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Reliability of a Damaged RC Slab Structure using Model Code 2010 Safety Formats for NLFEA

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ABSTRACT: The Dutch Ministry of Infrastructure and the Environment is concerned with the safety of existing infrastructure and assessment thereof, which can be done with nonlinear finite element analysis. The 2017 updated version of the Dutch NLFEA Guidelines can be used for reinforced and prestressed beams and slabs. The purpose of the guidelines is twofold: 1) advice analysists on NLFEA of concrete structures, and 2) explain choices and model uncertainty. This paper introduces the latest version of the guideline, and shows the application for a reinforced concrete slab on top of a concrete beam grid. An overheight vehicle caused damage to the concrete cover and the reinforcement of the beams. The analysis shows the results of modeling the bridge without and with damage. The outcome of the assessment is that the structure fulfils the code requirements. This case study shows that the Dutch NLFEA Guidelines can be used for the assessment of damaged concrete bridges.

1 INTRODUCTION

Assessment of existing structures will benefit from additional nonlinear analyses. Very often, the structure is assumed to have extra capacity, which can only be revealed by a nonlinear finite element analysis. If this extra (or “hidden”) capacity would not be used in the assessment of a structure, a substantial number of structures would be deemed to be replaced by new ones.

The *fib* ModelCode2010 (MC2010) (fib 2012), published in 2012, provides four levels of approximation, where level IV refers to nonlinear analyses. Within this level IV three so-called safety format methods are defined, whereas Eurocode 2 (CEN 2005) only describes one safety format. The three different safety format methods in the *fib* ModelCode 2010 are:

- the Partial Safety Factor method (PF),
- the Global Resistance Factor (GRF) and
- the Estimation of Coefficient Of Variation of resistance (ECOV).

The main difference between the safety format methods is the use of either mean material values, characteristic material values or design material values as input in the nonlinear analysis. Only the ECOV safety method involves two analyses, the other two safety format methods require only one nonlinear analysis. Details of the safety formats can be found in the ModelCode 2010 (fib 2012) or in the Eurocode (CEN 2005).

To facilitate the analyst and the checking authorities in the process of a nonlinear analysis, a guideline was needed. Handbooks on the use of nonlinear analysis were already available, but it was envisioned that more guidance on the selection and use of material models was needed. Also, more validation studies of nonlinear analysis results was required.

The objectives are threefold:

1. Limit the scatter of finite element results, attributed to relatively arbitrary finite element modelling choices made by finite element users, by standardizing safe guidelines.
2. Limit the work for finite element users for justifying the finite element modelling choices made.
3. Limit the work for reviewers of nonlinear finite element assessments by standardizing guidelines for reporting finite element analyses.

Initially, the guideline focused on beam structures. Afterwards, in a second edition, the focus was extended to slab structures. Girder, slab and culvert structures cover 90% of the total amount of existing structure types.

The format of the guideline is similar to the *fib* Model Code documents:

- On the right-hand side, the guidelines as brief as possible.

- On the left-hand side, the comments and explanations of the guidelines and, where appropriate, references to literature.

The calibration of this guideline is made by re-examination of a set of experiments, which are published in international journal papers. These experiments are related to failure modes, like bending, flexural shear and punching shear failure in slabs. More in detail for the girders the shear failure can be distinguished into yielding of shear reinforcement, shear-compression failure, flexure shear. The failure mode for slabs can be distinguished into shear, mixed mode and one-way shear. The results of the validations show that the girder results are satisfying, where the slab results require some improvements. So, the NLFEA Guideline (Rijkswaterstaat 2017a) is still under development, where the simulated experiments are published simultaneously (Rijkswaterstaat 2017b).

A real-life example how to use the NLFEA Guideline can be found in (Lantsoght et al. 2019), where a reinforced concrete slab bridge which was subjected to a proof load test is simulated. The experimental data from the proof load test was available verify the results of the nonlinear analysis of the structure.

This paper will show the assessment by nonlinear analysis of a reinforced concrete viaduct from 1963 with damage.

2 DESCRIPTION OF THE SUPERSTRUCTURE

2.1 *Concrete slab*

Viaduct Watergoorschesteeg (see Figure 1) is a four-span reinforced slab in the central region of the Netherlands, which was designed and built in 1963. The viaduct is a slab on a beam grid. The viaduct has been inspected every six years.

The length of the central spans is 14.5 m and the outermost spans are 10.5 m in length. The overall width of the bridge deck is 5.7 m; the carriageway width is only 3.8 m. The central spans are divided into three parts with two intermediate cross-beams. The outermost spans are divided into two parts with only one intermediate cross beam. The bridge deck slab has a thickness of 0.30-0.36 m, the cross beams at the support lines have a width of 1.0 m and a height of 0.95 m. The intermediate cross beams have a width of 0.5 m and a height of 0.65 m. These divisions can be seen in Figure 2.

The main capacity is coming from the two edge beams, which are heavily reinforced. The traffic forces of the wheel prints will be transferred by the slab panels to the intermediate cross beams and finally to the edge beams.

The cross section of the bridge deck can be seen in Figure 3. The reinforcement in the cross beam of Figure 3 shows some bent up reinforcement. The Dutch Concrete recommendation before the Eurocode(NEN 6720 1995) assigns no extra capacity to the bent up reinforcement, so a nonlinear analysis where this reinforcement layout is incorporated will bring automatically extra capacity.

Figure 1: Photograph Watergoorschesteeg viaduct.

Figure 2: Photograph different cross beams.

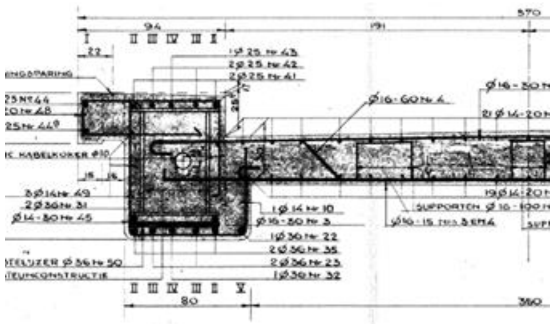

Figure 3: Half of the cross section of the bridge deck.

The longitudinal reinforcement of the edge beams contains a lot of bent up bars, so also here there is extra capacity expected by running a nonlinear analysis. 4 shows an impression of the amount of bent up bars in the edge beams. Figure 4 shows also the side view of the reinforcement layout in longitudinal direction of a side span and a main span. **Error! Reference source not found.**4 shows on the right side of the figure the five different layers of the reinforcement, which can be found also in Figure 3 under the edge beams. In Figure 3 the order of the layers starts with II followed by III, IV, III and II. The outside of the edge beam shows a layer I at the top, where the inside of the edge beams shows a layer V at the bottom. The diameter of the reinforcement bars is 36 and 25 mm, which is rather heavy for the time period of design (1963).

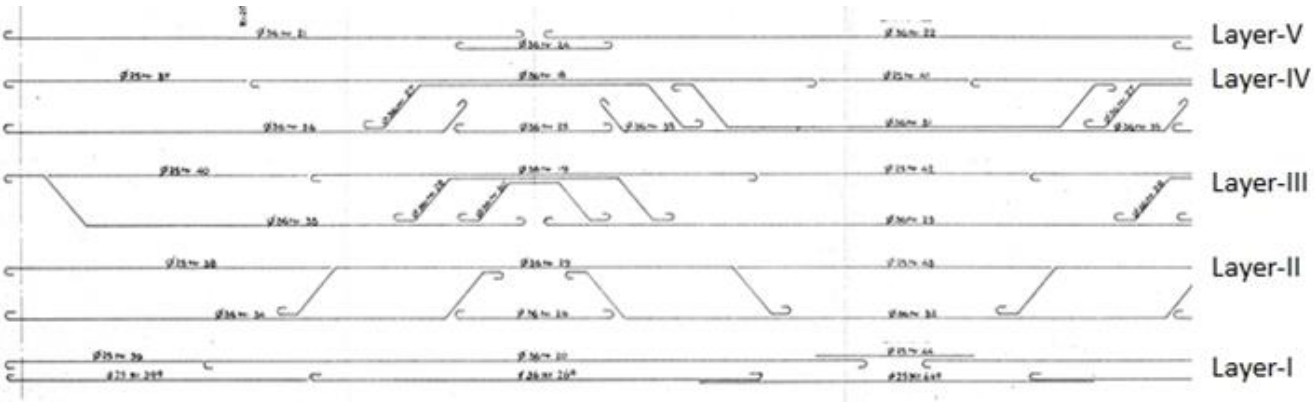

Figure 4. Side view of reinforcement layers of the edge beams.

2.2 *Material properties*

The actual material properties of the bridge deck are unknown. Since no concrete cores are drilled from the structure, the default properties from the Dutch recommendation of the Ministry of Infrastructure (Rijkswaterstaat 2013) are used. The international concrete class C37.5/B45 is used.
The reinforcement bars are from the time period when plain bars were used, so the usual 209 N/mm2 yield strength is assumed. The number of bent up bars is rather high in the three main layers II, III and IV.

2.3 *Loads*

The usual load cases of the slab are the dead weight, the asphalt layer and the two traffic load cases, a uniformly distributed load case and a set of wheel print loads. The thickness of the asphalt layer is 140 mm, according the RBK1.1(Rijkswaterstaat 2013) for existing structures. The damage of the structure was located in span 2 at the second intermediate panel edge beam. For that reason, four different traffic load cases were chosen, one bending and three shear force live load wheel print load cases, represented by the position of the design tandem. The bending moment load case (TS4) is located in the middle of span 2, and one shear force load case (TS3) is located near the first support crossbeam between span 1 and span 2. The second shear force load case (TS1) is near the first intermediate crossbeam of span 2 in the center over the width of the bridge deck. Finally, the third shear force load case (TS2) in span 2 is located near the first intermediate crossbeam of span 2 but now also near the edge beam in the width direction. Figure 5 shows the locations of the tandem wheel prints.

The original design traffic wheel print load case consists of two axles of 100 kN each and a distributed load of 3.5 kN/m^2.

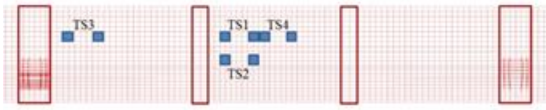

Figure 5. Load locations tandem wheel prints span 2

2.4 *Supports*

The supports of the slab are a mix of steel/rubber bearings, The first and final crossbeam are supported by three rectangular bearings with dimensions of 250×300×66 mm. The intermediate cross beams re supported by two circular bearings with a diameter of 500 mm and a thickness of 45 mm.

The material properties are unknown, so the normal and shear stiffness of the bearings can be calculated from the maximum allowed vertical displacement of 1 mm according to the Dutch RBK1.1 (Rijkswaterstaat 2013), when the slab is loaded by the both permanent load cases (dead weight and asphalt layer).

3 NONLINEAR ANALYSIS OF THE SLAB STRUCTURE

3.1 *Input geometry nonlinear analysis*

The FEA model (DIANA FEA 2019) can be minimized by the small width of the bridge deck, so a half bridge deck with its beam grid is modelled.

This means a reduction of input, especially for amount of the reinforcement bars. A side view can be seen in Figure 6 and an iso view of the bottom side of the concrete structure is shown in Figure 7.

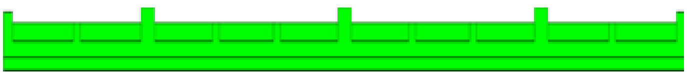
Figure 6. Side view of the NLFEA model

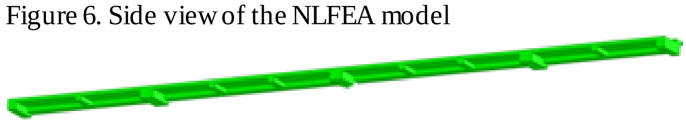
Figure 7. Iso view of the NLFEA model

Both figures show clearly the differences in dimensions of the crossbeams and the intermediate crossbeams. The NLFEA model contains only quadratic hexa solid elements with a maximum element side length of 100 mm. The dimension of 100 mm (3 stress points over the element side length) is related to the face to face dimension (150mm) of the different layers in the edge beam reinforcement. The reinforcement has been modelled as shown in Figures 4.

The bearings are modelled by quadratic interface elements. The circular shape of the intermediate beam bearings is simplified by using an equivalent rectangular area.

Figure 8 shows the element distribution over the height of the bridge deck and the edge beam at the location of the intermediate support cross beam.

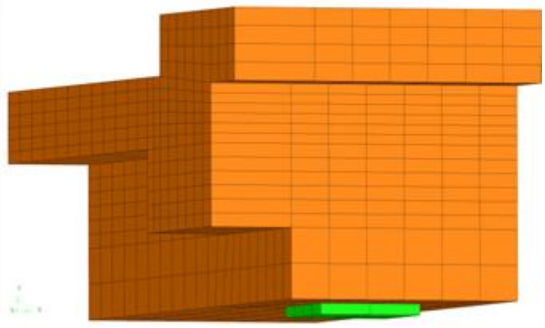

Figure 8. Element distribution at the support crossbeam

Figure 8 shows the concrete part and one bearing. The thickness of the bridge deck on the left side is divided over six elements, the edge of the bridge deck on the right side shows four elements over the height. The edge beam itself counts six elements on longitudinal direction and 14 elements over the height. The bearing part consists of 2×2×1 elements.

3.2 *Input material properties*

Besides the usual linear static input parameters, such as the Young's modulus, Poisson ratio and density of the concrete and reinforcement, additional properties are needed for the concrete crack model, the crushing behavior of the concrete and the yielding behavior of the reinforcement. These options and associated model type and values are summarized in Table 1.

3.3 *Load cases*

The load case dead weight is automatically coupled by the density material property. The asphalt load case is a distributed surface load case located on the bridge deck. Both load cases have a partial load factor of 1.15 according the Dutch RBK1.1 (Rijkswaterstaat 2013) for existing structures. The distributed traffic load case is also a distributed surface load case. All wheel print load cases are distributed load cases on the area of the wheel print 400×400 mm^2 of the bridge deck top surface, according the Eurocode 1 (NEN EC1 2011). The axle load of all four TSx load cases is 300 kN. The partial factor according traffic load cases is 1.25 (NEN 8700 2011).

Table 1. Material properties concrete and reinforcement

Concrete	**Input [N, mm]**	
Young's Modulus	YOUNG	3.35E+04
Poisson ratio	POISON	1.50E-01
Density	DENSIT	2.50E-06
Crack Model	TOTCRK	ROTATE
Govindjee's Projection Method	CBSPEC	GOVIND
Softening Curve	TENCRV	HORDYK
Poisson reduction Model	POIRED	DAMAGE
Tensile strength	TENSTR	1.65
Tensile Fracture Energy	GF1	0.12
Compression Curve	COMCRV	PARABO
Lateral Influence of Cracking Model	REDCRV	VC1993
Maximum reduction factor	REDMIN	4.00E-01
Lateral Influence of Confinement	CNFCRV	VECCHI
Compression strength	COMSTR	27.0
Compressive Fracture Energy	GC	30.0
Reinforcement	**Input**	
Young's Modulus	YOUNG	2.00E+05
Poisson ratio	POISON	3.00E-01
Density	DENSIT	7.85E-06
Plasticity Model	YIELD	VMISES
Hardening Model	HARDEN	WORK
Hardening values	KAPSIG	0. 209. 0.05 240.

3.4 *Nonlinear analysis process*

The nonlinear analysis is fulfilled by the Newton-Raphson method with additional options like arc length and line search to run the nonlinear analysis in a comfortable and steady process.

To control the analysis, the convergence norm is related to energy, because of the cracking behaviour of concrete, with an energy tolerance of 1.E-03.

This process can be visualized by Figure 9, where all iterations of the different load steps converge below this limit of 1.E-03.

3.5 *Results of the nonlinear analysis*

The first usual result of a nonlinear analysis is the load deflection curve. Here, Figure 10 shows the results for the four different wheel print load cases. Figure 10 shows that the difference in deformation between load cases TS1 (shear force) and TS4 (bending moment) is very small, where the shear force load case TS3, near the support cross beam, isn't relevant. Further important results are given in Table 2.

Table 2 shows also that the results of load case TS3 are far below the other 3 wheel print load cases. Every load case reaches at least a load factor 1.7, so there is more reserve capacity than the load factor 1.6, which is recommended in the *fib* ModelCode 2010 for nonlinear analysis. The factor 1.6 results of multiplying the uncertainty of the model (1.2) with the uncertainty of the end-user of NLFEA (1.06) and the common partial load factor for traffic loads

(1.25). The maximum value of the reinforcement strains at a load factor of 1.6 shows 1.83%, which is far below the limit strain of 5%. However, the value for the concrete is -5.8‰, which is below the limit of -3.5‰. These concrete elements in the structure are located at the support cross beam and are very local at the edge, which is indicated in Figure 11.

Table 2. Results NLFEA undamaged structure

Load-case	Total number of itera-tions	Maxi-mum loadfac-tor TS load	Maximum strain reinforce-ment load-factor 1.6	Minimum strain concrete loadfac-tor 1.6
TS1	2723	1.7	0.0174	0.0058
TS2	2421	1.7	0.0116	0.0052
TS3	1440	1.7	0.0084	0.0027
TS4	2482	1.7	0.0183	0.0055

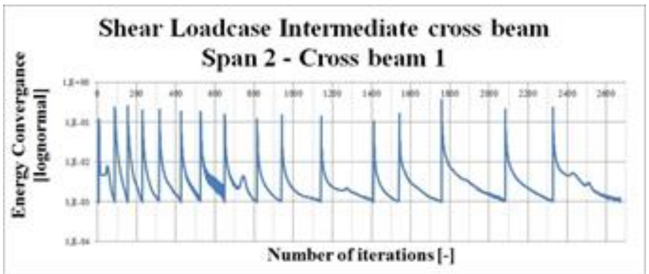

Figure 9. Energy convergence development versus total number of iterations

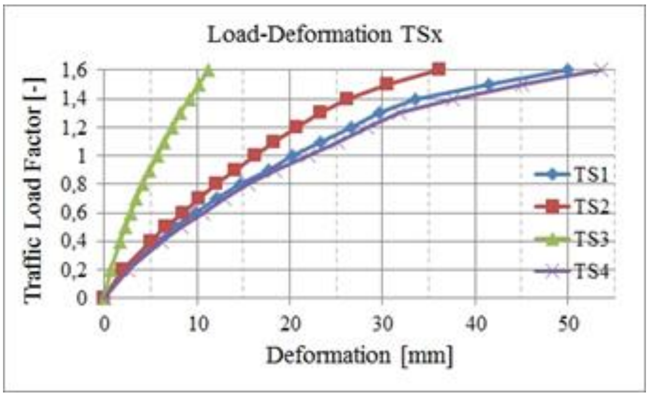

Figure 10. Load-deformation diagram traffic load cases

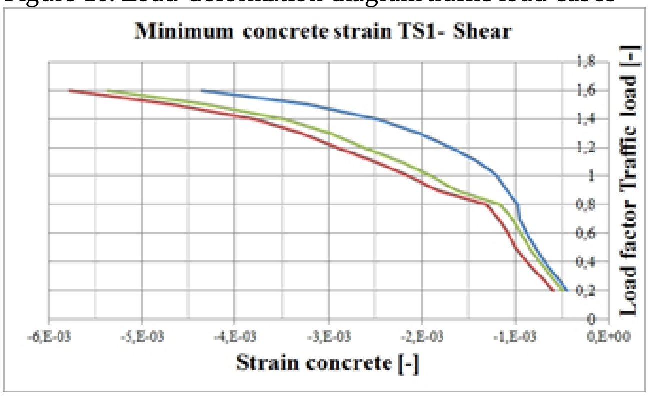

Figure 11. Concrete strain elements below the -3.5‰

Important to know is the fact when this crushing behaviour of the concrete starts. This development is shown in Figure 12 for load case TS1 for the three finite elements.

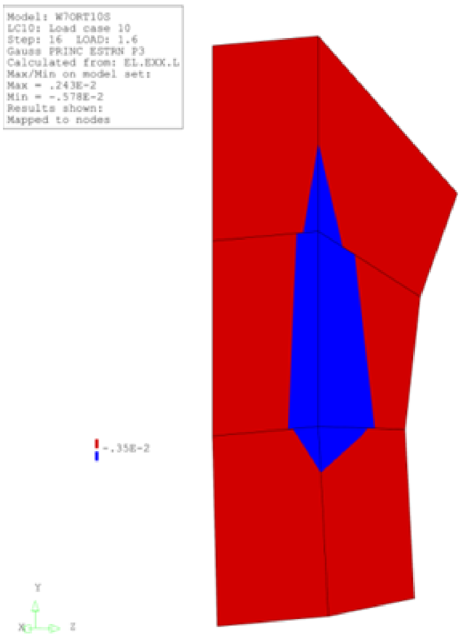

Figure 12. Development of the critical concrete strains

Figure 12 shows that the concrete strain of -3.5‰ is reached at the level of load factor 1.18. Conclusion from Figures 11 and 12 is that the minimum strain of the concrete is only localized in some stress points of three elements, which can be seen as a local aspect.

4 NONLINEAR ANALYSIS OF THE DAMAGED STRUCTURE

4.1 *Damage*

The damage of the edge beam can be seen on the photograph of Figure 13. Two reinforcement bars are broken over a length of 1 meter and also some concrete cover is missing.

Figure 13. Photograph of the edge beam damage

The damage results in a modification of the reinforcement bars of the edge beams, which is done by splitting two bottom level longitudinal reinforcement bars into two separate bars. This can be seen in Figure 14. The nonlinear analysis of the three traffic load cases belonging to mid part of span 2 are relevant.

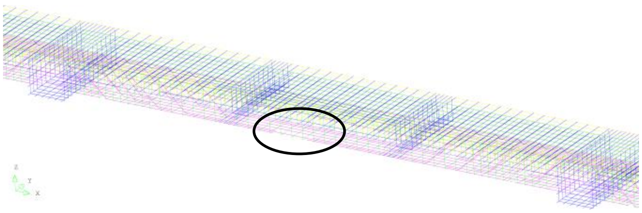

Figure 14. Excluded reinforcement edge beam mid span 2

4.2 *Results of the nonlinear analysis*

Again also here the most usual output is the load deformation of the structure can be given, now for the three relevant load cases TS1, TS2 and TS4. This is shown in Figure 15.

Figure 15 shows that the deformation results of the load case TS1 and TS4 are close to each other, where TS4 has still the maximum deformation. However, the increase of the deformation is rather large, changing from 53 mm to 83 mm!

From this nonlinear analysis, some more strain results are given in Table 3.

The strain results in Table 3 show again that the maximum reinforcement strains are increased to a value of 2.44%, but this is still under the limit of 5%.

The concrete strain is decreased from -5.8‰ to a value of -8.5‰. Again the location of these higher strains can be shown in Figure 16.

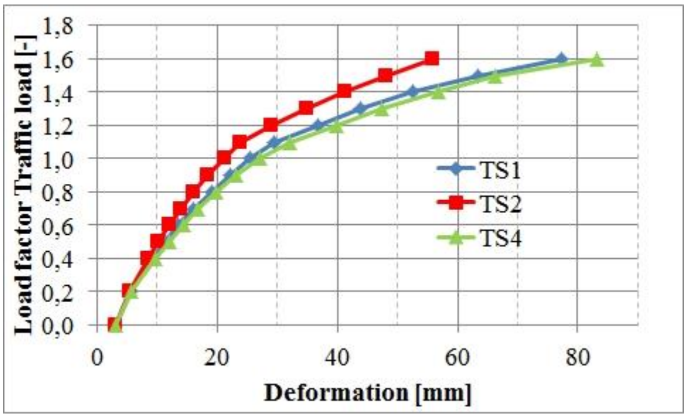

Figure 15. Load-deformation wheel print load cases

Table 3. Overview strains damaged structure

Load-case	Total number of itera-tions	Maxi-mum loadfac-tor TS load	Max strain reinforce-ment at loadfactor 1.6	Min strain concrete at loadfactor 1.6
TS1	2326	1.7	0.0244	0.00854
TS2	2039	1.7*	0.0205	0.00780
TS4	2192	1.6	0.0120	0.00852

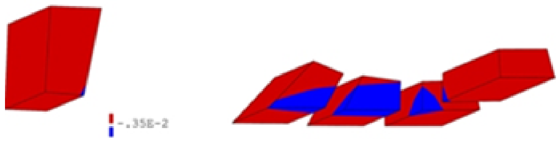

Figure 16. Concrete strains elements below the -3.5‰

The location of these finite elements with concrete strains below the -3.5‰ is above the bearing support plate of the damaged edge beam supporting crossbeam between span 1 and span 2.

This means that the location is shifted from the edge of the crossbeam to the region above the bearing plate of the same crossbeam. The edge side got a redistribution of strains, where the minimum value is coming below the limit of -3.5‰. Inspection of this edge side location didn't show any damage (crushing) of the concrete. The development of the concrete strain is shown in Figure 17.

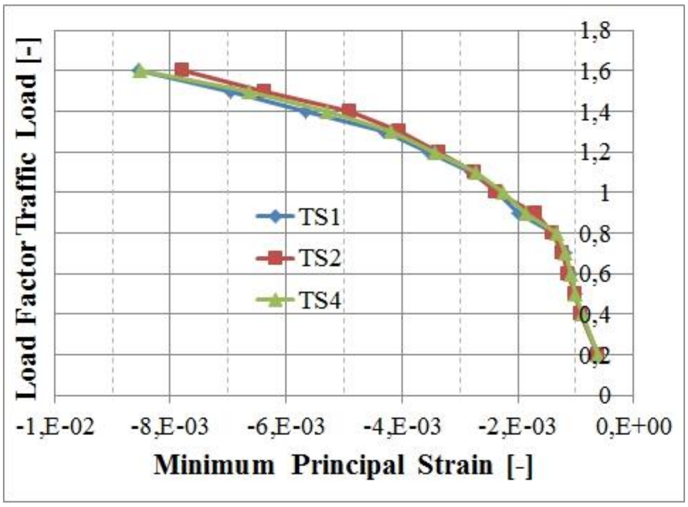

Figure 17. Development of the critical concrete strains

The critical principal concrete strains (Figure 17) shows still a load factor of 1.18, when the minimum limit of -3.5‰ is reached. The change from the edge of the crossbeam to the bearing plate region doesn't have effect on the load factor for the wheel print load.

Figure 17 counts five elements with a lower minimum strain, where Figure 11 shows only three elements with a lower minimum strain. However two elements (the most left and right element) in Figure 17 have a very small nodal area, where the minimum strain is reached.

To identify the principal concrete strain in the bottom fibre of the supporting cross beam, the strain is plotted at the area of the bearing and the surrounded row of elements of load case TS4. This plot is shown in Figure 18.

Figure 18 shows at the edge of the bearing area only low strain, where most of the area at the bottom fibre has a tensile strain. This behavior is rather overestimated by the way of modelling no pylons in this calculation. When the pylons have a normal length in the model and automatically also the belonging stiffness the strains would be presented in a more smooth way without extreme values.

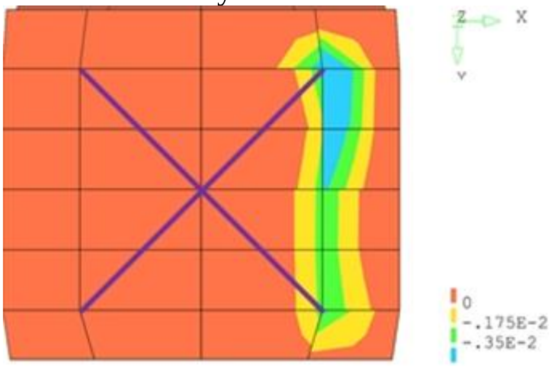

Figure 18. Concrete strain values around the bearing area

Figure 18 shows a principal concrete strain over half of the right side of the bearing area, which is indicated by the purple cross, with a value below -3.5‰. The other half of the right side has values between -1.75 and -3.5‰.

5 COMPARING RESULTS BOTH NONLINEAR ANALYSIS

When the localized overestimated principle concrete strains are accepted as a modelling aspect, the conclusion can be made that the damaged structure has still enough bearing capacity according the Dutch NEN 8700 2011.

The original design of this structure was for the traffic class 'C', which means 2 axles of 100 kN. The undamaged nonlinear analysis proved that the structure can carry the traffic load of 2 axles with 300 kN.

There is a decrease of the load factor of the representative traffic load case (TS4), but still the recommended load factor of 1.6 can be reached.

The strains of the reinforcement bars are increasing between the undamaged and damaged analysis, but are still below the ultimate value of 5%.

The maximum deformations are increasing from 53 to 83 mm, which is rather high, because the ratio to the span length becomes 1:175.

An alternative crack model regarding softening, in this case the Cervenka model, shows similar results. The benefit of this softening model is the cut-off after an ultimate limit of the crack strain. However there is no automatic FE input possible at this moment regarding the relation to the Fracture Energy, so that should be calculated based on the chosen FE element side and the belonging integration point scheme of the element.

6 CONCLUSIONS

The following conclusions can be made:

1. Both nonlinear analyses could reach the recommended load factor for traffic of 1.6, which means that the structure has no restrictions for common Dutch traffic.
2. Exceptional traffic should be checked by incident on forehand, which is the usual procedure in the Netherlands.
3. Both nonlinear analyses have reached full energy convergence at all load steps.
4. The used nonlinear options in both calculations are according to the NLFEA Guideline.
5. The applications of real concrete structure can be extended by other concrete structures, as well as simulations of lab experiments.

ACKNOWLEDGEMENT

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