

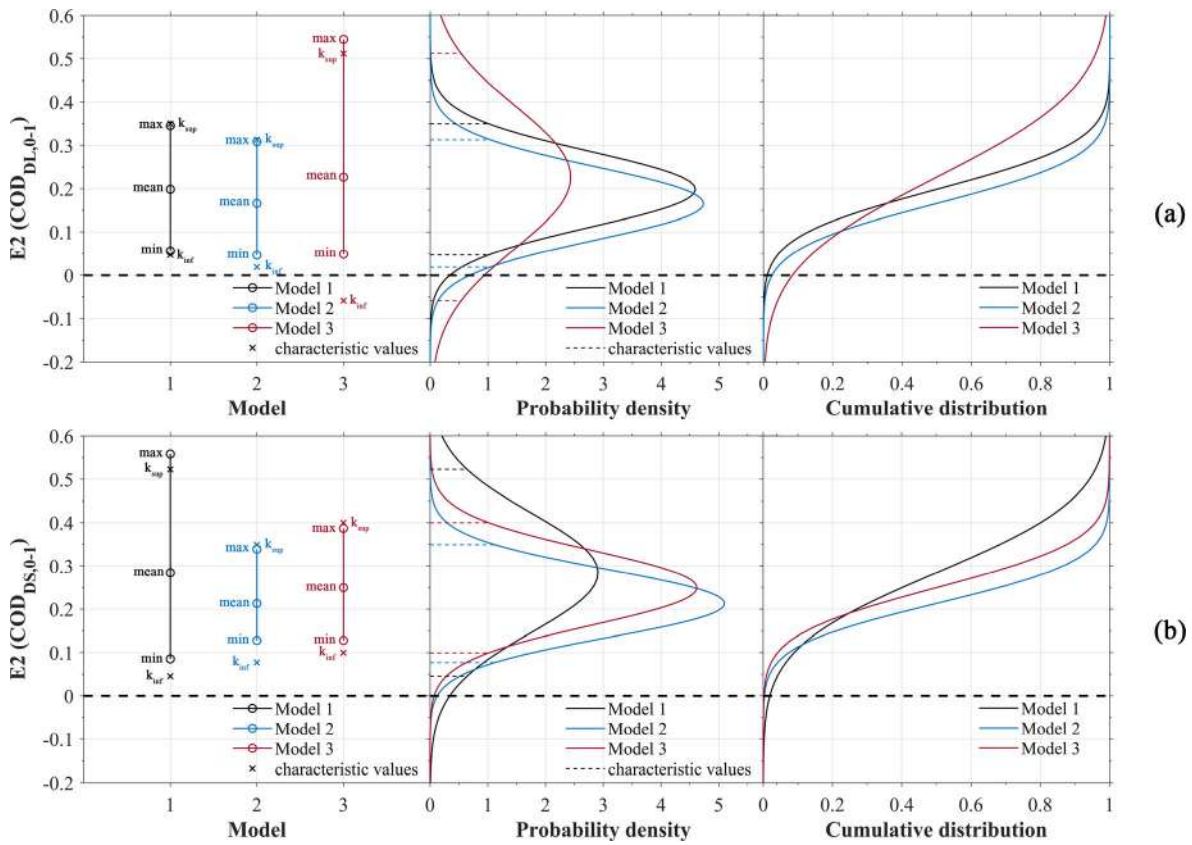


FIGURE 10 Computed values of the average errors E2, probability density and cumulative distribution: $E2(COD_{DL,0-1})$ (a), and $E2(COD_{DS,0-1})$ (b).

of the relative vertical displacement ($\alpha_2 = 0.25$), while lower weights are given to errors computed considering the entire curves ($\alpha_3 = \alpha_4 = \alpha_5 = 0.1$).

$$WI1 = \alpha_1 \cdot |E1(P_{max})_{avg}| + \alpha_2 \cdot |E1(\delta_1)_{avg}| + \alpha_3 \cdot E2(\delta_1)_{avg} + \alpha_4 \cdot E2(COD_{DS})_{avg} + \alpha_5 \cdot E2(COD_{DL})_{avg} \quad (5)$$

Meanwhile, the second index, WI2, is chosen to illustrate the accuracy of the simulation in the initial phase of the response, up to the reaching of 0.1 and 0.3 mm of crack openings, which typically represent serviceability conditions. The weighting parameters $\beta_1, \beta_2, \beta_3, \beta_4$ reported in Equation (6) are, respectively, set equal to 0.2, 0.4, 0.2, and 0.2. Similar to the weighting parameters of Equation (5), the load value at a crack opening of 0.3 mm (limit at SLS) is more significant ($\beta_1 = 0.4$) compared to the other parameters ($\beta_2 = \beta_3 = \beta_4 = 0.2$).

$$WI2 = \beta_1 \cdot |E1(P_{0.1})_{avg}| + \beta_2 \cdot |E1(P_{0.3})_{avg}| + \beta_3 \cdot E2(COD_{DL,0-1})_{avg} + \beta_4 \cdot E2(COD_{DS,0-1})_{avg} \quad (6)$$

Please note that all errors E1, previously indicated with a sign to describe overestimation or underestimation of the specific phenomenon (see Equation 1), are taken in absolute value in Equations 5 and 6, due to the need to combine them with errors E2, which are always positive.

The weighted indices are distinctly computed in the three phases of the simulation process for each RU, and their evolution is reported in Figure 11. Observing the average values between the RU results (red points), it is possible to appreciate the increasing of accuracy within the numerical phases, especially regarding the WI1 parameter. The slight increase of WI2 in the third phase of the benchmark can be attributed to the fact that the various research units, once given the opportunity to choose their modeling approach (see phases description in Section 4.1), deemed it of greater importance to capture the ultimate capacity of the element rather than focusing on the response at the initial stage.

The WI2 index, representing the behavior of the half-joint girder beam in service conditions, consistently reaches higher maximum values compared to the WI1 index across all three benchmark phases. This indicates a greater challenge for research units in accurately replicating experimental results at the initial stages, compared to

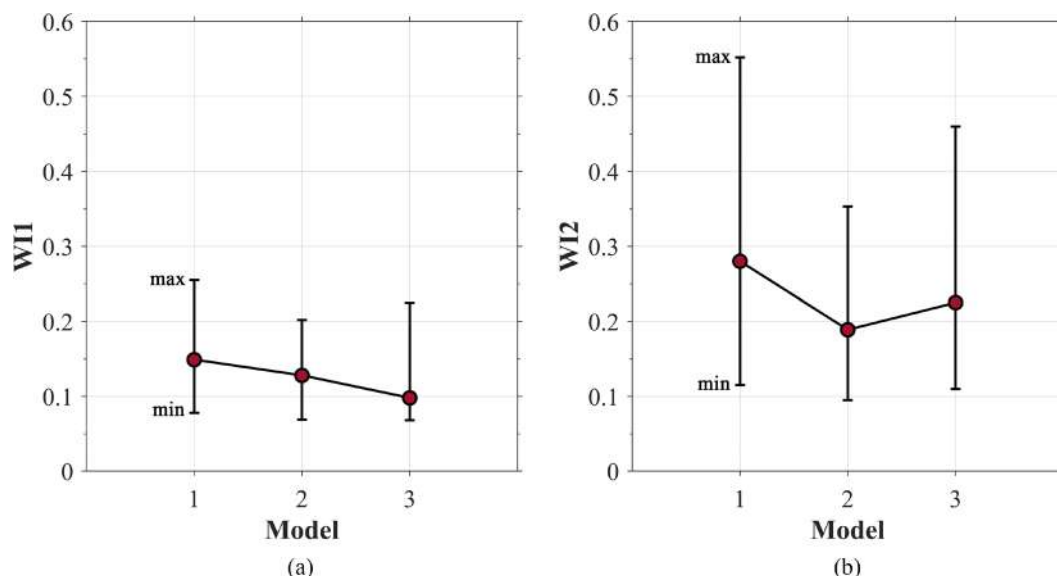


FIGURE 11 Evolution of the weighted indices in the three phases of the simulation process: WI1 (a) and WI2 (b). Average values are plotted in red.

estimating parameters generally associated with an ultimate condition (i.e., WI1).

5 | LESSONS LEARNED

The proposed procedure has proven effective in reducing the average error, especially concerning failure conditions. The curves from individual units and crack patterns in the three analysis phases are provided in Appendix A, accompanied by comprehensive comments on both the modeling procedure and strategies. This serves as a valuable resource for further scrutiny and a reference for future studies, while also providing support for designers engaged in the numerical modeling of RC dapped ends.

From the analysis of the evidence emerged in Phases 1, 2, and 3 developed in this study by the individual RUs, it is possible to draw some common observations. Firstly, there is a shared emphasis on the crucial role of accurate geometrical replication for a better reproduction of crack patterns. While RU8 emphasizes that 3D models offer greater precision compared to 2D models, it must be noted that this comes with an increase in analysis time, potentially mitigated by the use of explicit dynamic solvers. This trend is corroborated by RU7, indicating that despite 2D models provide a satisfactory approximation of the structural response, the choice between a 2D and 3D approach involves a trade-off between precision and computational burdens. Secondly, the importance of care in the modeling of

boundary conditions is consistently stressed, as evidenced by the analyses in Phase 3. RU4, RU6, RU7, and RU8 emphasize the significance of faithfully reproducing the geometries, kinematics, and stiffness of the supports and the load introduction system to capture experimental outcomes. Thirdly, the influence of material parameters emerges as a common issue, as indicated by RU1, RU3, RU4, and RU6, emphasizing the substantial impact of factors such as concrete fracture energy on numerical results, as demonstrated by the comparison between Phase 1 and Phase 2 responses. Additionally, the role of mesh discretization is acknowledged, with RU5 noting the potential for improved results through refined modeling of specific localization regions. RU8 suggests that geometric nonlinear analysis provides a better description of the load versus deformation curve. Finally, the importance of accurately determining the mechanical properties of concrete and steel is universally recognized: the effects of post-peak behaviors, both in tension and compression, are indeed apparent, while the role of the assumption of perfect bond between rebar and concrete seems less significant, probably due to the concrete strength class not particularly low.

6 | CONCLUSIONS

The paper presents the results of a partially blind benchmark organized within the framework of the WP4-T4.4 ReLUIIS project, addressing the challenges associated

with Gerber saddles in existing RC bridges. Its aim was to assess the reliability of available FEM-based models for the prediction of the structural behavior of RC dapped-end beams, and to identify their advantages with respect to widely used limit analyses, such as STMs.

The results obtained from one experimental test carried out by Politecnico di Milano were considered as benchmark solution and three different modeling phases were adopted. Based on the obtained results, the following observations can be drawn:

1. FE models proved to be valuable tools for examining the structural behavior of elements under serviceability conditions, such as the development of crack patterns and the identification of mechanical phenomena. In contrast, STMs are more suitable for estimating design loads at the ULS.
2. The transition from the semi-blind phase (Phase 1) to the phase with homogenized parameters (Phase 2) and finally to the post-blind phase (Phase 3) demonstrated a general decrease in both average error and dispersion of results, proving the ability of the numerical models to predict the experimental response. Moreover, a common trend was observed in capturing the sequence of different phenomena characterizing the behavior of a RC element, including cracking, yielding of reinforcing bars, and failure, confirming the adequacy of the simulations.
3. The semi-blind procedure facilitated modeling by substantially decreasing potential deviations/errors between the numerical curves and the experimental reference curve, compared to fully blind benchmarks. Furthermore, the discussion of the results obtained in Phase 1 through the semi-blind procedure allowed for a more informed definition of boundary conditions and geometric discretization methods in Phase 2. This resulted in a decrease in errors and final indices when transitioning from Phase 1 to Phase 2.
4. The utilization of homogenized parameters in Phase 2, collectively determined by research units specialized in nonlinear modeling of RC structures, entails a reduction of errors in predicting the structural response of half-joint beams. The selection of these homogenized parameters offers practical guidance for establishing initial material parameters or determining the modeling approach, mesh, and related aspects. However, it is important to note that, even with homogenized parameters, non-negligible errors were obtained.
5. The post-blind phase (Phase 3), wherein all experimental results were accessible and detailed comparisons among various research units occurred, enabled the incorporation of local effects—such as boundary

conditions—into the FE models, further refining their accuracy.

6. Research units specialized in the field, equipped with two experimental curves, initially achieved errors ranging from -14% to $+4\%$ concerning the maximum load (in the third phase this error range was reduced from -2.4% to 4%). Notably, researchers had the option to verify the numerically obtained maximum load using simplified tools like STMs. However, no significant improvement in displacement estimation throughout the three benchmark phases was observed. In fact, error $E1(\delta_1)$ deviations of approximately $\pm 50\%$ were registered, in spite an average value close to zero.
7. An important reduction from Phase 1 to Phases 2 and 3 in the error and dispersion of the loads at which the cracks on the lower nib of the half-joint widen ($E1(P_{0.1})$ and $E1(P_{0.3})$), was noted. Accurately capturing the cracking in this specific area holds significant importance in characterizing the response of half-joint beams during in-service condition, with considerable implications for the structural element's durability.
8. The synthetic indices WI1 and WI2 proposed in this paper can be considered representative for the ability to capture the behavior of RC half-joint girders in service and ultimate conditions.

Finally, it is important to note that while the procedure aimed to simulate the experimental response of a half-joint beam, designers need to estimate the design capacity instead. In this scenario, the average material properties used in the benchmark should be substituted with the design properties of the materials. Additionally, the numerically obtained response should be adjusted using specific coefficients, such as those found in methods like the partial factor method or the global resistant factor method.

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CONFLICT OF INTEREST STATEMENT

No potential conflict of interest was reported by the authors.

DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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APPENDIX A: MODEL DESCRIPTION AND RESULTS FROM THE SINGLE RESEARCH UNITS

This section presents the numerical results of all the research units (RUs) and a detailed description of the finite element (FE) model carried out in each of the three phases. Information concerning the software and the main numerical parameters adopted for the definition of the materials are provided, with particular attention given to the modifications implemented throughout the modeling phases.

The numerical results of the single RUs are reported in Figures A1–A8 in terms of load- δ_1 , load-COD_{DL}, and load-COD_{DS} curves. In addition, the crack patterns at loads equal to 270 and 370 kN are shown for Phases 1, 2, and 3. Finally, a list of relevant conclusions drawn from the numerical modeling is provided by each RU.

A.1. | Research unit: 1

Two FE models were created with DIANA FEA²⁴ using eight-node quadrilateral isoparametric plane stress elements; the mesh size was 25 mm. Smeared material properties acquired via a rotating crack approach are used to simulate concrete cracking; the reinforcement is modeled as embedded truss elements with a bilinear hardening stress-strain curve. Nonlinear analyses are performed considering both mechanical and geometrical nonlinearity.

A.1.1. | Description of Model 1

In Model 1, the following mechanical parameters are adopted for the concrete: $f_{cm} = 48.6$ MPa, $f_{ctm} = 3.73$ MPa, $G_f = 0.146$ N/mm, and $E_c = 35.35$ GPa. The Eurocode 2²⁵ constitutive law is used for concrete in

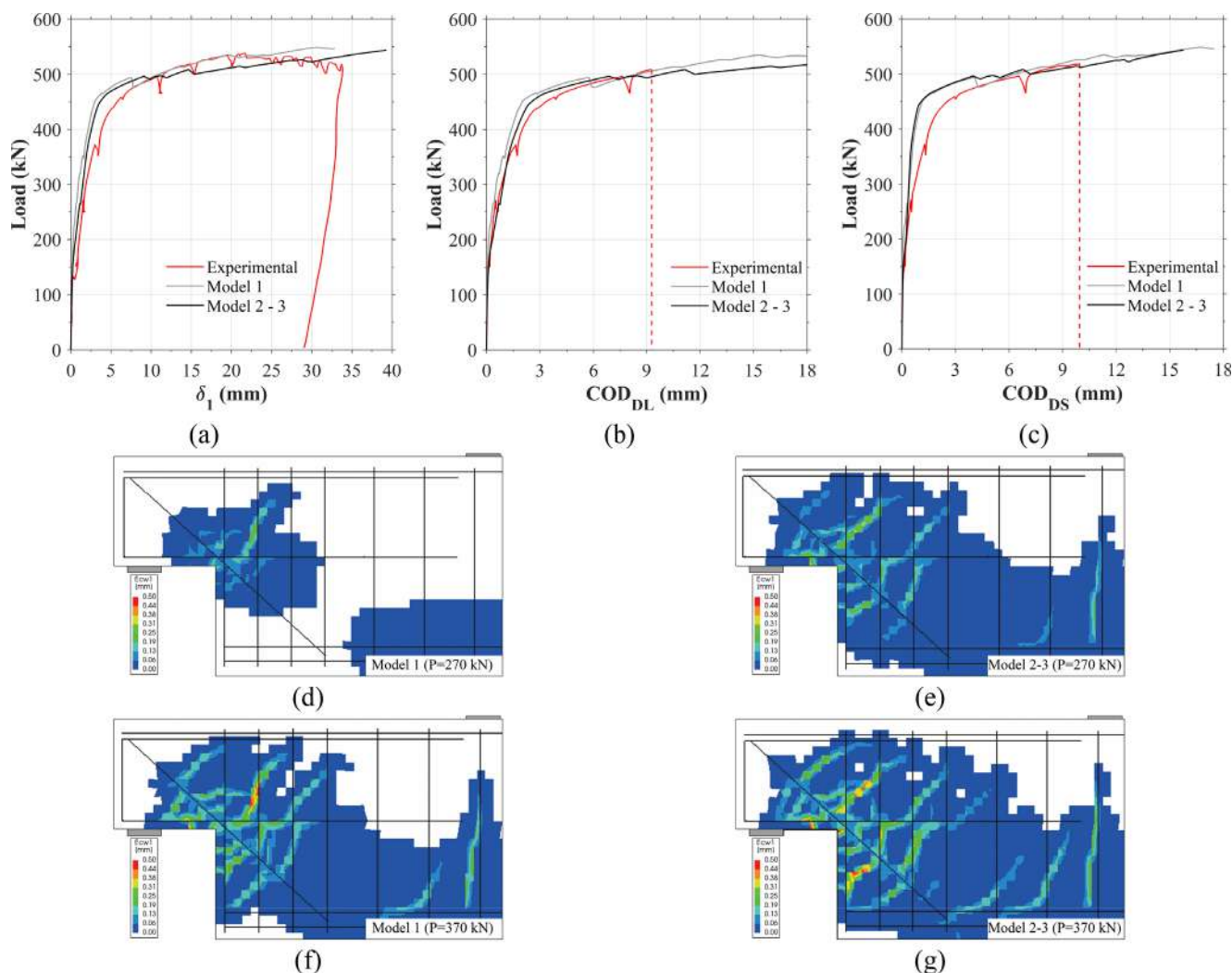


FIGURE A1 Results from RU1: Load- δ_1 (a), load-COD_{DL} (b), and load-COD_{DS} (c) curves of Phases 1–3, crack pattern at 270 kN for Phase 1 (d), and Phases 2, 3 (e), and crack pattern at 370 kN for Phase 1 (f) and Phases 2, 3 (g).

compression, and the *fib* Model Code 2010²⁰ constitutive law is adopted for concrete in tension. Restraints are given by a roller support at point A and a pinned support at point B, while the point load is free to move horizontally. Nearly the same numerical results are achieved if restraint conditions at points A and B are interchanged or if a horizontal restraint is applied at the load point, while points A and B are equipped with roller supports.

A.1.2. | Description of changes implemented for Model 2

In Model 2, the mechanical parameters and constitutive laws proposed by PoliMI RU are adopted, while the restraint conditions are given by a pinned support at point A and a roller support at point B. The constitutive law of *fib* Model Code 2010²⁰ is adopted for concrete in compression, while for concrete in tension, the fracture energy is evaluated considering only the area subtended by the first bilinear branch of the constitutive tensile law of *fib* Model Code 2010.²⁰ Model 2 provides a good approximation of the experimental results; therefore, it was deemed unnecessary to introduce additional modifications to the numerical model. The software DIANA FEA adopts a constitutive model based on total strain according to the modified compression field theory rather than the concrete damaged plasticity (CDP) model; therefore, in Model 2, it was not possible to use the standardized parameters given in Table 3 for the CDP model.

A.1.3. | Lessons learned

- The main parameter affecting the numerical results seems to be the fracture energy of the concrete.
- The numerical ultimate displacement is highly affected by the value assumed for the ultimate concrete strain, as the numerical collapse is governed by the crushing of the concrete cover at the top.
- The modeling of reinforcement as embedded trusses is found to be adequate for simulating experimental results; nevertheless, this approach is not reliable for low-strength concrete, where the slip between bars and concrete cannot be negligible.
- The cracking pattern of Model 1, both in terms of the number of cracks and opening of the main cracks, is less pronounced than that of Model 2 due to the lower values of the tensile strength and fracture energy adopted in Model 2. Moreover, the slope of the initial branch in the load–displacement curve is slightly less

pronounced for Model 2, while no significant variation in the ultimate capacity is observed between the two models.

- The second part of the experimental load–displacement curve is better described by a geometrical nonlinear analysis than by a geometrical linear analysis.

A.2. | Research unit: 2

A.2.1. | Description of Model 1

The numerical FE model was built with VecTor2 software based on the modified compression field theory “MCFT”²⁶ mainly developed by Prof. Vecchio of the University of Toronto. The mesh consists of rectangular and triangular elements (approximately 3% by number). Reinforcements are modeled with truss elements. The beam, modeled entirely, is supported at the ends by a roller and a hinge.

A.2.2. | Description of changes implemented for Model 2

Only the laws that define the properties of materials have been modified. Therefore, some material models already implemented in the software were chosen to get closer to what was agreed upon by the task group. It was not possible to adopt all the requested parameters (listed in Table 3) because the damage plasticity model was not implemented in the adopted software.

A.2.3. | Description of changes implemented for Model 3

To simulate the initial experimental stiffness more accurately, a uniformly distributed shrinkage was introduced, with a concrete prestrain of -0.2 millistrain.

A.2.4. | Lessons learned

- The numerical analyses were able to model the elastic stage and the post-cracking stiffness in a rather accurate but simple way.
- The formation and progression of the crack pattern (up to very significant transverse crack widths of approximately 10 mm), the bearing capacity, and the collapse mechanism were modeled with high accuracy.

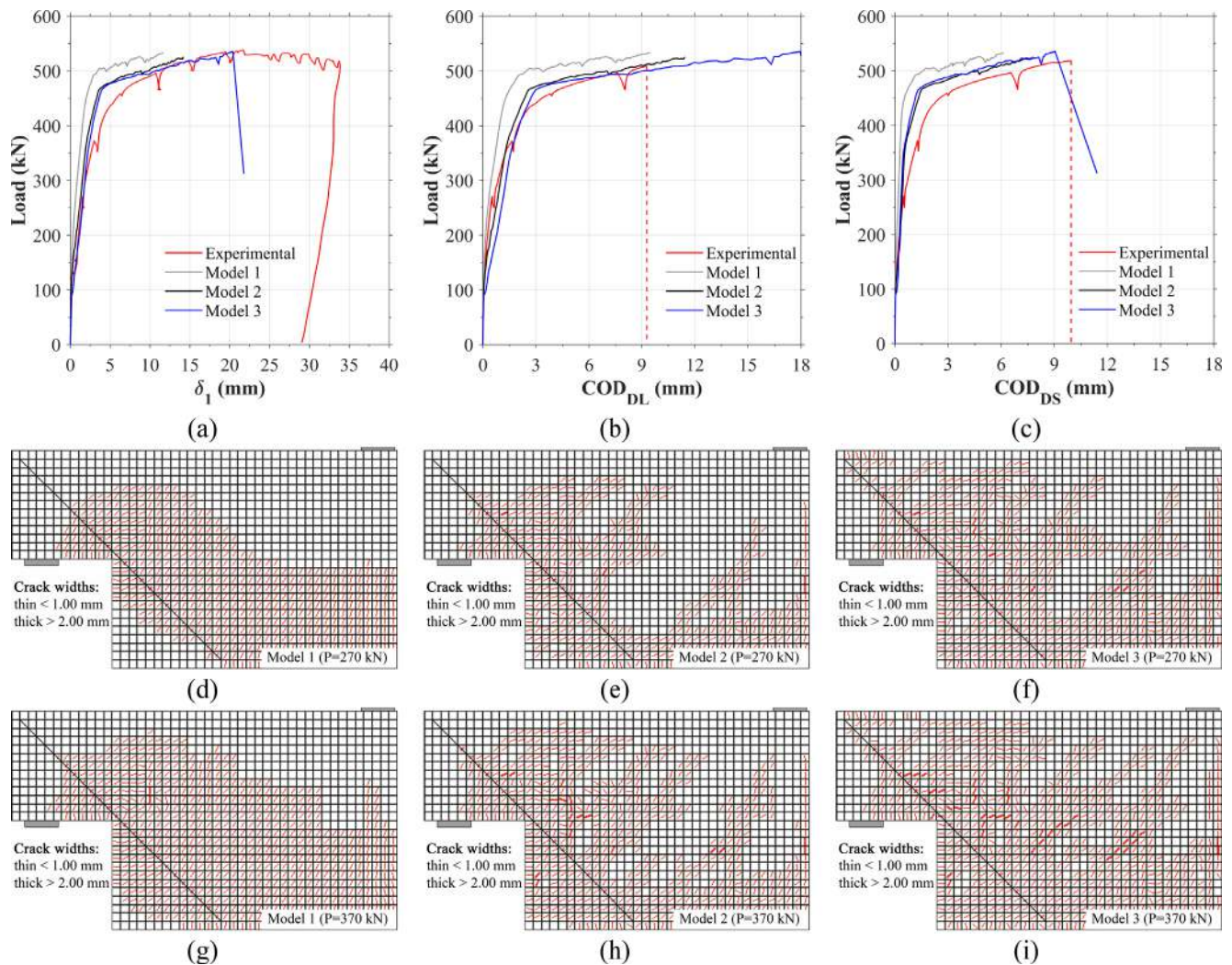


FIGURE A2 Results from RU2: Load- δ_1 (a), load-COD_{DL} (b), and load-COD_{DS} curves of Phases 1–3 (c); crack pattern at 270 kN for Phase 1 (d), Phase 2 (e), and Phase 3 (f); crack pattern at 370 kN for Phase 1 (g), Phase 2 (h), and Phase 3 (i).

- Model 3 best approximates the experimental crack pattern at 370 kN. This indicates that the assumption of the initial uniformly distributed shrinkage is not only able to better capture the stiffness but also determines a more consistent cracking pattern.

A.3. | Research unit: 3

A.3.1. | Description of Model 1

A 3D FE model was initially implemented (Model 1) in MIDAS-FEA software with a concrete mesh made of 8-node brick elements and embedded reinforcements made of 2D truss elements, for a total of 120,039 DOF. The average dimension of the mesh was 25 mm. A smeared cracking model was adopted with fixed cracks;

the tensile constitutive law for concrete was exponential, while the Thorenfeldt constitutive law¹⁹ was used in compression. No confinement effect was taken into account. A perfect bond between the concrete and steel reinforcement was considered.

A.3.2. | Description of changes implemented for Model 2

Models 2 and 3 are instead 2D FE models with plane stress elements and embedded truss reinforcements. The smeared cracking model with a rotating crack is applied in this case through MIDAS FEA software, with a bilinear constitutive law of concrete in tension and a parabolic curve in compression, maintaining the hypothesis of a perfect bond and no confinement. In Model 2, the standardized parameters of Table 3 are used apart the type of

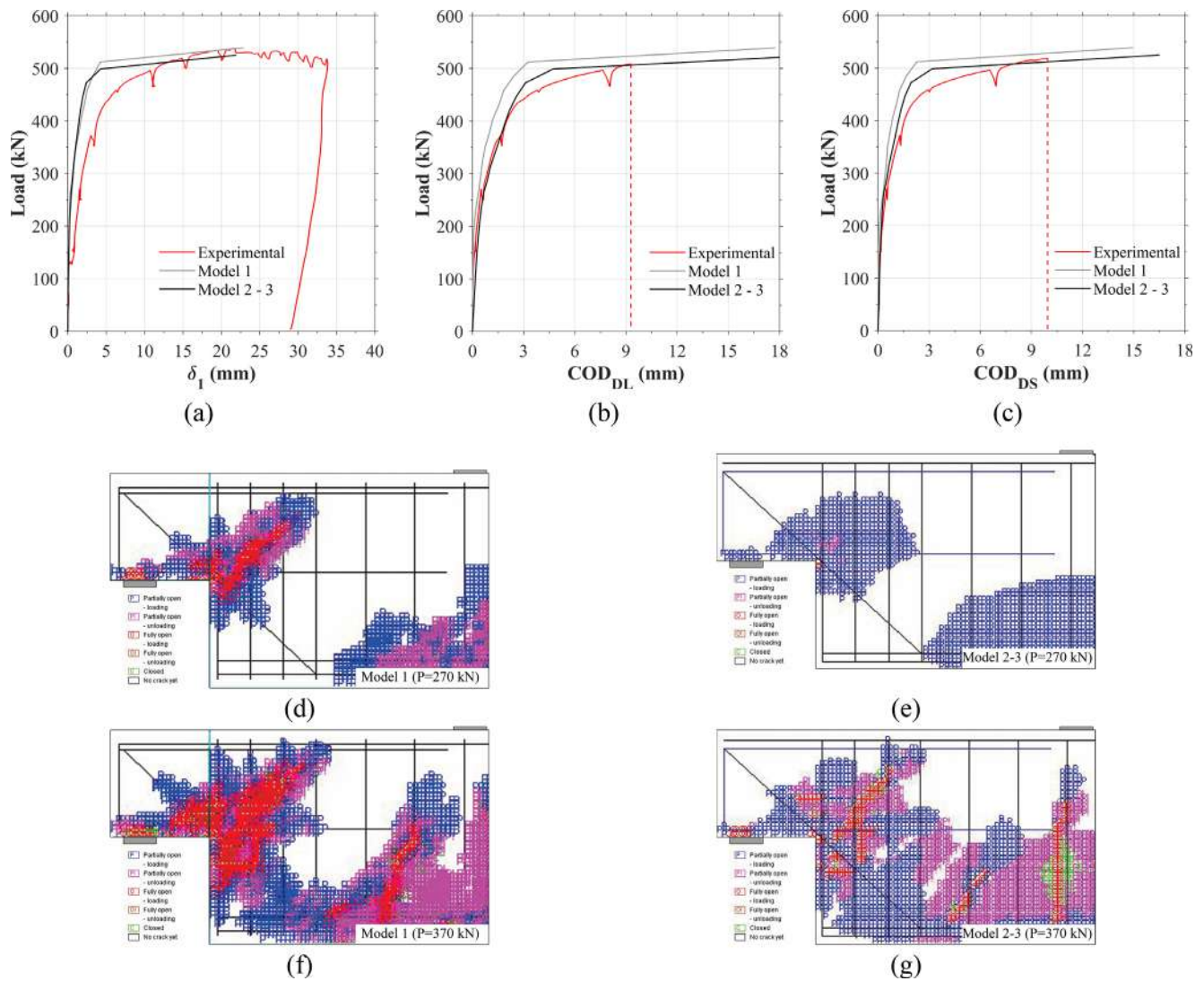


FIGURE A3 Results from RU3: Load- δ_I (a), Load-COD_{DL} (b), and Load-COD_{DS} (c) curves of Phases 1–3, crack pattern at 270 kN for Phase 1 (d), and Phases 2, 3 (e), and crack pattern at 370 kN for Phase 1 (f) and Phases 2, 3 (g).

analysis with the force control and smeared cracking model instead of CDP model. This kind of analysis provided better stability than Model 1 with rapid convergence of iterations.

A.3.3. | Description of changes implemented for Model 3

Model 2 reproduces the experimental results with good accuracy; thus, no changes were made to the numerical model.

A.3.4. | Lessons learned

- The 3D model with brick elements provides more reliable results from the point of view of the cracking pattern even if the model parameters (constitutive laws) were not standardized.
- The 2D model with standardized parameters provides less compliance with the cracking framework but improves the prediction of displacement curves and thus the overall deformability of the system. The cracking pattern well shows the strain concentration at the corner, but the perfect bond without slip does not allow for a change in the direction of the crack near the top bar.
- As the law of concrete compression varies, the cracking pattern can vary greatly, providing significantly different results. The softening branch provided by the Thorenfeldt model favors the simulation of the crack opening phase and the failure of the compressed struts.
- The choice of the parameters “fracture energy” in tension and “crack band width” significantly affects the

result, implying the necessity of a careful calibration phase of the model.

A.4. | Research unit: 4

A.4.1. | Description of Model 1

Nonlinear finite element analyses (NLFEAs) were performed by using a multi-layered shell approach with the PARC_CL 2.1 crack model,^{27,28} implemented as a user subroutine in Abaqus software. Eight-node shell elements with a reduced integration scheme (S8R) were used. The thickness of the member was subdivided into 11 layers to correctly place the reinforcement. Steel

rebars were modeled by using both smeared and discrete approaches. For the concrete, the following mechanical properties were adopted: $E_c = 33$ GPa, $f_{cm} = 48.6$ MPa, $f_{ctm} = 3.73$ MPa, $G_F = 0.103$ N/mm, and $\varepsilon_r = 2.5e-4$ (estimated shrinkage strain).

A.4.2. | Description of changes implemented for Model 2

The following parameters were adopted for Model 2 according to the common model input data: $f_{ctm} = 2.48$ MPa, $G_F = 0.0918$ N/mm, and $\varepsilon_r = 0$. The steel properties are set according to common data. The concrete parameters related to the CDP model cannot be adopted since different constitutive laws are

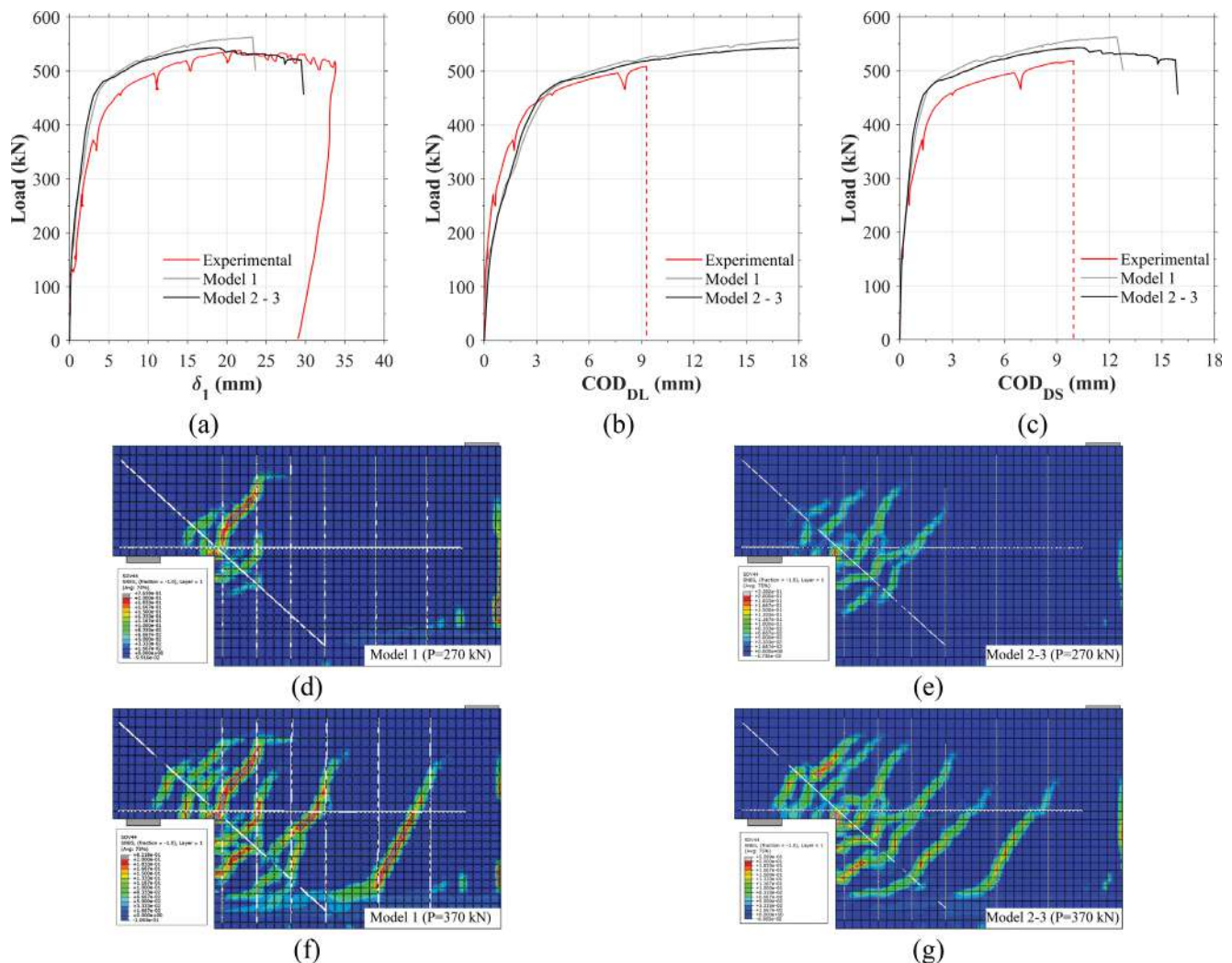


FIGURE A4 Results from RU4: Load- δ_1 (a), load-COD_{DL} (b), and load-COD_{DS} (c) curves of Phases 1–3, crack pattern at 270 kN for Phase 1 (d), and Phases 2, 3 (e), and crack pattern at 370 kN for Phase 1 (f) and Phases 2, 3 (g).

implemented in the adopted user subroutine.²⁷ The following boundary conditions are adopted: roller restraint at the supports and pin restraint at the point load position. This configuration does not affect the results compared to the boundary conditions of the common data.

A.4.3. | Description of changes implemented for Model 3

Model 2 reproduces the experimental results with good accuracy; thus, no changes were made to the numerical model.

A.4.4. | Lessons learned

- The correct calculation of the mechanical properties of concrete and steel input data, as well as the corresponding boundary conditions, are fundamental for obtaining reliable numerical results for the investigated RC member. Indeed, modest changes in such parameters may considerably affect the results of FE simulations.
- From the point of view of mechanical nonlinearity, the compressive and tensile strengths and the fracture energy of concrete as well as the yielding and ultimate stress–strain properties of steel are crucial parameters for obtaining realistic crack patterns and detecting the correct failure modes.
- The long-term effects related to shrinkage and creep of concrete trigger greater crack openings (the crack openings of Model 1 are greater than those of Models 2 and 3). Such aspects should be carefully evaluated during the construction of the FE model.

A.5. | Research unit: 5

A.5.1. | Description of Model 1

The numerical analyses were performed by 2D-FE models built by means of VecTor2 software. The latter is a program based on the Modified Compression Field Theory²⁶ and the Disturbed Stress Field Model,²⁹ both of which were developed during recent decades at the University of Toronto. The mesh of Model 1 was made of four-node elements 50×50 mm in size for the concrete

regions and two-node truss elements for the steel bars. The f_{ctm} of concrete was assumed to be equal to the default value suggested by VecTor2 and equal to 2.30 MPa.

A.5.2. | Description of changes implemented for Model 2

Models 2 and 3 were refined using a tighter mesh with square elements of 25×25 mm in size. The mechanical characteristics of the materials were calibrated based on the results of compressive tests carried out on cylindrical concrete specimens and tensile tests performed on steel rebar. The uniaxial constitutive behavior of steel was modeled by adopting a trilinear law for the embedded rebars. The constitutive behavior of the concrete was modeled by using the basic curves suggested by VecTor2: the Hognestad parabola and the Park-Kent models for the pre- and post-peak branches of the compression stress–strain curve, respectively. These constitutive models available in the version of VecTor2 used do not require the fracture energy as an input parameter; thus, the value of G_f reported in Table 3 was not considered.

A.5.3. | Lessons learned

- The analyzed specimen showed a main crack at the re-entrant corner without significant damage in the rest of the beam.¹⁴ Models 1–3 accurately reproduce the experimentally observed failure mode and crack pattern.
- The use of a large mesh (Model 1) allowed us to limit the analysis cost and to obtain a rapid estimate of the load at collapse. By introducing the mesh refinement and the standardized values of the parameters (Table 3) in Models 2 and 3, the results of the analyses improved, especially in terms of the crack pattern at the different load stages. The crack patterns in Figure A5 e,g show widespread cracking around the corner of the saddle, as well as the onset of several more pronounced cracks, as observed experimentally.
- The bearing load capacity of the specimen was predicted by numerical models with sufficient precision.
- The ascending branch of the numerical load–deflection curve is characterized by a slope greater than that of the experimental curve. In addition, the deflection at failure is slightly overestimated by the numerical

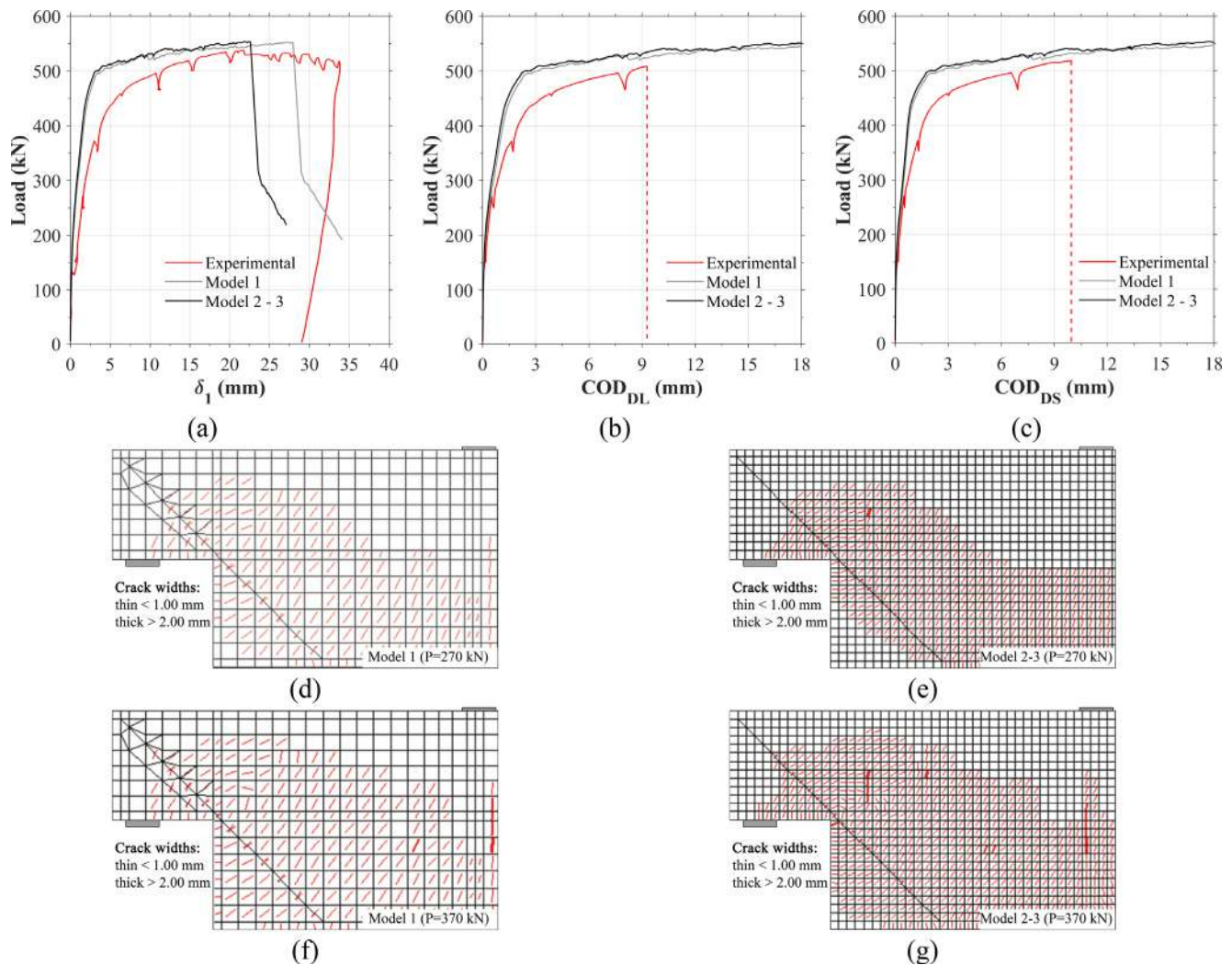


FIGURE A5 Results from RU5: Load- δ_1 (a), load- COD_{DL} (b), and load- COD_{DS} (c) curves of Phases 1–3, crack pattern at 270 kN for Phase 1 (d), and Phases 2, 3 (e), and crack pattern at 370 kN for Phase 1 (f) and Phases 2, 3 (g).

models. This can be attributed to the assumption of a perfect bond between the steel bar and the concrete, especially for the reinforcement across the critical crack. A refined model of the region at the re-entrant corner, with a more accurate description of the interaction between the concrete and steel, could lead to a better reproduction of the experimental results.

A.6. | Research unit: 6

A.6.1. | Description of Model 1

The 3D FE model was developed with STKO³⁰ and OpenSees.³¹ The concrete is discretized with 3D linear brick elements with a mesh size equal to 25 mm. The rebars

are discretized with 1D beam-column displacement-based elements with fiber sections. The model has 57,590 elements. The rebars are embedded in the concrete using embedded constraints. The constitutive model used for the concrete is a tension-compression plastic-damage IM-PLEX model^{32,33} and steel is a hysteretic material.³⁴ The boundary conditions reflect the experimental setup (see picture of the model). The analysis is performed with OpenSees MP using 10 processors.

A.6.2. | Description of changes implemented for Model 2

The standardized parameters were adopted for the Phase 2 model. The fracture energy in compression is equal to 36 N/mm.

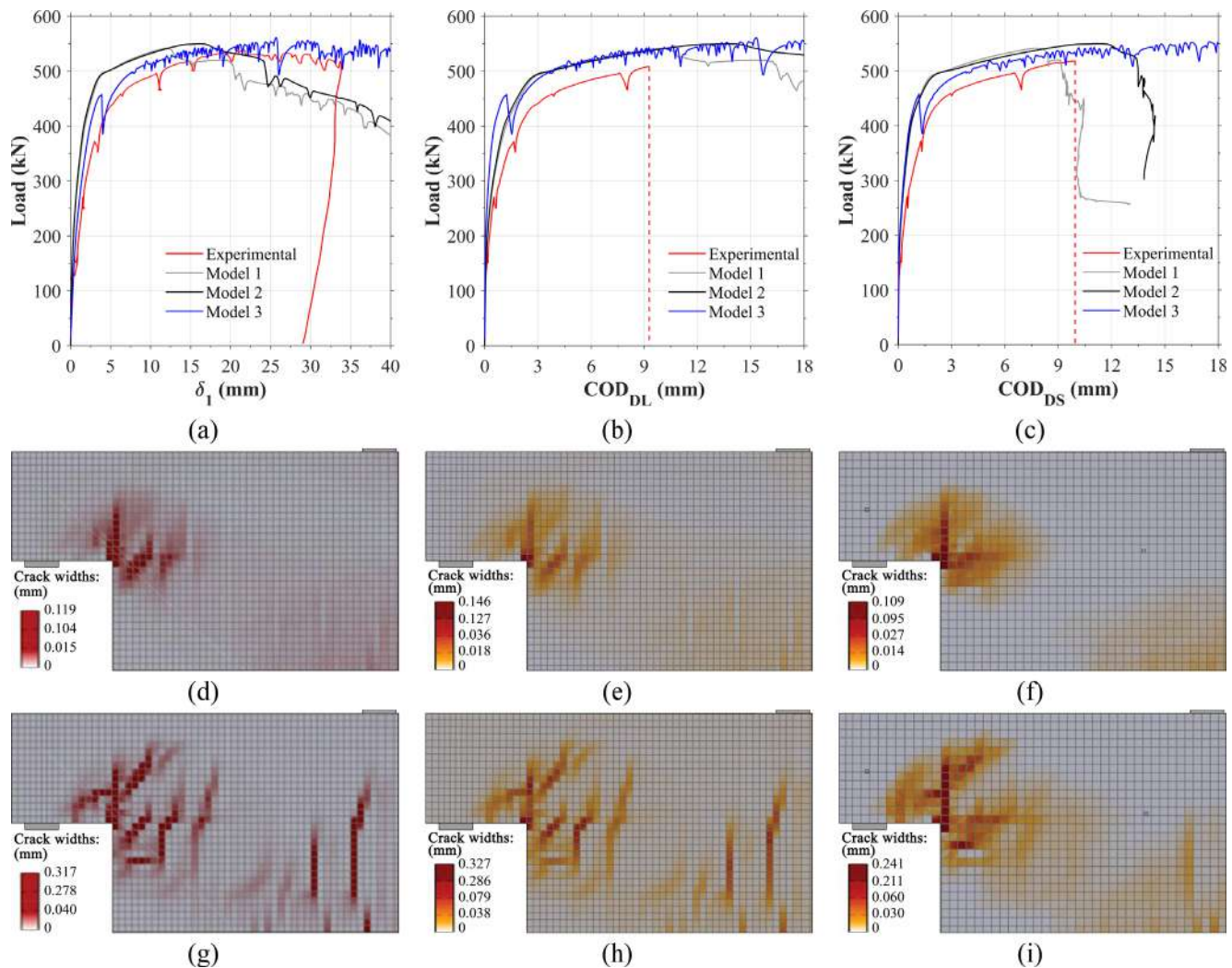


FIGURE A6 Results from RU6: Load- δ_1 (a), load- COD_{DL} (b), and load- COD_{DS} (c) curves of Phases 1–3; crack pattern at 270 kN for Phase 1 (d), Phase 2 (e), and Phase 3 (f); crack pattern at 370 kN for Phase 1 (g), Phase 2 (h), and Phase 3 (i).

A.6.3. | Description of changes implemented for Model 3

The load application and boundary conditions were modified to better represent the test. A zero-length IM-PLEX contact element³⁴ was placed between the loading plate, the left support, and the concrete with a friction coefficient $\mu = 0.07$. The concrete constitutive model was improved as described in Ref. 35. Bond slip between the rebars and the concrete is introduced using *fib* Model Code 2010²⁰ with the assumption of a good bond.

A.6.4. | Lessons learned

- The crack pattern and failure mode are accurately described by nonlinear analysis.

- The correct assumption of the concrete fracture energies, bond-slip behavior, and boundary conditions is central to obtaining accurate results.

A.7. | Research unit: 7

A.7.1. | Description of Model 1

For Phase 1 prediction (Model 1), a 3D FE model was developed in Abaqus. The nonlinear behavior of concrete was defined through a concrete damage plasticity model, with constitutive laws defined according to *fib* Model Code 2010.²⁰ A non-structured mesh was adopted in the model. The concrete was discretized as four-node linear tetrahedron elements (C3D4), and the steel reinforcement was

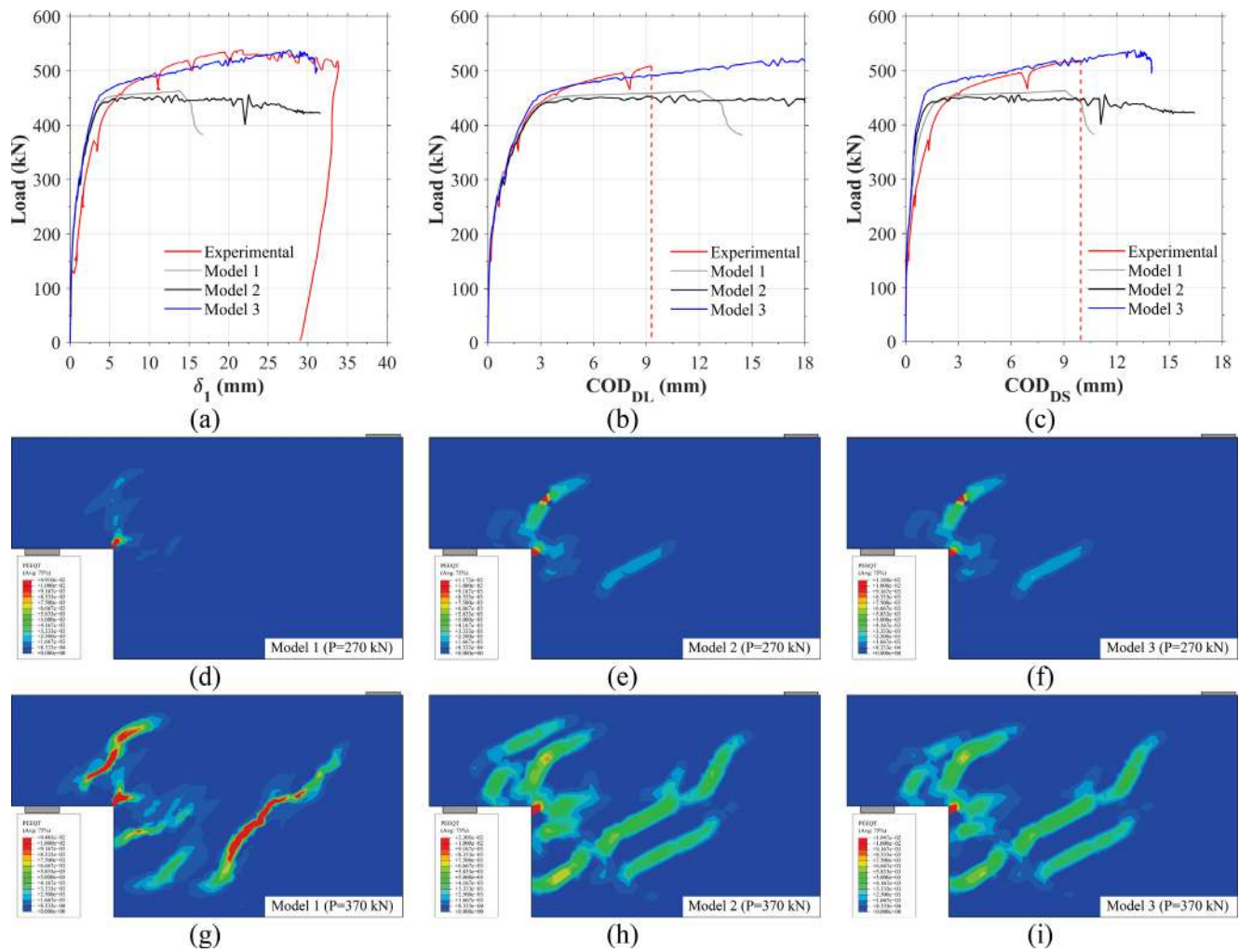


FIGURE A7 Results from RU7: Load- δ_1 (a), load-COD_{DL} (b), and load-COD_{DS} (c) curves of Phases 1–3; crack pattern at 270 kN for Phase 1 (d), Phase 2 (e), and Phase 3 (f); crack pattern at 370 kN for Phase 1 (g), Phase 2 (h), and Phase 3 (i).

discretized as two-node linear beam elements (B31). The test was displacement-controlled, with simply supported boundary conditions imposed on the beam.³⁶

A.7.2. | Description of changes implemented for Model 2

The standardized parameters were adopted for Model 2. The main modifications with respect to Model 1 were the 2D discretization, the 25 mm structured mesh, and the implementation of a dynamic analysis.

A.7.3. | Description of changes implemented for Model 3

Model 3 was developed considering all the properties set during Phase 2. In addition, a confinement effect was

induced to better reproduce the experimental results. To do so, a translational spring was introduced both in correspondence to the point of application of the load and in correspondence to the dapped-end support.

A.7.4. | Lessons learned

- Geometrical discretization is crucial for a better representation of the crack pattern. In particular, four-node linear tetrahedron elements for concrete (Model 1) provide a more refined distribution of cracks.
- Despite the distribution of steel reinforcement along the thickness of the beam, a good approximation of the response can be obtained with a 2D model approach, which represents a reduction in computational cost compared to 3D modeling.
- Correctly modeling the boundary conditions plays a very important role when attempting to reproduce

the experimental behavior of a specimen. In the reference experimental test, the beam was subjected to confinement effects due to the type of loading device used.¹⁷ This effect can only be numerically recreated with properly calibrated boundary conditions.

- A good agreement with the experimental behavior was obtained in terms of the crack pattern, with an initial diagonal crack at the inner corner of the nib and shear cracks propagating as the applied load increased. Nevertheless, some discrepancies can be observed when comparing Model 1 with Model 2 and Model 3 at load values of 270 kN and 370 kN. Fewer cracks with higher plastic strain (PE) values were obtained in Model 1, which can be attributed to the mesh (non-structured) and discretization (3D) adopted for this model.

A.8. | Research unit: 8

A.8.1. | Description of Model 1

3D four-node tetrahedral elements were used to model concrete via CDP modeling. Key CDP parameters, that is, dilation angle, flow potential eccentricity, strength ratio, second stress invariant ratio, and viscosity, were set equal to 31° , 0.1, 1.16, 0.67, and $5E-04$, respectively. The reinforcement bars were modeled as truss elements with a linear elastic-hardening response. The concrete compressive behavior was derived from Eurocode 2.²⁵ The tensile behavior was modeled as linear up to the elastic limit ($f_{ct} = 3.73$ MPa), with a linear tensile stress-inelastic strain softening branch. The fracture energy (G_f) was set equal to 1.47 N/m. A non-structured mesh was used. A roller constraint ($U_Y = 0$) was assigned to point A, and a hinge constraint ($U_X = U_Y = 0$) was assigned to point B. Dynamic/explicit analyses were carried out under displacement control.

A.8.2. | Description of changes implemented for Model 2

Model 2 differs from Model 1 in terms of (a) 2D modeling; (b) reinforcement modeled as a beam element; (c) lower f_{ct} (2.46 MPa); (d) lower G_f (0.918 N/m); (e) different CDP parameters (variation of dilation angle and viscosity, set equal to 20° and $1E-05$, respectively); (f) inversion of boundary constraints in A and B; (g) structured mesh; and (h) static analysis (displacement

control). The damage suboption parameters (tension and compression degradation variables DAMAGET and DAMAGEC in Abaqus, respectively) were not included in the standardized parameter set, and these parameters were assumed to increase linearly with increasing inelastic tensile strain (DAMAGET) and increase linearly with increasing inelastic compressive strain (DAMAGEC) according to a model compliant with the Abaqus Manual.

A.8.3. | Description of changes implemented for Model 3

Model 3 differs from Model 2 for (a) higher f_{ct} (3.73 MPa); (b) different CDP parameters (variation in the dilation angle from 20° to 30°); (c) more refined constraint modeling of the boundary conditions associated with the loading point and the dapped-end supports by means of *translator connector* elements; and (d) dynamic/explicit analysis (displacement control) instead of static analysis (displacement control).

A.8.4. | Lessons learned

- 3D models are more accurate and less irregular than 2D models, but the analysis time is generally longer. The analysis time associated with 3D models often ranges within 5 to 10 times the 2D model's analysis time. This result cannot be generalized as it might be conditioned by model/analysis specificities and technical features of the computing device. Furthermore, the assessed dataset is limited and does not have statistical significance.
- The variations in the boundary conditions (Models 2 and 3 from Model 1) led to better estimates of the stiffness and capacity.
- Static analysis has a longer analysis time than explicit dynamic analysis.
- 2D dynamic explicit analysis provides more disturbed results (than static analysis or 3D dynamic explicit analysis) even though the kinetic energy is lower than the recommended threshold; however, dynamic analysis results can be filtered to smooth the curves.
- The crack patterns confirm the greater accuracy of the 3D model (Model 1) with respect to the 2D models (Model 2 and Model 3). The cracks in the corner area of Model 3 have a more significant horizontal component than those in the other models; for Model 2, the cracks are not located exactly as detected in Model 1, and the damage zone is more widespread.

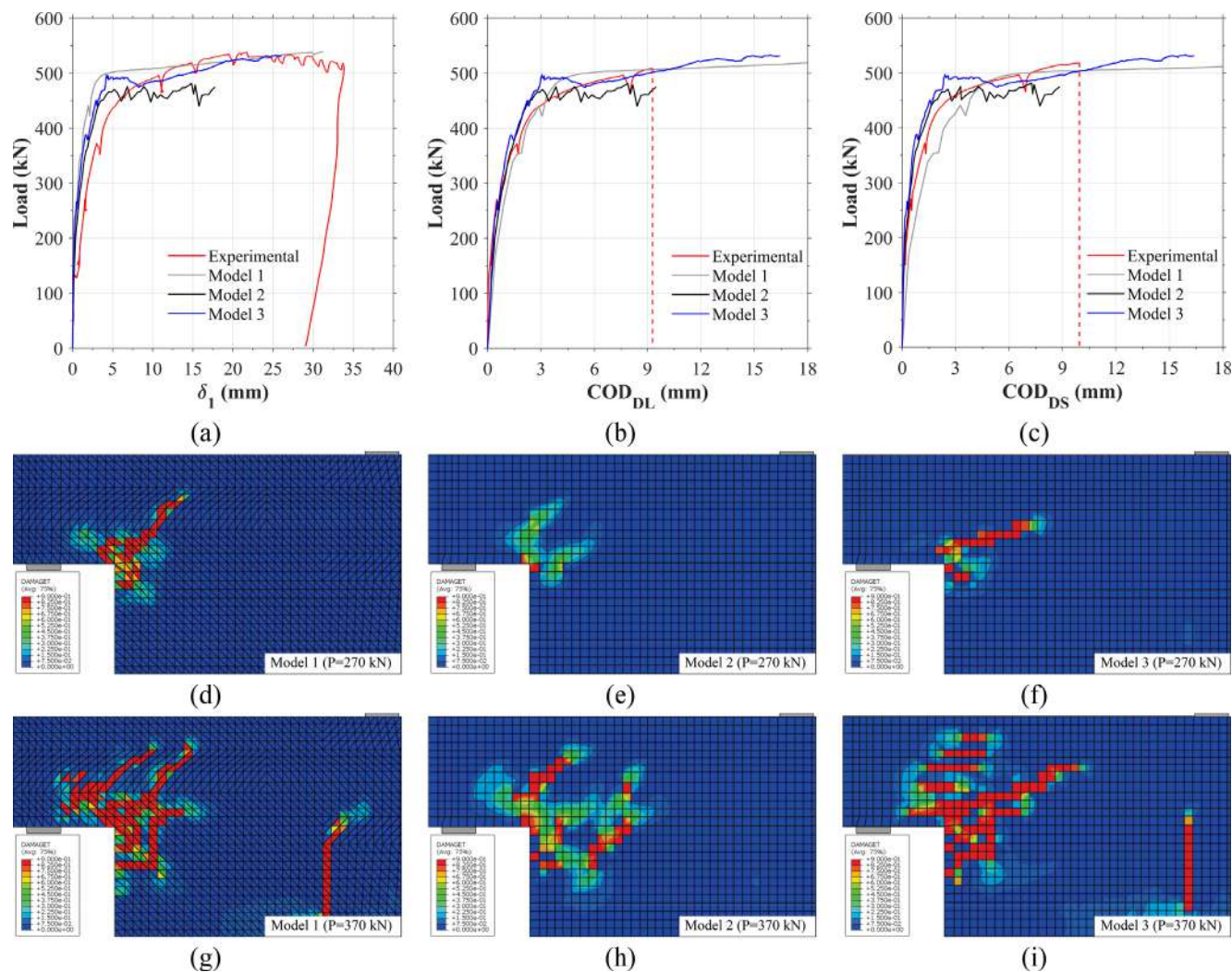


FIGURE A8 Results from RU8: Load- δ_1 (a), load-COD_{DL} (b), and load-COD_{DS} (c) curves of Phases 1–3; crack pattern at 270 kN for Phase 1 (d), Phase 2 (e), and Phase 3 (f); crack pattern at 370 kN for Phase 1 (g), Phase 2 (h), and Phase 3 (i).

APPENDIX B: Numerical identification of the main physical events from the single units

The discrete values of the variables used for the error calculations (E1, E2), and the load and vertical displacement (δ_1) associated to the identified mechanical phenomena are reported in Tables B1–B4.

TABLE B1 Results from the numerical estimations presented by the research units for Phases 1, 2, and 3: Discrete values of the identified key points (V), average values, coefficient of variation, and corresponding relative errors E1.

Variable (V)	Phase	RU1	RU2	RU3	RU4	RU5	RU6	RU7	RU8	Average	COV (%)
$P_{\max}$ (kN); E1($P_{\max}$) (%)	1	548.70;	532.88;	539.00;	562.94;	552.26;	541.66;	462.67;	538.76;	534.86;	5.36;
		2.00	−0.94	0.19	4.64	2.66	0.69	−14.00	0.15	−0.58	9.25
	2	543.53;	525.76;	525.00;	543.24;	553.69;	549.66;	455.96;	481.43;	522.28;	6.32;
		1.04	2.27	−2.41	0.98	2.92	2.17	−15.24	−10.51	−2.91	2.11
	3	543.53;	535.56;	525.00;	543.24;	553.69;	560.88;	537.56;	532.94;	541.55;	1.99;
		1.04	−0.45	−2.41	0.98	2.92	4.26	−0.07	−0.93	0.67	3.00
δ_1 (mm); E1(δ_1) (%)	1	30.78;	11.69;	22.88;	23.26;	27.38;	11.86;	13.82;	31.23;	21.61;	35.41;
		42.83	−45.75	6.16	7.94	27.05	−44.97	−35.87	44.92	0.29	123.39
	2	39.30;	14.20;	21.95;	18.30;	21.91;	16.17;	22.53;	14.84;	21.15;	35.60;
		82.37	−34.11	1.86	−15.10	1.66	−24.95	4.55	−31.15	−1.86	18.79
	3	39.30;	20.40;	21.95;	18.30;	21.91;	25.66;	27.60;	25.03;	25.02;	24.34;
		82.37	−5.32	1.86	−15.10	1.66	19.09	28.06	16.14	16.09	1.76
$P_{0.1}$ (kN); E1($P_{0.1}$) (%)	1	220.78;	209.20;	215.60;	170.86;	168.45;	196.66;	176.19;	83.55;	180.16;	24.46;
		53.27	45.23	49.68	18.61	16.94	36.52	22.32	−42.00	25.07	1.22
	2	167.69;	144.08;	131.25;	139.66;	183.44;	211.85;	192.18;	203.43;	171.70;	17.91;
		16.42	0.03	−8.88	−3.05	27.35	47.08	33.42	41.23	19.20	1.11
	3	167.69;	112.04;	131.25;	139.66;	183.44;	216.51;	194.12;	188.00;	166.59;	21.46;
		16.42	−22.22	−8.88	−3.05	27.35	50.31	34.77	30.52	15.65	1.58
$P_{0.3}$ (kN); E1($P_{0.3}$) (%)	1	269.23;	376.42;	269.50;	220.70;	252.90;	286.28;	261.91;	170.41;	263.42;	22.21;
		25.70	75.75	25.83	3.05	18.08	33.67	22.29	−20.43	23.00	1.19
	2	248.31;	197.66;	262.50;	222.07;	256.17;	293.51;	262.19;	244.18;	248.32;	11.56;
		15.94	−7.71	22.57	3.69	19.61	37.05	22.42	14.02	15.95	0.84
	3	248.31;	247.58;	262.50;	222.07;	256.17;	306.96;	262.46;	255.57;	257.70;	9.21;
		15.94	15.60	22.57	3.69	19.61	43.33	22.55	19.33	20.33	0.54

Note: Negative values of E1 indicate an underestimation of the response.

TABLE B2 Results from the numerical estimations presented by the research units for Phases 1, 2, and 3: Discrete values of relative error E2, average values and coefficient of variation.

Error E2 (%)	Phase	RU1	RU2	RU3	RU4	RU5	RU6	RU7	RU8	Average	COV (%)
E2(δ_1)	1	6.48	73.65	40.46	36.22	16.78	12.73	57.10	15.99	32.43	73.38
	2	5.01	61.90	43.31	18.28	35.20	11.73	19.86	54.16	31.18	66.32
	3	5.01	38.97	43.31	18.28	35.20	5.57	12.63	29.16	23.51	64.49
E2(COD _{DL})	1	5.70	12.58	11.22	7.11	10.25	10.64	4.54	6.70	8.59	34.15
	2	2.96	4.57	4.87	6.71	12.34	11.11	5.42	5.85	6.73	48.82
	3	2.96	6.01	4.87	6.71	12.34	14.28	2.06	4.94	6.77	64.10
E2(COD _{DS})	1	6.59	50.18	10.11	8.85	8.97	11.68	8.27	7.17	13.98	105.28
	2	6.46	27.69	4.97	7.95	10.64	11.42	10.52	16.55	12.06	59.82
	3	6.76	8.53	4.97	7.95	10.64	8.70	7.13	6.36	7.63	22.59
E2(COD _{DL,0-1})	1	16.73	23.25	23.50	24.12	12.36	18.66	5.62	34.53	19.85	43.78
	2	6.30	13.68	17.94	30.83	19.29	21.59	4.68	18.58	16.61	50.73
	3	6.30	33.46	17.94	30.83	19.29	54.47	4.90	14.05	22.66	72.46
E2(COD _{DS,0-1})	1	29.82	55.83	28.16	8.53	19.70	31.01	20.78	33.54	28.42	48.25
	2	26.78	18.52	12.77	13.72	26.07	33.89	28.04	13.69	21.68	37.03
	3	26.78	26.45	12.77	13.72	26.07	38.70	31.95	23.23	24.96	34.61

Note: E2 represents the absolute difference of the selected curve with respect to the experimental one. Hence, E2 values are always positive.

TABLE B3 Results from the numerical estimations presented by the research units for Phases 1, 2, and 3: Numerical load associated to several mechanical phenomena, average values and coefficient of variation.

Mechanical phenomena	Phase	RU1	RU2	RU3	RU4	RU5	RU6	RU7	RU8	Average	COV (%)
First cracking	1	73.21	75.64	188.65	153.67	—	182.86	58.89	300.73	147.67	58.65
	2	33.68	185.61	236.25	126.94	—	185.61	44.97	203.43	138.48	61.15
	3	33.68	90.02	236.25	126.94	—	258.47	45.30	177.00	138.24	64.42
Yielding of diagonal bar	1	356.94	410.02	296.45	350.46	428.84	364.35	337.78	396.67	365.14	10.13
	2	359.69	334.75	367.50	373.14	448.04	326.42	354.49	357.04	359.55	11.42
	3	359.69	332.73	367.50	373.14	448.04	331.17	359.26	385.57	369.64	9.95
Yielding of 1st stirrup	1	419.73	—	431.20	383.17	443.53	417.85	377.52	421.99	413.57	5.89
	2	393.65	404.86	446.25	407.69	461.17	383.32	427.23	449.44	421.70	6.77
	3	393.65	408.28	446.25	407.69	461.17	389.50	444.32	463.57	426.80	7.09
Yielding of horizontal bar	1	442.79	—	485.10	462.03	508.97	—	404.84	479.43	463.86	7.87
	2	438.01	468.31	498.75	455.55	514.84	—	427.23	470.49	467.60	6.68
	3	438.01	465.27	498.75	455.55	514.84	357.15	444.32	485.10	457.37	10.60
Yielding of 2nd stirrup	1	462.73	—	512.05	478.23	520.56	482.43	448.50	509.60	487.73	5.57
	2	461.94	—	472.50	479.30	521.10	445.72	450.56	478.69	472.83	5.29
	3	461.94	—	472.50	479.30	521.10	461.39	470.82	494.76	480.26	4.43
Yielding of 3rd stirrup	1	494.29	—	—	517.26	—	494.29	—	—	494.29	2.38
	2	496.35	—	525.00	531.63	—	485.77	—	—	509.69	5.29
	3	496.35	—	525.00	531.63	—	491.26	526.37	—	514.12	3.66

Note: Values reported in kN.

TABLE B4 Results from the numerical estimations presented by the research units for Phases 1, 2, and 3: Vertical nib displacement (δ_1) associated to several mechanical phenomena, average values and coefficient of variation.

Mechanical phenomena	Phase	RU1	RU2	RU3	RU4	RU5	RU6	RU7	RU8	Average	COV (%)
First cracking	1	0.08	0.08	0.17	0.26	—	0.32	0.08	0.69	0.24	91.17
	2	0.04	—	0.29	0.18	—	0.31	0.05	0.37	0.21	67.97
	3	0.04	0.17	0.29	0.18	—	1.36	0.05	0.17	0.32	143.13
Yielding of diagonal bar	1	1.41	1.60	0.71	2.00	1.96	1.41	1.62	1.34	1.51	26.84
	2	2.00	1.75	1.17	1.89	1.99	1.03	1.95	1.64	1.68	22.48
	3	2.00	2.04	1.17	1.89	1.99	2.03	1.80	1.74	1.83	15.79
Yielding of 1st stirrup	1	2.43	—	2.16	2.34	2.16	2.06	2.14	2.13	2.20	5.89
	2	2.45	2.52	1.95	2.30	2.20	1.49	3.39	3.48	2.47	27.45
	3	2.45	2.89	1.95	2.30	2.20	2.70	3.32	3.53	2.67	20.73
Yielding of horizontal bar	1	2.83	—	3.24	3.73	5.44	—	2.68	2.76	3.45	30.56
	2	3.25	3.77	4.27	3.09	5.53	—	3.39	5.20	4.07	23.78
	3	3.25	3.94	4.27	3.09	5.53	2.28	3.32	7.18	4.11	38.20
Yielding of 2nd stirrup	1	3.64	—	4.23	4.55	7.76	3.41	4.34	11.17	5.59	51.11
	2	4.32	—	2.40	4.22	7.85	2.42	8.44	15.30	6.42	71.51
	3	4.32	—	2.40	4.22	7.85	5.92	5.56	12.94	6.17	55.57
Yielding of 3rd stirrup	1	10.06	—	—	8.32	—	4.74	—	—	7.71	35.17
	2	9.10	—	21.95	12.85	—	3.40	—	—	11.82	65.88
	3	9.10	—	21.95	12.85	—	8.22	23.24	—	15.07	47.09

Note: Values reported in mm.