# COMPORTAMENTO A COLLASSO PROGRESSIVO DI COLLEGAMENTI TRAVE-COLONNA IN ACCIAIO IN SCENARI DI RIMOZIONE DELLA COLONNA

# PROGRESSIVE COLLAPSE BEHAVIOUR OF STEEL BEAM-TO-COLUMN CONNECTIONS UNDER A COLUMN REMOVAL SCENARIO

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## ABSTRACT

The paper investigates the progressive collapse performance of steel beam-to-column connections. While undergoing large deformation, the beam-to-column connections are subjected to moment, shear, and tension in conjunction with high ductility demand. Their behavior under monotonic loading depends on the moment-axial tension interaction and greatly affects the progressive collapse resistance of the structure. A review of experimental tests of different types of beam-column joints (flexible, rigid, and semi-rigid) under a central-column-removal scenario is presented in the study. In all simple connections considered herein, the plastic rotation capacity obtained by tests was found much higher than the codified values. On the contrary, for some rigid or semi-rigid connections, the values of plastic rotation capacity suggested by acceptance criteria prove to be unconservative.

## SOMMARIO

La memoria indaga le prestazioni a collasso progressivo di collegamenti trave-colonna in acciaio. In caso di grandi deformazioni, le connessioni trave e colonna sono soggette a momento, taglio e trazione unitamente ad un'elevata richiesta di duttilità. Il loro comportamento sotto carico monotono dipende dall'interazione momento-trazione e condiziona notevolmente la resistenza al collasso progressivo della struttura. Nello studio viene presentata una revisione di test sperimentali su diversi tipi di giunti trave-colonna in acciaio (flessibili, rigidi e semirigidi) in uno scenario di rimozione della colonna centrale. In tutti i collegamenti a cerniera qui considerati, la capacità rotazionale ottenuta dai

test è risultata molto superiore ai valori codificati. Al contrario, per alcuni giunti rigidi o semirigidi i valori di capacità rotazionale suggeriti dai criteri di accettazione si rivelano non conservativi.

## 1 INTRODUCTION

Over the last decade, a great effort has been made to investigate the progressive collapse resistance of structures [1-4]. Due to hazards not explicitly considered in the design – namely impacts, explosions, etc. – the structure can be at risk of initial local damage that can result in a spread of failure to surrounding members and, eventually, lead to partial or even global collapse of the structure. This event is known as progressive collapse. In place of calculations demonstrating the effects of abnormal loads on buildings, the existing guidelines make use of the alternate path approach to determine the susceptibility to progressive collapse. This approach assumes that one critical member (typically vertical load-carrying columns or bearing walls) is removed because incapable of supporting the load and the remaining structure must be able to span across the removed member. After a column is destroyed by abnormal loads, different mechanisms are mobilized to transfer the gravity loads to superstructures and avoid collapse: Vierendeel action, catenary action, columns acting as suspension, compressive arching in beams, membrane action in slabs, etc. The so-called "catenary action" refers to the ability of beams to resist vertical loads by developing a string-like mechanism under large displacements and rotations [5-6]. After column removal, a new equilibrium is found by redistributing the internal forces in the remaining structure. Initially, the beams adjacent to the removed column tend to resist the vertical load by the generation of a bending moment. However, with increasing vertical displacement (i.e., in the large deformation field) the gravity loads are mainly resisted by the vertical components of the axial forces that develop in the beams (i.e., catenary forces). This "catenary action" generates a supplemental resistance that prevents the progressive collapse of the structure. Practically, the load-carrying mechanism changes from plastic hinge action to tensile catenary action. However, its activation depends on the geometrical nonlinear behavior of the structure. At first, the structure should be ductile enough to allow large inelastic deformations and make possible the transition from the flexural resistance to the tensile catenary resistance. Then, the constraints of the beam end provided by the side columns should give a sufficient lateral anchorage to these catenary actions. Ultimately, the beam-to-column connections have to be able to transfer the increased axial force in the beams due to the catenary action.

In recent years many experimental studies have been developed to study both the dual effect of axial tension in beam-to-column connections and its impact on the progressive collapse resistance including the catenary effect. Nevertheless, most existing analytical models for beam assemblies under a progressive collapse scenario essentially assume that the plastic strength remains available despite very large deformation. Consequently, the strength degradation due to local failure is neglected and, thus, the effect of catenary action on the resistance can be largely overestimated. Such studies have shown that different beam-to-column joints have different performances under a column removal scenario, as far as both the progressive collapse resistance and the catenary mechanism. Therefore, it seems necessary to develop a database of full-scale test results on double-span assemblies. This paper, which summarizes the results coming out from [7], tries to overcome this gap by presenting a review of experimental tests of different types of beam-to-column connections under bending and axial tension due to column removal. To this aim, fully rigid, semi-rigid, and flexible connections are investigated. Their load-displacement response both in flexural and catenary action stages is described and commented below. Both resistance and rotational capacities are investigated, as well as their effect on the catenary action. The failure modes under sudden column loss as determined experimentally are discussed in detail. The test results are finally used to evaluate the rotation capacity of several steel subassemblies. The values obtained are finally compared with the acceptance criteria specified by the main progressive collapse guidelines for several beam-to-column connection categories.

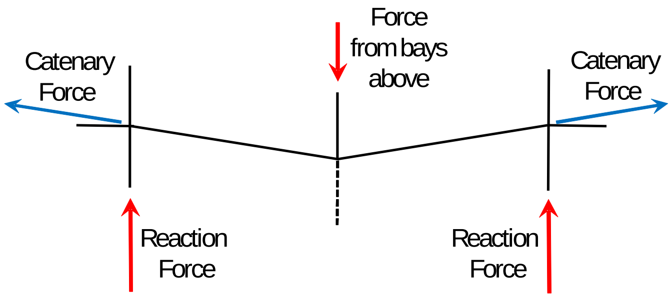


**Fig. 1.** Scheme of double-span assembly for the test of joints

## 2 BEHAVIOR OF FRAME CONNECTIONS

Beam–to–column connections play a critical role in the progressive collapse resistance of steel frames since the rotation capacity of joints usually controls the activation of the catenary action. The corresponding acceptance criteria were defined using both test results and numerical simulations available in the literature. In the 2005 version of UFC 4-023-03 [8], the rotational capacity values for connections were based on the 2003 version of GSA [9] and reasonably agreed with those given in ASCE 41 [10]. However, the ASCE 41 acceptance criteria were based on experimental tests of beam-to-column connections under cyclic loads, where the rotational capacity is limited by degradation and loss of strength due to low cycle fatigue. On the contrary, in the experimental tests under a central-column-removal scenario, the beam-to-column connections are subjected to monotonic loads and significant axial tension forces. Therefore, significant performance differences were found, due to the effects of loading (monotonic vs. cyclic) and the ultimate state (moment-axial tension interaction versus moment only). As a consequence, the modeling and acceptance criteria in the 2016 version of UFC and GSA were defined based on a comparison between the deformation limits contained in various documents and guidelines, such as ASCE 41, Eurocode 3 [11], UFC 4-023-03[8] and GSA Test Program [9].
As stated, it seemed necessary to develop an extended database of test results [7], since different beam-to-column connections show significant behavioral differences, especially in the catenary mechanism. After column removal, a “double-span” scenario arises, which is reproduced in the tests by suitable assemblies comprising two adjacent beams connected by a failing column at the center (Fig. 1). In this situation, the moment connections are the weakest components, as they can have either a low rotational capacity (case of simple joints) or reduced axial strength (case of rigid and semi-rigid joints). In both cases, the development of an effective catenary action is impaired. This is the reason why many types of beam-to-column connections have been experimentally investigated, including simple, rigid, and semi-rigid connections.
The following types of connections have been considered:

- Simple (flexible) connections: bolted-angle connections; web cleat, top and seat angle, top and seat with web angle and fin plate connections; web cleat connections; double web angle connections; shear connections; bolted-angle connections under tension.
- Rigid connections: Beam-to-tubular column moment connections; welded unreinforced flange, bolted web and reduced beam section connections; welded unreinforced flange-bolted web connections; welded flange-weld web connections, welded flange-bolted web connection and welded flange-bolted web connection with shear diaphragm; welded unreinforced flange-welded web connections; welded cover plate flange connection and haunch end plate bolted connection; welded unreinforced flange-bolted web connections; bolted flange plate connections; SidePlate moment connections.

- Semi-rigid connections: welded unreinforced flange-fillet welded web connections and bolted unstiffened extended end-plate with pre-tensioned high-strength bolts connections; complete joint penetration and reduced-web-section welded connections; flush end-plate, extended end-plate and top and seat-web angle connections; flush end-plate connections; top and seat-web angle connections; reduced beam section welded connection and unstiffened extended end plate bolted connection; flush end-plate connections.

In the next section, the performance of different beam-to-column connections against progressive collapse is examined based on full-scale test results on double-span assemblies available in the technical literature. A more detailed description of the testing activities is given in [7].

## 3 PLASTIC ROTATION CAPACITY AND ACCEPTANCE CRITERIA

The key parameter for assessing the joint performance, both in seismic and progressive collapse assessment, is the plastic rotation angle $\theta_p$, defined as the inelastic (permanent) rotation that occurs after the yield rotation is reached and the entire cross-section has yielded. Contrary to the seismic context, in which the cyclic behavior of connection is of concern, under a column removal scenario the nonlinear acceptance criteria of connections should be based on monotonic loading tests with flexural action as well as catenary action. As mentioned, the interaction of bending and axial load may significantly limit the rotational capacity of the connections. Many beam-column joints are unable to develop significant axial tension loads upon reaching the ultimate moment capacity of the beam. On the other side, the recent progressive collapse research has demonstrated that rotational capacities for monotonic loading are most of the times higher than for cyclic loading. Therefore, the main progressive collapse guidelines [8-11] provide specific modeling parameters and acceptance criteria due to the performance differences related to the loading conditions, between bending moment only and moment-axial tension interaction, and between monotonic and cyclic loads (Table 1). However, it seems necessary to enlarge the database of test results, since different beam-to-column connections have different behaviors, especially in the catenary mechanism. Moreover, it is useful to check if the acceptance criteria are conservative for progressive collapse design. For this reason, the findings of experimental tests were used [7] to evaluate the capacities of different types of steel beam-to-column connections and compare them with the acceptance criteria of the progressive collapse guidelines [8-11]. To this aim, the experimental force-vertical displacement curves of the tested specimen were used. The vertical deflection of the column is produced by both the rigid-body rotation of the joint and the connection rotation. The connection rotation is measured through the chord rotation, using the overall deflection profile of the beams to define the point of contraflexure (i.e., the point at which the bending moment is zero). This allows to plot the vertical load against the chord rotation. This curve is then idealized using a bilinear model, which defines the yield rotation, the plastic rotation, and the total rotation. Its value is finally compared with the allowable plastic rotation angle specified by the acceptance criteria for that connection. The value of the plastic rotation is finally compared with the allowable plastic rotation angle provided for that connection by GSA [9] (Table 1), estimated based on the depth of beam ($d$) and the depth of the connection ($d_{bg}$).

Some of the results obtained from the study carried out in [7] are shown in Figs 2-4. For flexible connections, the maximum rotation capacity varies from 0.108 rad to 0.208 rad. The test values are well beyond the recommended acceptance criteria by GSA [9] given in Table 1. This highlights that the tested connections exhibited a very large plastic rotation without a significant reduction in strength (Fig. 2). Moreover, it should be observed that in Table 1 the plastic rotation capacity of partially restrained simple connections (flexible) depend only on the connection depth. However, the test results show that different joint configurations with similar values of the connection depth result in very different values of the maximum rotation capacity. Thus, the connection depth alone does not seem to be an effective parameter to predict the rotational capacity of beam-to-column connections. In any case, the obtained results are always on the safe side with respect to codified values.

**Table 1**. Parameters and Acceptance Criteria for Nonlinear Modeling of Steel Frame Connections [9]

| Connection type | Nonlinear Modeling Parameters | | | Nonlinear Acceptance Criteria | |
|---|---|---|---|---|---|
| | Plastic rotation angles (rad) | | Res. Strength ratio | Plastic rotation angles (rad) | |
| | a | b | c | Primary | Secondary |
| Partially Restrained Simple Connections (Flexible) | | | | | |
| Double Angles | | | | | |
| a. Shear in Bolt | $0.0502 - 0.0015d_{bg}$ | $0.072 - 0.0022d_{bg}$ | 0.2 | $0.0502 - 0.0015d_{bg}$ | $0.0503 - 0.0011d_{bg}$ |
| b. Tension in Bolt | $0.0502 - 0.0015d_{bg}$ | $0.072 - 0.0022d_{bg}$ | 0.2 | $0.0502 - 0.0015d_{bg}$ | $0.0503 - 0.0011d_{bg}$ |
| c. Flex. in Angles | $0.1125 - 0.0027d_{bg}$ | $0.150 - 0.0036d_{bg}$ | 0.4 | $0.1125 - 0.0027d_{bg}$ | $0.150 - 0.0036d_{bg}$ |
| Simple Shear Tab | $0.0502 - 0.0015d_{bg}$ | $0.1125 - 0.0027d_{bg}$ | 0.2 | $0.0502 - 0.0015d_{bg}$ | $0.1125 - 0.0027d_{bg}$ |
| Fully Restrained Moment Connections | | | | | |
| Improved WUF with Bolted Web | $0.021 - 0.0003d$ | $0.050 - 0.0006d$ | 0.2 | $0.021 - 0.0003d$ | $0.050 - 0.0006d$ |
| Reduced Beam Section | $0.050 - 0.0003d$ | $0.070 - 0.0003d$ | 0.2 | $0.050 - 0.0003d$ | $0.070 - 0.0003d$ |
| WUF | $0.0284 - 0.0004d$ | $0.043 - 0.0006d$ | 0.2 | $0.0284 - 0.0004d$ | $0.043 - 0.0006d$ |
| SidePlate® | $0.089 - 0.0005d$ | $0.169 - 0.0001d$ | 0.6 | $0.089 - 0.0005d$ | $0.169 - 0.0001d$ |
| Partially Restrained Moment Connections (Relatively Stiff) | | | | | |
| Double Split Tee | | | | | |
| a. Shear in Bolt | 0.036 | 0.048 | 0.2 | 0.03 | 0.04 |
| b. Tension in Bolt | 0.016 | 0.024 | 0.8 | 0.013 | 0.02 |
| c. Tension in Tee | 0.012 | 0.018 | 0.8 | 0.01 | 0.015 |
| d. Flexure in Tee | 0.042 | 0.084 | 0.2 | 0.035 | 0.07 |

Legenda: $d$ = depth of beam, inch; $d_{bg}$ = depth of bolt group, inch
WUF: Welded Unreinforced Flange-Welded Web joint.

For rigid connections (Fig. 3), the test values of rotation capacity range between 0.02206 rad and 0.193 rad. For Welded Unreinforced Flange-Welded Web specimens, the test value (0.0678 rad) is lower than GSA value (0.0852 rad). Eventually, for the semi-rigid connections (Fig. 4), the maximum rotation capacity varies from 0.011 rad for the SJHT (hogging moment and tensile force) specimen and 0.226 rad for the Top and Seat-Web Angle (TSDWA L = 10 mm) specimen. Two values are lower than the GSA recommended acceptance criterion: the plastic rotation $\theta_p$ = 0.0678 rad (GSA value = 0.0852 rad) for the Welded Unreinforced Flange-Fillet Welded (WUF-FW) specimen and $\theta_p$ = 0.011 rad (GSA value=0.013 rad) for the Flush End-Plate (SJHT hogging moment and tensile force) specimen. The results show that although the stiffness of rigid and semi-rigid connections is higher than flexible connections, both categories result in similar rotation capacity. Generally, the suggested acceptance criteria are far beyond the allowable connection rotation according to the GSA standard. Therefore, the values prescribed by this standard are probably too conservative. Similar conclusions have been drawn from other studies. However, it should be highlighted that for some beam-to-column connection specimens (i.e., WUF, WUF-FW, and SJHT), both rigid and semi-rigid, the plastic rotation angles obtained from the experimental tests are lower than the allowable value, meaning that in some cases the GSA acceptance criteria are not on the conservative side.

## CONCLUSIONS

Experimental tests under a column removal scenario show that different beam-to-column connections behave differently as to the progressive collapse resistance and the catenary mechanism. In a critical review of experimental tests of different types of beam-to-column connections (flexible, rigid, and semi-rigid) under a central-column-removal scenario, results, including load-displacement relationships, failure modes, and catenary effects, show that in simple (flexible) joints, the stiffness and strength at higher drift angles essentially depend on the tensile capacity of the connection. In all simple connections considered herein, the plastic rotation capacity obtained by tests was found much higher than the values recommended by GSA. This means that these code values are probably too conservative. In fully rigid and semi-rigid connections the flexural resistance controls the behavior at the preliminary phase, and the tensile force is almost zero.

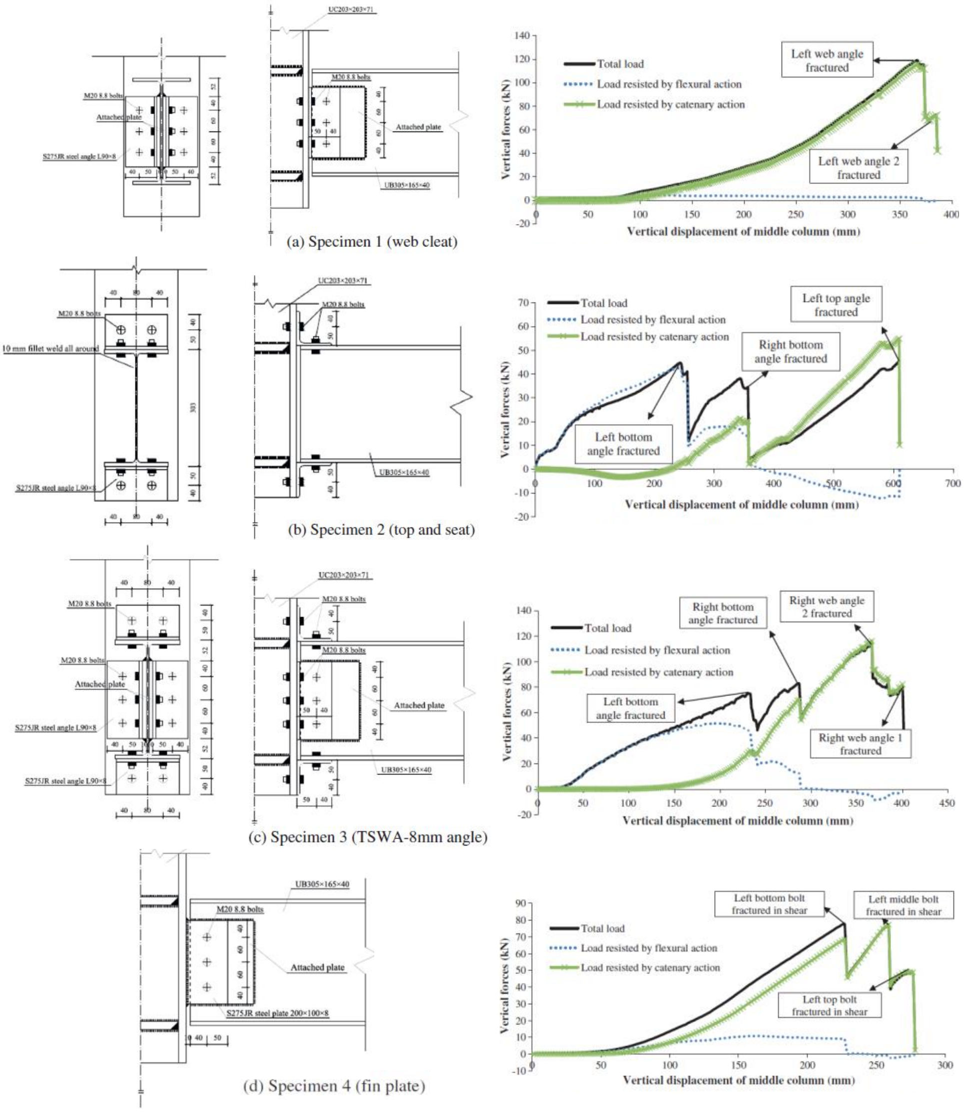


**Fig. 2.** Types of flexible joints configurations and corresponding load-displacement curves [12]

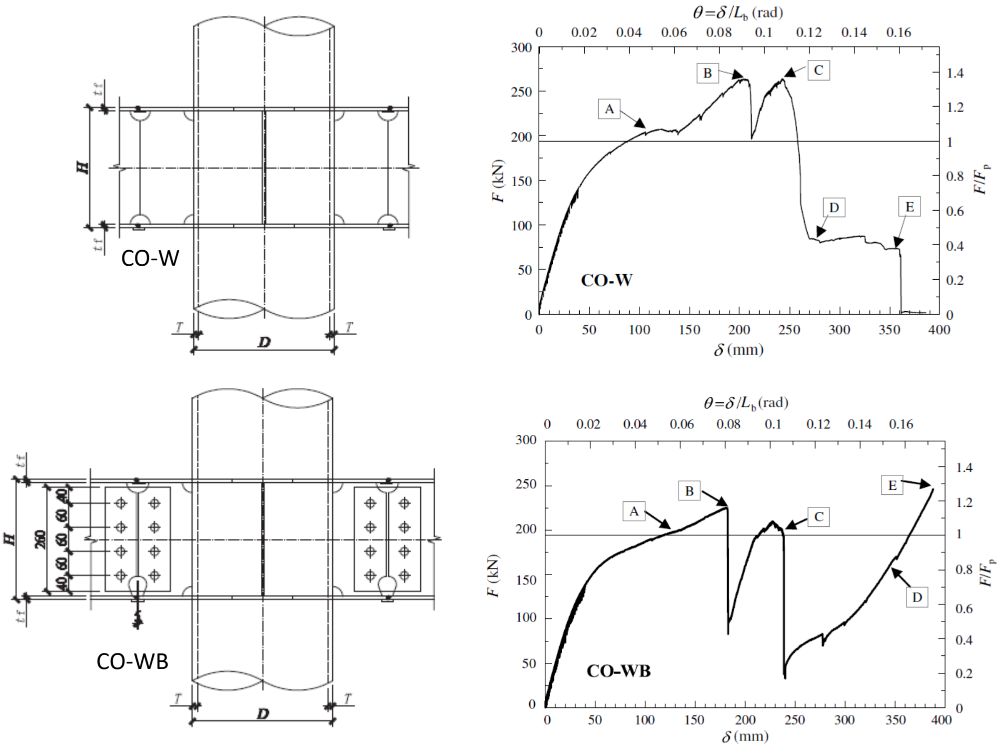


**Fig. 3.** Types of rigid joints and corresponding load-displacement curves [13]

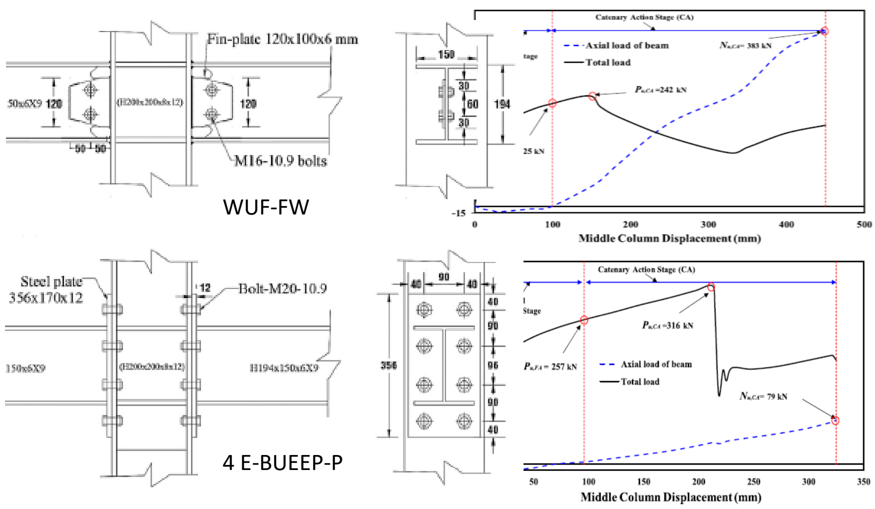


**Fig. 4.** Types of semi-rigid joints and corresponding load-displacement curves [14]

Experimental results show that although the stiffness of rigid and semi-rigid connections is higher than flexible connections, both categories result in similar rotation capacity. However, for a few rigid and semi-rigid joints the values of the plastic rotation capacity obtained by tests is lower than the corresponding recommended values, which turns to be out of the conservative side. In these cases, the results of experimental tests available in the literature would suggest the opportunity to pay special attention to both joint geometry and details which can affect the rotational ductility in presence of large tension loads, so as to prevent the onset of local brittle failures.

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## KEYWORDS

Progressive collapse, joints, tension strength, catenary effect, double-span test, acceptance criteria.