Repair, strengthening and upgrading of steel bridges in The Netherlands

Repair, Strengthening and Upgrading of Steel Bridges in The Netherlands

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Abstract

After World War II, many steel bridges were designed and built in The Netherlands. Many of these bridges are now of a substantial age and were designed for static and fatigue loading less severe than eventually present during their lifetime. Many of these bridges now show (fatigue) damage. Depending on the nature of the damage, these bridges can either be replaced, or a reinforcing substructure can be added, or the visible damage can be repaired and the bridge can be locally strengthened. In many cases, it has to be shown that the bridge is fit for purpose for future use, meaning that upgrading may be necessary. These bridges are now being reassessed and any possible conservatism in these assessments needs to be excluded to avoid unnecessary strengthening. The paper gives practical examples of repair, strengthening and upgrading techniques for steel bridges.

Keywords: steel; bridges; assessment; repair; strengthening; upgrading; fatigue; codes; standards.

1 Introduction

From the fifties to the eighties of the 20th century many steel bridges were designed and built in The Netherlands. Railways had to be reinstalled after World War II and the highway road network was developed. Many of the bridges built in that era are now of a substantial age. Modern bridges are designed differently than shortly after World War II [1]. At that time, traffic loads were moderate and also traffic intensity was much lower than nowadays. So these bridges were designed for static and fatigue loading less severe than eventually present during their lifetime, in some cases leading to (fatigue) damage. For that reason, these bridges are being reassessed. When reassessing existing bridges usually an inspection is carried out to see if damage has occurred. Damage of course needs to be repaired, often in such a way that the structure is also strengthened and the bridge is fit for purpose again for the years to come.

Section 2 further elaborates on the aspects inducing reassessment of existing bridges. In section 3 calculation aspects are treated briefly. Section 4 gives examples of repair, strengthening and upgrading of existing steel road bridges while section 5 does so for steel railway bridges. The conclusions are presented in section 6.

2 Aspects inducing reassessment

Reassessment of existing steel bridges is necessary if visible damage is present or due to new heavier bridge loads in design codes, which better represent the actual loads that bridges have to resist nowadays, due to increased traffic intensity and vehicle weights.

2.1 New heavier road bridge loads

2.1.1 Code provisions

In The Netherlands, most steel bridges constructed directly after World War II were designed for load configurations indicated by the Dutch design code VOSB1963 (Figure 1) and its predecessors. This code dates back to 1963 and is based on the actual traffic load of that time and shortly beyond. At this moment, the Eurocode EN 1991-2 prescribes a heavier traffic load, reflecting the increased traffic and axle loads (Figure 2). The traffic loads in the Eurocode EN 1991-2, Load Model 1, consist of a uniformly distributed load (UDL) and a tandem system (TS) and are considerably heavier than those in the code VOSB1963, Traffic Class 60. The axle loads increased by 50% and the UDL increased by 125% for the heaviest loaded lane. Taking into account

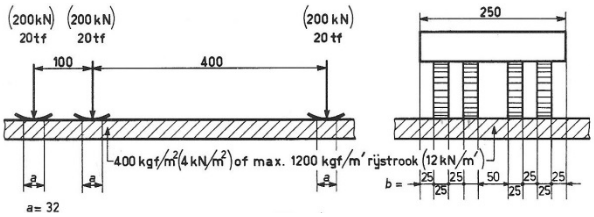


Figure 1. Traffic loads on bridges according to Dutch code VOSB1963, Traffic Class 60

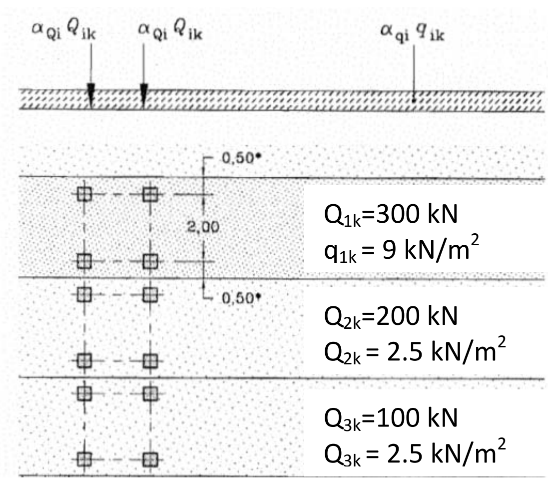


Figure 2. Traffic loads on bridges according to Eurocode EN1991-2, Load Model 1

that the code VOSB1963 would allow an 80% reduction on the UDL for multiple lane bridges, the increase in load level is even more pronounced. Reassessment for these increased loads almost certainly leads to bridge parts not fulfilling the requirements for static loading.

In the Netherlands there are supplementary codes for the assessment of existing structures in case of reconstruction: NEN 8700 (basic rules) and NEN 8701 (Actions). These codes should be read in conjunction with the Eurocode. In addition to these codes, a supplementary guideline from the Directorate-General for Public Works and Water Management (RWS) should be taken into account. The loads given in the Eurocode may be reduced because of:

- a shorter reference period than 100 years for the bridge's residual design working life;
- a trend reduction if the period under consideration falls before the year 2060;
- a large influence length, for UDL only.

In codes before 1963, fatigue is not an issue. In the code VOSB1963 specific fatigue loads are not given. Fatigue is covered by a reduced allowable stress.

2.1.2 More traffic lanes

Over the last decades not only the traffic loads increased but also the traffic intensity. There is also much heavier vehicle traffic on the highways these days than shortly after World War II. Therefore, many bridges over the years were equipped with more lanes by making the lanes narrower and by changing emergency lanes into normal lanes. This also has increased to loading on existing bridges and these bridges therefore need reassessment.

2.2 Upgrading the railway system

To upgrade the Dutch railway system for more and heavier train traffic, all the existing bridges in the main cargo lines were recalculated for so-called UIC D4 train traffic with axle loads of P = 225 kN (Figure 3) travelling at 100 km/h (the so-called D4 / V100 program).

These recalculations showed that many existing bridges are critical and therefore actions are

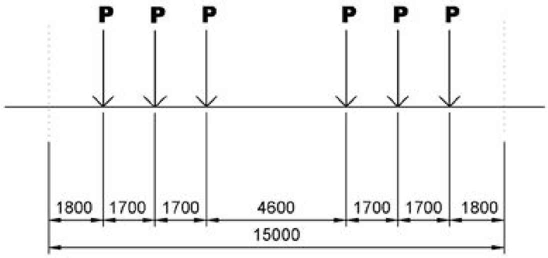


Figure 3. Train load configuration D4 / V100

necessary. These railway bridges were built with a traditional deck system and designed without considering the fatigue limit state. Fatigue problems occur directly beneath de rail in the connection between the longitudinal girder and the cross girder. This caused a lot of maintenance to be carried out on these connections. The other major bridge parts, as the cross girders themselves and the main girders, did not show fatigue problems because here the stress levels are lower.

3 Recalculation

If a recalculation is made of an existing steel bridge, special attention needs to be paid to local buckling, connections, fatigue and first of all the loading to be considered. Other than for new bridges, in case of existing bridges conservatism in the loading, the design rules and calculation methods should be avoided in order to keep the required strengthening measures to a minimum.

3.1 Project dedicated bridge loading

For specific bridge projects, the load can be further lowered. As an example, for the Merwede Bridge [2], the Netherlands Organisation for Applied Scientific Research (TNO) calculated an additional reduction factor for the static traffic load, based on measurements of freight traffic across the comparable Moerdijk Bridge. Based on a probabilistic calculation, the UDL has been recalculated for an influence length of 170 meters, leading to a reduction factor 0.85.

A step further was made for the Hagestein Bridge, where TNO determined load models specifically for the left main girder based on current and anticipated future loads, Figure 4. The specifically calculated UDL for the Hagestein Bridge on lane 3 is about 68% of the UDL on this lane according to

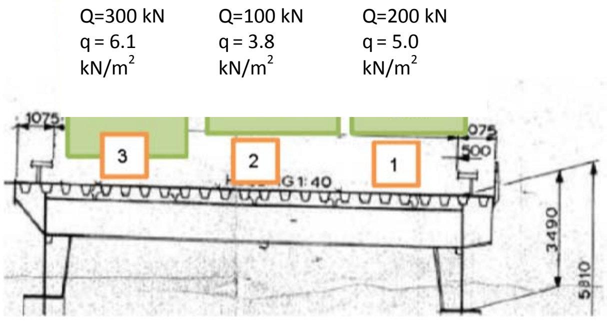


Figure 4. Specific traffic load calculated for the Hagestein Bridge

EN 1991-2. However, the specifically calculated UDL's for the Hagestein Bridge on the lanes 1 and 2 are substantially greater than those according to EN 1991-2, up to two times. Nevertheless, the specific traffic load of Figure 4 is more favorable for the left main girder than the traffic load EN 1991-2 since less load is distributed transversely to the left main girder.

3.2 Local buckling

It is more favorable to check local buckling with the effective width method than with the reduced stress method. Therefore, the effective width method of EN 1993-1-5 is used together with design checks of the resulting effective cross-sections in accordance with EN 1993-1-1. For the Hagestein Bridge, the main girders were checked for plate buckling at 31 cross-sections per main girder. In these evaluations, the normal forces, bending moments and shear forces per cross-section were determined for the normative load combination, and the stresses occurring as a result of these forces were calculated as input for the effective width calculations. For the global analysis, a model with plate elements for the deck and beam elements for the girders underneath was chosen. This model is called 'global beam model'. Using a global beam model has advantages over using a model completely consisting of plate elements, a so-called 'global plate model', since the global beam model well describes the global force distribution. If a global plate model is used, the plate buckling calculation can be disturbed by local effects such as peak stresses in the webs of the main girders. It was found from the global calculation models that the main girders of the Hagestein Bridge can be regarded as bending girders. In that case, the

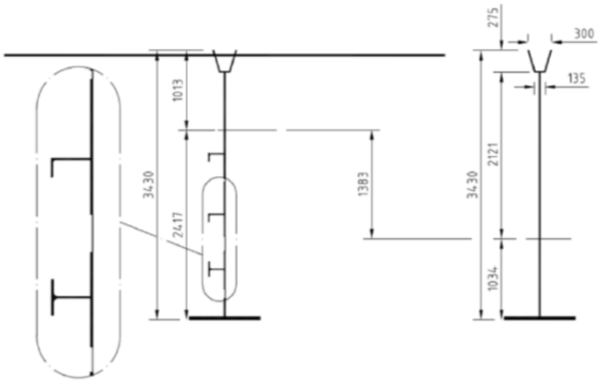


Figure 5. Cross-section reduction method applied to the main girders

resulting normal forces over each cross-section of the bridge is about equal to zero. The effective width of the deck that is to be included as part of the main girder is determined by converting the normal force in the center of gravity of the inverted T-girder from the global beam model into a bending moment in relation to the center of gravity of the deck structure. See Figure 5.

3.3 Connections

In many existing bridges shingle connections with preloaded bolts occur, which have to be evaluated. Normally, preloaded bolts are not allowed to slip in either the serviceability limit state (SLS), category B bolts according to EN 1993-1-8, or the ultimate limit state (ULS), category C

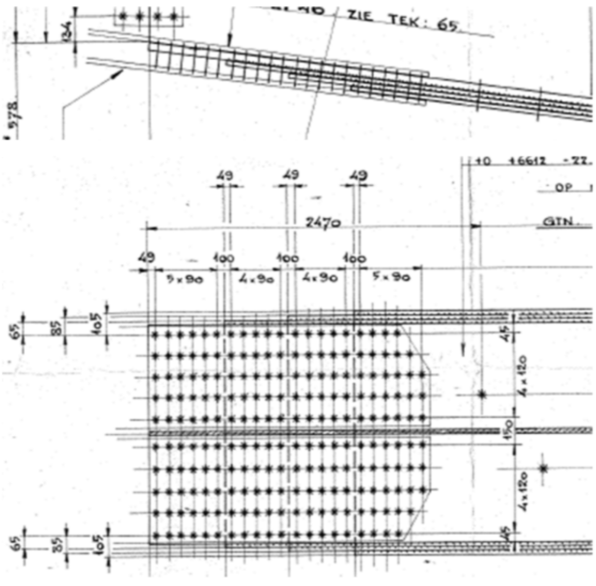


Figure 6. Shingle connection in the Hagestein Bridge

bolts according to EN 1993-1-8. For existing bridges, it is sufficient if the bolts do not slip in the SLS, category B. However, sometimes the bolts do not fulfill this requirement and then more sophisticated calculation methods have to be used, allowing the bolt to slip once. The bottom flanges of the main girders of the Hagestein Bridge all consist of a 40 mm thick flange plate with up to 3 additional plates, each 20mm thick. These plates are connected at various places using shingle connections, see Figure 6. The shear capacity of the shingle connections is determined using an elastic spring model. Each preloaded bolt in the shingle connection is modelled as a spring with a

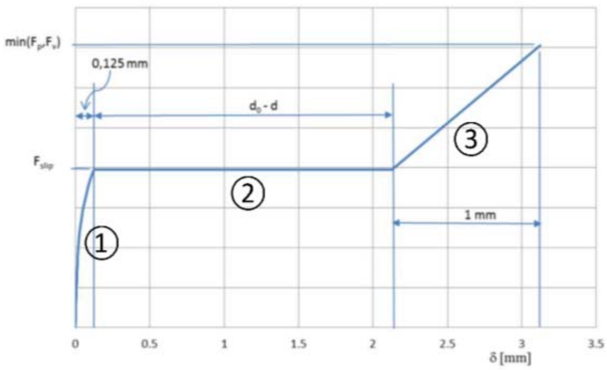


Figure 7. Load-displacement diagram for a single preloaded bolt in a lap joint

non-linear spring characteristic, Figure 7. Three different branches can be observed:

1) Behavior before slip occurs
2) Behavior during slip, before contact occurs between plate and bolt
3) Behavior after slip, after contact occurs between plate and bolt

The mechanical model of the shingle connection is shown in Figure 8. The advantage of this model is that the relative force distribution between the flange plates is described better. The connection is allowed to slip once, until the maximum of the bolt shear or bearing resistance (just before the end of branch 3). Slip is not permitted in the opposite direction, and the load-displacement behavior then must remain within branch 1. This

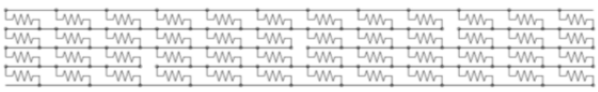


Figure 8. Elastic spring model of a shingle connection

is to prevent bolts from unscrewing themselves due to changing loads.

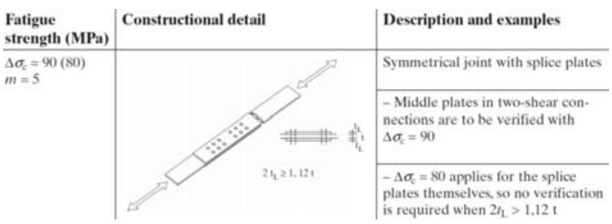
3.4 Fatigue

An important part of recalculating existing bridges is the evaluation of the fatigue strength. For bridges built before 1963 fatigue was not taken into account. The code VOS 1963 only pays limited attention to fatigue. Therefore, many existing bridge structures show signs of fatigue damage.

When recalculating existing road bridges for fatigue, the presence of asphalt on the bridge deck should be taken into account since asphalt spreads the wheel loads reducing the stress levels for fatigue. Asphalt can be modelled by volume elements with the correct temperature dependent rigidity. The asphalt may be bonded to the deck plate by contact elements, conservatively only vertically and not horizontally.

Also the traffic causing fatigue has to be taken into account carefully. For the Merwede Bridge [2] three different time periods were taken into account with typical wheel type and axle load each. Preferably, fatigue calculations shall be performed taking lorry numbers into account based on traffic counts, interpolating these counts with respect to the past en extrapolating them into the future. Also traffic measures, like prohibition of overtaking on the bridge, shall be taken into account if known.

As an example, the riveted joints in the Merwede Bridge are considered, which have been assessed for fatigue [2]. The Eurocode EN 1993-1-9 does not give any fatigue classifications for riveted joints. RWS has included fatigue classifications in its own guideline (based on [3]), Figure 9. After assessment, it was found that not all the riveted joints in the bridge were satisfactory. Reinforcement measures were necessary to reduce the stress ranges.



Fatigue strength (MPa)	Constructional detail	Description and examples
$\Delta\sigma_c = 90$ (80) $m = 5$	$2t_L \geq 1,12\,t$	Symmetrical joint with splice plates – Middle plates in two-shear connections are to be verified with $\Delta\sigma_c = 90$ – $\Delta\sigma_c = 80$ applies for the splice plates themselves, so no verification is required when $2t_L > 1,12$ t

Figure 9. Example of fatigue classification for a riveted joint [3]

4 Steel road bridges

For steel road bridges, one can distinguish between global and local deficiencies. If reassessment the bridge shows global strength and stability problems and the bridge also has fatigue damage, the remedy is either to replace the complete bridge, or to place an additional reinforcing substructure, or to repair the visible damage and locally strengthen the bridge. If reassessment shows only local deficiencies, damage repair and local strengthening is sufficient. It has to be shown that the bridge is fit for purpose for future use, meaning that upgrading may be necessary.

4.1 Global strengthening of road bridges

4.1.1 Additionally reinforcing the substructure

At the Galecopper Bridge, the bridge load was not distributed evenly across all main beams due to torsional weakness of the bridges. As a result, the bridge is strengthened by means of four steel box girders either side of the bridges, Figure 10.

Figure 10. Installation of strengthening girder, Galecopper Bridge

4.1.2 Strengthening the main girder

The recalculation of the Kreekrak Bridge showed that the main girder did not fulfill the strength and fatigue requirements. Therefore, reinforcing measures were taken, consisting of a T-shaped profile with a thick bottom flange in the areas where the stresses are too high (Figure 11), at the same time strengthening the connections in the main girder.

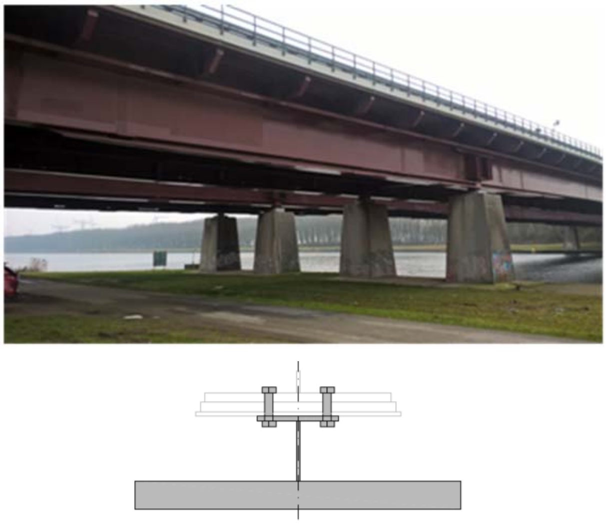


Figure 11. Strengthening of the main girder, Kreekrak Bridge

4.2 Local strengthening of road bridges

Fatigue cracks found during inspection in several important steel highway bridges in The Netherlands, are the main reason to reassess these bridges. The fatigue problems occurred in the orthotropic steel bridge decks. As an extreme example, cracks were observed in the orthotropic deck plate of the Van Brienenoord bascule bridge in Rotterdam in 1997, when the bridge was only 7 years old [4, 5]. These fatigue cracks were caused by the wheels of heavy vehicles. The designs of those days had rather thin deck plates making the orthotropic deck very susceptible to fatigue [6]. More cracks are expected to occur in the near future [4, 5]. One of the most promising ways to cope with fatigue in existing orthotropic decks is to add a layer of high strength concrete (HSC) on top of the deck plate, reducing the stress level in the orthotropic deck [7], as done at the Kreekrak Bridge, Figure 12. Also, at the Kreekrak Bridge, the web was strengthened against local buckling,

Figure 12. Strengthening by HSC, Kreekrak Bridge

Figure 13. Strengthening of the web preventing local buckling, Kreekrak Bridge

Figure 13.

When recalculating the Medewede Bridge, the riveted connections in the main girders were shown to be possibly critical for fatigue. After inspection, these connections turned out to have fatigue cracks. As an urgent strengthening measure, these connections were bridged by new additional connections with preloaded bolts, Figure 14.

Figure 14. Original riveted connection bridged by a new preloaded bolt connection, Merwede Bridge

5 Steel railway bridges

For railway bridges, in many case the main load bearing structure, such as the main plate or truss girders or arches, does not show any deficiencies contrary to local details which may need strengthening.

The Dutch railway network is one of the busiest in the world and it is necessary to have a reliable railway system. In order to prevent disruptions, the track must be in perfect shape. Preventing rail track maintenance is of extreme importance. Preventing corrosion protection maintenance is another important aspect. Preventing maintenance strongly influences the competitiveness of steel railway bridges [8]. The railway bridges built just after World War II have in most cases a traditional deck design consisting of cross girders and two longitudinal steel girders with bridge sleepers on top of them (Figure 15). This solution needs a lot of track maintenance and produces a lot of noise. Also, fatigue problems at the connection between longitudinal and cross girder may occur.

Figure 15. Traditional deck system

5.1 Local strengthening of railway bridges

Local strengthening may concern fatigue, local buckling and connections.

5.1.1 Silent bridge® decks

For fatigue, it is a good solution to replace the bridge sleepers by especially designed silent bridge® decks. This is a design with an acoustically optimal plate thickness configuration and rails embedded in a trough using corkelast. The stresses in the longitudinal girders are reduced and therefore the lifetime increased. To connect the silent bridge® decks to the longitudinal girders, the complete connection of the bridge sleepers is removed. By reusing the existing holes in de longitudinal girders, the silent bridge® decks can be connected using preloaded bolts. The upper flange plates of the new decks are continuous over the cross girders and therefore strengthen the longitudinal girders at their connections with the cross girders (Figure 16). This gives lower stress levels at the connections and also the dynamic factor for the embedded rail of the silent bridge® decks is lower. Thus, the fatigue problems at the connection between longitudinal and cross girder can be avoided.

Figure 16. New silent bridge® decks

5.1.2 Preloaded injection bolts replacing rivets

If a reassessment shows that the rivets in the connections between cross and longitudinal girders may form a fatigue problem (Figure 15), they are removed and replaced by preloaded injection bolts. This is only done in case fatigue cracks have not yet been observed. The hole is then drilled up, removing any possible invisible fatigue damage at the bolt holes, and the bolt is placed, preloaded and injected.

5.1.3 Strengthening against local buckling

An example of strengthening against local buckling

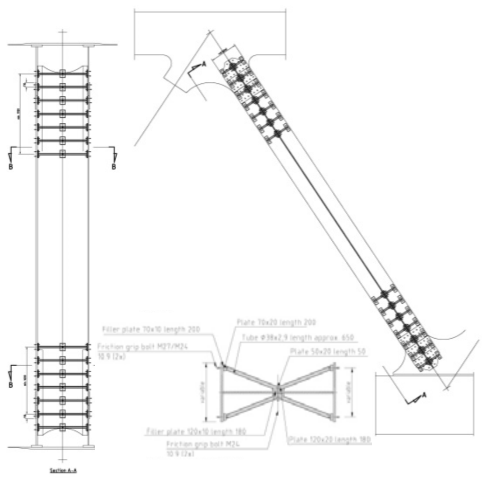


Figure 17. Strengthening of diagonals by preventing local buckling, Caland Bridge [9]

in the diagonals of a truss bridge is shown in Figure 17 [9]. Near the ends of the diagonal, where the compressive stresses cause local buckling, the flange tips are supported.

5.1.4 Strengthening of connections

An example of strengthening a connection in a truss for high stress levels is shown in Figure 18 [9]. To connect a reinforcing plate, it was necessary to loosen the rivets in the connection. Then, the reinforcing plate was welded in place. The rivets were then replaced by preloaded bolts.

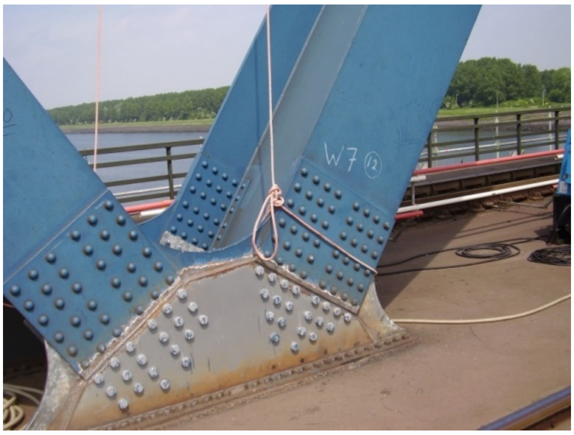


Figure 18. Strengthening the connection by a reinforcing plate, Caland Bridge [9]

6 Conclusions

Reassessment of existing steel bridges is induced by observed (fatigue) damage or due to increased traffic load prescribed by modern codes. Extensive reassessment and reinforcement programs for road and railway steel bridges are in progress in The Netherlands. The work is done in strong collaboration between engineering offices, clients, research institutes and universities to exclude any possible conservatism in the reassessments, in order to avoid unnecessary reinforcements. The paper gives examples of the repair, strengthening and upgrading techniques in practical examples of steel bridges. In The Netherlands, a lot of experience is available in reassessment, repair, strengthening and upgrading of existing steel bridges.

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