SEISMIC BEHAVIOUR OF NON-PREQUALIFIED CONNECTIONS OF REDUCED SECTION BEAMS WITH COMPOSITE COLUMNS

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ABSTRACT
Reduced beam section (RBS) connection is one of the pre-qualified systems used to ensure the formation of plastic hinges in earthquake-resistant frames. In the case of constructive systems used in buildings in Ecuador, it is common to use composite columns formed by two longitudinally welded steel U-sections filled with concrete and rigidly connected to steel I-beams using welding of webs and flanges. In order to ensure the formation of plastic hinges on the beam flanges at a safe distance from the column, the flange's cross-sections are reduced by symmetrical circular cut-outs. This type of rigid connections does not form part of the pre-qualified joints, in which steel columns with double T-sections are connected with RBS I-beams so it is necessary to carry out experimental and numerical studies to ensure its ductility due to the high seismicity in the areas in which they are used. In this work we study experimentally and numerically one of the most common designs for beam–column connections used in Ecuador. The design has been calculated numerically against cyclic loads and tested experimentally at the Housing Research Centre of the National Polytechnic School in Quito, following the specifications of the American codes used in Ecuador. The numerical results obtained show a good correlation with the experimental tests in the initial load cycles, but for medium loads a shear fracture develops in the welds of continuity plates and the ductility requirements of the regulations are not fulfilled. Therefore, new designs are proposed in order to improve the seismic response of the connections.
Keywords: seismic design, ductility, reduced beam sections, beam–column connections.

1 INTRODUCTION

Ecuador is located in an area of high seismicity within the Pacific Ring of Fire, between the Cocos, Nazca and Caribbean tectonic plates. In the seismic design of building structures, it is common to use three-dimensional moment-resisting frames with medium ductility, following the Ecuadorian Construction Standards, NEC-SE-DS [1], which is based on the United States AISC-358 [2] and FEMA-350 [3] standards and recommendations.

One common method for constructing moment-resisting beam-to-column connections involves using steel I-beams that are rigidly attached to the column. This is typically done by applying butt welds to the flanges and fillet welds to the beam web, which is connected to the column through a shear plate. The column is constructed from two longitudinally welded steel U-profiles and filled with concrete. To ensure the ductility of the connection, the beam flange sections are reduced in the proximity of the column (reduced beam section (RBS) connection) according to the AISC-358 [2], and continuity plates are attached to connect the beam flanges to the rear face of the column. These continuity plates must ensure the transmission of tensile stresses on the opposite face of the column to be absorbed by the concrete working under compression, and at the same time allow the concrete filling of the column, and a straightforward assembly of the steel column and the beam, with as minimum amount of welding as possible and an easy welding process.

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The use of a reduction of the beam flange section in the vicinity of the column has been extensively studied in the seismic design of moment resisting frames [4]–[6]. It has the fundamental advantage of securing the position at which the plastic hinge is formed, preventing it from forming at the connection with the column where the bending moment reaches its maximum.

The connection design evaluated in this work is shown schematically in Fig. 1. In this case, only a single beam is considered, connected to the column using two continuity plates. These plates match the dimensions of the beam flanges and extend across to the opposite face of the column. In the orthogonal horizontal planes, the continuity plates are split into two halves, each pre-welded to one of the steel U-profiles that make up the column. This setup requires a fillet weld on the back side of the plates connected to the beam, which is performed after welding the two column profiles together with the beam.

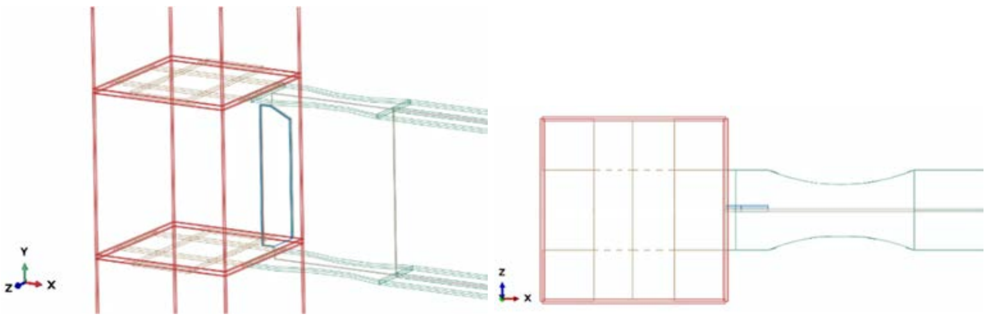

Figure 1: Three-dimensional and plan views of the column–beam connection.

This type of connection does not appear among the pre-qualified joints connections defined in the AISC-358 standard [2], in which steel columns with double T-profiles are connected with RBS I-beams, making it necessary to carry out experimental and numerical studies to ensure its ductility.

2 METHODOLOGY

This section describes the design of the proposed beam–column connection, the numerical model developed, the manufacture of the connection at 1:1 scale and the experimental test developed.

2.1 Proposed design

Based on the theoretical design of a high-rise building located in Quito (Ecuador) as shown in Fig. 2 and developed using ETABS software [7]; applying the seismic design regulations previously mentioned [1]–[3], a design of a typical beam–column connection located at one of the corners of the structure has been obtained with the dimensions and layout shown in Fig. 3.

From the design obtained, the development of a numerical model and its experimental testing is proposed.

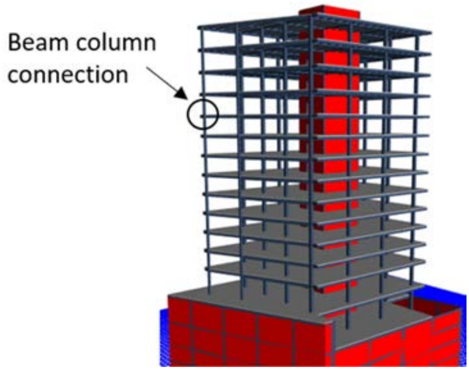

Figure 2: Three-dimensional building model and beam–column connection under study.

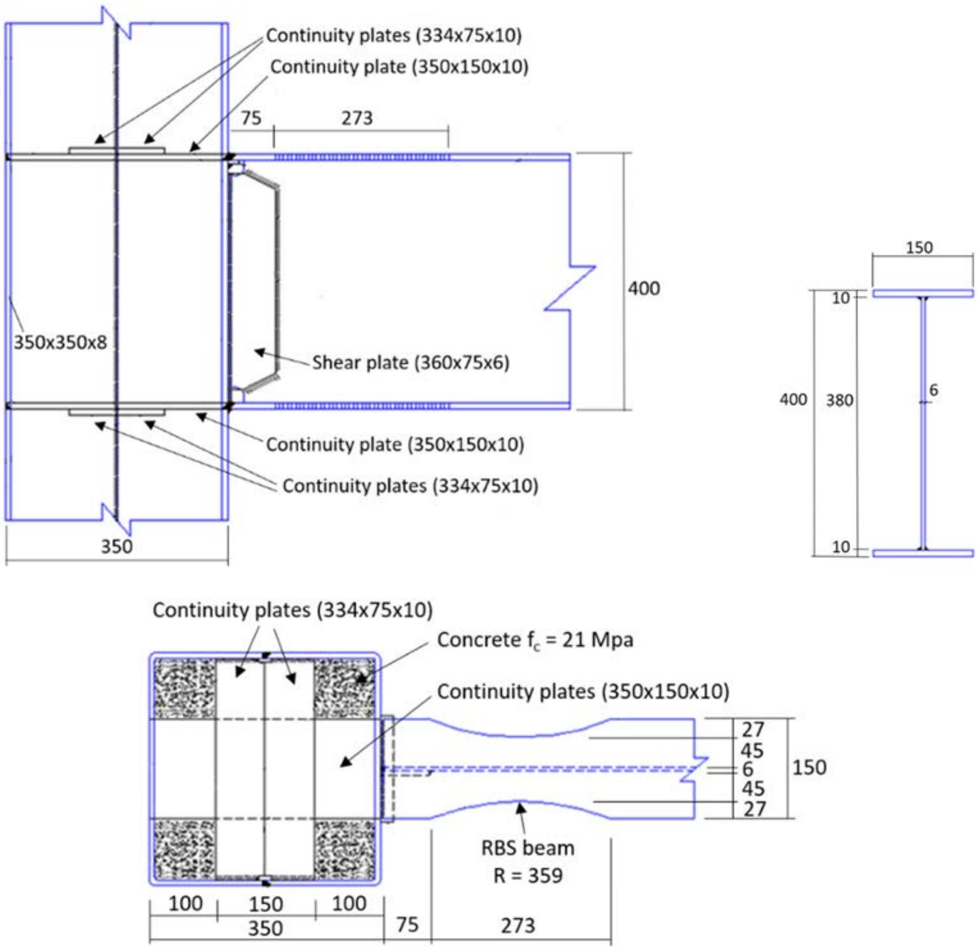

Figure 3: Connection component dimensions (mm) and welds.

2.2 Numerical model

As a preliminary step to the experimental test, a numerical finite element model (FEM) of the structure subject to cyclic load test was developed with Abaqus software [8], in order to verify the validity of the experimental test, and after that to validate the FEM with the experimental results. The final objective is to use this FEM later for the future study of cases with multiple beams connected to one column, and the development and optimisation of new configurations of the continuity plates.

The dimensions considered are those of the full-scale beam–column connection, described in the previous section, with a column length of 3.3 m and a beam length of 3.25 m (Fig. 4).

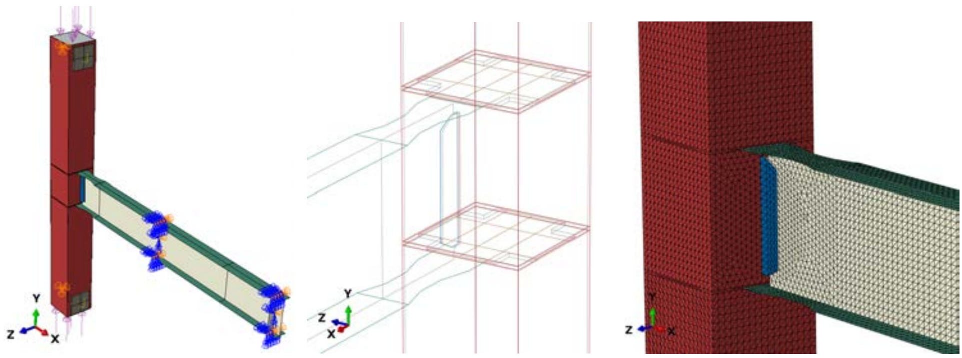

Figure 4: FE model: geometry, loads, boundary conditions and mesh.

With regard to the loads and boundary conditions, the AISC-341(Seismic Provisions for Structural Steel Buildings) standard and FEMA-350 are followed [3], [9]. These require that, to ensure the ductility of the connection, the beam–column connection must sustain at least 80% of the nominal plastic bending moment (M_p) for a rotation of 0.04 radians during two load cycles (see Fig. 5).

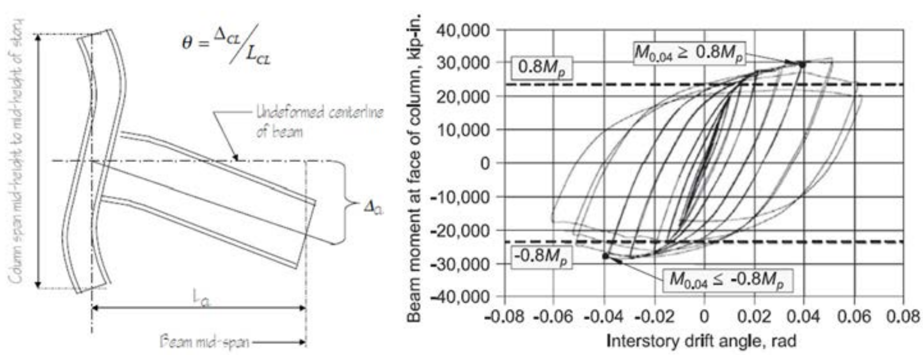

Figure 5: Cyclic test requirements (FEMA-350, AISC-341) and hysteresis cycles.

The load cycles are applied statically to the structure, at the beam end, with the incremental pattern defined in Table 1 for the proposed beam and column dimensions. The displacement is measured at the beam end where the cyclic load is applied, and the beam rotation is defined as shown in Fig. 5.

Table 1: Cyclic loading [3], [9].

Test phase	Number of cycles	Rotation θ (rad)	Displacement u (mm)
1	6	0.00375	12.2
2	6	0.005	16.3
3	6	0.008	24.4
4	4	0.010	32.5
5	2	0.015	48.8
6	2	0.020	65.0
7	2	0.030	97.5
8	2	0.040	130.1

Considering the dimensions of the I-beam (Fig. 3), and that the structural steel is an A36, the following values are obtained: M_p = 183.75 kNm, 0.8M_p = 147 kNm, and F_{min} = 0.8M_p/ L_{beam} = 45.2 kN. This means the structure must withstand two load cycles of ± 45 kN with a minimum displacement of ± 130.1 mm.

In the case of the FE model, the column ends are modelled as fixed hinged supports, and the load is applied at the beam end in the *Y* direction (Fig. 4), as a cyclic imposed displacement (Fig. 6), following the values in Table 1 with a linear variation. Both at the beam end and at an intermediate beam section, lateral buckling is prevented, as the actual beam is continuously braced by the floor slab.

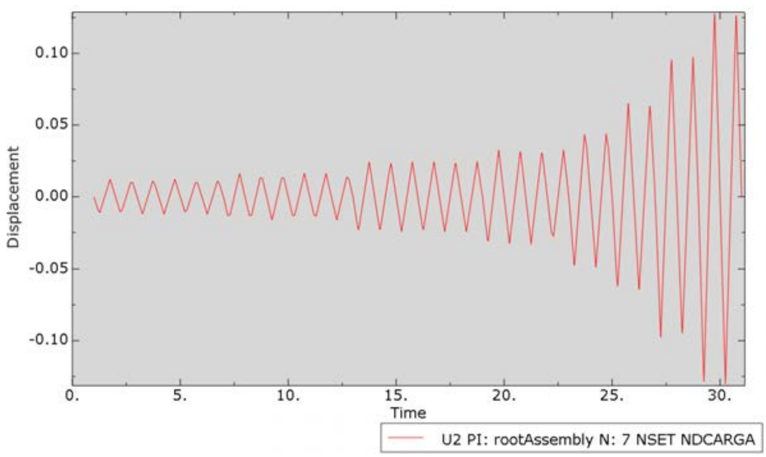

Figure 6: FE model: cyclic load introduced through imposed displacements (mm).

The type of analysis carried out is nonlinear implicit with large displacements, nonlinear concrete and steel materials, and contact constraints between concrete and steel column parts. The steel column and beam have been modelled with linear shell elements with reduced integration (S3R and S4R), whereas the concrete infill has been modelled with tetrahedral or cubic linear solid elements (C3D4 and C3D8R). Beam 3D elements B31 have been used for pre-stress bars in the hinged supports. The average element size varies between 10 mm and 50 mm, with the finest mesh located at the connection of the beam to the column.

The contact conditions between the filling concrete, the steel column and the continuity plates, are simulated with a hard contact model in normal direction and penalty friction coefficient $\mu = 0.25$ in tangential directions.

Two sequential steps are considered in the analysis; in the first step, an axial pressure of 5 MPa is applied to the concrete column in order to stabilise the contacts, and in the second step the load cycles (Table 1, Fig. 6) imposed as displacements at the beam end, at the point of load application by the hydraulic actuator, are applied.

Finally, the nonlinear material models considered are:

- A36 steel: Combined isotropic-kinematic plasticity model with cyclic hardening and ductile damage with the following properties:
  - Constants: $\rho = 7.85$ T/m^3, $\alpha = 1.2 \times 10^{-5}$°C^{-1}
  - Elastic parameters: $E = 200$ GPa, $\nu = 0.3$
  - Plastic parameters: $f_y = 260$ MPa, $b = 5.8$, $Q = 227.8$ MPa, $g = 10.7$ and $C =$ 1,617 MPa
  - Ductile damage parameters: $\varepsilon_u = 0.46$, $G_f = 168$ MPa
- HA20 concrete: Concrete damage plasticity (CDP) model, with the following properties:
  - Constants: $\rho = 7.85$ T/m^3
  - Elastic parameters: $E_c = 21.2$ GPa, $\nu = 0.2$
  - Ductile damage parameters: $\psi = 31°$, $e = 0.1$, $f_{bo}/f_{co} = 1.16$, $K_c = 0.667$, and viscosity $= 1 \times 10^{-5}$
  - Hardening behaviour in compression plasticity: $f_{cm} = 20$ MPa, $e_{cu} = 0.0035$
  - Tensile behaviour: $f_{ct} = 1.6$ MPa, $\varepsilon_{tu} = 0.001167$.

The properties have been obtained from compression tests of cylindrical samples for concrete and tensile specimens in the case of steel (Fig. 7), and from the existing literature [10], [11].

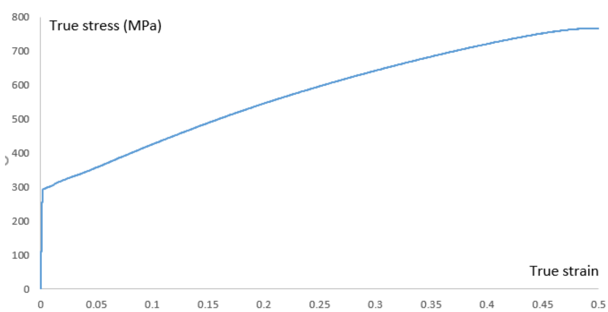

Figure 7: A36 true stress–stain test on 8 mm thick tensile specimens.

2.3 Manufacturing process

The manufacturing process of the tested beam–column connection was carried out at the Housing Research Centre of the National Polytechnic School in Quito (Fig. 8).

All welding processes followed the AWS (American Welding Society) standards for welding of structural steel AWS-D1.1 [12] and additional seismic requirements by the AWS-D1.8 [13].

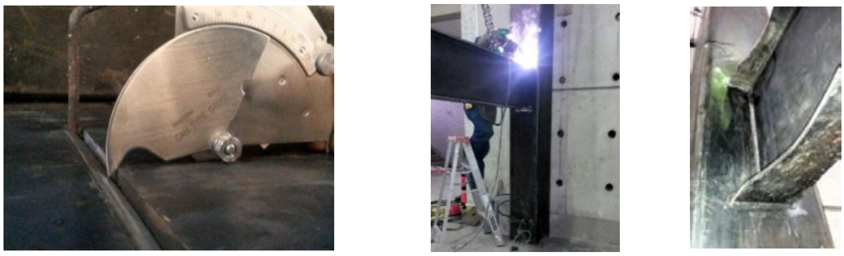

Figure 8: Welding and assembly work.

2.4 Experimental test

The test set-up and the instrumentation used are shown in Fig. 9.

The test was controlled in displacement by means of an LDVT extensometer located at the top of the beam. In addition, there was a load cell in the hydraulic actuator, and 4 extensometers for strain measurement in the central area of the reduced section of the beam flanges and at the connection of the beam flanges with the column.

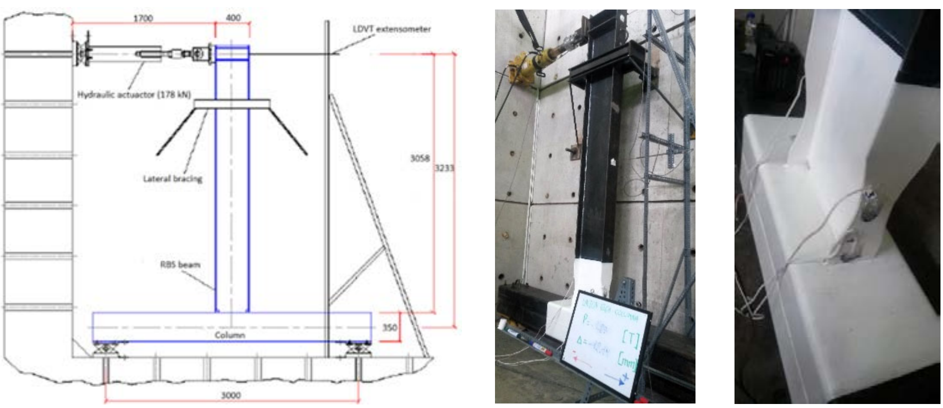

Figure 9: Test scheme, final assembly and extensometers on the beam flanges.

3 RESULTS

In the experimental test during load cycle 6, when displacements exceeded 50 mm, a brittle fracture occurred in the fillet weld connecting the continuity plate to the face of the column opposite the beam flange connection (Fig. 10).

The weld of the continuity plate under tension breaks on the face of the column opposed to the beam and then breaks on the face of the column next to the beam. When the load is reversed, the two welds of the other continuity plate break in a similar way. No local buckling of the flanges or web of the beam appear (Fig. 11).

The numerical model is not able to reproduce the shear fracture of the fillet welds of the continuity plates with the column observed in the experimental test. According to the numerical results, the connection withstands maximum loads close to 80 kN (Fig. 10), with complete plastic behaviour of the continuity plates and the beam area with reduced cross-section, and local end buckling of the flanges and web of the beam (Fig. 12).

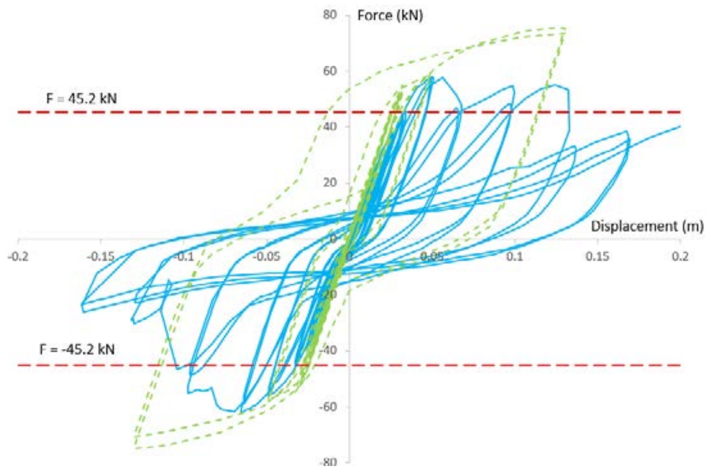

Figure 10: Hysteresis cycles: experimental test (blue lines) and in the numerical model (green dashed lines).

Figure 11: Brittle fracture of continuity plate welds in the experimental test.

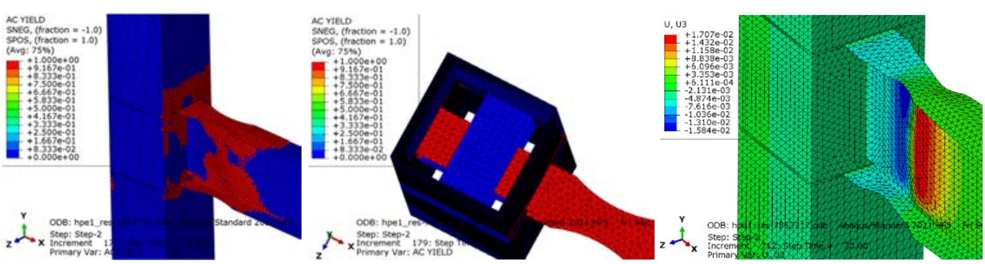

Figure 12: FE of steel in a plastic state in the last cycles of the numerical model (centre and left), and final transversal displacements u_z (m, right).

The numerical model is not able to reproduce the brittle fracture of the continuity plate welds as it works with shell elements in the steel and does not model the welds. As can be seen in the Fig. 12, the continuity plates enter into plastic zone like the beam flanges in the reduced section and concentrate the stresses in the connections with the 8 mm thick column plates (Fig. 13), causing shear failure.

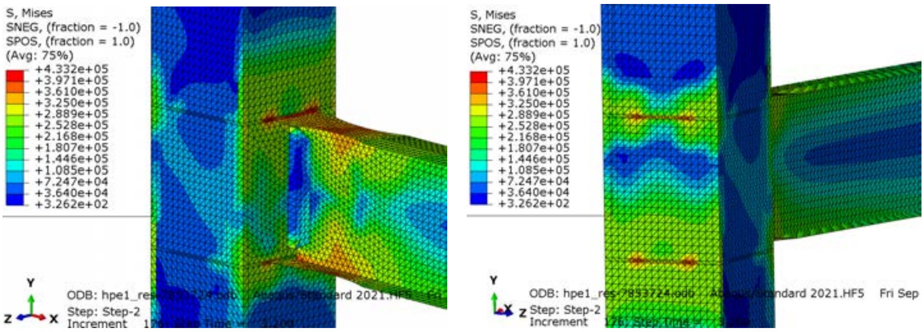

Figure 13: Equivalent von Mises stresses (kN/m^2) in the last cycle of the numerical model.

To reproduce this breaking mechanism, it is necessary to use solid elements in the steel plates and model this area with solid elements in an explicit analysis (Fig. 14). The problem with this model is that it increases the calculation times making them excessive.

Analytically, if the shear strength of the connection between the continuity plates and the column is calculated, a maximum load of 47 kN is obtained, whereas loads of about 55 kN are introduced in the experimental test.

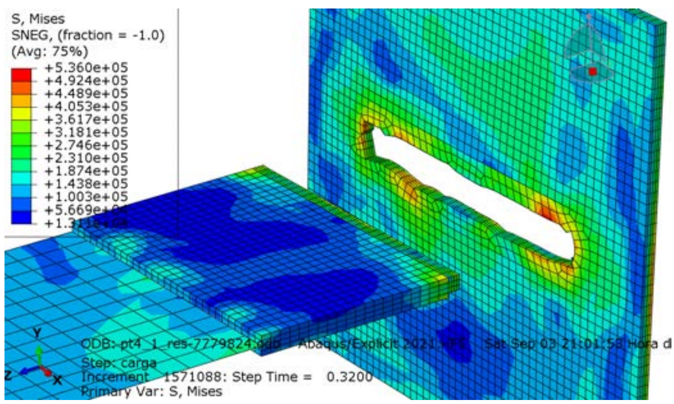

Figure 14: Shear fracture in a model with 3D solid elements.

4 PROPOSED DESIGN MODIFICATIONS

To improve the ductility of the beam–column connection, configurations with a longer weld length in the continuity plates as shown in Fig. 15 are proposed, which at the same time facilitate the assembly and the concrete filling of the columns.

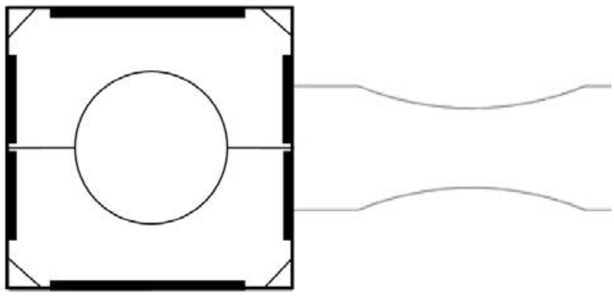

Figure 15: Proposed continuity plate configuration.

5 CONCLUSIONS

In this work, the seismic ductility of a beam–column connection with reduced-section beams and composite columns consisting of two longitudinally welded U-profiles filled with concrete has been evaluated, and it has been experimentally verified that the proposed connection does not have the necessary seismic ductility according to the requirements of AISC 358 and FEMA 350 standards.

The developed numerical model does not allow for the simulation of shear fracture of fillet welds, but it has a reasonable fit with the experimental tests until brittle fracture appears.

Based on the proposed modifications to the continuity plates, where shear failure should not appear as the welding lengths increases, the numerical model is being used to verify the new designs that improve the seismic response of the connections and subsequently validate them with experimental tests.

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REFERENCES

[1] NEC-SE-DS (Norma Ecuatoriana de la Construcción, Cargas Sísmicas y Diseño Sismorresistente), Ministerio de Desarrollo Urbano y Vivienda, 2014. https://www.habitatyvivienda.gob.ec/documentos-normativos-nec-norma-ecuatoriana-de-la-construccion/

[2] American Institute of Steel Construction, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-22. American Institute of Steel Construction, 2022. https://www.aisc.org/globalassets/aisc/publications/standards/a358-22w.pdf.

[3] SAC Joint Venture, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350. SAC Joint Venture, June 2000. https://nehrpsearch.nist.gov/static/files/FEMA/PB2007111285.pdf.

[4] Swati, A.K. & Gaurang, V., Study of steel moment connection with and without reduced beam section. *Case Studies in Structural Engineering*, **1**(1), pp. 26–31, 2014.

[5] Oshaki, M., Tagawa, H. & Pan, P., Shape optimization of reduced beam section under cyclic loads. *Journal of Construction Steel Research*, **65**, pp. 1511–1519, 2009.

[6] Pillajo, H., Romera, L. & Gutiérrez, R., Experimental and numerical seismic evaluation of connection systems between reduced-section beams and steel-concrete composite columns. *Third Conference on Structural Dynamics (DinEst2024)*, Seville, Spain, 2024.
[7] Computers and Structures Inc., ETABS v.19.1 Manuals, CSI, Building Analysis and Design, 2020.
[8] Abaqus/Cae 2022 manuals, Dassault Systèmes, Simulia, 2022.
[9] American Welding Society, Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341, American Welding Society, ANSI, 2022.
[10] Alfarah, B., López-Almansa, F. & Oller, S., New methodology for calculating damage variables evolution in plastic damage model for RC structures. *Engineering Structures*, **132**(1), pp. 70–86, 2017.
[11] Hartloper, A.R., Sousa, A.C. & Lignos, D.G., Constitutive modeling of structural steels: Nonlinear isotropic/kinematic hardening material model and its calibration. *Journal of Structural Engineering*, **147**(4), 2021.
[12] American Welding Society, ANSI, Structural Welding Code-Steel, AWS-D1.1, American Welding Society, ANSI, 2020. https://www.aisc.org/globalassets/aisc/publications/standards/a341-22w.pdf.
[13] American Welding Society, ANSI, Structural Welding Code-Seismic Supplement, AWS-D1.8/D1.8M, American Welding Society, ANSI, 2016.