Evaluation of Concrete Structures
Strength Development and Fatigue Capacity

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Preface

This licentiate thesis presents work that has been carried out at the Division of Structural Engineering, Department of Civil and Mining Engineering at Luleå University of Technology (LTU). The work has mainly been funded by the Swedish National Rail Administration, LTU and European Rail Research Institute (ERRI), the Netherlands.

First, I would like to thank my supervisor, Prof. Tech. Dr. Lennart Elfgren, head of the division, for being a valuable source of knowledge and enthusiasm. Further, I thank Ass. Prof. Tech. Dr. Ulf T. Ohlsson and Prof. Tech. Dr. Thomas Olofsson for always having time for discussions. I would also like to thank Tech. Dr. Hans Hedlund and Tech. Dr. Patrik Groth for all their advice and support during my time at LTU and Ass. Prof. Tech. Dr. Mats Emborg for “bringing” me back to LTU and PhD studies!

I would like to express my gratitude to the staff at Testlab, Mr. Håkan Johansson (head of “Testlab”), Mr. Georg Danielsson, Mr. Lars Åström and Mr. Mikael Kågström. They have performed, helped and advised me during all the tests/experiments. In this context I will also mention MSc. Jörgen Andersson who performed all the fatigue tests while working with his master thesis.

Moreover, I owe a debt of gratitude to the rest of the staff at the Division of Structural Engineering for making the environment pleasant and creative, especially PhD students Anders Carolin, Tech. Lic. Martin Nilsson, Håkan Nordin and last but not least Sofia Utsi for all the laughs. See you soon at an “Lp-start”!

Finally, it has been an eventful year with this thesis as some kind of “grand final”. A supporting and very patient family is needed in order to be able to finish something like this and I express my deepest gratitude to my fiancée Anna and my seven–month-old daughter Cecilia for always being there for me during this time.

Luleå in May, 2001

Håkan Thun
Abstract

This licentiate thesis consists of four papers, A-D.

In paper A the development of tensile and compression strength is presented for old concrete railway trough bridges. The compression strength has usually increased 50 to 100% over the years for these trough bridges. However, the same cannot be said for the tensile strength. Further, when the results from drilled out cores were compared for different structural parts (i.e. the bottom slab and the longitudinal beams) for one bridge. It turned out that the compressive strength was approximately 15% higher in the beam than in the slab. In the paper a rather well known method, the Capo-test, is used to determine the in-situ concrete compression strength in old railway trough bridges. The Capo-test has been compared with the results from drilled cores, which is the common way to determine the in-situ strength of a concrete structure. The study indicates that the Capo-test can be used on objects with old age, but with some restrictions, especially regarding the aggregate size. Results from tests performed on one bridge, the Lautajokki Bridge, show that the Capo-test might in some cases overestimate the compressive strength. An improved formula is presented for evaluation of the compression strength from Capo-tests for old structures. It gives a better correlation than the general one given by the manufacturer of the equipment and the one proposed by Rockström & Molin (1989). However, further studies are necessary.

In paper B load carrying capacities of cracked as well as un-cracked concrete railway sleepers have been investigated. The cracking is believed to be caused by delayed ettringite formation. The tests that have been performed are:

1. Bending moment capacity of the midsection.
2. Bending moment capacity of the rail section.
3. Horizontal load capacity of the fasteners.
4. Concrete properties of the tested sleepers.
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Information on how the visual inspection and the classification of damages are performed is presented together with possible failure mechanisms of the fastener when loaded horizontally. The purpose with the tests have been to get information on how the cracking influences the load carrying capacity compared to an un-cracked sleeper. The test results have been compared with calculations according to the Swedish Rail Code for sleepers. The tests show that small cracks do not seem to influence the load carrying capacity and first when cracking is very severe the load-carrying capacity is reduced significantly.

In paper C results and analyses are presented from cyclic tensile fatigue tests. The tests can be considered as initial ones. The fatigue tests in this paper are compared to a deformation criterion proposed by Daerga & Pöntinen (1993) based on an idea of Balázs (1991). The growth in deformation during fatigue can according to the model be described as three phases. In the beginning of the first phase the deformation rate is high but stagnates after a while. The second phase is characterised by a constant deformation rate. These two phases can be described as stable. During the third phase, the failure phase, the deformation rate increases rapidly leading to failure within a short time. The strain criterion for fatigue failure means that the strain at peak load during a static failure load corresponds to the strain at the changeover between phases two and three during a fatigue failure. These tests indicate that the strain criterion proposed by Balázs (1991) might also be applied on plain concrete subjected to cyclic tensile fatigue load. Though, further studies are needed.

In paper D a comparison is made between the Swedish Codes and Eurocode regarding fatigue load for concrete. If the results are compared there are differences. The calculation with EC2-2 (1995) gives the least conservative values. The Swedish code is slightly more conservative than the EC2-2, but they are still fairly similar. The new version of Eurocode, EC2-draft (1999), is by far the most conservative one. Some of the differences are:

- The expression to calculate the shear force capacity.
- The expression to calculate the design fatigue strength.
- The Swedish Code gives an approximate value of the number of cycles before failure (the value is taken from a graph).

The reason why EC2-draft (1999) is much more conservative than the EC2-2 (1995)/EC2-1 (1991) is the suggested reduction of the design shear strength. This leads to the fact that the trend regarding concrete fatigue goes in a conservative direction. The result presented in this paper indicates instead, that it rather would be possible to allow higher stresses.

Keywords: concrete, bridges, strength development, Capo-test, cyclic tensile fatigue test, fatigue, strain criterion, sleepers.
Sammanfattning

Denna licentiatuppsats består av fyra artiklar, A-D.


I artikel B redovisas provning och utvärdering av bärförmågan för uppspruckna respektive ej uppspruckna järnvägssliprar av betong. Uppsprickningen tros bero på s.k. försenad ettringitbildning. De test som utförts är:

1. Böjprov i mittsnitt.
2. Böjprov i rälläge.
3. Dragprov av befastning (horisontalkrafsskapacitet).
4. Kontroll av sliparnas betonghållfasthet.

- vii -


- Ekvationen för att bestämma tvärkraftskapaciteten.
- Ekvationen för att bestämma utmattningshållfastheten.
- Den svenska normen ger endast approximativa värden för antalet cykler till brott (värden läses ur ett diagram).


Nyckelord: betong, trågbroar, hållfasthetsutveckling, Capo-test, cykliska dragförsök, utmatning, töningskriterium, slipar.
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Papers included in this thesis:

**Paper A:**
Håkan Thun, Prof. Lennart Elfgren and Tech Dr. Ulf Ohlsson: *Concrete Strength Development in Swedish Bridges*.

**Paper B:**
Håkan Thun, Sofia Utsi and Prof. Lennart Elfgren: *Load Carrying Capacity of Cracked Concrete Railway Sleepers*.

**Paper C:**
Håkan Thun, Prof. Lennart Elfgren and Tech Dr. Ulf Ohlsson: *Concrete fatigue capacity. A study of deformations at tensile forces*.

**Paper D:**
Håkan Thun, Prof. Lennart Elfgren and Tech Dr. Ulf Ohlsson: *Fatigue capacity of the concrete slab – a comparison between the Swedish Concrete Code and Eurocode*. 
1 Introduction

1.1 Background. Aim and Scope
During the last decades it has become more and more important to assess, maintain and strengthen structures like bridges, dams and buildings. This is due to the increased age of many structures and the high costs to construct new infrastructure. Therefore it is of great interest to find methods to evaluate these concrete structures, since there are so many factors that influence, e.g. the load carrying capacity or the general condition of the structure. Such factors are e.g. in-situ strength, cover of reinforcement, amount and quality of the reinforcement, degree of degradation of resistance (frost attack etc.). An important factor to keep in mind is also why the structure is going to be evaluated so that the right action is taken.

In Figure 1 an idea is presented of how such an evaluation could be performed. It consists of three steps. The first is the simplest of them: an inspection at site and recalculation according to the present Design Code. The next step is a more refined check and might be more costly and time consuming. It consists of e.g. strength tests and measuring at site of some parameters (strain, deflection etc.). With this information new and more advanced calculations could be performed. The last step is what action that must be taken with the new information. Will the structure be:

- Supervised i.e. e.g. measurement of the strain development over a longer time (this is already done today with the help of a modem and a data logger see Utsi et.al (2001))?

- Or must the bridge be strengthened with e.g. carbon fibre reinforced polymers (see e.g. Carolin (2001))?

- Or must a new bridge be built?

Some of the checks in Figure 1 are not easy to perform e.g. the measuring of the strain development in concrete at site. An earlier research project - 30 ton på Malmbanan - has shown that field measurements of the deflection during passing trains are an important instrument when evaluating the condition of a bridge.
Figure 1 Idea of how an evaluation of e.g. a concrete bridge could be performed.

Of all these parameters/factors mentioned above that influence e.g. the load carrying capacity, the focus has been set on the in-situ concrete strength in old railway trough bridges, see paper A. Here, among other things, different semi-destructive and non-destructive methods to determine the strength of concrete in existing structures have been evaluated.

When a calculation is performed with the present Design Code on an existing bridge in e.g. step 1 in Figure 1, it can fail to live up to the demands in several ways. An old bridge that was recalculated according to the present concrete Design Code in an earlier research project – 30 ton på Malmbanan- showed that it would not manage the increase in axle load from 25 tons to 30 tons, especially regarding the concrete shear fatigue capacity. To check whether this was correct it was exposed to a full-scale fatigue test in the laboratory at LTU in 1996. It managed 6 million load cycles with an axle load of 360 kN instead of the 500 cycles that the Swedish Concrete Code indicated. The reason for this discrepancy is partly due to the fact that concrete fatigue is roughly modelled and often with large safety factors in the present Design Codes (see the comparison in paper D). This led to the study presented in paper C, a study of deformations at cyclic tensile load, which hopefully in the end will lead to a proposal for how the fatigue capacity might be calculated for tensile fatigue load.

This thesis also contains study of a concrete railway element and that has, in fact, been investigated according to the idea presented in Figure 1. This concrete element is a railway sleeper made of prestressed concrete and results can be found in paper B. Load carrying capacity tests have been performed on cracked as well as un-cracked railway sleepers.

1.2 Contents
In chapter 2 a summary is presented of paper A that has the title: *Concrete Strength Development in Swedish Bridges*. In the paper the development of tensile and compression strength is presented for old concrete railway trough bridges.

Chapter 3 presents a summary of paper B that has the title: *Load Carrying Capacity of Cracked Concrete Railway Sleepers*. In this paper load carrying capacities of cracked as well as un-cracked concrete railway sleepers have been investigated. The paper is co-authored by PhD student Sofia Utsi. The laboratory tests of the bending capacity of the midsection and the rail section have been performed by Sofia Utsi while the tests of the
horizontal load carrying capacity of the fastener have been performed by Håkan Thun except for one of the tests.

In chapter 4 a summary is presented of paper C that has the title: Concrete fatigue capacity. A study of deformations at tensile forces. In this paper results and analyses are presented from uniaxial tensile cyclic fatigue tests. The tests can be considered as pilot tests.

In chapter 4 a summary is presented of paper D that has the title: Fatigue capacity of the concrete slab – a comparison between the Swedish Concrete Code and Eurocode. In this paper a comparison is made between the Swedish Codes and Eurocode regarding the fatigue shear capacity for concrete.

Chapter 6 contains an outlook.

The tests and the analyses in the presented papers have in general been performed by the author. Guidance and comments have been given by the co-authors Prof. Lennart Elfgren and Tech. Dr. Ulf Ohlsson.

1.3 New and original features
To the knowledge of the author, the following features are new and original contributions:

- Concrete strength evaluation for old trough railway bridges.
- Revised expression for correlating the Capo-test to compressive strength for old concrete structures.
- Tests and evaluation of load carrying capacity of deteriorating concrete sleepers.
- Cyclic testing of concrete at tensile load and checking of a fatigue failure strain criterion.
- Comparison of Swedish and European Codes for concrete shear fatigue capacity.
2 Strength development in Swedish Concrete Bridges – Paper A

2.1 Background. Aim and Scope
Field studies of the concrete strength development were initiated when an increase of the axle load from 25 tons to 30 tons was planned on the iron ore railway line between Luleå in Sweden and Narvik in Norway. Due to this a program was initiated by the Swedish and Norwegian Railway Authorities to check the bearing capacity of the existing bridges strength and what kind of strengthening procedures that would be necessary to allow the enlarged load.

One of the first steps in the investigation was to recalculate a 29-year-old standard trough bridge from Lautajokki with the present design codes. The calculations indicated that the shear fatigue capacity was inadequate so in order to check this a full-scale fatigue test was carried out on the Lautajokki Bridge. The bridge was exposed to 6 million load cycles (axle load 360 kN) without any signs of a reduction of its capacity (this is considerably more than what the Swedish Code indicated).

There are some 70 concrete bridges of trough type, see Figure 2, between Luleå and Narvik, and some 200 in the rest of Sweden. The trough consists of a slab, filled with ballast, connected to and carried by two longitudinal beams. The bridges of this standard type were usually built between 1950 and 1980 with a design compression strength of about 40 MPa (tested on 150 mm cubes).

Figure 2 Typical cross-section of a Swedish concrete railway trough bridge. The trough, filled with ballast, consists of a slab connected to two longitudinal beams, Nilsson et al. (1999).
One of the reasons why it turned out so well in the fatigue test was due to the fact that the concrete compression strength had increased considerably over the years compared to the design strength. The reason to this increase can be due to several factors. The two most likely are that the concrete delivered to the building site was “better” than what was stipulated or that the hydration of concrete had continued over the years (see e.g. Rådman (1998)). This is possible when the cement is coarsely ground.

If this increase in strength that was found in the Lautajokki Bridge also could be a fact for all other bridges along the railway line it could lead to great savings of time and money. Therefore suitable test methods and a large number of bridges have been investigated regarding the strength development. In this paper different semi-destructive and non-destructive methods to determine the strength of concrete in existing structures have been used. The results have made it possible for a decision for each bridge if it could be kept or if it must be replaced.

2.2 Test methods
The methods that have been used in the field investigation to determine the concrete strength of the concrete railway trough bridges are:

- Capo-test
- Drilled out cores
- Rebound-hammer (Schmidt hammer)

The Capo-test and the rebound-hammer are simpler and less expensive to perform compared to drilling out cores. They also have the advantage that the equipment is lighter and easier to move compared to the equipment used for drilling out cores. This was one of the key-advantages since many of the bridges could only be reached either by train or by hiking.

To use drilled out cores to estimate the in-situ strength of a structure is a common method. Most countries have adopted standard procedures for how a core should be prepared, stored, etc. before testing. In this study the preparation, the storage etc. have been made according to the Swedish Concrete Code, BBK94 (1994,1996). The cores have been air cured for at least three days before testing.

The Capo-test on the other hand is a method to determine the concrete strength of the cover-layer for an existing structure. It was developed in Denmark by German Petersen & Poulsen (1993) in the middle of the 1970s. The Capo-test is a further development of the so-called Lok-test where the pullout bolt is embedded in fresh concrete.

The test procedure of the Capo-test, see Figure 3, consists of drilling a 65 mm deep hole with a diameter of 18 mm using a water-cooled diamond bit. Then a 25 mm recess is made at a depth of 25 mm using a portable router. An expandable split steel ring is inserted through the hole in the recess and expanded by the means of a special tool. Finally the ring is pulled through a 55 mm counter pressure placed concentrically on the surface. The pullout force is measured by the pull machine and can be converted into compressive strength by means of calibration charts provided by
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German Petersen & Poulsen (1993). A description can also be found in for example Bungey & Millard (1996).

Figure 3 Schematic drawing of the Capo-test, German Petersen & Poulsen (1993) and Bungey & Millard (1996).

The background of the correlation charts is several laboratory and field studies made by the manufacturer as well as by other researchers. Results from the Lok-test are mostly the basis of the correlations charts.

Especially at the end of the 1970s and during the 1980s the Lok-test and similar methods were heavily discussed in the concrete society, particularly regarding what property that is actually measured in a pullout test. In the literature you can find several theories and some of them are, Yener (1994):

- Compression strength: Ottosen (1981) published a non-linear finite element analysis of the Lok-test in 1981. His study indicated that the cracking initiated in the circumferential direction behind the disk (compare with the steel ring in Figure 3) at 7% loading. Radial cracking was found to initiate at 18% loading near the outer concrete surface. With increased loading radial cracks developed gradually and at 64% loading a considerable development of new circumferential cracks occurs. These were found to extend from the outer part of the steel disk towards the support. At 70% loading the distribution of principal stresses indicated that large compression forces run from the disk in a narrow band toward the support. Hence, the pullout capacity above 64% of the loading was due to crushing of the concrete within this band. His conclusion was that the force that is needed in a pullout test to extract the disk directly depends on the compressive strength of the concrete.

  In Yener (1994) one can find support for this theory. That the reasonably good correlation between the compression strength and the pullout force is explained by the fact that the residual strength in a pullout test is a consequence of the crushing of the concrete in the vicinity of the support ring.

- Aggregate interlocking: Stone & Carino (1983) carried out large-scale pullout tests and their observations refute the thesis with narrow bands. Their tests showed no visible evidence of such bands. They speculated that aggregate-
interlock across the failure surface is the reason to the load capacity above the 64% mentioned in Ottosen (1981).

This idea was rejected in Yener (1984) since aggregate interlocking would be very sensitive to different types of aggregate. This would in turn had led to reports of high variations within performed tests.

- Tensile strength: Stone & Carino (1984) concluded on basis of a comparison between the predicted elastic tensile trajectories of the un-cracked concrete and the experimental failure surface profile, that the formation of the complete failure surface is governed primarily by the tensile strength of mortar. They proposed that the observed correlation exists because both the pullout strength and the compressive strength of concrete are related to the tensile strength of the mortar.

- Combined compression and bending actions: Yener (1994) made a FEM-analysis of the problem and suggested that the behaviour of concrete subjected to a pullout test is primarily controlled by combined compression and bending actions. The bending action is pronounced in the later stages of the loading and the concrete within the eventual failure surface is compressed in the general direction of the applied load. His study also indicates that this is combined with tensile stresses in the earlier stages of the applied load.

However, a common thing for all studies is that a fairly good correlation has been found between the pullout force and the compression strength. In this paper the failure is not analysed. Instead the reasonably good correlation between the pullout force and the compression strength has been accepted and utilized to determine the in-situ compression strength.

Not many studies have been made concerning comparison between the Capo-test and drilled out cores (dimensions 100×100 mm) from old structures. To the authors’ knowledge only one study has been made, Rockström & Molin (1989). Their study showed that the correlation proposed by German Petersen (1997) might not give a perfect correlation when the test object is an old structure, i.e. old road bridges.

Together with the results obtained by Rockström & Mohlin and the results in this study a new correlation has been proposed for old structures, see also Figure 4:

\[ F = 0.59 \times f' + 2.46 \times N \]  \hspace{1cm} (1)
2.3 Results and Discussion

This study indicates that the Capo-test can be used on objects with old age, but with some restrictions. The results from the tests performed on the Lautajokki Bridge show that the Capo-test might overestimate the compressive strength compared with drilled out cores. The Capo-test gave higher values when it was performed on the longitudinal beams and the slab than drilled out cores from the slab. If this is something general for all actual bridges is hard to say while the basis is poor - there are only results from one bridge.

This study along with the study performed by Rockström & Molin (1989) show that if the general correlation proposed by the manufacturer is used, values on the safe side ought to be received for the compression strength. The correlation proposed by the authors gives a better correlation than the general one and the one proposed by Rockström & Molin (1989) for old structures, but it might give results on the unsafe side. More studies are needed of old structures.

Regarding very old structures it is essential to use the Capo-test with caution. This is due to the risk of a big difference in aggregate size in used concrete. For bridge No. 10 in the survey the Capo-test resulted in the compression strength 29.7 MPa, but when cores were taken out later on they gave the compression strength of 56 MPa. The difference was probably due to the great difference in aggregate size.

The Capo-test is included in the new proposal for International as well as Swedish standard for assessing the concrete strength in structures (see prEN 13791:1999: Assessment of concrete compressive strength in structures or in structural elements).

When the results from the drilled out cores were compared for the structural parts, i.e. the slab and the longitudinal beams, it showed that the compressive strength was
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approximately 15% higher in the longitudinal beam than in the slab for the Lautajokki Bridge. This indicates that there is a difference in strength between the side beams and the slab. Until this difference is verified with further studies, a reduction of the concrete strength with 15% should be performed when the tests are done in the side beam and the analysis concerns the slab. This reduction could be introduced as a partial coefficient, see Nilsson et.al. (1999).

Regarding the strength development of the concrete in the bridges it has increased heavily over the years, just for the Lautajokki Bridge the strength has increased from about 40 MPa to 72 MPa for the slab. The difference in strength between the side beams and the slab needs further studies.

The results of tensile strength vary considerably between the studied bridges (i.e. the longitudinal beams) and it is not possible to say from the results obtained in this study that the tensile strength has increased as much as compressive strength with time. When the tensile strength is determined from compressive strength it should be limited because of the possibility of cracks due to e.g. constraint. In Thun et.al. (1999) a method is included how to achieve characteristic tensile strength from test results according to the Swedish Concrete Code.
3 Load Carrying Capacity of Cracked Concrete Railway Sleepers – paper B

3.1 Background. Aim and Scope
Normally concrete sleepers sustain their properties for more than 50 years. However in Sweden some sleepers made between 1992 and 1996 have started to deteriorate. They have obtained cracking of a more or less severe kind and some of them have even lost most of their bearing capacity. The cracking is believed to be caused by delayed ettringite formation, which leads to an internal expansion and with time cracks. This process might reduce the service life to as few years as five.

This has of course led to extensive investigations by the Swedish National Rail Administration. They have inspected about 3.5 millions sleepers and there are about 300 000 sleepers that are cracked and the only way of finding them is by walking along the railway tracks. Knowing this, it is easy to understand that the investigation is very difficult and time consuming to perform. The investigations are performed in the way that two inspectors walk along the railway track on opposite sides.

The origin of the delayed ettringite formation is believed to be inferior production methods. In order to increase the production speed, the cement amount was increased from ordinarily 420kg/m³ to 500kg/m³ and steam curing was used during the hardening process in some of the production plants.

In Figure 5 a principal drawing of a Swedish railway sleeper is presented. The sleepers are made of prestressed concrete with the concrete class K60 (compression strength 60 MPa tested on 150 mm cubes). The sleepers are prestressed with 10 strands (each strand consists of 3 wires with the diameter 3 mm each).
3.2 Visual inspection and Classification

The first inspections led to a categorization of the sleepers depending on the cracking. They were divided into three classes by the Swedish National Rail Administration. The typical damages for each class are:

**Class 1. Acute / Red:**

The cracking is so severe that there is a considerable reduction of the load carrying capacity. There are typical longitudinal cracks in the middle part of the sleeper. There are also cracks at the end of the sleepers with a crackled pattern. The sleepers may have a crack from the fastener and downwards. The concrete surface is discoloured by yellow spots. A typical crack-pattern is shown in Figure 6.

**Class 2. Initial degradation / Yellow:**

Some cracks. The cracking is of the kind that the load carrying capacity is almost intact. There may be cracks with a crackled pattern at the end of the sleepers. The sleepers might have a crack from the fastener and downwards. There is presence of yellow spots. Typical crack-patterns are given in Figure 7.

**Class 3. OK / Green:**

No visible cracks. No visible tendencies to develop major faults. No change in colour. The load carrying capacity is intact.

Figure 6 Characteristic crack patterns for sleepers of class 1 (red sleepers).
Figure 7 Typical cracks for sleepers of class 2 (yellow). a) Sleepers in group 3 i.e. no visible cracks on the upper side but there might be cracks on the side at the lower edge. b) Sleepers in group 2 i.e. only 1 or 2 visible cracks on the upper side. They have fewer cracks on the side towards the lower edge than the sleepers in group 1 (the crackled pattern is not yet as “developed” as for group 1 sleepers). c) Sleepers in group 1 i.e. cracks in a crackled pattern on the side as well as on the upper side.

Since the yellow sleepers are so many (about 300 000 up to this date) and the variation in cracking is so large, they have been divided into subcategories in the hope of finding out if there is a variation in load carrying capacity among them. The criterion that has been used as a basis is what kind of cracks an inspector has a chance of discovering when he/she walks along the railway track. Since the sleepers are covered with macadam, it is only possible to notice damages that are on the upper side of the sleeper and 1 to 2 cm along the top parts of the sides. The cracks that have been used as target have a width larger than 0.05mm. These are possible to see with the naked eye and can be discovered without the need to get down on one’s knees. These cracks are in this paper called visible cracks.

The area on the sleeper where the first visible cracks appear (when they lie in the track) seems to be on the upper side at the end, near the edge. This leads to a problem since most yellow sleepers also have cracks on the side towards the lower edge, see Figure 7. These cracks are not possible to detect at an inspection as long as the macadam is not removed. This might lead to the fact that a yellow sleeper is given the class green.

The subdividing of the yellow sleepers is thus only based on visible cracks on the upper side of the sleeper, at the end. Worth pointing out is that not all sleepers have two ends with the same type of damages. Some sleepers have several cracks at one end but no cracks at the other.

The yellow sleepers have therefore in turn been subdivided into three categories:

Group 1 Several cracks on the upper side with a crackled pattern Figure 7 c).

Group 2 One or two cracks on the upper side, see Figure 7 b).

Group 3 No cracks on the upper side, see Figure 7 a).

3.3 Results
The tests that have been performed are:

(1) Bending capacity of the midsection.
(2) Bending capacity of the rail section.

(3) Horizontal load carrying capacity of the fasteners.

The material properties of the concrete in the tested sleepers have also been studied. The results have been compared to theoretical calculations according to the Swedish Rail Code.

### 3.3.1 Bending capacity of midsection

The red sleepers have a moment capacity of approximately 19 kNm and the green sleepers approximately 32 kNm, see Figure 8. According to BVF 522.32 (1995) the sleepers must manage 19.25 kNm.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Class</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$M_{\text{mid}}$ [kNm]</th>
<th>Failure type</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>red</td>
<td>65</td>
<td>19.5</td>
<td>shear</td>
</tr>
<tr>
<td>4</td>
<td>green</td>
<td>109</td>
<td>32.7</td>
<td>wire fracture</td>
</tr>
<tr>
<td>5</td>
<td>green</td>
<td>106</td>
<td>31.8</td>
<td>wire fracture</td>
</tr>
<tr>
<td>6</td>
<td>red</td>
<td>64</td>
<td>19.2</td>
<td>wire slip</td>
</tr>
</tbody>
</table>

Figure 8 Results from tests of the bending capacity at midsection.

### 3.3.2 Bending capacity of rail section

The red sleepers have a moment capacity of between 9 and 11 kNm while the green sleepers manage approximately 45 kNm, see Figure 9. According to BVF 522.32 (1995) the sleepers must manage 26.25 kNm.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Class</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$M_{\text{max}}$ [kNm]</th>
<th>Failure type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>red</td>
<td>60</td>
<td>9</td>
<td>wire slip</td>
</tr>
<tr>
<td>3</td>
<td>red</td>
<td>72</td>
<td>10.8</td>
<td>wire slip</td>
</tr>
<tr>
<td>4</td>
<td>green</td>
<td>301</td>
<td>43.2</td>
<td>wire fracture</td>
</tr>
<tr>
<td>6</td>
<td>red</td>
<td>69</td>
<td>10.4</td>
<td>wire slip</td>
</tr>
</tbody>
</table>

Figure 9 Results from tests of the bending capacity at the rail section.
3.3.3 **Horizontal load capacity of the fastener**

The horizontal load carrying capacity, 100-130 kN see Figure 10, for the fasteners in the green and yellow sleepers are much beyond the load imposed by the trains, which is of the order of 30 kN. Even the red sleeper with the lowest maximum capacity of 18 kN for a deformation of 5 mm may function if it is surrounded by green and yellow sleepers.

![Figure 10 Results from the tests of the load carrying capacity of the fastener for the tested sleepers.](image)

**Discussion**

The bending capacity of the midsection of the tested sleepers is enough to prevent failure with a safety factor of 1.75 against failure even for the sleepers in class 1 (red sleepers).

The bending capacity of the rail section of the tested sleepers is on the other hand not high enough for the sleepers in class 1 (red sleepers). This is probably due to bad anchorage for the strands when the cracking is so severe.

Small cracks, corresponding to class 2 (yellow sleepers), do not seem to influence the horizontal load carrying capacity of the tested fasteners significantly. It is first when the cracking is very severe (red sleepers, where both the longitudinal cracks and the vertical cracks appear) that the load carrying capacity is reduced so much that it is approaching the level of the applied load. The vertical crack at the fastener and downwards probably comes from track forces, i.e. the presence of this crack depends on where it has been lying in the track (curves etc) where it has been exposed to high forces.

It was also possible to see a variation of the load carrying capacity of the fastener for the yellow sleepers. But the results show that there is nothing to gain by dividing the yellow sleepers into groups since they manage approximately the same loads.
The material properties of the concrete in the tested sleepers were high. The mean value for 16 compression tests was 100.7 MPa and the mean value for 13 tensile tests was 3.9 MPa.

The sleepers produced with inferior methods are now inspected annually in order to see at what rate the cracking is progressing.

The big question is now how fast a green or yellow sleeper turns into a red one, i.e. how fast is the formation of ettringite? Probably weather conditions have an important role. If a sleeper will be exposed to cyclic frost erosion the process ought to accelerate considerably. An indication of that the weather conditions may play an important role has been discovered on one sleeper. The midsection of this sleeper had been covered with equipment for the signalling system a so-called “Balis” and this part seemed to be in better conditions (no visible cracks) in comparison with the ends where the crack system had developed quite a bit.
4 Concrete fatigue capacity. A study of deformations at tensile forces – Paper C

4.1 Background. Aim and Scope
In this paper results and analyses are presented from uniaxial cyclic tensile fatigue tests. The tests can be considered as pilot tests. The fatigue tests in this paper can be compared to a deformation criterion proposed by Daerga & Pöntinen (1993) based on an idea of Balázs (1991). The model has successfully been used to describe bond failure between rebars and the concrete. The growth in deformation during a fatigue test can according to the model be divided into three phases, see Figure 11. In the beginning of the first phase the deformation rate is high but stagnates after a while. The second phase is characterised by a constant deformation rate. These two phases can be described as stable. During the third phase, the failure phase, the deformation rate increases rapidly leading to failure within a short time. The strain criterion for fatigue failure is that the strain at peak load during a static test corresponds to the strain at the changeover between phases two and three during a fatigue failure, see Figure 11.

Figure 11 Strain criterion for fatigue failure according to Balázs (1991).
When $\varepsilon(\sigma_u)$ has been reached only a limited number of cycles are needed until failure occurs. Since there is a difference between the number of cycles at failure and at initiation of phase three one can consider the criterion as safe, Balázs (1991).

4.2 Experimental program
The tensile fatigue tests have been performed on two types of specimens with different shapes. One type was casted in a special steel mould equipped with a half-circle notch with the radius 15 mm. The other type of specimen was drilled out from a small concrete beam. It was later cut into a suitable length and a notch was milled. For dimensions see Figure 12 and Figure 13.

![Figure 12 Dimensions of the two different types of specimens used in the tensile fatigue test, Andersson (2000).](image)

![Figure 13 Photo showing the two types of shapes that were used in the tensile fatigue tests, Andersson (2000).](image)

All fatigue tests have been performed under load control with a sinusoidal load cycle. The lower load level (A) was held constant and the higher level (B) was varied. Since a fatigue test can last a very long time depending on what load levels that are used, the decision was made to increase the higher level if the strain still were small after 2000 load cycles. During the stop when the level was adjusted the load was kept at a mean value of the two earlier load levels by the machine. The load frequency has during all
tests been 2.004 Hz. The test set-up is shown in Figure 14. In order to decide the maximum load capacity of the concrete (so that the load levels in the fatigue test could be set) a number of uniaxial tensile tests were performed.

![Test set-up uniaxial tensile test and fatigue tensile test.](image)

**Figure 14** Test set-up uniaxial tensile test and fatigue tensile test.

### 4.3 Results

In Figure 15 and Figure 16 some results from the fatigue tests are presented.

![Result from the cyclic tensile fatigue test of specimen A5.](image)

**Figure 15** Result from the cyclic tensile fatigue test of specimen A5. The strain development for the COD-gauge that has been exposed to the highest strain is shown in (a). This curve is compared with the normalized average peak load obtained in the uniaxial tensile tests (b).
Figure 16 Results from cyclic fatigue tests of specimen S2. (a) The strain development is displayed versus the number of load cycles. (b) Tensile strength displayed versus the total strain. For the test the upper load level B corresponds to 90, 94, 96, 99 and 102% of the average peak load obtained in the uniaxial tensile tests (note that the “average” peak load for series S is based only on two tests).

4.4 Discussion

Strain Criterion

The conformity between the maximum strain obtained in the uniaxial tensile test and the total strain when phase 3 begin (according to Figure 11) is reasonably good for some of the specimens see Figure 15. The results from the tests indicate that the strain criterion proposed by Balázs (1991) might also be applied to plain concrete exposed to cyclic tensile fatigue load.

Fatigue tests

An interesting observation from the tests in Figure 16 is that it is the initial and the final load level that seems to give the largest contributions to the total strain development. The load cycles in between give rise to only about 20% to the total strain. If this phenomenon is compared with the three phases described in Figure 11 it indicates that the increase of $S_{\text{max}}$, i.e. $\sigma_{\text{max}} / f_{\text{ct}}$, (strength levels 3.33, 3.42 and 3.51 MPa) belongs to phase 2, the stable part of the fatigue process with a nearly constant strain rate. The reason for that so many levels have been needed in order to obtain a fatigue failure for this specimen is that the value of the average peak load for series S is not accurate. The other tests show almost the same behaviour as specimen S6. A thing that needs further studies is the influence the increase in load during a test has on the final result.

One of the most interesting parts of the fatigue curve in e.g. Figure 15 is the strain rate (equal to the angle of the slope) for phase 2 since it indicates how close you are to failure. A high strain rate (i.e. steep slope) means that not so many cycles are left until fatigue failure occurs and a low one (i.e. flat curve) that fatigue failure will not occur in a near future (at the same conditions). It is not easy to perform an analysis of the factors that influences the strain rate during phase 2 (i.e. the slope) from the results obtained in this study. This is due to the reason that there are too few tests with enough load cycles (i.e. before the failure) in order to make it possible to calculate the strain rate ($C =$
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d(ε/dN). A hypothesis would be that C is decreasing with decreasing $S_{max}$ and increasing $R$-values (decreasing amplitudes).

The tests have also given valuable information on how a test like this shall be performed (type of adhesive, the way the data are collected etc.).
5 Shear fatigue capacity – a comparison between the Swedish Code and Eurocode – Paper D

5.1 Background. Aim and Scope
In this paper a comparison is made between the Swedish Codes and Eurocode regarding concrete fatigue in shear. The aim is to see how the fatigue phenomenon is treated in the two codes. It is also very interesting to see in what direction the fatigue ideas are going in Europe since there is a continuous work with new drafts of the Eurocodes. A thing that makes it even more interesting is the fact that we must soon use the Eurocode here in Sweden.

The codes that have been used and are referred to in this chapter:


Since a comparison of this kind is not “easy” due to the differences in strength classes, partial safety factors, load combinations etc., this comparison is made in such a way that the load calculation follows the Swedish Codes. However, the strength classes are taken from the EC2-draft (1999).

The bridge that has been used in this example is the Lautajokki Bridge. This bridge was exposed to 6 million load cycles (axle load 360 kN) without collapsing, which was considerably more than what the Swedish Code indicated. The test was performed in the laboratory at Luleå University of Technology during 1996. The calculation is made on 1 m of the slab.
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The trough consists of a slab connected to and carried by two longitudinal beams. The bridges of this standard type were usually built between 1950 and 1980 with a concrete compression strength of about 40 MPa (tested on 150 mm cubes). The older bridges were designed according to principles where the shear capacity of the concrete was somewhat overestimated. This leads to problems when the bridges are recalculated with the design strength for the higher axle loads.

The load case that has been used for the calculation is shown in Figure 5.1, Figure 5.2 and Figure 5.3.

Figure 5.1 Load case with two bogies from two iron ore wagons, Nilsson et al. (1999).

Figure 5.2 Cross-section of the Lautajokki Bridge, Nilsson et al. (1999).

Figure 5.3 Reinforcement in the longitudinal direction in the slab in the Lautajokki Bridge.

The comparison can also be found in the report Thun et al. (2000).

5.2 Results and Discussion

First of all it must be said once again that it is very difficult to make a comparison of this kind between the codes. Though, this comparison nevertheless gives an idea how the two codes are designed.

One general difference between the codes is how you check the fatigue capacity. In Eurocode you have three different “levels”. The first check is very easy to perform since the calculations are very basic. If your structure passes this level you do not have to go any further. The following level needs more calculations and more information
and the last level is very complicated since it is connected to so-called $\gamma$-values (factors for loads, traffic etc.). These “levels” cannot be found in BBK94.

In Table 5.1 you can see a summary of the number of load cycles the Lautajokki Bridge could manage with the compared codes. In the table the more refined methods from the different codes are compared.

If we compare the results in Table 5.1 we can see that there are differences. The calculation with EC2-2 (1995) gives the least conservative values. The Swedish code is slightly more conservative than the EC2-2, but they are still fairly similar. The new version of Eurocode, EC2-draft (1999), is by far the most conservative one.

For example: for the strength class C30 at $P = 30$ tons the Swedish Code gives that the bridge should manage 2000 cycles before failure, the EC2-draft (1999) gives 50 cycles and the EC2-2 (1995) gives 3800 load cycles. We then have in mind that the bridge managed 6M cycles without failure (full-scale test performed at LTU, 1996), which points out that none of the three compared codes is especially accurate.

Table 5.1 Number of load cycles (n in million load cycles) for the Lautajokki bridge. Comparison between EC2-1 (1991)/EC2-2 (1995), BBK94 (1994) and EC2-draft (1999) at different strength classes.

<table>
<thead>
<tr>
<th>A) Strength Class</th>
<th>Code</th>
<th>Number of load cycles, n [M cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>22.5 tons</td>
</tr>
<tr>
<td>C30</td>
<td>Swedish</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>0.053</td>
</tr>
<tr>
<td>C40</td>
<td>Swedish</td>
<td>&gt;1</td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>223</td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>0.44</td>
</tr>
<tr>
<td>C50</td>
<td>Swedish</td>
<td>&gt;1</td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>3093</td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>1.8</td>
</tr>
<tr>
<td>C60</td>
<td>Swedish</td>
<td>&gt;1</td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>8929</td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>5.1</td>
</tr>
<tr>
<td>C80</td>
<td>Swedish</td>
<td>&gt;1</td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>34659</td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>18.8</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

What are the differences between the three codes? Some of them are:

- The expression to calculate the shear force capacity, see Table 5.2.
Evaluation of Concrete Structures

- The expression to calculate the design fatigue strength.
- The Swedish Code gives an approximate value of the number of cycles before failure (the value is taken from a graph).

The reason why EC2-draft (1999) is much more conservative than the EC2-2 (1995)/EC2-1 (1991) depends on how the shear strength is calculated. The new equation in EC2-draft (1999) to calculate the design value for the shear strength gives a lower value compared to the EC2-1 (1991).

The “new” equation for the design shear strength in EC2-draft (1999) is taken from MC 90 (1993), section 6.4.2.3; “Shear in cracked zones without shear reinforcement”. It has the following formula:

$$ V_{Rd,l} = 0.12 \cdot k \left( 100 \cdot \rho \cdot f_{ck} \right)^{1/3} \cdot b_{c} \cdot d $$

(Eq. 6.4-8 in MC 90 (1993))

The reason to this change is that there has been some uncertainty when it comes to high strength ordinary concrete and to be on the safe side a reduction of the design shear strength has been made.

This reduction has led to, for example that the difference in fatigue shear strength is as much as 35% for C80 ($V_{Rd,ct}$ and $V_{Rd,ct-fat}$), see Table 5.2 and Figure 5.4.

Table 5.2 Design value for the shear strength according to EC2-1 (1991) and EC2-draft (1999).

<table>
<thead>
<tr>
<th>A) Strength Class</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC2-1 (1991) / EC2-2 (1995)</td>
<td>$V_{Rd,l}$ =</td>
<td>189.0</td>
<td>236.2</td>
<td>274.0</td>
<td>292.9</td>
</tr>
<tr>
<td></td>
<td>$V_{Rd,ct}$ =</td>
<td>189.0</td>
<td>236.2</td>
<td>274.0</td>
<td>292.9</td>
</tr>
<tr>
<td>EC2-draft (1999)</td>
<td>$V_{Rd,ct}$ =</td>
<td>176.5</td>
<td>194.2</td>
<td>209.2</td>
<td>222.4</td>
</tr>
<tr>
<td></td>
<td>$V_{Rd,ct-fat}$ =</td>
<td>164.3</td>
<td>178.0</td>
<td>188.7</td>
<td>197.1</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

Figure 5.4 Design value for the shear strength according to EC2-1 (1991) and EC2-draft (1999).
Evaluation of Concrete Structures

An interesting thing is that the trend seems to go in a more conservative direction when it comes to concrete fatigue. Because the newest Code is the most conservative one!

More work is needed in this area and it might be an idea to keep the “old” equation (the one in the EC2-1 (1991)) to calculate the design shear strength when the fatigue capacity is studied.

The studied codes generally give too conservative values, especially for high strength concrete. Further work is needed in this area, especially regarding the shear force capacity.
6 Outlook

Strength development
The next step regarding the Capo-test can be to analyse the failure. A slab has been casted in the laboratory and the Capo-test will be performed on it. The results can then e.g. be compared with different FEM-models.

Concrete sleepers
A possible next step can be to test the fatigue capacity of yellow/red sleepers and to perform some more tests of the bending capacity in the midsection and rail section.

A very interesting part is also the influence of weather conditions on the development of the cracks. This ought to be investigated.

Fatigue tests
The strain criterion may be developed to form the basis for a design and assessment procedure for concrete structures.

Concrete Fatigue Codes
The concrete fatigue codes are probably too conservative. The safety levels ought to be analysed and possibly modified.
References


Evaluation of Concrete Structures


Paper A

CONCRETE STRENGTH DEVELOPMENT IN SWEDISH BRIDGES
by Håkan Thun, Lennart Elfgren and Ulf Ohlsson
CONCRETE STRENGTH DEVELOPMENT IN SWEDISH BRIDGES
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ABSTRACT
In the paper the development of tensile and compression strength is presented for old concrete railway trough bridges. The compression strength has usually increased 50 to 100% over the years for these trough bridges. However the same could not be said for the tensile strength. When the results from drilled out cores were compared for different structural parts, i.e. the slab and the longitudinal beams, it turned out for one bridge that the compressive strength was approximately 15% higher in the longitudinal beam than in the slab. This indicates that there is a difference in strength between the side beams and the slab. Until this difference is verified with further studies, a reduction of the concrete strength with 15% has been proposed when the tests are done in the side beam and the analysis concerns the slab.

In the paper a rather well-known method, the Capo-test, is used to determine the in-situ concrete compression strength in old railway trough bridges. The Capo-test has been compared with the results from drilled cores, which is the common way to determine the in-situ strength of a concrete structure. The study indicates that the Capo-test can be used on objects with old age, but with some restrictions, especially regarding the aggregate size. Results from tests performed on the Lautajokki Bridge in Northern Sweden show that the Capo-test might in some cases overestimate the compressive strength. An improved formula is presented for evaluation of the compression strength from Capo tests. It gives a better correlation than the general one and the one proposed by Rockström & Molin (1989) for old structures, but it might give results on the unsafe side and further studies are necessary.

Keywords: strength development, Capo-test, drilled cores, concrete, bridges

RESEARCH SIGNIFICANCE
During the last decades it has become more and more important to assess, maintain and strengthen structures like bridges, dams and buildings due to increased age of many structures and the high costs to construct new infrastructure. Therefore it is of great interest to find methods to evaluate the concrete strength. In this paper a rather well-known method, the Capo-test, is used to determine the in-situ concrete compression strength in old railway trough bridges. Up to this date the method has primarily been used to estimate the strength of the cover-layer of new structures. The Capo-test has been compared with the results from drilled cores, which is the common way to
determine the in-place strength of a concrete structure. This comparison has led to a new proposal for the correlation between the pullout force from the Capo-test and the compression strength from drilled out cores for old structures. In this paper is also the development of tensile and compression strength for old railway trough bridges presented.

INTRODUCTION

Field studies of the concrete strength development were initiated when an increase of the axle load from 25 tons to 30 tons was planned on the iron ore railway line between Luleå in Sweden and Narvik in Norway. Due to this a program was initiated by the Swedish and Norwegian Railway Authorities to check the actual bearing capacity of the existing bridges and what kind of strengthening procedures that would be necessary to allow the enlarged load.

The railway was originally built between 1884 and 1902, and has over the years been repaired and rebuilt. It has a total length of 473 km and annually about 20 million tons of iron ore and pellets are transported on it. There are some 70 bridges of trough type, see Figure 1, between Luleå and Narvik, and some 200 in the rest of Sweden. The trough consists of a slab, filled with ballast, connected to and carried by two longitudinal beams. The bridges of this standard type were usually built between 1950 and 1980 with design strength of about 40 MPa (tested on 150 mm cubes).

Figure 1 Typical cross-section of a Swedish concrete railway trough bridge. The trough, filled with ballast, consists of a slab connected to two longitudinal beams, Nilsson (1999).

One of the first steps in the investigation was to recalculate a bridge from Lautajokki close to the Arctic Circle with the present design codes. Since this old bridge was designed according to principles where the shear capacity of the concrete sometimes was somewhat overestimated it has led to problems, e.g. regarding the fatigue capacity. In order to check the concrete fatigue capacity, a full-scale fatigue test was carried out on the Lautajokki Bridge, a 29-year-old trough bridge, which had a span length of 6.1 m and a width of 4.1 m and was built in 1967. It had carried traffic for 20 years, up to 1988, when it was replaced. The test was performed in the laboratory at Luleå University of Technology during 1996, Paulsson et al. (1996, 1997). The bridge was exposed to 6 million load cycles (axle load 360 kN) without any signs of a reduction of its capacity (this is considerably more than what the Swedish Code indicated). After the fatigue test the properties of the concrete were examined. The result was that the compression strength had increased over the years. This led to an investigation of the
strength of all bridges along the railway line in question. In this paper some results from this investigation are presented.

STRENGTH DEVELOPMENT AND VARIATION

It is a well-known fact that there is a variation of properties within a member of a structure. According to Bungey & Millard (1996) this can be due to:

- Differences in concrete compaction and curing.
- Differences in the quality of concrete delivered.

Here the influence of difference in quality of delivered concrete is assumed to be random whereas concrete compaction and curing follow patterns according to the member type. Usually the bottom parts are better compacted with higher density than the top parts, where the percentage of ballast may be smaller. This is due to the influence of the gravity force and the stability of the concrete mixture.

The strength variations that can be found in a member differ. According to Bungey & Millard (1996) the variation between the top and the bottom for a beam can be up to 40% and for a slab 20% (here the loss in strength is concentrated to the top 50 mm). It is also said that it is difficult to make any general assumptions since individual circumstances vary widely from structure to structure. This variation of strength in a member, i.e. higher in the bottom than in the top, could also be found in e.g. Bartlett & MacGregor (1999). The authors have not been able to find any investigations regarding the possible strength variations between members in bridge structures with, e.g. a slab and longitudinal beams.

Most of the bridges in this study are built in the 1960s and there are no records left from the building sites. For the oldest bridges (built in the 1940s and 1950s) even the concrete compaction could be done by “foot”. One thing that must also be kept in mind is the development/improvement that has been over the years. Things that you could do in the 1940s you don’t dream of doing today, e.g. to put plums (i.e. large stones) in the structure to save concrete.

The two things that are of most importance regarding strength variation in this case are: the possible strength variation between the structural members (slab versus the longitudinal beams) and the reliability of the Capo-test as a method itself compared to drilled cores. One interesting question is also how the tensile strength is to be determined for those bridges where only the Capo-test has been performed.

One of the reasons why the Lautajokki Bridge turned out so well in the fatigue test was due to the fact that the concrete strength had increased considerably over the years compared to the design strength. The reason for this increase can be due to several factors. The two most likely are that the concrete delivered to the building site was “better” than what was stipulated or that the hydration of the concrete had continued over the years. This is possible when the cement is coarsely grinded. If this increase of
strength could be the fact for all other bridges along the railway line it could lead to great savings of time and money.

In order to check the strength development with time, data from 45 bridges were collected from the Swedish National Road Administration and the Swedish National Rail Administration, Rådman (1998). The increase in compressive strength from the design strength could be set to approximately 19% based on all the collected data. The actual increase in compressive strength is for the majority of studied bridges considerably larger. The 19% given are therefore to some extent an underestimation, but gives an indication on the safe side, which is important to account for.

The evaluated data indicate that concrete with a lower initial strength for instance K16, K20 experiences the greatest increase in strength (explanation: e.g. K16 means the compression strength of 16 MPa tested on 150mm cubes after 28 days). This is advantageous since most of the older bridges have an initial strength of K16, K20 etc. In Figure 2 a comparison between the design K-value and the K-value after x years is presented which gives the tendency of lower intended K-value giving larger increase of K-value when being tested x years after construction. Two of the bridges are built in the eighties and two are built in the sixties. The rest of them are built during the thirties and forties.

Figure 2 Development of strength (K-value according to drawing to K-value after x years) for bridges from the archives of the Swedish National Road Administration and the Swedish National Rail Administration. The K-value is the 28-days cube strength, tested on 150x150x150 specimens. K40 corresponds to (the nearest one) C30 in Eurocode. From Rådman (1998). (Drawing value = design value).
USED METHODS

The methods that have been used in the field investigation to determine the concrete strength of the concrete railway trough bridges are:

- Capo-test
- Drilled out cores
- Rebound-hammer (Schmidt hammer)

The Capo-test and the rebound-hammer are simpler and less expensive to perform compared to drilling out cores. They also have the advantage that the equipment is lighter and easier to move compared to the equipment used for drilling out cores. This was one of the key-advantages since many of the bridges could only be reached either by train or by hiking.

The Capo-test

The Capo-test is a method to determine the concrete strength of the cover-layer for an existing structure. It was developed in Denmark by German Petersen & Poulsen (1993) in the middle of the 1970s. The Capo-test is a further development of the so-called Lok-test where the pullout bolt is embedded in fresh concrete. Especially at the end of the 1970s and during the 1980s the Lok-test and similar methods were heavily discussed in the concrete society, particularly regarding what property that is actually measured in a pullout test. In the literature you can find that the pullout strength is a measure of the tensile strength, the shear strength or the punching shear or aggregate interlocking, see e.g. Yener (1991). However, a common thing for all studies is that a fairly good correlation has been found between the pullout force and the compression strength.

In this paper the failure is not analysed. Instead the reasonably good correlation between the pullout force and the compression strength has been accepted and utilized to determine the in-situ compression strength.

As mentioned earlier the Capo-test is designed to determine the strength of the cover-layer, but in this investigation efforts have been made to use it as a complement/substitution to the drilled out cores that are stipulated by all Concrete Codes including the Swedish one to determine the in-place concrete compression strength. But, in the Swedish code it is allowed to use other methods, as long as it can be shown that they are as reliable as drilled out cores.

The test procedure of the Capo-test, see Figure 3, consists of drilling a 65 mm deep hole with a diameter of 18 mm using a water-cooled diamond bit. Then a 25 mm recess is made at a depth of 25 mm using a portable router. An expandable split steel ring is inserted through the hole in the recess and expanded by the means of a special tool. Finally the ring is pulled through a 55 mm counter pressure placed concentrically on the surface. The pullout force is measured by the pull machine and can be converted into compressive strength by means of calibration charts provided by
German Petersen & Poulsen (1993), see Figure 4. A description can also be found in for example Bungey & Millard (1996).

![Figure 3 Schematic drawing of the Capo-test, German Petersen & Poulsen (1993) and Bungey & Millard (1996).](image)

The background of the correlation charts is several laboratory and field studies made by the manufacturer as well as by other researchers. Results from the Lok-test are mostly the basis of the correlations charts. The general correlation that must be used to fit the Swedish Code is shown in Figure 4.

![Figure 4 Suggested general correlation between pullout strength measured by Lok-Test and Capo-test and 150 mm standard cube compressive strength. The confidence limits are 95% for 16mm and 32mm maximum aggregate size for an average of 2 reference tests and 4 pullout tests, German Petersen (1997).](image)

Not many studies have been made concerning comparison between the Capo-test and drilled out cores (dimensions 100×100 mm) from old structures. To the authors’ knowledge only one study has been made, Rockström & Molin (1989). Their study showed that the correlation proposed by German Petersen (1997) might not give a perfect correlation when the test object is an old structure, i.e. old road bridges. They performed tests with both the Capo-test and drilled out cores on six road bridges that had ages up to 54 years (1989). Their result showed that if the general correlation was used the compression strength from the Capo-test, Eq. 1, differed considerably compared to the drilled out cores, in the sense that the compression strength was
underestimated. Rockström & Molin then proposed a revised equation for 100×100 mm cores, see Figure 5. One can see that the difference increases with higher compressions strength.

![Figure 5 Correlation between Capo-test and 100 mm × 100 mm cores, trimmed and air cured 3 days before testing, made by Rockström & Molin (1989) on 5 old Swedish bridges. The correlation is compared to the general correlation for standard cubes (Eq. 1), German Petersen (1997).](image)

The reasons for this discrepancy in correlation for old structures could according to Rockström & Molin be due to: (a) Difference in concrete strength of the cover-layer and concrete further into the structure (the cover-layer represents the strength 25 mm into the structure and a core 50 mm or more, Bellander (1977)). (b) For older structures the aggregate size may vary greatly (an increase in aggregate size from 8 to 32 mm increases the pullout force between 10 to 35%). (c) Risk for irregular and insufficient concrete compaction.

Worth mentioning is that the study in Rockström & Molin (1989) was based on five objects and where the Capo-test and cores were taken from the same test spot. The results from one bridge were rejected due to the great difference between the Capo-test and the drilled out cores (low strength of the cover-layer due to high porosity).

### Drilled cores

To use drilled out cores to estimate the in-situ strength of a structure is a common method. Most countries have adopted standard procedures for how a core should be prepared, stored, etc. before testing. In this study the preparation, the storage etc. have been made according to the Swedish Concrete Code, BBK94 (1994,1996). The cores have been air cured for at least three days before testing.

The cores have been used for uniaxial tensile tests, splitting tensile tests and compression tests. A drill with diamond edges has been used. The inner diameter was 93.8 mm. The length/diameter ratio has been 1.0. The cores have been marked with a drill hole number and a serial number, see Figure 6.
Rebound test (Schmidt-hammer)
The rebound test with a Schmidt-hammer is an old and well-known method to determine the surface hardness. Since the first variant of this kind was developed in the late thirties it has been a subject for researchers ever since. It has by several researchers been suggested not to use it as a substitution for other methods to determine the strength since it is not a reliable method due to the fact that there are so many factors that influence the result, see for example Bungey & Millard (1996) or Malhotra & Carino (1991).

In this study the method has only been used in order to determine the uniformity of the concrete for the bridges respectively and therefore no results are presented.

FIELD INVESTIGATIONS AND TEST PROGRAM

Lautajokki
An enlarged investigation of the concrete strength was performed on the Lautajokki Bridge. In Figure 6 the dimensions of the bridge are shown. A total of 12 cores were taken from the slab and 10 from the longitudinal beams. In most tests the section of the core that has been used is the “upper” core (i.e. part i:1 in Figure 6) due to heavy reinforcement in the bottom of the slab. In the longitudinal beams the cores are taken from the upper side, see Figure 6.

Figure 6 Dimensions and cross-section of the Lautajokki Bridge. The illustration also shows in principle where the cores have been taken.
Eight road underpasses

In order to get a reference material regarding the strength of the trough bridges the compression strength was examined for eight bridges. The method that was used was drilled out cores which is the method stipulated by the Swedish Concrete Code BBK94 (1994,1996). At a later stage of the project also Capo-tests were also performed so that a comparison between the two methods could be made. The bridges are built between 1965 and 1980 with the concrete class K40 (compression strength 40 MPa tested on standard 150mm cubes) with a maximum aggregate size of 32 mm.

RESULTS AND ANALYSIS

Compression strength

In Table 1 the results are presented from the extended investigation of the Lautajokki Bridge. It is shown that the compression strength determined by the drilled cores is dependent on where the core is taken. The difference between cores from the longitudinal beams and the slab regarding compression strength is 12.7 MPa, which corresponds to about 15%.

<table>
<thead>
<tr>
<th>Part of structure</th>
<th>Compression strength, drilled cores No.</th>
<th>f_{cc} [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>slab</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1:2</td>
<td></td>
<td>69.9</td>
</tr>
<tr>
<td>P3:2</td>
<td></td>
<td>70.8</td>
</tr>
<tr>
<td>P5:2</td>
<td></td>
<td>78.2</td>
</tr>
<tr>
<td>P6:2</td>
<td></td>
<td>66.5</td>
</tr>
<tr>
<td>P2:1</td>
<td></td>
<td>69.3</td>
</tr>
<tr>
<td>P2:2</td>
<td></td>
<td>74.1</td>
</tr>
<tr>
<td>P4:1</td>
<td></td>
<td>74.0</td>
</tr>
<tr>
<td>P4:2</td>
<td></td>
<td>77.8</td>
</tr>
<tr>
<td>m =</td>
<td></td>
<td>72.6</td>
</tr>
<tr>
<td>s =</td>
<td></td>
<td>3.9</td>
</tr>
<tr>
<td>V =</td>
<td></td>
<td>0.05</td>
</tr>
</tbody>
</table>

In Table 2 a summary is presented of the results from the eight road underpasses. The correlation used to calculate the compression strength obtained with the Capo-test, is the general one proposed by German Petersen (1997), see Eq.1.

In Table 2 and Figure 7 one can see that for only one bridge (bridge no. 6) the Capo-test gives a result higher than the core, i.e. on the unsafe side, if the Eq.1 is used to evaluate the pullout force. One reason for this could be that there are only three tests that have been performed on this bridge. For bridges no. 7 and 8 Eq.1 gives a difference between the Capo-test and the core strength on a level that could be
expected (about 8 MPa). But for the other bridges Eq. 1 gives very low strengths compared to the core strength. It is almost a difference of 20 MPa, in other words the correlation equation could be improved.

It is also interesting that the difference that exists between the core strength of the slab and the longitudinal beam, is not reflected on the result from the Capo-test. Instead the Capo-test indicates that the strength is higher in the slab than in the longitudinal beam. One explanation might be that too few Capo-tests are performed on the slab (they are also performed vertically). In German Petersen (1993) one can read that testing the upper side of a slab gives a higher standard deviation than testing on a vertical surface.

The mean value of all core compression tests is 73.7 MPa (standard deviation $s = 8$ MPa and the coefficient of variation $V=0.11$) and 64.4 MPa for the Capo-test (standard deviation $s = 7.56$ MPa and the coefficient of variation $V=0.12$).

![Figure 7 Mean value of the concrete compression strength for eight road underpasses. The names of the bridges could be found in Table 2.](image-url)
Table 2 Concrete compression strength for eight trough bridges. Explanations: m = mean value, s = standard deviation and V= coefficient of variation. The cores are taken from the longitudinal beams if nothing else is said.

<table>
<thead>
<tr>
<th>Bridge No.</th>
<th>Bridge Name</th>
<th>Compression strength, Drilled Cores</th>
<th>Bridge No.</th>
<th>Bridge Name</th>
<th>Compression strength, Drilled Cores</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Capo-test F a</td>
<td>f c' b)</td>
<td></td>
<td>Capo-test F a</td>
<td>f c' b)</td>
</tr>
<tr>
<td></td>
<td>[kN]</td>
<td>[MPa]</td>
<td></td>
<td>[kN]</td>
<td>[MPa]</td>
</tr>
<tr>
<td>1</td>
<td>Boden C</td>
<td>34.9 46.3 -</td>
<td>6</td>
<td>Kallkällevägen</td>
<td>- 71.7 -</td>
</tr>
<tr>
<td></td>
<td>m = 53.6 s = 8.4</td>
<td>v = 0.16</td>
<td></td>
<td>m = 68.8 s = 10.1</td>
<td>v = 0.15</td>
</tr>
<tr>
<td>2</td>
<td>Garnisonsgatan</td>
<td>48.8 65.9 -</td>
<td>7</td>
<td>Bensbyvägen</td>
<td>42.6 57.2 -</td>
</tr>
<tr>
<td></td>
<td>m = 68.9 s = 5.2</td>
<td>v = 0.07</td>
<td></td>
<td>m = 58.4 s = 8.1</td>
<td>v = 0.14</td>
</tr>
<tr>
<td>3</td>
<td>Gammelstad</td>
<td>42.7 57.3 -</td>
<td>8</td>
<td>Lautajokki (long. beam)</td>
<td>54.6 77.1 -</td>
</tr>
<tr>
<td></td>
<td>m = 57.7 s = 2.8</td>
<td>v = 0.05</td>
<td></td>
<td>m = 76.9 s = 5.4</td>
<td>v = 0.03</td>
</tr>
<tr>
<td>4</td>
<td>Notviken</td>
<td>- -</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>m = - s = -</td>
<td>v = -</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Haparandavägen</td>
<td>49.1 66.3 -</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>m = 66.5 s = 9.3</td>
<td>v = 0.14</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In Figure 9 the results from the field survey of 36 trough bridges are presented. All bridges have been tested with the Capo-test. Eq. 1 has been used to correlate the pullout force from the Capo-test to compression strength. The bridges in Figure 9 are sorted after the year of construction (the oldest to the left). The bridges are built between 1953 and 1980 with the concrete class K40 or K45 (40MPa or 45 MPa compression strength tested on standard 150mm cubes).

Bridges No. 1 to 8 are from the fifties, No. 9 to 24 from the sixties and 25 to 36 from the eighties. Maximum aggregate size is 32 mm for the bridges built in the eighties. It is probably also 32 mm for the bridges built during the fifties and sixties due to the heavy reinforcement in the bridges but it can be as high as 128 mm. For bridges No. 10 and 23 big differences in aggregate size has been confirmed by drilled out cores.
Figure 8 shows the location where the Capo-test has been performed on the bridges (tests on both sides).

Figure 8 Figure showing where the Capo-test has been performed on the bridges.

Figure 9 Graph showing the mean value of the compression strength and its standard deviation obtained with the Capo-test on 36 trough bridges. The pullout force from the Capo-test is evaluated with the general correlation, Eq. 1.

The next step in the survey was to determine the concrete class according to the Swedish code for each bridge in Figure 9, see Thun et al. (1999). After that the bridges were recalculated with the new axle load 30 tons, by means of the characteristic compression strength, which is connected to each concrete class.

**Tensile strength**

One difficulty was how to determine the tensile strength for the bridges where only the Capo-test had been performed. For the Lautajokki Bridge on the other hand there were two possible ways: the uniaxial tensile test and the splitting test. There was also the possibility of using the compression strength (mean value) obtained by the Capo-test and then use empirical equations between compression strength and tensile strength. In the literature some equations to convert compression strength into tensile strength could be found. In Möller et al. (1994) the following equation is proposed:

\[ f_t = A \cdot f_c^B \quad \text{MPa} \]  

(3)

where \( A = 0.21 \) and \( B = 2/3 \) or \( A = 0.24 \) and \( B = 2/3 \).
Another correlation to determine the tensile strength is the one that could be found in CEB-FIP (1993):

\[ f_{tk} = \alpha \left( \frac{f_{tk}}{f_{d0}} \right)^2 \text{ MPa} \tag{4} \]

where \( \alpha_{\text{min}} = 0.95 \text{ MPa}, \alpha_{\text{mean}} = 1.4 \text{ MPa}, \alpha_{\text{max}} = 1.85 \text{ MPa}, f_{d0} = 10 \text{ MPa} \)

In Figure 10 examples show the variations obtained when empirical expressions are used to determine the tensile strength. This variation could also be observed using the same equation but with different constants \( \alpha \) and \( A \), see equation 3 and 4. The two equations have also been established on young concrete.

![Figure 10 Variation obtained when different correlations between compression and tensile strength are used.](image)

The result from these equations differs a great deal from each other as can be seen in Figure 11. The tensile strength calculated with the compression strength from the Capo-test as basis with equation 4 is higher than the results from the splitting test and the uniaxial test. If Eq 3. is used only the results from the uniaxial tensile test are lower. The difference between the splitting test and the uniaxial test is about 35%. The result in Figure 11 shows the difficulty in using equations to correlate the tensile strength with the compression strength, since it is very difficult to choose factors for the equation.
Figure 11 Tensile strength determined with different methods on the Lautajokki Bridge. The methods that have been used are: uniaxial tensile tests (cores), splitting tensile tests (cores) and tensile strength calculated with Eq.3 and 4 with the mean value from the Capo-test. Note: For the splitting tensile strength the tensile strength is set to 80% of the tensile splitting strength according to the Swedish Concrete Code BBK94 (1994,1996). It is also revised with 7% due to the smaller dimensions (100×100mm) instead of the standard dimensions 150×300mm, Möller et al. (1994).

Since the methods discussed above do not work for the bridges where only the Capo-test has been performed another method must be used to determine the tensile strength for these bridges. The problem was approached in the way that a concrete class was established for each bridge (with the mean value of the compression strength from the Capo-test), e.g. K60. In the Swedish concrete code K60 means that the strength of the concrete should be 60 MPa tested on standard 150mm cubes after 28 days. The corresponding characteristic design value for the tensile strength was then used but with the limitation that the tensile strength could not be higher than 2.25 MPa (which corresponds to concrete class K50 in the Swedish code) even if the Capo-test resulted in a higher concrete class. In Thun et.al. (1999) a method is included how to achieve characteristic tensile strength from test results according to the Swedish Concrete Code BBK94 (1994,1996).

This conservative limitation is due to the fact that from these studies it is not possible to say that the concrete tensile strength has increased as much as the compression strength over the years. The results vary too much. Micro cracks may also form in the tensile zones of the bridges, reducing the tensile strength.

In Table 3 the results for the Lautajokki Bridge are presented. The tensile strength for the Lautajokki Bridge shows a similar variation as the compressive strength, i.e. variation between the structural parts. For the uniaxial tensile test the difference is 0.3 MPa (8.5%) and for the splitting test 0.2 MPa (4%), see Table 3. If this difference is significant for all bridges in the survey is of course uncertain since there has only been a thorough investigation of one bridge. This topic could be target future studies.
Table 3 Concrete tensile strength for the Lautajokki Bridge performed on 100×100 mm cores. The test methods that have been used are uniaxial tensile test and splitting test. Note: For the splitting tensile strength the tensile strength is set to 80% of the tensile splitting strength according to the Swedish Concrete Code BBK 94 (1994, 1996). It is also revised with 7% due to the smaller dimensions (100×100mm) instead of the standard dimensions 150×300mm since a smaller core give higher values, Möller et al. (1994).

<table>
<thead>
<tr>
<th>Part of structure</th>
<th>Tensile strength, drilled cores</th>
<th>Splittings strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniaxial test</td>
<td>No.</td>
</tr>
<tr>
<td>slab</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

In Table 4 and Figure 12 values are presented from the uniaxial tensile tests for eight road underpasses. The uniaxial tensile test on the drilled cores resulted in a mean value for all bridges of 3.3 MPa (standard deviation s = 0.39 and the coefficient of variation V=0.12).

Figure 12 Mean value and standard deviation of the tensile strength from the uniaxial tensile test for eight road underpasses.
### Table 4 Mean value and standard deviation of the tensile strength from the uniaxial tensile test for eight road underpasses.

<table>
<thead>
<tr>
<th>Bridge No.</th>
<th>Drilled Cores</th>
<th>Tensile strength, $f_{ct}$ [MPa]</th>
<th>Bridge No.</th>
<th>Drilled Cores</th>
<th>Tensile strength, $f_{ct}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.28</td>
<td>5</td>
<td>3.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boden C</td>
<td>3.16</td>
<td>3.31</td>
<td>3.49</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.6</td>
<td>0.4</td>
<td>0.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 5</td>
<td>V = 2.6</td>
<td>m = 3.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.4</td>
<td>V = 0.3</td>
<td>s = 0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.16</td>
<td>V = 0.09</td>
<td>s = 0.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>4.06</td>
<td>6</td>
<td>2.72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Garnisonsgatan</td>
<td>3.60</td>
<td>3.82</td>
<td>3.26</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.05</td>
<td>V = 0.07</td>
<td>V = 0.07</td>
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<td></td>
</tr>
<tr>
<td>3</td>
<td>2.77</td>
<td>7</td>
<td>2.72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gammelstad</td>
<td>3.76</td>
<td>3.3</td>
<td>2.53</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.30</td>
<td>0.4</td>
<td>0.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.12</td>
<td>V = 0.22</td>
<td>V = 0.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3.76</td>
<td>8</td>
<td>3.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Notviken</td>
<td>4.31</td>
<td>3.27</td>
<td>3.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>0.4</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.3</td>
<td>V = 0.17</td>
<td>V = 0.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>s = 0.2</td>
<td>V = 0.08</td>
<td>V = 0.08</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*For individual values see table 3*

### A proposal for a new correlation for the CAPO –test for old structures

If the correlation proposed by Rockström & Molin (1989) is used instead of the general one the correlation for bridge no.1 is better (67.1 MPa instead of 53.6 MPa). For the other bridges, except bridge no. 3, the values of the compressive strength are higher than the core strength, i.e. on the unsafe side. This is probably due to the fact that the correlation proposed by Rockström & Molin (1989) is valid in the range of 11 - 66 MPa and the bridges in this study are on the edge or outside this range.

If the mean values of the pullout forces are plotted versus the mean values of the compression strength in Table 2, together with the data from Rockström & Molin (1989) the result is according to Figure 13. For these data a regression analysis (linear model) has been performed, see Figure 13 and appendix A. Unfortunately it is not possible to use all the data obtained in this study. Thus it is not possible to connect the pullout force from a Capo-test to a certain core. This is due to the fact that several companies have performed the tests and there is no record of core numbers (the cores were taken prior to this study). So it is only possible to connect the pullout force from the Capo-tests to the core strength for a whole bridge.
The regression equation is:

\[ F = 0.59 \times f'_c + 2.46 \quad N \]  

(5)

In Figure 13 two values for very old bridges are also presented, the ones with a circle. They are not included in the regression analysis since the cores from these bridges showed that the concrete used in the bridges was not a “modern” one. The mixture consisted of a few very big aggregates (approximate 70 mm) and the rest were small ones (approximate 8 mm) combined with mortar. As one can see in Figure 13 these two bridges give somewhat different values compared to the others.

![Figure 13 Proposal for new correlation between Capo-test and 100 x100 mm cores for old bridges. Regression analysis is based on y = kx + m, where the standard error is 1.8683 for m and 0.0359 for k. Coefficient of determination, R-squared, is 0.927. Estimate standard error is 3.61 MPa. The confidence limit is 95%, dotted lines. The equation is valid in the interval 11 to 85 MPa. It is similar to the equation proposed by Rockström & Molin (1989) but it reaches further than that correlation, up to 85 MPa, and the slope for the higher regions is steeper, which leads to lower values of the compression strength, see Figure 14.](image-url)
If Eq. 5 is used, the correlation is better between the Capo-test and the core strength. But still, for bridges 6, 7 and 8 the equation overestimates the compressive strength, i.e. on the unsafe side. The conclusion of this regression analysis is that more studies are needed if a good and reliable correlation for old structures will be possible to develop. The decision that was made was to use Eq. 1 since the other correlation might give results on the unsafe side.

**SUMMARY AND CONCLUSIONS**

**Capo-test**
This study indicates that the Capo-test can be used on objects with old age, but with some restrictions. The results from the tests performed on the Lautajokki Bridge show that the Capo-test might overestimate the compressive strength compared with drilled out cores. The Capo-test gave higher values when it was performed on the longitudinal beams and the slab than drilled out cores from the slab. If this is something general for all actual bridges is hard to say while the basis is poor – there are only results from one bridge.

This study along with the study performed by Rockström & Molin (1989) show that if the general correlation proposed by the manufacturer is used, values on the safe side ought to be received for the compression strength. The correlation proposed by the authors gives a better correlation than the general one and the one proposed by Rockström & Molin (1989) for old structures, but it might give results on the unsafe side. More studies are needed of old structures.

Regarding very old structures it is essential to use the Capo-test with caution. This is due to the risk of a big difference in aggregate size in used concrete. For bridge No. 10 in Figure 9 the Capo-test resulted in the compression strength 29.7 MPa, but when
cores were taken out later on they gave the compression strength of 56 MPa. The difference was probably due to the great difference in aggregate size.

The Capo-test is included in the new proposal for International as well as Swedish standard for assessing the concrete strength in structures (see prEN 13791:1999: Assessment of concrete compressive strength in structures or in structural elements).

**Compression strength**
When the results from the drilled out cores were compared for the structural parts, i.e. the slab and the longitudinal beams, it showed that the compressive strength was approximately 15% higher in the longitudinal beam than in the slab for the Lautajokki Bridge. This indicates that there is a difference in strength between the side beams and the slab. Until this difference is verified with further studies, a reduction of the concrete strength with 15% should be performed when the tests are done in the side beam and the analysis concerns the slab. This reduction could be introduced as a partial coefficient, see Nilsson et.al. (1999).

Regarding the strength development of the concrete in the bridges it has increased considerably over the years, just for the Lautajokki Bridge the strength has increased from about 40 MPa to 72 MPa for the slab.

The difference in strength between the side beams and the slab needs further studies.

**Tensile strength**
The results of tensile strength vary considerably between the studied bridges (i.e. the longitudinal beams) and it is not possible to say from the results obtained in this study that the tensile strength has increased as much as compressive strength with time. When the tensile strength is determined from compressive strength it should be limited because of the possibility of cracks due to e.g. constraint. In Thun et.al. (1999) a method is included how to achieve characteristic tensile strength from test results according to the Swedish Concrete Code BBK94 (1994,1996).

**ACKNOWLEDGEMENTS**
The investigation was carried out on behalf of the Swedish National Rail Administration. Björn Töyrä and Anders Kronborg together with their staff have provided much information and help. TESTLAB (the test laboratory at Luleå University of Technology) has performed the tests.

**NOTATION**
- $f_t$ calculated tensile strength from equation in Möller et.al. (1994)
- $f_{tk}$ calculated tensile strength from equation in CEB-FIP (1993).
- $f'_c$ Compression strength evaluated from Capo-test
- $F$ pullout force from Capo-test [kN]
concrete compression strength, from cores

\( f_{c,c} \)

concrete tensile strength, uniaxial tensile strength (cores)

\( f_{t,c} \)

concrete tensile strength derived from splitting strength test (cores). The splitting strength is first reduced with 80% according to the Swedish Concrete Code and then additionally reduced with 7% due to different dimension than what is stipulated in the code. The core dimension used in this study is smaller and gives higher strengths than the standard dimension 150*300mm. These two reductions result in \( f_{t,s} \).

\( m \) mean value. Calculated as, 

\[
m = \frac{\sum f_i}{n}
\]

\( s \) standard deviation. Calculated as, 

\[
s = \sqrt{\frac{\sum (f_i - m)^2}{n-1}}
\]

\( V \) coefficient of variation. Calculated as, 

\[
V = \frac{s}{m}
\]

REFERENCES


APPENDIX A REGRESSION ANALYSIS
For the bridges where both cores have been taken and Capo-tests have been performed a linear regression analysis has been performed with a computer program called: STATGRAPHICS Plus for Windows 4.0 (Statistical Graphics Corp.).
The values that have been used are:

### Rockström & Molin (1989):

<table>
<thead>
<tr>
<th>$f_{cc}$ [MPa]</th>
<th>$F$ [kN]</th>
<th>Bridge</th>
</tr>
</thead>
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<tr>
<td>34.5</td>
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</tr>
<tr>
<td>39</td>
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<td>D119</td>
</tr>
<tr>
<td>43.5</td>
<td>25.2</td>
<td>D119</td>
</tr>
<tr>
<td>46</td>
<td>26.5</td>
<td>D119</td>
</tr>
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</tr>
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<td>BD245</td>
</tr>
<tr>
<td>43.4</td>
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<tr>
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<td>11.9</td>
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<tr>
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<td>28.9</td>
<td>H29</td>
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</table>

### Ltu investigation:

<table>
<thead>
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<th>$F$ [kN]</th>
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<tr>
<td>73</td>
<td>40.1</td>
<td>Boden C</td>
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<tr>
<td>83.4</td>
<td>50</td>
<td>Garmsongatan</td>
</tr>
<tr>
<td>85.3</td>
<td>54.4</td>
<td>Lautajokki, beam</td>
</tr>
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<td>61.3</td>
<td>50</td>
<td>Kalkällevägen</td>
</tr>
<tr>
<td>79.5</td>
<td>48.3</td>
<td>Haparandavägen</td>
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<tr>
<td>65.3</td>
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Paper B

LOAD CARRYING CAPACITY OF CRACKED CONCRETE RAILWAY SLEEPERS
by Håkan Thun, Sofia Utsi and Lennart Elfgren
LOAD CARRYING CAPACITY OF CRACKED CONCRETE RAILWAY SLEEPERS
by Håkan Thun, Sofia Utsi and Lennart Elf gren

ABSTRACT
In this paper load carrying capacities of cracked as well as un-cracked concrete railway sleepers have been investigated. The cracking is believed to be caused by delayed ettringite formation. The tests that have been performed are:

1. Bending capacity of the midsection.
2. Bending capacity of the rail section.
3. Horizontal load carrying capacity of the fasteners.
4. Control of the concrete properties of the tested sleepers.

Information on how the visual inspection and the classification of damages are performed is presented together with possible failure mechanisms of the fastener when loaded horizontally. The purpose with the tests have been to get information on how the cracking influences the load carrying capacity compared to an un-cracked sleeper.

The sleepers are made of prestressed concrete with the concrete class K60 (compression strength 60 MPa tested on 150mm cubes). The sleepers are prestressed with 10 strands (each strand consists of 3 wires with the diameter 3 mm each).

The test results have been compared with calculations according to the Swedish Rail Code for sleepers. The tests show that small cracks do not seem to influence the load-carrying capacity and it is first when cracking is very severe that the load-carrying capacity is reduced significantly.

Keywords: sleeper, classification, inspection, cracks, concrete, delayed ettringite, load carrying capacity, fastener.

RESEARCH SIGNIFICANCE
The problems that delayed ettringite formation cause for these concrete sleepers are very serious. Therefore it is important for economic but mostly safety reasons to investigate their load carrying capacity. An enlarged understanding of the factors that influence the process is needed in order to be able to give reasonably good predictions of the remaining life length of these sleepers.
INTRODUCTION

Railway sleepers made of concrete have only been the standard during the last decades in Sweden. According to Andersson & Berg (1999) pine was substituted for concrete as material in the 1950s when prestressed concrete became available for sleepers. The advantages of the concrete sleepers were e.g. the longer service life, higher bearing capacity and no use of environmental hazardous chemicals such as creosote (used to increase the service life of the timber).

A railway sleeper has several functions such as: being elastic foundations to the rails, keeping the right distance between the rails, cooperating with the rail so the railway track is resistant to flexuous movement in the horizontal direction, etc. If all above-mentioned reasons are considered it is easily understood that a sleeper must withstand various types of loads, weather conditions etc., without losing its properties during its service life.

Normally concrete sleepers sustain their properties for more than 50 years. However in Sweden some sleepers made between 1992 and 1996 have started to deteriorate. They have obtained cracking of a more or less severe kind and some of them have even lost most of their bearing capacity. The cracking is believed to be caused by delayed ettringite formation, which leads to an internal expansion and with time cracks, see e.g. Tepponen & Eriksson (1987). This process might reduce the service life to as few years as five.

The Swedish National Rail Administration has started investigations. Bad production methods are believed to be the cause of the deterioration. In order to increase the production speed, the cement amount was increased from ordinarily 420kg/m³ to 500kg/m³ and steam curing was used during the hardening process in some of the production plants.

In this paper results from strength capacity tests are presented. The purpose of the tests has been to get an idea of how the cracking influences the load carrying capacity.

In Figure 1 a principal drawing of a Swedish railway sleeper is presented. The sleepers are made of prestressed concrete with the concrete class K60 (compression strength 60 MPa tested on 150mm cubes). The sleepers are prestressed with 10 strands (each strand consists of 3 wires with the diameter 3mm each).
VISUAL INSPECTION AND CLASSIFICATION

Visual inspection

When the Swedish National Rail Administration became aware of the problem with the cracked sleepers, several investigations were initiated. These showed among other things that the damaged sleepers could be found all over Sweden. About 3.5 million sleepers have been inspected and there are about 300,000 sleepers that are cracked. The only way of finding them is by walking along the railway tracks making a visual inspection. Knowing this, it is easy to understand that the investigation is very difficult and time consuming to perform. The investigations are performed in the way that two inspectors walk along the railway track on opposite sides.

Since the sleepers are covered with macadam as in Figure 2, it is only possible to notice damages that are on the upper side of the sleeper and the top 1 to 2 cm along the sides. This makes it very difficult to discover sleepers that are in the beginning of the failure process.
Figure 2 Picture showing how much of a sleeper that is visible when it is placed in the railway track.

Classification

The first inspections led to a categorization of the sleepers depending on the cracking. They were divided into three classes by the Swedish National Rail Administration. The typical damages for each class are:

Class 1. Acute / Red:

The cracking is so severe that there is a considerable reduction of the load carrying capacity. There are typical longitudinal cracks in the middle of the sleeper. There are also cracks at the end of the sleepers with a crackled pattern. The sleepers might have a crack from the fastener and downwards. The concrete surface is discoloured by yellow spots. A typical crack-pattern is shown in Figure 3.

Figure 3 Characteristic crack patterns for sleepers in class 1 (red sleepers).

Class 2. Initial degradation / Yellow:

Some cracks. The cracking is of the kind that the load carrying capacity is almost intact. There might be cracks with a crackled pattern at the end of the sleepers. The sleepers might have a crack from the fastener and downwards. There is presence of yellow spots. Typical crack-patterns are given in Figure 5.
**Class 3. OK / Green:**

No visible cracks. No visible tendencies to develop major faults. No change in colour. The load carrying capacity is intact.

Combined with these classes the Swedish National Rail Administration has set up safety rules like: maximum number of damaged sleepers that can lie next to each other for a certain curve radius etc. before they must be replaced.

Since the yellow sleepers are so many (about 300 000 up to this date) and the variation in cracking is so large, they have been divided into subcategories in the hope of finding out if there is a variation in load carrying capacity among them. The criterion that has been used as a basis is what kind of cracks an inspector has a chance of discovering when he/she walks along the railway track. Since the sleepers are covered with macadam as in Figure 2, it is only possible to notice damages that are on the upper side of the sleeper and 1 to 2 cm along the top parts of the sides. The cracks that have been used as target have a width larger than 0.05mm. These cracks are possible to see with the naked eye and can be discovered without the need to get down on one’s knees. These cracks are in this paper called *visible cracks*.

The area on the sleeper where the first visible cracks appear (when they lie in the track) seems to be on the upper side at the end, near the edge. This leads to a problem since most yellow sleepers also have cracks on the side towards the lower edge, see Figure 5 and Figure 4. These cracks are not possible to detect at an inspection as long as the macadam is not removed. This might lead to the fact that a yellow sleeper is given the class green.

![Figure 4 Illustration showing where in a sleeper the cracks first seem to appear.](image)

The subdividing of the yellow sleepers is thus only based on visible cracks on the upper side of the sleeper, at the end. Worth pointing out is that not all sleepers have two ends with the same type of damages. Some sleepers have several cracks at one end but no cracks at the other.

The yellow sleepers have therefore in turn been subdivided into three categories:

*Group 1* Several cracks on the upper side with a crackled pattern, see Figure 5 c).

*Group 2* One or two cracks on the upper side, see Figure 5 b).
Group 3  No cracks on the upper side, see Figure 5 a).

Figure 5 Typical cracks for sleepers of class 2 (yellow). a) Sleepers in group 3 i.e. no visible cracks on the upper side but there might be cracks on the side at the lower edge. b) Sleepers in group 2 i.e. only 1 or 2 visible cracks on the upper side. They have fewer cracks on the side towards the lower edge than the sleepers in group 1 (the crackled pattern is not yet as “developed” as for group 1 sleepers). c) Sleepers in group 1 i.e. cracks in a crackled pattern on the side as well as on the upper side.

Before the tests started every sleeper delivered to LTU was visually inspected and photographed. The condition of the sleepers varied from OK to very damaged. To be able to see the cracks more easily the sleepers were washed and especially interesting parts were moistened.

The following could be said after the laboratory inspection:

- Generally one can say that if a sleeper only has one or two cracks on the upper side at the sleeper end, it also has fewer cracks on the side compared to a sleeper who has a lot of cracks on the upper side (at the end).
- The concrete was very fragile and pieces had a tendency to loosen.
- The colour of the concrete was brighter compared to the colour of the concrete for sleepers in class 3 (green sleepers).
- Some of the sleepers had no visible cracks on the upper side but they had cracks on the sides, see Figure 5 a).
- All sleepers have discolorations here and there. The colour is brown-yellow.

The sleepers in class 3 (green sleepers) showed none of the signs mentioned above.

LOADS

Bending capacity of midsection
According to the Swedish National Rail Code, BVF 522.32 (1995), a sleeper must have the moment bearing capacity according to Figure 6.
The moment carrying capacity of the midsection must be $M_{\text{max}} = 11$ kNm. Additionally it is stipulated that the safety factor against failure must be at least 1.75. This leads to a moment carrying capacity in the midsection of:

$$M_{f,\text{mid}} = 1.75 \cdot 11 = 19.25 \text{ kNm}$$  \hspace{1cm} (1)

**Bending capacity of rail section**

Figure 6 gives that the moment carrying capacity of the section where the rail is placed must be 15 kNm. With a safety factor against failure of 1.75 this results in a moment carrying capacity of:

$$M_{f,\text{rail}} = 1.75 \cdot 15 = 26.25 \text{ kNm}$$ \hspace{1cm} (2)

**Horizontal load carrying capacity of fastener**

In the Swedish National Rail Code, BVF 522.32 (1995), there is no specification of how the load carrying capacity could be tested or calculated. In this section efforts have been made to estimate the level of the horizontal forces that act on a fastener.

**Horizontal forces caused by the centripetal acceleration**

The horizontal forces that act on a sleeper are partly caused by the centripetal acceleration. It can be written as $v^2/R$, for a train travelling with the speed $v$ in a curve with the radius $R$. In order to reduce this force, the curve can be sloped, i.e. one of the rails is placed higher than the other one, see Figure 7.
Using the level of the track as reference the horizontal acceleration component $a_y$ is:

$$a_y = \frac{v^2}{R} \cdot \cos \varphi_y - g \cdot \sin \varphi_y$$

where

$$\sin \varphi_y = \frac{h_a}{s}$$

$h_a$ = heightening of sleeper ($h_a=h$ in Figure 7)

$s = 1.50$ m i.e. the distance between one pair of wheels

Small angles are assumed.

The horizontal force, $F$, can be written as:

$$F = m \cdot a_y$$

In the calculation it is also assumed that one fastener takes all the forces from one axle when the train is standing still. No consideration is taken of possible tilting of high-speed trains.

Horizontal forces $F$ from equation (4) are shown in Figure 8 for two cases, a freight train carrying iron ore (axle load 30 tons) and a high-speed train (axle load 18.75 tons). The smallest radii $R$ are used, which exist on the railway line they traffic. From the figure it can be seen that the maximum force from one axle is approximately 35 kN for the freight train at 70 km/h and 50 kN for the high speed train at 130-140 km/h. If a train is standing still in a curve the maximum force will come from the freight train, approximate 30 kN. This load is distributed by the rail to two or three neighbouring sleepers. For one fastener the maximum horizontal load will thus be of the order of 12 to 25 kN.
Figure 8 Horizontal force, $F$, as defined in Eq. 4, as function of train speed, $v$, and heightening, $h_a$, of one sleeper end. (a) Freight train with iron ore, $R=335$ m. (b) High speed train, $R=600$ m.

**Horizontal force on track**

In Figure 9 the forces are presented that act between the rails and the wheels.

![Diagram of forces between rails and wheels](image)

Figure 9 Forces that act between the rails and the wheels. Vertical forces $Q_v$ and $Q_h$, lateral forces $Y_v$ och $Y_h$ and the horizontal track on force $S$. The wheel set is seen from behind. Positive direction of forces. From Sahlin & Sundqvist (1995).

It is difficult to calculate the highest possible horizontal force. This is due to the complexity of the forces acting between the rails and the wheels. If the duration of a horizontal $S$-force is very brief, it is not capable to move the track. A duration of two meters is a commonly used distance, see Figure 10.
Figure 10 The horizontal track force, $S$, must have the duration corresponding to a distance of 2m when comparing to allowed value. From Andersson & Berg (1999).

The maximum allowed horizontal force on the track, $S_{2m,till}$, is according to Andersson & Berg (1999) calculated as:

$$S_{2m,till} = K \cdot (10 + \frac{2Q_0}{3}) \text{ kN} \tag{5}$$

where $K$ is a constant depending on the track structure and the degree of consolidation. For freight trains $K$ is 0.85 and for engines and passenger coaches $K$ is 1.0. These values are valid for new tracks.

$2Q_0$ is the static axle load [kN]

Assuming a new and well consolidated track we will get the following for a freight train transporting iron ore, with an axle load of 30 tons:

$$S_{2m,till} = 1.0 \cdot (10 + \frac{300}{3}) = 110 \text{ kN}$$

This force will in turn be distributed on a distance of 2m (for more details see Andersson & Berg (1999)). The iron ore railway has a distance between the sleepers of 0.55m, which leads to the following force that every sleeper must manage:

$$F_{\text{sleeper}} = \frac{110 \cdot 0.55}{2} = 30.2 \text{ kN}$$

**Measured track forces**

Track forces have been measured by the Swedish National Rail Administration on an iron ore engine (class IORE 101) along the iron ore railway line, “Malmbanan”, by direction of Adtranz Kassel. The measurements have been conducted during September and October 2000.

The maximum track force that was measured was 70 kN. The largest lateral forces (Y-forces) that were measured were on average 60 –80 kN. Single “spikes” up to 120 kN appeared though.

These loads correspond approximately to the loads in the calculations above.
TEST SET-UP AND RESULTS

Bending capacity of midsection

The sleepers were placed upside down on bearings at each rail section (distance between bearings: 1500mm). Each bearing consisted of a steel cylinder (diameter 60mm) and two steel plates (thickness 100mm). Between the bearings and the sleeper a rubber pad was mounted.

At the middle of the sleeper two bearings were mounted on the upper side at a distance of 150mm from the symmetry line. Each bearing consisted of a steel cylinder (diameter 15mm) and two steel plates (thickness 50mm). Between the bearings and the sleeper a rubber pad was mounted. On the bearings a steel girder was placed so the load could be applied symmetrically in proportion to the rail sections, see Figure 11.

The displacement was measured by four LVDT-gauges, one on each side of the midsection to compensate for a possible rotation during the test. An LVDT-gauge was also mounted at each rail section to compensate for the possible setting of the supports. The test was run in displacement control and the load was applied with a rate of 0.02mm/s. The sleepers were loaded until failure.

![Figure 11 Illustration showing the test set-up during the test of the bending capacity of midsection.](image)

The results from the tests are shown in Figure 13 and a typical failure is shown in Figure 12. The maximum moment is calculated as:

$$M(0.6) = M_{\text{max}} = \frac{F}{2} \cdot 0.6 \text{ kNm}$$  \hspace{1cm} (6)

The red sleepers have a moment capacity of approximately 19 kNm and the green sleepers approximately 32 kNm. According to BVF 522.32 (1995) the sleepers must manage 19.25 kNm.
Figure 12 Failure of sleeper no. 2 in shear.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Class</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$M_{\text{max}}$ [kNm]</th>
<th>Failure type</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>red</td>
<td>65</td>
<td>19.5</td>
<td>shear</td>
</tr>
<tr>
<td>4</td>
<td>green</td>
<td>109</td>
<td>32.7</td>
<td>wire fracture</td>
</tr>
<tr>
<td>5</td>
<td>green</td>
<td>106</td>
<td>31.8</td>
<td>wire fracture</td>
</tr>
<tr>
<td>6</td>
<td>red</td>
<td>64</td>
<td>19.2</td>
<td>wire slip</td>
</tr>
</tbody>
</table>

Figure 13 Results from the tests of the bending capacity of the midsection for the tested sleepers.

Bending capacity of rail section

The sleepers were placed on two bearings at a distance of 300mm on each side of the symmetry line of the rail section.

Each bearing consisted of a steel cylinder (diameter 60mm) and two steel plates (thickness 100mm). Between the bearings and the sleeper a rubber pad was mounted.

In the middle of the rail section a steel plate, thickness 15 mm and the width 50 mm, was placed with a rubber pad between the concrete and the steel. The test was run in displacement control and the load was applied via a hinged edge with a rate of 0.02 mm/s. The sleepers were loaded until failure.
The displacement was measured by four LVDT-gauges, one on each side of the midsection of the rail section to compensate for a possible rotation during the test. A LVDT-gauge was also mounted at each of the supports.

![Figure 14 Illustration showing the test set-up during the test of the bending capacity of rail section.](image)

The results from the tests are shown in Figure 16 and a typical failure is shown in Figure 15. The maximum moment is calculated from:

\[
M(0.3) = M_{\max} = \frac{F}{2} \cdot 0.3 \text{ kNm}
\]  

(7)

The red sleepers have a moment capacity of between 9 and 11 kNm while the green sleepers manage approximately 45 kNm. According to BVF 522.32 (1995) the sleepers must manage 26.25 kNm.

![Figure 15 Failure of sleeper no. 3 after wire slip.](image)
Figure 16 Results from the tests of the bending capacity of the rail section for the tested sleepers.

**Horizontal load capacity of the fastener**

The test arrangement is shown in Figure 17. The outer of the two fasteners has been tested. The sleeper was placed on a steel girder and tightened to prevent movement. A hydraulic jack and a load cell were mounted on a bar. The tests have been run in load control with a load velocity of 0.4 kN/s. To measure the displacement, an LVDT-gauge was placed horizontally against the fastener.

![Figure 17 Illustration showing the test set-up.](image)

Typical failures are shown in Figure 18 and Figure 19 and test results are given in Figure 20. In Table 1 a summary of all horizontal load carrying capacity tests is presented. According to the calculations presented earlier the sleeper must manage about 30 kN.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Class</th>
<th>( F_{\text{max}} ) [kN]</th>
<th>( M_{\text{max}} ) [kNm]</th>
<th>Failure type</th>
</tr>
</thead>
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<tr>
<td>1</td>
<td>red</td>
<td>60</td>
<td>9</td>
<td>wire slip</td>
</tr>
<tr>
<td>3</td>
<td>red</td>
<td>72</td>
<td>10.8</td>
<td>wire slip</td>
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<td>4</td>
<td>green</td>
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<td>45.2</td>
<td>wire fracture</td>
</tr>
<tr>
<td>6</td>
<td>red</td>
<td>69</td>
<td>10.4</td>
<td>wire slip</td>
</tr>
</tbody>
</table>
The horizontal load carrying capacity, 100-130 kN, for the fasteners in the green and yellow sleepers are much beyond the load imposed by the trains. Even the red sleeper with the lowest maximum capacity of 18 kN for a deformation of 5 mm may function if it is surrounded by green and yellow sleepers.
Figure 20 Results from the tests of fasteners capacity for horizontal load.

Table 1 Summary of all horizontal load carrying capacity tests.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Class a)</th>
<th>Group b)</th>
<th>Max. horizontal force $F_{max}$, [kN]</th>
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<tbody>
<tr>
<td>7</td>
<td>red</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td>16a</td>
<td>red</td>
<td>-</td>
<td>65.5</td>
</tr>
<tr>
<td>16b</td>
<td>red</td>
<td>-</td>
<td>52.5</td>
</tr>
<tr>
<td>5</td>
<td>green</td>
<td>-</td>
<td>117.1</td>
</tr>
<tr>
<td>8</td>
<td>green</td>
<td>-</td>
<td>133.5</td>
</tr>
<tr>
<td>9</td>
<td>green</td>
<td>-</td>
<td>111</td>
</tr>
<tr>
<td>10</td>
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<td>103.8</td>
</tr>
<tr>
<td>12</td>
<td>yellow</td>
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<td>85.2</td>
</tr>
<tr>
<td>18</td>
<td>yellow</td>
<td>3</td>
<td>109.5</td>
</tr>
</tbody>
</table>

  a) Classification performed by the Swedish National Rail Administration.

  b) Classification performed by LTU.

Sleepers No. 16–18 have been tested at a later stage than the others. Worth mentioning is that sleeper No. 7 was in a much worse condition than the other red sleeper (the outer fastener and the concrete surrounding it had fallen off at one end). Note also that sleepers No. 16a and 16b is the same sleeper but different sides. The main reason why sleeper No. 7 managed a much lower load than 16 and 17 is probably because it has a vertical crack from the fastener and downwards. This was also combined with a very developed crack system.
Sleeper No. 17 looks more like the yellow sleepers because only the ends had a lot of cracks. Sleeper No. 16 looks more like sleeper No. 7 due to the fact that it has long horizontal cracks. As mentioned above it does not have the vertical crack at the fastener.

For sleepers No. 5, 7-16a and 17 the tests have been performed according to Figure 21, i.e. with a steel pad between the end of the sleeper and the vertical girder. For sleeper No. 16b and 18 the steel pad was removed to be able to see the influence of this steel pad. In Figure 20 it is shown that for a sleeper of class 2 or 3 (compare with sleeper No. 18) it does not influence the test. For a sleeper that is in class 1 (red sleepers) that have severe cracking the steel pad may influence the test, compare No. 16a and 16b.

![Figure 21 Test set-up. Illustration showing the steel pad used in the tests.](image)

### Material properties

The material properties of the concrete have been determined from uniaxial tensile and compression tests on drilled out cores with a diameter of 68 mm, see Figure 22.

The concrete was specified to have a cube strength of 60 MPa. The cement content was ordinarily 420 kg/m³. In order to increase the production speed, the cement amount was increased to 500 kg/m³ and heat was used during the hardening process in some of the production plants. The test results are summarised in Table 2.

![Figure 22 Test of material properties. (a) Location of cores. (b) Crack planes for the test specimens 9:1p, 9:1 and 12:1p.](image)

The mean value for 22 compression tests was 100.4 MPa with a standard deviation of 6.6 MPa and a coefficient of variation of 0.07 (the lowest value was 85 MPa).
The mean value for 18 tensile tests was 3.8 MPa with a standard deviation of 0.4 MPa and a coefficient of variation of 0.11. Four test specimens that had cracks according to Figure 5 (b) have not been included in the mean value. If these tests are also included in the mean value the results are 3.3 MPa with a standard deviation 1.13 MPa and a coefficient of variation of 0.34.

**Table 2** Summary of the results from the compression and tensile tests. Index p means that the core comes from that half of the sleeper that has been exposed to the horizontal pull test.

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>Sleeper class /part no.</th>
<th>Compression strength</th>
<th>单元拉伸强度</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$F_u$ [kN]</td>
<td>$f_{cu}$ [MPa]</td>
<td>mean</td>
<td>$F_u$ [kN]</td>
</tr>
<tr>
<td>16</td>
<td>red 16a</td>
<td>379.3</td>
<td>85</td>
<td>88.9</td>
<td>7.691</td>
</tr>
<tr>
<td></td>
<td>16b</td>
<td>351.3</td>
<td>92.7</td>
<td></td>
<td>7.309</td>
</tr>
<tr>
<td>8</td>
<td>green 8</td>
<td>356.6</td>
<td>96.6</td>
<td></td>
<td>10.0125</td>
</tr>
<tr>
<td></td>
<td>8p</td>
<td>375.2</td>
<td>101.5</td>
<td></td>
<td>9.7354</td>
</tr>
<tr>
<td>9</td>
<td>green 9</td>
<td>379.3</td>
<td>102.6</td>
<td>98.9</td>
<td>3.3008 (0.90)</td>
</tr>
<tr>
<td></td>
<td>9p</td>
<td>351.3</td>
<td>95</td>
<td></td>
<td>3.6701 (0.99)</td>
</tr>
<tr>
<td>10</td>
<td>yellow/group1 10p</td>
<td>390.6</td>
<td>105.7</td>
<td>105.5</td>
<td>8.9805</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>362.3</td>
<td>98</td>
<td></td>
<td>8.1252</td>
</tr>
<tr>
<td>11</td>
<td>yellow/group1 11p</td>
<td>402.6</td>
<td>108.9</td>
<td>105.5</td>
<td>9.5888</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>404</td>
<td>109.3</td>
<td></td>
<td>7.8843</td>
</tr>
<tr>
<td>12</td>
<td>yellow/group2 12p</td>
<td>386.2</td>
<td>104.5</td>
<td>102.2</td>
<td>9.5105</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>367.2</td>
<td>99.3</td>
<td></td>
<td>1.4154 (0.38)</td>
</tr>
<tr>
<td>13</td>
<td>yellow/group2 13p</td>
<td>385.3</td>
<td>104.2</td>
<td>102.2</td>
<td>8.9323</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>372.4</td>
<td>100.8</td>
<td></td>
<td>10.0566</td>
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<td>14</td>
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<td>320.1</td>
<td>86.6</td>
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<tr>
<td></td>
<td>14</td>
<td>344.9</td>
<td>93.3</td>
<td></td>
<td>7.8265</td>
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<tr>
<td>15</td>
<td>yellow/group3 15p</td>
<td>376.4</td>
<td>101.8</td>
<td></td>
<td>10.4542</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>381.7</td>
<td>103.3</td>
<td></td>
<td>8.8922</td>
</tr>
<tr>
<td>17</td>
<td>yellow/group3 17p</td>
<td>370.6</td>
<td>100.3</td>
<td></td>
<td>8.833</td>
</tr>
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<td></td>
<td>17</td>
<td>383.7</td>
<td>103.8</td>
<td></td>
<td>9.766</td>
</tr>
<tr>
<td>18</td>
<td>yellow/group3 18p</td>
<td>399.1</td>
<td>108</td>
<td>104.0</td>
<td>9.033 (2.44)</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>395.3</td>
<td>107</td>
<td></td>
<td>9.483</td>
</tr>
</tbody>
</table>

**FAIL URMECHANIS M OF FASTENER**

If the fastener is compared to an ordinary bolt the load-carrying capacity according to the $\Psi$-method, Eligehausen et.al. (1994), can be written as:

$$V_s = \Psi \cdot d^{0.5} \cdot f_u^{0.5} \cdot \left( \frac{h_{nf}}{d} \right)^{0.2} \cdot \varepsilon^{1.5}$$

(8)
where $\psi' = \frac{c_2}{1.5 \epsilon_1} = \frac{75 \cdot 100}{1.5 \cdot 320} = 0.156 \cdot 0.208, \text{ see Figure 23.}$

Dimension of the fastener: $d =$ bolt diameter, varies between 12 and 60 mm see Figure 24, $f_c =$ concrete cube strength, 100 MPa, and $h_{ef} =$ effective depth, 110 mm.

For $c_2 = 75$ to 100 mm and $d = 16$ to 60 mm, the ultimate load $V_u$ varies between 52 and 104 kN which can be compared to the test results of 103-133 kN for the sleepers in classes 1 and 2 (green and yellow).

When the failure mechanism is compared for the three classes, the red sleeper shows a completely different failure process than the green and yellow ones. The failure process for the red sleeper is calm and steady, i.e. the fastener was slowly pulled out with no large concrete parts falling off. On the other hand, the failure process for the yellow and green sleepers was explosive. The failure started with a crack growing from the fastener and down towards the base where it was divided into two horizontal cracks, one going towards the end and the other towards the mid section. When enough energy was built up large sections of the concrete fell off.
Possible failure mechanisms are illustrated in Figures 23–27.

If a simplified stress distribution according to Figure 26 is assumed, where the tensile stress decreases linearly along the length, \( l_b \), an equilibrium equation around \( A \):

\[
\left\{ \begin{array}{l}
\sigma \cdot b = F \cdot e - \left( \frac{\sigma \cdot l_b \cdot 2l}{3} \right) = 0 \\
\tau = \frac{F}{b \cdot l} = \frac{103800 \cdot 130}{200 \cdot 320} = 1,62 \text{ MPa}
\end{array} \right.
\]

The obtained stresses \( \sigma \) and \( \tau \) are rather small. A more realistic assumption is that the tensile stress, \( \sigma \), is in the beginning distributed only along the length \( l_b/3 \), see Figure 27.

\[
\left\{ \begin{array}{l}
\sigma = \frac{27 \cdot F \cdot e}{4 \cdot b \cdot l} = \frac{27 \cdot 103800 \cdot 130}{4 \cdot 200 \cdot 320} = 4,45 \text{ MPa}
\end{array} \right.
\]
The obtained stress $\sigma_t = 4.45$ MPa is higher than what can be expected from a sleeper concrete with $f_{ct}$ about 4 MPa. Consequently a crack is likely to form and to propagate. The propagation of the crack may be studied if the softening properties of the concrete are taken into consideration, see Figure 28, see also Elfgrén (1989, 1998), Ohlsson (1995) and Noghabai (1998). Assume as an example that the horizontal crack has propagated the length $l_b/3$ and that the tensile stresses are distributed along the length $2l_b/3$ according to Figure 28. If we further assume that the tensile strength along the distance $l_b/2$ in Figure 28 is half the area of a triangle we get rather high tensile stresses.

Assume as an example that the horizontal crack has propagated the length $l_b/3$ and that the tensile stresses are distributed along the length $2l_b/3$ according to Figure 28. If we further assume that the tensile strength along the distance $l_b/2$ in the figure is half the area of a triangle. $F=103.8$ kN (sleeper no. 11).

The tensile stresses obtained from the calculations in Figure 24 to 26 can be compared to the concrete strength that has been obtained in the tests of the material properties, see Table 2. Sleeper No. 11 had the tensile strength of 3.3 MPa, which is lower than the stresses for the cases in Figure 25 and 26.

The calculations confirm to a certain part the observations made during the tests, i.e. when the horizontal crack is propagating towards the end of the sleeper and has reached a part of its full length, the failure process accelerates considerably. This leads to the conclusion that it would take an unreasonably high tensile strength to prevent the process.

**SUMMARY AND CONCLUSIONS**

The bending capacity of the midsection of the tested sleepers is enough to prevent failure with a safety factor of 1.75 against failure even for the sleepers in class 1 (red sleepers).

The bending capacity of the rail section of the tested sleepers is on the other hand not high enough for the sleepers in class 1 (red sleepers). This is probably due to bad anchorage for the strands when the cracking is so severe.

Small cracks, corresponding to class 2 (yellow sleepers), do not seem to influence the horizontal load carrying capacity of the tested fasteners significantly. It is first when the cracking is very severe (red sleepers, where both the longitudinal cracks and the vertical cracks appear) that the load carrying capacity is reduced so much that it is approaching
the level of the applied load. The vertical crack at the fastener and downwards probably comes from track forces, i.e. the presence of this crack depends on where it has been lying in the track (curves etc) where it has been exposed to high forces.

It was also possible to see a variation of the load carrying capacity of the fastener for the yellow sleepers. But the results show that there is nothing to gain by dividing the yellow sleepers into groups since they manage approximately the same loads.

The material properties of the concrete in the tested sleepers were high. The mean value for 16 compression tests was 100.7 MPa and the mean value for 13 tensile tests was 3.9 MPa.

The sleepers produced with inferior methods are now inspected annually in order to see at what rate the cracking is progressing.

The big question is now how fast a green or yellow sleeper turns into a red one, i.e. how fast is the formation of ettringite? Probably weather conditions have an important role. If a sleeper will be exposed to cyclic frost erosion the process ought to accelerate considerably. An indication of that the weather conditions may play an important role has been discovered on one sleeper. The midsection of this sleeper had been covered with equipment for the signalling system a so-called “Balis” and this part seemed to be in better conditions (no visible cracks) in comparison with the ends where the crack system had developed quite a bit.

The next steps in the project will be to test the fatigue capacity of a yellow/red sleeper and to perform some more tests of the bending capacity in the midsection and rail section with yellow/red sleepers. It might also be tests where a rail is mounted on the sleeper and the load is applied on this instead of directly in the fastener and see if this changes the result.

**Acknowledgements**

The investigation was carried out on behalf of the Swedish National Rail Administration. Björn Paulsson and Paul Nilsson together with their staff have provided much information and help. TESTLAB (the test laboratory at Luleå University of Technology) has performed the tests in collaboration with the first two authors.

**Notation**

- $f_c$: concrete compression strength
- $f_t$: concrete tensile strength
- $F_u$: ultimate force at failure
- $M_{f, \text{mid}}$: moment bearing capacity at the midsection of the sleeper including 1.75 safety factor against failure
M_{rail} moment bearing capacity at the rail section of the sleeper including 1.75 safety factor against failure

REFERENCES


Paper C

CONCRETE FATIGUE CAPACITY. A STUDY OF DEFORMATIONS AT TENSILE FORCES
by Håkan Thun, Lennart Elfgren and Ulf Ohlsson
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ABSTRACT
In this paper results and analyses are presented from uniaxial cyclic tensile fatigue tests. The tests can be considered as pilot tests. The results from the tests indicate that the strain criterion proposed by Balázs (1991) might also be applied to plain concrete exposed to cyclic tensile fatigue load. The tests have also given valuable information on how a test like this shall be performed (type of adhesive, the way the data are collected etc.). Further studies are needed.

Keywords: concrete, fatigue, tensile strength, strain criterion, uniaxial tensile test.

RESEARCH SIGNIFICANCE
Since the concrete design codes of today are conservative concerning the concrete shear fatigue capacity, there is a need to develop new methods that describes the phenomenon in a more refined way. This may in turn lead to economic savings when existing structures are evaluated and approved for higher loads instead of being replaced.

INTRODUCTION
When a program was initiated to check the condition of railway concrete trough bridges in Sweden, one of the first actions was to recalculate the bridges with the present design code. The standard bridges have the form of a concrete trough that is filled with ballast and consists of a slab connected to and carried by two longitudinal beams. These old bridges were designed according to principles where the shear capacity of the concrete sometimes was somewhat overestimated and a critical point in the recalculations was the shear fatigue capacity of the slab and the connection between the slab and the longitudinal beams. Since the shear capacity is dependent on the tensile strength studies have been performed on specimens exposed to cyclic tensile load. These tests will hopefully lead to a refined fatigue model for the shear capacity.
FATIGUE OF CONCRETE

A common way to consider fatigue load has up to now been to calculate the load effect of the fatigue load, as it were a static load and then manually calculate the stress levels for a number of critical sections. The actual stress levels will then be converted into service life for the actual section with the help of so called "Wöhler curves", Wöhler (1858-70). The problem is often to transfer the local fatigue strength to the fatigue strength of the whole structure. Another problem is that the criterion of fatigue strength is often connected to the ultimate limit state and with that total collapse. To be able to calculate the service life of concrete bridges, criteria for the serviceability limit state should be developed.

Fatigue in concrete was observed rather late in comparison to the early studies of fatigue in iron and steel cables and railway carriage axles, Albert (1837) and Wöhler (1858-70). For concrete subjected to cyclic compression load several studies have been performed over the years. But regarding uniaxial tensile fatigue tests, the focus in this study, not so many studies have been performed. The most recent one that is similar to the present study seems to be the one performed by Plizzari et al. (2000). This study was focused on the effect that fiber reinforcement has on the low cyclic fatigue strength of concrete. When they performed the uniaxial tensile fatigue tests they began the cyclic loading when the load dropped to 95% of the peak load in the softening branch of the load-displacement curve. They also compared the material response obtained from uniaxial tensile tests with the structural behaviour obtained in notched beams.

Fatigue capacity in concrete is normally described by so-called Wöhler curves. They are named after the German engineer A. Wöhler who conducted studies of the fatigue capacity of railway axles in the late nineteenth century. In Figure 1 results from fatigue tests are presented. The figure shows the number of load cycles for different amplitudes of stresses. The curve is also sometimes called an S-N-curve. If the number of load cycles at failure, N, is presented on a logarithmic axis the curve becomes linear.

![Figure 1 Number of load cycles N for different amplitudes of stresses, $\sigma_A$. $\sigma_B$ is the static failure load. $\sigma_{\text{fat}}$ is the fatigue limit, if the loading does not exceed this limit no fatigue failure occurs, Elfgren & Gylltoft (1997).](image-url)
The fatigue tests in this paper can be compared to a deformation criterion proposed by Daerga & Pöntinen (1993) based on an idea of Balázs (1991). The model has successfully been used to describe bond failure between rebars and the concrete. The growth in deformation during a fatigue test can according to the model be divided into three phases, see Figure 2. In the beginning of the first phase the deformation rate is high but stagnates after a while. The second phase is characterised by a constant deformation rate. These two phases can be described as stable. During the third phase, the failure phase, the deformation rate increases rapidly leading to failure within a short time.

The strain criterion for fatigue failure is that the strain at peak load during a static test corresponds to the strain at the changeover between phases two and three during a fatigue failure, see Figure 2.

When \( \varepsilon(\sigma_u) \) has been reached only a limited number of cycles are needed until failure occurs. Since there is a difference between the number of cycles at failure and at initiation of phase three one can consider the criterion as safe, Balázs (1991).

**EXPERIMENTAL PROGRAM**

**Test specimens and Used Concrete**

The tensile fatigue tests have been performed on two types of specimens with different shapes. One type was casted in a special steel mould equipped with a half-circle notch with the radius 15 mm. The other type of specimen was drilled out from a small concrete beam. It was later cut into a suitable length and a notch was milled. For dimensions see Figure 3 and Figure 4.

One reason why two different types of specimens were tested was an uncertainty of crack initiation when it was subjected to the cyclic load. According to Noghabai (1998) the two types of notches behave differently to one another. When an uniaxial tensile test is performed a sharp notch results in that the crack follows the intended path (if the
notch is deep enough that is). When the half-circle-like notch is used the crack is free to choose its path and might detour from the intended crack zone. Another reason is that it takes less time to cast a beam and drill out cores compared to using the special steel mould.

Figure 3 Dimensions of the two different types of specimens used in the tensile fatigue test, Andersson (2000).

Figure 4 Photo showing the two types of shapes that were used in the tensile fatigue tests, Andersson (2000).

The concrete used in the tests was a normal strength concrete (NSC). The mixture that was used was a K45, which means that the compression strength should be 45 MPa tested on 150 mm cubes after 28 days (according to the Swedish Concrete Code, BBK94 (1994)). After casting the specimens were stored in water for twenty-eight days. Before they were tested they were stored in the laboratory at room temperature.
The concrete mixtures used for the test specimens (K45):

<table>
<thead>
<tr>
<th></th>
<th>Series A</th>
<th>Series B and S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cem I 42.5 BV/SR/LA</td>
<td>450 kg/m³</td>
<td>450 kg/m³</td>
</tr>
<tr>
<td>Fine aggregate 0-8 mm</td>
<td>825.1 kg/m³</td>
<td>837.1 kg/m³</td>
</tr>
<tr>
<td>Coarse aggregate 8-16 mm</td>
<td>893.8 kg/m³</td>
<td>906.9 kg/m³</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plasticiser</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water-to-cement ratio, vct</td>
<td>0.40</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Slump: 90 mm (series A), 80 mm (series B), 75 mm (series S).
Air content: 2.5 % (series A), 2.8 % (series B), 3.1 % (series C).

Series S, consists of 12 specimens. Series A and Series B have been casted on two different occasions. Each time 16 specimens were casted.

**Uniaxial tensile tests**

In order to decide the maximum load capacity of the concrete (so that the load levels in the fatigue test could be set) a number of uniaxial tensile tests were performed.

Today there is no international standard on how this test should be performed. The influence that the shape, the dimension etc. have on a uniaxial tensile test has been studied by several researchers, e.g. Noghabai (1998), Daerga (1992), Hordijk (1991) etc.

If the uniaxial tensile test is performed under displacement control instead of load control it is also possible to obtain the tension-softening branch of the material. At LTU several researchers have performed the uniaxial tensile test using a so-called closed-loop sevo-hydraulic test machine, Noghabai (1998). This has led to a “standard“ within the division.

Before the test every test specimen were grinded and cleaned with acetone before they were attached to the two steel plates with an adhesive. The adhesive that was used is a two-component adhesive manufactured by Hottinger Baldwin Messtechnik (HBM) called Schnellklebstoff X-60. The adhesive was first put on the lower steel plate (see Figure 5) under a small compressive load. When it has hardened the test specimen and the lower steel plate are mounted in the test machine. Then adhesive is put on the upper side of the test specimen. Finally a compressive load is put on for the hardening process. Dartec manufactures the servo-hydraulic test machine that is used. The measuring device for collecting all the data has been a Spider8, manufactured by HBM.

A steel ring with holders for the COD-gauges were mounted on the specimen at each side of the notch with a distance of 45 mm in between. When the steel ring is in place a total of four Crack Opening Displacement gauges (COD-gauges) can be mounted with 90 degrees between each of them. The feed-back signal to the machine is the mean value of all four COD-gauges. Figure 5 shows a photo of the test set-up.
Figure 5 Test set-up uniaxial tensile test and fatigue tensile test.

One peculiarity with the uniaxial tensile test is that you get a very large scatter of the test results. You might get a variation of as much as 10% on tests performed on concrete from the same batch, Hedlund (2000).

**Fatigue Tests**

All fatigue tests have been performed under load control with a sinusoidal load cycle. The lower load level (A) was held constant and the higher level (B) was varied. Since a fatigue test can last a very long time depending on what load levels that are used, the decision was made to increase the higher level if the strain still were small after 2000 load cycles. During the stop when the level was adjusted the load was kept at a mean value of the two earlier load levels by the machine. The load frequency has during all tests been 2.004 Hz.

In order to get a “soft” start of the test the load was increased with a rate of 0.5 kN/s until it reached a level that was higher than level A, see Figure 6. This was done in order to make sure that the machine did not break the test immediately.
In order to get a “soft” start of the test the load was increased with a rate of 0.5 kN/s until it reached a level that was higher than level A.

The load levels that have been used in the beginning of the tests correspond to 90% (level B) and 60% (level A) of the average peak load obtained from the uniaxial tensile tests.

Later on in the tests of series A the following was used for level A: 60%, 48% and 79%. For level B the tests started at 90. If no failure occurred during 2000 load cycles the level was increased to 94%, 98%, 102% and 108%.

For series B level A was set to 58% and 66%. Level B was set to 87%, 99%, 103% and 106%. For series S two different levels were used for level A: 54% and 80%. For level B the following was used: 90, 94, 96, 99, 102, 104, 107, 109 and 112%.

The two last tests that were made with a specimen from series A and S differ from the others: The higher level B was kept at the same value instead of increasing it. The tests lasted for 9 respectively 5 days. As failure did not occur during this time for none of them, uniaxial tensile test were performed.

The data from the COD-gauges have been sampled during the fatigue tests with a frequency of 50 Hz, which in some cases has led to very large data files. This has in turn led to some problems when the data were going to be evaluated since only few computer programs can handle these large amounts of data. In this study MATLAB™ (The MathWorks Inc.) has been used.

RESULTS

Uniaxial tensile tests

In Table 1 a summary is presented of the results from the uniaxial tensile tests. Note that for series S and specimen S3 the fracture occurred in the adhesive layer. The “mean value” is therefore only based on two values.
Table 1: A summary of all uniaxial tensile tests. Explanations: \( m \) = mean value, \( s \) = standard deviation and \( V \) = coefficient of variation.

<table>
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<tbody>
<tr>
<td>A1</td>
<td>28.23</td>
<td>4.44</td>
<td>B1</td>
<td>22.44</td>
<td>3.53</td>
<td>S1</td>
<td>18.67</td>
<td>3.71</td>
</tr>
<tr>
<td>A2</td>
<td>24.88</td>
<td>3.91</td>
<td>B2</td>
<td>27.8</td>
<td>4.37</td>
<td>S2</td>
<td>19.69</td>
<td>3.92</td>
</tr>
<tr>
<td>A3</td>
<td>23.31</td>
<td>3.66</td>
<td>B3</td>
<td>25.9</td>
<td>4.07</td>
<td>S3</td>
<td>17.11(^a)</td>
<td>3.40(^a)</td>
</tr>
<tr>
<td>A4</td>
<td>25.58</td>
<td>4.02</td>
<td>B4</td>
<td>25.28</td>
<td>3.97</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) Specimen failed in adhesive layer.

In Figure 7 the results are presented from the tests in series A. The curves show the applied stress and the strain for the COD-gauge from each test that was exposed to the highest strain.

Figure 7: Results of the uniaxial tensile tests for series A. (a) Curves showing the strains of the COD-gauges that were exposed to the highest strain during each test. (b) The same curves as in (a) but with normalized stress in order to get a mean curve for the whole test series. The curve that has (+) is the mean curve used in Figure 11 and Figure 12.

In Figure 8 the results are presented from the tests in series B. The curves show the stress and the strain for the COD-gauge from each test that was exposed to the highest strain.
Figure 8 Results of the uniaxial tensile tests for series B. (a) Curves showing the strains of the COD-gauges that were exposed to the highest strain during each test. (b) The same curves as in (a) but with normalized stress in order to get a mean curve for the whole test series. The curve that has (+) is the mean curve used in Figure 10.

Fatigue test

In Figure 9 to Figure 13 some typical results from the fatigue tests are presented.

In Figure 9 the tensile stress versus the strain is displayed for specimen B9. The specimen failed after 350 load cycles. In Figure 9 the plotted curve is the COD-gauge that was exposed to the highest strain.

Figure 9 Curve showing the tensile strength versus the strain development from specimen B9. The specimen failed after 350 load cycles. The two load levels corresponds to 94% respectively 64% of the average peak load for the uniaxial tensile tests for the series.

In Figure 10 the total strain versus the number of load cycles is displayed for specimen B9. In the figure this curve is compared to the average normalized stress-strain curve for series B.
Figure 10 Result from the cyclic tensile fatigue test of specimen B9. The strain development for the COD-gauge that has been exposed to the highest strain is shown in (a). This curve is compared with the normalized average peak load obtained in the uniaxial tensile tests (b).

In Figure 11 the total strain versus the number of load cycles is displayed for specimen A7. In the figure this curve is compared to the average normalized stress-strain curve for series A.

Figure 11 Result from the cyclic tensile fatigue test of specimen A7. The strain development for the COD-gauge that has been exposed to the highest strain is shown in (a). This curve is compared with the normalized average peak load obtained in the uniaxial tensile tests (b).

In Figure 12 the total strain versus the number of load cycles is displayed for specimen A5. In the figure this curve is compared to the average normalized stress-strain curve for series A.
Figure 12 Result from the cyclic tensile fatigue test of specimen A5. The strain development for the COD-gauge that has been exposed to the highest strain is shown in (a). This curve is compared with the normalized average peak load obtained in the uniaxial tensile tests (b).

In Figure 13 (a) the total strain versus the number of load cycles is displayed for specimen S2. The figure also shows the tensile strength versus the strain for the test (b).

Figure 13 Results from cyclic fatigue tests of specimen S2. (a) The strain development is displayed versus the number of load cycles. (b) Tensile strength displayed versus the total strain. For the test the upper load level B corresponds to 90, 94, 96, 99 and 102% of the average peak load obtained in the uniaxial tensile tests (note that the “average” peak load for series S is based only on two tests).

DISCUSSION

Strain Criterion
The conformity between the maximum total strain obtained in the uniaxial tensile test and the total strain when phase 3 begin (according to Figure 2) is reasonably good for specimen B9 and A5, see Figure 10 and Figure 12. However, for specimen A7 (Figure
11) the start of phase 3 takes place for a higher strain than the normalized stress-strain curve indicates (this is on the safe side). The variation depends most likely on that too few uniaxial tensile tests have been performed. If more uniaxial tensile tests are done a more exact average of the peak load will be obtained for the comparison.

**Fatigue tests**

An interesting observation from the tests in Figure 13 is that it is the initial and the final load level that seems to give the largest contributions to the total strain development. The load cycles in between give rise to only about 20% to the total strain. If this phenomenon is compared with the three phases described in Figure 2 it indicates that the increase of $S_{\text{max}}$, i.e. $\sigma_{\text{max}} / f_{\text{ct}}$ (strength levels 3.33, 3.42 and 3.51 MPa) belongs to phase 2, the stable part of the fatigue process with a nearly constant strain rate. The reason for that so many levels have been needed in order to obtain a fatigue failure for this specimen is that the value of the average peak load for series S is not accurate. The other tests show almost the same behaviour as specimen S6. A thing that needs further studies is the influence the increase in load during a test has on the final result.

For the two tests where no increase of $S_{\text{max}}$ was performed the tests were terminated after 9 respectively 5 days. They had run for 1.37 and 0.85 million load cycles and had reached the total strain of 0.36‰ respectively 0.24‰. Uniaxial tensile tests were then performed on the two specimens and the residual strength was for specimen A9 4.45 MPa but for specimen S7 the failure occurred near the adhesive. This leads to the fact that for specimen A9 the higher level $S_{\text{max}}$ was approximately 81% of the average peak-load and not about 86%. It was probably even lower than 81% if you assume that the 1.37 million load cycles had led to a reduction of the original tensile strength.

One of the most interesting parts of the fatigue curve in e.g. Figure 6 is the strain rate (equal to the angle of the slope) for phase 2 since it indicates how close you are to failure. A high strain rate (i.e. steep slope) means that not so many cycles are left until fatigue failure occurs and a low one (i.e. flat curve) that fatigue failure will not occur in a near future (at the same conditions). It is not easy to perform an analysis of the factors that influences the strain rate during phase 2 (i.e. the slope) from the results obtained in this study. This is due to the reason that there are too few tests with enough load cycles (i.e. before the failure) in order to make it possible to calculate the strain rate ($C = \frac{d\varepsilon}{dN}$). A hypothesis would be that $C$ is decreasing with decreasing $S_{\text{max}}$ and increasing $R$-values (decreasing amplitudes).

If the results in appendix A are studied for series B the result is somewhat different to what you might expect, see Figure 19 to Figure 22. Specimen B2 has a higher strain rate than the others in series B (together with specimen B9) even though $S_{\text{min}}$ and $S_{\text{max}}$ are somewhat lower compared to specimens B8-B10. Note also that for specimen B2 the fatigue test only lasted for about 130 load cycles (specimen B9 for about 350 load cycles). The main reason to these ambiguous results is most likely the variation of the tensile strength for each series. For specimen B2 and B9 $S_{\text{max}}$ was most likely very close to the maximum load, which was not the case for the others. This is confirmed in some sense of the result in Figure 22 in appendix A, where it can be seen that specimen B2 and B9 obtained failure for a lower number of load cycles than the other two.
specimens. For series A no indications have been obtained of factors influencing the strain rate.

Another conclusion from Figure 15 to Figure 28 is that you need to vary the factors more. In these test $S_{\text{min}}$, $S_{\text{max}}$ and $R$ have almost been the same.

The main reason why the increase of load $S_{\text{min}}$ was performed during the test was to get a quick result (besides of simulating e.g. an increase in axle load for an existing railway bridge).

**General**

Since these tests are pilot tests there were a few things concerning the testing procedure that were uncertain before the tests:

- Will the adhesive between the specimen and the steel plates manage repeated loading?
- Will the COD-gauges show any signs of fatigue?
- Will it be possible to collect data from tests that last a longer time, e.g. 5 days, with kept accuracy?

For these questions, there were only positive answers. The adhesive between the steel plates and the specimen showed no signs of “fatigue failure” nor did the COD-gauges. In only one test, the uniaxial tensile test for specimen S3, the adhesive failed and this was due to human errors in the mixing procedure. Regarding the collection of the data there will be improvements made before the next series of tests. New equipment that has been bought will make it possible to handle the data from the more time consuming tests better. For the tests that lasted 5 and 9 days, the data files became so large that the database section of the measuring program failed.

When the analysis of the data begun, it was easy to see at what gauges the failure was going to happen, so a better set-up of the measuring program during the tests will make it easier to see the development of the strain for the different COD-gauges.

**CONCLUSIONS**

These tests indicate that the strain criterion proposed by Balázs (1991) may also be applied to plain concrete exposed to cyclic tensile fatigue load. Though, further studies are needed.

If these further studies also show that the criterion can be used as a safe indicator of a coming fatigue failure, it may lead to a proposal for how the fatigue capacity can be calculated for tensile fatigue load. A challenge is to find a way to use the results in design and assessment of reinforced structures.
ACKNOWLEDGEMENTS

The study has been financially supported by the Swedish National Rail Administration. MSc. Jörgen Andersson has performed all the experimental work during his studies at LTU together with the staff at TESTLAB (the test laboratory at Luleå University of Technology).

NOTATION AND ABBREVIATIONS

LVDT Linear Voltage Differential Transducers
COD Crack Opening Displacement
Level A Lower limit of the load during the cyclic uniaxial fatigue test
Level B Upper limit of the load during the cyclic uniaxial fatigue test

$m$  mean value. Calculated as, \[ m = \frac{\sum f_i}{n} \]

$s$  standard deviation. Calculated as, \[ s = \sqrt{\frac{\sum (f_i - m)^2}{n - 1}} \]

$V$  coefficient of variation. Calculated as, \[ V = \frac{s}{m} \]

$N$  number of load cycles until failure

$S_{\text{min}}$  normalized lower stress level, \[ S_{\text{min}} = \frac{\sigma_{\text{min}}}{\sigma_{\text{ct}}} \]

$S_{\text{max}}$  normalized higher stress level, \[ S_{\text{max}} = \frac{\sigma_{\text{max}}}{\sigma_{\text{ct}}} \]

$R$  normalized stress amplitude, \[ R = \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}} \]

$f_{\text{avg}}$  average value of the peak load during the uniaxial tensile test for each series

$\varepsilon(\sigma_u)$  strain at peak load during a static test, Balázs (1991)

$\Delta \varepsilon$  increase in strain during phase 2 in Figure 2

$dN$  number of load cycles during phase 2 in Figure 2 (see also Figure 14).

$N_f$  number of load cycles during failure phase, i.e. phase 3.

$C$  strain rate, \[ C = \frac{\Delta \varepsilon}{dN} \]

$\sigma_{\text{max}}$  maximum value of level B during the cyclic test

$\sigma_{\text{min}}$  minimum value of level A during the cyclic test

$\varepsilon_{\text{max}}$  strain at the changeover from phase 2 to phase 3 in Figure 2 (inflection point)
REFERENCES


APPENDIX A

Evaluated data from the fatigue tests and used in Figure 15 to Figure 28 are presented in Table 2 and Table 3.

Table 2 Results from cyclic tensile fatigue tests for series A. The specimens that are presented in the table are for the ones there a deflection rate has been possible to calculate in the failure phase.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Part</th>
<th>(\varepsilon_{\text{max}})</th>
<th>(d\varepsilon/dN)</th>
<th>(\sigma_{\text{min}})</th>
<th>(\sigma_{\text{max}})</th>
<th>(\sigma_{\text{fct}})</th>
<th>(S_{\text{min}}/f_{\text{ct}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>utm_a4</td>
<td>3</td>
<td>5.42E-04</td>
<td>7.36E-05</td>
<td>1415</td>
<td>3.14</td>
<td>4.09</td>
<td>4.01</td>
</tr>
<tr>
<td>utm_a5</td>
<td>2</td>
<td>3.21E-04</td>
<td>4.28E-05</td>
<td>590</td>
<td>1.57</td>
<td>3.93</td>
<td>4.01</td>
</tr>
<tr>
<td>utm_a6</td>
<td>1</td>
<td>4.71E-04</td>
<td>5.30E-05</td>
<td>23</td>
<td>1.57</td>
<td>3.77</td>
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<tr>
<td>utm_a7</td>
<td>1</td>
<td>4.27E-04</td>
<td>8.31E-05</td>
<td>1029</td>
<td>1.57</td>
<td>3.77</td>
<td>4.01</td>
</tr>
</tbody>
</table>

Table 3 Results from cyclic tensile fatigue tests for series B. The specimens that are presented in the table are for the ones there a deflection rate has been possible to calculate in the failure phase.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(\varepsilon_{\text{max}})</th>
<th>(d\varepsilon/dN)</th>
<th>(\sigma_{\text{min}})</th>
<th>(\sigma_{\text{max}})</th>
<th>(\sigma_{\text{fct}})</th>
<th>(S_{\text{min}}/f_{\text{ct}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>utm_b2</td>
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<td>2.36E-04</td>
<td>4.37E-05</td>
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<td>3.49</td>
</tr>
<tr>
<td>utm_b8</td>
<td>2</td>
<td>3.23E-04</td>
<td>1.94E-05</td>
<td>700</td>
<td>2.63</td>
<td>3.93</td>
</tr>
<tr>
<td>utm_b9</td>
<td>1</td>
<td>3.78E-04</td>
<td>1.46E-04</td>
<td>267</td>
<td>2.63</td>
<td>3.93</td>
</tr>
<tr>
<td>utm_b10</td>
<td>2</td>
<td>4.58E-04</td>
<td>9.94E-05</td>
<td>1120</td>
<td>2.63</td>
<td>4.09</td>
</tr>
</tbody>
</table>

Table 4 Results from cyclic tensile fatigue tests for series C. The specimens that are presented in the table are for the ones there a deflection rate has been possible to calculate in the failure phase.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(\varepsilon_{\text{max}})</th>
<th>(d\varepsilon/dN)</th>
<th>(\sigma_{\text{min}})</th>
<th>(\sigma_{\text{max}})</th>
<th>(\sigma_{\text{fct}})</th>
<th>(S_{\text{min}}/f_{\text{ct}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>utm_c2</td>
<td>1</td>
<td>2.05E-04</td>
<td>3.17E-05</td>
<td>70</td>
<td>2.13</td>
<td>3.49</td>
</tr>
<tr>
<td>utm_c8</td>
<td>2</td>
<td>3.23E-04</td>
<td>1.94E-05</td>
<td>700</td>
<td>2.63</td>
<td>3.93</td>
</tr>
<tr>
<td>utm_c9</td>
<td>1</td>
<td>3.78E-04</td>
<td>1.46E-04</td>
<td>267</td>
<td>2.63</td>
<td>3.93</td>
</tr>
<tr>
<td>utm_c10</td>
<td>2</td>
<td>4.58E-04</td>
<td>9.94E-05</td>
<td>1120</td>
<td>2.63</td>
<td>4.09</td>
</tr>
</tbody>
</table>
Figure 14 Explanation of notations in Table 2 and Table 3.

Test series A:

Figure 15 Strain rate $C$ versus the normalized lower stress level $S_{\text{min}}$ for series A.

Figure 16 Strain rate $C$ versus the normalized higher stress level $S_{\text{max}}$ for series A.
Figure 17 Strain rate $C$ versus the normalized stress amplitude $R$ for series A.

Figure 18 Strain rate $C$ versus the number of load cycles during phase 2, $dN$, for series A.

Test series B:

Figure 19 Strain rate $C$ versus the normalized lower stress level $S_{min}$ for series B.
Figure 20 Strain rate $C$ versus the normalized higher stress level $S_{\text{max}}$ for series B.

Figure 21 Strain rate $C$ versus the normalized stress amplitude $R$ for series B.

Figure 22 Strain rate $C$ versus the number of load cycles during phase 2, $dN$, for series B.
Test series A:

Figure 23 Normalized lower stress level $S_{\text{min}}$ versus total number of load cycles $N$ for series A.

Figure 24 Normalized higher stress level $S_{\text{max}}$ versus total number of load cycles $N$ for series A.

Figure 25 Normalized stress amplitude $R$ versus total number of load cycles $N$ for series A.
Test series B:

Figure 26 Normalized lower stress level \( S_{\text{min}} \) versus total number of load cycles \( N \) for series B.

Figure 27 Normalized higher stress level \( S_{\text{max}} \) versus total number of load cycles \( N \) for series B.

Figure 28 Normalized stress amplitude \( R \) versus total number of load cycles \( N \) for series B.
Paper D

SHEAR FATIGUE CAPACITY – A COMPARISON BETWEEN THE SWEDISH CODE AND EUROCODE

by Håkan Thun, Lennart Elfgren and Ulf Ohlsson
SHEAR FATIGUE CAPACITY – A COMPARISON BETWEEN THE SWEDISH CODE AND EUROCODE

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Abstract

In this paper a comparison is made between the Swedish Codes and Eurocode regarding fatigue load for concrete. The comparison is also presented in Thun et al. (2000).

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1 Introduction

The codes that have been used and are referred to in this paper:

**Swedish Codes**: BBK94 (1995) and BV BRO94 (1996).


**EC2-2 (1995)**: Eurocode 2: Design of Concrete Structures, Part 2: Concrete Bridges.


Since a comparison of this kind is not “easy” due to the differences in strength classes, partial safety factors, load combinations etc., this comparison is made in such a way that the load calculation is made according the Swedish Codes. However, the strength classes are taken from the EC2-draft (1999).

The bridge that has been used in this example is the Lautajokki Bridge (which was exposed to fatigue tests at LTU in 1996). The calculation is made on 1 m of the slab. The load case that has been used for the calculation is shown in Figure 1.1, Figure 1.2 and Figure 1.3.

![Figure 1.1 Load case with two bogies from two iron ore wagons, Nilsson et al. (1999).](image)

![Figure 1.2 Cross-section of the Lautajokki Bridge, Nilsson et al. (1999).](image)
2 Loads

Load history

To give an idea of how many load cycles the bridge already had been subjected to during its service life, the following estimation has been made:

Variable load: According to Banverket (Swedish National Rail Administration) this particular railway line between Boden-Gällivare has been subjected to the following traffic:

<table>
<thead>
<tr>
<th>Year</th>
<th>Gross loads [Mtons]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1968</td>
<td>11.39</td>
</tr>
<tr>
<td>1969</td>
<td>13.24</td>
</tr>
<tr>
<td>1970</td>
<td>14.508</td>
</tr>
<tr>
<td>1971</td>
<td>13.214</td>
</tr>
<tr>
<td>1972</td>
<td>12.647</td>
</tr>
<tr>
<td>1973</td>
<td>14.486</td>
</tr>
<tr>
<td>1974</td>
<td>19.225</td>
</tr>
<tr>
<td>1975</td>
<td>15.947</td>
</tr>
<tr>
<td>1976</td>
<td>13.379</td>
</tr>
</tbody>
</table>

Sum: 128.036

This corresponds to 128.036 / 9 = 14.2 Mtons/year. About 81 % of this load is iron ore trains with the axle load 22.5 tons while the rest is trains with lower axle loads. With a load cycle of four axles (two axles on one wagon + two axles on the next wagon, see Figure 1.1) we will have:

\[
\frac{0.81 \cdot 14.2}{4 \cdot 22.5} = 0.13 \text{ M cycles/year with the axle load 22.5 tons.}
\]

During the 21 years (1968-1988) the bridge was in traffic it has been exposed to approximately, \(21 \cdot 0.13 = 2.7\) M cycles.

Loads

The loads are calculated according to the Swedish Codes. In the Swedish Code the self-weight of reinforced concrete is 24 kN/m³.
Dead Loads:

Slab: \(0.320 \, \text{m} \times 24 \, \text{kN/m}^2 = 7.68 \, \text{kN/m}^2\)

Ballast: \(0.600 \, \text{m} \times 20 \, \text{kN/m}^2 = 12.00 \, \text{kN/m}^2\)

Traffic loads:

If the load from one bogie pair is spread over \(1.70 + \frac{1.5}{2} + 0.75 = 3.20 \, \text{m}\) (in the longitudinal direction of the bridge, see Figure 1.1), we will have:

\[
\begin{align*}
P &= 22.5 \, \text{tons} \quad \Rightarrow q &= \frac{2 \times 22.5}{3.20 \times 3.10} = 45.36 \, \text{kN/m}^2 \\
P &= 25 \, \text{tons} \quad \Rightarrow q &= \frac{2 \times 25}{3.20 \times 3.10} = 50.40 \, \text{kN/m}^2 \\
P &= 30 \, \text{tons} \quad \Rightarrow q &= \frac{2 \times 30}{3.20 \times 3.10} = 60.48 \, \text{kN/m}^2
\end{align*}
\]

Load combination

Fatigue load according to Load Combination C in section 22:23

“Bärgihetsbestämmelser”, Banverket (1995) [Design handbook for Railway Bridges]:

The partial factors, which must be used, are:

- Dead loads = 1.0 (\(\psi_l\))
- Ballast = 1.2 (\(\psi_l\))
- Traffic load: For iron ore load, 1.2 (a dynamic factor)

Summation of loads:

Dead Loads

Slab: \(7.68 \times 1.0 = 7.68 \, \text{kN/m}^2\)

Ballast: \(12 \times 1.2 = 14.40 \, \text{kN/m}^2\)

Total = 22.08 kN/m²

Traffic loads

\[
\begin{align*}
P &= 22.5 \, \text{tons} \quad \Rightarrow q &= 45.36 \times 1.2 = 54.43 \, \text{kN/m}^2 \\
P &= 25 \, \text{tons} \quad \Rightarrow q &= 50.40 \times 1.2 = 60.48 \, \text{kN/m}^2 \\
P &= 30 \, \text{tons} \quad \Rightarrow q &= 60.48 \times 1.2 = 72.58 \, \text{kN/m}^2
\end{align*}
\]

The critical section for the bridge, regarding the load \(q\) above, is the connection between the slab and the longitudinal beams.

The traffic load, \(V_{\text{dim}}\), on one half of the bridge, i.e. \(B / 2 = 3.1 / 2 = 1.55 \, \text{m}\), results in:
P = 22.5 tons ⇒ V_{dim} = 54.43 \cdot 1.55 = 83.36 \text{kN/m}

P = 25 tons ⇒ V_{dim} = 60.48 \cdot 1.55 = 93.74 \text{kN/m}

P = 30 tons ⇒ V_{dim} = 72.58 \cdot 1.55 = 112.5 \text{kN/m}

To get the maximum shear load, V_2, the dead load must be added (which is equal to the minimum shear load, V_1):

V_1 = 22.08 \cdot 1.55 = 34.22 \text{kN/m}

P = 22.5 tons ⇒ V_2 = V_1 + V_{dim} = 83.36 + 34.22 = 117.6 \text{kN/m}

P = 25 tons ⇒ V_2 = V_1 + V_{dim} = 93.74 + 34.22 = 128.0 \text{kN/m}

P = 30 tons ⇒ V_2 = V_1 + V_{dim} = 112.5 + 34.22 = 146.7 \text{kN/m}

### 3 Codes for concrete fatigue

Fatigue in concrete was observed rather late in comparison to the early studies of fatigue in iron and steel cables and railway carriage axles, Albert (1837) and Wöhler (1858-70).

Concrete codes long treated fatigue of concrete in a simplified way. UIC 774-1 (1984), for example, states in chapter 8.1 that fatigue in compression is OK as long as the maximum stress is smaller than 0.5 f_{cc} \leq 18 \text{MPa}. For slabs without shear reinforcement no concern is required as long as the shear stresses comply with the common static rules. In section 6.4.2 the shear stress is given as \tau_{rd1} = \tau_{rd0} (1 + 50 A_s/b_d)(1 + M_s/M_{du}) with \tau_{rd0} = 0.30 \text{MPa} for f_{ik} = 25 \text{MPa} and \tau_{rd0} = 0.50 \text{MPa} for f_{ik} = 50 \text{MPa}.

The Swedish Code BBK 94 (1994) is based on a Wöhler curve proposed by Aas-Jakobsen (1970), see Figure 3.1:

\[
\frac{\sigma_{c,max}}{f_c} = S_{c,max} = 1 - \frac{1}{C} \left( 1 - \frac{\sigma_{c,min}}{\sigma_{c,max}} \right) \log N
\]

which with R = S_{c,min} / S_{c,max}, can also be written as

\[
\log N = C \frac{1-S_{c,max}}{1-R}
\]

In the Eurocodes EC2-1 (1991), EC2-2 (1995) and EC2-draft (1999) the term 1-R in the denominator is increased to \sqrt{1-R}. This gives somewhat smaller N-values for R > 0. A thing worth pointing out regarding today’s Eurocode and the new draft is that the fatigue expressions are almost the same. But if a closer comparison is made you can find some significant differences. In the newest draft of Eurocode, EC2-draft (1999),...
the equations to calculate the design shear resistance and the design fatigue strength are changed. These changes give that the new draft is much more conservative than the EC2-1 (1991) (see later sections).

In the MC90 (1993), also presented in a Fib textbook (1999), a more refined method is proposed based on work by Stemland et al (1990) and others:

\[
\begin{align*}
\log N_1 &= \left( 12 + 16S_{c,\text{min}} + 8S_{c,\text{min}}^2 \right) \left( 1 - S_{c,\text{min}} \right) \quad \text{for } N_1 \leq 6 \\
\log N_2 &= 0.2 \log N_1 \cdot \left( \log N_1 - 1 \right) \quad \text{for } N_1 > 6 \text{ and } \Delta S_i \geq 0.3 - 3S_{c,\text{min}} / 8 \\
\log N_3 &= \frac{\log N_2 \cdot \left( 0.3 - 3S_{c,\text{min}} / 8 \right)}{\Delta S_i} \quad \text{for } N_i > 6 \text{ and } \Delta S_i < 0.3 - 3S_{c,\text{min}} / 8 \\
\text{with} \quad \Delta S_i = S_{c,\text{max}} - S_{c,\text{min}}
\end{align*}
\]

The equations are illustrated in Figure 3.1 for the two cases \(R = S_{c,\text{min}}/S_{c,\text{max}} = 0\) and = 0.5.

Some observations can be made:

- EC2 is the most conservative code (the new draft, EC2-draft (1999), is even more conservative).
- Aas-Jacobsen (and the Swedish code) is somewhat less conservative for \(S_{c,\text{min}} > 0\).
- MC90 gives lower \(N\)-values for \(N < 6\) and low \(S_{c,\text{min}}\)-values. But it gives higher \(N\)-values for \(N > 6\) in comparison to EC2 and Aas-Jakobsen.
- UIC gives always the same \(S_{c,\text{max}} = 0.5\). For \(R=0\) and \(N < 6\) this is conservative but for \(N > 6\) it can be un-conservative. For higher \(R\)-values the change from conservative to un-conservative will take place for higher \(N\)-values, see Figure 3.1.
Figure 3.1 Comparison between concrete fatigue capacities for different codes. (The Swedish code BBK94 is based on the Wöhler curves proposed by Aas-Jacobsen). (a) \( R = \frac{S_{c,\text{min}}}{S_{c,\text{max}}} = 0 \). (b) \( R = 0.5 \).

4 Fatigue capacity of concrete according to the Swedish Code BBK94 (1994,1996)

4.1 Partial safety factors for strength

Since \( f_s \) – the formal shear strength – (\( f_{s,\text{sid}} \) in E2-1 (1991)) is depending on \( f_{\text{sid}} \) there is a need to establish the strength values for concrete according to BBK94.

BBK94, section 2.3.1:

\[
f_d = \frac{f_s}{\eta \gamma_u \cdot \gamma_u} \quad (a)
\]
where $\eta \gamma_n$ is 1.5 for concrete (could be compared with $\gamma$ in Eurocode)

$\gamma_n$ is depending on safety class (1, 2 or 3) defined with regard to the consequences of a failure: 1- low risk with regard to the safety of people, 2 – Moderate risk, 3 – High risk.

$f_k$ is the characteristic value of the strength.

Compression: This characteristic value is based on the 5%-fractile of the cylinder strength (cylinders: $\phi 150$ and height 300mm) and reduced to 85% to consider long-term effects.

In our example the safety class defined with regard to the consequences of a failure with regard to the safety of people must be 3, i.e. high risk.

Which gives: $f_s = \frac{f_k}{1.5 \cdot 1.2}$

Since the strength classes in EC2-draft (1999) are used for the strength in this example the characteristic value, $f_k$, for compression must be multiplied with the factor 0.85.

Note: It is not possible to directly “translate” the Swedish strength classes into the strength classes used in EC2-draft (1999). The Swedish strength classes are based on the cube strength (but the characteristic values are based on the cylinder strength), for example: The Swedish strength class K40, corresponds to (the nearest one) strength class C30/37 in EC2-draft (1999).

### 4.2 Shear force capacity for concrete, $V_c$

According to BBK94, section 3.7.3.2: “Shear force capacity for concrete, $V_c$,” the shear force capacity is calculated in the following way:

For a structure with constant section, which is not influenced by tension, the shear force capacity, $V_c$, can be defined as

$$V_c = b_w \cdot d \cdot f_v$$  \hspace{1cm} (3.7.3.2a)

where $b_w$ is the smallest width of the beam width within the effective depth in the actual section

$d$ is the effective depth in the actual section

$f_v$ is the formal shear strength, defined by equation 3.7.3.2b

$$f_v = \xi \left( 1 + 50 \cdot \rho \right) \cdot 0.3 \cdot f_{sd}$$  \hspace{1cm} (3.7.3.2b)

where

$\xi = 1.4$ \hspace{1cm} for $d \leq 0.2$ m

$\xi = 1.6 - d$ \hspace{1cm} for $0.2$ m < $d \leq 0.5$ m

$\xi = 1.3 - 0.4$ \hspace{1cm} for $0.5$ m < $d \leq 1.0$ m

$\xi = 0.9$ \hspace{1cm} for $1.0$m < $d$
\[ \rho = \frac{A_{so}}{b_{w} \cdot d}, \text{ not higher than } \rho = 0.02 \text{ (reinforcement content)} \]

\( A_{so} \) is the smallest amount of reinforcement due to bending in the tension zone, in actual beam section, between the zero-point of the bending moment and its maximum point. As an alternative the amount of reinforcement that has a length of \((d + l_b)\) and that passes the actual section can be used. \( l_b \) is the length that is required to anchor the design tension force.

### 4.3 Shear force capacity for the Lautajokki Bridge

From the example we will get the following input data:

- **Width:** \( b_w = 1.0 \text{ m} \)
- **Effective depth:** \( d = 0.295 \text{ m} \)
- **Factor, effective depth:** \( \xi = 1.6 - d = 1.6 - 0.295 = 1.305 \text{ m} \)
- **Reinforcement content:** \( \rho = \frac{A_{so}}{b \cdot d} = \frac{201}{100 \cdot 295} = 0.0068 \) (φ16 s100)

**Formal shear strength:**

\[ f_v = 1.305 \left(1 + 50 \cdot 0.0068\right) \cdot 0.3 \cdot f_{cd} \]

The shear force capacity, \( V_c \), for different strength classes can now be calculated and the result is displayed in Table 4.1.

| Table 4.1 The shear force capacity, \( V_c \) according to Swedish Code BBK94 for different strength classes. |
|:--:|:--:|:--:|:--:|:--:|
| Strength Class | C30 | C40 | C50 | C60 | C80 |
| \( f_{cd} = \frac{f_{ck} \cdot 0.85}{1.5/1.2} \) | 14.17 | 18.89 | 23.61 | 28.33 | 37.78 |
| \( f_{td} = \frac{f_{ck} \cdot 0.85}{1.5/1.2} \) | 1.11 | 1.39 | 1.61 | 1.72 | 1.89 |
| \( f_v = 1.305 \left(1 + 50 \cdot 0.0068\right) \cdot 0.3 \cdot f_{cd} \) | 0.58 | 0.73 | 0.85 | 0.90 | 0.99 |
| \( V_c = 1.0 \cdot 0.295 \cdot f_v \) | 172.0 | 214.9 | 249.3 | 266.5 | 292.3 |

\(^{A1}\) Strength classes according to EC2-draft (1999).

Note: The calculation in this section is not made strictly according to the Swedish code BBK94, since the strength classes are taken from the Eurocode. As mentioned earlier in this chapter it is difficult to compare the codes since there are several factors that are
different besides the strength classes (e.g. load factors and curing conditions for the test specimens).

In this chapter we have chosen to use the “real” loads that the Lautajokki Bridge has been and will be exposed to and the strength values from EC2-draft. To exemplify the complexity of the comparison the following factors can be taken in to account:

Let $C_{\text{cube}}$ refer to the last number in the strength class, e.g. C30/37 ($f_{ck}/f_{ck,cube}$) in EC2-draft.

**BBK94:**

The expression to calculate the characteristic strength value:

$$ f_{ck} = 0.7 \cdot C_{\text{cube}} + 0.5 = 0.7 \cdot C_{\text{cube}} \quad \text{(Table a in section 2.4.1 in BBK94).} $$

The design strength is determined with:

$$ f_{d} = \frac{f_{ck}}{\eta_{m} \cdot \gamma_{n} \cdot 1.5 \cdot 1.2} = \frac{C_{\text{cube}} \cdot 0.7}{2.57} \quad \text{(} \eta_{m} \text{ and } \gamma_{n} \text{ see section 4.1, safety class 3 is used).} $$

**EC2-draft:**

The characteristic value is determined with:

$$ f_{ck} = 0.8 \cdot C_{\text{cube}} \quad \text{(0.85: long term effect. Expression 3.11 in EC2-draft).} $$

The design strength is determined with $f_{d}$:

$$ f_{d} = 0.85 \cdot f_{ck} \cdot \frac{0.85 \cdot C_{\text{cube}}}{1.5} = C_{\text{cube}} \cdot \frac{0.85}{2.21} \quad \text{(0.85: long term effect. Expression 3.11 in EC2-draft).} $$

The load factors in EC2-draft are approximately 20% higher than in BBK94. This gives a design strength value of:

$$ f_{d} = \frac{C_{\text{cube}}}{2.21 \cdot 1.2} = \frac{C_{\text{cube}}}{2.65} $$

On the other hand the strength classes in the Swedish code are based on tests with wet test specimens and in Eurocode on dry test specimens. This results in ca. 10% lower values in BBK94:

$$ f_{d} = \frac{C_{\text{cube}}}{2.57 \cdot 1.1} = \frac{C_{\text{cube}}}{2.82} $$

So, if the strength classes in EC2-draft are used in a calculation with BBK94 the compressive design strength will be approximately 6% lower than with EC2-draft (the factor 2.65 compared to 2.82).

### 4.4 Fatigue capacity of the Lautajokki Bridge

The following method can be used to consider the decrease in strength due to fatigue load (section 2.4.3 in BBK94):
No risk of fatigue failure is assumed to occur, at \( n \) load cycles between the limits \( \sigma_1 \) and \( \sigma_2 \), if \( \sigma_1 \) and \( \sigma_2 \) is a point that is within the curves for actual \( n \) in the graph in Figure 4.1. Between the curves linear interpolation is accepted for log \( n \). For \( n \leq 5 \cdot 10^2 \) it is assumed that fatigue has no influence on the strength.

Example: In Figure 4.1 the critical number of load cycles where fatigue failure can be assumed is shown for one of the strength classes in, C40 at \( P=22.5 \) tons.

Strength class C40, \( P=22.5 \) tons:

\[
\frac{V_c}{f_c} = \frac{\tau}{f_a} = \frac{\sigma_c}{f_{cd}} = 0.16
\]

\[
\Rightarrow n > 1000 \text{ kc}
\]

\[
\frac{V_c}{f_c} = \frac{\tau}{f_a} = \frac{\sigma_c}{f_{cd}} = 0.55
\]

Figure 4.1 Graph for fatigue strength in concrete.

The number of cycles without risk for any fatigue failure is shown in Table 4.2 for the chosen strength classes.
Table 4.2 Fatigue strength. Number of cycles without risk for any fatigue failure according to BBK94 at different strength classes.

<table>
<thead>
<tr>
<th>BBK94 (1994)</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_1/V_c ) for ( P=22.5 ) tons</td>
<td>0.68</td>
<td>0.55</td>
<td>0.47</td>
<td>0.44</td>
<td>0.40</td>
</tr>
<tr>
<td>( V_2/V_c ) for ( P=25 ) tons</td>
<td>0.74</td>
<td>0.60</td>
<td>0.51</td>
<td>0.48</td>
<td>0.44</td>
</tr>
<tr>
<td>( V_2/V_c ) for ( P=30 ) tons</td>
<td>0.85</td>
<td>0.68</td>
<td>0.59</td>
<td>0.55</td>
<td>0.50</td>
</tr>
<tr>
<td>( n ) [kc] for ( P=22.5 ) tons</td>
<td>900</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>( n ) [kc] for ( P=25 ) tons</td>
<td>100</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>( n ) [kc] for ( P=30 ) tons</td>
<td>2</td>
<td>800</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

B) The actual value is higher than 1000 kilocycles but the Swedish code does not give a more precise value.

Table 4.2 shows that for strength class C30 the concrete only manages 900kc at \( P=22.5 \) tons, 100kc at \( P=25 \) tons and 2kc at \( P=30 \) tons.

The fatigue tested bridge has without any visible damage managed 6000kc at \( P=36 \) tons.

5 Fatigue capacity of concrete according to EC2-1 (1991) and EC2-2 (1995)

5.1 The design value for the shear capacity

The design value for the shear capacity, \( V_{rd1} \), according to EC2-1 (1991), section 4.3.2.3: “Elements not requiring design shear reinforcement”:

(1) The design shear resistance, \( V_{rd1} \), is given by:

\[
V_{rd1} = \left[ \tau_{rd} \cdot k \left( 1.2 + 40 \cdot \rho_1 \right) + 0.15 \cdot \sigma_{\gamma} \right] b_w \cdot d
\]

where

- \( \tau_{rd} \) = basic design shear strength = \( \frac{0.25 \cdot f_{ck,0,0}}{\gamma_c} \)
- \( \gamma_c \) should be taken as 1.5.
- \( k = 1 \) for members where more than 50% of the bottom reinforcement is curtailed.
- Otherwise, \( k = 1.6 \cdot d < 1 \) (d in metres)
- \( \rho_1 = \frac{A_{sl}}{b_w \cdot d} \leq 0.02 \)
$A_{sl} =$ the area of tension reinforcement extending not less than $d + l_{b,net}$ beyond the section considered (see Figure 4.12 in EC2-1 (1991)). $l_{b,net}$ is defined in 5.2.2.3 and Figure 5.2 in EC2-1 (1991).

$b_{w} =$ minimum width of the section over the effective depth.

$\sigma_{cp} = N_{Sd}/A_{c}$

$N_{Sd} =$ longitudinal force in section due to loading or prestressing (compression positive).

**5.2 Strength values at fatigue load, section 4.3.7.4 in EC2-2 (1995)**

Fatigue verification according to EC2-2 (1995), section 4.3.7.4: “Fatigue verification for concrete under compression, shear and punching shear”:

(103) In members without shear reinforcement, adequate fatigue resistance of concrete under shear may be assumed if either Equation (4.189) or (4.190), illustrated graphically in Figure 4.135 in EC2-2 (1995), is satisfied. Otherwise a more refined fatigue verification may be necessary (see Appendix 106 for railway bridges).

\[
\text{for } \frac{\tau_{\text{max}}}{\tau_{\text{max}}} \geq 0 : \left[ \frac{\tau_{\text{min}}}{\tau_{Rd1}} \right] \leq 0.5 + 0.45 \cdot \left[ \frac{\tau_{\text{min}}}{\tau_{Rd1}} \right] \leq 0.9 \quad (4.189)
\]

\[
\text{for } \frac{\tau_{\text{max}}}{\tau_{\text{max}}} < 0 : \left[ \frac{\tau_{\text{min}}}{\tau_{Rd1}} \right] \leq 0.5 \cdot \left[ \frac{\tau_{\text{min}}}{\tau_{Rd1}} \right] \quad (4.190)
\]

where $\tau_{\text{max}}$ is the maximum nominal shear stress under frequent combination of actions.

$\tau_{\text{min}}$ is the minimum nominal shear stress under frequent combination of actions at the section where $\tau_{\text{max}}$ occurs.

$\tau_{Rd1}$ design shear resistance according to equation (4.18) in EC2-1 (1991).

From the example we will get the following input data:

Width: $b_{w} = 1000$ mm

Effective depth: $d = 295$ mm

Factor, effective depth: $k = 1.6 - d = 1.6 - 0.295 = 1.305$ m

Reinforcement content: $\rho = \frac{A_{sl}}{b \cdot d} = \frac{201 \cdot 10}{1000 \cdot 295} = 0.0068 \ (\phi 16 \ s 100) \ ; \leq 0.02$

Basic design shear strength:

$\tau_{Rd} = \frac{(0.25 \cdot f_{uk0.05})}{\gamma_{t}} = \frac{(0.25 \cdot f_{uk0.05})}{1.5} = 0.16667 \cdot f_{uk0.05} \text{ MPa}$

The design shear resistance: $V_{Rd1} = \left[ \tau_{Rd} \cdot k (1.2 + 40 \cdot \rho_{t}) \right] 1000 \cdot 295 \text{ N}$

- 13 -
The shear force capacity, $V_{Rd1}$, for different strength classes can now be calculated and the result is displayed in Table 5.1.

**Table 5.1 The shear force capacity, $V_{Rd1}$ according to EC2-1 (1991) for different strength classes.**

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck}$</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>80</td>
</tr>
<tr>
<td>$f_{ck,0,05}$</td>
<td>2.0</td>
<td>2.5</td>
<td>2.9</td>
<td>3.1</td>
<td>3.4</td>
</tr>
</tbody>
</table>

EC2-1 (1991)

[$\tau = \frac{R_{ctk}}{0.05} = 0.33 \cdot 0.42 \cdot 0.48 \cdot 0.52 \cdot 0.57$ MPa]

[$V_{Rd1} = \left( \frac{1.305 \cdot \left( 1.2 + 0.0068 \right)}{1000} \right) \cdot 295 = 189.0 \cdot 236.2 \cdot 274.0 \cdot 292.9 \cdot 321.2$ kN]

Strength classes according to EC2-draft (1999).

The minimum nominal shear force under frequent combination of actions at the section where $V_{max}$ occurs, $V_{min}$, is the same as $V_1$ in section 2 (load on 1m of the slab):

$V_{min} = V_1 = 22.08 \cdot 1.55 = 34.22$ kN

The maximum nominal shear force under frequent combination of actions, $V_{max}$ is the same as $V_2$ in section 2 (load on 1m of the slab):

$P = 22.5$ tons $\Rightarrow V_{max} = V_2 = 117.6$ kN

$P = 25$ tons $\Rightarrow V_{max} = V_2 = 128.0$ kN

$P = 30$ tons $\Rightarrow V_{max} = V_2 = 146.7$ kN

The result of the equation (4.189) is displayed in Table 5.2 for the chosen strength classes:

$$\frac{V_{min}}{V_{max}} \geq 0 : \left| \frac{V_{max}}{V_{Rd1}} \right| \leq 0.5 + 0.45 \left| \frac{V_{min}}{V_{Rd1}} \right|$$

Example: Strength class C40 at $P = 22.5$ tons

$$\frac{117.6}{236.2} \leq 0.5 + 0.45 \cdot \frac{34.22}{236.2} \Rightarrow 0.489 \leq 0.565 \Rightarrow OK! \quad (;< 0.9)$$

Table 5.2 shows that for almost all strength classes the concrete fulfills the condition. For the other strength classes a fatigue verification must be made.
Table 5.2 Fatigue strength. Fatigue verification according to section 4.3.7.4: “Fatigue verification for concrete under compression, shear and punching shear” in EC2-2 (1995).

<table>
<thead>
<tr>
<th>Load</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>P=22.5 tons</td>
<td>0.622 ≥ 0.581</td>
<td>0.498 ≤ 0.565</td>
<td>0.429 ≤ 0.556</td>
<td>0.402 ≤ 0.553</td>
<td>0.366 ≤ 0.548</td>
</tr>
<tr>
<td></td>
<td>⇒ NOT OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
</tr>
<tr>
<td>P=25 tons</td>
<td>0.677 ≥ 0.581</td>
<td>0.542 ≤ 0.565</td>
<td>0.467 ≤ 0.556</td>
<td>0.437 ≤ 0.553</td>
<td>0.398 ≤ 0.548</td>
</tr>
<tr>
<td></td>
<td>⇒ NOT OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
</tr>
<tr>
<td>P=30 tons</td>
<td>0.776 ≥ 0.581</td>
<td>0.621 ≥ 0.565</td>
<td>0.535 ≤ 0.556</td>
<td>0.501 ≤ 0.553</td>
<td>0.457 ≤ 0.548</td>
</tr>
<tr>
<td></td>
<td>⇒ NOT OK</td>
<td>⇒ NOT OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
<td>⇒ OK</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

5.3 Strength values at fatigue load, Appendix 106: Damage equivalent stresses for fatigue verification in EC2-2 (1995)

In EC2-2 (1995), Appendix 106: “Damage equivalent stresses for fatigue verification”, the following is stated:

(101) For concrete subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

\[ 14 \cdot \frac{1 - S_{d,\text{max,eq}}}{\sqrt{1 - R_{eq}}} \geq 6 \quad (A106.11) \]

where

\[ R_{eq} = \frac{S_{d,\text{min,eq}}}{S_{d,\text{max,eq}}} \]

\[ S_{d,\text{min,eq}} = \frac{\sigma_{d,\text{min,eq}}}{f_{d,\text{fat}}} \]

\[ S_{d,\text{max,eq}} = \frac{\sigma_{d,\text{max,eq}}}{f_{d,\text{fat}}} \]

where

\[ f_{d,\text{fat}} = [\text{Assume:}] f_d / \gamma_{d,\text{fat}} \]

\[ \sigma_{d,\text{max,eq}} \] upper stresses of the damage equivalent stress range with a number of cycles \( N=10^6 \)
\[ \sigma_{d,\text{min},\text{equ}} \text{ lower stresses of the damage equivalent stress range with a number of cycles } N=10^6 \]

If this method is applied also for tension you will get the following:

Table 5.1, C30 at \( P = 22.5 \text{ tons} \Rightarrow f_{\text{ck}} = 30 \text{ MPa} \)

\[ f_{\text{ck},0.05} = 2.0 \text{ MPa} \]

Partial safety factor for concrete for fatigue verification: \( \gamma_{\text{fat}} = 1.5 \) (table 4.115 in EC2-2 (1995)).

Basic design shear strength (fatigue): \( \tau_{Rd,\text{fat}} = \frac{0.25 \cdot f_{\text{ck},0.05}}{\gamma_{\text{fat}}} = \frac{0.25 \cdot 2.0}{1.5} = 0.3333 \text{ MPa} \)

The fatigue design shear resistance:

\[ V_{Rd,\text{fat}} = \left[ 0.3333 \cdot 1.305 \cdot (1.2 + 40 \cdot 0.0068) \right] 1000 \cdot 295 = 188.9 \text{ kN} \]

NOTE: The values for the fatigue design shear resistance will be the same as in the “normal” case, i.e. \( V_{Rd,\text{fat}} = V_{Rd} \).

If the following is stated:

\[ S_{d,\text{min},\text{equ}} = \frac{\sigma_{d,\text{min},\text{equ}}}{f_{d,\text{fat}}} = \frac{V_{\text{min}}}{V_{Rd,\text{fat}}} = \frac{34.22}{188.9} = 0.181 \quad (6.45) \]

\[ S_{d,\text{max},\text{equ}} = \frac{\sigma_{d,\text{max},\text{equ}}}{f_{d,\text{fat}}} = \frac{V_{\text{max}}}{V_{Rd,\text{fat}}} = \frac{117.6}{188.9} = 0.623 \quad (6.46) \]

\[ R_{\text{equ}} = \frac{S_{d,\text{min},\text{equ}}}{S_{d,\text{max},\text{equ}}} = \frac{0.181}{0.623} = 0.291 \quad (6.44) \]

Finally:

\[ 14 \cdot \frac{1 - 0.623}{\sqrt{1 - 0.291}} = 6.26 \geq 6 \Rightarrow \text{OK}! \quad (6.43) \]

The value 6.26 corresponds to \( \sim 1.9 \text{ M cycles} \).

The number of cycles without risk for any fatigue failure is shown in Table 5.3 for the chosen strength classes.
Table 5.3 Number of cycles to failure for the Lautajokki Bridge according to EC2-1 (1991)/EC2-2 (1995) at different axle loads and strength classes.

<table>
<thead>
<tr>
<th></th>
<th>A) Strength Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C30</td>
</tr>
<tr>
<td>[log n], P=22.5 tons</td>
<td>6.28</td>
</tr>
<tr>
<td>[log n], P=25 tons</td>
<td>5.28</td>
</tr>
<tr>
<td>[log n], P=30 tons</td>
<td>3.58</td>
</tr>
<tr>
<td>n [kc], P=22.5 tons</td>
<td>1901</td>
</tr>
<tr>
<td>n [kc], P=25 tons</td>
<td>189</td>
</tr>
<tr>
<td>n [kc], P=30 tons</td>
<td>3.8</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

6 Fatigue capacity of concrete according to EC2-draft (1999)

6.1 The design value for the shear capacity

The design value for the shear capacity, \( V_{rd,ct} \), according to EC2-draft (1999), section 6.2.3: “Members not requiring design shear reinforcement”:

1. The design value for the shear capacity, \( V_{rd,ct} \), follows from:

\[
V_{rd,ct} = \left[ 0.12 \cdot k \left( 100 \cdot \rho_t \cdot f_{ck} \right)^{1/3} - 0.15 \cdot \sigma_{cd} \right] b_w \cdot d
\]

where

- \( f_{ck} \) in MPa
- \( k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \) with \( d \) in [mm]
- \( \rho_t = \frac{A_d}{b_w \cdot d} \leq 0.02 \)
- \( A_d \) Cross-sectional area of the tensile reinforcement, which continues at least over an additional distance \( d \) beyond the section considered and is there effectively anchored (figure 6.2 in EC2-draft (1999))
- \( b_w \) smallest width of the cross-section in the tensile area
- \( \sigma_{cd} = N_E/A \) in MPa
- \( N_E \) axial force in the cross-section due to loading or prestressing \((N_E<0 \) for compression)
6.2 Strength values at fatigue load, section 6.8.6: Simplified verification

In this calculation the simplified verification is used, according to EC2-draft (1999), section 6.8.6: “Simplified verification”:

(6) In members without shear reinforcement, a fatigue verification for concrete under shear need not be performed if the following conditions are met:

\[
\frac{V_{\text{max}}}{V_{\text{Rd1}}} \geq 0.5 + 0.45 \cdot \frac{V_{\text{min}}}{V_{\text{Rd1}}} \quad \text{for} \quad f_d \leq 50 \text{ N/mm}^2 \\
\frac{V_{\text{max}}}{V_{\text{Rd1}}} \geq 0.8 \quad \text{for} \quad f_d \geq 55 \text{ N/mm}^2
\]  

where

- \( V_{\text{max}} \): maximum nominal shear force under frequent combination of actions
- \( V_{\text{min}} \): minimum nominal shear force under frequent combination of actions at the section where \( V_{\text{max}} \) occurs
- \( V_{\text{Rd1}} \): design shear resistance according to section 6.2 in prEN 1992-1:2001 (1st draft)

From the example we will get the following input data:

Width: \( b_w = 1000 \text{ mm} \)

Effective depth: \( d = 295 \text{ mm} \)

Factor, effective depth: \( k = 1 + \frac{200}{295} = 1.823 \leq 2.0 \)

Reinforcement content: \( \rho \frac{A_d}{b \cdot d} = \frac{201 \cdot 10}{1000 \cdot 295} = 0.0068 \) (ø16 8 100) ; \( \leq 0.02 \)

The shear capacity: \( V_{\text{Rd,ct}} = \left[ 0.12 \cdot 1.823 \cdot \left( 100 \cdot 0.0068 \cdot f_d \right)^{1/3} \right] \cdot 1000 \cdot 295 \text{ N} \)

The shear force capacity, \( V_{\text{Rd,ct}} (= V_{\text{Rd1}}) \), for different strength classes can now be calculated and the result is displayed in Table 6.1.

### Table 6.1 The shear force capacity, \( V_{\text{Rd,ct}} \), according to EC2-draft (1999) for different strength classes.

<table>
<thead>
<tr>
<th>EC2-draft (1999)</th>
<th>( f_d )</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{cd} ) = ( f_d / 1.5 )</td>
<td>20 26.7 33.3 40.0 53.3</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>80</td>
</tr>
<tr>
<td>( V_{\text{Rd,ct}} )</td>
<td>176.5 194.2 209.2 222.4 244.7</td>
<td>1000 1000 1000 1000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( f_{cd} \): Strength class according to EC2-draft (1999).
\( f_d \):according to equation 3.11; \( \gamma = 1.5 \) (concrete).

The minimum nominal shear force under frequent combination of actions at the section where \( V_{\text{max}} \) occurs, \( V_{\text{min}} \), is the same as \( V_1 \) in section 2 (load on 1m of the slab):
\[ V_{\text{min}} = V_1 = 22.08 \cdot 1.55 = 34.22 \text{ kN} \]

The maximum nominal shear force under frequent combination of actions, \( V_{\text{max}} \) is the same as \( V_2 \) in section 2 (load on 1m of the slab):

- \( P = 22.5 \text{ tons} \Rightarrow V_{\text{max}} = V_2 = 117.6 \text{ kN} \)
- \( P = 25 \text{ tons} \Rightarrow V_{\text{max}} = V_2 = 128.0 \text{ kN} \)
- \( P = 30 \text{ tons} \Rightarrow V_{\text{max}} = V_2 = 146.7 \text{ kN} \)

The result of the equation (6.49) is displayed in Table 6.2 for the chosen strength classes:

\[
\frac{V_{\text{min}}}{V_{\text{max}}} \geq 0 ; \quad \frac{V_{\text{max}}}{V_{Rd1}} \leq 0.5 + 0.45 \left( \frac{V_{\text{min}}}{V_{Rd1}} \right)
\]

Example: Strength class C40 at \( P = 22.5 \) tons:

\[
\frac{117.6}{194.2} \leq 0.5 + 0.45 \left( \frac{34.22}{194.2} \right) \Rightarrow 0.605 \geq 0.579 \Rightarrow \text{NOT OK!} \quad (;< 0.9)
\]

Table 6.2 shows that for almost all strength classes the concrete does not fulfil the condition. For these fatigue verification must be made.

**Table 6.2 Fatigue strength. Fatigue verification according to section 6.8.6: “Simplified verification” in EC2-draft (1999).**

<table>
<thead>
<tr>
<th>EC2-draft (1999)</th>
<th>( P=22.5 ) tons</th>
<th>( P=25 ) tons</th>
<th>( P=30 ) tons</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>C30</strong></td>
<td>0.666 ( \geq ) 0.587</td>
<td>0.650 ( \geq ) 0.579</td>
<td>0.612 ( \geq ) 0.574</td>
</tr>
<tr>
<td>( \Rightarrow ) NOT OK</td>
<td>( \Rightarrow ) NOT OK</td>
<td>( \Rightarrow ) OK</td>
<td>( \Rightarrow ) OK</td>
</tr>
<tr>
<td><strong>C40</strong></td>
<td>0.562 ( \leq ) 0.569</td>
<td>0.576 ( \leq ) 0.569</td>
<td>0.576 ( \leq ) 0.569</td>
</tr>
<tr>
<td><strong>C50</strong></td>
<td>0.529 ( \leq ) 0.569</td>
<td>0.523 ( \leq ) 0.565</td>
<td>0.523 ( \leq ) 0.565</td>
</tr>
<tr>
<td><strong>C60</strong></td>
<td>0.481 ( \leq ) 0.563</td>
<td>0.599 ( \geq ) 0.563</td>
<td></td>
</tr>
<tr>
<td><strong>C80</strong></td>
<td>( \Rightarrow ) OK</td>
<td>( \Rightarrow ) OK</td>
<td>( \Rightarrow ) OK</td>
</tr>
</tbody>
</table>

\( ^1 \) Strength classes according to EC2-draft (1999).
6.3 Strength values at fatigue load, section 6.8.5: Verification using damage equivalent stress

In EC2-draft (1999), section 6.8.5: “Verification using damage equivalent stress”, the following is stated:

(1) Instead of an explicit verification of the operational strength according to 6.8.4 the fatigue verification of standard cases with known loads (railway and road bridges) may also be performed as follows:

by damage equivalent stress ranges for steel according to (2)
damage equivalent compression stresses for concrete according to (3)

(3) A satisfying fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

\[ 14 \cdot \frac{1 - S_{\Delta d, \text{max, e}}}{\sqrt{1 - R_{\text{e, qu}}}} \geq 6 \]  \hspace{1cm} (6.43)

where

\[ R_{\text{e, qu}} = \frac{S_{\Delta d, \text{min, e}}}{S_{\Delta d, \text{max, e}}} \]  \hspace{1cm} (6.44)

\[ S_{\Delta d, \text{min, e}} = \frac{\sigma_{\Delta d, \text{min, e}}}{f_{d, \text{fat}}} \]  \hspace{1cm} (6.45)

\[ S_{\Delta d, \text{max, e}} = \frac{\sigma_{\Delta d, \text{max, e}}}{f_{d, \text{fat}}} \]  \hspace{1cm} (6.46)

where

\( f_{d, \text{fat}} \) is the design fatigue strength of concrete according to expression (6.48)
\( \sigma_{\Delta d, \text{max, e}} \) upper stress of the ultimate amplitude for \( N=10^6 \) cycles
\( \sigma_{\Delta d, \text{min, e}} \) lower stress of the ultimate amplitude for \( N=10^6 \) cycles

Expression (6.48):

\[ f_{d, \text{fat}} = 0.85 \cdot \beta_c(t_0) \cdot f_d \left( 1 - \frac{f_{d, \text{fat}}}{250} \right) \]  \hspace{1cm} (6.48)

where

\( \beta_c \) coefficient for concrete strength at first load application, \( \beta_c(t_0) = 10^{-0.2 \left[ 1 - \left( \frac{t_0}{t_1} \right)^{0.5} \right]} \)

\( t_1 \) reference time; \( t_1 = 1 \) day
\( t_0 \) time of first loading of concrete (in days)
If this method is applied also for tension you will get the following:

Table 6.1, C30 at P = 22.5 tons ⇒ $f_{ck} = 30 \text{ MPa}, f_{cd} = 20 \text{ MPa}$

Expression (6.48) gives:

$$f_{d, fa} = 0.85 \cdot \beta(t_0) \cdot f_{d} \left(1 - \frac{f_k}{250}\right)$$

Assume: $t_0 = 40$ days ⇒ $\beta(t_0) = 10 \cdot \sqrt{\frac{23}{380}} = 1.078$

(if $t_0$ is assumed to 150 days ⇒ $\beta(t_0) = 1.299$ ⇒ $V_{Rd,ct-fat} = 174.6 \text{ kN}$, which is 6% higher than $V_{Rd,ct-fat}$ for 40 days in the example below)

$$f_{d, fa} = 0.85 \cdot \beta(t_0) \cdot f_{d} \left(1 - \frac{f_k}{250}\right) = 0.85 \cdot 1.078 \cdot 20 \left(1 - \frac{30}{250}\right) = 16.13 \text{ MPa}$$

To be able to use the design value for the shear capacity, $V_{Rd,ct}$ (according to section 6.2.3 in EC2-draft (1999): Members not requiring design shear reinforcement), the design value must be made to a characteristic value, $f_{ck, fat}$ according to:

$$f_{ck, fat} = f_{d, fa} \cdot \gamma = 16.13 \cdot 1.5 = 24.19 \text{ MPa}$$

The design fatigue value for the shear capacity, $V_{Rd,ct-fat}$:

$$V_{Rd,ct-fat} = \left[0.12 \cdot 1.823 \cdot \left(100 \cdot 0.0068 \cdot f_{ck, fat}\right)^{1/3}\right] \cdot 1000 \cdot 295 =$$

$$= \left[0.12 \cdot 1.823 \cdot \left(100 \cdot 0.0068 \cdot 24.13\right)^{1/3}\right] \cdot 1000 \cdot 295 = 164.3 \text{ kN}$$

If the following is stated:

$$S_{d, min, equ} = \sigma_{d, min, equ} \frac{f_{d, fa}}{V_{Rd,ct-fat}} = \frac{V_{min}}{V_{Rd,ct-fat}} = \frac{34.22}{164.1} = 0.2085 \quad (6.45)$$

$$S_{d, max, equ} = \sigma_{d, max, equ} \frac{f_{d, fa}}{V_{Rd,ct-fat}} = \frac{V_{max}}{V_{Rd,ct-fat}} = \frac{117.6}{164.1} = 0.7166 \quad (6.46)$$

$$R_{equ} = \frac{S_{d, min, equ}}{S_{d, max, equ}} = \frac{0.2085}{0.7166} = 0.2910 \quad (6.44)$$

Finally:
The value 4.71 corresponds to ~52 kilo cycles.

The fatigue shear force capacity, $V_{Rd,ct} (= V_{Rd,i})$, for different strength classes can now be calculated and the result is displayed in Table 6.1.

Table 6.3 The fatigue shear force capacity, $V_{Rd,ct-fat}$ according to EC2-draft (1999) for different strength classes.

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck}$</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>80</td>
</tr>
<tr>
<td>$f_{cd}$</td>
<td>20</td>
<td>26.7</td>
<td>33.3</td>
<td>40.0</td>
<td>53.3</td>
</tr>
<tr>
<td>$f_{cd,fat}$</td>
<td>16.1</td>
<td>20.5</td>
<td>24.4</td>
<td>27.9</td>
<td>33.2</td>
</tr>
<tr>
<td>$f_{ck,fat}$</td>
<td>24.19</td>
<td>30.79</td>
<td>36.66</td>
<td>41.79</td>
<td>49.85</td>
</tr>
</tbody>
</table>

\[
V_{Rd,ct-fat} = 164.3 \cdot 100 \cdot 295 = 178.0 \cdot 5086 = 188.7 \cdot 18751 = 197.1 \cdot 18571 = 209.0 \cdot 18451
\]

A) Strength classes according to EC2-draft (1999).

The number of cycles, $n$, without risk for any fatigue failure is shown in Table 6.4 for the chosen strength classes.

Table 6.4 Number of cycles, $n$, to failure for the Lautajokki-bridge according to EC2-draft (1999) at different axle loads and strength classes.

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td>$[\log n]$, P=22.5 tons</td>
<td>4.72</td>
<td>5.64</td>
<td>6.26</td>
<td>6.71</td>
<td>7.27</td>
</tr>
<tr>
<td>$[\log n]$, P=25 tons</td>
<td>3.61</td>
<td>4.60</td>
<td>5.26</td>
<td>5.73</td>
<td>6.34</td>
</tr>
<tr>
<td>$[\log n]$, P=30 tons</td>
<td>1.71</td>
<td>2.81</td>
<td>3.56</td>
<td>4.09</td>
<td>4.77</td>
</tr>
<tr>
<td>$n [\text{kC}]$, P=22.5 tons</td>
<td>53</td>
<td>440</td>
<td>1834</td>
<td>5086</td>
<td>18751</td>
</tr>
<tr>
<td>$n [\text{kC}]$, P=25 tons</td>
<td>4</td>
<td>39</td>
<td>182</td>
<td>542</td>
<td>2193</td>
</tr>
<tr>
<td>$n [\text{kC}]$, P=30 tons</td>
<td>0.05</td>
<td>0.65</td>
<td>3.6</td>
<td>12</td>
<td>59</td>
</tr>
</tbody>
</table>

A) Strength classes according to EC2-draft (1999).

7 Discussion

In Table 7.1 you can see a summary of the number of load cycles the Lautajokki Bridge could manage with the compared codes. In the table the more refined methods from the different codes are compared (see sections 4.3, 5.3 and 6.3).

If we compare the results in Table 7.1 we can see that there are differences. The calculation with EC2-2 (1995) gives the least conservative values. The Swedish code is slightly more conservative than the EC2-2, but they are still fairly similar. The new version of Eurocode, EC2-draft (1999), is by far the most conservative one.
For example: for the strength class C30 at P = 30 tons the Swedish Code gives that the bridge should manage 2000 cycles before failure, the EC2-draft (1999) gives 50 cycles and the EC2-2 (1995) gives 3800 load cycles. We then have in mind that the bridge managed 6M cycles without failure (full-scale test performed at LTU, 1996), which points out that none of the three compared codes is especially accurate.

Table 7.1 Number of load cycles (n in million load cycles) for the Lautajokki bridge. Comparison between EC2-1 (1991)/EC2-2 (1995), BBK94 (1994) and EC2-draft (1999) at different strength classes.

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>Code</th>
<th>Number of load cycles, n [M cycles]</th>
<th>22.5 tons</th>
<th>25 tons</th>
<th>30 tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>C30</td>
<td>Swedish</td>
<td>0.90</td>
<td>0.10</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>1.9</td>
<td>0.19</td>
<td>0.0038</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>0.053</td>
<td>0.004</td>
<td>0.00005</td>
<td></td>
</tr>
<tr>
<td>C40</td>
<td>Swedish</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>223</td>
<td>31</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>0.44</td>
<td>0.039</td>
<td>0.0007</td>
<td></td>
</tr>
<tr>
<td>C50</td>
<td>Swedish</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>3093</td>
<td>519</td>
<td>26.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>1.8</td>
<td>0.18</td>
<td>0.0036</td>
<td></td>
</tr>
<tr>
<td>C60</td>
<td>Swedish</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>8929</td>
<td>1615</td>
<td>95.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>5.1</td>
<td>0.5</td>
<td>0.012</td>
<td></td>
</tr>
<tr>
<td>C80</td>
<td>Swedish</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td>&gt;1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC2</td>
<td>34659</td>
<td>6900</td>
<td>486</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Draft</td>
<td>18.8</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
</tbody>
</table>

\(^{a)}\text{Strength classes according to EC2-draft (1999).}\)

What are the differences between the three codes? Some of them are:

- The expression to calculate the shear force capacity, see Table 7.2.
- The expression to calculate the design fatigue strength.
- The Swedish Code gives an approximate value of the number of cycles before failure (the value is taken from a graph).

The reason why EC2-draft (1999) is much more conservative than the EC2-2 (1995)/EC2-1 (1991) depends on how the shear strength is calculated, see Table 7.2. The new equation in EC2-draft (1999) to calculate the design value for the shear strength gives a lower value compared to the EC2-1 (1991).
The “new” equation for the design shear strength in EC2-draft (1999) is taken from MC 90 (1993), section 6.4.2.3: “Shear in cracked zones without shear reinforcement”. It has the following formula:

\[ V_{\text{RdI}} = 0.12 \cdot \xi \cdot (100 \cdot \rho \cdot f_{cd})^{1/3} \cdot b_{rd} \cdot d \]

where \( \xi = k \) (Eq. 6.4-8 in MC 90 (1993))

The reason for this change is that there has been some uncertainty when it comes to high strength ordinary concrete and to be on the safe side a reduction of the design shear strength has been made.

This reduction has led to, for example, that the difference in fatigue shear strength is as much as 35% for C80 (\( V_{\text{RdI-fat}} \) and \( V_{\text{RdI,ct-fat}} \)).

### Table 7.2 Design value for the shear strength according to EC2-1 (1991) and EC2-draft (1999).

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C80</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EC2-1 (1991) / EC2-2 (1995)</strong></td>
<td>( V_{\text{Rdi}} = )</td>
<td>189.0</td>
<td>236.2</td>
<td>274.0</td>
<td>292.9</td>
</tr>
<tr>
<td></td>
<td>( V_{\text{RdI-fat}} = )</td>
<td>189.0</td>
<td>236.2</td>
<td>274.0</td>
<td>292.9</td>
</tr>
<tr>
<td><strong>EC2-draft (1999)</strong></td>
<td>( V_{\text{RdI}} = )</td>
<td>176.5</td>
<td>194.2</td>
<td>209.2</td>
<td>222.4</td>
</tr>
<tr>
<td></td>
<td>( V_{\text{RdI,ct-fat}} = )</td>
<td>164.3</td>
<td>178.0</td>
<td>188.7</td>
<td>197.1</td>
</tr>
</tbody>
</table>

*Strength classes according to EC2-draft (1999).*

An interesting thing is that the trend seems to go in a more conservative direction when it comes to concrete fatigue. Thus, the newest Code is the most conservative one!

More work is needed in this area and it might be an idea to keep the “old” equation (the one in the EC2-1 (1991)) to calculate the design shear strength when the fatigue capacity is studied.

Regarding the difficulties with a comparison of this kind (i.e. different strength classes, load factors etc. between the codes) – see the note at the end of section 4.3.

The studied codes generally give too conservative values, especially for high strength concrete. Further work is needed in this area, especially regarding the shear force capacity.

### References


Appendix A. Concrete Codes

Excerpt from:

- EC2-draft (1999)
- MC90 (1990)
Excerpt from EC2-draft (1999)


Chapters/sections:

- 3.1 Concrete (materials), Table 3.1: Stress and deformation characteristics for normal concrete. Page 33.

- 6.2 Shear, sections 6.2.1 – 6.2.3. Pages 93-96.

- 6.8 Fatigue, sections 6.8.1 – 6.8.6. Pages 121-126
<table>
<thead>
<tr>
<th>$f_{ck}$ (MPa)</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck,ne}$ (MPa)</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>30</td>
<td>37</td>
<td>45</td>
<td>50</td>
<td>55</td>
<td>60</td>
<td>67</td>
<td>75</td>
<td>85</td>
<td>95</td>
<td>105</td>
<td>115</td>
</tr>
<tr>
<td>$f_{cm}$ (MPa)</td>
<td>20</td>
<td>24</td>
<td>28</td>
<td>33</td>
<td>38</td>
<td>43</td>
<td>48</td>
<td>53</td>
<td>58</td>
<td>63</td>
<td>68</td>
<td>78</td>
<td>88</td>
<td>98</td>
<td>108</td>
</tr>
<tr>
<td>$f_{cm}$ (MPa)</td>
<td>1.6</td>
<td>1.9</td>
<td>2.2</td>
<td>2.6</td>
<td>2.9</td>
<td>3.2</td>
<td>3.5</td>
<td>3.8</td>
<td>4.1</td>
<td>4.2</td>
<td>4.4</td>
<td>4.6</td>
<td>4.8</td>
<td>5.0</td>
<td>5.2</td>
</tr>
<tr>
<td>$f_{cm,0.05}$ (MPa)</td>
<td>1.1</td>
<td>1.3</td>
<td>1.5</td>
<td>1.8</td>
<td>2.0</td>
<td>2.2</td>
<td>2.5</td>
<td>2.7</td>
<td>2.9</td>
<td>3.0</td>
<td>3.1</td>
<td>3.2</td>
<td>3.4</td>
<td>3.5</td>
<td>3.7</td>
</tr>
<tr>
<td>$f_{cm,0.95}$ (MPa)</td>
<td>2.0</td>
<td>2.5</td>
<td>2.9</td>
<td>3.3</td>
<td>3.8</td>
<td>4.2</td>
<td>4.6</td>
<td>4.9</td>
<td>5.3</td>
<td>5.5</td>
<td>5.7</td>
<td>6.0</td>
<td>6.3</td>
<td>6.6</td>
<td>6.8</td>
</tr>
<tr>
<td>$E_{cm}^{10}$ (GPa)</td>
<td>27</td>
<td>29</td>
<td>30</td>
<td>31.5</td>
<td>33</td>
<td>34</td>
<td>35</td>
<td>36</td>
<td>37</td>
<td>38</td>
<td>39</td>
<td>41</td>
<td>42</td>
<td>44</td>
<td>45</td>
</tr>
<tr>
<td>$e_{cc}$ (%)</td>
<td>-1.8</td>
<td>-1.9</td>
<td>-2.1</td>
<td>-2.2</td>
<td>-2.3</td>
<td>-2.4</td>
<td>-2.5</td>
<td>-2.6</td>
<td>-2.6</td>
<td>-2.7</td>
<td>-2.8</td>
<td>-2.9</td>
<td>-2.9</td>
<td>-3.0</td>
<td>see Fig. 33</td>
</tr>
<tr>
<td>$e_{cd}$ (%)</td>
<td>-3.5</td>
<td>-3.4</td>
<td>-3.3</td>
<td>-3.2</td>
<td>-3.1</td>
<td>-3.0</td>
<td>-3.0</td>
<td>see Fig. 33</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$e_{cd}$ (%)</td>
<td>-2.0</td>
<td>-2.03</td>
<td>-2.06</td>
<td>-2.1</td>
<td>-2.14</td>
<td>-2.17</td>
<td>-2.2</td>
<td>see Fig. 34</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$e_{cd}$ (%)</td>
<td>-3.5</td>
<td>-3.1</td>
<td>-2.7</td>
<td>-2.5</td>
<td>-2.4</td>
<td>-2.3</td>
<td>-2.2</td>
<td>see Fig. 34</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$n$</td>
<td>2.0</td>
<td>2.0</td>
<td>1.9</td>
<td>1.8</td>
<td>1.7</td>
<td>1.6</td>
<td>1.6</td>
<td>1.55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$e$</td>
<td>-1.35</td>
<td>-1.35</td>
<td>-1.4</td>
<td>-1.5</td>
<td>-1.6</td>
<td>-1.65</td>
<td>-1.7</td>
<td>see Fig. 34</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$e$</td>
<td>-3.5</td>
<td>-3.5</td>
<td>-2.7</td>
<td>-2.5</td>
<td>-2.4</td>
<td>-2.3</td>
<td>-2.2</td>
<td>see Fig. 35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
(6) For prestressed members with permanently unbonded tendons, it is generally necessary to take the deformation of the whole member into account when calculating the increase of the stress in the prestressing steel.

(7) For external prestressing tendons the strain in the prestressing steel between two subsequent contact points (anchors or deviation saddles) is assumed to be constant. The strain in the prestressing steel is then equal to the initial strain, realised just after finishing the prestressing operation, increased with the part resulting from the structural deformation between the contact areas considered.

6.2 Shear

6.2.1 General verification procedure

(1) For the verification of the capacity in regions with shear forces the following design values are defined:

\[ V_{rd,ct} \] the design shear resistance of the member without shear reinforcement

\[ V_{rd,sy} \] the design value of the shear force which can be transmitted by the yielding shear reinforcement

\[ V_{rd,max} \] the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts

(2) In regions of the member where the condition \( V_{Ed,w} < V_{rd,ct} \) applies, no calculated shear reinforcement is necessary. For the definition of \( V_{Ed,w} \), the design value of the shear force, see Expression (6.1) in Section 6.2.2.

(3) When, on the basis of the design shear calculation, no shear reinforcement is necessary, a minimum shear reinforcement according to 8.1.2 should be provided. The minimum shear reinforcement can be deleted in members such as slabs (solid, ribbed or hollow core slabs) and those of secondary importance.

(4) In regions where the design shear force \( V_{Ed,w} \) is larger than \( V_{rd,ct} \), according to Expression 6.2 or 6.3, sufficient shear reinforcement should be provided in order that \( V_{Ed,w} < V_{rd,sy} \) (see 6.2.4).

(5) In no region of the member should the design shear force exceed the value \( V_{rd,max} \) (see 6.2.4).

(6) With regard to the design of the longitudinal reinforcement in regions subjected to shear, the increase of the force in this reinforcement, according to the truss analogy, should be taken into account.

6.2.2 Design value of the shear force

(1) In members with variable cross-sectional depths or inclined prestressing tendons the design value of the shear force \( V_{Ed,w} \), taking account of the shear carrying components in the inclined compression- and tensile chords, is given by:

\[ V_{Ed,w} = V_{Ed} - V_{cod} - V_{td} - V_{cd} \] (6.1)

where

- \( V_{Ed} \) design value of the shear force in the section considered caused by external loading.
- \( V_{cld} \) design value of the shear component of the force in the compression area, in the case of an inclined compression chord.
- \( V_{ld} \) design value of the shear component of the force in the tensile area, in the case of an inclined tensile chord.
- \( V_{pld} \) shear component of inclined prestressing cables, at the ultimate limit state.

In members with constant cross-sectional depths the terms \( V_{cld} \) and \( V_{pld} \) are 0.

(2) For the calculation of the shear reinforcement in members subjected to uniform loading the design shear force in the cross-section can be calculated at a distance \( d \) from the inner edge of the support. This is because the loads between this section and the support are transmitted directly to the support through concrete compression, without participation of the shear reinforcement. For the control of the maximum shear capacity \( V_{Rd,\text{ct}} \), reached by concrete crushing in the web of the member, no reduction applies.

### 6.2.3 Members not requiring design shear reinforcement

(1) The design value for the shear capacity \( V_{Rd,\text{ct}} \) is given by:

\[
V_{Rd,\text{ct}} = [0,12k(100 \rho f_{cd})^{1/2} - 0,15 \sigma_{cd}]b_w d
\]

(6.2)

where

- \( f_{cd} \) in MPa
- \( k = 1 + \frac{200}{d} \) ≤ 2,0 with \( d \) in mm
- \( \rho = \frac{A_d}{b_w d} \) ≤ 0,02
- \( A_d \) Cross-sectional area of the tensile reinforcement, which continues at least over an additional distance \( d \) beyond the section considered and is effectively anchored (Figure 6.2).
- \( b_w \) smallest width of the cross-section in the tensile area
- \( \sigma_{cd} = \frac{N_{Ed}}{A_d} \) in MPa
- \( N_{Ed} \) axial force in the cross-section due to loading or prestressing (\( N_{Ed}<0 \) for compression)
Figure 6.2 Definition of $A_d$ in Expression (6.3)

(2) In single span members without shear reinforcement, prestressed with strands or wires with direct bond, the shear capacity of the regions cracked in bending should be controlled using Expression (6.3). In regions uncracked in bending (where the flexural tensile stress is smaller than $f_{ck,0.05}/\gamma_c$), however, the shear capacity is limited by the tensile strength of the concrete in the web. For those regions the shear capacity is given by:

$$V_{\text{max}} = \frac{l \cdot b_c}{S} \left( \frac{f_{ck,0.05}}{\gamma_c} \right)^{2} - 0.9 \alpha \sigma_{\text{cym}} \frac{f_{ck,0.05}}{\gamma_c}$$

(6.3)

where

- $l$ Moment of inertia
- $S$ static moment
- $\alpha_i$ $l_d/l_{\text{net}} < 1.0$
- $l_c$ distance of section considered from the starting point of the anchorage length
- $l_{\text{ud}}$ upper bound value of the transmission length of the prestressing element according to Expression 8.18.
- $\sigma_{\text{cym}}$ mean concrete compressive stress due to axial loading or prestressing ($\sigma_{\text{cym}} = N_d/N_c$).

(3) The calculation of the shear capacity according to Expression (6.3) need not be carried out for cross-sections that are nearer to the support than the point which is the intersection of the member axis and a line inclined from the inner edge of the loading area at an angle of $45^\circ$.

(4) For the design of the longitudinal reinforcement the $M_d$ -line should be shifted over a distance $a_i < 2.5 d$ from the edge of a support such as in a short beam or a corbel (Figure 6.3) the load is assumed to be transmitted to the support by a concrete strut.

Figure 6.3 Direct strut action

By virtue of this strut action, the shear capacity in the region between the loads is increased to:

\[
V_{Rd,ct} = \left[0.12k(100\rho f_{\alpha})^{1/3}\left(\frac{2.5}{a/d}\right) - 0.15\sigma_{cd}\right]b_sd
\]  

(6.4)

which is directly transmitted to the support.

The increased shear capacity may only be allowed if the anchorage of the longitudinal reinforcement at the node is designed for the full tensile force.

The value \(V_{Rd,ct}\) in Expression (6.4) shall not exceed

\[
V_{Rd,max} = \left[4f_{cd,0.5\alpha} \sqrt{\frac{A_d}{bd}}\right]b_sd
\]  

(6.5)

where \(A_d\) is the area of the bearing plate (Figure 6.3)

6.2.4 Members requiring design shear reinforcement

1. The design of members with shear reinforcement is based on a truss model (Figure 6.4). Limiting values for the angle of the inclined struts in the web are given in Section 6.2.4 (2).
The value of $F_{n,\text{av}}$ shall be reduced if the load is not uniformly distributed on the area $A_{\text{av}}$ or if high shear forces are existent.

P(3) The justification of anchorages for prestressing tendons shall be checked by means of suitable strut-and-tie models.

P(4) The transverse tension forces in the loaded area shall be taken by supplementary reinforcement (links or layers of reinforcement bent in the shape of hair pins)

6.8 Fatigue

6.8.1 Verification conditions

P(1) The resistance of structures to fatigue shall be verified. This verification shall be performed separately for concrete and steel.

(2) A fatigue verification is necessary for structures and structural components which are subjected to regular load cycles (e.g. crane-rails, bridges exposed to high traffic loads).

6.8.2 Internal forces and stresses for fatigue verification

P(1) The stress calculation shall be based on the assumption of cracked cross sections neglecting the tensile strength of concrete but satisfying compatibility of strains.

P(2) The effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account by increasing the conventional steel stress in the reinforcing steel calculated under the assumption of perfect bond by the factor:

$$\eta = \frac{A_s + A_p}{A_s + A_p \sqrt[3]{\phi / \phi_r}}$$  \hfill (6.40)

where:

- $A_s$ area of reinforcing steel
- $A_p$ area of prestressing tendon or tendons
- $\phi$ largest diameter of reinforcement
- $\phi_r$ diameter or equivalent diameter of prestressing steel
- $\phi_r = 1.6 \sqrt{A_p}$ for bundles
- $\phi_r = 1.75 \phi_{\text{ave}}$ for single strands (7 wires)
- $\phi_r = 1.20 \phi_{\text{ave}}$ for single strands (3 wires)
- $\xi$ ratio of bond strength between bonded tendons and ribbed steel in concrete (Table 6.1)

(3) When using the standard method for fatigue strength of the shear reinforcement the contribution $V_{\text{con},\theta}$ of the concrete to $V_{\text{con}}$ should be taken not greater than

---

\footnote{If the standard method will not exist any longer in the future EC2, this paragraph may be deleted}
Table 6.1: Ratio of bond strength between tendons and reinforcing steel

<table>
<thead>
<tr>
<th>Prestressing steel</th>
<th>Pre-tensioned</th>
<th>Post-tensioned</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth bars or strands</td>
<td>-</td>
<td>0.3</td>
</tr>
<tr>
<td>Strands</td>
<td>0.6</td>
<td>0.5</td>
</tr>
<tr>
<td>Profiled wires</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Ribbed bars</td>
<td>0.8</td>
<td>0.7</td>
</tr>
</tbody>
</table>

(4) When action effects due to shear are determined using the variable strut inclination method, the inclination of the compressive struts $\theta_{\text{str}}$ may be taken according to Expression 6.41

$$\tan \theta_{\text{str}} = \sqrt{\tan \theta} \leq 1.0$$  \hspace{1cm} (6.41)

where:
- $\theta$ is the angle of concrete compression struts to the beam axis assumed in ULS design (see Chapter 6.2)

6.8.3 Combination of actions

P(1) Fatigue verification for steel and concrete shall be performed taking into account the effects of the following combination of actions:
- permanent actions
- characteristic value of the prestressing force
- settlements (mean value)
- frequent value of temperature if unfavourable
- relevant fatigue traffic load

6.8.4 Verification procedure for reinforcing steel and prestressing steel

P(1) It shall be proved that the fatigue damage factor $D_{\text{fat}}$ of steel caused by the relevant fatigue loads satisfies the condition $D_{\text{fat}} \leq 1$. For the calculation of $D_{\text{fat}}$ the Palmgren-Miner Rule applies. The damage can be determined by using the corresponding S-N curves (Figure 6.22) for reinforcing and prestressing steel and Table 6.2 or 6.3. The values obtained shall be divided by the safety factor $\lambda_{\text{fat}}$. The values given in Table 6.2 and 6.3 apply for reinforcing and prestressing steel according to 3.2 and 3.3 and for other steel unless otherwise determined in the approval documents.

Draft Note: $\lambda_{\text{fat}}$ shall be determined in Chapter 2!

P(2) For the design of members in non-corrosive environments (Table 4.1 X0, XC1) the S-N curves of reinforcing steel shall be determined by the values given in Table 6.3. For
members in corrosive environments (Table 4.1 XC2-4, XD1-3, XS1-3) the value for the stress exponent \( k_2 \) for straight and bent reinforcing bars given in Table 6.3 shall be reduced to \( k_2 = 5 \).

P(3) The stress range of welded bars shall be reduced to \( \Delta \sigma_{\text{req}} = 300 \text{ N/mm}^2 \).

### Table 6.2: Parameters for S-N curves of prestressing steel

<table>
<thead>
<tr>
<th>S-N-curve of prestressing steel used for</th>
<th>( N^* )</th>
<th>stress exponent</th>
<th>( \Delta \sigma_{\text{req}} ) (N/mm(^2)) at ( N^* ) cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>pretensioning</td>
<td>( 10^6 )</td>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>post-tensioning</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- single strands in plastic ducts</td>
<td>( 10^6 )</td>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>- straight tendons or curved tendons in plastic ducts</td>
<td>( 10^6 )</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>- curved tendons in steel ducts</td>
<td>( 10^6 )</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>- splicing devices(^1)</td>
<td>( 10^6 )</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

\(^1\) unless other S-N-curves can be justified by test results

### Table 6.3: Parameters of S-N curves for reinforcing steel

<table>
<thead>
<tr>
<th>Type of reinforcement</th>
<th>( N^* )</th>
<th>stress exponent</th>
<th>( \Delta \sigma_{\text{req}} ) (N/mm(^2)) at ( N^* ) cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight and bent bars(^1)</td>
<td>( 10^6 )</td>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>Welded bars (^2)</td>
<td>( 10^7 )</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Splicing devices (^2)</td>
<td>( 10^7 )</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>

\(^1\) Values for \( \Delta \sigma_{\text{req}} \) are those of the appropriate straight bar and should be obtained by multiplying the straight bar values by a reduction factor \( \zeta = 0.35 + 0.026 \cdot D / \phi \). For stirrups \( \zeta \) should be taken as 0.9.

\(^2\) unless other S-N-curves can be justified by test results
6.8.5 Verification using damage equivalent stress

Using an explicit verification of the operational strength according to 6.8.4 the fatigue verification of standard cases with known loads (railway and road bridges) may also be performed as follows:

- by damage equivalent stress ranges for steel according to (2)
- damage equivalent compression stresses for concrete according to (3)

For reinforcing or prestressing steel and splicing devices adequate fatigue resistance shall be assumed if the following expression is satisfied:

\[
Y_F \cdot Y_{sl} \cdot \Delta \sigma_{s,eq}(N^*) \leq \frac{\Delta \sigma_{Ral}(N^*)}{Y_{sl}}
\]

where:

\(\Delta \sigma_{s,eq}(N^*)\) is the stress range at \(N^*\) cycles from the appropriate S-N-curves given in Figure 6.22 and Table 6.2 and 6.3.

\(\Delta \sigma_{Ral}(N^*)\) is the damage equivalent stress range for different types of reinforcement and considering the number of loading cycles \(N^*\).

A satisfying fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

\[
14 \frac{1 - S_{ed, max, eqv}}{\sqrt{1 - R_{eqv}}} \geq 6
\]

where:

\(R_{eqv} = \frac{S_{ed, max, eqv}}{S_{ed, max}}\)


- 39 -
\[ S_{ed, min} = \frac{\sigma_{ed, min}}{f_{ed, fu}} \]  
(6.45)

\[ S_{ed, max} = \frac{\sigma_{ed, max}}{f_{ed, fu}} \]  
(6.46)

where:
- \( f_{ed, fu} \) is the design fatigue strength of concrete according to expression (6.48)
- \( \sigma_{ed, max, eq} \) upper stress of the ultimate amplitude for \( N = 10^6 \) cycles
- \( \sigma_{ed, min, eq} \) lower stress of the ultimate amplitude for \( N = 10^6 \) cycles

(4) Special standards give relevant fatigue loading models and procedures for the calculation of the equivalent stress range \( \Delta \sigma_{eq} \) for superstructures of road and railway bridges.

6.8.6 Simplified verifications

P(1) Simplified verifications shall be performed with actions according to Chapter 2.4 and EN 1990.

(2) Adequate fatigue resistance may be assumed for not-welded reinforcing bars under tension, if the stress range under frequent load combination is:
\[ \Delta \sigma \leq 70 \text{ MPa} \]  
For welded reinforcing bars under tension adequate fatigue resistance may be assumed if the stress range under frequent load combination is:
\[ \Delta \sigma \leq 35 \text{ MPa} \]

(3) The verification of prestressing and reinforcing steel with welded joints and splicing devices is performed under the following conditions: Under the frequent load combination and under consideration of a reducing factor 0.9 for the mean value of the prestressing force \( P_{ml} \), only compression stresses at all fibres should exist.

(4) The fatigue verification for concrete under compression is performed, if the following condition is satisfied:

\[ \frac{|\sigma_{C, max}|}{f_{ed, fu}} \leq 0.5 + 0.45 \frac{|\sigma_{C, min}|}{f_{ed, fu}} \leq 0.9 \quad f_{c} \leq 50 \text{ N/mm}^2 \quad f_{c} \geq 55 \text{ N/mm}^2 \]  
(6.47)

where:
- \( |\sigma_{C, max}| \) is the maximum compressive stress at a fibre under the frequent combination
- \( |\sigma_{C, min}| \) is the minimum compressive strength at the same fibre where \( \sigma_{C, max} \) occurs. If \( \sigma_{C, min} > 0 \) (tension), then \( \sigma_{C, min} = 0 \).
\[ f_{cd,cr} = 0.85 \beta_{cr} (t_0) f_{cd} \left( 1 - \frac{f_{cr}}{250} \right) \]  \hspace{1cm} (6.48)

- \beta_{cr} \quad \text{coefficient for concrete strength at first load application}
- \beta_{cr}(t_0) = 10^{0.01(t_0 - \ln t_0)}
- t_1 \quad \text{reference time: } t_1 = 1 \text{ day}
- t_0 \quad \text{time of first loading of concrete (in days)}

**(P5)** Expression 6.47 also applies to the compression struts of members subjected to shear. In this case the concrete strength \( f_{cd,cr} \) shall be reduced by the effectiveness factor \( \nu \) (see 6.2).

**(6)** In members without shear reinforcement, a fatigue verification for concrete under shear need not be performed if the following conditions are met:

\[ \frac{V_{\text{min}}}{V_{\text{max}}} \geq 0; \quad \frac{V_{\text{min}}}{V_{\text{cr}}} \leq 0,5 + 0,45 \frac{V_{\text{min}}}{V_{\text{cr}}} \quad \text{for} \quad \frac{V_{\text{min}}}{V_{\text{max}}} \leq 0,9 \quad f_c \leq 50 \text{ N/mm}^2 \]

\[ \frac{V_{\text{min}}}{V_{\text{max}}} < 0; \quad \frac{V_{\text{min}}}{V_{\text{cr}}} \leq 0,5 - \frac{V_{\text{min}}}{V_{\text{cr}}} \quad \text{for} \quad \frac{V_{\text{min}}}{V_{\text{max}}} \leq 0,8 \quad f_c \geq 55 \text{ N/mm}^2 \]  \hspace{1cm} (6.49)

\[ \frac{V_{\text{min}}}{V_{\text{max}}} < 0; \quad \frac{V_{\text{min}}}{V_{\text{cr}}} \leq 0,5 - \frac{V_{\text{min}}}{V_{\text{cr}}} \]  \hspace{1cm} (6.50)

where:
- \( V_{\text{max}} \) maximum nominal shear force under frequent combination of actions
- \( V_{\text{min}} \) minimum nominal shear force under frequent combination of actions at the section where \( V_{\text{max}} \) occurs
- \( V_{\text{cr}} \) design shear resistance according to 6.2


Chapters/sections:

- 4.3.2 : Shear, sections 4.3.2.0 – 4.3.2.4. Pages 118-123. [EC2-1 (1991)].

- 4.3.7 : Verification of Fatigue, sections 4.3.7.1 – 4.3.7.4. Pages 18-21. [EC2-2 (1995)].

- A106.3.2: Concrete subjected to compression. Page 44. [EC2-2 (1995)].
4.3.2 SHEAR

4.3.2.0 Notation (See also 1.6 and 1.7)

- $A_{sf}$: Area of reinforcement across the flange of a flanged beam
- $A_{sl}$: Area of tension reinforcement effective at a section
- $F_C$: Compressive force in the concrete in the direction of the longitudinal axis
- $a_{F_d}$: Variation of the longitudinal force acting in a section of flange within the distance $a_v$ [See 4.3.2.5 (3)]
- $F_s$: Tensile force in longitudinal reinforcement
- $V_{cdd}$: Force component in the compression zone, parallel to $V_{od}$, of elements with variable depth
- $V_{cd}$: Shear capacity of the concrete compression zone
- $V_{od}$: Design shear force in the section, uncorrected for effects of variable section depth
- $V_{pd}$: Force component due to inclined prestressing tendons
- $V_{rd1}$: Design shear resistance of a section in elements without shear reinforcement
- $V_{rd2}$: Maximum design shear force that can be carried without web failure
- $V_{rd2\text{, red.}}$: Reduced value of $V_{rd2}$, due to axial force
- $V_{rd3}$: Design shear resistance of a section, in elements with shear reinforcement
- $V_{td}$: Force component in the tensile zone, parallel to $V_{od}$, in elements with variable depth
- $V_{wd}$: Contribution of shear reinforcement
- $a_v$: Distance between points of zero and maximum moment
- $b_{w\text{, nom}}$: Nominal web thickness
- $f_{ywd}$: Design yield strength of shear reinforcement
- $h_f$: Flange depth
- $k$: A constant relating to section depth and curtailment
- $s_f$: Spacing of reinforcing bars across the flange of flanged beams
- $a$: Angle of the shear reinforcement to the longitudinal axis of a member
$S$  Shear force enhancement coefficient  
$\theta$  Angle of the concrete struts with the longitudinal axis of the member  
$\psi$  Efficiency factor  
$\rho_l$  Reinforcement ratio corresponding to $A_{sl}$  
$\sigma_{cp}$  Average stress in concrete due to axial force  
$\sigma_{cp,\text{eff}}$  Effective average stress in concrete due to axial force  
$\tau_{rd}$  Basic design shear strength of members without shear reinforcement  
$2\sigma$  Sum of diameter of prestressing ducts at a given level

4.3.2.1 General

P(1) This section applies to beams and slabs designed for flexure in accordance with 4.3.1. It also applies to prestressed elements and columns subjected to significant shear forces designed in accordance with 4.3.1 and 4.3.3.

P(2) In general, a minimum amount of shear reinforcement shall be provided, even where calculation shows that shear reinforcement is unnecessary. This minimum may be omitted in elements such as slabs, (solid, ribbed, hollow), having adequate provision for the transverse distribution of loads, where these are not subjected to significant tensile forces. Minimum shear reinforcement may also be omitted in members of minor importance which do not contribute significantly to the overall strength and stability of the structure.

(3) Rules for minimum shear reinforcement are given in 5.4. An example of a member of minor importance would be a lintel of less than 2 m span.

P(4) In structures of variable depth, the design shear forces shall be modified by a contribution corresponding to the components of the compressive and tensile resultant perpendicular to the member axis.

P(5) In prestressed structures, in the calculation of $\psi_{gd}$ account shall be taken of the effect of inclined prestressing tendons.

P(6) When determining the necessary longitudinal reinforcement in areas subjected to shear, account shall be taken of the possible increase of the tensile force beyond the value corresponding to the bending moment.

(7) This increase is covered by the 'shift' rules given in Section 5.4.2.1.

4.3.2.2 Design method for shear

(1) The method for shear design, set out in the following sections, is based on three values of design shear resistance:
- \( V_{Rd1} \) - the design shear resistance of the member without shear reinforcement. (See 4.3.2.3).

- \( V_{Rd2} \) - the maximum design shear force that can be carried without crushing of the notional concrete compressive struts. (See 4.3.2.3, 4.3.2.4.3, 4.3.2.4.4).

- \( V_{Rd3} \) - the design shear force that can be carried by a member with shear reinforcement. (See 4.3.2.4.3, 4.3.2.4.4).

(2) Any section for which the design shear, \( V_{gd} \), is less than \( V_{Rd1} \), requires no design shear reinforcement but except in the cases defined in 4.3.2.1 F(2) and (3), minimum shear reinforcement should be provided in accordance with 3.4.

(3) For sections where \( V_{gd} \) exceeds \( V_{Rd1} \), shear reinforcement should be provided such that:

\[
V_{gd} \leq V_{Rd3}
\]

The amount of shear reinforcement should not be less than the minimum given in 3.4.2.2.

(4) In the absence of more rigorous analysis, at no section in any element should the design shear force exceed \( V_{Rd2} \). (See 4.3.2.3). Where the member is subjected to an applied axial compression, \( V_{Rd2} \) should be reduced in accordance with Equation (4.15) below.

\[
V_{Rd2,\text{red}} = 1.67 V_{Rd2} \left(1 - \frac{\sigma_{cp,\text{eff}}}{f_{cd}}\right) \leq V_{Rd2}
\]  

where:

- \( V_{Rd2,\text{red}} \) is the reduced value of \( V_{Rd2} \)
- \( \sigma_{cp,\text{eff}} \) is the effective average stress in the concrete due to axial force. \( \sigma_{cp,\text{eff}} \) is given by Equation (4.16) below.

\[
\sigma_{cp,\text{eff}} = \left(N_{gd} - f_{yk} A_{s2}/f_{y}\right)/A_c
\]

where:

- \( N_{gd} \) is the design axial force
- \( A_{s2} \) is the area of reinforcement in the compression zone at the ultimate limit state
- \( f_{yk} \) is the yield strength of the compression steel. (\( f_{yk}/f_y \) should not exceed 400 N/mm\(^2\))
- \( A_c \) is the total area of the concrete cross-section.

(5) Close to supports where the configuration of concentrated loads and support reaction is such that a proportion of the loads may be carried to the support by direct compression (direct support). an allowance may be made for an enhancement of the shear resistance \( V_{Rd1} \) (see (9) below). Any such enhancement of \( V_{Rd1} \) should be ignored when checking \( V_{Rd2} \).
(6) The attainment of $V_{Ed1}$ depends significantly on the proper anchorage of the tension reinforcement or prestressing tendons on each side of any possible plane of failure. Rules are provided to ensure this, in Chapter 5.

(7) For cases where $V_{Ed} > V_{Ed1}$, two design methods are given in the following clauses:

- the standard method (4.3.2.4.3) and
- the variable truss angle method (4.3.2.4.4).

The variable truss angle method allows more freedom in the arrangement of reinforcement than the standard method. It will frequently lead to substantial economies in shear reinforcement but may require increases in the longitudinal tension steel.

It should be used when a member is subjected to combined shear and torsion.

(8) If the web contains grouted ducts with a diameter $d > b_w/8$ the shear resistance $V_{Ed2}$ should be calculated on the basis of a nominal web thickness given by:

$$b_{w, nom} = \frac{b_w - 1/2 L}{\pi}$$

where $L$ is determined for the most unfavourable level.

(9) For members without shear reinforcement, and for members with shear reinforcement where the Standard Method of shear design is used (4.3.2.4.3) and where the conditions set out in (11) below are satisfied, an enhancement of shear resistance, only for concentrated loads situated at a distance $x \leq 2.5$ d from the face of the support, is permitted [(5) above]. Solely for this purpose, the value $\tau_{Ed}$ in Equation (4.18) may be multiplied by a factor $\beta$, when estimating $V_{Ed1}$, where:

$$\beta = 2.5d/x \quad \text{with} \quad 1.0 \leq \beta \leq 5.0$$  \hspace{1cm} (4.17)

When this enhancement is taken into account, $V_{Ed1}$ and shear reinforcement should be calculated at all critical sections over the length $2.5$ d from the face of the support, with $\beta = 1.0$ on the span side of the relevant concentrated loads; the maximum shear reinforcement so obtained should be provided over this entire length.

Where the dominant load on a beam is a concentrated load close to a support, the above procedure may lead to minimum reinforcement throughout the beam. In these cases, care is required, and the designer may wish to base the resistance on the unenhanced $V_{Ed1}$.

(10) Because of the increased resistance due to direct transmission of loads close to supports, it will normally be conservative to evaluate $V_{Ed}$ at a distance $d$ from the face of a direct support on beams or slabs with continuously distributed loading.
(11) When taking account of the increased shear strength close to the supports in (9) or (10) above, the following conditions should be satisfied.

(a) the loading and support reactions are such that they cause diagonal compression in the element (direct support).

(b) at an end support, the whole tension reinforcement required within a distance of 2.5 \( d \) from the support should be anchored into the support.

(c) at an intermediate support the tension reinforcement required at the face of the support should continue for at least 2.5 \( d + \frac{l_{b,\text{net}}}{2} \) into the span.

4.3.2.3 Elements not requiring design shear reinforcement \((V_{\text{ed}} \leq V_{\text{Ed1}})\)

(1) The design shear resistance \(V_{\text{Ed1}}\) is given by:

\[
V_{\text{Ed1}} = \left[ p_{Rd} k (1.2 + 40 \rho_{l}) + 0.15 \sigma_{cp} b_{w} d \right] d_{c}
\]

where

\[
p_{Rd} = \text{basic design shear strength} = \frac{0.25 f_{c,\text{tk}} 0.03}{f_{c,\text{c}}}.
\]

\(\gamma_{c}\) should be taken as \([1.25]\). Values of \(p_{Rd}\) are given in Table 4.8.

\[
k = \begin{cases} 1 & \text{for members where more than 50% of the bottom reinforcement is curtailed.} \\ 1.6 - d < 1 & \text{(d in metres)} \\ 0.02 & \end{cases}
\]

A
d Somali

\[
\sigma_{cp} = \frac{N_{sd}}{A_{c}}
\]

\(N_{sd}\) = longitudinal force in section due to loading or prestressing (compression positive).

### Table 4.8: Values for \(\gamma_{Rd}\) (N/mm²) with \(\gamma_{c} = 1.5\) for different concrete strengths

<table>
<thead>
<tr>
<th>(f_{c,\text{tk}})</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\gamma_{Rd})</td>
<td>0.18</td>
<td>0.22</td>
<td>0.26</td>
<td>0.30</td>
<td>0.34</td>
<td>0.37</td>
<td>0.41</td>
<td>0.44</td>
<td>0.48</td>
</tr>
</tbody>
</table>

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Figure 4.12: Definition of \( A_s \) for use in Equation (4.18)

(2) Equation (4.18) only applies to the anchorage zones of pretensioned members where the anchorage requirements of 4.2.3.5.6 are satisfied.

(3) When checking sections without designed shear reinforcement, the design resistance \( V_{Rd2} \) is given by:

\[
V_{Rd2} = \frac{1}{3} V_{cd} b w \cdot 0.9 d
\]

where

\[
v = 0.7 - \frac{f_{ck}}{200} \leq 0.5 \quad \text{(fck in N/mm)}
\]

4.3.2.4 Elements requiring design shear reinforcement \((V_{sd} > V_{Rd1})\)

4.3.2.4.1 General

P(1) In beams, bent-up bars shall not be used as shear reinforcement except in combination with stirrups. At least 50% of \( V_{sd} \) shall be resisted by vertical stirrups.

P(2) Where inclined shear reinforcement is used, the angle between the reinforcement and the longitudinal axis of the beam should not be less than 45°.

P(3) Where the load is not acting at the top of the beam, or where the support is not at the bottom of the beam, horizontal reinforcement should be provided to transfer the load to the top of the design truss system.

4.3.2.4.2 Members with constant depth

P(1) For shear design, the member is assumed to consist of compressive and tensile zones separated by a distance equal to the internal lever arm \( z \). The shear zone has a depth equal to \( z \) and width \( b_w \). The internal lever arm is calculated perpendicular to the longitudinal reinforcement by ignoring the effect of any bent-up longitudinal reinforcement.
4.3.6.2 Measures

(101) In the absence of national rules, the following ones are applicable.
(102) Priority should be given to preventive measures which avoid or reduce the risk of impact.
(103) The effectiveness of shielding structures placed in front of the load bearing structure should be verified.
(104) Expendable members should lose their ability to transmit considerable horizontal forces after failure under impact.
(105) The deformations of the structure after failure of an expendable structural member should be checked to ensure clearance under the deformed structure.
(106) If the danger of impact cannot be eliminated and if a design with expendable members is not possible, the structural safety should be ensured by designing the structural members to have adequate resistance.

4.3.6.3 Detailing

P(101) In the absence of specific dynamic investigations, the impacted structural members shall be designed such that they can resist also an impact in the opposite direction.

P(102) In all cases, where plastic deformations of the impacted structural members are considered to absorb a substantial proportion of the kinetic energy, the ultimate strains shall be checked. The reinforcement shall not be welded in this case.

(103) The concrete cover outside the outer longitudinal reinforcement should not be taken into account for the determination of the ultimate capacity of the collision zones. Compression reinforcement can only be considered in the design when restrained against buckling (see 5.4.1.2.2 in ENV 1992-1-1).

(104) Appropriate stirrup reinforcement with small spacing should be provided. For very high impact forces even steel encasement to a collision zone may be required.

Insert new subclause after 4.3.6:

4.3.7 VERIFICATION OF FATIGUE

4.3.7.1 Verification conditions

P(101) Structural members which are subjected to a significant stress variation shall be designed for fatigue. In this case, the verification shall be performed separately for concrete and steel.

(102) A fatigue verification is generally not necessary for structures and structural components such as:

a) footbridges;
b) buried arch and frame structures with a minimum earth cover of 1.0 m for road bridges and 1.5 m for railway bridges;
c) foundations;
d) piers and columns, which are not rigidly connected to superstructures;
e) retaining walls of road bridges;[1];
f) abutments of road bridges] which are not rigidly connected to superstructures (except the slabs and walls of hollow abutments);
g) concrete in compression for road bridges if 4.4.1.1 (103) in this Part 2 is satisfied;
h) steel and prestressed reinforcement without welded connections or couplers for bridges designed according to category A, B, and C of Table 4.118 of this Part 2;

[1] to be checked for railway bridges
il prestressing and reinforcing steel with welded connections or couplers in those regions where, under the frequent combination of actions with a reduction factor of 0.85 applied to the characteristic value of the prestressing force, \( P_v \), only compression stresses occur in the extreme fibres.

4.3.7.2 Combination of actions and partial safety factors for fatigue verification

(101) The partial safety factors for load and model uncertainties of action effects should be taken as

\[
\gamma_F = 1.0 \quad \text{and} \quad \gamma_F = 1.0
\]

(4.186)

(102) Partial safety factors for material properties are given in Table 4.115.

Table 4.115: Partial safety factors for material properties for fatigue verification

<table>
<thead>
<tr>
<th>Verification for</th>
<th>Concrete</th>
<th>Steel Reinforcement, Tendons</th>
</tr>
</thead>
<tbody>
<tr>
<td>safety factor</td>
<td>([1.5])</td>
<td>([1.15])</td>
</tr>
</tbody>
</table>

P(103) In general, fatigue verification for steel and concrete shall be performed taking into account the effects of the following combination of actions:

- permanent actions
- characteristic value of the prestressing force (see 2.5.4.2)
- most unfavourable value of settlements (appropriate estimate values)
- most unfavourable frequent value of temperature
- relevant fatigue traffic load model (see ENV 1991-3 and Appendix 106)
- where relevant, wind fluctuations.

However, when using the methods given in 4.3.7.4 for concrete, the frequent combination of actions should be applied.

In the absence of a more refined verification method in coupling joints the characteristic value of the prestressing force shall be reduced by a factor of 0.85.

4.3.7.3 Internal forces and stresses for fatigue verification

P(101) The stress calculation shall be based on the assumption of cracked cross sections neglecting the tensile strength of concrete but satisfying compatibility of strains (plane sections remain planar).

(102) Internal forces may be calculated using linear elastic models throughout the structural elements considered. In cracked zones a reduced stiffness may be taken into account.

(103) For the calculation of internal forces according to (102) above and stresses (see P(101) above), the modular ratio may be taken as \( \alpha = 10 \).

P(104) For cracked cross sections subjected to fatigue the effect of the different bond behaviour of prestressing and reinforcing steel shall be taken into account, see 4.3.7.6.

P(105) For structural members with shear reinforcement, the determination of the forces in the reinforcement and in the concrete shall be carried out using the truss model.

(106) When using the standard method for fatigue verification of the shear reinforcement the contribution \( V_{sen} \) of the concrete to \( V_{sen} \) should be taken not greater than 0.5 \( V_{sen} \).

(107) When action effects due to shear are determined using the variable strut inclination method, the inclination of the compression struts \( \theta_{incl} \) may be taken according to Equation (4.187):

\[
\tan \theta_{incl} = \sqrt{\tan \theta} \leq 1.0
\]

(4.187)

where:

\( \theta \) is the angle of concrete compression struts to the beam axis assumed in the design for shear at the ultimate limit state according to 4.3.2.4 in ENV 1992-1-1.
4.3.7.4 Fatigue verification for concrete under compression, shear and punching shear

(101) For concrete under compression adequate fatigue resistance may be assumed if Equation (4.188) shown graphically in Figure 4.134 is satisfied. Otherwise, a more refined fatigue verification may be necessary (see Appendix 108 for railway bridges).

\[
\frac{\sigma_{\text{nom}}}{f_{ck}} \leq 0,5 + 0,45 \cdot \frac{\sigma_{\text{nom}}}{f_{ck}} \leq 0,9
\]

(4.188)

where:
- \(\sigma_{\text{c,max}}\) is the maximum compressive stress at a fibre under the frequent combination of actions.
- \(\sigma_{\text{c,min}}\) is the minimum compressive stress at the same fibre where \(\sigma_{\text{c,max}}\) occurs.

If \(\sigma_{\text{c,min}} < 0\) (tension) then \(\sigma_{\text{c,max}}/f_{ck} \leq 0,5\) should be fulfilled.

The increase of the reference compressive strength with the age of the concrete at the time \(t_0\) before the cyclic loading occurs, may be taken into account by applying the factor \(\varepsilon(t_0)\) to the design concrete strength \(f_{cd}\). For \(\varepsilon(t_0)\) see 4.4.3.2 (102) in this Part 2.

a) allowable region

![Figure 4.134: Allowable stress variation for concrete under compression according to Equation (4.188) without an explicit fatigue verification](image)

(102) Equation (4.188) and Figure (4.134) apply also to the compression struts of members subjected to shear. In this case the reference strength \(f_{cd}\) should be reduced by the effectiveness factor \(\alpha\) given by Equation (4.21) in ENV 1992-1-1.

(103) In members without shear reinforcement, adequate fatigue resistance of concrete under shear may be assumed if either Equation (4.189) or (4.190), illustrated graphically in Figure 4.135, is satisfied. Otherwise, a more refined fatigue verification may be necessary.

\[
\frac{f_{\text{nom}}}{f_{\text{nom}}/f_{\text{cd}}} \leq 0,5 + 0,45 \cdot \frac{f_{\text{nom}}}{f_{\text{cd}}} \leq 0,9
\]

(4.189)
for \( r_{\text{min}} < 0 \): \( \frac{r_{\text{min}}}{r_{\text{max}}} \leq 0.5 \), \( \frac{r_{\text{min}}}{r_{\text{x}}(s)} \)

where:

- \( r_{\text{max}} \) is the maximum nominal shear stress under frequent combination of actions.
- \( r_{\text{min}} \) is the minimum nominal shear stress under frequent combination of actions at the section where \( r_{\text{min}} \) occurs.
- \( r_{\text{c-x}} = \frac{V_{\text{d-x}}}{l b_{\text{yd}}} \) with the design shear resistance \( V_{\text{d-x}} \), according to Equation (4.18) in ENV 1992-1-1.

Figure 4.135: Allowable shear stress variation for members without shear reinforcement according to Equations (4.189) and (4.190)

(104) In the case of punching shear, the maximum and minimum design shear stresses should satisfy Equations (4.189) and (4.190) respectively.

For the calculation of the design shear stresses clause 4.3.4.2 of ENV 1992-1-1 applies, i.e. 
\( r = \frac{V_{\text{d-x}}}{V_{\text{x}}(s)} \) and \( r_{\text{c-x}} = \frac{V_{\text{d-x}}}{V_{\text{x}}(z)} \).

4.3.7.6 Fatigue verification for prestressing and reinforcing steel

(101) For unwelded reinforcing bars subjected to tension, adequate fatigue resistance may be assumed if, under the frequent combination of actions, the stress variation, \( \Delta a_{\text{t-w}} \), does not exceed \( 70 \) N/mm².

(102) For reinforcing or prestressing steel and couplers adequate fatigue resistance may be assumed if the following expression is satisfied:

\[
\gamma_F \cdot \gamma_{50} = \frac{\delta a_{\text{res}}(N)}{\gamma_{1-5}}
\]

(4.191)

where:

- \( \Delta a_{\text{res}}(N) \) is the stress range at \( N \) cycles from the appropriate S-N lines given in 4.3.7.7 or 4.3.7.8 of this Part 2.
- \( \Delta a_{\text{t-w}} \) is the damage equivalent stress range defined as the stress range of a constant stress spectrum with \( N \) stress cycles which results in the same damage as the spectrum of stress ranges caused by flowing traffic loads.

(103) For decks of road and railway bridges the damage equivalent stress range \( \Delta a_{\text{t-w}} \) may be calculated by the procedure given in Appendix 106.
A106.3.2 CONCRETE SUBJECTED TO COMPRESSION

(101) For concrete subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

\[
14 \cdot \frac{1 - \sigma_{\text{cd,max},\text{eq}}}{{\sigma_{\text{cd,min},\text{eq}}}} \leq 6
\]

where:

\[
\frac{\sigma_{\text{cd}}}{\sigma_{\text{cd,max},\text{eq}}} = \frac{\sigma_{\text{cd}}}{\sigma_{\text{cd,max},\text{eq}}} \quad ; \quad \sigma_{\text{cd,min},\text{eq}} = \sigma_{\text{d}} \cdot \frac{\sigma_{\text{cd,min},\text{eq}}}{\sigma_{\text{d}}} \quad ; \quad \sigma_{\text{cd,min},\text{eq}} = \sigma_{\text{d}} \cdot \frac{\sigma_{\text{cd,min},\text{eq}}}{\sigma_{\text{d}}}
\]

\( \sigma_{\text{cd, max}, \text{eq}} \) and \( \sigma_{\text{cd, min}, \text{eq}} \) are the upper and lower stresses of the damage equivalent stress range with a number of cycles \( N = 10^6 \).

(102) The upper and lower stresses of the damage equivalent stress range should be calculated according to Equation (A106.12):

\[
\sigma_{\text{cd, max}, \text{eq}} = \sigma_{\text{d, eq}} \cdot \lambda_2 \cdot (\sigma_{\text{d, max}, \text{eq}} - \sigma_{\text{d, min}, \text{eq}})
\]

\[
\sigma_{\text{cd, min}, \text{eq}} = \sigma_{\text{d, eq}} \cdot \lambda_2 \cdot (\sigma_{\text{d, min}, \text{eq}} - \sigma_{\text{d, max}, \text{eq}})
\]

where:

\( \sigma_{\text{d, eq}} \) is the compressive concrete stress under the infrequent combination of actions without load model 71.

\( \sigma_{\text{d, max}, \text{eq}} \) and \( \sigma_{\text{d, min}, \text{eq}} \) are the maximum and minimum compressive stress under the infrequent combination of actions, which includes the dynamic factor \( \lambda_2 \) according to ENV 1991-3.

\( \lambda_2 \) is the correction factor to calculate the upper and lower stresses of the damage equivalent stress range from the stresses caused by the load model 71. The values given in Table 107.3 are based on \( \lambda_1 = 1 \).
Excerpt from MC90 (1990)


Chapters/sections:

6.4.2.3: Shear in cracked zones without shear reinforcement. Pages 177 and 178.
where the effects are favourable and the higher ones where they are unfavourable.

The presence of prestressing ducts should be taken into account by subtracting from the web breadth the sum of the diameters of the ducts in one layer.

If the centroid of the cross-section does not lie within the web the principal tensile stress verification should be made at the intersection of the web and flange within which the centroid lies.

In pretensioned members account should be taken of the reduction of the effective prestress within the transmission lengths.

Where the centroid of the section is at mid-depth the reduction may be allowed for by making the calculations for a section distance $h/2$ from the inner edge of the support and taking the prestress at the centroid as

$$\sigma_c = \frac{P_{pl}}{A} \left(1 - \frac{s + 0.5h}{l_{tr}}\right)$$  \hspace{1cm} (6.4.5)

where

$P_{pl}$ is the design prestressing force under permanent loads
$A$ is the area of the section
$s$ is the distance from the end of the member to the inner edge of the supports
$l_{tr}$ is the transmission length.

6.4.2.3. Shear in cracked zones without shear reinforcement

The net shear forces acting on the concrete should be taken as

(a) for ordinary reinforced concrete members, the total shear $V_{sd} = V_d$
(b) for prestressed members, the shear corresponding to the part of the external loading system not balanced by the prestress system

$$V_{sd} = V_d - V_s$$  \hspace{1cm} (6.4.6)

where $V_s$ is the shear corresponding to $\lambda h_{ld}$ determined as in clause 6.3.3.3.3.

It is to be verified that

$$V_{sd} \leq V_{krit}$$  \hspace{1cm} (6.4.7)

The transverse shear resistance $V_{krit}$ may be described by means of an appropriate model.
Wherever a more precise analysis is not made, the following empirical expression may be used for members with parallel chords:

$$V_{sd} = 0.12\xi(100f_{ka})^{0.5} b_{net}d$$

(6.4-8)

where

$$\xi = 1 + \sqrt{\frac{200}{d}}$$

with $d$ in mm

$p$ is the ratio of bonded flexural tensile reinforcement $A_f b_{net}d$ or $(A_f + A_t) b_{net}d$ extending for a distance at least equal to $d$ beyond the section considered, except at end supports where the extension may be considered adequate if the length of bar beyond the centre-line of support is equal to at least 12 times the diameter.$b_{net}$ is the reduced web breadth equal to the full breadth minus the sum of the widths of tendon ducts situated within the web (note no deduction is necessary for ducts at the boundary of the web, i.e. at the level of the main tension reinforcement).

Fig. 6.4.3 illustrates indicative models of the actions resisting shear.

For high concrete strengths the relationship between the shear resistance of a member and the strength of the concrete depends upon the characteristics of the aggregate. If the aggregate fractures at flexural cracks, leaving smooth crack surfaces, the shear resistance may be below that given by eq. (6.4-8) unless $f_{ka}$ is restricted. Relevant experimental evidence may be provided by tests of beams of concrete with the intended value of $f_{ka}$ and the aggregate of the type to be used.

Unless relevant experimental evidence is available for the concrete in question, $f_{ka}$ should be limited to 50 MPa for the purpose of calculation according to eq. (6.4-8).

Except at simple supports, flexural tensile reinforcement should extend at least 0.6d beyond the section at which it is no longer required according to flexural calculations.

In the case of a member subjected to axial tensile loading the reinforcement required to resist this load should be discounted in the calculations of $p$.

Bonded prestressing tendons can be included in the calculation of $p$ but unbonded tendons should be excluded.

6.4.2.4. Shear in slabs with shear reinforcement

The verification of the resistance of a slab with shear reinforcement should
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