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### 44

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# <sup>47</sup> Postdiction of the flexural shear capacity of a

# deep beam without stirrups using NLFEM

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# 53 Abstract

54 A recent contest of shear tests modelling was carried out in 2019. Teams from universities and consultancies 55 around Europe were invited to predict the shear capacity of two reinforced concrete beams. The basics of the 56 numerical models should be setup according the Dutch NLFEM Guideline RTD 1016-1:2017. In the contest, two 57 reinforced concrete beams without stirrups but with a large depth (1200 mm) tested at Delft University of 58 Technology were selected as modelling target. Most participants of the contest did not get good agreement with 59 the test results. This paper presents a postdiction study on one of the two tests: H123. Based on this study, some 60 adaptations are made to the recommendations of RTD 1016-1:2017 in order to better approach the test results. 61 The intention of this contribution is to improve the existing NLFEA Guideline for practical engineering structures 62 with uncommon reinforcement layout.

63 **Keywords:** reinforced concrete, shear failure, deep beam, without shear reinforcement, NLFEA

# 64 Introduction

65 Application of smeared cracking approach based Non-Linear Finite Element Method (NLFEM) is becoming 66 more accepted in the engineering practice to model the nonlinear behaviour of structural concrete with 67 complex loading conditions and geometries. Nowadays, general design provisions for structural concrete 68 members such as Eurocode (CEN, 2005) and Model Code 2010 (fib, 2012) specify that the resistance of 69 structural concrete members can be evaluated using NLFEM when simplified analytical approach may not 70 provide an estimation with sufficient accuracy. However, modelling with NLFEM was shown to be sensitive to 71 the choices of the modelling techniques and parameters (Belletti, et al., 2010). And it becomes time consuming 72 if one needs many trials to get a reliable simulation. With the intension of simplifying the modelling process for 73 engineering application, the Dutch Ministry of Infrastructure and Water Management (Rijkswaterstaat) 74 provided a Guideline for Nonlinear Finite Element Analysis of Concrete Structures RTD 1016-1:2017 75 (Rijkswaterstaat, 2017a), which deals with the modelling of concrete structures using smeared cracking based 76 NLFEM. The intension of the guideline is provide a simple general modelling approach, which will yield reliable 77 and conservative predictions without significantly losing accuracy compared to more tailored NLFEM models.

- 78
- To gain experience and build up confidence on RTD 1016-1, several validation studies on well documented
   experiments have been published in RTD1016-2, 3A, 3B, 3C (Rijkswaterstaat, 2017b), covering three types of
- 81 structures, namely reinforced concrete beams, prestressed beams and slabs. In addition to that, two
- 82 international contests based on unpublished experiments have been organized by the User Association of
- 83 DIANA (Ensink, et al., 2015; Yang, et al., 2021b). The participants are from research institutes and engineering
- 84 companies who use NLFEM and are familiar with RTD 1016-1. Its recent edition, in 2019, aimed at the
- 85 simulation of two shear tests on reinforced concrete beams without shear reinforcement carried out at Delft
- 86 University of Technology. The specimens were selected from a large research program on shear behaviour of

87 RC slab strips (Yang, et al., 2021b). The original goal of the research program was to investigate the size effect on the shear capacity of existing RC slab bridges without shear reinforcement. 88

89

90 In the contest, two unpublished shear tests on RC beams (slab strips) with 1200 mm depth were selected. The 91 participants were asked to predict the shear capacity of these beams with any approach including numerical 92 and analytical models. When NLFEM would be applied, it was advised to follow RTD 1016-1 (but this was not 93 compulsory). The provided information before the competition included the detailed geometry of the 94 specimen, the reinforcement configurations, the test setup and the mechanical properties of concrete and 95 reinforcement (also listed in Table 1). The results of the simulations were disappointing, despite that most 96 contributions followed the aforementioned design codes and guidelines. On average, an overestimation of 97 more than 140 % of the experimental capacity was obtained from the total 10 contributions submitted to the 98 contest. A summary of the contest and the results was reported in (Yang, et al., 2021b; Yang, et al., 2021b). To 99 the owners of existing large infrastructural structures, like Rijkswaterstaat, the contest results might raise the 100 following question:

- 101
- 102

Is RTD 1016-1 still reliable, and how should RTD 1016-1 be improved with the obtained information? 103

104 Out of the two beams in the contest: H123 ( $\rho_l = 1.14\%$ ) and H352 ( $\rho_l = 0.36\%$ ), H123 was selected in the 105 present study considering that it had a more practical longitudinal reinforcement ratio. With the test results 106 known, this paper focuses on searching for an improved set of choices based on a postdiction study. As for 107 NLFEM, in addition to the loading conditions and the material properties, the choices of modelling parameters 108 and solution strategies may affect the simulation results as well. The intention of the present study is to 109 investigate the possibility of approaching the test results by adjusting these parameters and demonstrate a 110 step-by-step approach of improving the modelling flexural shear failure of deep RC members without shear 111 reinforcement. The study is mainly based on the commercially available NLFEM software package DIANA, to

112 avoid bias another software package ATENA is used in an additional validation case.

#### Shear test on beam H123 113

114 The shear test H123 was designed to investigate the size effect of a realistic configured RC member without 115 shear reinforcement. The dimensions and the reinforcement configurations of the test specimen are given in 116 Figure 1. As shown, the specimen was loaded by a single point load at mid-span, with the left side being 117 considered as test span. Half of the longitudinal reinforcement bars were bent up to the top side of the 118 specimen with the other half welded at the bar ends to the bent-up bars, in order to ensure sufficient 119 anchorage at the beam ends. The reinforcement bars were arranged in two layers, however no spacing was 120 specified between the two layers, which were connected to each other by pit welding at a few spots. 121



- 123 Figure 1. Dimensions, test conditions and reinforcement configurations of H123.
- 124

122

125 At the date of the experiment, several material properties were tested in the lab using concrete cubes of 150 126 mm. The average values of these properties, which were given to the participants of the contest, are shown in 127 Table 1. In the contest, it depended on the users' interpretation for the input in their NLFEM simulations based 128 on this information.

129



#### 130 Table 1. Material parameters obtained from lab tests.

Parameter		value	units
Concrete strength (from 150 mm cube tests)	fc,cube	86.9	MPa
Concrete tensile strength (from splitting tests of 150 mm cubes)	fct,split	5.7	MPa
Maximum aggregate size	$d_a$	16	mm
Density of concrete	$ ho_{ m c}$	23.9	kN/m³
Yield stress reinforcement	$f_{yk}$	583.9	MPa
Ultimate stress reinforcement	ftk	683.9	MPa

131

132 In the test, the beam failed at a maximum load of 445 kN. The crack patterns of the specimen at P = 400 kN

and after the formation of the flexural shear crack are shown in Figure 2. As indicated by the crack pattern at

400 kN and 445 kN, the critical shear crack initiated from the last flexural crack. Further propagation of the
 flexural shear crack resulted in failure of the specimen. The observed crack pattern is utilized for comparison

136 with the output of the simulation results.



Figure 2. Crack pattern of H123 at the last load level before failure (top figure) and the crack pattern after failure(bottom figure).

139

140 In the test, the critical shear crack formed from the already present flexural crack at 400 kN. At the

- 141 peak load, two secondary cracks developed from one of the major flexural cracks into the
- 142 compression zone and along the longitudinal reinforcement. The propagation process of these two
- branches was very unstable, which leads to the sudden drop of the bearing capacity and the increase
- of the deflection. This type of failure is typically defined as flexural shear failure as suggested by
- 145 (Yang, 2014). The unstable propagation of the flexural shear crack at failure leads to a drastic change
- 146 of the deformation and stress distribution in the whole beam. This is usually difficult to be captured
- 147 by numerical models without lack of convergency.

# 148 Modelling choices based on the RTD 1016-1 guideline

- 149 In addition to the information provided by the call of the contest (shown in Table 1), the missing parameters
- 150 for the NLFEM are obtained following the instructions of RTD1016-1, which leads to the additional material151 and analysis parameters given in Table 2.
- 152

153 Out of the listed parameters, this study will first discuss the choices of the concrete parameters which are not

- always directly reflected by lab specimen tests, such as the concrete tensile strength  $f_{ctm}$ . Next, the study
- discusses the influences of the modelling choices and analysis parameters to the simulation results. These
- parameters includes: rotating/fixed crack model, element size and bond-slip model. After comparing with the

- 157 test results, further improvements of the modelling choices and analysis parameters that do not relate to the
- 158 material properties are made.

#### 159

160 Table 2. Additional material and nonlinear analysis parameters

Parameter	Value
fcm	71.2 MPa
$G_{f}$	157.3 N/m
$G_c$	250×G <sub>f</sub>
Poisson ratio	0.15
Crack model	rotated
fctm	4.44 MPa
Young's modulus	39.2 GPa
Convergence	Energy + Force
Element size over height girder	100 mm (Mapped mesh)
Arclength	Regula
Model of Reinforcement	Bar element
Bonded/bond slip	Perfect bond

#### 161

162Regarding the concrete tensile strength, in the announcement of the contest, the tensile strength of the163concrete was reported to be 5.7 MPa, which was obtained by splitting tensile tests. However, the direct tensile164strength  $f_{ctm}$  should be used when analysing the tensile behaviour of concrete in a NLFEM. This is also165recommended by RTD1016-1 and other design codes such as the *fib* Model Code 2010. In this study the166concrete tensile strength  $f_{ctm}$  is directly derived from the concrete compressive strength.

167

168 The validation studies reported by (Rijkswaterstaat, 2017c), suggest to apply the total strain rotating crack 169 model as the default choice of the material model. As demonstrated by (Rots, 1989), the rotating crack model 170 turns out to be more robust against shear locking, thus it typically provides a lower prediction than the fixed 171 crack model type. For engineering practice, the ease of use of the rotating crack model combined with its 172 conservative prediction is considered as great benefit. Thus in RTD 1016-1 rotating crack model is suggested. 173 However, as suggested in (Yang, et al., 2017) the shear capacity of RC members without shear reinforcement 174 may be affected by the crack pattern, while rotating crack model is known not to be able to accurately 175 simulate the crack pattern. In the study, both the rotating crack model and the fixed crack model are therefore 176 employed. Following the default settings of the software, the rotating crack model is typically used in DIANA and the fixed crack model in ATENA.

177 178 179 In terms of element size, mesh sensitivity study is generally recommended by most NLFEM packages (DIANA 180 FEA, 2020; Cervenka & Jendele, 2009) as well as design codes (fib, 2012). Although with the introduction of the 181 crack band theory proposed by (Bažant & Oh, 1983), the effect of element size on the cracking behaviour of 182 concrete is taken into account for models with regular mesh layout. For shear simulations, recent studies 183 reported by (Slobbe, et al., 2013) and (Cervenka, et al., 2016) showed that the element size and orientation 184 still have a clear influence on the prediction results. Therefore, the influence of element size is chosen as a 185 modelling parameter to be studied in this paper. RTD 1016-1 suggests a maximum element size of h/6 over the 186 height of the beam. It is expected that a smaller element size will lead to more accurate prediction. Thus to 187 simulate the H123 beam with a height of 1200 mm, a larger number of elements (more than 6 elements) over 188 the height of the beam is foreseen. In (Červenka, et al., 2018), the maximum element size for members with 189 tensile cracking is suggested to be the expected crack spacing. The minimum element size was not introduced 190 yet in RTD 1016-1 in 2017. In (Bažant, et al., 1984; Červenka, et al., 2018) the minimum element size was 191 suggested to be 1.5 - 3 times the maximum aggregate size in order to fulfil the basic assumption of local

- 192 continuum theory.
- 193



194 The third study parameter is the bond-slip model of the reinforcement bars. In RTD 1016-1, a reinforcing bar is 195 modelled by embedded elements with perfect bond to concrete. It means that when the tensile strain of the 196 reinforcement becomes larger than the cracking strain of concrete, the concrete elements crack in order to 197 fulfil the kinematics conditions. The introduction of a bond-slip model when modelling the embedded 198 reinforcement leads to a more realistic crack pattern at the level of the tensile reinforcement. This is 199 considered as an option to provide a more accurate prediction to model failure modes which are sensitive to 200 the crack propagation like the H123 beam. In this study the bond-slip model proposed by (Shima, et al., 1987) 201 is selected in the reference model. The model describes the interaction between a reinforcing bar and the 202 surrounding concrete at macro level. In this model reduction of bond stress due to local failure of the interface 203 is not considered. Thus it needs only an input value for the compression strength. The bond-slip model of 204 Shima is adapted in some of the DIANA models. The bond-slip model of the MC2010 (fib, 2012), on the other 205 hand, takes into account different bond failure modes (pull-out and splitting), which results in different bond 206 stress-slip relations. With the bond-slip model suggested by Model Code 2010, the unloading of the bond 207 stress due to local failure under large slip can be modelled as well. The Model Code 2010 bond-slip model is 208 adapted in the ATENA model (Jendele & Cervenka, 2006) presented in this study.

209

210 Beside the modelling choices of the simulation, the convergence criteria may affect the results as well. They 211 may be chosen amongst displacement, force or energy based criteria, coupling two or all three criteria. For 212 each choice, the values for the tolerance of the criteria will affect the simulation results. A recent study 213 reported in (de Putter, 2020) shows this effect. In the study, it was recommended that for simulations with 214 brittle failures like the flexural shear failure of RC members without shear reinforcement, in order to continue 215 the simulation with reasonable accuracy, it is not necessary to continue simulation steps with all the criteria 216 fulfilled. In RTD 1016-1, using the maximum values of 0.001 for the energy criterion and 0.01 for the force 217 criterion is recommended. Imposing multiple convergence criteria in a simulation often leads to unnecessary 218 load steps or even early divergence in critical load step(s). In engineering practice, in order to go over such 219 critical load step(s), a choice of relaxing the convergence criteria or reducing the number of criteria is 220 employed. In this paper, with the intention of simplifying the study, only the Energy norm (criterion) amongst 221 the other criteria is selected to study further, to be in line with RTD 1016-1.

222

In test H123A, the load was applied by an actuator using displacement control due to safety considerations.
 However, most structures in engineering practice are designed for force controlled loads such as gravity, wind
 pressure or traffic loading. Ideally NLFEM simulations loaded by either displacement or force control should
 give comparable results. Taking that into account, in this study both loading methods are applied to the same
 model in order to evaluate the potential difference between the two loading approaches. They are
 distinguished by a (displacement control) or b (force control) after the simulation No. when they are referred
 in the text.

# 230 Effect of element size

The effect of element size in the simulation is studied first in three simulations using element sizes varying from 200 mm to 50 mm, which leads to 6 to 24 elements in the height of the beam. The configurations of Simulations 01 – 03 are listed in Table 3. The basic modelling choices follow RTD 1016-1 and Table 2. The convergence limit of Energy is set to 1.0E-4. According to RTD 1016-1, all load steps should converge till the ULS is reached with a convergence value for Energy tolerance of 1.0E-3.

236

237 Table 3. Results of element size simulations

Simulation No.	Element size [mm]	P <sub>max,disp</sub> 1) [kN]	P <sub>max,disp</sub> /P <sub>Test</sub> [-]	P <sub>max,forc</sub> <sup>2)</sup> [kN]	P <sub>max,disp</sub> /P <sub>Test</sub> [-]	P <sub>max,forc</sub> /P <sub>max,disp</sub> [-]
D01	200	432	0.97	441	0.99	1.02
D02	100	365	0.82	346	0.77	0.95
D03	50	301	0.68	279	0.63	0.93

238 239 240 P<sub>max,disp</sub> is the maximum applied load before the convergence criterion was reached. In the simulation, the load was applied by displacement control. Accordingly, the simulations are named as D01a – D03a.

- 2) *P<sub>max,forc</sub>* is the maximum applied load before the convergence criterion was reached. In the simulation,
   the load was applied by force control. Accordingly, the simulations are named as D01b D03b.
- 243

Table 3 shows the results of the three simulations with different element sizes. The maximum load level of
Simulation D01a is close to the experimental load level. Using a finer mesh results in a reduction of the
ultimate load till a maximum load of 279 kN from D03b. Both smaller element models show a lower ultimate
load with force controlled (b series models) than with displacement controlled loading (a series models).
However the difference between both loading methods is rather small, at most 7.3%. Figure 3 shows the crack
patterns of Simulation D02 using both displacement control and force control analyses.

250

The results of the simulation indeed demonstrates the influence of element size upon the failure load. However this conclusion is rather different from what was reported in (Cervenka, et al., 2016). In the study of Cervenka et.al., models with larger element size leads to lower capacity. However, our simulations using both loading methods show that by using a larger element size, higher capacity is predicted. Further study is still needed to investigate the reason of the mesh dependency in both studies and the possible solutions to resolve it.



Figure 3 Crack pattern just before ULS Load level from Simulation D02a displacement controlled (top) and from
 Simulation D02b force controlled (bottom).

263

264 In terms of crack patterns, Figure 3 shows that the different loading methods give rather similar predictions, 265 with slightly different magnitude of the maximum crack strains before failure. The red and yellow coloured 266 strains show that cracks initiate along the longitudinal reinforcement. Similar cracking is reported in (de Putter, 267 2020). Further loading leads to the loss of load bearing capacity. As comparison, the crack pattern of H123 at 268 the load step before failure (P = 400 kN) is shown in Figure 2. The crack pattern of both methods turns out to 269 be rather comparable to the measured crack pattern, although the final crack pattern at failure cannot be 270 shown because the critical shear crack only occurred at the very last load step which leads to divergence of the 271 simulation. Most cracks before failure were flexural cracks, with which limited rotation of the crack is 272 expected. Nevertheless, at several spots at the bottom of the flexural cracks, initiation of longitudinal cracks 273 can be observed along the reinforcing bars. This observation shows that an accurate bond-slip model is needed 274 as an additional input option to get a more realistic behaviour around the reinforcement bar in order to obtain 275 a more accurate failure load.

# 276 Bond slip model and analysis parameters

277 In Simulation D01 – D03, no clear shear crack was observed before failure. This could also because of a too 278 relaxed convergence criterion. In order to avoid the discussions about the choice of the convergence criterion, 279 another set of simulations (Simulations D04 – D06) was developed. From Simulation D01 – D03 it was 280 concluded that an accurate bond-slip model might be a critical requisite for the simulation. Thus, the bond-slip 281 model of Shima (Shima, et al., 1987) is also introduced in the new set of simulations. The FE models have a 282 mesh with element size of 50 mm. This choice is based on the assumption that models with finer mesh size are 283 able to represent the behaviour of a structure in more detail, thus has the potential to provide better 284 accuracy. The main difference between the three simulations is the value of the energy norm. In Simulation 285 D04, the energy norm is set to 1.0E-3 as suggested by RTD 1016-1, while the other two simulations employ an 286 energy norm of 10.E-4 and 10.E-5, respectively, see Table 4. Both the displacement control and force control 287 method are used in these simulations.

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#### 288 289

Table 4. Results of analyses on convergence energy criterion, all simulations use element size of 50 mm.

Simulation No.	Energy norm	Arc length control	P <sub>max,disp</sub> [kN]	P <sub>max,disp</sub> /P <sub>Test</sub> [-]	P <sub>max,forc</sub> [kN]	P <sub>max,forc</sub> /P <sub>Test</sub> [-]	P <sub>max,forc</sub> /P <sub>max,disp</sub> [-]
D04	1.0E-3	Automatic	392.3	0.88	348.5	0.78	0.89
D05	1.0E-4	Automatic	348.7	0.78	372.5	0.84	1.07
D06	1.0E-5	Automatic	378.2	0.85	364.4	0.82	0.96
D07	1.0E-4	Manual	-	-	456.9	1.03	-
D08	1.0E-5	Manual	-	-	405.9	0.91	-

#### 290

Table 4 shows the results of the second series of simulations. Comparison of the results with the 50 mm element size simulation in table 3 and the results of table 4 yields the first conclusion that the ultimate load level is increased by implementing the bond-slip model. A second conclusion is that the load level does not change significantly when the tolerance value is increased. Both loading methods show this aspect. The difference between model predictions and test results is around 20%. Although more simulations are still needed to draw solid conclusions, one may consider that introduction of a bond-slip model may change the

results of the simulations, and could provide a more reasonable estimation.

298

The crack pattern of Simulation D05 is shown in Figure 4. Compared to Figure 3, a more developed pattern of cracks is present by the introduction of the bond-slip model. Also the crack spacing at mid-depth of the beams turns out to be more representative to that observed in the experiments in Figure 2. Compared to the perfect bond model, a realistic bond-slip model enables the localization of crack opening amongst the elements at the rebar level, that clearly results in a more realistic crack pattern. Thus this can be seen as an improved result.

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305 306 307

Figure 4. Overview crack pattern force controlled method for Simulation D05.

308 The ratio between the maximum load observed in the experiments (445 kN) and the simulations (348 – 392 309 kN) is still low. A further improvement was made by adjusting the arc length analysis. As suggested by 310 (Verhoosel, et al., 2008), the adaption of arc length analysis may improve the stability of the analysis, thus 311 being able to obtain the snap-back behaviour of the structure. RTD 1016-1 recommends arc length analysis to 312 improve the convergence of the simulation. As an additional improvement, the default automatic arc length 313 approach is replaced. In the new simulations, the control displacement is set to the bottom fibre of the left 314 half length of the girder. Thus, the crack length of the bottom fibre of the concrete becomes more important in 315 determining the load factor in the arc length calculation. Two new simulations are made in the additional 316 study, Simulation D07 and D08 in table 4. Since from the previous study, the different loading approaches 317 show very limited influence to the simulation results, in Simulation D07 – D08 only force control was applied. 318 When comparing with the test results, the simulated ultimate load reached 90% of the experimental result and 319 on the lower side. Hence, the adjustment of the arc length approach can be considered as a further 320 improvement.

321

322 As was done in (Belletti, et al., 2010) and (Rijkswaterstaat, 2017b), test H123 is also simulated with the 323 software package ATENA. Being different from the models in DIANA, the rebars in ATENA are modelled using embedded elements, but incorporated explicitly with a bond-slip model. The bond-slip model proposed in the 324 325 Model Code 2010 (fib, 2012) is employed. The bond-slip model suggested in Model Code 2010 includes the 326 softening parts of bond-slip relationship. And it can consider different bond failure modes. The model is 327 numbered as Simulation A01 in this paper. The ultimate load level of Simulation A01 is 479 kN, which is similar 328 to the simulation with the Shima bond-slip model and – more important – still a reasonable estimation. The 329 corresponding crack pattern is given in figure 5. As a standard approach in ATENA, a fixed smeared crack

- 330 model with a crack width based shear retention factor is used in the simulation. Besides, when the predefined
- convergence criterion is not reached within 50 iterations, the program accept a relaxed convergence criterion.
- The crack patterns of the specimen before and after the peak load are indicated in Figure 5. With the fixed
- 333 crack model, the crack pattern does not change with the change of the principal stress direction, thus a 334 realistic flexural shear crack can be simulated as shown in Figure 5. In addition, in Simulation A01, it is possible
- realistic flexural shear crack can be simulated as shown in Figure 5. In addition, in Simulation A01, it is possible
   to reach the descending branch of the load deflection relations, see figure 6. That provides additional
- 336 confidence of the simulations provided by DIANA, in which further loading were not possible. The crack
- 337 pattern given by the simulations of ATENA compares well with the crack pattern after failure in figure 2. In
- 338 ATENA, a realistic crack pattern is also found with a reference model with perfect bond. The simulation is not
- demonstrated in the paper.



Figure 5. Crack pattern (crack width > 0.01 mm) and horizontal strains in ATENA simulation (Simulation A01) with MC2010 bond splitting model at ULS load level (top, 479 kN) and just after maximum load (bottom) (2D-model with 100x100 mm quadratic elements, displacement control, Arc Length solution method).



345

Figure 6. Load displacement diagrams of Simulation D07 with Shima bond slip model using DIANA and Simulation A01
 with MC2010 A01 bond-slip model using ATENA.

348

Based on the discussion above, Simulation D07 and Simulation A01 are chosen as the final representative
 models. Figure 6 shows the load-displacement diagram of both models and that from the experiment. The
 displacement shown in the figure is the maximum deflection under the loading point. The comparison shows
 that both Simulation D07 and A01 provide rather similar load-deflection relationship, and they compare well

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- 353 with the experimental observation. The main difference lies in the stiffness, which can be attributed the long
- term deformation of concrete during the loading process. Comparing the stiffness of the first branches in
   Figure 6 shows a reduction with a factor 0.74 from simulation to the test (Yang, et al., 2021b) and 0.77 in the
- 355 righte 0 shows a reduction with a factor 0.74 from simulation to the test (rang, et al., 2021b) and 0.77 in the 356 second branches. In an experiment that takes several hours a certain amount of creep will occur, which can be
- 357 simulated with a lower elastic modulus. Similar observation has been reported in (Cervenka, et al., 2016).
- 358 Besides, the first cracking loads in the simulations are higher than that observed in the experiments. This could
- 359 partly relate to the same phenomenon. Under sustained loading a reduced tensile strength may be expected
- with comparable deformation as suggested by (Rusch, 1960; Reinhardt & Cornelissen, 1985). In addition, the
- 361 variation in concrete strength may result in a lower fracturing load (Tran & Graubner, 2018).

# 362 Discussions and recommendations

- In this study, the following steps are made to improve the simulation, which can be summarized as advices forthe engineering practice when dealing with similar simulations:
- 365 1. Evaluation of mesh dependency.
  - 2. Introduction of a bond-slip model when crack propagation at the level of the longitudinal reinforcement is critical to the failure.
- 3683. Further refinement of solving strategy (for example, manual selection of control displacement in an arc length analysis).
- 370 4. For better crack pattern adaption of fixed crack model.
- 371

366

367

372 In this paper it is shown that indeed the influence of the so called modelling choices that were previously 373 considered to be less important may affect the simulation results considerably in this special situation. It 374 shows the risk of using NLFEM when it is applied on modelling of structure members with large dimensions 375 and brittle failure mode. In addition to that, the proposed improvement steps, have shown to give a realistic 376 approach to further improvement of the simulations. With this approach, simulations with good agreement to 377 the experimental results can be obtained. Despite that, as the intension of the paper is not to provide a 378 systematic study on how to accurately perform non-linear simulation of the flexural shear behaviour of RC 379 beams, further studies on the effect of the following aspects remain open:

- **380** 1. The effect of element size to the simulation of structural members with large dimensions.
- 381As discussed earlier, the presented study shows a clear influence of element size on the simulation382results. However, an opposite conclusion from that reported in a previous study (Cervenka, et al.,3832016) was obtained. To get a clear picture on the influence of element size in shear simulations,384further study is still necessary.
- 385 2. The effect of the bond-slip model in the simulation.
- In this study, better results were reported after the introduction of a bond-slip model. Further
   validation is needed on the choice of bond-slip model and the robustness of the simulation when such
   bond-slip model is introduced.
- 389
  3. In the simulations, a dowel crack along the longitudinal reinforcement was often observed during the propagation of the critical shear crack. As the embedded reinforcement is usually considered as bar elements, the bending stiffness, which is considered as the reason of dowel cracking, cannot be simulated with this element type. Introduction of a beam type reinforcement element may further improve the simulation.

# 394 Conclusions

- This paper presents a post-diction study on the shear failure of specimen H123, with the intention to provide a simple stepwise approach to improve simulations including flexural shear failure. It is demonstrated that the simulation of the experimental results of H123 can be improved by refining modelling choices. The study provides a practical stepwise example starting with a basic model, proposed by RTD 1016-1. The following conclusions can be drawn:
- 400 1. The influence of the element size on the model simulation turns out to be rather complicated. This
   401 study yields a different conclusion than the earlier study reported by (Cervenka, et al., 2016). Further
   402 study on this topic is still needed.

- 403
   2. Both the total strain rotating crack model and the total strain fixed crack model may provide sufficient accuracy. However, the fixed crack model is able to provide a more realistic crack pattern.
- 4053.For the simulation of the shear behaviour of RC members without stirrups, introduction of a bond-slip406model may improve the simulation results. Within this study both the bond-slip model suggested in407Model Code 2010 and the model proposed by Shima give predictions closer to the test results than408simulations with perfect bond.
- 409 4. More beams without stirrups should be simulated to quantify the model uncertainty for NLFEM410 simulation on the shear behaviour of RC members.
- 411 5. The work presented in the paper can be interpretated as a warning for users of RTD 1016-1 in case of
  412 deep beams without shear reinforcement. The simulation of such type of structures turns out to be
  413 more sensitive to the choices of the modelling parameters than for other types of structures.

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