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Finite element modelling of a beam to column dowel connection calibrated on experimental data

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Abstract

This paper investigates the failure mechanism of beam to column dowel connections for precast structures under lateral loads. Cantilever columns tied together with beams through steel dowels have been commonly used for many years, particularly in Europe. However, the structural behaviour and dissipative capabilities of these systems are not fully understood. To address these aspects large-scale experiments were conducted through monotonic loading by using a common beam-to-column dowel assembly. Additionally, the study aims to provide a comprehensive finite element analysis calibrated versus the experimental data. These calibrated models replicate the experimental results and provide additional information on the behaviour of the beam to column dowel connections. The results of this research demonstrate that these connections are a weak point that determines the premature failure of the assembly. The calibrated finite element models were able to reproduce the observed failure mechanism and provide in-depth information which was not possible to record during testing. The findings of this study suggest that design considerations and procedures for these connections should be improved.

Rezumat

Această lucrare investighează mecanismul de cedare a conexiunilor dintre grinzi și stâlpi pentru structurile prefabricate acționate de încărcări laterale. Stâlpii în consolă conectați de grinzi prin intermediul unor dornuri de oțel sunt utilizați frecvent de mulți ani, în special în Europa. Cu toate acestea, comportamentul structural și capacitățile de disipare ale acestor sisteme nu sunt pe deplin cunoscute. Pentru a aborda aceste aspecte, au fost efectuate experimente la scară reală prin încărcări monoton crescătoare asupra unui sistem uzual de conexiune grindă-stâlp. În plus, studiul prezintă o analiză numerică bazată pe Metoda Elementului Finit, calibrată în funcție de datele experimentale. Aceste modele calibrate reproduc rezultatele experimentale și oferă informații suplimentare privind comportamentul conexiunii grindă-stâlp. Rezultatele acestei cercetări demonstrează că aceste conexiuni reprezintă un punct slab care determină cedarea prematură a ansamblului prefabricat. Modelele cu elemente finite calibrate au fost capabile să reproducă mecanismul de cedare observat și să furnizeze informații detaliate care nu au putut fi înregistrate în timpul testelor. Rezultatele acestui studiu sugerează că ar trebui îmbunătățite considerațiile și

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procedurile de proiectare pentru aceste conexiuni.

Keywords: Beam-to-column dowel connections, precast structures, structural behaviour, experimental testing, finite element.

1. Introduction

The design of seismic-resistant precast concrete structures presents a challenge due to the inherent discontinuity in the structural elements. Ensuring continuity, redundancy, and proper diaphragm behaviour at each story is crucial for the seismic resistance of these structures. While there are multiple solutions available in the precast industry, the "Portal frame system" is the most common system used. Typically, the columns are secured in moment resisting foundations, while the beams are supported on top of the columns, forming the skeleton of the structure. The behaviour and safety of these precast structures under lateral loads, and more specifically under seismic actions, has been a source of numerous unanswered questions. It is clear that the connections used in precast structures play a crucial role in their behaviour, but assessing the wide range of connection types is challenging due to the significant impact that even small changes can have on the overall behaviour. European codes and joint research programs suggest that each type of connection should be thoroughly investigated and verified using both numerical and experimental methods. In Europe, there is limited understanding of seismic design for precast elements, which is further exacerbated by the absence of specific design procedures in Eurocode 8 [1]. To address this gap, ongoing efforts are focused on improving knowledge and understanding of the seismic response of precast structures, including initiatives such as the SAFECAST project (Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions) [2], and fib Bulletin 78 - Precast-concrete buildings in seismic areas [3]. Previous studies have focused primarily on the local failure of dowels, making it difficult to apply design recommendations for seismic structures where global failure is possible, [4]. However, the Precast Seismic Structural Systems project (PRESSS) has been used by some researchers to investigate precast structures, including large-scale testing to determine the seismic response of precast buildings [5]. The goal of this research was to develop design recommendations to increase acceptance of precast concrete structures in seismic areas and to develop concepts, and techniques for high seismicity regions. The COST C1 action in Europe [6], which involved over 125 research centres from 25 countries, studied the behaviour of semi-rigid connections, with precast connections being a minor part of the project. Tampere University of Technology in Finland conducted full-scale tests on portal frames to determine the behaviour of beam-to-column connections under lateral loads [7]. Despite the promising results, it was evident that connections have a crucial role in the structural response of precast constructions, and additional research is required [8].

The seismic events of recent years have demonstrated the susceptibility of precast concrete structures to damage and even failure, with industrial buildings being particularly affected [9, 10, 11]. The Emilia Romagna earthquake in 2011 resulted in significant damage to many precast structures, highlighting the need for improved design and detailing of beam-to-column connections [12]. In many cases, failures in precast structures are traced back to the connections between the individual elements, with beam-to-column connections being a common failure point for these structures [10, 11, 13]. These last events have only confirmed what has already been shown during the Kocaely earthquake (Turkey, 1999) [14], and the L'Aquila earthquake (Italy, 2009) [15, 16].

According to The Eurocode 8 [1] guidelines, during the design phase of a structure, it is necessary to identify the function of each structural element and the impact of the connection on overall energy dissipation capacity. Connections located in critical regions should be appropriately overdesigned to have an elastic behaviour or provide sufficient ductility to contribute to the dissipation mechanism, while connections outside of critical regions should not impact the energy dissipation capacity of the structure. This approach follows the capacity design principle, where a structure should be designed to dissipate sufficient energy without significant reduction in resistance for horizontal and vertical loads. In the case of simply supported beams, which are typical for precast industrial halls, connections must transmit the seismic-induced horizontal forces. If the connections used are not sufficiently covered in the Eurocode provisions, experimental testing should be conducted to assess their capacity [1].

The SAFECAST report [2] recommends conducting thorough experimental and numerical investigations on the behaviour of any connection system and configuration that was not previously investigated. Figure 1 illustrates a common beam to column dowel connection used in Eastern Europe where the dowels are embedded in the column forks and inserted into the beam flanges during assembly and later grouted. However, there is a gap between the column fork and beam flange where the dowel is not embedded in any element, and this could affect its capacity. Since there is no similar connection studied in existing literature, it is difficult to evaluate its capacity. To validate its behaviour and suitability for seismic areas, it is essential to conduct experimental investigations on this specific geometry. The common reasons for failure of such connections are dowel yielding, and concrete spalling around the dowel, due to inadequate concrete coverage or confining reinforcements and it is considered a local failure [4]. As it was previously mentioned these connections should have an elastic behaviour and transmit the lateral loads so that the dissipative mechanism start forming based on the capacity design principles. The experimental part of this research as well as the finite element (FE) analysis investigate the behaviour of this dowel connection under lateral loads.

According to the research, experimental tests are suitable for determining failure mechanisms [17], but they have limitations in terms of data recording channels [18]. FE models, on the other hand, can provide more details regarding the behaviour and failure of the elements. These models can fill in the shortcomings of test results and provide information on the internal behaviour and deformation of the elements, which cannot be observed otherwise. Additionally, FE investigations can perform analyses using alternative loading/boundary conditions or design [17].

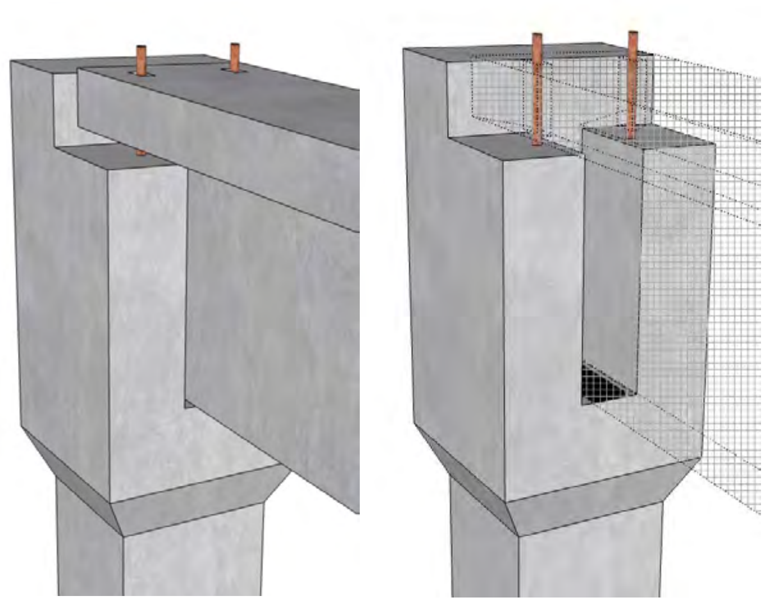

Figure 1. Typical precast dowel connection used in Eastern Europe

Various researchers have utilized FE modelling to assess the behaviour and strength of several types of concrete connections, as evidenced by the following references [19, 20, 21, 22, 23]. Additionally, other researchers have conducted investigations on the dowel mechanisms using FE models, finding that the contact interface between the dowel and surrounding concrete is critical to the overall behaviour of the precast structure [24, 25, 26]. However, it is challenging to model connections between precast elements with precision, mainly due to the fragmentation between the precast components. Calibrated FE models that are based on monotonically loaded test specimens can be employed to perform a cyclic or dynamic analysis to determine the strength degradation of connections, which may be lower for loading-unloading scenarios.

The focus of this chapter is to define and calibrate the FE models based on the experimental results presented in the previous chapter. This was done using DIANA FEA (Displacement ANAlyzer) [27] FE software, which is a versatile FE software with advanced modelling features and material models that are particularly suitable for modelling concrete.

The specific aspect of the investigated dowel connection is that it joins two large elements through a small connector element, which makes the area around the connection susceptible to significant local effects. Despite this, it is important to assess the overall behaviour of the structure in addition to the local effects.

2. Experimental testing under monotonic loads and results

2.1 Elements selection and test description

The lateral load capacity of beam to column connections can be affected by small changes in geometry, making it difficult to assess their performance using design codes alone. Therefore, a specific dowel connection commonly used in the precast industry in Romania was selected for investigation in this study. Figure 2 illustrates a variation of this dowel connection.

Figure 2. Beam to column connection solution used in Romania

Due to the large size of the precast elements some geometry adjustments were required in order to facilitate the manipulation of the elements in the testing laboratory. The objective was to minimize modifications to the test elements as much as possible to accurately capture the behaviour of a real precast frame structure with a beam to column dowel connection. The experimental investigation focused on the connection between an end column and a main beam. The dimensions of the column and dowel position used for the experiment can be seen in Figure 3 and the beam dimensions in Figure 4.

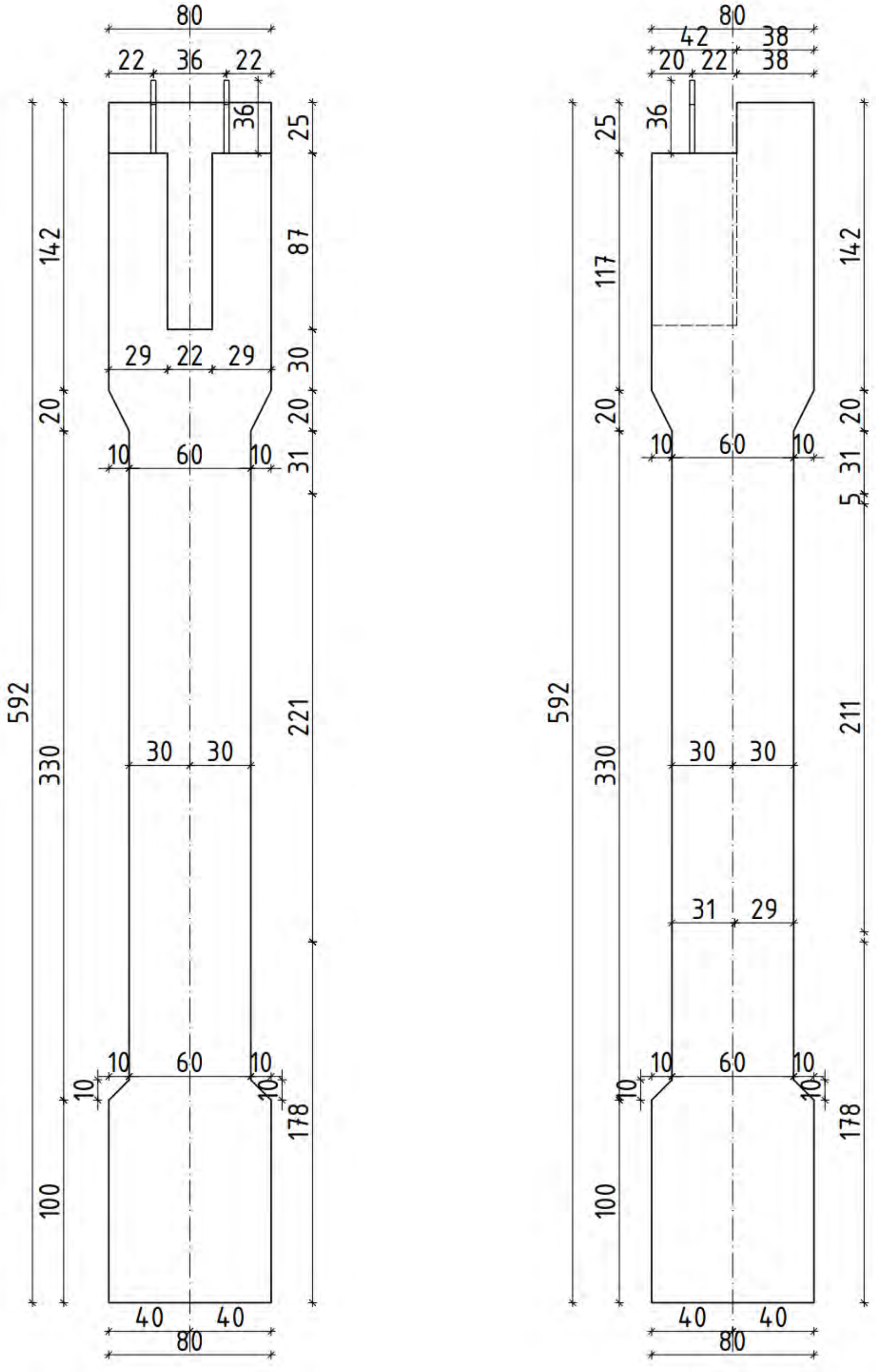

Figure 3. Column dimensions and dowels position

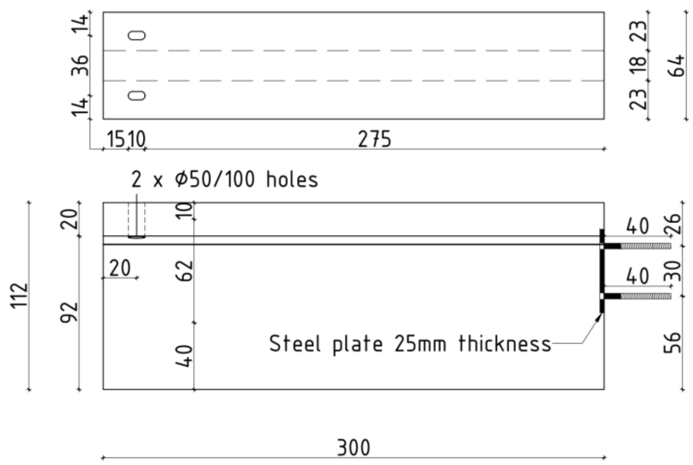

Figure 4. Beam dimensions

Due to the large size of testing specimens and for the safety considerations, the tests were performed horizontally on the laboratory floor. The downside of this approach is that it induces friction on the supported side of the test specimens. In order to reduce this effect, sliding plates were placed under the concrete specimens. The test setup is shown in Figure 5.

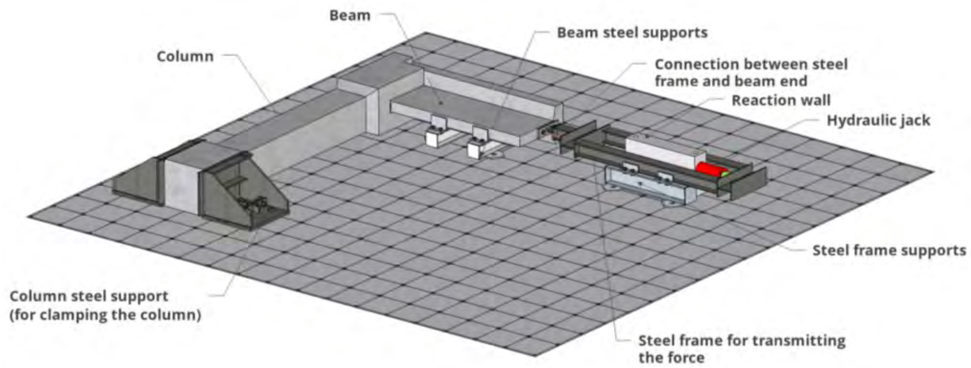

Figure 5. Perspective view of the testing setup

For the data acquisition two QUANTUM MX840B measuring amplifiers were used. Data was measured from a total of 16 channels and recorded using a sampling rate of 10Hz. Figure 6 shows the positions of the measuring equipment for this experiment. On this figure the position of the strain gauges (SG) and the force/displacement transducers (CH) are indicated. CH1 and CH2 shown in Figure 6 measure the relative displacement between the column fork and the beam end.

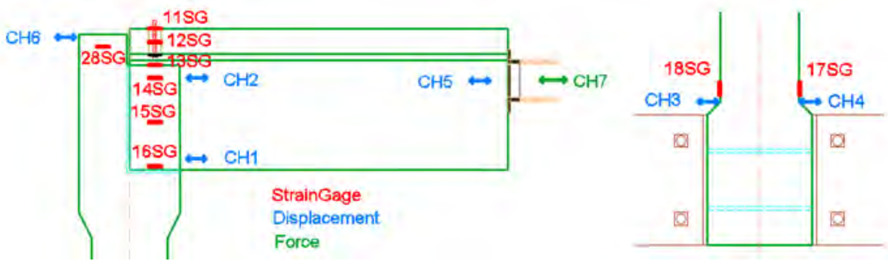

Figure 6. Measuring equipment position

2.2 Material properties

During the casting of the tested elements, concrete specimens were collected. The material properties determined on concrete specimens according to the Eurocode provisions [28, 29, 30] on the day the tests were performed are: compressive strength (for both cube and cylinder), tensile strength, modulus of elasticity (see Table 1). As for the reinforcements, including the dowels, S500B steel class was used with the following average steel properties: f_y=561MPa, f_{max}=656MPa, ε_{max}=10.73%. The dowels were grouted using Sika AnchorFix®-3030 product [31].

Table 1. Compressive strength results on cubes (individual and average values)

Specimen		Compressive strength cube (f_c)	Average cube (f_{cm})	Compressive strength cylinder (f_c)	Average cylinder (f_{cm})	Tensile strength (f_{cf})	Average ($f_{cf,m}$)	Elasticity modulus (E_c)	Average (E_{cm})
		(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
MT	1	73.53	75.45	68.1	68.6	5.16	4.76	34795	33115
	2	78.46		71.2		4.33		31051	
	3	74.36		66.4		4.78		33499	

2.3 Monotonic loading test results

The test was force controlled with a Δd =10kN increment at each step until failure. After each step, the cracks and the connection were checked. The maximum load reached was 208kN which corresponded with the failure of the dowel connection. Because the tests were performed horizontally after analyzing the results it was determined that the kinetic friction force was constant during the test, and it was estimated at around 30kN. The measurements of the displacement transducers in relationship with the force measurement from the applied load are shown in Figure 7 to Figure 11. During the test it was noticed that the column clamp allowed for some movement leading to unwanted rotations of the column base. This is visible in the measurements recorded by the transducers located at the base of the column, CH3 and CH4 (Figure 11). The column base measurements show a continuous increase in displacement up to the failure of the connection. The recorded values do not correspond to a deformation of the column itself; they show the rigid body rotation of the entire column. Because of this the top displacement of the column at failure was approximately 180mm.

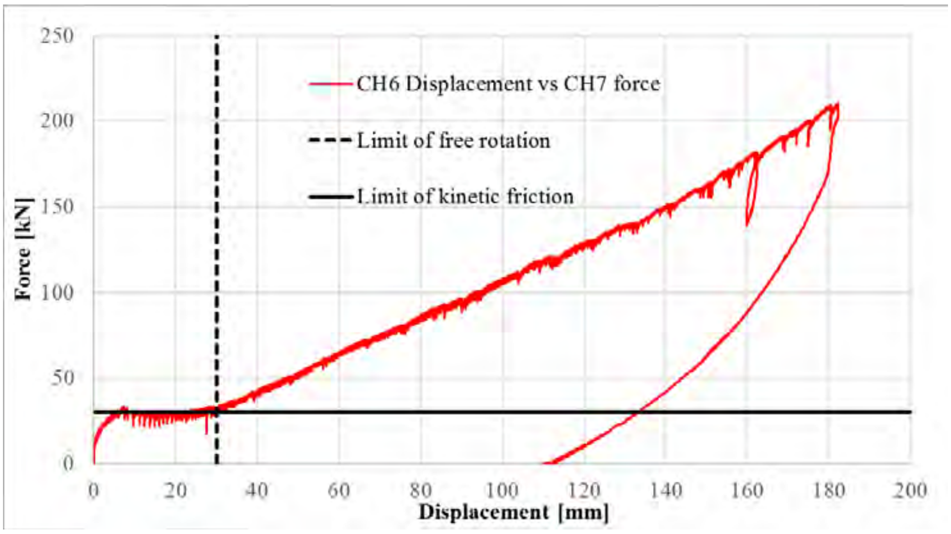

Figure 7. CH6 measurement

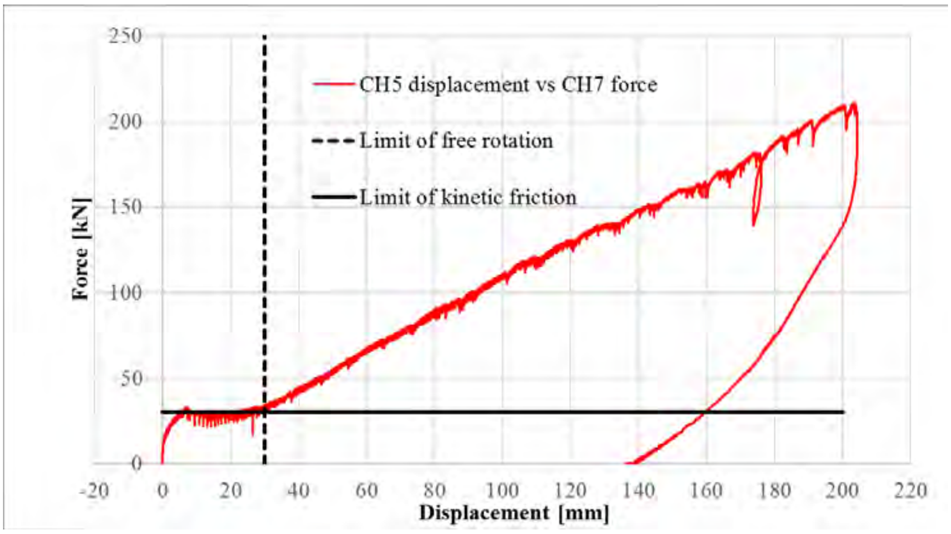

Figure 8. CH5 measurement

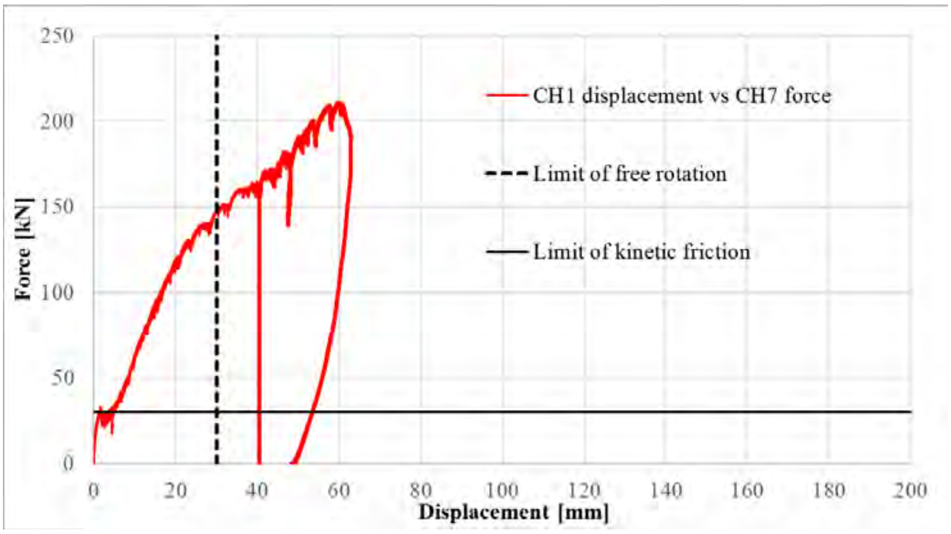

Figure 9. CH1 measurement

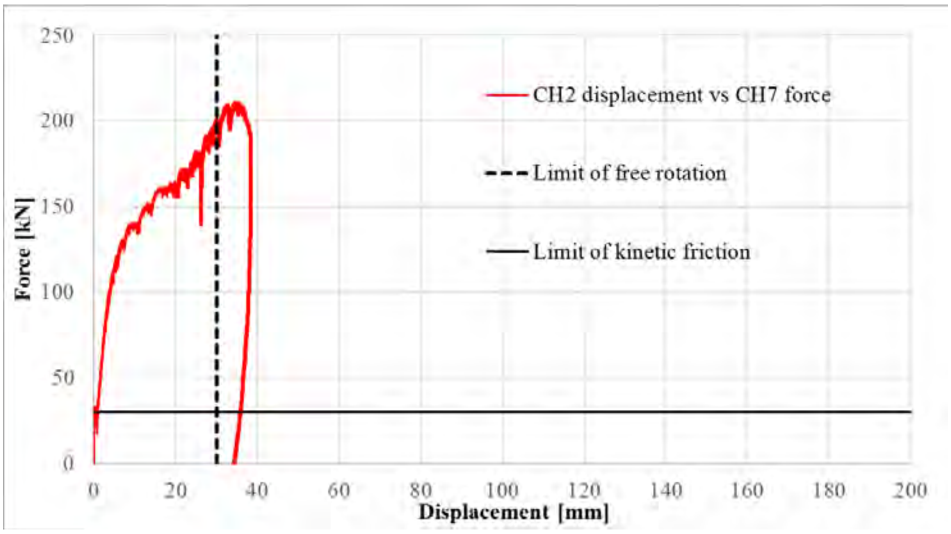

Figure 10. CH2 measurement

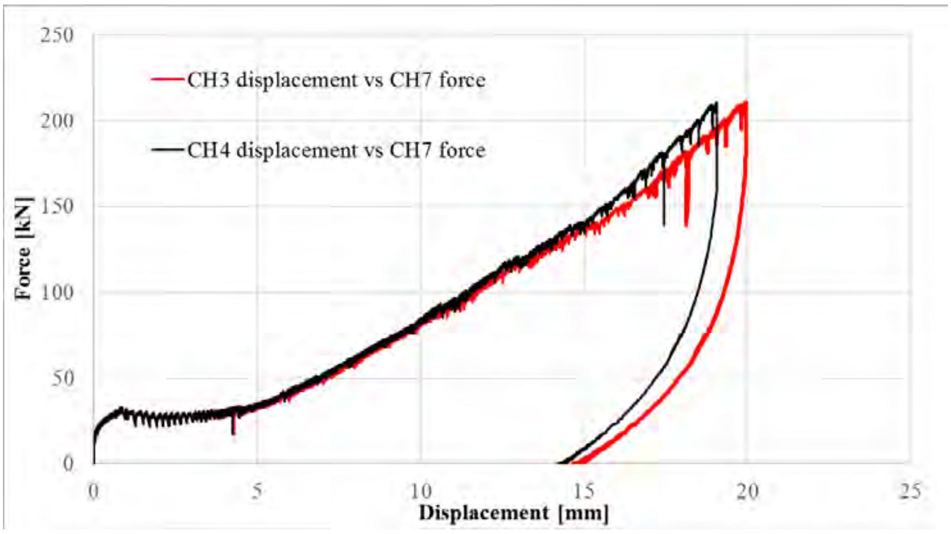

Figure 11. CH3 and CH4 measurement

The dowel connection experienced high shear and bending stresses, which caused the connection to slide leading to significant relative displacements between the beam and column fork. As these stresses increased the deformation became more visible. The failure of the dowel connection was a combination due to a combination of shear and bending, and it was accompanied by localized concrete crushing around the dowels, in both the column and the beam side. Because the test was stopped before the complete failure of the dowel connection due to safety concerns the concrete crushing was not significant. The area around the dowel and the deformed shape of the dowel at failure can be seen in Figure 12.

At the maximum load of 208kN the rest of the elements did not show significant damage. The maximum crack width with a value of 0.45mm was observed at failure at the base of the column (Figure 13), which confirms that the failure of the tested assembly was due to the failure of the dowel connection.

Figure 12. Failure - dowel yielding

Figure 13. Position of maximum crack width

Because there was no sudden rupture of the dowel at the end of the test, the beam was dismantled in order to see the deformed shape of the dowel. Figure 14 shows the deformed shape of the dowel which was inside the beam flange. It can be seen that the formation of the plastic hinge for the dowel was inside the grouted area of the beam flange.

Figure 14. The dowel after dismantling the beam

3. FE modelling and calibration

3.1 FEM geometry and material properties

Because of the complex geometry of the column fork and its assembly with the column, a simplified 2D approach was not feasible. The overlap between the column fork and the beam required the use of 3D geometry for building the models (Figure 15).

The concrete elements, beam and column, were modelled using isoparametric solid brick CHX60 finite elements [27]. The software automatically selects the type of reinforcement elements based on the type of element used for the concrete volume. For CHX60 concrete elements, truss reinforcements are modelled using CL9TR [27] elements that can only take axial strains and stresses, while beam reinforcements, such as dowels that also resist shear forces, are modelled using CL18B [27] elements. Quadrilateral interface element type CQ48I [27] was used for the plane interfaces. This element is commonly used for surface-to-surface contact interfaces and boundary interfaces. It can be assigned values for normal stiffness and shear stiffness.

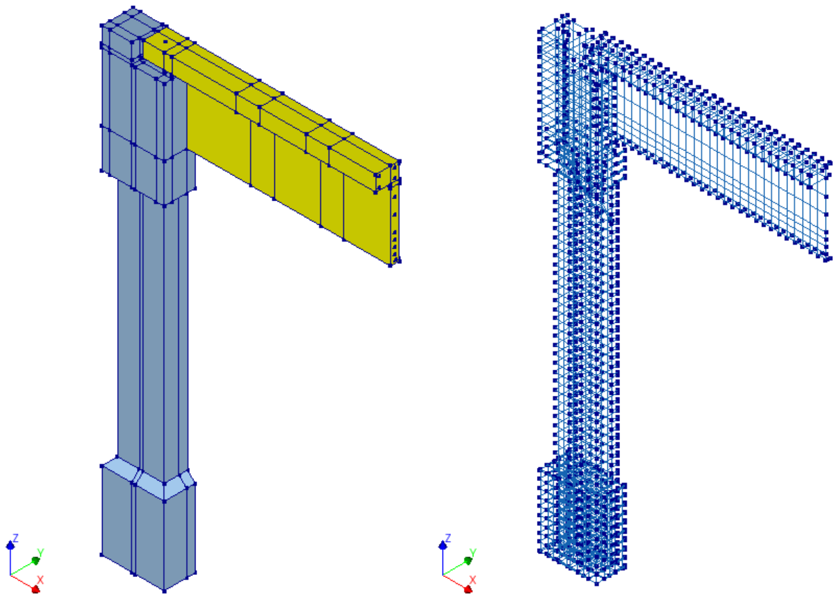

Figure 15. Column and beam model geometry/ Reinforcements geometry

The model geometry was based on the shop drawings used in the experiments and was constructed at full scale. Only half of the geometry was modelled due to its symmetry, as shown in Figure 15. To achieve a structured mesh and prevent inconsistencies caused by section changes and the cut-out of the column fork, additional guiding lines were drawn on the surface of the elements to constrict the meshing algorithm and maintain the structure of the mesh.The selection of material constitutive models for the numerical simulations took into account the observed deformations in the experimental tests, particularly in the vicinity of the dowel. In order to accurately capture the plastic behaviour of materials in these critical areas, nonlinear constitutive models were utilized. The concrete elements were modelled using the "Maekawa-Fukuura Concrete Model" available in the DIANA FEA material library. This model was chosen due to its ability to capture the plasticization of materials in critical regions and its good performance in quasi-static and hysteretic analysis. The model is based on the research of Maekawa [32] and is an extension of a smeared cracking model, constructed using cyclic loading data. The implementation of the model was defined according to the software documentation [27], [33]. The concrete model was combined with the model proposed by Vecchio and Collins [34] in order to account for the influence of lateral cracking that decreases the concrete's compressive strength for large tensile strains perpendicular to the principal compressive direction. The values used for the model are shown in Table 2.

Table 2. Maekawa-Fukuura Concrete model parameters

Linear properties	Mass density	2.5 t/m^3
	Strain at compressive strength	0.2%
Tensile behaviour	Tensile strength	4.1 N/mm^2
	Plateau end strain	0.02%
	Power C	0.4
	Threshold angle	22.5°
Compressive Behaviour	Uniaxial compressive strength	68 N/mm^2
Reduction model	Vecchio and Collins 1993	0.6

The Menegotto-Pinto plasticity model [35] was used to model the behaviour of all the longitudinal reinforcements and stirrups. The model parameters were determined based on the S500 steel class utilized in the experiment. Additionally, specific parameters for this model were set according to the technical documentation of the software and the material constitutive model's experimentally determined specifications [36, 37]. The values used in this FE model are shown in Table 3.

Table 3. Menegotto-Pinto steel reinforcement parameters

Menegotto-Pinto steel reinforcement parameters	Young's modulus	194600 N/mm^2
	Poisson's ratio	0.3
	Mass density	7.85 T/m^3
	Yield stress	432.63N/mm^2
	Initial tangent slope	0.00286359
	Initial curvature parameter	20
	Constant a_1	18.5
	Constant a_2	0.15
	Constant a_3	0.01
	Constant a_4	7

The Von Mises plasticity model was used for the steel dowel. This was necessary because of the gap between the column fork and the beam, which results in a significant buckling component for the dowel in addition to tension and compression along its axis. Table 4 contains all the model parameters for the Von Mises plasticity model used for the steel dowel. The hardening distribution factor of 0.5 is defined to represent an equal distribution between isotropic and kinematic hardening.

For modelling the interfaces between the steel plate assemblies used to fix the column and the beam (Figure 5), a Coulomb Friction model was used. This model requires defining the normal and shear stiffness modules, as well as the characteristic parameters for Coulomb friction, including cohesion, friction angle, and dilatancy angle. A relatively small friction angle was used due to the low friction strength of the interface. The model was set up according to the guidelines in the DIANA FEA technical manual [27]. The objective of the model was not to study the interaction of the structure with the supports, so models were chosen to minimize the concentration of large stresses in the interaction areas to avoid convergence issues. The values used for the column and beam interfaces can be seen in Table 5 and Table 6.

Table 4. Steel dowel Von Mises material model parameters

Linear properties	Young's modulus	200000 N/mm^2
	Poisson's ratio	0.3
	Mass density	7.85 T/m^3
Nonlinear properties	Hardening function	Plastic strain-yield stress
	Hardening hypothesis	Strain hardening
	Hardening type	Mixed isotropic-kinematic
	Hardening distribution factor	0.5

Table 5. Column interface – Coulomb surface interface material properties

Linear properties	Normal stiffness modulus (z-dir)	0.15 N/mm^3
	Shear stiffness modulus (x-dir)	0.001 N/mm^3
	Shear stiffness modulus (y-dir)	0.001 N/mm^3
Nonlinear properties	Cohesion	0.01 N/mm^2
	Friction angle	40°
	Dilatancy angle	0°
	Tension cut-off value	0 N/mm^2

Table 6. Beam interface – Coulomb surface interface material properties

Linear properties	Normal stiffness modulus (z-dir)	5 N/mm^3
	Shear stiffness modulus (x-dir)	1e-08 N/mm^3
	Shear stiffness modulus (y-dir)	1e-08 N/mm^3
Nonlinear properties	Cohesion	1e-08 N/mm^2
	Friction angle	10°
	Dilatancy angle	10°
	Gapping model -tensile strength	0 N/mm^2

The FE model was created using quadratic elements. The size of the FE mesh was determined based on the size of the model and the need for accurate results within a reasonable computational time. The mesh was generated using the division meshing algorithm of the software, with edges divided into a specific number of elements to ensure uniform size across all mesh elements. The mesh element size ranged between 50mm and 100mm (Figure 16), with a finer mesh used in the column fork where damage was observed during the experimental tests.The applied load was in the form of a point displacement at the end of the beam. To ensure equal displacements in the X direction, a tie was created between the selected point and the face of the beam. The purpose of the tie was to simulate the effect of the thick steel plate that was present at the end of the beam during the experimental tests, which was used to uniformly distribute the load.

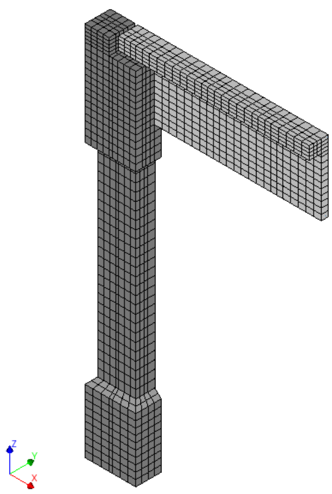

Figure 16. Qquadratic/hexagonal meshing

3.2 FEM results

To enhance the accuracy of the FE model, the force-displacement results of the monotonic test were adjusted by subtracting the values of the limit of the free rotation and the limit of the kinematic friction values, as shown in Figure 17. This was necessary as it is difficult to define the interface values to replicate this effect without additional experimental tests. Before implementing this new approach, the actual results from the monotonic test were used. However, this led to numerical instability because the model behaved like a mechanism up to the limit of free rotation, due to the extremely low stiffness required.

The FE results (Figure 18) exhibit a favourable correlation with the experimental tests, with both experiencing failure at approximately 180kN force. The FE maximum displacement was 155mm, whereas the experimental results were slightly lower at 153mm. The stiffness up to the failure point shows a minor difference, which may be attributed to the unique interactions between the guiding steel elements used for the beam and force transmitting frame during experimental tests. These effects cannot be entirely reproduced by the interface boundary conditions utilized in the FE model.

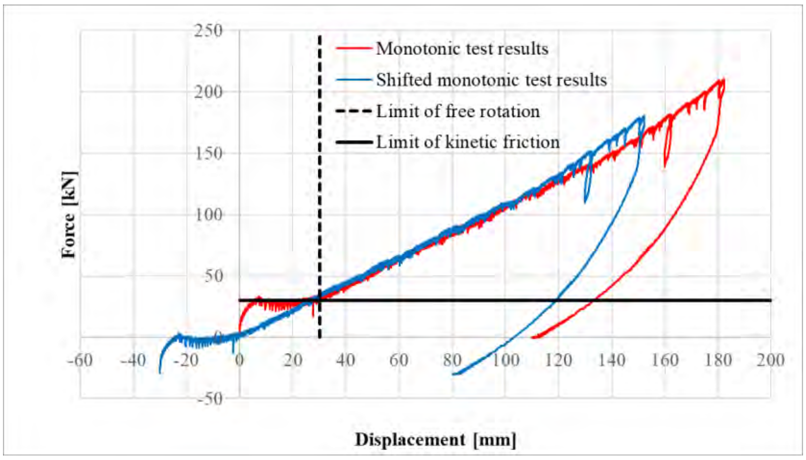

Figure 17. FE – Load displacement values

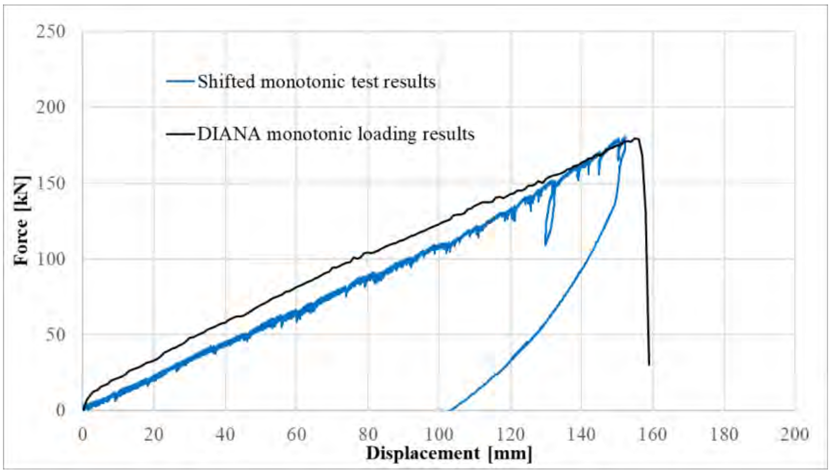

Figure 18. DIANA FEA results vs. Experimental results

The stress distribution for both the left and right sides of the dowel were examined. The failure of the assembly in the FE model was attributed to the high stresses in the dowel, as seen in Figure 19.

The remaining structure did not suffer significant damage, as indicated by the stresses in the longitudinal reinforcements of the column, which were three times smaller than those seen in the dowels at failure (Figure 20). Thus, it can be inferred that the column reinforcements have sufficient capacity remaining, indicating that the column was not close to failure. The maximum crack width in the numerical model was 0.98mm, located in the column fork near the dowel. A maximum crack width of 0.63mm was observed near the base of the column, which is 40% higher than the 0.45mm observed during the experiments. However, this may be a result of the coarse mesh used in the column to improve convergence.

The CL18B element type used for the dowel reinforcements can display both tensile and compressive stresses on the outer layers of the bar. As shown in Figure 19, the stress distribution inside the dowels corresponds to the location of the two plastic hinges, which are present in each concrete element. Additionally, stresses are concentrated near the concrete surfaces of both the beam and column fork. The significant stresses are only present up to a depth of approximately 10 cm.

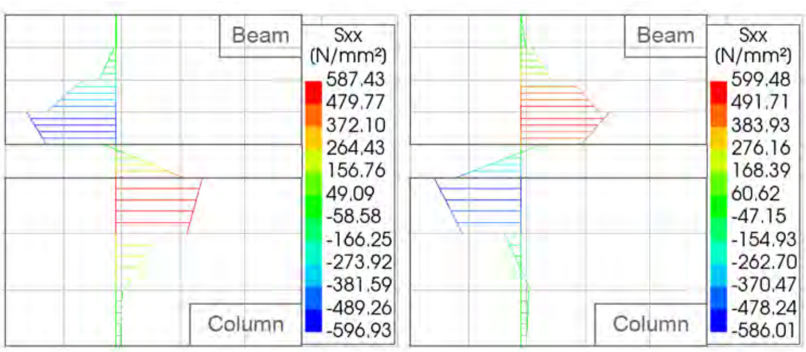

Figure 19. DIANA FEA – Stresses on the edge of the section along the steel dowel at failure – Left side of the dowel and Right side of the dowel

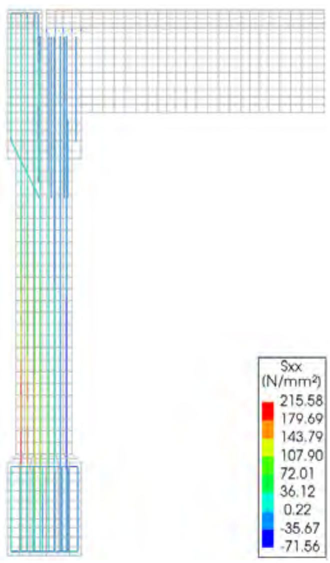

Figure 20. DIANA FEA – Stresses in the longitudinal reinforcements of the column

The dowels begin to yield at 80mm displacement in the lower part of the beam (Figure 21) with a Von Mises stress value of 577.92 N/mm2. At the same point, the longitudinal reinforcements in the column exhibit a stress value of around 100 N/mm2, as shown in Figure 22. It is important to note that this value is 75% lower than the yield limit for the reinforcement steel, this confirms that the dowel yielding, which ultimately led to the failure of the connection, started while the precast column was not exhibiting any significant stresses in the longitudinal reinforcements. Based on these results, it can be concluded that the column did not fail prior to the failure of the connection, and the longitudinal reinforcements of the column had a remaining capacity of 50% when compared to the design yield capacity of the reinforcement, which is 435 N/mm2.

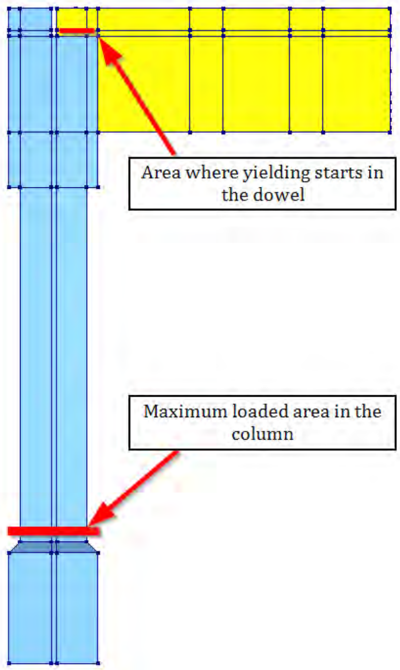

Figure 21. DIANA FEA – Location of maximum stress for the dowel and the column reinforcements

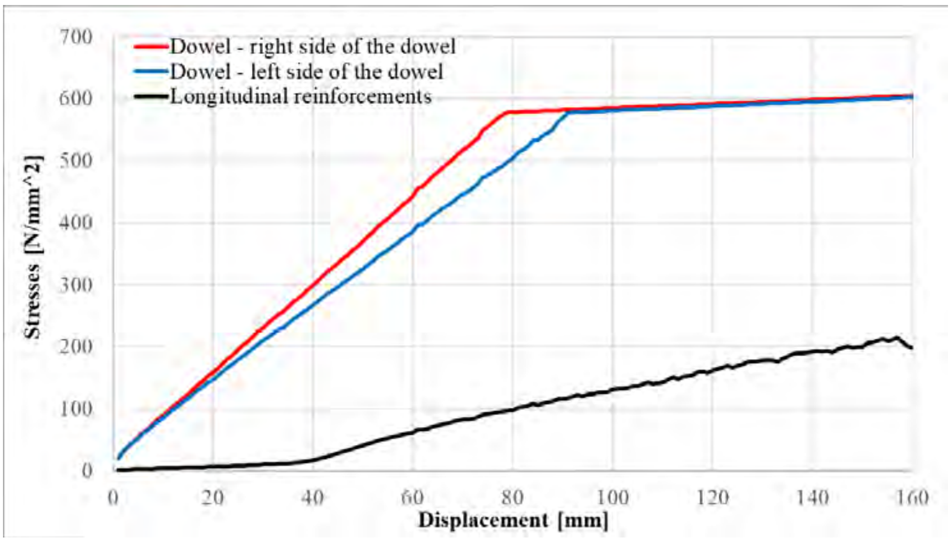

Figure 22. DIANA FEA – Von Mises stress variation in the dowel and longitudinal reinforcements

According to the design specifications outlined in EC8 [1] and P100 [38] the precast element connection for the inverted pendulum structural type should maintain its elasticity and the formation of a plastic hinge at the bottom of the column should serve as the mechanism for dissipation. The results of the experimental test and the FE model have shown that this is not the case for this particular dowel connection.

4. Conclusions

The findings of this study provide valuable insights into the behavior of precast dowel connections subjected to lateral monotonic loading. The obtained results confirm the hypothesis that the dowel connection has a lower capacity than the column, and therefore, it fails before any significant damage is observed in the precast elements. This situation may result in an undesirable failure mechanism under extreme lateral loads, such as those generated by a seismic event. In conclusion:

1. The FE model which was calibrated based on the experimental results was able to accurately predict the failure of the dowel connection and its displacement values. The FE model confirms that the failure of the assembly is due to the failure of the dowel connection rather than the failure of the precast elements.
2. The results obtained from the FE model were consistent with the experimental results, showing a failure load of 178 kN when accounting for the 30kN kinetic friction value.
3. The stresses in the longitudinal reinforcements of the column were much lower than those in the dowels at failure, indicating that the reinforcements still had significant capacity left. The column did not fail before the failure of the connection, and the longitudinal reinforcements showed a 50% capacity left considering the design yield force of the reinforcement.
4. The FE model indicated that the maximum crack width before the connection failed was 0.63 mm, which is 40% higher than the 0.45 mm observed during the experimental test. The crack width reached its maximum value of 0.98 mm at the top of the column fork, near the dowel, at the point of failure.
5. The difference in stiffness between the experimental and FE results up to the point of failure may be due to the different interactions between the test specimens and the guiding steel elements used in the experiments, which cannot be fully replicated in the model.

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