Improvements of a Nonlinear Analysis Guideline for the Re-examination of Existing Urban Concrete Structures

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Improvements of a Nonlinear Analysis Guideline for the Re-examination of Existing Urban Concrete Structures

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Abstract

The Dutch Ministry of Infrastructure and the Environment is concerned with the safety of existing infrastructure and expected re-analysis of a large number of bridges and viaducts. Nonlinear finite element analysis can provide a tool to assess safety; a more realistic estimation of the existing safety can be obtained.

Dutch Guidelines, based on scientific research, general consensus among peers, and a long-term experience with nonlinear analysis, allow for a reduction of model and user factors and improve the robustness of nonlinear finite element analyses.

The 2017 version of the guidelines can be used for the finite element analysis of basic concrete structural elements like beams, girders and slabs, reinforced or prestressed. Existing structures, like box-girder structures, culverts and bridge decks with prestressed girders in composite structures can be analysed.

The guidelines have been developed with a two-fold purpose. First, to advice analysts on nonlinear finite element analysis of reinforced and pre-stressed concrete structures. Second, to explain the choices made and to educate analysts, related to the responsibility of limiting model uncertainty.

This paper contains an overview of the latest version of the guideline and its latest validation extensions. Most important impact is the extended operational lifetime of an existing reinforced concrete slab structure.

Keywords: Guideline, nonlinear analysis, validation, re-examination, assessment, reinforced concrete

1 Introduction

The publication of the ModelCode 2010 and the Eurocode 2012 by fib and CEN motivated the use of perform a nonlinear analyses for the design or re-examination of concrete structures. Before 2012, only so-called level of approximation I analysis, including the partial safety factor method, which is also recommended in the Eurocode serie, was used in practical engineering.

Re-examination of existing structures will profit from additional nonlinear analyses. Very often, the structure is supposed to have an extra bearing capacity, which could only be revealed by a nonlinear finite element analysis. If this extra (or "hidden") capacity would not be used in a re-examination of a structure, a substantial number of structures would be deemed to be replaced by new ones.

The fib ModelCode2010 (MC2010), the final version was published in 2012, provides four levels of approximation, where level IV refers to nonlinear analyses. Within this level IV three alternative so-called safety format methods are defined, where the Eurocode 2 only describes one safety format method.

The three different safety format methods 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 ModelCode2010[1] or in the Eurocode[2] as well.

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:

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

Initially, the guideline was focusing on beam structures,. Afterwards, in a second edition, the focus was extended to slab structures. The 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 worldwide journals or papers. These experiments are related to failure modes, like bending, flexural shear and shear failure in slabs. More in detail for the girders the shear failure can be distinguished into yielding of shear reinforcement, compressive shear, diagonal critical crack. The failure mode for slabs can be distinguished into shear, mixed mode and one-way shear. An overview of the experiments is given in Table 1. In this table a column is added with a result $P_{max,exp}/P_{max,NLFEA}$ of the re-examinations of the experiments. These results show that the girder results are satisfying, where the slab results require some improvements. After publishing the first results, the slab RS1 is updated by adding reinforcement to the model at the edges of the slab, where the upper and lower reinforcement is bending in the vertical direction. This shows a better results, with a $P_{max,exp}/P_{max,NLFEA}$ of 1.35. Nevertheless, the girder results remain better than the slab results. More elements over the height of the slab would give improved results.

Table 1: Overview NLFEA experiments

Failure mode		Case	Transversal reinforcement	$P_{max,exp}$/ $P_{max,NLFEA}$
Bending		RB1	Yes	0.99
		RB3	Yes	1.00
		PB1	Yes	0.93
Flexural-shear	Yielding of shear reinforcement	RB3A	Yes	1.14
	Compressive shear	PB2	Yes	0.94
		PB3	Yes	1.04
		PB4	Yes	0.98
	Diagonal critical crack	RB2	No	0.95
Shear failure in slabs*	Shear	RS1	No	1.62
	Shear	RS2	No	1.36
	Mixed mode	RS3	No	1.29
	One-way shear	RS4	No	1.33
	Mixed mode	RS5	No	1.33
			Mean	1.15
			CoV	0.19

* one-way shear or punching shear or combination of one-way shear and punching shear

Looking to the main failure mode the following Table 2 can be setup. Table 2 shows accurate results for the beams and rather good results for the slabs in shear. The results for the slabs are biased on the conservative side.

Table 2: Statistical properties of the modelling uncertainty per *failure mode*

Failure mode	**Mean**	**CoV**
Bending	0.97	0.04
Flexural shear in beams	1.01	0.08
Shear in slabs	1.39	0.10
All	1.15	0.19

Table 3 shows the results of the extra hidden capacity of the experiments using the different safety formats, by comparing design capacities obtained by nonlinear finite element analysis to the design capacities obtained with more traditional methods.

Table 3: Results using different safety formats for LoA IV

Member	**MC 2010 LoA IV [kN]**			$P_{Rd,\,LoA\,IV}$/ $P_{Rd,min}$	$P_{Rd,\,LoA\,IV}$ / $P_{Rd,max}$
	GRF	PF	ECOV		
RB1	190	193	203	1.12	1.12
RB3	116	115	120	1.21	1.21
PB1	1352	1376	1514	1.38	1.38
RB3A	110	114	119	2.06	1.22
PB2	4639	4774	5391	1.65	1.36
PB3	1549	1857	1952	3.28	1.36
PB4	809	589	874	1.59	1.4
RB2	54	56	57	1.62	0.97
RS1	-	-	-	-	-
RS2	785	917	890	2.09	1.4
RS3	502	582	588	4.0	2.1
RS4	521	613	607	4.0	2.23
RS5	610	726	677	4.29	2.34

Table 3 shows that in almost all cases a substantial extra hidden capacity is demonstrated by using level of approximation IV of the ModelCode2010.

The results of these validation studies for reinforced concrete components, encouraged the use of the guidelines at structural level. Various publications on proof loading of reinforced concrete structures are available. One of these structures is the Viaduct De Beek in the Netherlands, where detailed measurements are available. This viaduct is selected to compare these measurements with the results of a nonlinear analysis.

2 Viaduct De Beek

2.1 History and modelling

Viaduct De Beek (see Figure 1) is a four span reinforced slab in the south region of the Netherlands. The viaduct has been inspected every six years.

Fig. 1: Photograph of viaduct De Beek

The latest inspection revealed many small cracks at the main span. However this span is over the highway, where it is not allowed as a proofloading location. Instead, the side span can be proofloaded to quantify the capacity of the slab. In the Netherlands these local viaducts have a yearly maximum of 20.000 heavy vehicle passages. These heavy vehicle passages are related to agricultural traffic. Measurements of these passages have been, showing an upper limit of 50.000 passages.

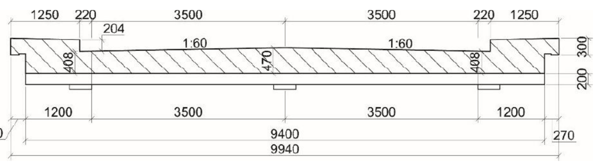

Fig 2: Cross-section of the viaduct near the support

A simple linear static analysis, based on FE shell elements, resulted in a restriction of usage of both traffic lanes. Consequently, the width of the bridge deck was narrowed by barriers to one traffic lane. However, the local authorities preferred to know the real capacity of the viaduct, so a proofloading was planned.

In 2015 the first version of the NLFEA Guideline was published and the first solid FE model of this viaduct shows also the same traffic restriction, but there was some extra hidden capacity in the viaduct by the bent-upward reinforcements, which capacity is not incorporated into the Dutch concrete recommendation. Figure 3 shows the layers of the reinforcement including the layers with the bent-upward reinforcement of a small section of the side span. Each reinforcement bar is a separated input item in the FE model. The layers have a head to head distance of 560 mm, so the longitudinal reinforcement have a head to head distance of 140mm.

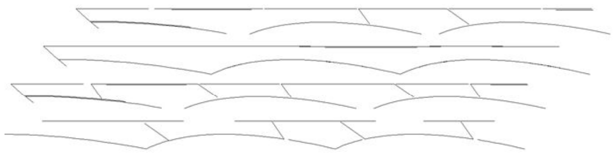

Fig.3: Layers of reinforcement with the bent-upward particles

The sizes of the concrete elements in the FE solid model are related to the head to head distance of the reinforcement. If cracking would occur within an element, there must be sufficient stress points available along a side of the element. Consequently, an element size of 2/3 of the head to head reinforcement distance was selected. For the transverse direction a structural element side of 100 mm is used and for the longitudinal direction a structural element side of 150 mm used.

The connection between the side curbs are very soft. The concrete-concrete surface and only one reinforcement bar with a diameter of 12mm is present. So an upper and lower load capacity can be analyzed by a FE model with and without the curbs.

The material properties are obtained from drilled cylinders out of the bridge deck so the actual physical properties with the related equations form the ModelCode2010 are the input for the nonlinear analysis.

Measurements of the proofloading with two loading locations, one for bending and one for shear force can be compared for the side span. Measurements are deflections in longitudinal and transverse direction of the first span and concrete strains at the bottom surface of the first span over sensor lengths of 1 m can be used to compare. A view of the location of the deflection points is given in Figure 4 with the indication of LVDTxx.

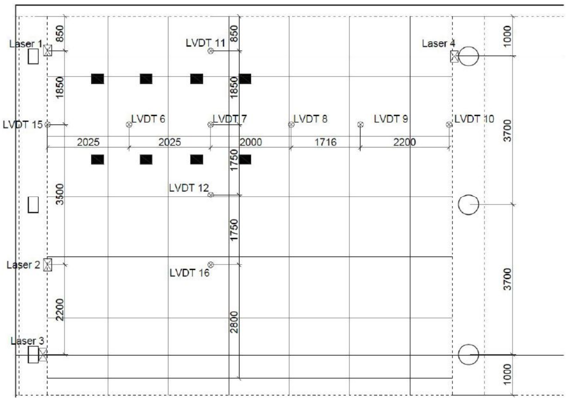

Fig.4: Deflections points side span and bending and shear force location proofloading

2.2 Compared results

To get confidence in the nonlinear analysis and get the right convergence in the nonlinear process a figure with the convergence energy values is given over all 120 iterations in Figure 5.

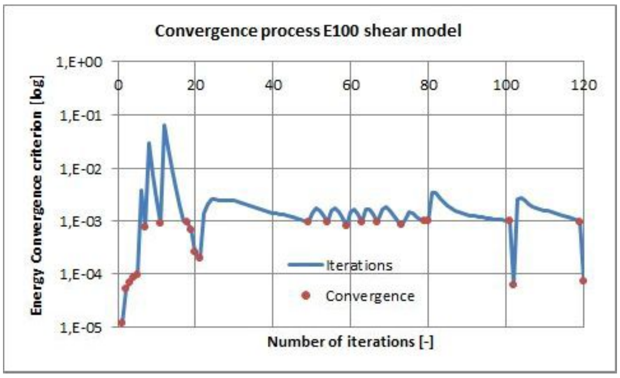

Fig. 5: Convergence process Shear Load Location

Figure 5 shows that at all load steps of the nonlinear analysis simulation the convergence criterion is reached. At both simulations with the shear and bending location this convergence process was successful.

The next result is a load deflection of the location of LVDT7, which is the intersection point of the longitudinal and transverse LVDTxx lines in both models(with and without curbs), which is shown in Figure 6.

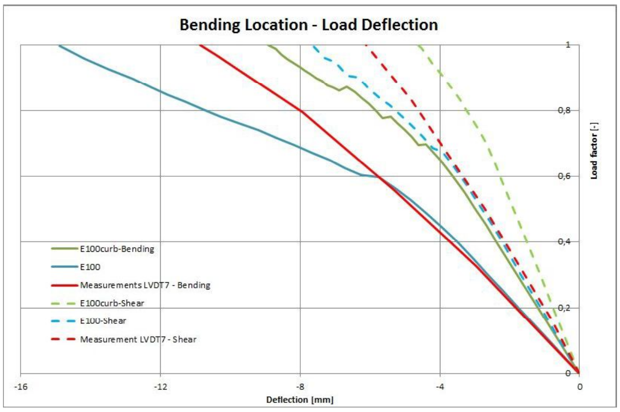

Fig. 6: Load deflection of LVDT7

The results of the deflection at both proofload locations of LVDT7 gives similar results as the FE model without curbs till a load factor level of 0.6. The second branch of the diagram shows a larger deflection then the measurements. Probably there is some additional stiffness of the overall bridge deck coming from the connection between the curbs and the bridge deck.

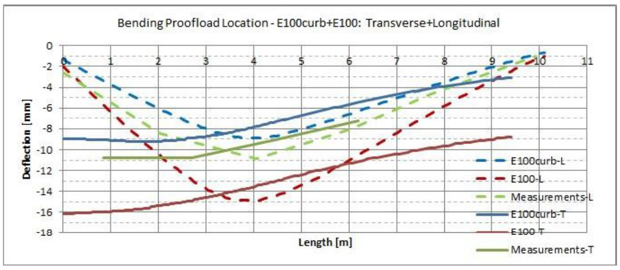

Fig. 7: Load deflection of LVDTxx lines

Figure 7 shows that the measurements of the deflections at the final load stage of the proofloading with the bending location are between the results of the two FE models with and without the curb. The simulation of the shear load location gives similar results as the bending location. The total load of the shear force location was 1500 kN and for the bending location 1700 kN. The total load was distributed over 4 wheel print loads, so the minimum axle load was 750 kN, which is not allowed in the Netherlands. The specific conclusion is that the concentrated axle load is reached with a range of crack widths of about 0.1-0.2 mm.

The overall conclusion is that the FE simulations are close to the measurements given by both proofloading locations. The simulations of the main span can now be setup in a similar way.

3 Assessment main span De Beek

The results of the main span can also be given in the same way like the side span, but now without measurements of LVDT's. The FE model is without the curbs, since the results in the side span showed a rather good result for model without the curbs

Again the convergence process is rather stable and reaches at every load step the relative energy convergence tolerance of 10^{-3}.

The load deflection of both load cases, one lane or two lanes, with traffic is the most important result. This is shown in figure 8.

Fig. 8: Load deflection 1 or 2 traffic lanes

Figure 8 shows that the overall load on one lane is larger than the overall load on two lanes. The specific load at two lanes is of course higher than on one lane. The load factor belonging to one lane traffic is 1.82, where the reached load factor belonging to two lane traffic stops at 1.15. This means that the lane restriction should be remained, because the load factor should be 1.58, following the Dutch traffic recommendation partial factor and the extra ModelCode2010 uncertainty factor for nonlinear analysis.

4 Conclusions

The following conclusions can be drawn:

- The comparison between the results of measurements on structures and FE simulations shows good results
- The NLFEA Guideline gives guidance in analyzing reinforced concrete structures by restrictions in material models and checking results related to the ModelCode2010
- The extra bearing capacity of one lane traffic shows a factor 0.2, where the linear elastic analysis shows an extra factor of 0.0.
- Improving the confidence in using nonlinear analysis for re-examinations of more existing concrete structure will give an extension of the existing assessment tools
- Urban structures do not have to be demolished, in case of demonstrating extra bearing capacity by analyzing the structure in an advanced way.

References

[1] fib 2012. Model Code MC2010 Final draft. Lausanne: International Federation for Structural Concrete (fib)

[2] CEN 2005. Eurocode 2: Design of Concrete Structures - Part 1-1 General Rules and Rules for Buildings. EN 1992-1-1:2005. Brussels, Belgium: Comité Européen de Normalisation

[3] M.A.N. Hendriks, A. de Boer, B. Belletti, "Guidelines for Nonlinear Finite Element Analysis of Concrete Structures", Rijkswaterstaat Centre for Infrastructure, Report RTD:1016-1:2017, version 2.1.1, 2017 (to be published by Rijkswaterstaat).

[4] M.A.N. Hendriks, A. de Boer, B. Belletti, "Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures - Part: Overview of results", Rijkswaterstaat Centre for Infrastructure, Report RTD:1016-2:2017, version 1.0, 2017 (to be published by Rijkswaterstaat)

[5] E.O.L. Lantsoght, A. de Boer, C. van der Veen, D.A. Hordijk, "Optimizing Finite Element Models for Concrete Bridge Assessment with

Proof Load Testing", www.frontiers.org, (in review, will be published soon)