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Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures

Part: Review of results

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PREFACE

At an international workshop on shear force capacities of concrete structural element, held in Rotterdam, the Netherlands in 2007, predictions of the ultimate limit state of three different girder experiments were presented. This workshop was initiated by the Dutch Ministry of Infrastructure and organized by TNO (Vervuurt & Leeghwater, 2008). The ultimate capacities, predicted by six teams using different nonlinear software packages, showed a large scatter. Also the predicted crack patterns showed a large scatter.

With this in mind, research on the development of a “guideline for nonlinear analysis of concrete girders” was started. The *fib* Model Code 1990 was the background document when Peter Feenstra started with the development of the guideline. Also, Joop den Uijl was involved in validating the guidelines. From 2010 the draft version of the *fib* Model Code 2010 was used as background document. Today, both the MC2010 and the Eurocode2 allow the use of nonlinear analysis to verify the design capacity of concrete objects.

The validation of the guidelines is done by simulating old and new experiments. To verify human and software factors, several people were involved in this project and two commercially available software packages were used. Finally the first version of the guideline was published in May 2012. It is used by the Dutch Ministry of Infrastructure and the Environment when commissioning engineering work for re-examinations of existing concrete structures in the Netherlands to reveal extra remaining structural capacity.

To verify whether the guideline is also valid for a larger group of international end-users and for other software packages, a prediction contest of T-shaped prestressed girders was set up in 2014. The tests were performed by Sebastiaan Ensink in the Stevin Laboratory of the Delft University of Technology. The participants of the contest gathered in a workshop in Parma. The outcome of this contest showed that the guidelines are indeed helpful for reducing model and human factors when predicting the behaviour of concrete structures by means of nonlinear finite element analysis.

As a result of additional validation studies and making use of the experiences of the workshop in Parma a new version of the guidelines has been published in 2016. The present document gives an overview of validations studies for this version of the guideline. Maciej Kraczla has contributed to this document.

This document is one from a series of documents. At the time of writing, the following documents have been drafted:

RTD 1016-1:	Guidelines for Nonlinear Finite Element Analysis of Concrete Structures
RTD 1016-2:	Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures - Part: Overview of results
RTD 1016-3A:	Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures - Part: Reinforced beams
RTD 1016-3B:	Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures - Part: Prestressed beams
RTD 1016-3C:	Validation of the Guidelines for Nonlinear Finite Element Analysis of Concrete Structures - Part: Slabs

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1 Introduction

The main reason of developing guidelines is ensuring a “reasonable” modelling approach with knowledge of the modelling uncertainty. The guidelines are definitely not envisioned as the best and only modelling approach. In this report, the guidelines have been validated by strictly and consistently applying the guidelines to 13 cases.
In this part we present (and shortly discuss) three overviews of results: one on the model uncertainty by comparing experimental failure loads to failure loads predicted by the nonlinear finite element analyses (NLFEA), one on the influence of the level of approximation on the design strength, and one the prediction of the failure mode. The overview of all results is presented Table 4-1.

2 Modelling uncertainty

Table 2-1 gives an overview of the most important outcomes of the analyses i.e. the ultimate loading capacity. The cases are categorized per failure mode and show the presence of transversal reinforcement. The member name corresponds to the names as used in the titles of the chapters of other parts of the validation: reinforced beams (RB), prestressed beams (PB) and reinforced slabs (RS). The analyses of reinforced slabs have resulted in shear failure mechanisms of a complex nature being typically a combination of one and two-ways shear. Full details have been given in the respective chapters. The table shows the maximum experimental load and the resulting maximum load according to the nonlinear finite element analyses with mean properties of material applied.

Table 2-1: Overview of the case studies categorized per failure mode, showing the ratio of the experimental failure load to the numerical failure load (using mean material properties) and statistical properties of this ratio. The loads are in kN

Failure mode		**Case**	**Transversal reinf.**	$P_{max,exp.}$	$P_{max,NLFEA}$	$P_{max,exp.}$ / $P_{max,NLFEA}$
Bending		RB1	Yes	265	268	0.99
		RB3	Yes	142	142	1.00
		PB1	Yes	1897	2044	0.93
Flexural-shear	Yielding of shear reinforcement	RB3A	Yes	155	137	1.14
	Compressive shear	PB2	Yes	6983	7413	0.94
		PB3	Yes	2313	2220	1.04
		PB4	Yes	1491	1527	0.98
	Diagonal critical crack	RB2	No	69	73	0.95
Shear failure in slabs*	Shear	RS1	No	180	111	1.62
	Shear	RS2	No	1397	1028	1.36
	Mixed mode	RS3	No	952	736	1.29
	One-way shear	RS4	No	1023	769	1.33
	Mixed mode	RS5	No	1154	867	1.33
					Mean	1.15
					CoV	0.19

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

The ratio of the experimental failure load to the numerical failure load (using mean material properties) is a standard way of defining the modelling uncertainty. Safety formats for NLFEA are used to ensure a certain safety level. Within these safety formats, material uncertainty and geometrical uncertainty are usually accounted for directly whereas all other uncertainties are accounted for by the modelling uncertainty. The sources of the modelling uncertainty comprise both the inherent variability of the experiments and the accuracy of the nonlinear finite element models.

The table shows a mean value of 1.15 (i.e. on the "safe side" of 1.00) and a coefficient of variation (i.e. the standard deviation divided by the mean value, denoted as CoV) of 0.19. Due to insufficiency of references concerning a similar subject, it is difficult to compare and verify the obtained numbers. In the available references the following can be found. Engen et al. (2016) presents similar values, but for a different modelling approach which is more suited for large scale analyses (with relatively large elements). Schlune et al. (2012) investigated the modelling uncertainty by studying the statistics of various round robin analyses results, i.e. the results of international blind prediction competitions. They reported values of the CoV in the range of 0.03 to 0.39.

As indicated by Schlune, it is reasonable to distinguish easy cases from relatively difficult modelling cases. The round robin analyses usually fall in the latter category, including over-reinforced beams, shear panels and slabs. With this in mind, we calculate the properties of the modelling uncertainty ratio per failure mode. This is presented in Table 2-2.

Table 2-2: 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

We immediately emphasize that the statistics in this table are based on very few case studies. However, although the statistical significance of these properties of the modelling uncertainty is questionable, the table shows that the "difficult cases" are the slabs failing in shear. Following the current guidelines, they give a relatively high coefficient of variation (0.27) in combination with a "safe" mean of 1.35.

3 Comparison of the design resistance using different levels of approximation

Table 3-1 summarizes the design resistances. It distinguishes analytical models following the Eurocode and the *fib* Model Code 2010. For the Model Code 2010, different levels of approximations (LoA's) have been considered, where applicable. The highest level IV of approximation employs nonlinear numerical simulations. Verification of the design resistance according to this method has been executed by means of three safety formats introduced in the Model Code 2010.

The last column in Table 3-1, shows that increasing the level of approximation indeed reveals an increase of the established design resistance. This column shows the ratio of the highest LoA IV design resistance to the lowest analytical design resistance.

Table 3-1: The design resistance for different levels of approximation (LoA's) and using different safety formats for LoA IV

Member	EC2	LoA I	LoA II	LoA III	LoA IV			$P_{Rd,\ LoA\ IV} / P_{Rd,min}$	$P_{Rd,\ LoA\ IV} / P_{Rd,max}$
		MC2010			GRF	PF	ECOV		
	[kN]								
RB1	181				190	193	203	1.12	1.12
RB3	99				116	115	120	1.21	1.21
PB1	1097				1352	1376	1514	1.38	1.38
RB3A	85.9	57.67	69.2	97.65	110	114	119	2.06	1.22
PB2	3859	3275	3968	3968	4639	4774	5391	1.65	1.36
PB3	668.4	596	761.6	998	1549	1857	1952	3.28	1.36
PB4	625.4	-	548.8	548.8	809	589	874	1.59	1.4
RB2	52	35.2	59	-	54	56	57	1.62	0.97
RS1	43.28				-	-	-	-	-
RS2	636.7	425.4	536.3	-	785	917	890	2.09	1.4
RS3	232.6	146.3	282.2	-	502	582	588	4.0	2.1
RS4	224	133	272.6	-	521	613	607	4.0	2.23
RS5	235.7	157.7	289.3	-	610	726	677	4.29	2.34

4 Comparison of failure modes

The analyses of slabs failing in shear were classified as "rather difficult cases" with relatively low predicted resistances in comparison with experimental observations (1.39). Based on the results of the selected benchmark elements, analyses of slabs resulted also in the highest ratio between the lowest and highest resistances from analytical approach as compared to numerical analyses. Besides this high range of values, the adopted approaches resulted in different or incomplete information about the governing failure mechanism. The low resistances from analytical calculations as presented in Table 3-1 and Table 4-1 (i.e. for cases RS 2 to 5) were consistently governed by one-way shear. The NLFEA, apart from revealing much higher resistances, have been concluded with additional information about the exact failure mechanism. This in turn was close or the same as the description of failure in the source references.

This also applies to the case RS1 where the analytical solution was almost four times lower than at the actual ultimate resistance to punching shear. The numerical analyses, even though terminated at a much lower load level due to shear failure, was able to correctly predict the occurrence of critical shear crack at load 95kN for NLFEA and 100 kN in case of the experiment. Table 4-1

summarizes analytically and numerically predicted resistances as well as experimental ultimate resistances of the benchmark elements. The inconsistences regarding the predicted failure mechanism are relevant only for the cases of reinforced slabs. The analytical solutions of beams failing in shear are governed by adjusting the angle of inclination of compressive struts so that no brittle failure occurs.

Table 4-1: Summary of results with specified failure mechanism

No.	Case study	EC	LoA I	LoA II	LoA III			LoA IV				Pexp
								GRF	PF	ECOV	Mean	
1	RB1	181	-	-	-			190	193	203	268	265
2	RB2	52	35.2	59				54	56	57	73	69
3	RB3	98.87	-	-	-			116	115	120	142	141.9
4	RB3A	85.88	57.67	69.15	97.65			110	114	119	137	155.7
5	PB1	1097	-	-	-			1352	1376	1514	2044	1898
6	PB2	3859	3275	3968	3968			4638	4773.96	5391	7413.75	6983.4
7	PB3	668.42	595.64	761.6	998.23			1548.74	1857.49	1951.98	2220	2313
8	PB4	625.4	-	548.8	548.8			809.43	588.6	874.19	1527.6	1491.12
		EC	LoA I	LoA II	LoA III	Regan	PS EC2					
9	RS1	-	-	-	-		43.28	-	-	-	111	180
10	RS2	636.7	425.4	536.3	-	-	741	785.6	917	889.92	1028.2	1397
11	RS3	232.63	146.3	282.18	-	464.56	454.71	501.9	581.7	588	736.2	952.38
12	RS4	223.98	132.79	272.57	-	522.98	458.01	520.8	613.4	607.3	769	1023
13	RS5	235.73	157.7	289.29	-	512.5	436.23	610	725.6	676.5	867	1153.85

Legend	
bending	
shear compression	- Failure in shear with/ without yielding of shear reinforcement and the governing failure mechanism due to crushing of concrete compressive struts.
shear yielding of stirrups	
one-way shear	
I and II way shear	- Combination of one and two shear
II way shear (punching shear)	

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