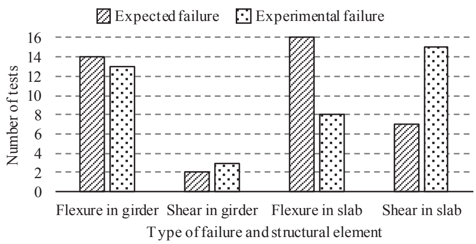

Figure 35. Summary of expected and actual failure modes in bridges tested to failure.

Full-scale tests of bridges have been reported for over a half century. Despite this, a review of the conclusions from each full-scale test indicates that the lessons from these tests have not been fully learnt. The same kinds of lessons and mistakes are being continually repeated rather than being recognized and fixed for subsequent bridge tests and assessments. The outcomes can be summarized as follows:

– In general, the bridge tests demonstrated load-carrying capacities appreciably higher than those calculated using traditional methods for structural assessment.
– Flexural failures can be precisely predicted and, for statically indeterminate structures, methods accounting for redistributions of internal forces can be beneficial. However, aspects associated to shear should be accounted for. Alternatively, nonlinear analyses can be used to model the nonlinear behaviour.
– Shear types of failure are not fully understood and local resistance models for shear are not always able to predict the actual behaviour. Shear models that are too conservative have been used in several studies so, in order to predict the behaviour and capacity better, nonlinear FE analysis is suggested. The studies also show the importance of accounting for concrete elements without shear reinforcement, which often produce brittle failure with no warning.
– For existing bridges, the in-situ material parameters often differ significantly from the values used in design and there are many examples where tested material parameters considerably improved the accuracy of the assessment. Thus, an assessment should be based on bridge-specific information from material tests.
– In the tests, both support conditions and interaction between structural elements have been shown to have a crucial impact on the structural behaviour and, in some cases, this was a cause of an unexpected failure mode. After several studies, uncertainties remained with regard to the boundary conditions and, consequently, it was not possible to explain differences between experiment and theory. Particular attention should be paid to the choice

of boundary conditions, both in structural analysis during assessment and for future tests.
- Usually, non-structural elements (e.g. curbs) are ignored when assessing the capacity of bridge decks and, in several tests, such geometrical simplifications made an appreciable difference between tested and theoretically determined load-carrying capacities.
- In two bridge tests, strengthening using CFRP reinforcement has been shown to work for upgrading the load-carrying capacity. However, both studies highlight the importance of proper consideration of the deformation capacity in order to avoid sudden failures.

In connection to the learning outcomes listed above, it should be noted that many of the bridges were not loaded in a way that corresponded with a real load, and some not even in accordance with the design or assessment code. Consequently, the failure modes obtained experimentally can differ to the ones critical for the bridge and conclusions regarding the structural safety in relation to the theoretical assessment are not fully representative for the bridge in general. Despite this, it is believed that the full-scale failure tests have great value for understanding the real structural behaviour of bridges and improvement of design and assessment methods. These tests also show the need and benefits of further research in order to achieve a more accurate and, thus, sustainable management of the network of bridges.

As stated in the introduction, it is of importance to test full-scale structures and to take advantage of the experience from these tests. In several projects, the tested bridge was not adequately analysed, either before or after the test. Analysis should be further prioritized in order for research and assessment to benefit from the test. Associated with this, it appears from this literature review that a range of studies have not been fully reported and, thus, the outcomes from the specific test are not completely clear. From these findings, a recommended procedure for full-scale testing has been defined and is shown in Figure 36. The test is based on research needs identified for the improvement of bridge assessment or design. From the very beginning, a strategy should be defined describing how to fill these gaps, and this should be continuously used throughout the project, although of course, it may be necessary to revise the strategy due to unexpected results. For many bridge tests, it can be concluded that the aim of the project is somewhat vaguely formulated, often "the aim is to understand the behaviour of this type of bridge better". To facilitate the valuable outcomes of a rare and challenging test, precise and achievable aims should be defined. Moreover, the outcomes of the project are greatly affected by the amount of attention paid to the analysis; before the test, this is an important part of the planning process in order to design a test whose measurements are able to provide the desired information. The bridge should preferably be analysed with successively improved accuracy and, if possible, also with improved information about the structure (e.g. information from pre-tests can be incorporated). A sensitivity study may highlight important aspects prior to the test. Coupled to previous studies discussed in this paper, such important aspects can, for instance, be associated with as-built geometrical characteristics, material properties, boundary conditions and residual prestressing. Afterwards, a predefined approach should be examined based on the test results. It should be updated if necessary and it should also evaluate and clarify any unexpected outcomes. Ultimately, an essential part of a bridge testing project is to implement and disseminate the

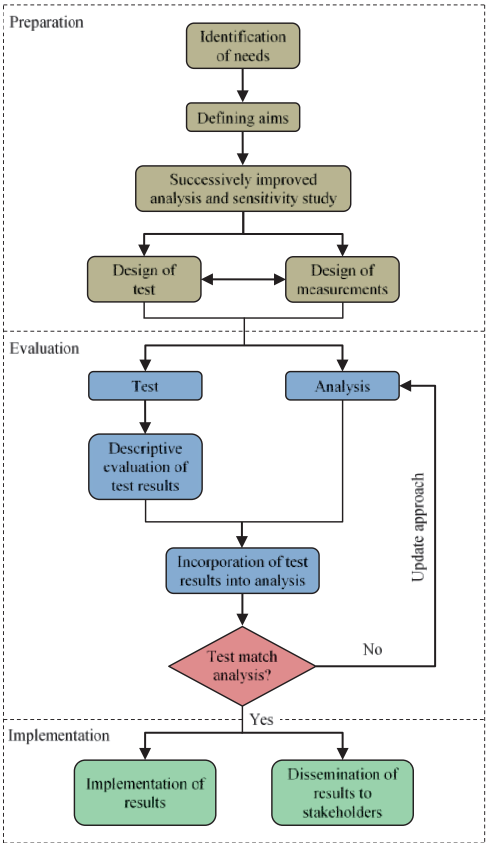

Figure 36. Procedure for full-scale testing.

outcomes. The dissemination is crucial in order to improve future assessment and design of bridges; however, an early involvement of stakeholders is often beneficial for a more effective dissemination of the outcomes. In the past, there were, in several cases, limitations in time and resources for preparing tests. Even in these cases, it is possible to follow the procedure presented, but the outcomes from the particular study are greatly affected by the early stage work.

6 Conclusions

This paper presents a literature review of full-scale concrete bridges tested to failure and provides feedback from previous studies. These kind of experiments are rare and play an important role, since they may raise aspects not seen in small-scale or large-scale tests in controlled and idealised laboratory environments. The objective of the review was to summarize the lessons learnt from those studies in order to improve future bridge assessment and to avoid repetition of the same mistakes. In addition, there were several tests where the ultimate capacity was not reached due to unexpectedly high capacities. A few such valuable examples have been listed.

Many unpredicted events took place in the failure tests and about 28 % of them ended with a failure mode different to that predicted. In some cases, this was related to inaccuracies in the methods for determining the load-carrying capacity but, in the majority of cases, it was caused by a lack of attention to aspects shown to be critical, particularly associated with the shear and punching capacities. In general, it can be concluded that the theoretical calculations of the load-carrying capacity are conservative, however, the "hidden" capacity can be utilized by using refined analyses. For instance, nonlinear FE analyses can precisely model the structural behaviour but, independent of the level of approximation used in the analyses, a proper representation of geometry, boundary conditions and materials is crucial.

This paper shows the advantages of full-scale failure tests and additional experiments should be carried out in order to improve further the understanding of existing bridges. However, the experience from previous tests should be taken into account in the preparation of new studies, which apparently has not always been the case. For instance, it is suggested that experimental tests are based on realistic load cases with consideration of aspects shown to be critical in the assessment, thus, leading to relevant and useful outcomes.

References

AASHO (1969). "Standard specifications of highway bridges." *10th edition*, The American Association of State Highway Officials (AASHO), Washington, DC, United States.

AASHTO (1973). "Standard specifications for highway bridges." *11th edition*, American Association of State Highway Transportation Officials (AASHTO), Washington, DC, United States.

AASHTO (1989). "Standard specifications for highway bridges." *14th edition*, American Association of State Highway Transportation Officials (AASHTO), Washington, DC, United States.

ACI 318 (2011). "Building code requirements for structural concrete and commentary." American Concrete Institute (ACI), Farmington Hills, MI, United States, 503.

Aktan, A. E., Zwick, M., Miller, R., & Shahrooz, B. (1992). "Nondestructive and destructive testing of decommissioned reinforced concrete slab highway bridge and associated analytical studies." *Transportation research record*(1371), 142-153.

Alkhrdaji, T., Nanni, A., Chen, G., & Barker, M. (1999). "Destructive and non-destructive testing of bridge J857 Phelps Country, Missouri." *Volume I: Strengthening and testing to failure of bridge decks*, University of Missouri, Rolla, MO, United States, 131.

Alkhrdaji, T., Nanni, A., & Mayo, R. (2000). "Upgrading Missouri transportation infrastructure. Solid reinforced-concrete decks strengthened with fiber-reinforced polymer systems." *Transportation research record*, 1740, 157-163.

Amir, S., van der Veen, C., Walraven, J. C., & de Boer, A. (2016). "Experiments on punching shear behavior of prestressed concrete bridge decks." *Aci Structural Journal*, 113(3), 627-636.

Austroads (1996a). "Australian bridge design code." *Section 5. Concrete*, Austroads, Sydney, Australia.

Austroads (1996b). "Australian bridge design code." *Section 2. Design loads*, Austroads, Sydney, Australia.

Azizinamini, A., Boothby, T., Shekar, Y., & Barnhill, G. (1994a). "Old concrete slab bridges. I: Experimental investigation." *Journal of Structural Engineering*, 120(11), 3284-3304.

Azizinamini, A., Shekar, Y., Boothby, T., & Barnhill, G. (1994b). "Old concrete slab bridges. II: Analysis." *Journal of Structural Engineering*, 120(11), 3305-3319.

Bagge, N., Nilimaa, J., Blanksvärd, T., & Elfgren, L. (2014). "Instrumentation and full-scale test of a post-tensioned concrete bridge." *Nordic Concrete Research*, 51, 63-83.

Bagge, N., Plos, M., & Blanksvärd, T. (2017a). "A multi-level structural assessment strategy examined with a prestressed concrete girder bridge. Part A: Initial level using current regulations." *Journal of Structure and Infrastructure Engineering*, (Submitted for publication).

Bagge, N., Plos, M., Popescu, C., & Sas, G. (2017b). "A multi-level structural assessment strategy examined with a prestressed concrete girder bridge. Part B: Enhanced level using nonlinear finite element analysis." *Journal of Structure and Infrastructure Engineering*, (Submitted for publication).

Barr, P., Halling, M. W., & Pettigrew, C. S. (2014). "Forensic testing of a double tee bridge." Utah State University, Logan, UT, United States, 240.

BBK 79 (1987). "Bestämmelser för betongkonstruktioner. Band 1: Konstruktion (Regulations for concrete structures. Part 1: Design. In Swedish)." AB Svensk Byggtjänst, Stockholm, Sweden, 157.

Bentz, E. C. (2000). "*Sectional analysis of reinforced concrete members*." Ph.D. Thesis, University of Toronto, Toronto, ON, Canada.

Bouwkamp, J. G., Scordelis, A. C., & Wasti, S. T. (1974). "Ultimate Strength of Concrete Box Girder Bridge." *Journal of the Structural Division*, 100(st1), 31-49.

Brownjohn, J. M. W., Moyo, P., Omenzetter, P., & Lu, Y. (2003). "Assessment of highway bridge upgrading by dynamic testing and finite-element model updating." *Journal of Bridge Engineering*, 8(3), 162-172.

Burdette, E. G., & Goodpasture, D. W. (1971). "Full-scale bridge testing: An evaluation of bridge design criteria." University of Tennessee, Knoxville, TN, United States.

Burdette, E. G., & Goodpasture, D. W. (1972). "Comparison of measured and computed ultimate strength of four highway bridges." *Highway Research Record*(382), 38-49.

Burdette, E. G., & Goodpasture, D. W. (1973). "Tests of four highway bridges." *Journal of the Structural Division*, 99(3), 335-348.

Burdette, E. G., & Goodpasture, D. W. (1974). "Test to failure of a prestressed concrete bridge." *Pci Journal*, 19(3), 92-103.

Byun, K.-J., Kim, Y.-J., Lee, K.-S., & Lee, S.-M. (1989). "Strength and structural performace of deteriorated reinfroced concrete bridges in Korea." *JCI Annual Convention*, Japan Concrete Institute (JCI), Tokyo, Japan, 667-672.

Bådholm, C., Bäcklund, J., Ottosson, H., & Ögård, L. (1967a). "Brottförsök på spännbetongbro (Failure test of a prestressed concrete bridge. In Swedish)." *Väg- och vattenbyggaren*, 11, 466-470.

Bådholm, C., Ottosson, H., Ögård, L., & Österlund, J. (1967b). "*Undersökning av huvudbalkarnas böjstyvhet, brobanans sammansatta vridstyvhet samt säkerheten mot böjbrott för en spännarmerad betongbro på tre stöd (Investigation of the flexural stiffness of the girders, the torsional stiffness of the bridge deck and the level of safety with regard to flexural failure of a prestressed concrete bridge on three supports . In Swedish)*." M.Sc. Thesis, Chalmers University of Technology, Gothenburg, Sweden.

Candy, C. C., Pressley, J. S., Walton, B. L., & Sanjayan, J. G. (2004). "Shear damage control in assessing flat slab bridge decks." *5th AustRoads Bridge Conference*, Austroads, Sydney, Australia, 1-12.

Cook, W. (2010). "*Destructive testing of composite precast concrete deck panels and built-up steel plate girders*." M.Sc. Thesis, Utah State University, Logan, UT, United States.

CSA A23.3 (2004). "Design of concrete structures." Canadian Standards Association (CSA), Mississauga, ON, Canada, 352.

Cullington, D. W., F, D. A., & E, H. M. (1996). "Assessment of reinforced concrete bridges: Collapse tests on the Thurloxton underpass." *3rd International Conference on Bridge Management Conference: Inspection, Maintenance and Repair*Guildford, United Kingdom, 667-674.

Daly, A. F. (1994). "Test to collapse of Thurloxton Underpass, Somerset." *PR/BR/12/94*, Transport Research Laboratory, Crowthorne, United Kingdom, 37.

Department of Transport (1990). "The assessment of concrete highway bridges and structures." *Departmental Standard BD 44/90*, Department of Transport, London, United Kingdom.

Eder, R. W., Miller, R. A., Baseheart, T. M., & Swanson, J. A. (2005). "Testing of two 50-year-old precast post-tensioned concrete bridge girders." *Pci Journal*, 50(3), 90-95.

Ferreira, D., Bairán, J., & Marí, A. (2015). "Efficient 1D model for blind assessment of existing bridges: simulation of a full-scale loading test and comparison with higher order continuum models." *Structure and Infrastructure Engineering*, 11(10), 1383-1397.

fib (2003). "Monitoring and safety evaluation of existing concrete structures." *Bulletin 22*, International Federation for Structural Concrete (fib), Lausanne, Switzerland, 304.

Goulet, J. A., & Smith, I. F. C. (2013). "Predicting the usefulness of monitoring for identifying the behavior of structures." *Journal of Structural Engineering (United States)*, 139(10), 1716-1727.

Haritos, N., Hira, A., Mendis, P., Heywood, R., & Giufre, A. (2000). "Load testing to collapse limit state of Barr Creek Bridge." *Transportation research record*, 2, 92-102.

Harries, K. A. (2009). "Structural testing of prestressed concrete girders from the Lake View Drive Bridge." *Journal of Bridge Engineering*, 14(2), 78-92.

Jang, J., & Smyth, A. W. (2017). "Model updating of a full-scale FE model with nonlinear constraint equations and sensitivity-based cluster analysis for updating parameters." *Mechanical Systems and Signal Processing*, 83, 337-355.

Jiaquan, X., Zanping, W., Bing, H., Guanhua, F., Yufeng, Z., & Jianfei, Z. (2006). "沪宁扩建桥梁极限承载能力实桥试验研究 (Practical Experimental study on ultimate bearing capacity of actual bridge in Huning expressway extension project. In Chinese)." *Modern Transportation Technology*(5), 77-84.

Jorgenson, J. L., & Larson, W. (1976). "Field testing of a reinforced concrete highway bridge to collapse." *Transportation research record*(607), 66-71.

Kjellgren, L., & Bergström, G. (1990). "*Skjuvförsök på spännarmerad rambro (Shear test of a prestressed frame bridge. In Swedish)*." M.Sc. Thesis, Chalmers University of Technology, Gothenburg, Sweden.

Labia, Y., Saiidi, M. S., & Douglas, B. (1997). "Full-scale testing and analysis of 20-year-old pretensioned concrete box girders." *Aci Structural Journal*, 94(5), 471-482.

Lantsoght, E. O. L., Yang, Y., vad det Veen, C., de Boer, A., & Hordijk, D. A. (2016). "Ruytenschildt Bridge: Field and laboratory testing." *Engineering Structures*, 128, 111-123.

Maanteemet (2015). "Vana Särevere silla koormuskatsetused ja teaduslikud analüüsid ning hüdroisoleerivate betoonide ja betoon-pinna hüdrofoobsete pinnakaitsevahendite teaduslik uuring." Talinn University of Technology, Talinn, Estonia.

McClure, R. M., & West, H. H. (1980). "Field testing of an experimental segmental bridge." *Interim report, Research project 75-3*, Pennsylvania State University, Pennsylvania Transportation Institute, University Park, PA, United States.

McClure, R. M., & West, H. H. (1984). "Full-scale testing of a prestressed concrete segmental bridge." *Canadian Journal of Civil Engineering*, 11(3), 505-515.

Miller, R. A., Aktan, A. E., & Shahrooz, B. M. (1994). "Destructive testing of decommissioned concrete slab bridge." *Journal of Structural Engineering*, 120(7), 2176-2198.

Ministry of Transport of the People's Republic of China (1989). "General code for design of highway bridges and culverts." *JTJ 021-89*, China Communications Press, Beijing, China.

N/A (1951). "Concrete structures at the Festival of Britain. South Bank, London, Exhibition." *Concrete and Constructional Engineering*, July, 199-206.

N/A (1952). "Test if a prestressed concrete footbridge." *Concrete and Constructional Engineering*, June, 185-188.

NBR (1992). "Prosjektering av betongkonstruksjoner - Beregnings- og konstruksjonsregler (Concrete structures - Design rules. In Norwegian)." *NS 3473:1992*, Norges Byggstandardiseringsråd (NBR), 103.

Nilimaa, J., Blanksvärd, T., Elfgren, L., & Täljsten, B. (2012). "Transversal post tensioning of RC trough bridges - Laboratory tests." *Nordic Concrete Research*, 46(2), 57-73.

Oh, B. H., Kim, K. S., & Lew, Y. (2002). "Ultimate load behavior of post-tensioned prestressed concrete girder bridge through in-place failure test." *Aci Structural Journal*, 99(2), 172-180.

Paulsson, B., Töyrä, B., Elfgren, L., Ohlsson, U., Danielsson, G., Johansson, H., & Åström, L. (1996). "30 ton på Malmbanan. Rapport 3.3 Infrastruktur: Forsknings- och utvecklingsprojekt avseende betongbroars bärighet (Static tests on four trough bridges and a laboratory fatigue test on one bridge. In Swedish)." Banverket and Luleå University of Technology, Borlänge, Sweden, 129.

Pedersen, E. S., Nielsen, P. M., & Lyngberg, B. S. (1980). "Investigation and failure test of a prestressed concrete bridge." *IABSE congress report*, 11, 849-854.

Plos, M. (1990). "Skjuvförsök i full skala på plattrambro i armerad betong (Full-scale shear test on concrete slab frame bridge. In Swedish)." *Report 90:3*, Chalmers University of Technology, Gothenburg, Sweden, 117.

Plos, M., Gylltoft, K., & Cederwall, K. (1990). "Full scale shear tests on modern highway concrete bridges." *Nordic Concrete Research*, 9, 134-144.

Plos, M. (1995). "*Application of fracture mechanics to concrete bridges. Finite element analyses and experiments*." Ph.D. Thesis Ph.D. Thesis, Chalmers University of Technology, Gothenburg, Sweden.

Plos, M., & Gylltoft, K. (1995). "Fracture mechanics analyses of the shear failure in a concrete bridge." *Nordic Concrete Research*, 16, 83-102.

Pressley, J. S., Candy, C. C. E., Walton, B. L., & Sanjayan, J. G. (2004). "Destructive load testing of bridge no. 1049 - analyses, predictions and testing." *5th Austroads Bridge Conference*, G. Nichols, ed., Austroads, Sydney, Australia, 1-12.

Pukl, R., Novak, D., & Eichinger, E. M. (2002). "Stochastic nonlinear fracture analysis." *1st International Conference on Bridge Maintenance, Safety and Management (IABMAS)*Barcelona, Spain.

Puurula, A. M., Enochsson, O., Sas, G., Blanksvärd, T., Ohlsson, U., Bernspång, L., Täljsten, B., Carolin, A., Paulsson, B., & Elfgren, L. (2015). "Assessment of the strengthening of an RC railway bridge with CFRP utilizing a full-scale failure test and finite-element analysis." *Journal of Structural Engineering*, 141(1), 1-11.

Rogers, R. A., Wotherspoon, L., Scott, A., & Ingham, J. M. (2012). "Residual strength assessment and destructive testing of decommissioned concrete bridge beams with corroded pretensioned reinforcement." *Pci Journal*, 57(3), 100-118.

Roschke, P. N., & Pruski, K. R. (2000). "Overload and ultimate load behavior of posttensioned slab bridge." *Journal of Bridge Engineering*, 5(2), 148-155.

Rösli, A., Kowalczuk, R., Hofacker, H., & Segelsdorff, R. (1963). "Die Versuche an der Glattbrücke in Opfikon (Test of the bridge over Glatt in Opfikon. In German)." *Report no. 192*, Swiss Federal Laboratories for Materials Science and Technology (EMPA), Dübendorf, Switzerland, 85.

Sanayei, M., Phelps, J. E., Sipple, J. D., Bell, E. S., & Brenner, B. R. (2012). "Instrumentation, nondestructive testing, and finite-element model updating for bridge evaluation using strain measurements." *Journal of Bridge Engineering*, 17(1), 130-138.

SB (2007). *Sustainable Bridges - Assessment for future traffic demands and longer lives*, Dolnośląskie Wydawnictwo Edukacyjne Wrocław, Poland.

SB (2008). "Field test of a concrete bridge in Örnsköldsvik, Sweden." *Deliverable 7.3*, Sustainable Bridges (SB) - Assessment for Future Traffic Demands and Longer Lives, Luleå, Sweden, 406.

Scanlon, A., & Mikhailovsky, L. (1987). "Full-scale load test of a three-span concrete highway bridge." *Canadian Journal of Civil Engineering*, 14(1), 19-23.

Scordelis, A. C., Larsen, P. K., & Elfgren, L. G. (1977). "Ultimate strength of curved RC box girder bridge." *Journal of the Structural Division*, 103(8), 1525-1542.

Shekar, Y., Azizinamini, A., Barnhill, G., & Boothby, T. E. (1993). "Performance of concrete slab bridges." Nebraska Department of Roads, Lincoln, NE, United States, 609.

Shenoy, C. V., & Frantz, G. C. (1991). "Structural tests of 27-year-old prestressed concrete bridge beams." *Pci Journal*, 36(5), 80-90.

Shu, J., Bagge, N., Plos, M., Johansson, M., Yang, Y., & Zandi, K. (2017). "Shear and punching capacity of a RC bridge deck slab loaded to failure in a field test." *Journal of Structural Engineering*, (Submitted for publication).

Song, H. W., You, D. W., Byun, K. J., & Maekawa, K. (2002). "Finite element failure analysis of reinforced concrete T-girder bridges." *Engineering Structures*, 24(2), 151-162.

SS-EN 1992-1-1 (2005). "Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings." European Committee for Standardization (CEN), Brussels, Belgium, 236.

Statens Vegvesen (1998). "Prøvebelastning av bru nr 02-1234 Smedstua bru: Forutsetninger, gjennomføring og måledata (Load test of bridge no 02-1234 Smedstua Bridge: Conditions, execution and measurements. In Norwegian)." Statens Vegvesen, 99.

Stillings, T. W. (2012). "*Load distribution and ultimate strength of an adjacent precast, prestressed concrete box girder bridge*." M.Sc. Thesis, University of Cincinnati, Cincinnati, OH, United States.

Täljsten, B. (1994). "*Plate bonding strengthening of existing concrete structures with epoxy bonded plates of steel or fibre reinforced plastics*." Ph.D. Thesis, Luleå University of Technology, Luleå, Sweden.

Wang, F. M., Kang, S. Z., Cai, Y. C., & Li, X. L. (2011). "Destructive test study of a prestressed concrete hollow slab beam bridge." *Geotechnical Special Publication*, 2014, 57-64.

Vaz Rodrigues, R., Fernández Ruiz, M., & Muttoni, A. (2008). "Shear strength of R/C bridge cantilever slabs." *Engineering Structures*, 30(11), 3024-3033.

Weder, C. (1977). "Die vorgespannte, zwanzigjährige Stahlbetonbrücke über die alte Glatt bei Schwamendingen, Zürich (Prestressed, twenty year-old RC bridge over the old Glatt at Schwamendingen, Zürich. In German)." *Report no. 203*, Swiss Federal Laboratories for Materials Science and Technology (EMPA), 49.

Vejdirektoratets broafdelningen, Statens Vejlaboratorium, & Cowiconsult (1979). "Belastningsforsøg med förspænd betonbro ved Brønsholm. Hovedrapport (Failure test of a prestressed concrete bridge at Brønsholm. Final report. In Danish)." Vejdirektoratet, 130.

Wenping, X. (2006). "***既有**预应力混凝土梁桥承载能力实桥试验及分析研究南京 (Experimental research and analysis on load-carrying capacity of the existing prestressed concrete bridge. In Chinese)*." Ph.D. Thesis, Southeast University, Nanjing, China.

Vill, M., Schweighofer, A., & Kollegger, J. (2012). "Großversuche an Spannbetonbrückenträgern zur Beurteilung des Schubtragverhaltens (Full scale tests on prestressed concrete bridges concerning the shear load bearing capacity. In German)." *Beton- und Stahlbetonbau*, 107, 86-95.

You, D.-W., Song, H.-W., & Byun, K.-J. (1998). "파괴시험에 의한 철근콘크리트 T형교 거동의 실험적 연구 (An experimental study on the behavior of reinforced concrete T-girder bridges by full-scale destructive testing. In Korean)." *Journal of The Korean Society of Civil Engineers*, 18(1-6), 851-862.

Zhang, J.-Q., Ren, H.-W., & Yu, B. (2011a). "Failure testing of a full-scale reinforced concrete T-girder bridge." *Advanced Materials Research*, 243-249, 1767-1773.

Zhang, J., Peng, H., & Cai, C. S. (2011b). "Field study of overload behavior of an existing reinforced concrete bridge under simulated vehicle loads." *Journal of Bridge Engineering*, 16(2), 226-237.

Zhang, J., Peng, H., & Cai, C. S. (2013). "Destructive testing of a decommissioned reinforced concrete bridge." *Journal of Bridge Engineering*, 18(6), 564-569.

Zhang, J. R., Peng, H., Zhang, K. B., & Hao, H. X. (2009). "Test study on overload and ultimate behavior of old reinforced concrete bridge through destructive test of corroded bridge." *Engineering Mechanics*, 26(2), 213-224.

Živanović, S., Pavic, A., & Reynolds, P. (2007). "Finite element modelling and updating of a lively footbridge: The complete process." *Journal of Sound and Vibration*, 301(1-2), 126-145.