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NONLINEAR ANALYSIS OF SHEAR CRITICAL RC MEMBERS USING CURRENT FE SOFTWARE

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Abstract. *The shear behaviour of concrete has been widely studied in the field of structural engineering, yet it is still considered an unresolved issue. In one hand, the design codes provide diverse methods to estimate the shear capacity of concrete with different results. On the other hand, analytical models to assess the performance of shear critical concrete members have been proposed by several authors. These models have been developed using different assumptions. Some of the theories have been implemented in finite element software packages that are used by engineers in academia and in the industry.*

This study presents an investigation of the state-of-the-art software packages for the analysis of shear critical concrete elements. The theory implemented on each software is detailed and analysed. To evaluate the capacities of the packages, some benchmark tests have been used for both monotonic and cyclic loading cases. The presented results show the capabilities and limitations of the programs from the experience obtained during the duration of this work. Just as well, the results among the programs are compared among themselves as well as the results from other authors.

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1 BACKGROUND AND MOTIVATION

Even though the mechanics of shear behaviour in reinforced concrete (RC) has been studied for a long time, it is still an unsolved problem. On one hand, many design codes provide different methods to define and estimate the shear capacity of concrete. Among these codes there are the National Building Code of Canada by the CSA [1], AASHTO LRFD Bridge Specifications by AASHTO [2], and the Fib Model Code by the International Federation for Structural Concrete [3] and ACI 318-11 by the ACI Committee 318 [4]. These codes and their methods of design generally lead to very different shear capacity estimations for RC elements (and therefore, different amount of required transverse reinforcement). On the other hand, several analytical models have been proposed in order to assess the performance of shear critical members. Authors such as Litton [5], Bazant and Oh [6], Cervenka [7], Vecchio and Collins [8], De Borst [9], Hsu [10] and Maekawa *et al.* [11], have developed methods to determine the shear behaviour of RC, using diverse types of assumptions in the process. As defined in Ceresa *et al.* [12] [13], in the framework of smeared crack theory for RC the following main models can be identified:

- Compression Field Theory (see Collins [14], Vecchio and Collins [15])
- Modified Compression Field Theory, MCFT (see Vecchio and Collins [16])
- Rotating-Angle Softened Truss Model, RA-STM (see Belardi and Hsu [17], Belardi and Hsu [18], Pang and Hsu [19])
- Fixed-Angle Softened Truss Model, FA-STM (see Pang and Hsu [20], Hsu and Zhang [21], Zhang and Hsu [22])
- Cracked membrane model (see Kaufmann and Marti [23])
- Softened membrane model (see Zhu [24], Hsu and Zhu [25])
- Disturbed Stress Field Model, DSFM (see Vecchio [26], Vecchio and Lai [27]).

Some of these analytical models have been used in the development of commercial finite element (FE) software packages used by researchers and engineers in both industry and academia. Yet the complete range and capabilities of these programs is not completely investigated. Therefore, the aim of this work is to provide a thorough review of the theoretical models implemented in state-of-the-art FE packages for the nonlinear analysis of shear critical RC members. Similarly, verification examples are provided for the assessment of the accuracy, performance, advantages and limitations for the studied programs. This is performed for both the monotonic and the cyclic behaviour of reinforced concrete.

2 BRIEF OVERVIEW OF THE ANALYZED SOFTWARE

Three are the software packages taken into account in this study: VecTor2 [28], DIANA [29] and ATENA[30]. The detailed review of the theory implemented in each software can be found elsewhere [31].

VecTor2 is a nonlinear finite element program for the analysis of two-dimensional RC member structures subjected to in-plane normal and shear stresses. It models cracked concrete as an orthotropic material with smeared, rotating cracks. It considers the element to act as in the plane stress idealization. The program uses an incremental total load, iterative secant stiffness algorithm to produce the nonlinear solution. The two main bases of the modelling of RC are the MCFT and the DSFM. The latter two are then complemented in VecTor2 with other models for compressive and tensile behaviour of concrete, tension softening and tension stiffening. The main aspects that greatly affect the response of RC elements are the crack width check, the crack slip calculation and the models for hysteretic response of concrete and reinforcement steel, using models that consider the damage due to load reversal, repetitions and plastic behaviour.

The software DIANA uses two smeared cracking models to simulate the behaviour of RC, mainly focused in the monotonic response: the Multi-Directional Fixed Crack Model and the Total Strain Crack Model. The Multi-Directional Fixed Crack Model does not allow for a clear modelling of the compressive behaviour of concrete and was not used in this study. A further constitutive model is presented in DIANA as the model more suitable for the cyclic response of concrete – the Maekawa Cracked Concrete model. While working in a two-dimensional model, DIANA offers the option of working with an element based either in the plane stress or plain strain idealisation. The selection of the crack direction has been determined to be one of the main parameters that affect the response of RC in FE modelling. The Total Strain Crack Model and the Maekawa Cracked Concrete model are capable of modelling both the fixed and the rotated crack approach. If the fixed crack has to be used, a shear retention model has to be defined to calculate the shear stresses that exist when the strain field analysed does not coincide with the principal stresses direction. It has been found that fracture mechanics principles, through the concepts of fracture energy and crack bandwidth, are widely used to describe the stress-strain relations of concrete. An important concept offered by DIANA is the possibility of modelling the discrete steel bars using the embedded reinforcement approach.

The program ATENA includes several constitutive models that can be used in order to predict the behaviour of RC. For the behaviour of concrete, two models have been considered – the SBETA and the Rankine fracture-plastic constitutive models. Both models include the possibility of modelling the crack direction as fixed or rotated. Contrary to DIANA, ATENA only includes on model for the shear retention calculation when the fixed crack direction is used. The SBETA model contains the nonlinear behaviour of concrete in compression, the reduction of strength due to cracking, the use of fracture mechanics for the tension softening and a biaxial failure criteria. The unloading and reloading in the SBETA model are performed linearly from the maximum experienced strain to the origin. The Fracture-plastic model is a combination of a fracture model based on a smeared crack formulation and crack band model for the tensile behaviour and a plasticity model that considers concrete crushing for the compressive behaviour. The loading and reloading for compression considers the plastic strains, while for the tension, the reloading and unloading can be performed as linear to the origin or parallel to the initial stiffness. The software also includes several possibilities to model the steel reinforcement.

3 MONOTONIC ANALYSIS OF SHEAR-CRITICAL BEAMS

The software analysed in this study has been evaluated through different tests. The programs have been first evaluated with an analysis of beams under monotonic load where the shear behaviour is critical. The series of beams tested by Bresler and Scordelis [32] and the beam used for a blind prediction at the University of Toronto in 2015 [33] were the benchmark tests for this evaluation. The finite element models for each of the programs are formulated and presented, including the evaluation of the most adequate material models for these numerical tests. The parameters that affect the response of the RC element modelling the most have been identified, such as the definition of the tensile strength and the fracture energy and the selected crack rotation approach (fixed or rotated). The results from the FE packages are then compared.

3.1 Beams tested by Bresler and Scordelis

The first studied beams are those tested by Bresler and Scordelis [32]. These beams have been often used as benchmark, as shown in [34] [35]. These beams were simply supported and subjected to concentrated load at midspan, with a span-depth ratio ranging from 3.3 to 5.7. They were designed such that the shear behaviour is critical, since they are heavily reinforced in flexure but with light to no amounts of shear reinforcement. The general properties of the beams are presented in Table 1, where the initial elastic modulus E_c, the tensile strength f_t and the fracture energy G_f are estimated as recommended in [36] and the Model Code [3]. The main focus was on the modelling of beams A-1 and OA-1. Using these results, the rest of the beams were analysed [31].

Beam	f'_c (MPa)	Bottom bars*	Top bars*	Stirrups*	E_c (MPa)	f_t (MPa)	G_f (kN/m)
A-1	24.1	4 #9	2 #4	#2@210mm	24816	1.62	0.129
OA-1	22.6	4 #9	-	-	24031	1.57	0.128
A-2	24.3	5 #9	2 #4	#2@210mm	24919	1.63	0.130
OA-2	23.7	5 #9	-	-	24609	1.61	0.129
A-3	35.1	6 #9	2 #4	#2@210mm	29948	1.96	0.139
OA-3	37.6	6 #9	-	-	30997	2.02	0.140

* bar # f_y(MPa), ϕ(mm): f_y = 325, ϕ = 6.35; #4: f_y = 345, ϕ = 12.7; #9: f_y = 555, ϕ = 28.65

Table 1: Properties of the Bresler-Scordelis beams, whose cross-section is b×h = 305mm×560mm.

Models and results with VecTor2

Beams A-1 and OA-1 have been modelled using 4-node rectangular elements for the concrete. The size of the element was taken as 25×25mm so that a uniform and sufficiently discrete mesh is used. Due to the symmetry of the beam's geometry, the boundary conditions and the loading conditions, only half of the beam has been modelled. The beam is modelled with bearing elements which are capable of simulating the loading plates and avoid any local issues in the point of support or load application. The boundary conditions consisted of a vertical support under the lower bearing element and of a lateral support throughout the middle of the beam due to symmetry. The reinforcement was modelled as discrete truss elements. The loading was applied with an imposed vertical displacement in the middle of the beam. By doing a displacement control of the test, the nonlinear post-peak response of the beam can be more precisely estimated. With the constitutive relationships listed below it was possible to obtain numerical results that more accurately reproduce the behaviour of these RC beams. All the details can be found in [31].

- Compression before peak: Popovics.
- Compression post-peak: Modified Park-Kent.
- Compression softening: Vecchio 1992-A (e1-e2-Form).
- Tension stiffening: Modified Bentz 2003.
- Tension softening: Nonlinear (Hordijk).
- Confined strength: Kupfer/Richart.
- Crack stress calculation: DSFM.
- Crack slip calculation: 2 mm maximum crack width.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: elasto-plastic behaviour with strain hardening in both tension and compression.

The numerical results showed that the selected models are able to replicate well the behaviour of the beams until the 70% of the peak-load reached during the experimental test. The nonlinear response is modelled correctly, where the stiffness of the specimens are well reproduced. However, the failure of the element is obtained prematurely. Convergence problems were observed in analyses of all the Bresler-Scorderlis beams.

Models and results with DIANA

Similarly to the models in VecTor2, 4-node rectangular 25×25 mm size elements were used for the modelling of concrete. Just as well, only half of the beam was modelled with the same support conditions and it included the bearing elements at the point support and the load application point in order to avoid localisation problems. The reinforcement steel was modelled using the discrete embedded reinforcement concept. With this modelling option, the reinforcement bars in the beam do not necessarily have to line-up with the nodes of the finite elements. The constitutive relationships used in DIANA for the RC beams under the monotonic load is the Total Strain Crack Model. It was deemed appropriate since it is capable of modelling several different stress-strain relations for tension and compression, considers the effects of cracking and confinement and the possibility of using several shear retention models. The following list presents the selected models (all the details are given in [31]):

- Compression model: Thorenfeldt.
- Tension model: Hordijk.
- Model for lateral cracking: JSCE 2012.
- Confinement model: Selby and Vecchio.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: Von Mises plasticity with strain hardening in both tension and compression.

For the beams A-1 and OA-1, a comparison was performed between the results of using the fixed crack model or the rotating crack model. For the beam with transverse reinforcement (A-1), the best result was obtained using the fixed crack approach with the aggregate-based shear retention model. A normal size aggregate (15 mm) was assumed. For the beam without stirrups (OA-1), this modelling strategy overestimated the capacity of the beam. Better results were obtained using the fixed crack approach with the damage-based retention model. Two difficulties appeared during the numerical simulations. First, the displacement corresponding to the peak load was generally lower than the experiment. Therefore, the degradation of stiffness at the ultimate stages of loading was not precisely modelled. Second, the models presented convergence problems at these stages, and, in some cases, the ultimate load was not reached.

Models and results with ATENA

The mesh of the models for ATENA has been generated with the same assumptions adopted in VecTor2 and in DIANA. Similarly to DIANA, the reinforcement steel was modelled using the discrete embedded reinforcement concept. ATENA has two models available for the RC elements: SBETA and Fracture-Plastic. Both were used to model the beams. The next constitutive model were used:

- Compression model for pre-peak branch: Model Code 2010 (for SBETA) and Hordijk model.
- Compression model for descending branch: Linear descend.
- Tension model: Exponential.
- Model for reduction of compressive stress due to cracking: Vecchio and Collins [16] with $c = 0.80$.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: Bi-linear response with strain hardening, active in both tension and compression.

Both the SBETA and the Fracture-plastic models are capable of using a fixed or rotating crack approach, so both approaches were tested. ATENA uses some formulas to define the missing parameters based on the compressive strength inputted into the software. In order to determine which values represent better the results from laboratory test, a simple sensitivity analysis was performed for beams A-1 and OA-1. The two parameters modified were the tensile strength of concrete (f_t) and/or the fracture energy (G_f). Only for the Fracture-plastic model, another parameter was the inclusion of the tension stiffening as defined in ATENA with a recommended value of c_t=0.40 (Table 2). This parameter represents the minimum value (40% of f_t) for the reduction of the tensile stress after cracking occurs.

Parameter	A-1		OA-1	
	Default	Modified	Default	Modified
f_t (MPa)	2.23	1.62*	2.14	1.57*
G_f (kN/m)	0.058	0.129	0.053	0.128
c_t	0	0.40	0	0.40

*value used in VecTor2 and DIANA

Table 2: Default and modified parameters for the sensitivity analysis in ATENA

Comparing the numerical results it was derived that the Fracture-plastic model was capable of precisely model the nonlinear behaviour of these beams. The models using the fixed crack approach with the modified tensile strength showed the most promising results. Furthermore, the tension stiffening parameter works fairly well for the beam with the transverse reinforcement. Therefore, the simulation of the Bresler-Scordelis beams was performed with the Fracture-plastic model, the modified tensile model and the inclusion of c_t=0.40 for the beams with transverse reinforcement only. The numerical simulations with this software showed accurate results for most of the cases. The models used were able to perform the complete analysis without convergence problems.

Comparison of the results among the programs

The results for the A-1 and OA-1 beams are compared in Figure 1. It is important to note that these results were obtained with only the experience and knowledge obtained during the time of this study. It is believed that these predictions could be improved given the time and experience to work more and perfect the models.

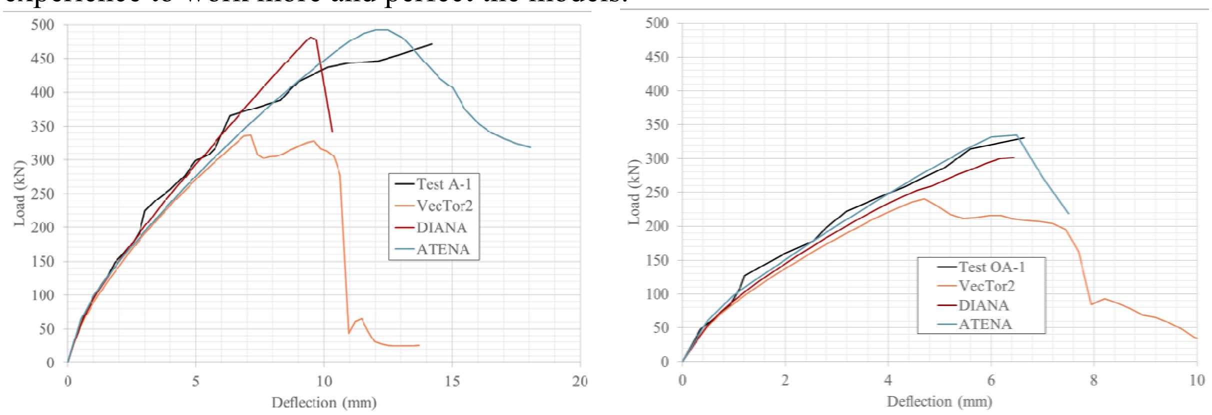

Figure 1: Comparison of the load-displacement curves for the beams A-1 (on the left) and OA-1 (on the right)

Comparison with other authors

A re-examination of these beams was performed by Vecchio and Shim [35]. An analysis of the beams was performed using VecTor2 and their results for beams A-1 and OA-1 are presented in Figure 2, as well as the results of this research with VecTor2. The results of Vecchio and Shim represent an enhanced prediction of the behaviour of the beams with respect to the numerical results obtained by the Authors. This confirms the idea that the models developed in this study could be improved with more expertise and software knowledge.

Very good numerical simulations were obtained by Kagermanov and Ceresa [37] [38] with a fiber-section model with an exact shear strain profile for 2D RC frame structures.

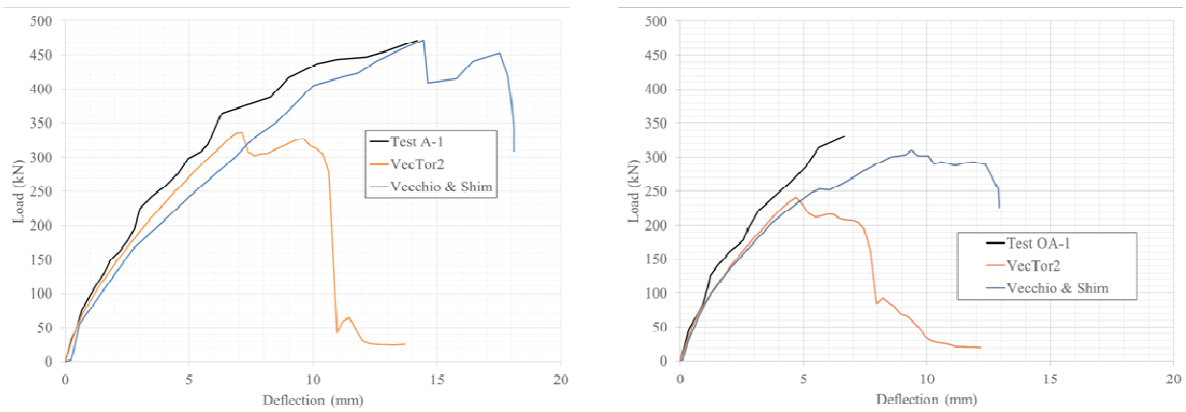

Figure 2: Comparison of the A-1 and OA-1 beams to the models of Vecchio and Shim [35]

3.2 Toronto beam

The University of Toronto, as described by Collins *et al.* [33], tested a deep beam in order to evaluate the shear behaviour of thick slabs, since various design code procedures give significantly different estimates of the shear capacity. These thick slabs may be present in several structures such as mat foundations for high rise buildings or intake structures for hydroelectric projects. The main issues with the codes generally used is that the size effect is not considered.

The test simulated a strip cut of a 4m thick slab, as shown in Figure 3. The properties of the specimen are presented in Table 3. The applied load was not located in the middle of the beam. The right side contained no shear reinforcement, while the left side contained the minimum shear reinforcement recommended by the ACI 381 Code. A blind prediction was established in order to assess the ability of the engineering profession (industry and academia) in the estimation of the shear response in this type of structures.

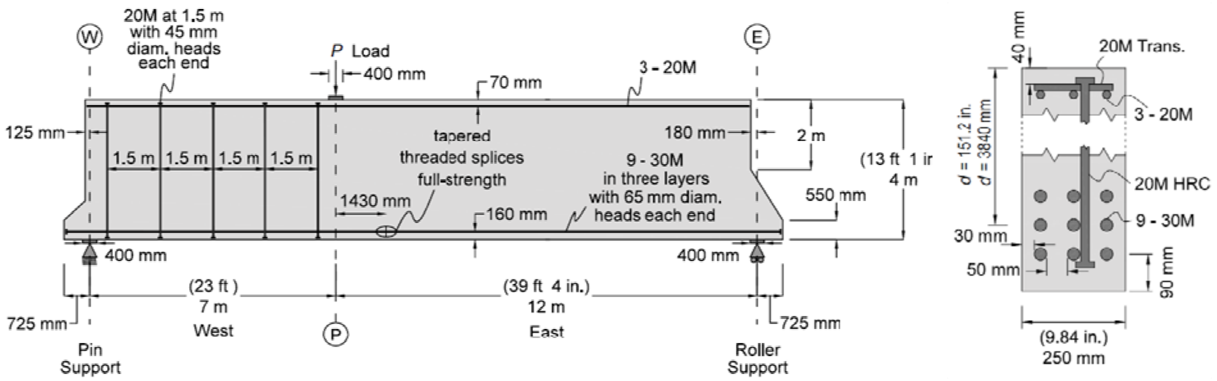

Figure 3: Elevation (on the left) and cross-section (on the right) of the Toronto beam

Model and results with VecTor2

Given the depth of the beam, the size of the elements of the mesh was 200×200 mm for the 4-node elements. The complete beam is modelled as simply support element. All the reinforcement was modelled as discrete truss elements. Also, a bearing elements was used under the load application point to avoid localization issues. Finally, the beam was modelled under an imposed vertical displacement. Note that for this beam the self-weight is significant. Hence, a first loading stage of only the weight of the beam was performed and then the imposed vertical displacement started to increase.

The constitutive models selected were those which model more realistically the behaviour of RC elements, including the models for compression softening and for the confined concrete:

- Compression before peak: Popovics.
- Compression post-peak: Modified Park-Kent.
- Compression softening: Vecchio 1992-A (e1-e2-Form).
- Tension stiffening: Modified Bentz 2003.
- Tension softening: Nonlinear (Hordijk).
- Confined strength: Kupfer/Richart.
- Crack stress calculation: Basic (DSFM/MCFT).
- Crack slip calculation: 1 mm maximum crack width.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: elasto-plastic behaviour with strain hardening in both tension and compression.

Span (mm)	f'_c (MPa)	Bottom bars*	Top bars*	Vertical bars*	E_c (MPa)	f_t (MPa)	G_f (kN/m)
19000	40	9 – 30M	3 – 20M	20M @1.5 m	32496	2.09	0.142
Bar	ϕ (mm)	Area (mm^2)	f_y (MPa)	ε_{sh}	f_u (MPa)	ε_u	
20 M	19.5	300	522	0.017	629	0.20	
30 M	29.9	700	573	0.014	685	0.18	

Table 3: Properties of the Toronto beam, whose cross-section is b×h = 250mm×4000mm

Model and results with DIANA

The mesh was generated automatically by the software using 4-node rectangular element of 100×150 mm. The modelling of the reinforcement steel was performed using the discrete embedded reinforcement concept. The support conditions and the modelling of the bearing elements were the same as in VecTor2. The analysis, differently from the ones performed with the other two programs, required an imposed vertical load instead of a displacement. This was necessary due to how DIANA performs the staged loading, first from the self-weight and later from the applied load.

The constitutive model used in DIANA is the Total Strain Crack Model, as for the Bresler-Scordelis beams. The laws selected were as follows:

- Compression model: Thorenfeldt.
- Tension model: Hordijk.
- Model for lateral cracking: JSCE 2012.
- Confinement model: Selby and Vecchio.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: Von Mises plasticity with strain hardening in both tension and compression.

As observed for the Bresler-Scordelis beams, the fixed crack model was deemed appropriate to model beams in DIANA. Similarly, the results of these beams showed that for lightly reinforced structures, the shear retention model based on damage provided the best suited results. Hence, the Toronto beam was modelled using this approach.

Model and results with ATENA

The FE model developed in ATENA is equal to the model developed in Vector2. The discrete embedded reinforcement model was used for the steel reinforcement. The beam was loaded imposing a vertical displacement, as in VecTor2.

From the two models available in ATENA for the RC elements, the Fracture-plastic model is used. The constitutive models considered in the analysis are listed below:

- Compression model for pre-peak branch: Hordijk model.
- Compression model for descending branch: Linear descend.
- Tension model: Exponential.
- Model for reduction of compressive stress due to cracking: Vecchio and Collins [16] with $c = 0.80$.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: Multi-linear response with strain hardening, with the steel being active in both tension and compression.

As concluded in the analysis of the Bresler-Scordelis beams, the fixed crack approach resulted in the best numerical predictions, and it is used for this beam as well. The tension stiffening parameter (c_t) was not included into the analysis of the Toronto beam. The rational for this decision was that the beam can be considered as very lightly reinforced, as seen in Figure 3. Even though the west side of the beam has transverse reinforcement, it is too far apart and the east side has no vertical reinforcement. Therefore, the inclusion of a minimum limit for the tensile strength after cracking could be overestimating the capacity of the beam.

Comparison of the results among the programs

The load-displacement curve calculated from the three software is presented in Figure 4. The load in the vertical axis only accounts for the load applied from the jacks during the test. It is because of this that the curve does not start at the origin but at the displacement that occurred due to the self-weight of the beam. The three programs predicted a similar the behaviour for the Toronto beam, in particular for the post-cracking stiffness.

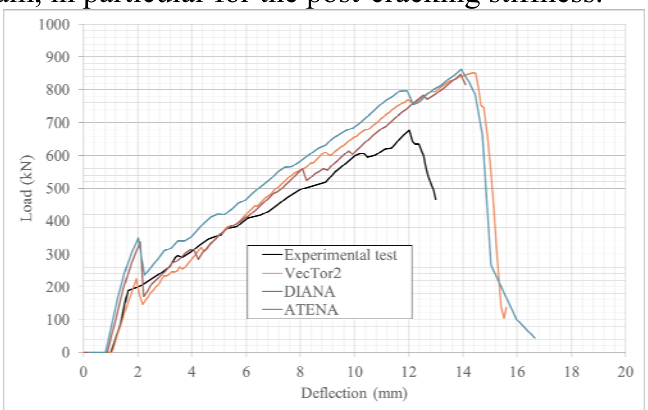

Figure 4: Comparison of the load-displacement curves for the Toronto beam

Comparison with other authors

The blind prediction for the Toronto beam included 66 entries. The resulting predictions are shown in Figure 5, with the inclusion of the three numerical results obtained in this study (carried out after the blind prediction). The aim of presenting this comparison is to show the variability of the results that were reported. This demonstrates that the shear problem is still an unsolved issue. As mentioned by Collins *et al.* [33], the prediction of shear strength of very thick slabs not containing shear reinforcement was a great challenge for the engineering profession. A 44% of the entries made dangerous overestimation of the capacity, while 20% of them were very accurate. The ACI 318-14 Code by the ACI Committee 318 [4], which does not account for size effect, estimated a capacity 3.7 times greater than the experimental value. This notes the limitations some codes have when shear critical elements are analysed.

Two of the entries used VecTor2: the prediction by Professor Vecchio and the prediction by the University of Brescia, led by Conforti and Facconi. The results of their numerical predictions are different and a better simulation was performed by Conforti and Facconi, proving that the engineering criteria and the assumptions made by a user greatly affect the results of the analysis.

The winning entry of the blind prediction was performed by Cervenka Consulting [39], the developers of the ATENA software. The reported results of their prediction are shown in Figure 6 and compared to the obtained results in this study using ATENA. Note that the winning model also displayed a greater initial elastic behaviour and a higher load at which the nonlinear behaviour start (440 kN) than the test (198 kN). After this point, the model shows a significant drop in capacity for a subsequent nonlinear behaviour that concurs with the test. The ATENA model from this study shows the same behaviour, that is greater post-cracking stiffness and capacity than the analysis performed by Cervenka Consulting. As it was stated, a probable cause for these differences is the lack of experience at the time of modelling and selecting the best parameters for the prediction.

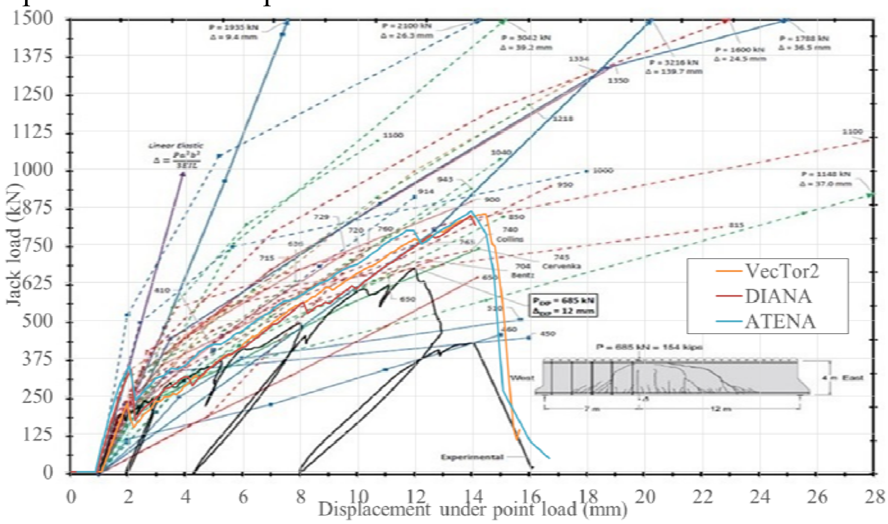

Figure 5: Comparison of the models from the three software with the blind predictions

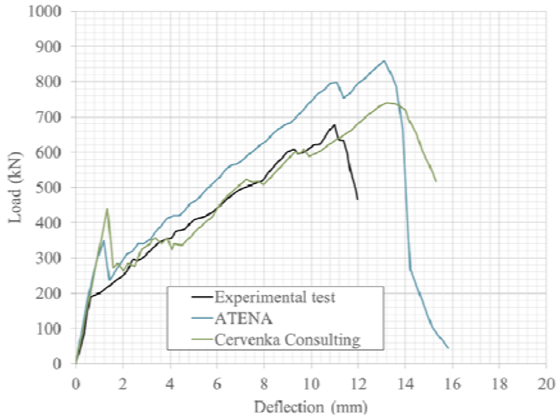

Figure 6: Comparison of the Toronto beam in ATENA to the Cervenka consulting prediction

4 CYCLIC ANALYSIS OF SHEAR-CRITICAL RC ELEMENTS

In order to model the response of shear-critical elements under cyclic and seismic loading, it is important to understand how the programs model the hysteretic behaviour of RC and to determine how they are able to predict the degradation due to the repetition and the reversal of the loading. The shear panels tested by Stevens *et al.* [40] were considered for evaluating the cyclic modelling capacities of the three programs. These cyclic tests were crucial to evaluate the hysteretic modelling of RC for shear-critical elements. All the details of the study can be found elsewhere [31].

4.1 SE shear panels

As described in [40], the University of Toronto tested three shear panels designated as SE8, SE9 and SE10. These specimens were tested in order to simulate the shear behaviour of a RC element under pure shear. The panels could represent a single element in a greater structure such as a silo or an offshore platform. The properties of the panels are summarised in Table 4. During the laboratory test, the panels were subjected to simultaneous compression and tension loading through hydraulic jacks. Due to the test setup, the applied loads are the principal stresses in the element. Given that the principal stresses are of the same magnitude and opposite directions, at a 45° angle the element is in a state of pure shear. For this reason, the FE model generated in each software consists of a 1×1 mesh (one single 4-node element of 1524×1524 mm). The loading condition is such that a uniformly distributed shear of 1MPa is applied at each side of the element.

Panel	f'_c (MPa)	ε_{co} (me)	Bar in x-direction				Bar y-direction				E_c (MPa)	f_t (MPa)	G_f (kN/m)
			Size	Area (mm^2)	ρ_{sx} %	f_{sx} (MPa)	Size	Area (mm^2)	ρ_{sx} %	f_{sx} (MPa)			
SE8	37.0	2.60	20M	300	2.93	492	10M	100	0.98	479	30753	2.01	0.140
SE9	44.2	2.65	20M	300	2.93	422	20M	300	2.93	422	33612	2.19	0.144
SE10	34.0	2.20	20M	300	2.93	422	10M	100	0.98	479	29480	1.92	0.138

Table 4: Properties of the SE panels, whose dimensions are 1524×1524×285 mm.

Model and results with VecTor2

A single 4-node element, simply supported in the bottom side, was used to model the SE panels, in which the steel bars were introduced as smeared, using the percentages given in Table 4 and assigning a crack spacing of 72 mm in x- and y-directions. The next models were chosen in order to model as best as possible the hysteric response that the RC panels would present during cyclic testing:

- Compression before peak: Hognestad (Parabola).
- Compression post-peak: Modified Park-Kent.
- Compression softening: Vecchio-Collins 1982.
- Tension stiffening: Modified Bentz 2003.
- Tension softening: Linear.
- Confined strength: Kupfer/Richart.
- Crack stress calculation: Basic (DSFM/MCFT).
- Hysteretic response of concrete: Palermo 2002 (with decay).
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: elasto-plastic behaviour with strain hardening in both tension and compression.
- Hysteretic response of reinforcement: Bauschinger effect (Seckin).

VecTor2 was able to perform the complete loading history quickly and without convergence issues. The models were capable of replicating the point at which cracking occurs in both panels. Similarly, the post-cracking stiffness in the first cycle was modelled in an accurate way. However, the cyclic behaviour of the tests differ from those obtained in VecTor2 in terms of stiffness degradation, energy dissipation and plastic strains.

Model and results with DIANA

The FE model of DIANA presents the same characteristics of the FE model of VecTor2. The model recommended in DIANA for the RC elements under cyclic loads is the Maekawa Cracked Concrete Model because it includes laws for the loading, unloading and reloading for both tension and compression. Since the Maekawa model was originally conceived as a fixed crack model, this approach has been followed to model the crack rotation. For the same reason, the shear retention model used is the Maekawa contact density. The selected models were as follows:

- Compression model: Maekawa compressive model.
- Tension model: Linear-ultimate crack strain.
- Model for lateral cracking: JSCE 2012.
- Concrete-reinforcement bond: Perfect bond
- Reinforcement stress-strain: Menegotto-Pinto plasticity, with the parameters suggested by Yu [41].

Due to convergence problems, the analysis was not able to complete the first cycle of loading. The maximum number of iterations and the step size were both varied in order to obtain convergence. Large values of iterations (~1000) and small step sizes (~0.01 multiplier) were used to achieve convergence but the results did not improve.

Model and results with ATENA

The specimens, the reinforcement and the boundary conditions were modelled as in VecTor2 and DIANA. The two constitutive models available in ATENA for RC elements are SBETA and Fracture-plastic. Furthermore, the next models were included in the analysis:

- Compression model for pre-peak branch: Model Code 2010 (for SBETA) and Hordijk model (for Fracture-Plastic).
- Compression model for descending branch: Linear descend.
- Tension model: Exponential.
- Model for reduction of compressive stress due to cracking: Vecchio and Collins [16] with $c = 0.80$.
- Concrete-reinforcement bond: Perfect bond.
- Reinforcement stress-strain: Multi-linear model with strain hardening, active in both tension and compression.

It should be noted when using the smeared reinforcement, the Menegotto-Pinto model is not available in ATENA, since it can be used only when the reinforcement steel is discretely modelled. According to the quality of the results obtained for the Bresler-Scordelis beams, the fixed crack approach was selected.

First the panels were tested using the SBETA constitutive models, which were able to compute all the loading history without convergence problems. The cracking point of the panels was adequately predicted. However, the estimated post-cracking stiffness is significantly larger than the experimental one. The unloading and reloading curves of the SBETA model are linear and no plastic strain is stored during the analysis. Because of this, the numerical results presented no stiffness degradation or damage representation after every cycle. Therefore, it was determined that the Fracture-plastic constitutive model was more appropriate since it allows to consider the plastic offset in compression and the tension stiffening (with c_t= 0.40).

The results when using the Fracture-plastic constitutive model showed promising results at the first cycle. Then convergence issues after the cracking point in the first reversal of the load were found. Several attempts were performed reducing significantly the step size and/or increasing the number of iterations. However, this problem persists and it was not possible to complete the test.

Comparison of the results among the programs

Due to convergence problems that were not overcome using DIANA and ATENA, the Authors were not capable of completing the whole loading history imposed during the experimental tests. VecTor2 did not present convergence problems. Therefore, to evaluate the obtained results, the comparison of the programs was performed for the available information on the first cycle only (Figure 7). The programs predicted properly cracking of concrete and the beginning o the nonlinear behaviour. ATENA presented a linear unloading behaviour to the origin, while VecTor2 showed a nonlinear unloading path but the residual plastic strains were almost insignificant. DIANA showed a plastic offset when the load reversed and decreased to zero, however the obtained strain overestimated the real response of the specimens.

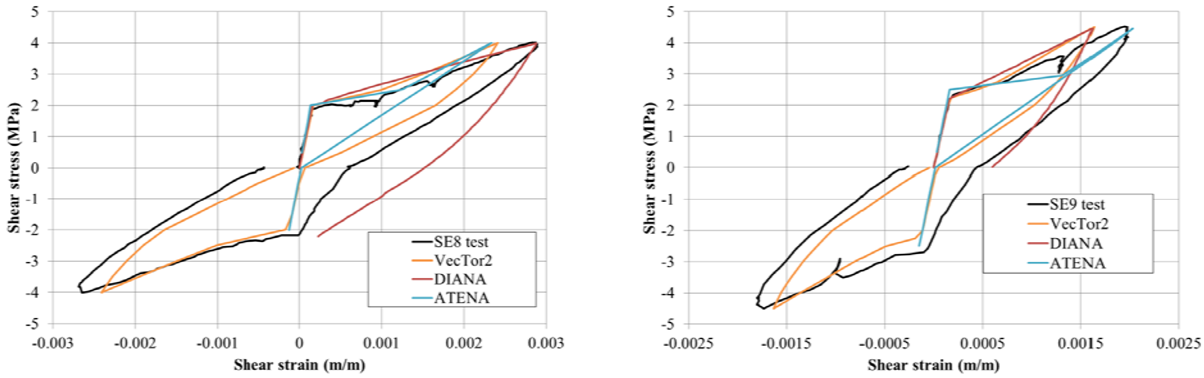

Figure 7: Comparison of the shear stress-strain curves for the SE8 and SE9 panels

Comparison with other authors

The results of the SE panels were numerically reproduced in literature using constitutive models based on the main concepts of the MCFT and its further extensions [40], [42], [13], [43]. The results obtained in this study with Vector2 are compared with the ones published by Stevens *et al*. [40] and Palermo and Vecchio [42]. Figure 8 shows the comparison for the SE8 panel. The study of Stevens *et al.* [1991] showed accurate results and, at the time being, these are the best published numerical predictions for the SE panels (Figure 8 right). The hysteretic model used in VecTor2 in this case is the Palermo and Vecchio constitutive model [42] (Figure 8 on the left) which includes a nonlinear unloading with plastic offsets only in compression, as well as a degradation of the reloading stiffness in order to model the damage in the concrete due to cyclic loading. However, it seems that this model does present a significant improvement from the previous analytical models which did not include plastic offsets in tension and for which the reloading was performed linearly returning to the previously experienced maximum (or minimum) strain. Comparing the results on the left of Figure 8, it can be stated that the numerical simulations carried out by the Authors using VecTor2 were performed with a proper selection of the available constitutive models and input parameters.

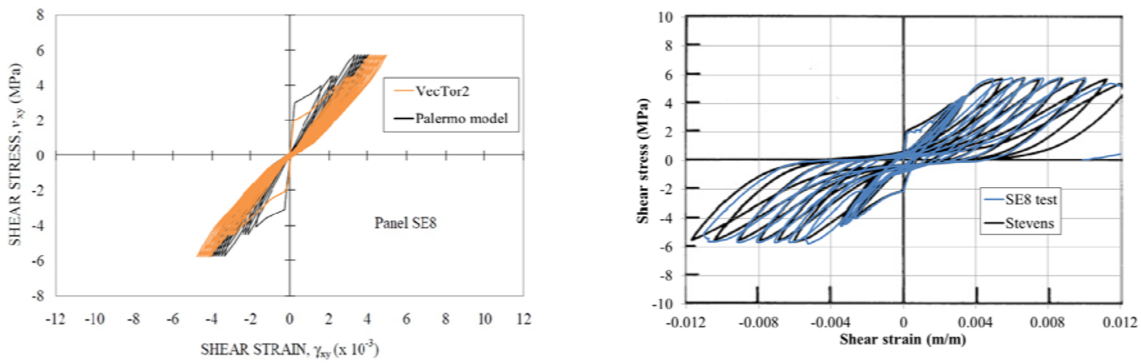

Figure 8: Comparison of the SE8 panel in VecTor2 to the results of [42] (left) and [40] (right)

5 CONCLUSIONS

The main limitation in the development of this investigation was the lack of experience and expertise in the use of the three finite element packages. The models presented in this work are the best simulations possible based on the knowledge gained during the duration of the investigation. It is believed that the numerical results could be improved given the resources and expertise required to appropriately simulate and reproduce shear-critical reinforced concrete elements in these FE software packages. Yet note that the validation with other authors (as Palermo and Vecchio [42] and the predictions reported by Collins *et al.* [33]) lead to the conclusion that the results of this research work are acceptable. Even further, it was demonstrated that not only the degree of expertise but the engineering criteria and selection of constitutive models and input data have a great influence in the results.

Other aspects should be mentioned. The phenomena being modelled, the shear behaviour of reinforced concrete, is complex and still an unresolved issue. This difficulty has been studied and is being studied for several years and it is still in development. Therefore, the constitutive models implemented in these software remain with limitations in their capabilities and applicability to the modelling of shear-critical RC elements.

Furthermore, the accessibility to educational licensing for the use of some of these programs restricted the available time to work on each program. Given the cost of the full license, some of the software were used thanks to a time-limited license granted by the developers of the software.

Future research in this topic should focus in including other different tests to further evaluate the capacities of the software packages. Other test could include beams under different loading conditions and different characteristics and reinforcement, as well as further shear panels and shear walls subjected to cyclic loading.

Moreover, other finite element programs that include the state-of-the-art in concrete modelling should be included in the evaluation. These programs could include Abaqus, Ansys and OpenSees. Also, commercial available software mainly used in the private industry (e.g SAP 2000) should also be included to evaluate their capacities, since, to the Authors' knowledge, these are the programs mainly used in the structural design companies.

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