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THESIS WORK
IN
THEORY AND DESIGN OF STEEL CONSTRUCTIONS

STRUCTURAL RESPONSE OF STEEL BEAM-TO-COLUMN JOINTS EQUIPPED WITH FRICTION DAMPERS

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1. Acknowledgements

The concept of resilience has always fascinated me, as I believe it is one of the ingredients of a happy life. Given the very low probability, almost zero in reality, of never encountering problems during the entire span of our existence, our ace in the hole is precisely that of sustaining the punches well and learn how to fall without getting hurt too much and to stand up faster possible.

Leaving aside now the philosophical side of this topic, which could deceive the reader about the type of text he is about to read, I have to sincerely say that it was really interesting for me to apply this concept to structures, dealing with a wide spectrum of interconnected aspects and to see all these theoretical hypotheses to take life by means of a FE simulation.

This my master thesis was written during and after my ERASMUS Traineeship in Liège, a wonderful challenge made possible by my dear two Professors Landolfo and D'Aniello, who I thank also for the kindness, the availability, the encouragement. My thanks go also to Eng. Tartaglia, for his time, his tips and his masterclass in FE modelling, things that helped me a lot.

My stay in Liège couldn't have been better, and my gratitude goes to the Professors Demonceau and Jaspart, aside with the entire MS2F department: they welcomed me and they made me feel at home and truly accepted, nonetheless my bad French!

A big thank you to my family, that was close to me along this my study journey as always have been.

Thank in the end to my friends, the Italians, the Belgians, the Europeans, the non-Europeans, for the great moments spent together, the decompressing time and for the think-out-of-the-box moments.

Thank you all

2. Abstract

2.1. Background

In recent years, under the thrust of *Low Damage Structures* philosophy, several projects involving friction connections have been carried out. From this perspective, one of the most promising typologies of joint is represented by the *Sliding Hinge Joint* (SHJ), where a sliding mechanism is provided at the beam ends. SHJs are usually equipped with a *Symmetric Friction Connection* (SFC), made by a set of surfaces (in form of shims, pads) with controlled friction coefficient, available both in vertical or horizontal configuration. Joints like this are designed to be stiff under serviceability loads while admitting a relative displacement at ultimate limit state. In this manner, energy that comes from seismic events or exceptional load conditions can be dissipated thanks to the sliding of the friction surfaces, using slotted holes.

A good example of SHJ with SFC is represented by the **FREEDAM** (acronym for “*__FREE__ from __DAM__age”*) joints, made by a modified Double Split Tee Joint, with a bottom haunch which hosts friction dampers. Experimental tests have proven that this type of connection is able to withstand severe seismic event without any damage to the structural members, resulting a very profitable cost-saving design solution.

The **DREAMERS** (acronym for *“__D__esign __RE__search, implementation __A__nd __M__onitoring of __E__merging technologies for a new generation of __R__esilient __S__teel buildings”*) demonstration project has the goal to prove the applicability and the enhanced performance against various destructive scenarios of the FREEDAM connections, in a real scale environment. Under this project a three-storey building will be built in Fisciano, in the University of Salerno Campus. Seismic resilient moment-resisting steel frames will be put in place along the two main directions of the building and provided with FREEDAM connections.

2.2. Aim and Objectives

The aim of this thesis is that to study the behaviour of FREEDAM joints when used in a real scale environment, such as the DREAMERS building, in order to prove the feasibility and the effectiveness of this type of structural solution, helping its spread in the Constructions Sector.

The main objectives of this thesis are: (i) to highlight the possible weaknesses in case of *Column Loss Scenario* of a FREEDAM MRF connection and (ii) to re-design such connection in order to guarantee a proper level of robustness.

2.3. Methods

Several Finite Element analyses of beam-to-column joint assemblies have been carried out by using Abaqus 2017 commercial software. The following Case Scenarios have been investigated: (i)

Monotonic loading, under hogging and sagging bending moment; (ii) Cyclic loading, by using the AISC 341 loading protocol; (iii) Column loss scenario, both in case of the notional removal of the column' joint assembly and in case of the notional removal of a column nearby the joint assembly.

Materials behaviour is modelled by using true stress-true strain curves taken from experimental tensile tests. Particular attention is used in imposing the adequate boundary conditions on each assembly, since then they are strictly dependent from the Case Scenario.

2.4. Results

FE analyses have shown that FREEDAM connections exhibit an excellent seismic behaviour both in monotonic and cyclic loading as it was expected, while on the other hand several weaknesses have been registered. In particular, under Column Loss Scenario unwanted brittle failures have been observed in vital zones of the connection:

- The bolts connecting the upper T-stub to the column flange, show a Failure Mode 2 (close to Failure Mode 3) with respect the equivalent T-stub method under hogging bending moment;
- The dampers bolts easily come in contact with the slotted holes walls of the haunch, resulting in an unforeseen necking type failure and in an excessive deformation of the haunch itself.

In order to reach the desired performance levels under Column Loss Scenario, the upper T-stub has been modified properly and this proposal has been presented in this thesis.

2.5. Conclusions

All MRF joints of the DREAMERS building are equipped with FREEDAM connections. FE analyses show that these joints exhibit an excellent behaviour in a Seismic Scenario, both under monotonic and cyclic loading. However, their behaviour in a Column Loss Scenario is characterized by brittle failure of the connection at medium rotations (around 15% of chord rotation). In this thesis an alternative design for the upper T-stub of the connection is proposed, with the aim to guarantee a proper level of robustness, namely to reach higher rotations under Column Loss Scenario.

Further studies should be directed towards including the Column Loss Scenario in the FREEDAM joints design procedure, enhancing in this way all the five different FREEDAM devices.

3. Brief Introduction on Robustness

New design criteria were created in Europe, particularly in Great Britain, after the Ronan Point Building collapse in London in 1968 to prevent the progressive collapse of structures caused by local damages. The terrorist attack against the Alfred P. Murrah Building (Oklahoma City, 1995) have once again brought attention to how inadequate standard design principles are in the event of accidental actions. These two events, along with several international research initiatives influenced the design standards approach to a new philosophy that takes structural robustness into account as a new safety criteria.

Fig 1: Ronan Point, 1968 (left), Alfred P. Murrah Building, 1995, (right)

The term "robustness" refers to a structure's response to accidental actions. The so-called progressive collapse, a kind of chain reaction that could lead the partial or complete collapse of the building, can be triggered by localized failures brought on by accidental actions if the structure is not strong enough. A structure is considered "robust" when accidental actions don't result in losses that are disproportionate to the cause. Therefore, the goal of a resilient structures is to limit the amount of failure, which should remain limited in particular regions to prevent spreading to the rest of the structure, rather than to prevent damages to structural parts. In other words, designing a system that permits no damages may result in a very high degree of complexity and, as a result, extremely expensive realization costs.

4. State of the Art

Constructions are nowadays designed for different identified actions to which they could be subjected. The intensity of these actions is characterised using semi-probabilistic methods or derived from the experience. However, structure collapses are still observed; they are generally induced by unforeseen actions or action intensities or combinations of actions. Indeed, these unforeseen events may induce local damages which may then spread within the structure and lead to a progressive and disproportionate collapse. For this reason, modern codes and standards are nowadays requesting for a proper structural **robustness**, defined in the Eurocodes (EN1991:1-7, § 1.5.14) as "*the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause*".

However, although widely recognized as fundamental and extremely important, robustness is only described in a general way in the present building codes with limited information on how to guarantee an appropriate robustness to the designed structures. It is the reason why, in the past decades, different research projects have been conducted on the topic in order to derive scientifically founded design recommendations allowing to ensure an appropriate robustness to structures. In particular, a valorisation project funded by the *Research Fund for Coal and Steel* (RFCS) named *FAILNOMORE* is under finalisation with the objective of proposing a practice-oriented "design for robustness" manual for steel and composite structures on the basis of the outcomes of recent research projects. Within the present section, it is intended to propose a "design for robustness" strategy for the *DREAMERS* structure by describing first the normative context and referring to the above-mentioned *FAILNOMORE* Design Manual (Demonceau et al., 2021) when required and then by proposing design approaches to be adopted for the *DREAMERS* structure.

The State of the Art related to the design for robustness will be briefly introduced with the objective of highlighting the key design recommendations. On an European and Italian framework, the following normative documents will be addressed:

European regulations:

- EN 1990 - (EN 1990, 2002)
- EN 1991 - (EN 1991-1-7, 2006)
- EN 1993 - (EN 1993-1-1, 2005) & (EN 1993-1-8, 2005)
- EN 1994 - (EN 1994-1-1, 2004)

Italian regulations:

- NTC 2018 and
- Circolare N°7 - (NTC, 2018)
- CNR DT214-2018 - (CNR, 2018)

4.1.1. EN 1990 – Basis of the structural design

EN 1990 provides principles and requirements for **safety**, **serviceability**, and **durability** of structures. This Eurocode applies for any structures designed according to the European norms and is intended to be used together with Eurocodes EN 1991 to EN 1998.

In Section 2.1 of this norm dedicated to the **Basic Requirements**, Clause (4) (reminded here below) can be seen as a request for structural robustness (even if not explicitly expressed). Then, Clause (5) (also reminded here below) is proposing different design strategies which can be contemplated. As it can be observed, these clauses are just providing the designer with general recommendations and refer to EN 1991-1-7 for more details. Accordingly, the next section is dedicated to EN 1991-1-7.

> *(4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as:*
> - *explosion*
> - *impact*
> - *consequences of human errors*
>
> *to an extent disproportionate to the original cause.*
> *NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.*
> *NOTE 2 Further information is given in EN 1991-1-7*

> *(5)P Potential damage shall be avoided or limited by appropriate choice of one or more of the following:*
> - *avoiding, eliminating or reducing the hazards to which the structure can be subjected;*
> - *selecting a structural form which has low sensitivity to the hazards considered;*
> - *selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;*
> - *avoiding as far as possible structural systems that can collapse without warning;*
> - *tying the structural members together.*

4.1.2. EN 1991:1-7 - Action on structures / General actions: accidental actions

EN 1991:1-7 provides strategies and rules for safeguarding buildings and other civil engineering works against identifiable and unidentifiable accidental action. For an identifiable action, the recommended design procedure involves i) strategies aiming at preventing or reducing the action, ii) strategies based on an explicit design of the structure to sustain the action and iii) strategies aiming at providing the structure with a sufficient minimum robustness. In EN 1991-1-7, only explosions and impact accidental actions are covered. For unidentifiable actions, the proposed design strategies aim at limiting the extend of a localised failure, whatever the initiating event is. It can be achieved by enhancing the redundancy of the structure, by following prescriptive rules, or by designing key elements to sustain a notional accidental action (a key element being defined as a structural element that is fundamental to guarantee the integrity of the entire structure. However, in the core of the text, there is limited information on how to apply the proposed design strategies in practice.

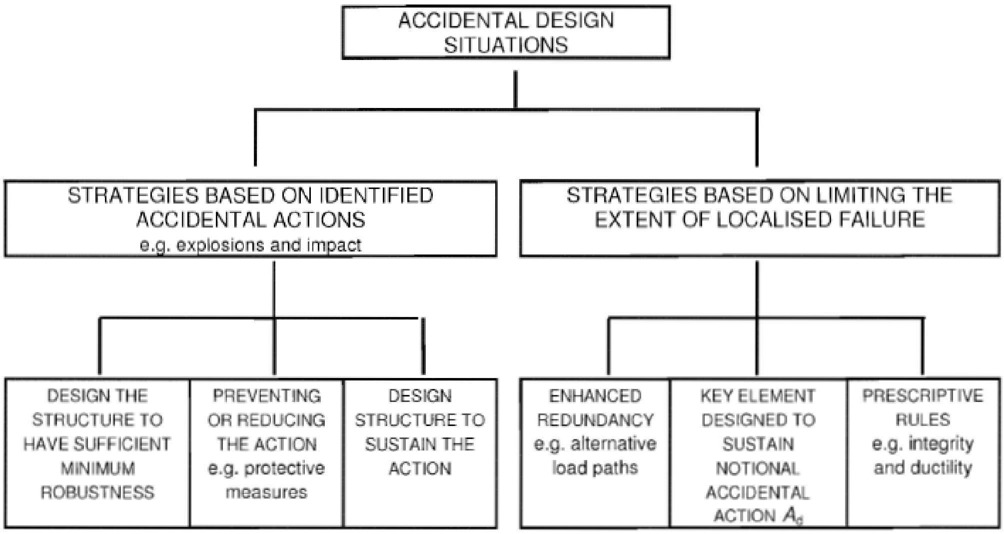


Fig 2: Strategies for accidental design situations recommended in EN1991-1-7

The informative Annex A of EN1991-1-7 entitled *Design for consequences of localised failure in buildings from an unspecified cause* proposes rules and methods for designing buildings to limit the extent of localised failure.

The selection of the method to be applied is related to the consequences class (CC) of the considered structure, the consequences classes being defined in Table B1 of EN1990 (**Errore. L 'origine riferimento non è stata trovata.**)

Tab 1: Definition of the consequences classes according to EN 1990

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

In Annex A of EN 1991-1-7, Table A.1 (**Errore. L'origine riferimento non è stata trovata.**) is p roviding additional information to classify specific structural systems. As it can be observed, the consequences class 2 as defined in EN 1990 is subdivided in two sub-groups: CC2a named lower risk group and CC2b named upper risk group.

Tab 2: Categorisation of the consequences classes according to EN 1991-1-7

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of $1^1/_2$ times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators Buildings containing hazardous substances and /or processes

According to EN 1991-1-7, no specific consideration is necessary for accidental actions for CC1 structures if they are designed respecting the rules given in EN 1990 to EN 1999 as applicable. For CC2 structures, a simplified analysis using static equivalent action models or prescriptive design/detailing rules may be adopted. For CC3 structures, a risk analysis may be required and the use of refined methods such as dynamic analyses, non-linear models and interaction between the load and the structure may be contemplated.

In the framework of the design strategies for identified accidental action, equivalent static loads for different types of impacts are proposed in Section 4 of EN 1991-1-7. For what concerns explosions, EN 1991-1-7 is only dealing with internal explosions proposing general recommendations in Section 5. More refined design strategies for impact and explosion are provided in Annex C and D of EN 1991-1-7 respectively.

In the framework of the design strategies for unidentified accidental actions, Section A.4 of EN 1991-1-7 lists some design approaches to be contemplated to limit the extent of a local damage. Provided a building has been designed in accordance with the rules given in EN 1990 to EN 1999, the recommended design approaches are as follows (**Errore. L'origine riferimento non è stata t rovata.**):

Tab 3: Recommended design approaches according to the consequences class

Consequences class	Recommended design approaches
CC1	• no further consideration is necessary with regards to accidental actions from unidentified causes
CC2a	• effective horizontal ties or effective anchorage of suspended floors to wall should be guaranteed using the prescriptive tying method
CC2b	• effective horizontal ties for framed and load-bearing wall construction and effective vertical ties in all supporting columns and walls should be guaranteed using the prescriptive tying method or; • check that, upon the notional removal of each supporting column and each beam supporting a column, the building remains stable and that any local damage does not exceed a certain limit (defined in EN 1991-1-7) – this approach is called the alternative load path method
CC3	• A systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards.

While the prescriptive tying method is detailed in Section A.5 and A.6 of EN 1991-1-7, limited information is provided on how to apply the alternative load path method. For what concerns the risk analysis, Annex B of EN 1991-1-7 provides guidance for the planning and execution of risk assessment in the field of buildings and civil engineering structures.

4.1.3. EN 1993 – Design of steel structures

EN 1993:1-1 – General rules and rules for buildings

EN 1993:1-1 provides general design rules for steel structures and specific guidance for structural steelwork used in buildings.

Although Section 2.1.3 of this normative document is entitled “Design working life, durability and robustness”, there is no specific guidance for the design for robustness of steel structures. It is only stated that “*steel structures shall be designed for accidental actions*” and further referring to EN 1991-1-7.

In EN 1991-1-7, the ductility of structural elements is identified as a key property (in particular at the level of the beam extremities) while no specific provisions are given in this regard. However, in EN 1993-1-1, interesting information may be found regarding the ductility of steel beams through the concept of cross-section classification reminded here below:

- Class 1: cross-sections can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance;
- Class 2: cross-sections can develop their plastic moment resistance, but have limited rotation capacity because of local buckling;
- Class 3: cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance;
- Class 4: cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Even if no specific recommendations are provided in EN 1993-1-1 regarding the cross-section class to be used in the framework of a design for robustness, it appears that the use of Class 1 cross-sections should be used at the extremities of the beams if full strength joints are used as reflected in the FAILNOMORE Design Manual (Demonceau et al., 2021). If partial-strength or pinned joints are used at the extremities of the beams, the ductility should come from the joint itself and reference should be made to EN 1993-1-8. This normative document is addressed in the next section.

EN 1993-1-8 – Design of joints

EN 1993-1-8 gives design recommendations for moment resisting steel joints where the use of the component method is proposed for the joint characterisation. This method is an analytical model that allows for the calculation of the main mechanical properties of the joints: (i) stiffness, (ii) resistance, and (iii) rotation capacity of the joints. Its concept makes use of the static and the kinematic theorem. Details for the implementation procedure and supplementary information to EN 1993-1-8 are available in (Jaspart and Weynand, 2016).

The rules as proposed in EN 1993-1-8 allow for the characterisation of the main steel joint configurations. The characterisation of other joint configurations may be possible referring to

(Jaspart et al., 2005) where a review of the literature dedicated to the characterisation of joint components not (yet) covered by EN 1993-1-8 is provided.

The calculation of the stiffness and resistance design properties of joints is basically possible, whatever the loading is (moment M only, axial force N only and combination of moment *M* and axial force *N*, in addition to shear forces). In EN 1993-1-8, however, precise application rules are only provided for joints subjected to bending moments and shear forces. Only a rough approach is proposed in which, first, the influence of the *M*-*N* interaction is disregarded as long as the axial force N_{Ed} applied to the joint is smaller than 5% of the axial design plastic resistance of the connected beam cross-section ($N_{pl,Rd}$):

$$\left|\frac{N_{Ed}}{N_{pl,Rd}}\right| \leq 0,05$$

If this criterion is not respected, EN 1993-1-8 recommend considering a linear interaction between *M* and *N*, the lines linking the hogging or sagging bending resistances, computed assuming that no axial loads are applied, to the axial resistances in tension or compression, computed assuming that no bending moments are applied to the joint. Both criteria result in a polygonal interaction diagram as illustrated below.

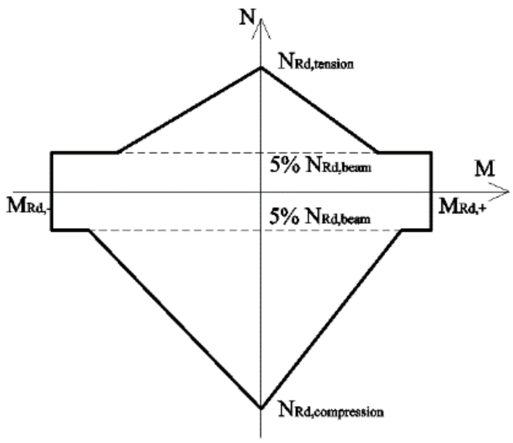


Fig 3: M-N domain interaction proposed in Eurocode 1993:1-8

In (Demonceau, Cerfontaine, and Jaspart, 2019), it has been shown that the Eurocode *M*-*N* interaction curve predicts sometimes rather precisely but often very safely the joint resistance, while the 5% rule leads generally to a significant overestimation of the joint resistance. Besides that, EN 1993-1-8 is not defining the way on how to evaluate the values of $N_{Rd,tension}$ and $N_{Rd,compression}$ reported in **Errore. L'origine riferimento non è stata trovata.**. In the same p ublication, an improved design analytical assembly procedure is also presented for steel and steel-concrete composite joints. It has been validated through comparisons to results obtained from experimental tests performed on composite beam-to-column joints in various loading situations, including fire and progressive collapse.

For what concerns the ductility, design criteria are provided in EN 1993-1-8 (see Section 6.4.2) for bolted joint configurations to ensure that joints will have a sufficient ductility to perform plastic analyses with formation of plastic hinges at the level of the joints:

$$t \leq 0{,}36d\sqrt{f_{ub}/f_y}$$

where f_{ub} is the ultimate strength of the bolt material and f_y the elastic strength of the material of the component in bending. This condition should at least by satisfied by one of the two connected plates. In the framework of a design for robustness, it is recommended to respect this criterion as reflected in the FAILNOMORE Design Manual (Demonceau et al., 2021).

The design of simple (hinged) joints is not covered by EN 1993-1-8. Ductility requirements for such joints are provided in (Jaspart et al., 2009), as well as procedures for the evaluation of the design shear resistance (in the form of design sheets allowing an easy application in practice); these procedures will be implemented in the future draft of EN 1993-1-8 (prEN 1993-1-8, 2020).

In this publication, it is also recommended to respect the criterion linking the thickness of the connected plates to the diameter of the bolts given above. In addition, requirements are expressed in terms of full-strength welds.

In the framework of a design for robustness, it is recommended to use full penetration welds or full-strength welds (respecting the requirements provided in (Jaspart et al., 2009) following the guidelines provided in the FAILNOMORE Design Manual (Demonceau et al., 2021).

4.1.4. EN 1994-1-1 – Design of composite steel and concrete structures

EN 1994-1-1 provides general design rules for composite steel-concrete structures. This normative document includes rules to characterise structural composite members but also rules to characterise composite joints. For the latter, EN 1994-1-1 is also recommending the use of the component method for the joint characterisation referring to EN 1993-1-8 for the steel components and providing rules for components specific to composite joints or which are affected by the presence of the concrete. However, the rules as presently provided in EN 1994-1-1 allow for the characterisation of composite joints under hogging moments only. For the characterisation of composite joints under sagging moments, reference can be made to (Demonceau, 2008) where rules in full agreement with the component method are proposed.

In EN 1994-1-1, the design for robustness of composite structures is not specifically addressed. However, the recommendations stated in Section 4.1.3 are also valid for composite structures, i.e.:

- use of Class 1 cross-sections if full strength joints are used;

- in case of partial-strength or simple joints, to ensure the rotation capacity/ductility of the joints using full penetration or full-strength welds and by respecting the criterion linking the thickness of the connected plates to the diameter of the bolts given in Section 0.

Additional design recommendations specific to composite structures are also provided in the FAILNOMORE Design Manual (Demonceau et al., 2021).

4.1.5. NTC2018 and Circolare N°7

For the design of buildings, Italy issued two main codes:

- NTC 2018 entitled *"Aggiornamento delle «Norme Tecniche per le costruzioni»"*, released on 17.01.2018 by Ministerial Decree, is a regulatory document which collects the rules governing the design, execution, and testing of buildings in order to guarantee, for established safety levels, public safety. It takes in consideration all the different structural typologies such as reinforced concrete, prestressed concrete, steel, masonry, and timber building structures;
- Circolare N°7 entitled *"Istruzioni per l'applicazione dell' «Aggiornamento delle "Norme tecniche per le costruzioni" » di cui al decreto ministeriale 17 gennaio 2018"*, released on 21.01.2019 by Ministerial Decree, clarifies the previously mentioned code NTC 2018 and provides additional instructions.

According to the Italian law, each building structure should comply with the recommendations from these two codes.

For what concerns robustness, NTC 2018, in its Section 2.1 defines robustness as the "*ability to avoid disproportionate damages which may be induced by exceptional events, as explosions and impacts*" and recalls in its Section 2.2.5 the robustness principles expressed in EN 1990 and reminded in Section 4.1.1 of the present document.

NTC 2018 also proposes a definition for **exceptional actions,** defined as "*actions that show up only exceptionally during the nominal life of the structures (such as fires, explosions, impacts)*" and recommends the following "exceptional load combination" to be used for exceptional ultimate limit states, i.e., involving exceptional actions:

$$G_1 + G_2 + P + A_d + \psi_{21} Q_{k1} + \psi_{22} Q_{k2} + \cdots$$

where:

- G_1 : permanent structural loads
- G_2 : permanent non-structural loads
- P : preload
- A_d : accidental/exceptional actions
- Q_{ki} : variable load i
- ψ_{2i} : combination coefficient for rare event, for variable load i, depending on the intended use for the construction as defined in Table 2.5.1 of NTC 2018

In addition, NTC 2018 requires to guarantee a proper level of robustness in constructions, based on the intended use of the building, by identifying risk scenarios and the exceptional actions relevant for the design. The following sections of NTC 2018 define the partial safety factors to be used into the verification checks and clearly state that robustness must be ensured:

- Section 4.1.4 for concrete structures;
- Section 4.2.6 for steel structures;
- Section 4.3.8 for composite concrete-steel structures;
- Sections 4.4.12 and 4.4.17 for timber structures;
- Section 4.5.10 for masonry structures.

As a conclusion, it can be stated that this design standard is providing limited information regarding the design for robustness; but, it suggests searching for other "documents of proven validity" such as the ones published by Italian CNR (Consiglio Nazionale delle Ricerche) to deal with this issue.

Further indications and clarifications can be found in the Circolare N°7. In particular, Section C2.2.5 of this document states that: "*In general, the design of buildings conducted according to the prescriptions contained in the NTC 2018, taking into account the criteria of design for seismic actions, guarantees the achievement of robustness levels that can be considered, in general, satisfactory. For buildings of particular importance or structural complexity or, where deemed necessary, also in relation to the specificities of the project, the level of robustness can be increased through the adoption of motivated strategies among those listed in § 2.2.5 (which are similar to the ones recommended in EN 1990) which can be combined*."

4.1.6. CNR DT214-2018

The National Research Council (Consiglio Nazionale delle Ricerche - CNR) is a public organization involved in different fields of research. The document CNR DT214-2018 entitled "*Instructions for the constructions robustness evaluation*" is organised as follows:

1. Introduction
2. Risk scenario and magnitude quantification of the relating action
3. Risk of disproportionate collapse
4. Risk reduction strategies
5. Conceptual design for robustness
6. Design for robustness
    6.1. Analysis methods
    6.2. Reinforced cast-in-situ concrete constructions
    6.3. Reinforced precast concrete constructions
    6.4. Steel constructions
    6.5. Timber constructions

7. Probabilistic and semi-probabilistic quantification of robustness
8. Examples and case studies
9. Annex A: a simplified method for the evaluation of the membrane contribution to ultimate bearing capacity of rectangular reinforced concrete slabs

Within this section, general recommendations coming from this document are first presented. Then, the specific ones for steel structures are reported.

General recommendations

Section 2.1 of CNR DT214-2018 provides general definitions. Several terms are covered, including: hazard, vulnerability, exposition, risk, exceptional action, disproportionate collapse, progressive collapse, robustness, ductility, structural redundancy, alternative load path, segmentation, and key element.

Chapter 2 splits the hazards that can lead to a failure in three categories:

- **category 1**: hazards associated to natural phenomena or involuntary human activities;
- **category 2**: hazards associated to voluntary human activities;
- **category 3**: hazards due to errors and mistakes in the conception/design/execution phases of the construction.

The document suggests different strategies according to the selected category:

1. for category 1, it is appropriate to:
   - 1.1. define a statistical model describing the frequency of occurrence of a given phenomenon
   - 1.2. define a model describing far field effects
   - 1.3. define a model fixing the intensity of the action and how the phenomenon interacts with the construction
   - 1.4. define a model describing possible mitigation interventions of the hazard

   with the objective of obtaining a statistical approach to manage the encountered risks through a robust design.

2. for category 2, a statistical approach is not applicable. For vandalism/terroristic acts, the following aspects should be taken into account:
   - 2.1. strategic role of the construction, based also on the hosted activity;
   - 2.2. potential relevance of an attack: possibility to cause a high number of casualties;
   - 2.3. typology of the building: hospital, public office, government building, building with symbolic relevance, etc.
3. for category 3, a statistical study is also not applicable. To avoid or minimize such hazards, a quality control process is strongly recommended, both in the design and construction phase.

A list of possible exceptional events is provided in CNR DT214-2018: seismic action (earthquake, tsunami), landslides, falling rocks, snow avalanche, volcanic eruptions (ash fall), foundation settlements, floods, meteorologic phenomena such as tornado, storms, snowfalls, fire, blasts, impacts (vehicles, aircrafts, ships, vandalism and terrorist acts and finally, conception/design/execution errors. For most of these events, recommendations are provided referring to existing guidelines in literature. Guidelines on how to conduct a risk assessment are also provided in Chapter 3 of the CNR document with examples of application.

In Chapter 4 of CNR DT214-2018, different measures to be contemplated are proposed:

3.1. Prevent the hazard (by reducing its probability of occurrence);
3.2. Prevent the local damages that could lead to a disproportionate collapse;
3.3. Prevent the spread of a local damages a such a spread could lead to a disproportionate collapse of extended parts of the construction.

These approaches are similar to the ones proposed in EN 1990 and EN 1991-1-7 and accordingly, the design strategies proposed in the CNR document are also similar to the ones prescribed by the Eurocodes. However, the CNR document provides the reader with more detailed information, instructions, and examples.

In Chapter 5, key structural properties/recommendations when considering the design for robustness are provided – they are briefly listed here below:

- **Redundancy**: only hyperstatic structures can offer alternative load paths;
- **Ties**: they allow to ensure links between structural elements and so to ensure a certain redundancy;
- **Ductility**: members have to be able to exhibit large deformations to allow the development of large displacements and rotations within the structure to find a new state of equilibrium in the deformed shape;
- **Uniform distribution of structural elements**: It allows for a load redistribution in case of failure of a single element;
- **Adequate shear stress resistance**: shear strength should always be greater than flexural strength, in order to exploit ductility;
- **Capacity to support sign inversion of actions and forces**: element-loss scenario often involves situations which are totally different from the usual distribution of internal forces.

The importance of **segmentation** is also highlighted in this chapter to limit the extent of a local damage by defining strong boundaries which stop the failure of weak elements.

Steel constructions, beams requirements

Section 6.4.3.1 of CNR DT214-2018 provides some guidelines intended to prevent local instability in the beams. It is strongly recommended to use class 1 cross-section for beams, what is in line with the recommendations provided in the FAILNOMORE Design Manual (see Section 0). Moreover, the column loss scenario can result in a sagging bending moment in the beam, with the lower beam flange in compression. For this reason, the lower beam flange must be properly restrained against lateral instability; spacing and strength of torsional restraints can be evaluated according to EN 1993-1-1.

Steel constructions, beam-to-column and beam-to-beam connections

In case of bolted end-plate connections, the welds between the end of the beam and the end-plates must be full penetration ones, performed in the workshop and properly checked – this recommendation is stronger than the one previously presented in Section 0 as, in the latter, full-strength welds are also allowed. The filler material of the welds must be of greater strength and resilience than the materials of the connected elements to avoid brittle failure of the welds.

In order to limit the request in terms of deformation capacity at the level of the joints, full-strength joints can be used. In the CNR document, rules are provided to guarantee the full-strength character of joints, including the possible overstrength and strain hardening effects. The proposed method is similar to the one provided for the design of full-strength joints in seismic regions. In addition, in order to guarantee a full-strength joint towards the catenary action, tensile resistance of bolt rows in the tensile part of the connection $\left(\sum F_{t,Rd}\right)$, namely bolt rows placed in the half height of the connection (in case of symmetric connection), must be designed to resist the whole axial plastic strength of the connected beam $\left(N_{pl,Rd,beam}\right)$, so that $\sum F_{t,Rd}/N_{pl,Rd,beam} \geq 1$. This simplified prescription allows to design full-strength joints in case of column loss (Tartaglia et al., 2018). In case where the ratio $\sum F_{t,Rd}/N_{pl,Rd,beam}$ is less than 1, the connection will develop plastic deformations and, therefore, it will be necessary to adopt additional prescriptions to ensure the ductility of the joint.

Additional recommendations are also provided to guarantee the ductility of partial-strength joints, in particular for bolted joints for which criteria linking the thickness of the connected plates to the diameter of the bolts are provided.

Steel Constructions, Column-to-column connections

Column-to-column connections must be placed at a height equal to half the inter-storey height and in any case 1200 mm above the beam-to-column joint. To guarantee the possibility of activating an alternative load path following the loss of a column at the floor below, these connections must be designed to resist to such combined actions:

- Tensile force N_{Ed} , equal in value to the vertical axial load, in the ultimate design combination;
- Shear forces $V_{c,Ed}$, computed alternatively along the two principal directions of the column cross-section, written as:

$$V_{c,Ed} = \frac{2M_{cpl,Rd}(N_{Ed})}{h}$$

where:

- $M_{cpl,Rd}(N_{Ed})$ is the design flexural plastic resistance of the column in the considered direction, under the axial load N_{Ed};
- h is the inter-storey height, calculated as the distance between the beam intrados quota of the upper floor and the beam extrados quota of the lower floor.

These recommendations are stronger than the ones proposed in the Eurocodes.

5. FAILNOMORE project

FAILNOMORE project was funded by the Research Fund for Coal and Steel (RFCS) of the European Commission, with the aim to produce a set of practical and easy-to-use guidelines for the mitigation of the risk of progressive collapse of steel structures and of composite steel-concrete structures subject to exceptional actions. These guidelines are based on scientific existing literature and new research projects.

In this document a summary of the existing prescriptions for robustness is reported, aside to further considerations and detailed computations of worked examples.

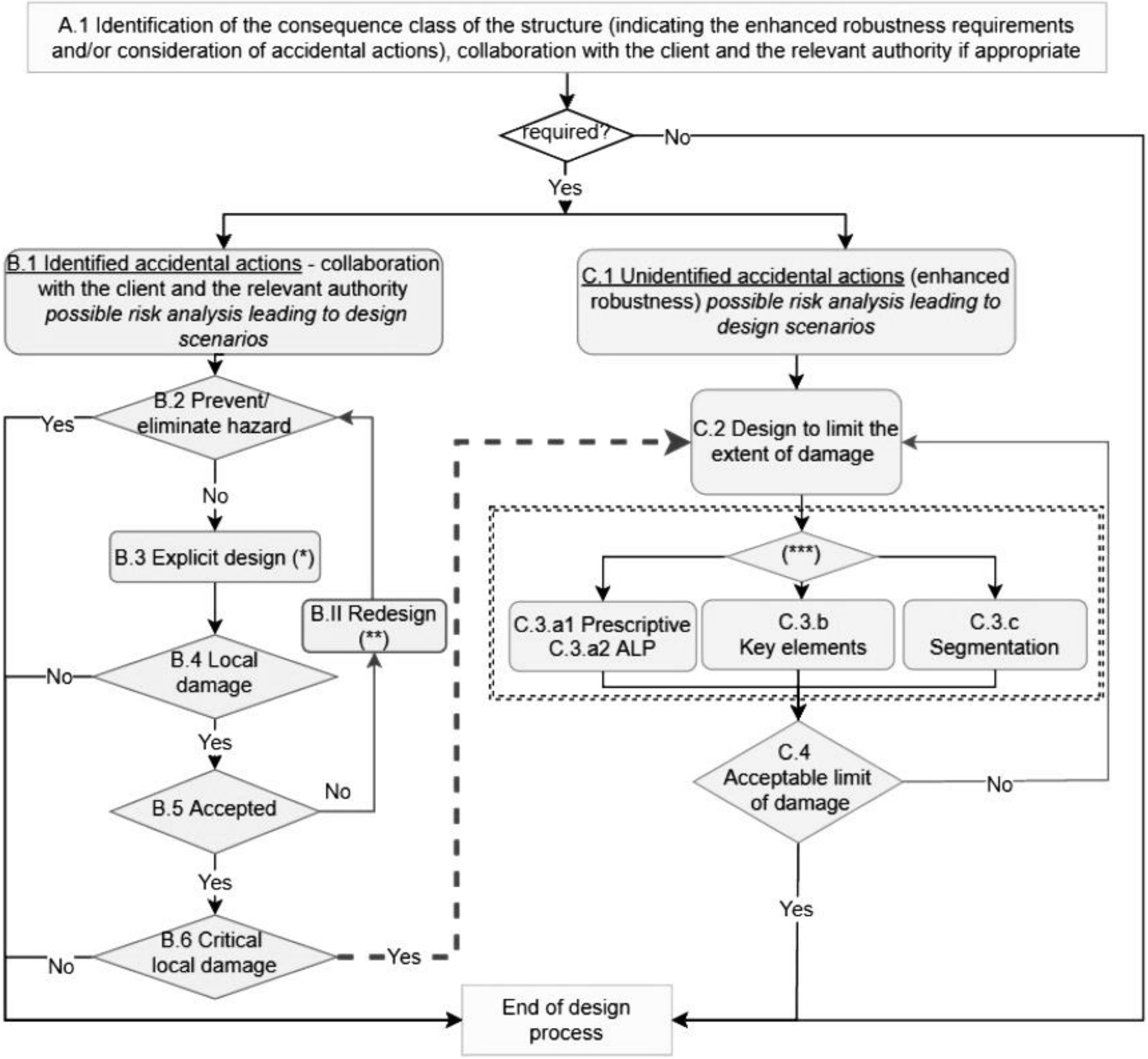
The FAILNOMORE project partners are:

- University of Liège (ULG) – Belgium
- University of Coimbra (UC) – Portugal
- Imperial College London (IC) – UK
- University of Stuttgart (USTUTT) – Germany
- University of Trento (UNITN) – Italy
- Politehnica University Timisoara (UPT) – Romania
- Czech Technical University of Prague (CVUT) – Czech Republic
- Rzeszow University of Technology (PRZ) – Poland
- Technical University of Delft (TUD) – The Netherlands
- Universitat Politècnicade Catalunya (UPC) – Spain
- INSA de Rennes (INSAR) – France
- European Convention for Constructional Steelwork (ECCS) – Europe
- Feldmann+ Weynand GmbH (F+W) – Germany
- ArcelorMittal Belval& Dufferdange S.A. (AM) – Luxembourg

One of the main outcome of this project is the development of a flow-chart for the robustness design. A strong distinction is made based on the magnitude of the accidental action, if this magnitude can be determined, the accidental action is denoted as “identified accidental action”, otherwise the accidental action is denoted as “unidentified accidental action”. This design process is composed of three blocks:

- Block A – identification of the Consequences Class of the building: this identification depends by the use of the building and it takes into account the loss in terms of human lives, economic, social, environmental losses.
- Block B – Design against identified accidental action: these procedures are directed to prevent or eliminate the hazard, often with active or passive countermeasures.

- Block C – Design against unidentified accidental action: these procedures are direct to limit the extent of the damage, by using three main strategies, which are i) the Alternate Load Paths method, ii) the Key Element method, iii) the Segmentation method.



* Appropriate design approaches for higher and lower consequences classes can be required

**When redesign/retrofit, more advanced methods may be used where appropriate

***Strategies for designing for robustness are not mutually exclusive and may be used singly or in combination

Fig 4: FAILNOMORE flow-chart for robustness design

6. FREEDAM and FREEDAM-PLUS projects

6.1. Projects overview

The European Union Research Fund for Coal and Steel co-funded the research project **FREEDAM**, which stands for "FREE from DAMage steel connections," in response to the RFCS Call 2015. The FREEDAM project aimed to develop a new design strategy whose goal is to design connections that can withstand rotation demands caused by destructive seismic events without damage. These novel beam-to-column connections include friction dampers located at the bottom flange level of the connected beam to dissipate earthquake input energy. The friction resistance is calibrated by varying the number and diameter of the bolts, as well as the tightening torque that governs the preloading. Flexural resistance is calculated as the product of damper friction resistance and lever arm. The connections are designed to have large and stable hysteresis loops while causing no damage to the connection steel plate elements. As a result, the work's basic concept was inspired by the strategy of supplementary energy dissipation, but it is based on the use of damping devices from a different perspective. While passive control strategies have commonly been based on the integration of the primary structure's energy dissipation capacity using supplementary dissipation from damping devices, the FREEDAM design strategy is based on the use of friction dampers designed to replace the traditional dissipative zones of moment-resisting frames, i.e. the beam ends. The development of the FREEDAM connections must be considered, on the one hand, a first important goal because of the benefits resulting from the elimination of connection repair costs in the aftermath of a destructive seismic event, and, on the other hand, a step towards the ambitious goal of damage-free buildings, which will require, additionally, the identification of connection details, between the non-structural components and the primary structure, capable of preventing damage.

The FREEDAM project involved 6 partners, including 4 universities and 2 industrial partners:

- University of Salerno (UNISA) - Italy, as project coordinator, coordinated by Prof. V. Piluso;
- University of Liege (ULG) - Belgium, coordinated by Prof. Jean-Pierre Jaspart;
- University of Naples “Federico II” (UNINA) - Italy, coordinated by Prof. Raffaele Landolfo;
- University of Coimbra (UC) - Portugal, coordinated by Prof. Luis Simoes Da Silva;
- FIP Industriale S.p.A. - Italy, coordinated by Dr. Eng. Maria Gabriella Castellano;
- O Feliz Metalomecanica S.A. - Portugal, coordinated by Dr. Eng. José Manuel Silva.

The European dimension of the research consortium allowed the execution of a wide experimental campaign concerning the friction dampers, beam-to-column sub-assemblages and the seismic testing of a two-storey building. In particular, 120 experimental tests on friction dampers subjected to cyclic loading conditions (60 under low-velocity conditions and 60 under high-velocity conditions, including loading histories simulating real earthquakes) were carried out for investigating the tribological properties of different coating procedures for realizing the friction pads of the dampers. In addition, the friction dampers were also subjected to 6 tests under impact loading, as a preliminary study concerning structural robustness, and to 6 long-term tests aimed to investigate the bolt preloading losses occurring during the life-cycle of the structure. The cyclic behaviour of beam-to-column connections equipped with friction dampers was investigated by performing 8 experimental tests on external beam-to-column connections and 8 tests on internal beam-to-column connections. Moreover, 6 experimental tests on beam-to-column connections subjected to impact loading were carried out to get important information for exceptional loading conditions requiring structural robustness. Finally, 10 seismic simulations on a two-storey building were performed using the pseudo-dynamic testing method. In particular, 5 earthquake' simulations concerned the building equipped with traditional connections and 5 simulations regarded the same building but equipped with FREEDAM connections. The comparison between the obtained displacement time histories and the structural damage occurring in the case of the building specimens equipped with traditional connections pointed out the advantages of FREEDAM connections that really behaved according to the free from damage design goal.

All the experimental tests were accompanied by advanced numerical simulations. In particular, the blind predictions of the cyclic response of the FREEDAM connections, carried out by advanced FE models, resulted in a very good agreement with the successive experimental observations. All these research efforts are described in the book "Valorisation of knowledge for FREE from DAMage steel connections" which is one of the main outcomes of the FREEDAM-PLUS Project. The FREEDAM-PLUS project has been funded by the European Union Research Fund for Coal and Steel following the RFCS Call 2019.

The FREEDAM-PLUS project, "Valorisation of knowledge for FREE from DAMage steel connections", is a dissemination project involving the following partners:

- University of Salerno (UNISA), as coordinator – Italy;
- University of Naples "Federico II" (UNINA) – Italy;
- University of Liege (ULG) – Belgium;
- University of Coimbra (UC) – Portugal;
- Politechnic University of Timisoara (UPT) – Romania;
- European Convention for Constructional Steelwork (ECCS);
- Ozyegin University (OZU) – Turkey
- National Technical University of Athens (NTUA) – Greece;

- "Ceske Vysoke Uceni Technicke" of Prague (CVUT) – Czech Republic;
- "Institut National des Sciences Appliquees de Rennes (INSA) – France;
- Technical University of Delft (TUD) – Netherland;
- University of Ljubljana (UL) – Slovenia;
- "Universitet Po Architektura Stroitelstvo I Geodezija" of Sophia (UASG) – Bulgaria;
- "Universitat Politecnica de Catalunya" of Barcelona (UPC) – Spain;
- "Rheinisch-Westfaelische Technische Hochschule Aachen" (RWTH) – Germany.

6.2. FREEDAM connection design behaviour

FREEDAM joints consists by friction pads coated by friction material located in the middle of a vertical sliding plates connected by high strength bolts to the bottom flange of the beam, a typology of connection called *Sliding Hinge Joint* (SHJ), where the slip resistance is provided by a *Symmetric Friction Connection* (SFC). Two L-stubs accommodate the friction pads and connect them to a vertical sliding plate. Moreover, a T-stub is located at the top beam flange to fix the center of rotation. Both the L-stubs and the T-stub are designed fusing the component method to remain in the elastic range. In other words, they have not to yield, and the dissipative behaviour of the connection has to be assured by the slippage of the friction pads only. The rotation demands occurring in beam-to-column joints under destructive seismic events are transformed into displacement demands at the level of the friction damper. Consequently, the yielding of the connections is prevented. If the columns of the building are designed to remain in the elastic range, also the structure of the building remains free from damage. However, to obtain a free from damage building, it is also needed that cladding elements and partition walls remain undamaged in the case of destructive seismic events. To this scope, the use of certified cladding elements with properly conceived connecting systems is needed.

The FREEDAM research project has achieved a TRL 5 (Technology Readiness Level 5: *"Technology validated in relevant environment (industrially relevant environment in the case of key enabling technologies)"*, on a 1-to-9 scale), ending with the testing of a building mock-up under seismic loading conditions. Many real earthquake records have been applied to the mock-up, pointing out that, compared to other traditional solutions, such as RBS (*Reduced Beam Section*) connections, the use of the FREEDAM joints, on average, gives rise to a top sway displacement reduction of about 35% with no damage. Nevertheless, to date, the first application of this technology in a real scale environment is still missing due to the need to demonstrate the actual possibility to build a free from damage structure with real-time and budget constraints, considering all the issues related to architecture, sustainability, and interaction with the non-structural elements.

6.3. FREEDAM joints features

In order to overcome the drawbacks of the traditional design approaches, the FREEDAM design strategy allows, easily, to design rigid frames with fully rigid connections (as in the case of full-strength continuous frames) with a resistance very close to the nominal value of the beam resistance (as in the case of partial – or equal - strength design) and with high energy dissipation supply (as in the case of supplementary energy dissipation strategies) avoiding, in the same time, the structural damage.

The adoption of FREEDAM connections allows to dissipate the seismic input energy avoiding damage both in the structural members and in the fastening elements of the connecting system, thanks to the inclusion of friction dampers. Such connections are detailed to include at the level of the lower beam flange a friction device realized with steel plates and friction pads pre-stressed with high-strength bolts. In particular, the typical configuration of a FREEDAM beam-to-column joint consists in a modification of the classical detail of a Double Split Tee Joint (DST) where, the bottom tee element, is substituted with a friction damper.

FREEDAM joints can be designed according to the following equation:

$$M_{f,Ed} \leq M_{f,Rd} = \frac{\mu_{st} n_b n_s P_f h_f}{\gamma_{F2}}$$

where μ_{st} is the average value of the static friction coefficient equal to 0.76, n_b is the number of bolts, n_s is the number of the contact surfaces equal to 2, h_f is the lever arm given as the sum of height of the haunch and height of the beam, γ_{F2} is the partial safety factor accounting for the randomness of friction and bolt preload, and it is equal to 1.26, P_f is the preloading force that has to be calibrated to assure that the FREEDAM connection resistance is as much close as possible to the design moment $M_{f,Ed}$ at the column face resulting from the seismic load combination. Therefore:

$$P_f \cong \frac{M_{f,Ed} \gamma_{F2}}{\mu_{st} n_b n_s h_f}$$

The bolt preloading should be between 40% and 100% of the maximum bolt preloading allowed by code provisions (EN 1993-1-8).

The number of bolts changes according to the standardised devices. The friction damper to be adopted has to be selected in function of the beam height and of the increase of the lever arm due to the haunch resulting from the damper geometry.

Device	Number of damper bolts
D1	4
D2	4
D3	6
D4	8
D5	8

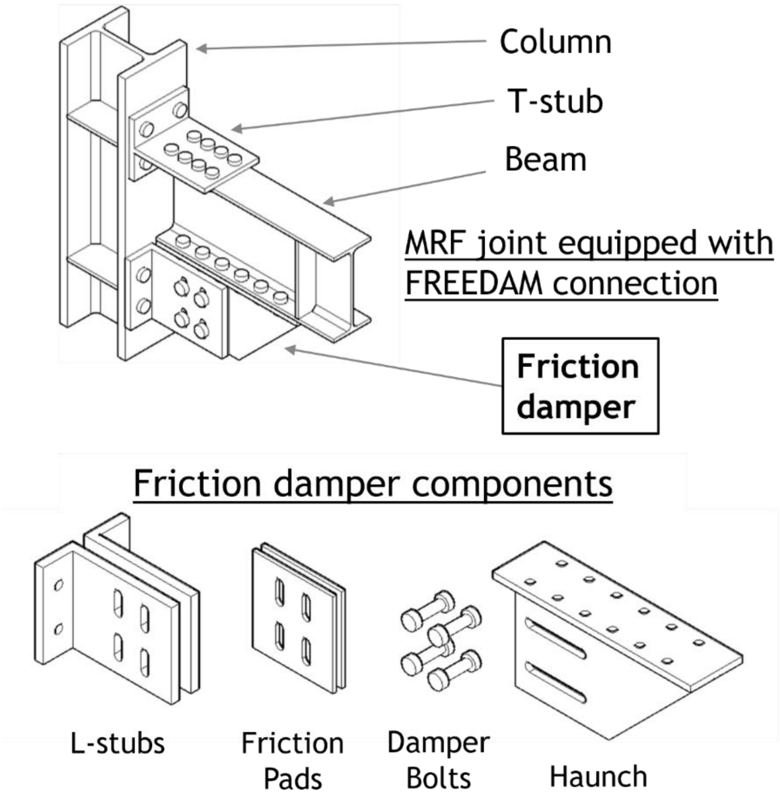


Fig 5: 3D view of a MRF joint with its components

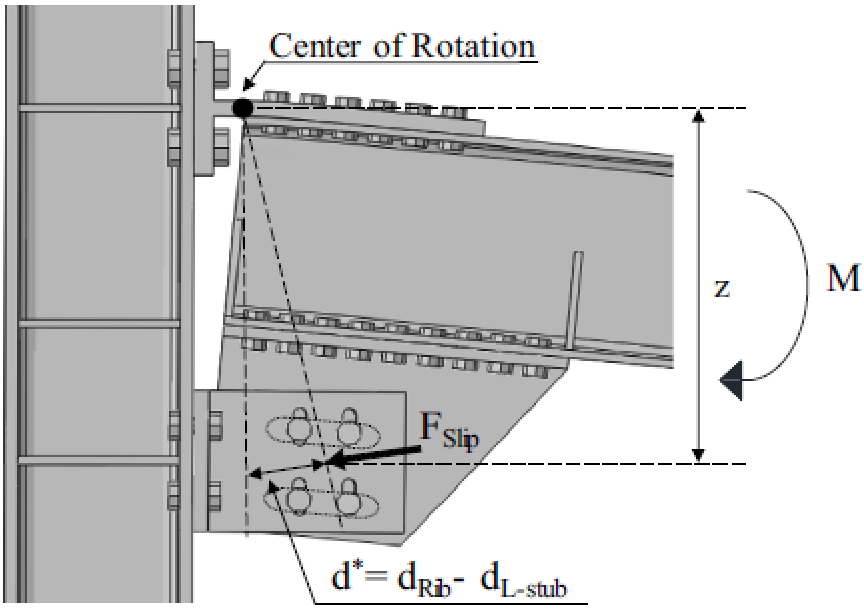


Fig 6: Side view of the FREEDAM joint in action

Another important aspect of the FREEDAM project is that Devices are designed to be equipped on a certain range of beam-column profiles; the Design Manual gives for a selected bending capacity level (ratio between the resistance of the FREEDAM connection and the plastic resistance of the connected beam) and for a given beam profile, the best Device to be used.

As rule of thumb, taller beam profiles requires Devices with more damper bolts and bigger elements.

Tab 4: Suggested Device to be equipped

Beam Size	m (Bending Capacity Level)			
	0.3	0.4	0.5	0.6
IPE 270			D1	D1
IPE 300		D1	D1	D1
IPE 360	D1	D1	D2	D2
IPE 400	D1	D2	D2	D2
IPE 450	D1	D2	D2	D3
IPE 500	D2	D2	D3	D3
IPE 550	D2	D3	D3	D4
IPE 600	D2	D3	D4	D4
IPE 750 x 147	D3	D4	D5	D5
IPE 750 x 161	D3	D4	D5	D5
IPE 750 x 173*	D3	D4	D5	D5
IPE 750 x 185	D4	D5	D5	D5

7. DREAMERS building

7.1. Generality

The DREAMERS demonstration project aims to show the applicability and the increased performance obtained through the application of the innovative FREEDAM connections in a real scale environment. The objective is the construction of an 800 sqm resilient steel building in the Campus of the University of Salerno already included in the public plan of investments.

The acronym DREAMERS stands for *"**D**esign **RE**search, implementation **A**nd **M**onitoring of **E**merging technologies for a new generation of **R**esilient **S**teel buildings"*.

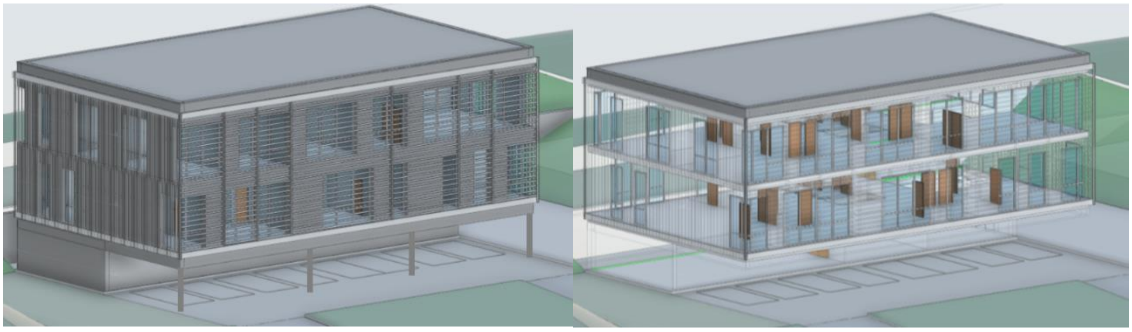


Fig 7: 3D rendering of the DREAMERS building, as it is (left) and without partitions, walls, columns (right)

The project has six partners, four of which are universities while the other two are industrial partners:

- Università degli Studi di Salerno (Italy);
- Università degli Studi di Napoli "Federico II" (Italy);
- Université de Liège (Belgium);
- Universidade de Coimbra (Portugal);
- Arcelormittal Belval & Differdange SA (Luxembourg);
- Knauf di Lothar Knauf SAS (Italy).

The proposed demonstration project will have a significant focus on the structural part, but the architectural components and the mechanical/electrical systems will also be designed considering the most advanced available standards.

The building is characterized by a 15 m x 25 m plan and 3 storeys with a total height of about 12 m. It has wide internal spaces that allow flexible use of the areas as lecture rooms or open space offices.

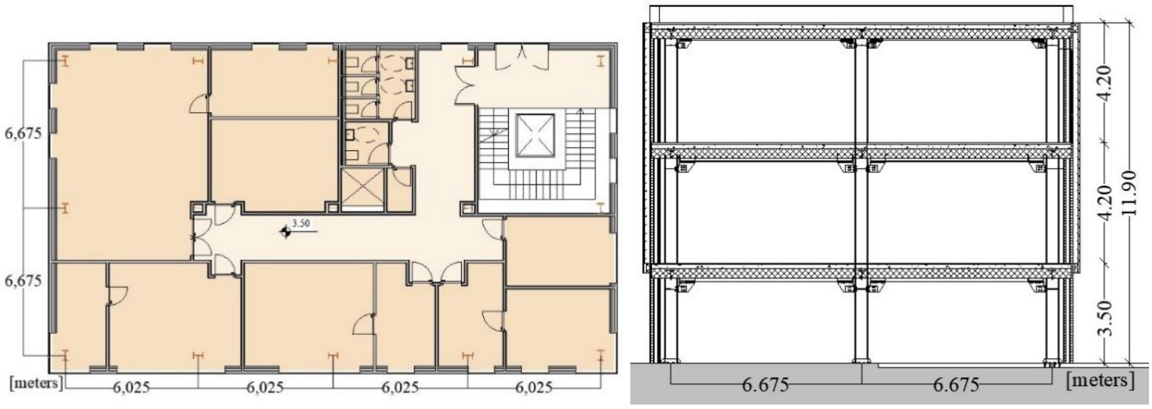


Fig 8: plan view (left) and elevation view of the Y-MRF (right)

The **structure** will have fifteen columns made with HEB 400 profiles and COFRADAL slim composite floors with HEB240-HEB300 cut-off beam, belonging to the system commercialized by Arcelor Mittal (CoSFB). This type of floors provides several technical advantages and they are particularly suitable for medium-long spans such as those adopted in the building, that will be around 7 m. In addition, the CoSFB are very thin (about 40 cm), allowing to maximize the internal spaces of the building.

The **seismic-resistant part** of the structure will be constructed, adopting four MRFs bays in the -X direction and four MRFs bays in the -Y direction. The beams of the seismic-resistant MRFs will be made of IPE 400 and IPE 450 profiles, and S355JR steel will be used for all structural components. The beam-to-column joints of the seismic-resistant MRFs will be realized adopting the devices standardized during the FREEDAM project; in particular, Device 1 and Device 2A will be used.

The non-structural elements will be conceived considering the damage issues, adopting partition walls, false ceilings, and façades able to follow the structural horizontal displacements without damage.

The DREAMERS consortium will design the building according to the new free from damage concept already validated within the research project FREEDAM. Since the building will become a benchmark for practitioners and construction companies, the objective of the project is also to provide up-to-date solutions in all the fields involved aiming at the overall sustainability of the intervention and the achievement of a positive social impact. To this scope, particular care will be devoted to the aspects related to the integration of the non-structural elements, the quality of the architecture, the energy efficiency, the fitting in the existing environment, and the adoption of mechanical/electrical systems based on the use of renewable resources.

The **usage** of the building will be for offices and classrooms, and its location is already defined. It will be built starting from 2023 by the University of Salerno, which is the public client. The structure will be part of the University of Salerno's new technological district. The new structure is part of the construction program for the next three years. It will be built in accordance with the public investment plan beginning in 2023.

The mechanical/electrical systems and claddings will also be designed with the LEED construction and building protocol in mind. BIM methodologies will be adopted to reduce critical risks for the project.

This demonstration building will provide an example that will have a resonance worldwide and will represent a benchmark for the future applications of this novel technology.

7.2. Added value of the project

The modern seismic design strategies implemented into international building codes require the occurrence of structural damage to dissipate the seismic input energy. This approach provides the development of a stable plasticization and the achievement of the safety requirements. Nevertheless, it leads to extensive damage, which is often distributed throughout the structure in many elements, significantly compromising the building reparability after strong seismic events, thus leading to high socioeconomic losses and downtime. To address these shortcomings and the need of new types of resilient structural systems, the use of friction joints has been recently proposed and studied demonstrating that beam-to-column connections equipped with friction dampers are an efficient solution to protect the frame components from damage, leaving the building fully operational even after destructive seismic events. Recent European research works have clearly demonstrated the high potential of such innovative connection typology for the development of a new generation of resilient constructions conceived to avoid damages and repair costs.

7.3. Structural details

In the DREAMERS building seismic action is resisted by moment-resistant frames, with four MRF bays along the X-direction and four MRF bays along the Y-direction. Each side of the building hosts two MRF bays. The MRF frames along the two directions are equipped with two kinds of FREEDAM connections.

In the following figure, MRFs bays are enlightened in red, together with the direction of the slab. As it can be seen, the slab loads don't bear on the seismic frames.

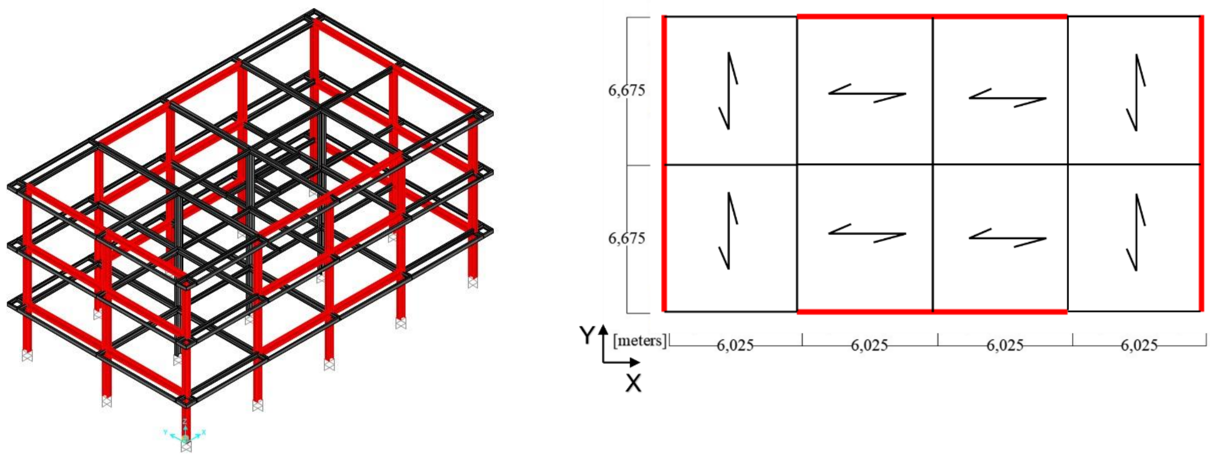


Fig 9: 3D view of the structural elements (left); plan view, with MRF bays in red (right)

Device 1 is used on all the MRF-X joints and on the third storey of the MRF-Y, while Device 2A is used only on the storey I and II of the MRF-Y. With this overall configuration, at I and II floor, it's possible to study the behaviour of two different devices, 1 and 2A, with the same beam-column set.

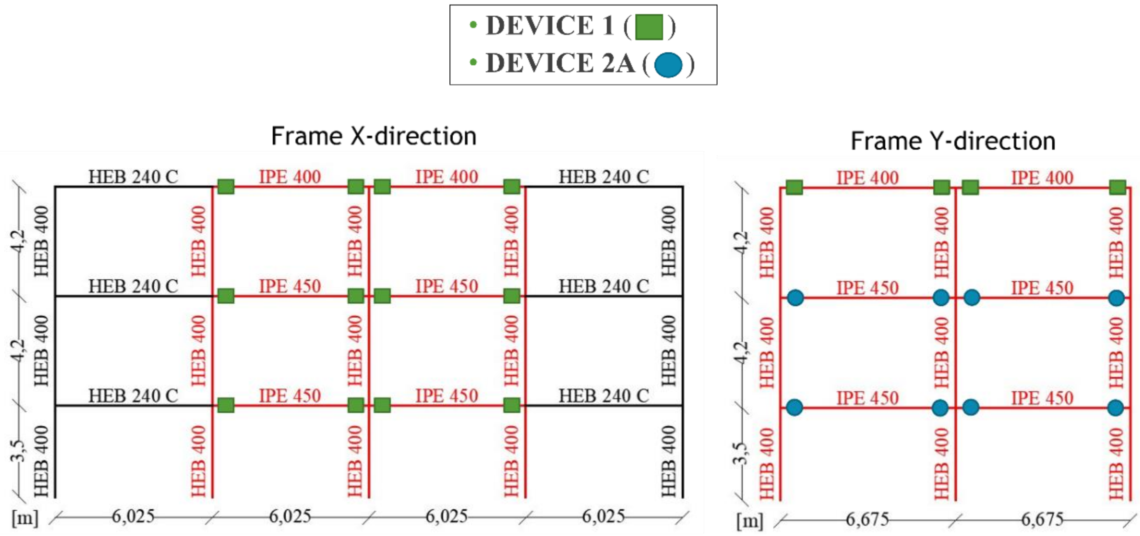


Fig 10: details of the X-MRF (left) and Y-MRF (right), with FREEDAM devices location

The following table is an overview of the modelled joint assemblies:

Tab 5: modelled joints, with details

Name	Device	Column	Beam	$F_{p,C}^{d}$ [kN]	$f = F_{p,C}^{d} / F_{p,C}^{EC3}$ [-]	h_s [mm]
X-III-TJ	D1-0.3	HEB400	IPE400	57,59	0,52	570
X-III-XJ	D1-0.3	HEB400	IPE400	57,59	0,52	570
X-II-TJ	D1-0.3	HEB400	IPE450	68,96	0,63	620
X-II-XJ	D1-0.3	HEB400	IPE450	68,96	0,63	620
Y-III-TJ	D1-0.3	HEB400	IPE400	57,59	0,52	570
Y-III-XJ	D1-0.3	HEB400	IPE400	57,59	0,52	570
Y-II-TJ	D2A-0.4	HEB400	IPE450	81,43	0,47	700
Y-II-XJ	D2A-0.4	HEB400	IPE450	81,43	0,47	700

Design Preload Force $F_{p,C,d}$ on device bolts is given by the desired **bending capacity level** m, namely the ratio between bending moment resistance of the connection and bending moment plastic resistance of the connected beam:

$$m = \frac{M_{j,Rd}}{M_{pl,Rd,beam}} = 0.3 \div 0.6$$

Where:

- $M_{j,Rd} = \mu_{dyn} F_{p,C,d} n_b n_s h_s$
    - μ_{dyn}: dynamic coefficient of the material of the friction pads, equal to 0,53 thanks to experimental tests;
    - $F_{p,C,d}$: design preload force of the damper bolts;
    - n_b: number of the damper bolts, for Device 1 and Device 2A this number is equal to 4
    - n_s: number of friction surfaces;
    - h_s: lever arm of the connection, that is to say the distance between the center of the damping bolts and the upper flange of the beam.
- $M_{pl,Rd,beam} = W_{pl,beam} f_y$
    - $W_{pl,beam}$: plastic first-order inertia modulus;
    - f_y: yield stress of the material of the beam.

In such way design preload force is given by:

$$F_{p,C,d} = \frac{M_{j,Rd}}{m \cdot \mu_{dyn} n_b n_s h_s} \in [0,4 \div 1] F_{p,C}^{EC3} = [0,4 \div 1] 0.7 A_{res} f_{ub}$$

7.4. Main goals

The main goals of the DREAMERS project will be the following:

- Structural design of a real scale demonstration building equipped with free from damage connections applying the design rules already established within the FREEDAM Project and working within the framework of Eurocodes (in particular, EN1993, EN1994 and EN1998);
- Design of the building to assure that the non-structural elements can accommodate without damage the lateral displacements of the building occurring in the case of destructive seismic events;
- Architectural design of the demonstration building to find an aesthetic solution for the façade that will assure a positive social impact of the structure;
- Design of the building optimizing the sustainability with particular reference to the principles set by the LEED construction and building protocol;
- Design of the electrical and mechanical services of the demonstration building finding solutions able to meet the sustainability requirements provided by the LEED construction protocol;
- Minimization of the critical risks through the application of 4D Building Information Modelling tools (BIM);
- Implementation of an effective project management strategy able to individuate the critical risks and the actions to be implemented whenever needed;
- Development of a multi-disciplinary design group with a holistic view of the design process involving experienced partners from the Academia, Industry and Professional world;
- Transferring the know-how of the FREEDAM research project from Academia to a wide audience of engineers and industries;
- Development of advanced seismic experimental tests to demonstrate the resilience of the connections employed in the building;
- Development of advanced large-scale experimental tests to demonstrate the robustness of the connections employed in the building;
- Development of advanced large-scale experimental tests to show the fire-resistance of the connections employed in the building and the solution for their protection;
- Development of advanced numerical models to quantify the seismic performance, robustness and fire resistance of the building considering also the effect of the non-structural components;

- Erection of a demonstration building to show the possibility to construct a free from damage building in a real scale environment, with actual time and budget constraints;
- Site testing of the building to demonstrate the resilience of the structure;
- Structural Health Monitoring of the building to control the structural performance over time, especially in case of critical events such as earthquakes, fires, extreme loading scenarios;
- Development of an impactful dissemination strategy based on workshops, training days, open days to the construction site and an informative book to be freely distributed.

8. FE modelling

8.1. Modelling assumptions

FE Modelling was used to characterize the actual behaviour of the joint. The finite element (FE) model is developed using the software Abaqus 2017. Dynamic implicit solver is used to reproduce the quasi-static behaviour of the investigated beam-to-column joint.

Abaqus has two analysis methods—Abaqus/Standard and Abaqus/Explicit—that it may use to solve structural problems. The implicit analysis approach, which employs numerical methods to solve ordinary and partial differential equations, discretizes the equation of motion by using *reverse Euler time integration*. The solution at a given time step depends on the state of the system at the previous step. Therefore, the state of a system calculated with implicit technique at a particular time step differs from the one calculated by using explicit techniques. Since the static response ignores any transitory behaviour that happens while the loads are being applied to the structure, static analysis can be thought of as implicit. However, solving a non-linear problem may require several iterations.

The constitutive laws of the materials are represented by means of the engineered curves (true stress-true strain curves) derived by coupon tensile tests. The Elastic Modulus is 210 GPa for generic steel and 130 GPa for bolts. Poisson ratio is 0.3 for generic steel and for bolts.

Based on the stated premises, the analysis is carried out in two steps, the first of which involves applying preload to the bolts and the second of which involves applying a vertical displacement to the selected reference point. By turning on the "Nlgeom" option in the Step Module, the geometric imperfections are taken into account.

Surface-to-surface contact formulation is used to model each interaction. For all interactions, normal "hard contact" and tangential behavior are defined. Steel-to-steel friction is assumed to have a value of 0.3, while experimental studies have determined that the friction dynamic coefficient of the damper device is equal to 0.53.

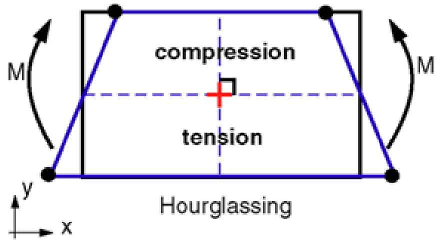
Loads and boundary conditions are applied to reference points, which are representative of the specific cross-section. All the nodes of a certain cross-section are bounded to the corresponding reference point by using a *rigid body constrain*. Welds are simulated by using *Tie constraint*.

The bolt preload is applied to the cross-section of the bolt shank, using the option "*Bolt Force*" available in Abaqus/CAE and the magnitude is calculated following the prescription given by Eurocode 3 Part 1-8.

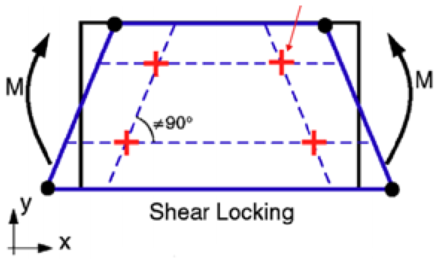
All elements are modelled by using **C3D8I** 8-node linear brick (incompatible modes, first-order integration) an enhanced version of the C3D8-element. Shear locking is specifically avoided, and volumetric locking is greatly diminished. This is accomplished by adding so-called bubble functions, which have nonzero values between all nodes and zero values at all nodes, to the common shape functions. When using linear elements that are susceptible to bending, the C3D8I element should always be utilized. Although the quality of the C3D8I element is far better than the C3D8 element, the best results are usually obtained with quadratic elements (C3D20 and C3D20R), but to lower the computational burden, the choice fell on the C3D8I element.

The main issues a FE element can encounter are:

- **Hourglassing** can occur in first-order, reduced-integration elements (C3D8R). Because there is only one integration point, neither of the visualization lines in the schematic below have changed in length, and the angle between them is also unchanged by the imparted deformation. This can result in zero-strain deformation modes that produce highly spurious results. This issue, however, can usually be mitigated by using second-order, reduced-integration elements (C3D20R).



- **Shear locking** can occur in first-order, fully-integrated elements (C3D8) that are subjected to bending. This occurs when artificial shear strain develops due to an inability of the element edges to bend. As a result of this nonphysical shear strain, these elements can be too stiff when used in bending-dominant problems. Fortunately, though, this issue can also be address by using second-order elements (C3D20).



- **Volumetric locking** can occur in first- and second-order, fully-integrated elements when material behaviour is nearly incompressible (e.g., rubber). This causes unrealistic pressure stresses to develop at the integration points, which can make the element behave too stiffly. Fortunately, first-order, fully-integrated hexahedral elements use selectively-reduced-integration (reduced integration on the volumetric terms only), so these elements do not experience volumetric locking with nearly-incompressible materials. (As a side note, one should utilize the Hybrid element formulation when working with nearly-incompressible materials).

8.2. Types of analysis

MRFs 2D joint assemblies have been modelled and tested under seismic scenario and robustness scenario. In such way two typologies of joints are investigated: T-joint, external joint of the MRF, and X-joint, internal joint of the MRF.

Five kinds of analyses have been carried out:

- **M-HOG**, Monotonic Loading "Hogging": downward imposed displacement of the beam free end, resulting in a Hogging bending moment on the beam, with upper flange in tension;
- **M-SAG**, Monotonic Loading "Sagging": upward imposed displacement of the beam free end, resulting in a Sagging bending moment on the beam, with lower flange in tension;
- **CYC**, Cyclic Loading: imposed displacement of the beam free end according to AISC-345 protocol
- **CL-HOG**, Column Loss "Hogging": imposed displacement of the beam free end due to the loss of a nearby column;
- **CL-SAG**, Column Loss "Sagging": imposed displacement of the column base due to the loss of the column itself

The first three analyses represent the Seismic Scenario, while the last two analyses represent the Robustness Scenario.

Regarding the Cyclic Loading, here's the AISC-345 loading protocol:

(1) 6 cycles at θ = 0,00375 rad;
(2) 6 cycles at θ = 0,005 rad;
(3) 6 cycles at θ = 0,0075 rad;
(4) 4 cycles at θ = 0,01 rad;
(5) 2 cycles at θ = 0,015 rad;
(6) 2 cycles at θ = 0,02 rad;
(7) 2 cycles at θ = 0,03 rad;
(8) 2 cycles at θ = 0,04 rad;
Continue loading at increments of θ = 0,01 rad, with two cycles of loading at each step.

Model boundary conditions are imposed with regards the typology of analysis; in fact different analysis calls for different static schemes to properly represent the scenario.

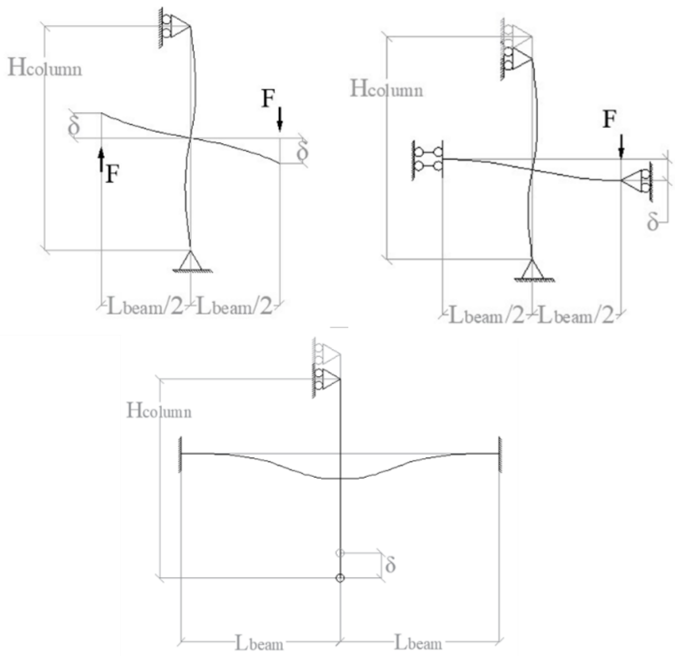


Fig 11: static schemes for: Seismic Scenario (top left), Hogging Column Loss Scenario (top right), Sagging Column Loss Scenario (bottom)

- An X-joint under Seismic Scenario is modelled by considering half length of the upper column, half length of the lower column, half length of the beams converging into the joint. The top-end of the upper column is restrained by a trolley while the bottom-end of the lower column is restrained by a hinge. The beams instead are considered free ended. These two ends are subject to a vertical displacement, equal in modulus, but opposite in sign.

To investigate the joint behaviour in terms of robustness two Scenarios have been planned. In fact, the Column Loss Scenario with Hogging bending moment represents the case where the column of the model joint assembly is part of the Indirectly Affected Part, namely joint C or D of the figure, while the Column Loss Scenario with Sagging bending moment represents the case where the column of the model joint assembly is part of the Directly Affected Part, namely joint A of the picture.

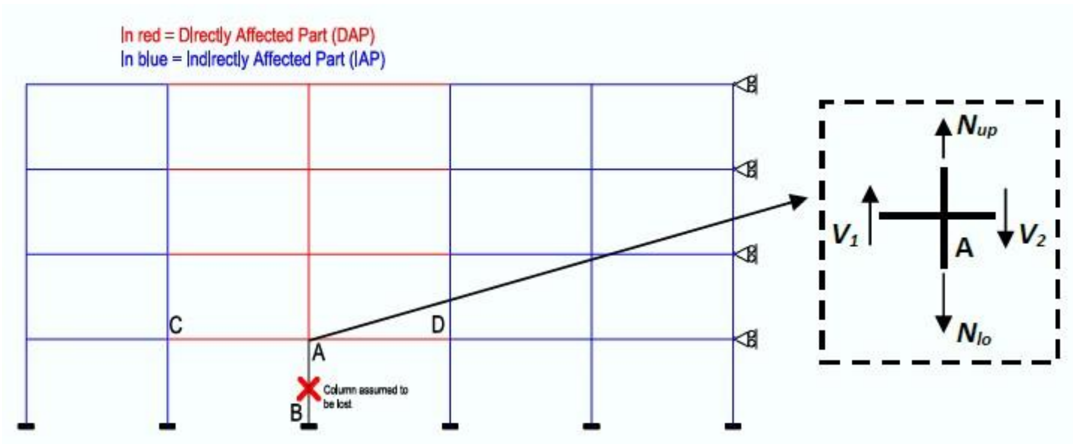


Fig 12: Directly Affected Part and Indirectly Affected Part of a Column Loss Scenario

Then:

- An X-joint under Hogging Column Loss is similarly modelled by considering half length of the upper column, half length of the lower column, half length of the beams converging into the joint. The top-end of the upper column is restrained by a trolley while the bottom-end of the lower column is restrained by an hinge. In order to evaluate the axial load of the catenary effect in the beams, the point where the imposed displacement is applied is restrained as a trolley, while the other not-loaded beam end is restrained as a double pendulum to simulate the flexural continuity at beam midspan.

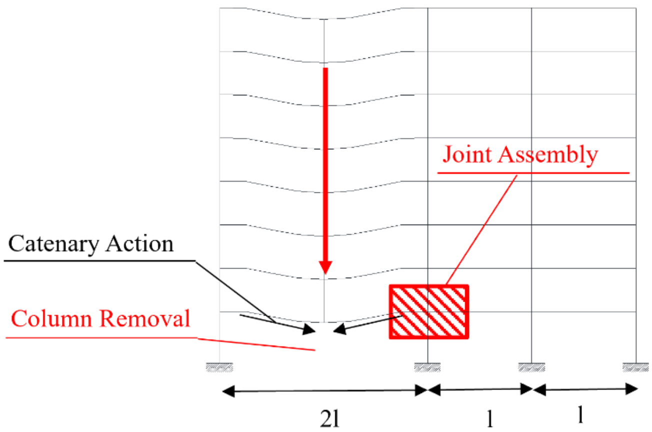


Fig 13: Hogging Column Loss

- An X-joint under Sagging Column Loss is modelled by considering half length of the upper column, half length of the lower column, the entire length of the beams converging into the joint. The top-end of the upper column is restrained by a trolley while the bottom-end of the lower column is free. The beams ends are fixed. In this case the displacement is imposed to the bottom end of the lower column

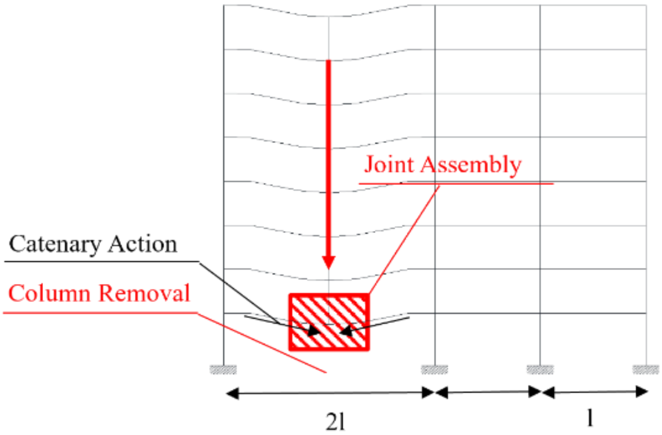


Fig 14: Sagging Column Loss

In Column Loss Scenario large deformation are involved, so it necessary to evaluate the second-order bending moment, defined as the first-order bending moment (calculated as vertical shear force multiplied by the length of the beam) minus the bending moment derived by the catenary action (product between the catenary action axial load and the vertical displacement of the beam end).

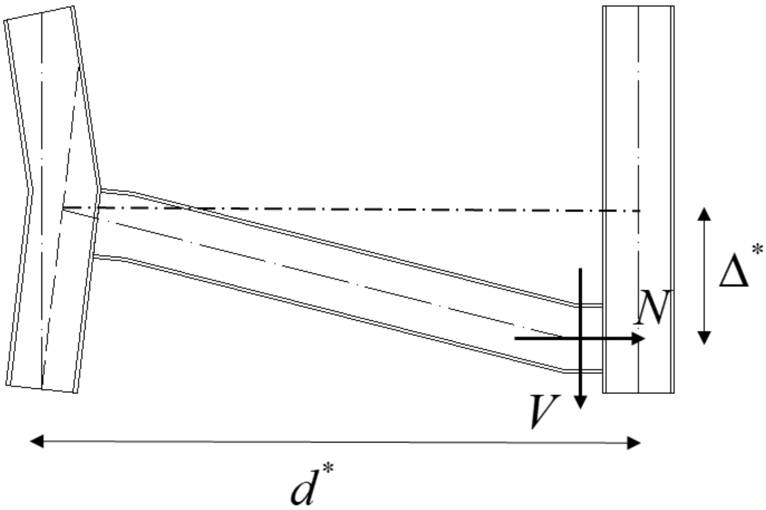


Fig 15: Forces involved in a Column Loss

$$M^{I^\circ Order} = V_{\text{App-point}} \cdot d^*$$

$$M^{II^\circ Order} = M^{I^\circ Order} - N_{\text{Cat-Act}} \cdot \Delta^*$$

9. Detailed analysis of (X-II-XJ) MRF joint

In this chapter a detailed analysis of a MRF joint of the DREAMERS building is shown. The selected joint is the X-II-XJ, that is to say MRF joint located on the X-MRF (X), II floor (II), X-joint configuration (XJ).

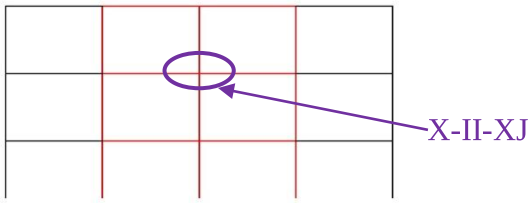


Fig 16: Selected joint with its location on the MRF

9.1. Monotonic Loading

Bending moment-chord rotation diagrams show a stable response curve both in Hogging and Sagging, with a plateau very close to the bending moment design resistance of the connection up to 0.06 rad of rotation. The lower resistance of the Sagging scenario is given by the high deformability of the L-stubs in tension, which causes a loss of clamping in the bolts.

For the Hogging, after 0.06 rad the bolts device go in contact with the slotted holes surfaces of the haunch, while the haunch itself is pushing on the column flange. As a result, the device doesn't work properly anymore and the M-rot curve shows an increase of stiffness: the damage extends also to structural parts.

For the Sagging the issue is similar, since the damper bolts go in contact with the slotted holes of the haunch.

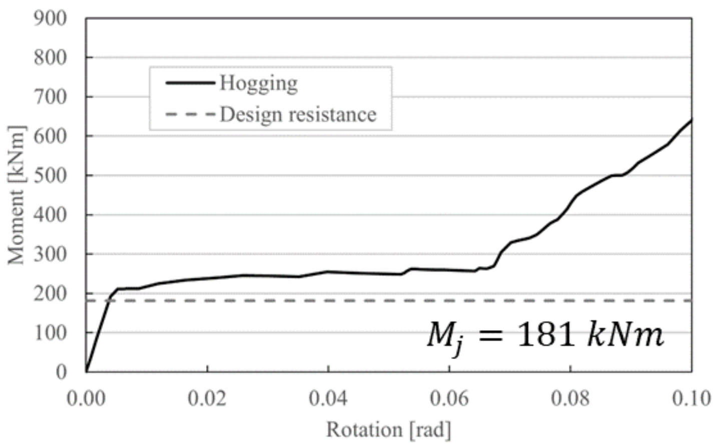


Fig 17: Hogging bending moment-rotation curve

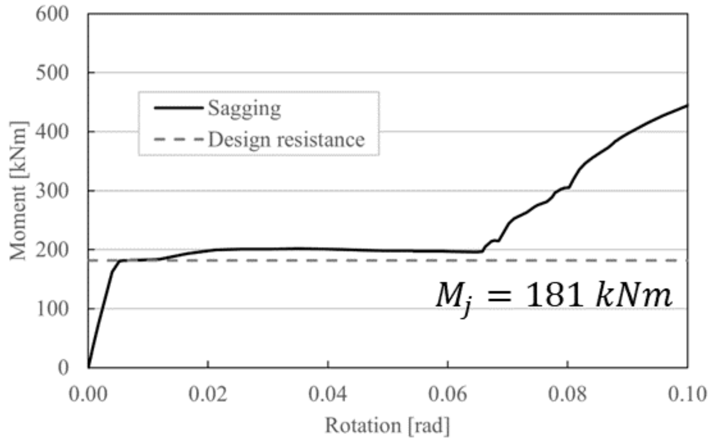


Fig 18: Sagging bending moment-rotation curve

Clamping force-rotation diagram shows that damper bolts are able to maintain the preload up to 0.06 of rotation. The first bolt that start losing its preload differs between the two scenarios: Bolt 1-2 (bottom bolt row, midspan side) in Hogging, Bolt 2-2 in Sagging (bottom bolt, column side).

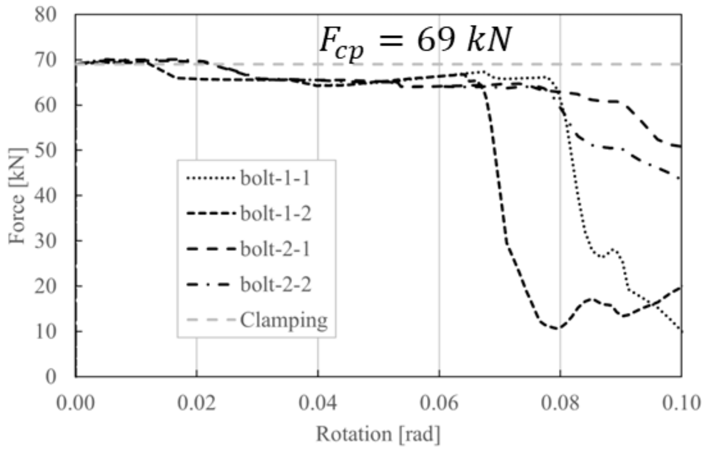


Fig 19: (Hogging) Evolution of the Preload Force in the damper bolts

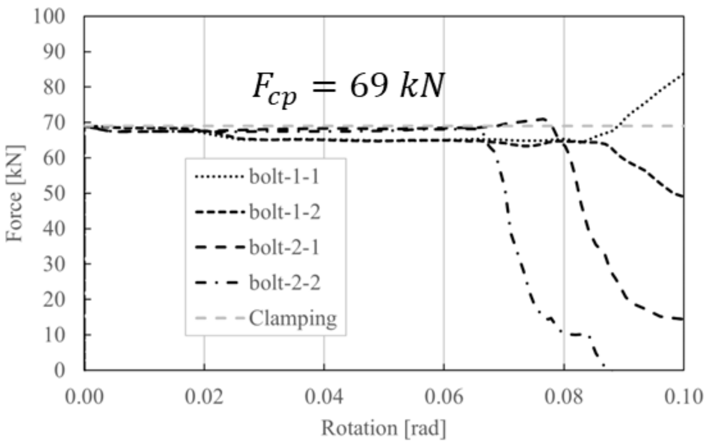


Fig 20: (Sagging) Evolution of the Preload Force in the damper bolts

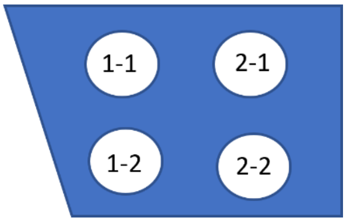


Fig 21: Damper bolts nomenclature

Some FE analysis screen-shots can be useful to visualize what it’s happening to the joint. The point of rotation on the T-stub web is clearly visible, with the sliding mechanism offered by the device. The lever arm of the connection is, as said before, the distance between the centre of damper bolts and the upper flange of the column.

Von Mises stress distribution shows:

- In Hogging (left), the formation of a diagonal tie at the end of the beam, which is coupled with a compressive diagonal strut on the haunch;
- In Sagging (right), the opening of the L-stubs, with the runaway of the haunch, restrained only by damper bolts.

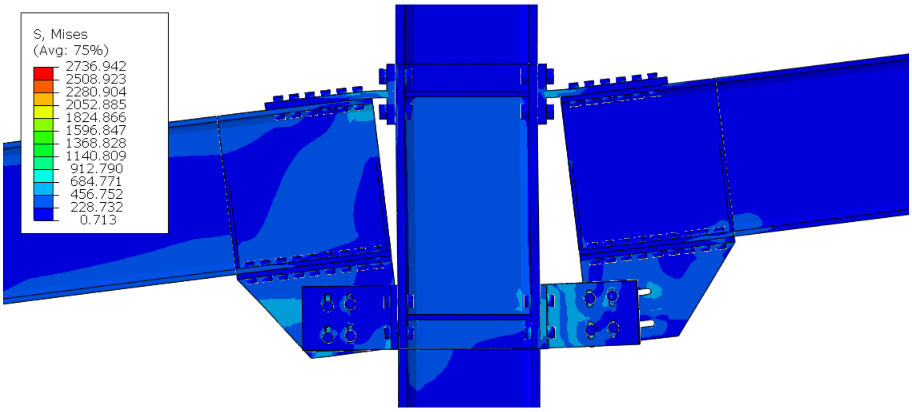


Fig 22: (Hogging) Von Mises stress distribution

PEEQ (equivalent plastic deformation) distribution shows no damage across the joint, validating the “FREE from DAMage” behaviour of the connection. The only damage registered by the software is at the level of the connection (damper bolts and haunch).

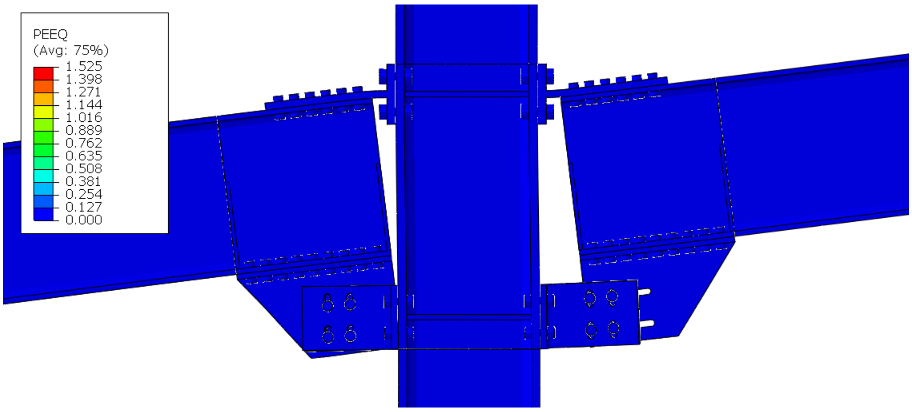


Fig 23: (Sagging) PEEQ distribution

9.2. Cyclic Loading

Joint exhibits a very stable cyclic behaviour with a regular and stable shape of hysteretic cycles, allowing the friction pads to dissipate energy while all the other components of the assembly remain within the elastic range. The slight asymmetry of the cycles, with the hogging side stronger than the sagging side, is still due to the deformability of the L-stubs in tension (as seen in the previous paragraph). Joint can successful reach rotation of 0,05 rad. For the Von Mises stress and PEEQ distributions is still valid what it was said for the Monotonic Loading Scenarios.

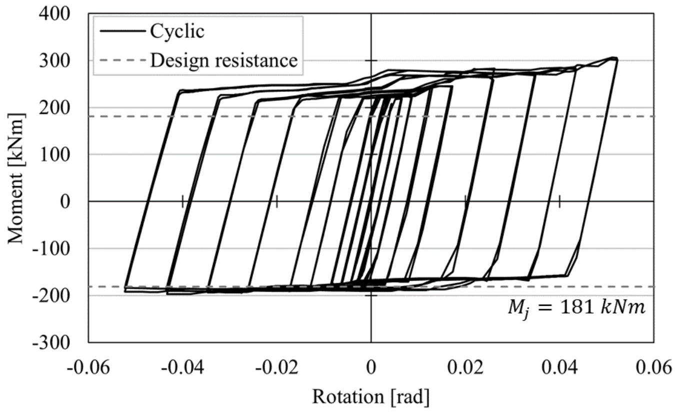


Fig 24: Cyclic response

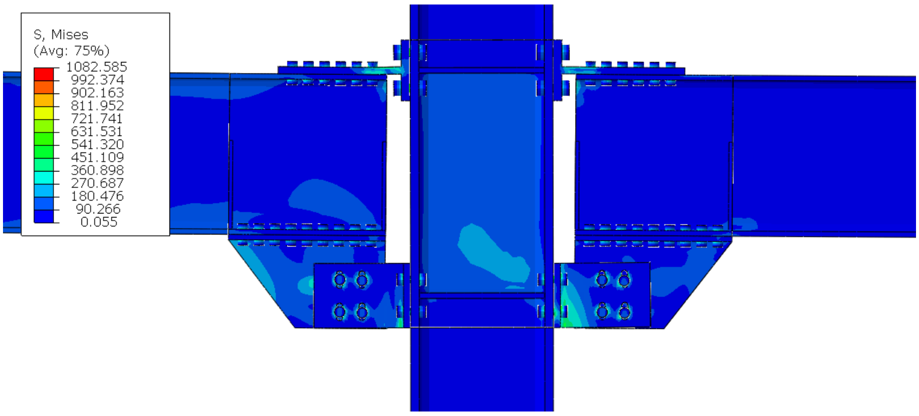


Fig 25: Von Mises stress distribution

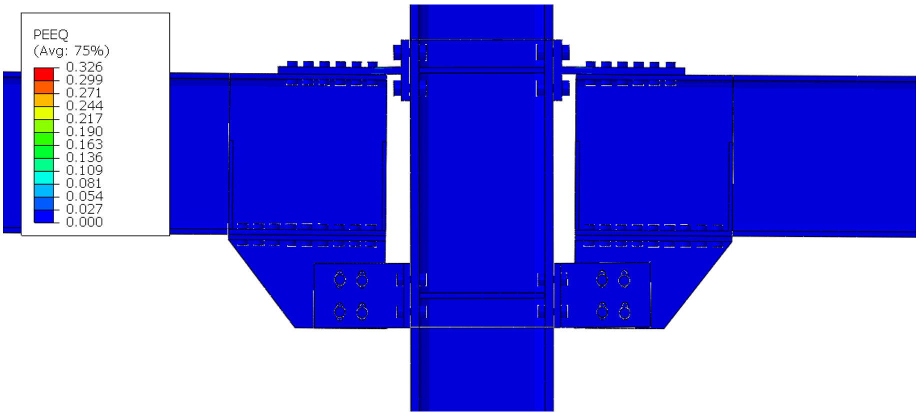


Fig 26: PEEQ stress distribution

9.3. Column Loss scenario

In this Scenario large displacement are involved, so it's reasonable to compute the second-order bending moment, defined previously. The horizontal displacement of the beams end is restrained and then beams are now subjected to a tensile axial load, responsible of the catenary effect.

9.3.1. Column Loss "Hogging"

In the hogging moment-rotation diagram is possible to see this catenary effect, that gives a big contribution in terms of resistance already for small rotations, when confronted with the Monotonic Loading Scenario.

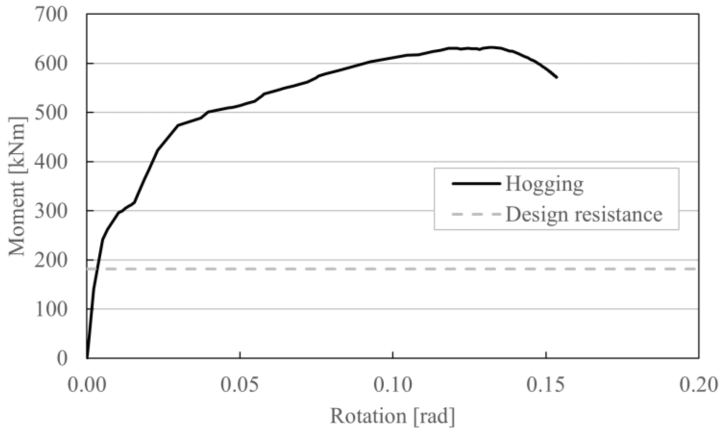


Fig 27: Hogging bending moment-rotation curve

The evolution of the external axial force is here divided by the axial plastic resistance of the connected beam. It has to be noted that the maximum external axial force is about $N_{Ed} = 0.4N_{pl,Rd,beam}$.

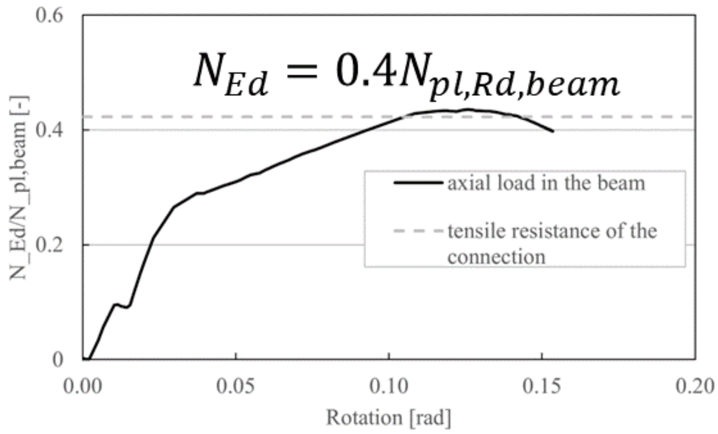


Fig 28: non-dimensional axial load on beam

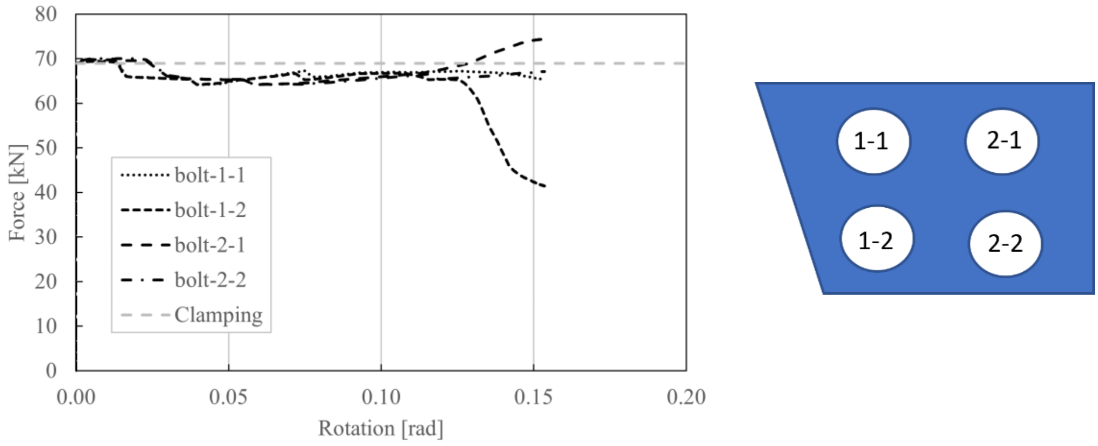


Fig 29: (Hogging) Evolution of the Preload Force in the damper bolts; damper bolts nomenclature (right)

By having a look at the FE results it becomes obvious what part of the entire connection fails first. The upper T-stub is characterized by a brittle Failure Mode 2 (very close to Failure Mode 3).

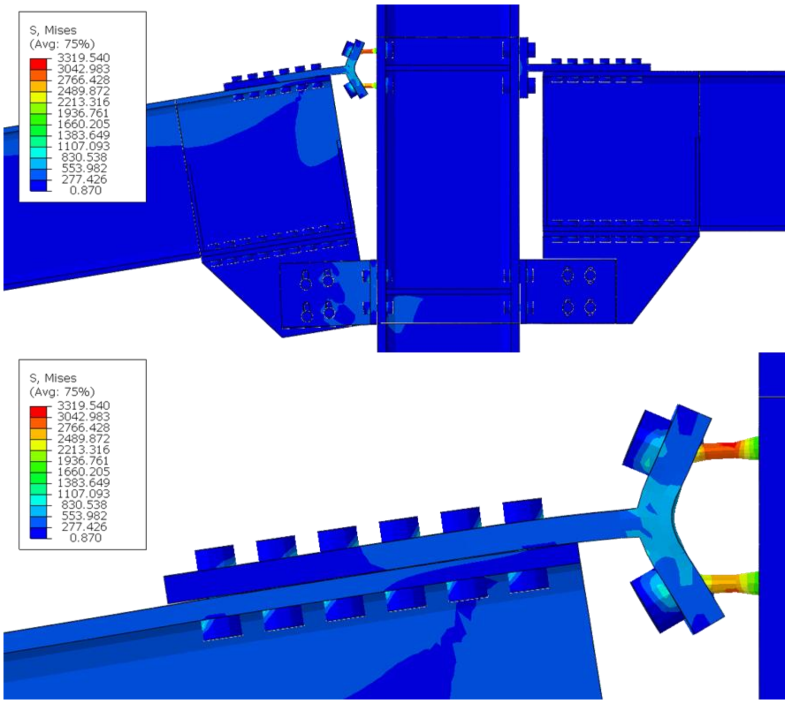


Fig 30: Von Mises stress distribution, with detail of the T-stub

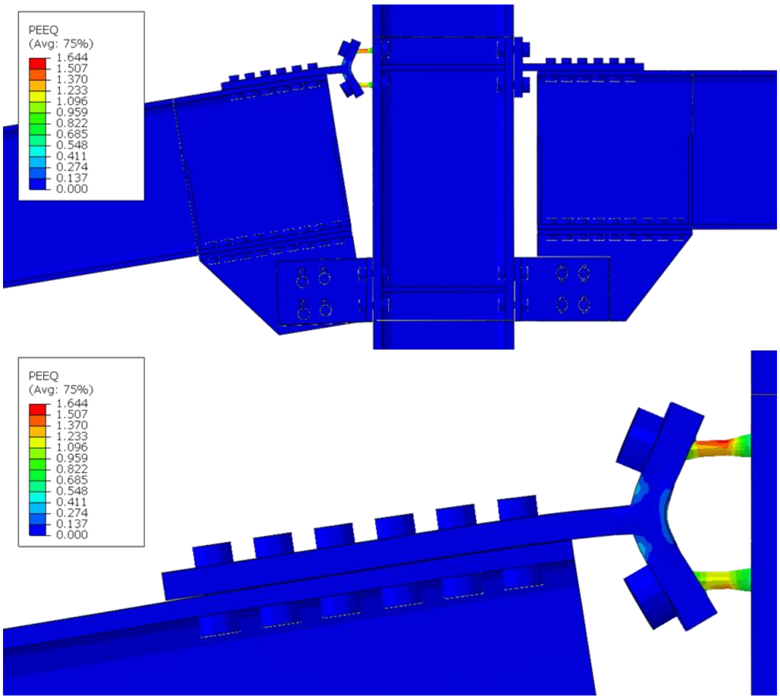


Fig 31: PEEQ distribution, with detail of the T-stub

9.3.2. Column Loss "Sagging"

For the Sagging Column Loss Scenario, the bending moment-rotation diagram shows an arching effect, followed by a weak catenary effect. The higher values of Von Mises stress are concentrated in the device and precisely at the haunch and the damper bolts. PEEQ higher values are registered on the shank of the damper bolts Bolt 2-1 and Bolt 2-2.

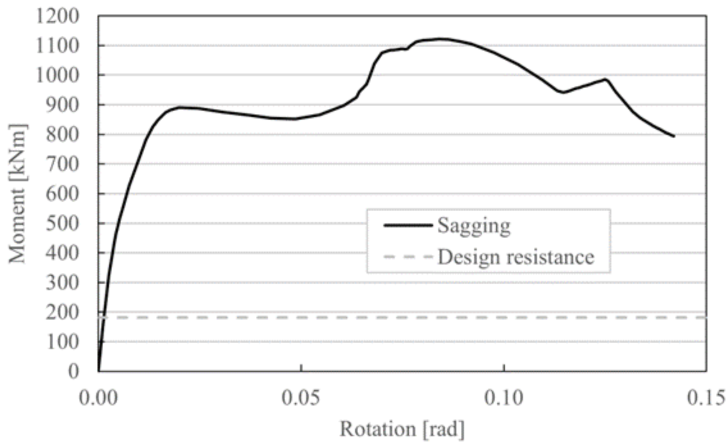


Fig 32: Sagging bending moment-rotation curve

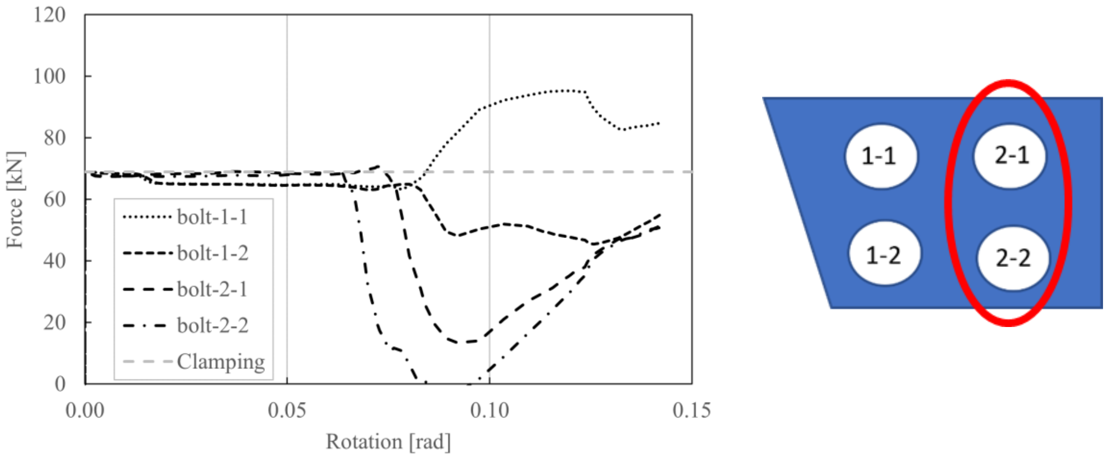


Fig 33: (Sagging) Evolution of the Preload Force in the damper bolts

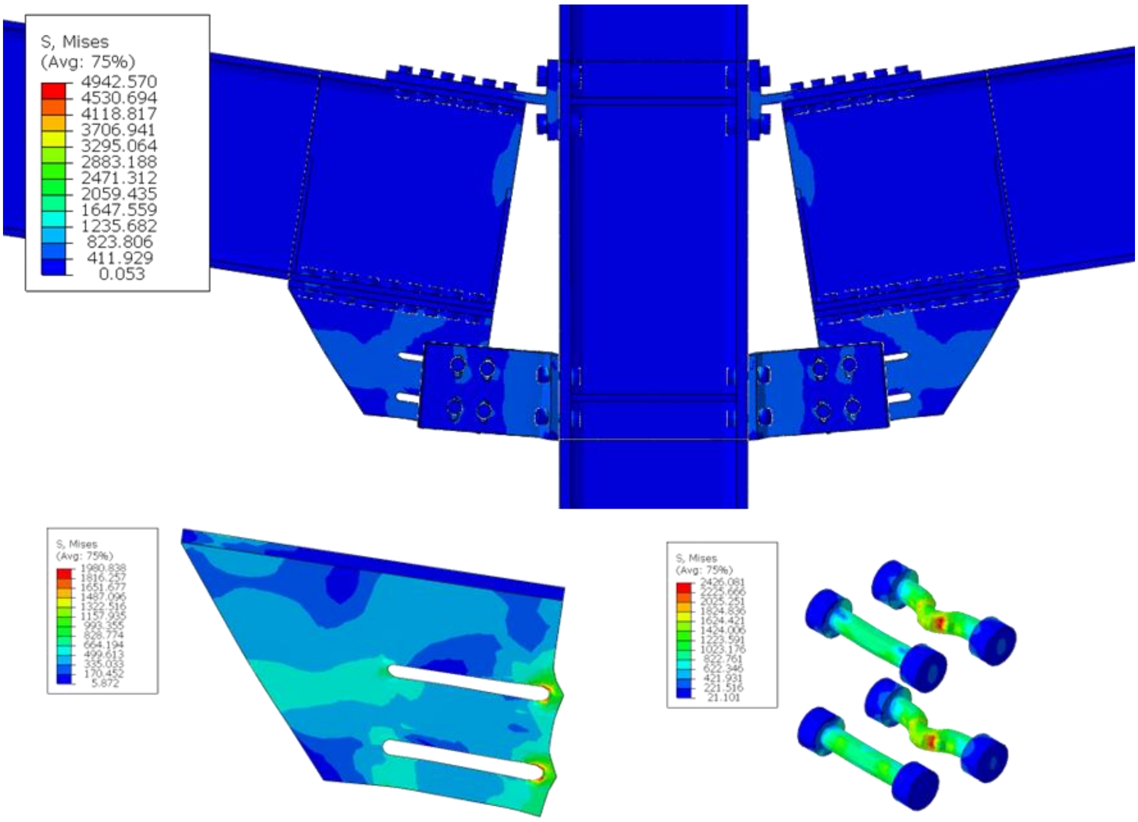


Fig 34: Von Mises stress distribution, with haunch and damper bolts in detail

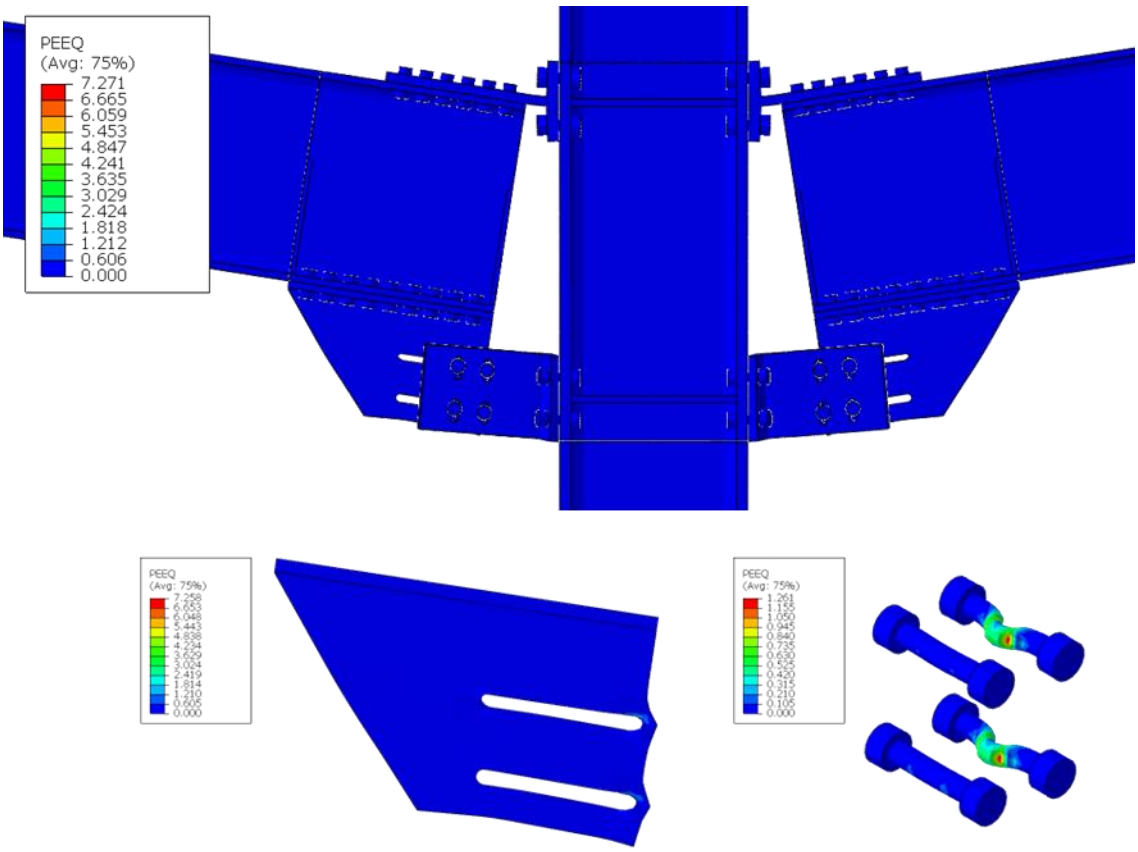


Fig 35: PEEQ distribution, with haunch and damper bolts in detail

10. Enhancement of upper T-stub response: re-design

10.1. Introduction

FE analysis of the Column Loss Scenario shows that the current version of the joint hasn't a sufficient rotational capacity. In fact, when the connection is subject to hogging bending moment, the bolts connecting the upper T-stub to the column flange undergo brittle failure (failure mode 2, referring to the equivalent T-stub method). It is therefore necessary to re-design the upper T-stub connection, in order to guarantee a more ductile type of failure.

A T-stub modification is here proposed with the aim of strengthening the connection and let it reach larger rotation. According to FE simulations, the external force (pull force) acting on the T-stub web can be set equal to $N_{Ed} = 0.4N_{pl,Rd,beam}$.

10.2. Bolted beam to column connections

In a bolted end plate connection, each bolt row contributes to the resistance of the connection. In case of pure flexure, with no axial force in the beam, the resultant in tension is equal to the resultant in compression. Bolts in bearing and shear resist vertical shear; the force is typically thought to be resisted mostly by bolts next to the compression flange. For ease of design, it may be assumed that the compression resistance is concentrated at the level of the center of the flange in the ultimate limit state, when the center of rotation is at or close to the compression flange.

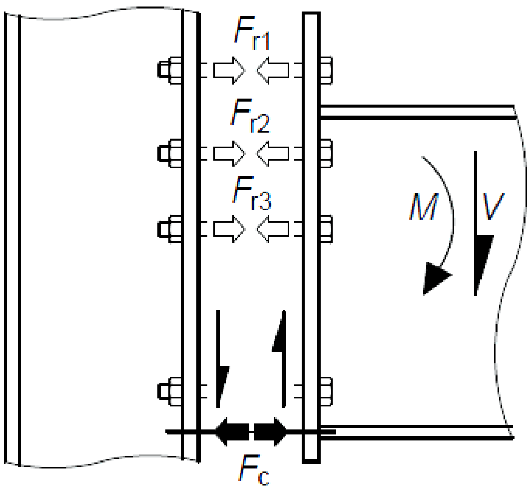


Fig 36a: Forces in an end plate connection

Design practice in the past has been to assume a "triangular" distribution of forces, pro rata to the distance from the bottom flange. This is because the bolt row that is farthest from the compression flange will tend to attract the largest tension force. However, the full resistance of the lower rows may be utilised if either the column flange or the end plate is sufficiently flexible (as described by NA.2.7 of the UK NA) to achieve a ductile failure mode (this is sometimes referred to as a plastic distribution of bolt row forces).

The design method is an iterative procedure. It requires calculating the minimum between the resistance of each bolt row's effective tension resistance, which is:

- bolts
- end plate
- column flange;
- beam web;
- column web

A "triangular" distribution of forces, which has its zero value point at the compression flange level, can be used to limit this effective tension resistance. The shear resistance of the bolt rows, as well as the quality of the welds in the connections, must be taken into consideration because they have an impact on the values of the m length (distance of the first plastic hinge from the bolt axis).

Bolt rows can be used alone or in group. Starting with the bolt row that is in tension closest to the beam flange, the resistances of each row are determined. For each bolt row, an effective length of equivalent T-stub is determined for each of the potential yield line patterns, which is defined by the "location" of the bolt row, which is the distance between bolts and stiffeners, as well as the edge distances and bolt spacing.

The row's effective design resistance is the lowest of the resistances determined for the connection's beam and column sides.

The same method is used to assess the resistances of groups of bolt rows, however in this instance, the effective length refers to the bolt rows functioning as a group and having various yielding line configurations. The effective design resistance of the collection of rows is still the lowest among those determined for the connection's beam and column sides. It is crucial to emphasize that no group effect is conceivable, and the resistance of the group is not assessed if rows are separated by a flange or stiffener.

Bolt rows that are not separated by a flange or a stiffener are typically near enough to function as a unit. The uppermost row, the one in tension adjacent to the beam flange, is believed to offer the resistance that it would have as a solo row, with lower rows in the group merely providing the additional resistance that each row adds as it is added to the group. Thus, to put it simply:

$$F_{t1,Rd} = [resistance\ of\ row\ 1\ alone]$$

$$F_{t2,Rd} = \min \begin{cases} resistance\ of\ row\ 2\ alone \\ (resistance\ of\ row\ 2 + 1) - F_{t1,Rd} \end{cases}$$

$$F_{t3,Rd} = \min \begin{cases} resistance\ of\ row\ 3\ alone \\ (resistance\ of\ row\ 3 + 2) - F_{t2,Rd} \\ (resistance\ of\ row\ 3 + 2 + 1) - F_{t2,Rd} - F_{t1,Rd} \end{cases}$$

and in a similar manner for the subsequent rows.

To determine a row's resistance, either on an individual or group level, is the process before thinking about the following (lower) row. Additionally, the effective design resistance of lower rows must be constrained to the aforementioned "triangular" distribution if the failure mode for any row is not ductile.

10.3. Checks for a bolted connection

According to EN1993:1-8, a bolted connection must pass the following checks:

- Minimum and maximum spacing, end and edge distances
- For the individual bolt: shear failure, tensile failure, combined shear and tension
- Connected plates: bearing resistance, punching shear resistance, block-tearing

10.4. Equivalent T-stub method

This simplified method allows to simulate the behaviour of a bolt row as if it belonged to an equivalent T-stub, for which several studies are available in the literature. The equivalent T-stub method can be found in EN1993:1-8 and in the SCI-P398.

The resistances of the equivalent T-stubs are evaluated separately for the end plate and the column flange. The resistances are calculated for three possible modes of failure. The resistance is taken as the minimum of the values for the three modes:

$$F_{T,Rd} = \min(F_{T,1,Rd};\ F_{T,2,Rd};\ F_{T,3,Rd})$$

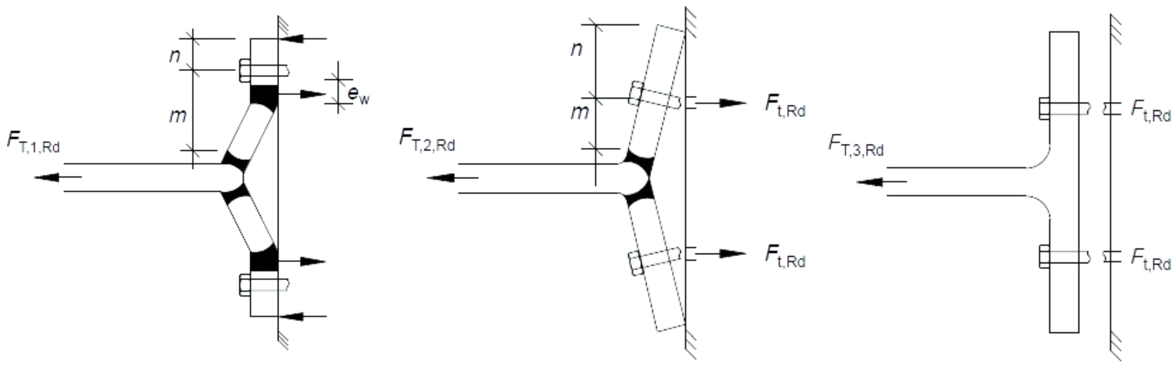


Fig 36b: different mode of failure for a T-stub; mode 1 (left), mode 2 (center), mode 3 (right)

Mode 1 consists in the complete flange yielding and it is the more ductile and the more preferable failure mode, since it allows to make the most of the plastic capacity of the materials involved. It gives rise to the formation of four plastic hinges on the T-stub flange, two in proximity of the T-stub web and two in proximity of the bolts. The resistance of this failure mode is highly dependent by the distances between the bolts, the plastic hinges zones, the edge distance of the plate. In the generic case:

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} \qquad\qquad M_{pl,1,Rd} = \frac{0.25\Sigma l_{eff,1} t_f^2 f_y}{\gamma_{M0}}$$

Mode 2 consists in the simultaneous failure of the bolts and the yielding of the T-stub flange in proximity of the web. As the previous failure mode, also this one is highly dependent by the distances between the bolts, the plastic hinges zones, the edge distance of the plate. In the generic case:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n} \qquad\qquad M_{pl,2,Rd} = \frac{0.25\Sigma l_{eff,2} t_f^2 f_y}{\gamma_{M0}}$$

Mode 3 is the brittlest failure mode, since it relies on the ultimate tensile resistance of the bolts only, disregarding in-toto the plastic capacity of the T-stub flange. Mode 3 design resistance is then the sum of the ultimate tensile resistance of the bolts involved:

$$F_{T,3,Rd} = \Sigma F_{t,Rd}$$

In case no prying forces develop, an intermediate mode of failure is possible, namely **Mode 1-2**. But, as stated in EC1993:1-8, Tab. 6.2, "*in bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop*".

The mode of failure of the equivalent T-stub is highly dependent from the following lengths:

- m: distance of the first plastic hinge from the bolt axis;
- $n = \min(e_{min}; 1.25m)$: distance of the free edge with regards the bolt axis (end distance).

Effective lengths are characterized by the shape of the yield lines; these can be in circular patterns ($l_{eff,cp}$) or non-circular patterns ($l_{eff,nc}$), and they depend on the location of the bolt row, with regards the stiffeners, the plate edges, the other bolt rows. The effective lengths for Mode 1 and Mode 2 are defined as:

- $l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc})$
- $l_{eff,2} = l_{eff,nc}$

<table>
<tr><td></td><td colspan="2">Prying forces may develop, i.e. $L_b \leq L_b^*$</td><td>No prying forces</td></tr>
<tr><td>Mode 1</td><td>Method 1</td><td>Method 2 (alternative method)</td><td rowspan="4">$F_{T,1-2,Rd} = \dfrac{2M_{pl,1,Rd}}{m}$</td></tr>
<tr><td>without backing plates</td><td>$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}}{m}$</td><td>$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}}{2mn-e_w(m+n)}$</td></tr>
<tr><td>with backing plates</td><td>$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}+2M_{bp,Rd}}{m}$</td><td>$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}+4nM_{bp,Rd}}{2mn-e_w(m+n)}$</td></tr>
<tr><td>Mode 2</td><td colspan="2">$F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd}+n\Sigma F_{t,Rd}}{m+n}$</td></tr>
<tr><td>Mode 3</td><td colspan="3">$F_{T,3,Rd} = \Sigma F_{t,Rd}$</td></tr>
</table>

Mode 1: Complete yielding of the flange
Mode 2: Bolt failure with yielding of the flange
Mode 3: Bolt failure

L_b is - the bolt elongation length, taken equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut or
- the anchor bolt elongation length, taken equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half the height of the nut

$L_b^* = \dfrac{8{,}8m^3 A_s}{\Sigma \ell_{eff,1} t_f^3}$

$F_{T,Rd}$ is the design tension resistance of a T-stub flange
Q is the prying force

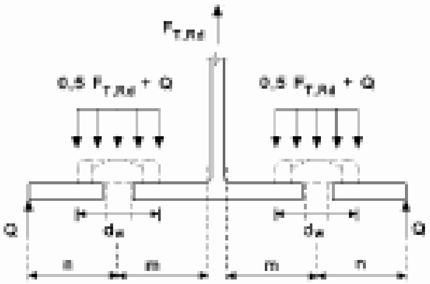
$M_{pl,1,Rd} = 0{,}25\Sigma \ell_{eff,1} t_f^2 f_y / \gamma_{M0}$

$M_{pl,2,Rd} = 0{,}25\Sigma \ell_{eff,2} t_f^2 f_y / \gamma_{M0}$

$M_{bp,Rd} = 0{,}25\Sigma \ell_{eff,1} t_{bp}^2 f_{y,bp} / \gamma_{M0}$

$n = e_{min}$ but $n \leq 1{,}25m$

$F_{t,Rd}$ is the design tension resistance of a bolt, see Table 3.4;
$\sum F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the T-stub;
$\sum \ell_{eff,1}$ is the value of $\sum \ell_{eff}$ for mode 1;
$\sum \ell_{eff,2}$ is the value of $\sum \ell_{eff}$ for mode 2;
e_{min}, m and t_f are as indicated in Figure 6.2.
$f_{y,bp}$ is the yield strength of the backing plates;
t_{bp} is the thickness of the backing plates;
$e_w = d_w / 4$;
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant.



NOTE 1: In bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop.

NOTE 2: In method 2, the force applied to the T-stub flange by a bolt is assumed to be uniformly distributed under the washer, the bolt head or the nut, as appropriate, see figure, instead of concentrated at the centre-line of the bolt. This assumption leads to a higher value for mode 1, but leaves the values for $F_{T,1-2,Rd}$ and modes 2 and 3 unchanged.

Fig 37: Design resistance of a T-stub flange (Tab. 6.2, EN1993:1-8)

It’s possible to investigate better the behaviour of the equivalent T-stub, as shown in [Tartaglia et al., 2020]. The dependency of failure mode on the mechanical and geometrical features of the T-stub can be expressed in function of the parameters β and η, explained in the following:

- ratio between Failure Mode 1 and Failure Mode 3:

$$\beta = \frac{4M_{pl,1,Rd}}{m\Sigma F_{t,Rd}} = \frac{F_{T,1,Rd}}{F_{T,3,Rd}}$$

- ratio between the resistance of T-stub and the Failure Mode 3:

$$\eta = \frac{F_{T,Rd}}{\Sigma F_{t,Rd}} = \frac{F_{T,Rd}}{F_{T,3,Rd}}$$

- ratio between the edge distance and the distance bolt axis-first plastic hinge:

$$\nu = \frac{e}{m}$$

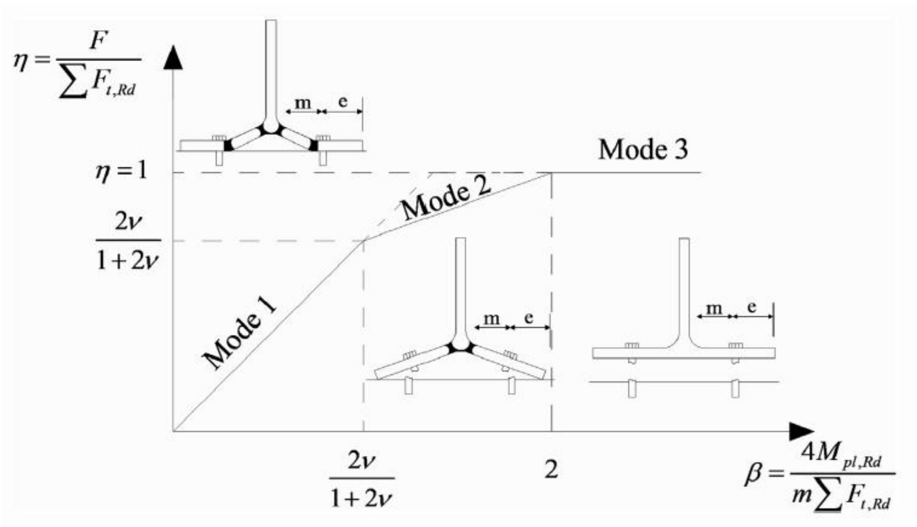


Fig 38: T-stub resistance and corresponding mechanism according to EN1993:1-8

The resistance, represented by a vertical line with abscissa equal to β, meets the failure domain in correspondence of the expected mode of failure. In such way this diagram shows how close the failure is to the other modes of failure. It’s considered acceptable (read ductile) a Failure Mode 1 or a Failure Mode 2 with $\beta \leq 1$.

10.5. T-stub current version

An initial assessment of the behaviour of the as-is T-stub has been carried on by using the equivalent T-stub method, as given in EN1993:1-8 and in the SCI-P398.

Several yielding line patterns are taken into account; in particular the T-stub flange can be modelled as “case (a): Pair of bolts in an unstiffened end plate extension”.

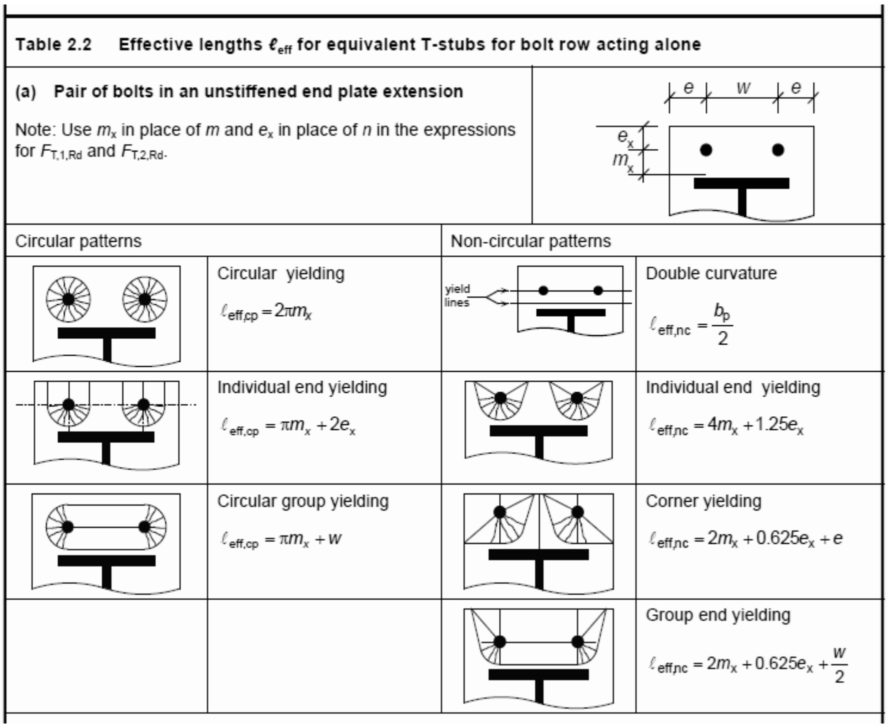


Table 2.2 Effective lengths ℓ_{eff} for equivalent T-stubs for bolt row acting alone			
(a) Pair of bolts in an unstiffened end plate extension Note: Use m_x in place of m and e_x in place of n in the expressions for $F_{T,1,Rd}$ and $F_{T,2,Rd}$.			
Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff,cp} = 2\pi m_x$		Double curvature $\ell_{eff,nc} = \frac{b_p}{2}$
	Individual end yielding $\ell_{eff,cp} = \pi m_x + 2e_x$		Individual end yielding $\ell_{eff,nc} = 4m_x + 1.25e_x$
	Circular group yielding $\ell_{eff,cp} = \pi m_x + w$		Corner yielding $\ell_{eff,nc} = 2m_x + 0.625e_x + e$
			Group end yielding $\ell_{eff,nc} = 2m_x + 0.625e_x + \frac{w}{2}$

Fig 39: Effective lengths for equivalent T-stub

While the column flange can be seen as "case (b): pair of bolts at end of column or on a stiffened end plate extension" (side yielding only) and as "case (c): pair of bolts in a column flange below a stiffener (or cap plate) or in an end plate below the beam flange".

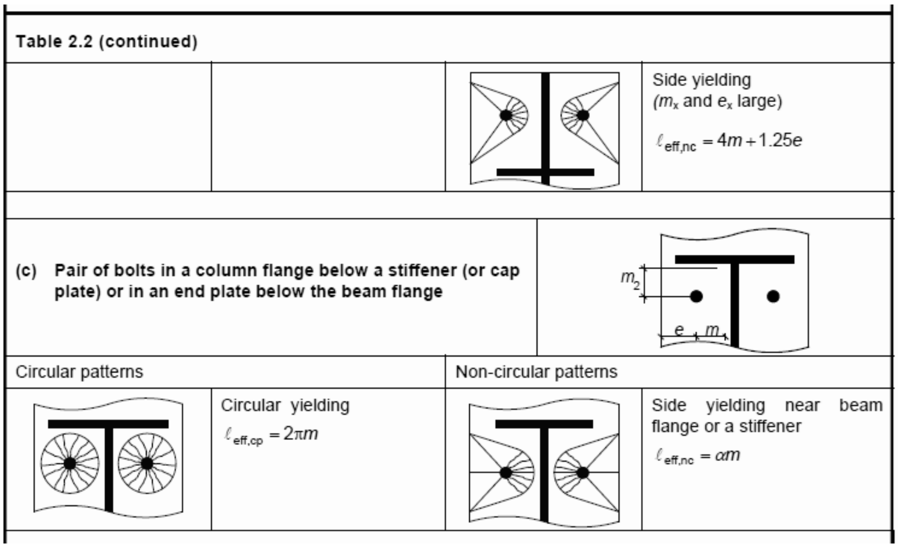


Table 2.2 (continued)			
			Side yielding (m_x and e_x large) $\ell_{eff,nc} = 4m + 1.25e$
(c) Pair of bolts in a column flange below a stiffener (or cap plate) or in an end plate below the beam flange			
Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff,cp} = 2\pi m$		Side yielding near beam flange or a stiffener $\ell_{eff,nc} = \alpha m$

Fig 40: Effective lengths for equivalent T-stub

The four bolts acting in combination have been considered, but this type of failure has a greater resistance with respect the single bolt rows, which means that at the ultimate state bolt rows work alone.

It is therefore possible to evaluate the effective lengths for Mode 1 and Mode 2, then the bending moment plastic resistance for Mode 1 and Mode 2, the design resistance for Modes 1, 2, 3 and finally the design resistance $F_{T,Rd}$ of the equivalent T-stub.

As the diagram $\eta - \beta$ below clearly show, all the four connections investigated are subject to brittle failure: the weakest component, namely the T-stub flange, undergoes to a Failure Mode 2, but it's too close to a Failure Mode 3 ($\beta > 1$).

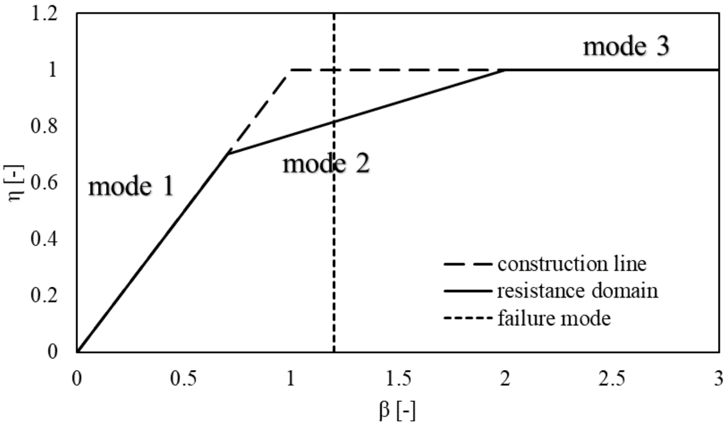


Fig 41: Resistance domain for T-stub flange

The diagram $\eta - \beta$ below is referred to the column flange

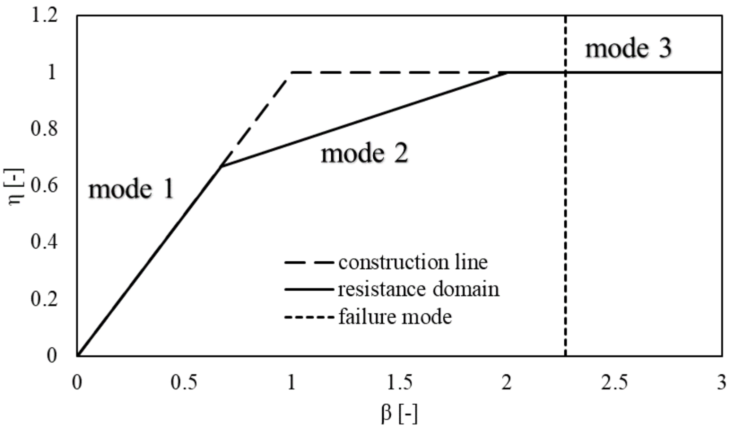


Fig 42: Resistance domain for column flange

Since failure is always due to the T-stub flange, a redesign targeted to this part of the connection is reasonable. On the other hand, column flange undergoes to a Failure Mode 3, but its bigger resistance means that this failure isn't achieved in reality, since the entire connection fails for a lower action due to the failure of the T-stub flange.

10.6. T-stub proposed version

10.6.1. Limitations

To ensure that the T-stub works as designed, several limitations must be taken into account:

- Maximum diameter of the bolt of the T-stub flange: $d_{max} = 30\ mm$, which means bolts not greater than M30;
- Bolts grade not higher than Grade 10.9, since stronger bolts (such as Grade 12.9) are more expensive and not so common;
- Overall dimensions of the T-stub flange: not larger than the column flange, not too high;
- Thickness of the T-stub flange and T-stub web: less than 35 mm due to manufacturing reasons;
- Distance between column flange and beam can't be modified, since the T-stub web close to the T-stub flange is designed to host a plastic hinge and by modifying this distance there is no clue on the effective result: there's the risk of compromising the behaviour of the FREEDAM joint;
- General geometrical limitations for bolted connections, such as edge distances, spacing between bolts…
- It was chosen to strengthen the joint based on the Device, for the love of the uniformity and consistency. As result all the Devices 1 will be modified in Devices 1-MOD, while the Devices 2A will be modified in Devices 2A-MOD. This process is done by strengthening the Device with regards the severest action and then applying these modifications to the other joints.

10.6.2. Design process

In order to enhance the prying action of the T-stub flange, the width of this plate is augmented up to the width of the column flange.

After calculation, it was observed that the as-is bolt configuration (namely 2x2 bolt grid) was not sufficient for the design action unless using M36 bolts, which as stated before are not an option.

A new bolt configuration was investigated, a 4x2 bolt grid, compensating to the lack of tensile resistance of the bolts by increasing their number. This new configuration is not considered in the Codes: in particular the outer bolts, with no transversal stiffener, are unusual to the common MRF connections. In addition, it was not possible to place a vertical stiffener on the T-stub flange, since it would hinder its designed functioning: a stiffener would lower the bending capacity of the T-stub.

As usual, effective length are evaluated for all the bolt rows, with respect circular yielding pattern and non-circular yielding pattern. For all the considerations stated above, the 4x2 bolt configuration is calculated with a blended method:

- **Bolt row 1** is evaluated according to the equivalent T-stub method, as row acting alone;

  $l_{eff,cp}^{BR1,alone}$ $\quad$ $l_{eff,nc}^{BR1,alone}$

- **Bolt row 2** is evaluated according to the equivalent T-stub method, as row acting in combination;

  $l_{eff,cp}^{BR2,group}$ $\quad$ $l_{eff,nc}^{BR2,group}$

- **Bolt row 1+2**, namely the two bolt rows acting in combination, are evaluated thanks to the *Demonceau method*, while the overall effective length is given by the sum of the effective length of the bolt row 1 acting alone and the effective length of the bolt row 2 acting in group:

$$\sum l_{eff,cp}^{BR(1+2)} = l_{eff,cp}^{BR1,alone} + l_{eff,cp}^{BR2,group}$$

$$\sum l_{eff,nc}^{BR(1+2)} = l_{eff,nc}^{BR1,alone} + l_{eff,nc}^{BR2,group}$$

This hypothesis was verified in detail with a FE model of the T-stub alone, which positive results.

Demonceau proposed an alternative formula to evaluate the Mode 2 failure of a 4-bolts T-stub, while the formulae for Mode 1 and Mode 3 are the same of the 2-bolts T-stub:

Tab 6: T-stub design resistances according to Demonceau et al.

Failure modes	T-stub with 2 bolts	T-stub with 4 bolts
Mode 1	$F_{Rd,1} = \dfrac{(8n-2e_w)M_{pl,1,Rd}}{2mn-e_w(m+n)}$	$F_{Rd,1} = \dfrac{(8n-2e_w)M_{pl,1,Rd}}{2mn-e_w(m+n)}$
Mode 2	$F_{Rd,2} = \dfrac{2M_{pl,2,Rd}+n\sum B_{t,Rd}}{m+n}$	$F_{Rd,2} = \min(F_{Rd,2,p};F_{Rd,2,np})$ with $F_{Rd,2,p} = \dfrac{2M_{pl,2,Rd}+\dfrac{\sum B_{t,Rd}}{2}.(\dfrac{n_1^2+2n_2^2+2n_1n_2}{n_1+n_2})}{(m+n_1+n_2)}$ $F_{Rd,2,np} = \dfrac{2M_{pl,1,Rd}+\dfrac{\sum B_{t,Rd}}{2}.n_1}{(m+n_1)}$
Mode 3	$F_{Rd,3} = \sum B_{t,Rd}$	$F_{Rd,3} = \sum B_{t,Rd}$ (but limited in practice to $0{,}9\sum B_{t,Rd}$ ([6] & [7])

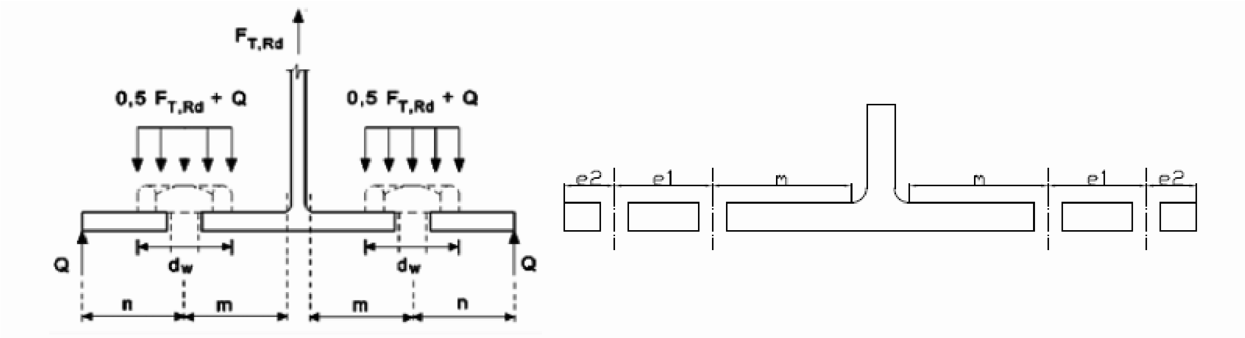


Fig 43: 4-bolts T-stub

In addition to the “classical” length m and edge distance n (called e_2 in this study), new lengths are introduced:

- spacing e_1 between bolts of the same flange side
- length $n_1 = \min(e_1 + e_2; 1.25m)$
- length $n_2 = \min(e_2; 1.25m + n_1)$

As it can be noticed, Failure Mode 1 and 3 proposed by Demonceau are essentially those indicated by EC1993:1-8, while Failure Mode 2 is different. This last Failure Mode is modified in order to take into account the new geometrical features of a 4-bolt T-stub.

10.6.3. Proposed version of the T-stub

These modifications refer to the T-stub of Device 1, with these changes:

- Flange:
  - Width, 300 mm instead 205 mm
  - Height, 500 mm instead 170 mm
- Web:
  - Width, 300 mm instead 205 mm
  - Length, 440 mm instead 345 mm
- Flange bolts:
  - 8 M27 bolts instead 4 M24 bolts
- Web bolts:
  - M20 instead M16 (same quantity)
- <u>Flange and web thickness has not been modified</u>

10.6.4. Results and comparisons

FE simulations proved the proposed version of the T-stub to be successful in guarantee a ductile failure mode. In fact, with the new T-stub design it was possible to move the vertical dotted lines identifying the failure mode to the left of the $\eta - \beta$ diagram, both for the T-stub flange and for the column. The overall failure of the T-stub flange to column flange connection is still governed by the T-stub flange, but now this crisis happens in Failure Mode 1. T-stub of the column side is now in Failure Mode 2.

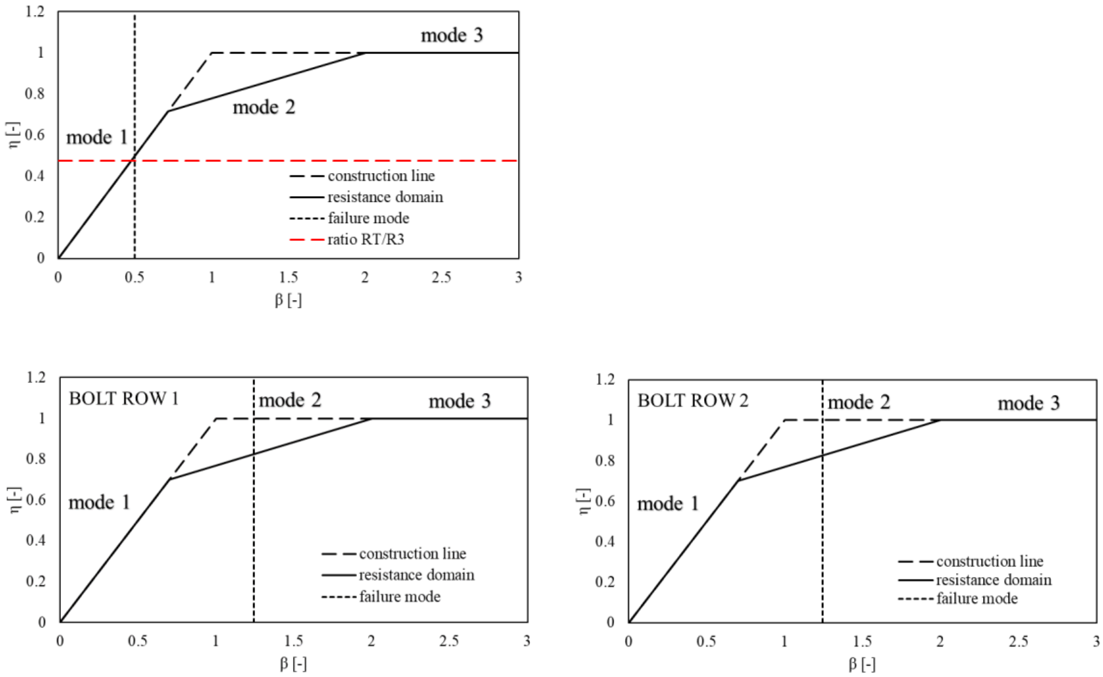


Fig 44: joint X-II: T-stub flange behaviour (up-left), column side bolt row 1 (bottom-left), column side bolt row 2 (bottom-right)

11. FE analyses of joints with modified T-stub

The following modified joint is the X-II-XJ, equipped with a FREEDAM Device 1.

11.1. Monotonic Loading Scenario

Bending moment-rotation diagram shows that the newly designed joint has the same seismic behaviour of the previous version, this means that all the considerations made in FREEDAM campaign for the joint behaviour are still valid.

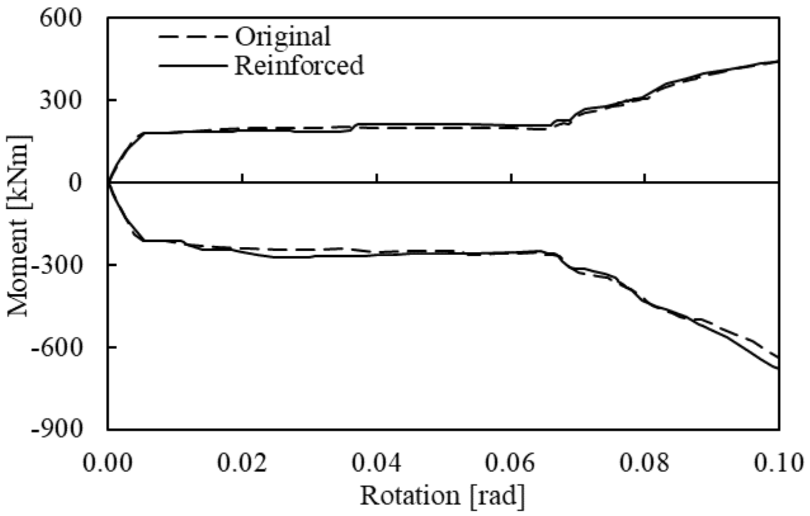


Fig 45: Bending moment-rotation diagrams (Hogging and Sagging)

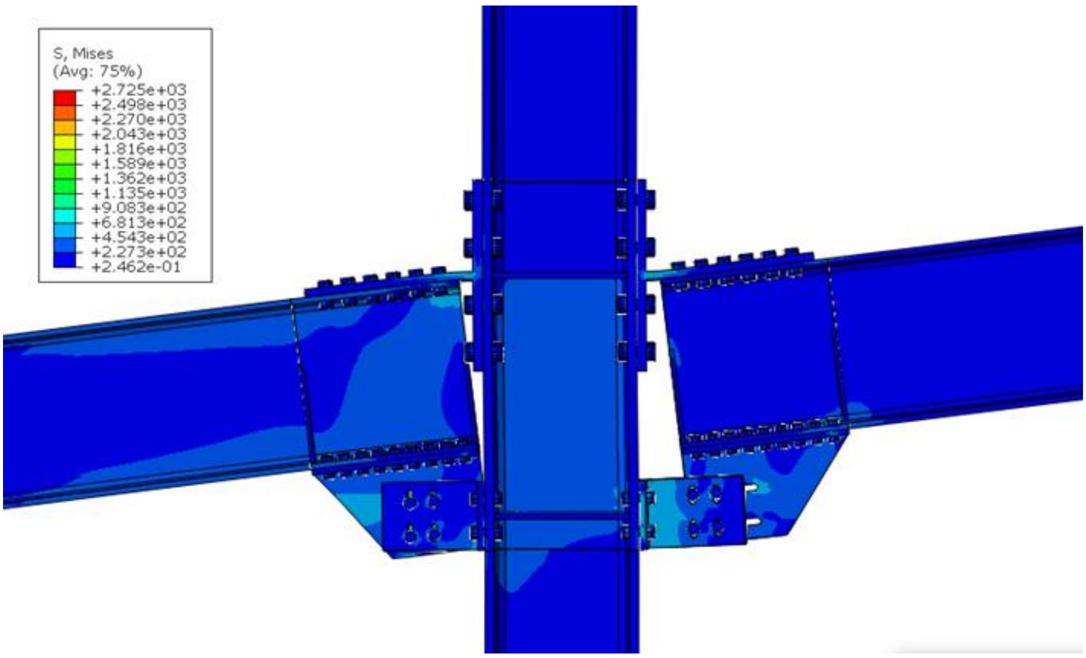


Fig 46: Von Mises stress distribution

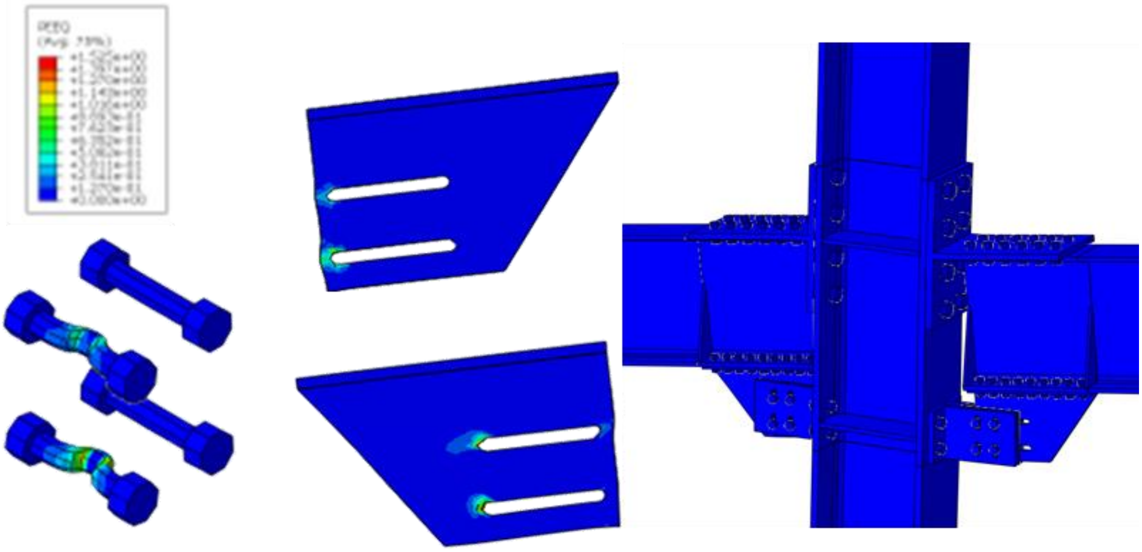


Fig 47: PEEQ distribution

11.2. Column Loss "Hogging" Scenario

These two diagrams show the great advantages of the proposed modification. By simply changing some geometrical parameters of the T-stub, the joint now reaches 0,25 rad of chord rotation. The catenary action can now fully develop and it can almost reach the plastic resistance of the beam.

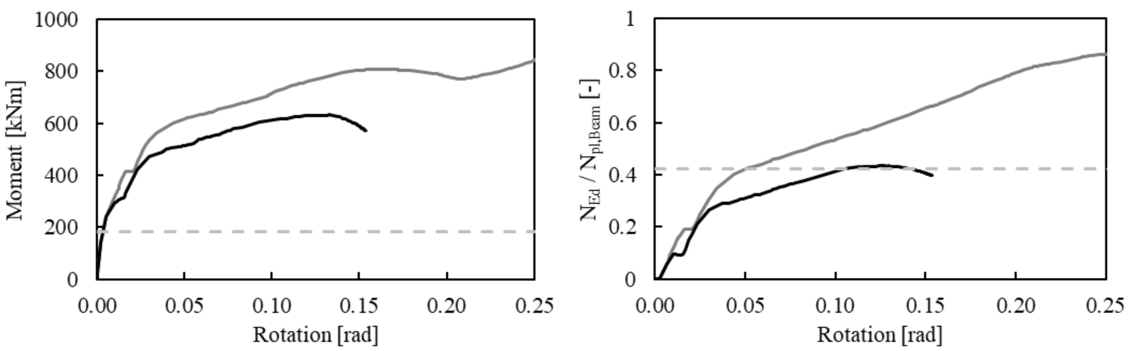


Fig 48: Bending moment – rotation (on the left) diagram and catenary axial load of the X-II-XJ-MOD the previous joint (in black) is confronted with the new version (in grey)

The Von Mises and PEEQ distribution confirm the hypotheses made during the design phase: the load acting on the T-stub is mainly resisted by the bolts nearby the web, then the outer rows share a portion of the external action. PEEQ are concentrated in the bolts next to the T-stub web.

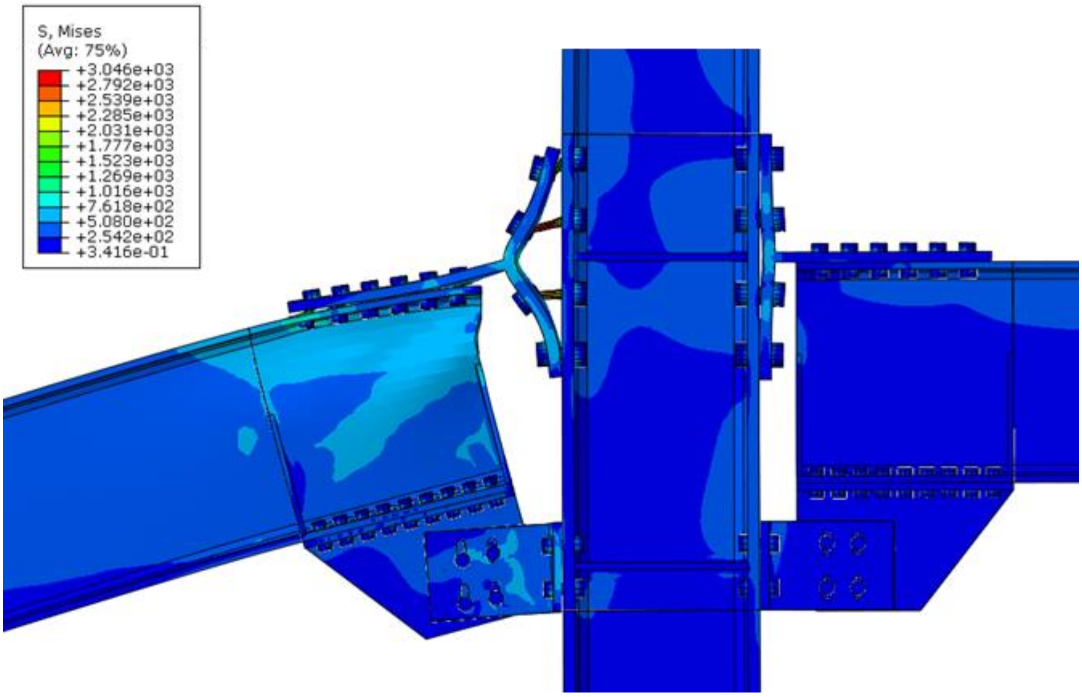


Fig 49: Von Mises stress distribution

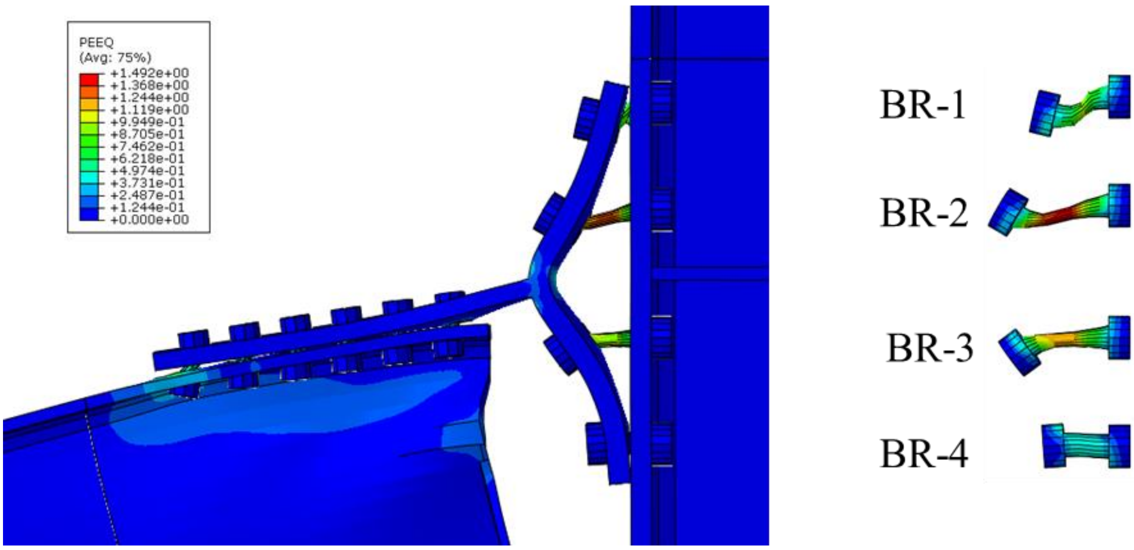


Fig 50: 4-bolts T-stub detail under Column Loss "Hogging"

11.3. Column Loss "Sagging" Scenario

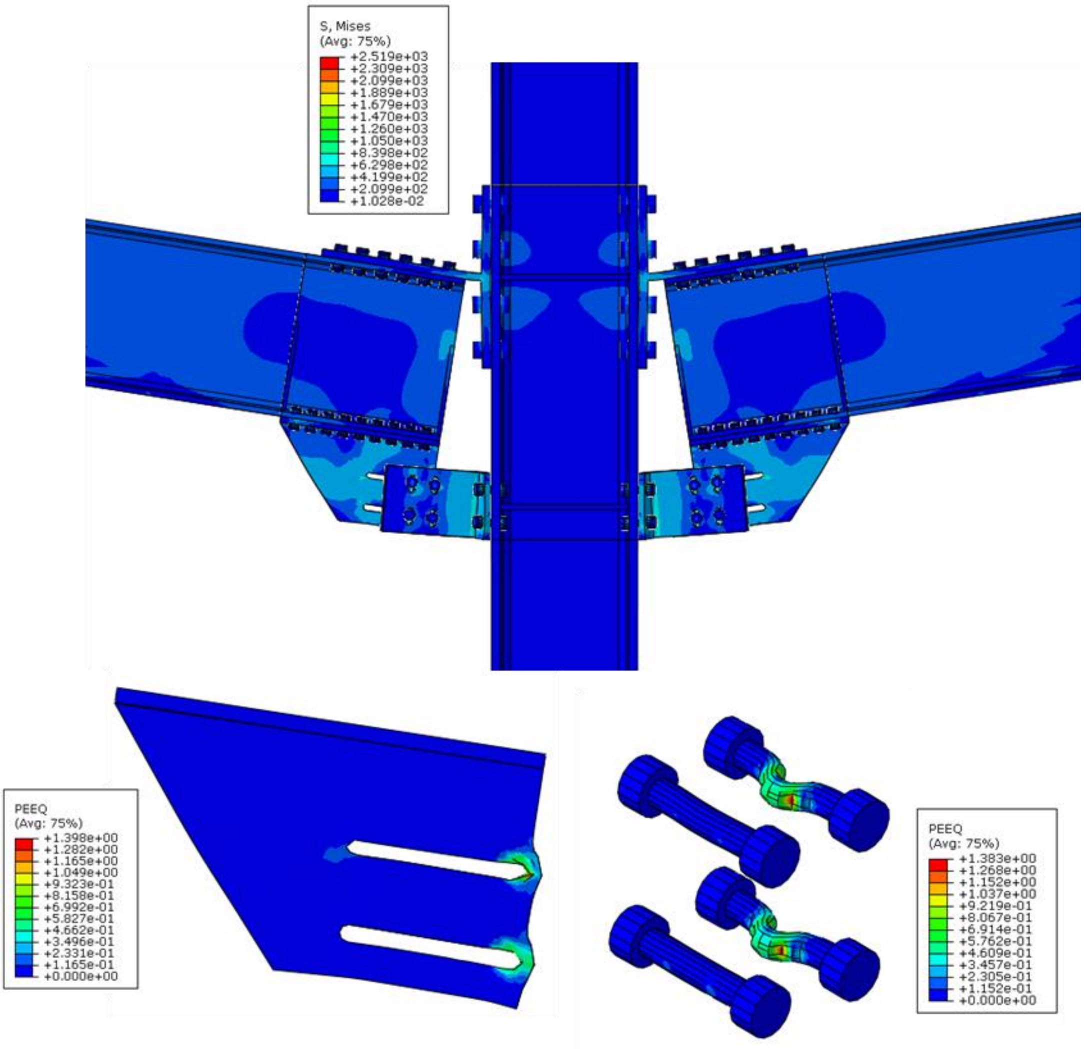


Fig 51: Von Mises stress distribution (top); PEEQ distribution on the haunch and damper bolts (bottom)

12. Analysis of joint Y-II-XJ

This joint has a X-configuration, it's localized at the second floor of the Y-MRF, it's equipped with the FREEDAM Device 2A. FE analyses are carried out for this joint, in the same way seen for the X-II-XJ. These FE analyses, briefly introduced here, show that also this joint cannot resist a Column Loss Hogging Scenario. For this reason, an upper T-stub re-design is made.

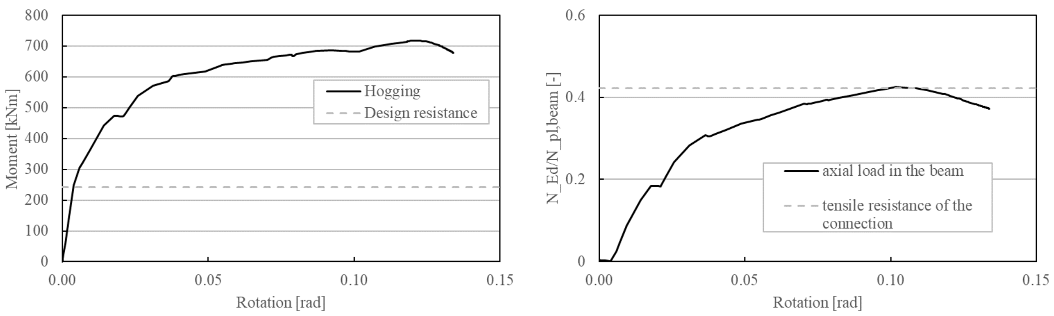


Fig 52: Hogging bending moment - rotation (left), non-dimensional axial load - rotation (right)

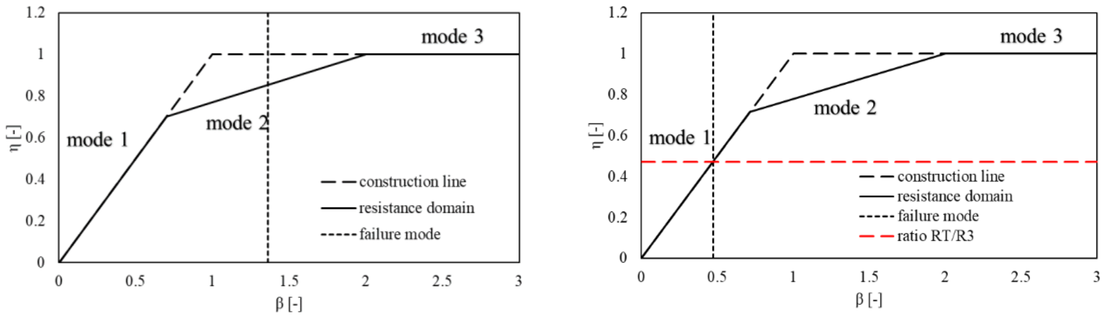


Fig 53: T-stub flange resistance domain: previous design (left) and proposed design (right)

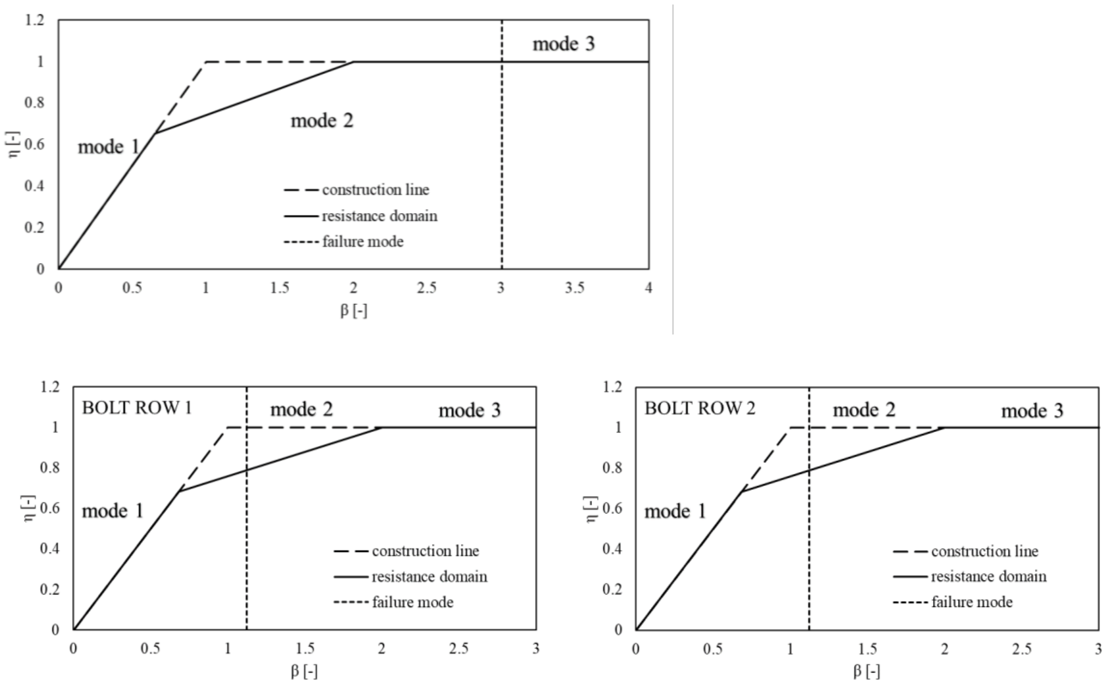


Fig 54: Column equivalent T-stub resistance domain: previous design (top) and proposed design (bottom)

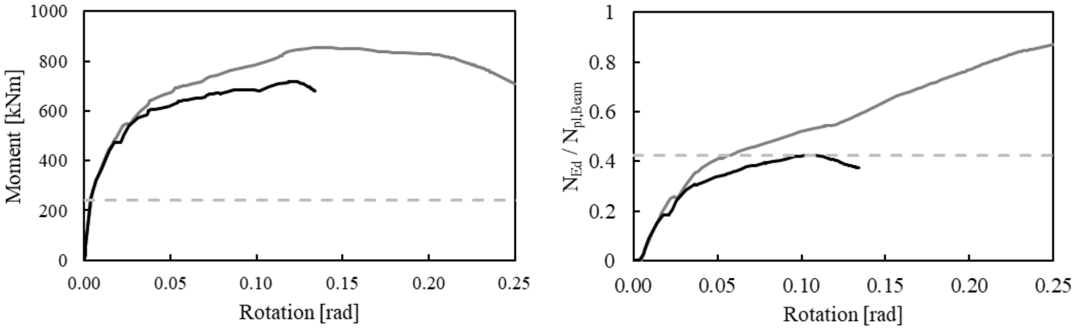


Fig 55: Bending moment – rotation diagram (on the left) and catenary axial load on the right. Previous design Y-II-XJ (in black) and proposed design Y-II-XJ-MOD (in grey)

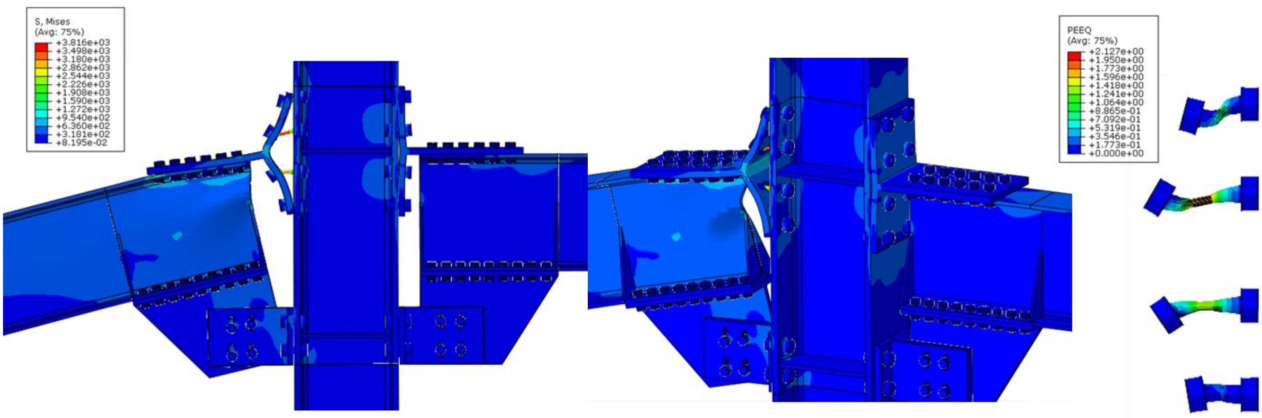


Fig 56: Von Mises stress distribution (left) and PEEQ distribution (right)

13. Conclusions

As it was expected, FREEDAM joints exhibit an excellent performance under seismic loading (both monotonic and cyclic). However, these types of connection are not enough to resist a Column Loss Scenario, resulting in a brittle Failure Mode located at the upper T-stub (in hogging column loss) or at the haunch and the damper bolts (in sagging column loss)

The proposed modifications ensure a proper level of robustness in case of hogging column loss: rotational capacity of the joint is enhanced up to 25% while upper T-stub Failure Mode is now sufficiently ductile.

<u>These considerations can be extended to the other joints of the DREAMERS building</u>

The proposed modifications, albeit localized, can guarantee an important increase also in terms of resistance for the FREEDAM connections, providing a ductile failure mode, in case of Column Loss Scenario, with beam subjected to Hogging bending moment. The simplicity of the intervention makes it very useful and feasible.

It has been shown that the modified T-stub doesn't interfere with the seismic behaviour of the joint, in such way all the previous studies made for the FREEDAM connections are still valid. In terms of behaviour, X-II-XJ and X-II-XJ-MOD behave in the same manner.

Few things between the actual version and the proposed version have changed, basically the geometrical dimensions of the plates, the number and the diameter of the bolts.

Here is a comparison between Device 1-MOD and Device 2A-MOD:

- Design tensile action depends by the beam profile: $N_{Ed} = 0.4N_{pl,beam}$, where the beam profiles are IPE 400 and IPE 450;
- With respect to the column flange to T-stub flange connection: same column profile, same plate thickness, same bolt diameter. In particular, the T-stubs of Device 1 and Device 2A differs on the width and height of the flange;
- With respect to the beam flange to T-stub web: plate width, height, thickness differs between Device 1 and Device 2A,

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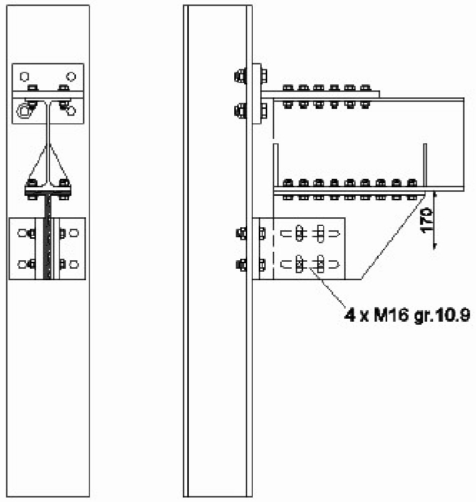
List of figures

ANNEX A

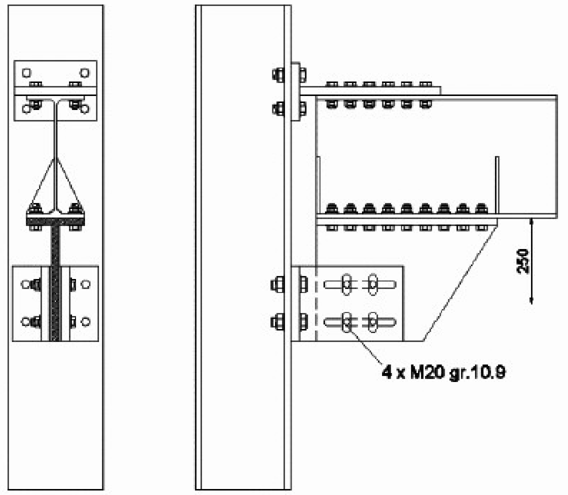
DEVICE D1

Beam range: IPE 270 - IPE 450



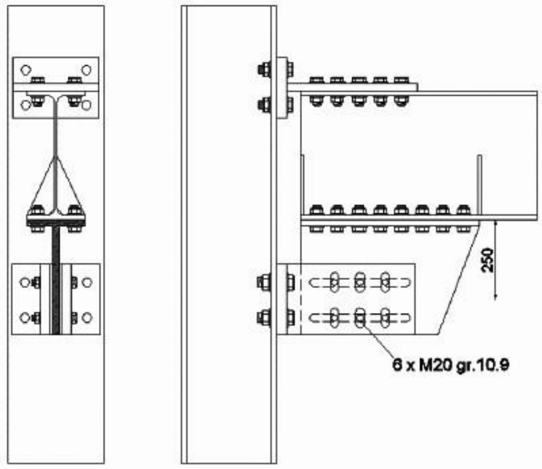
DEVICE D2

Beam range: IPE 360 - IPE 600



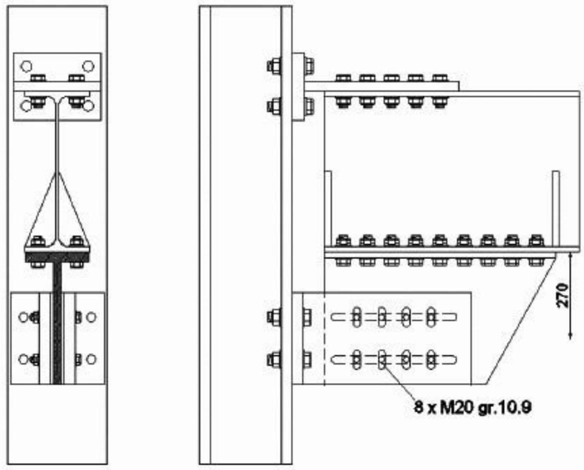
DEVICE D3

Beam range: IPE 400 - IPE 750x173



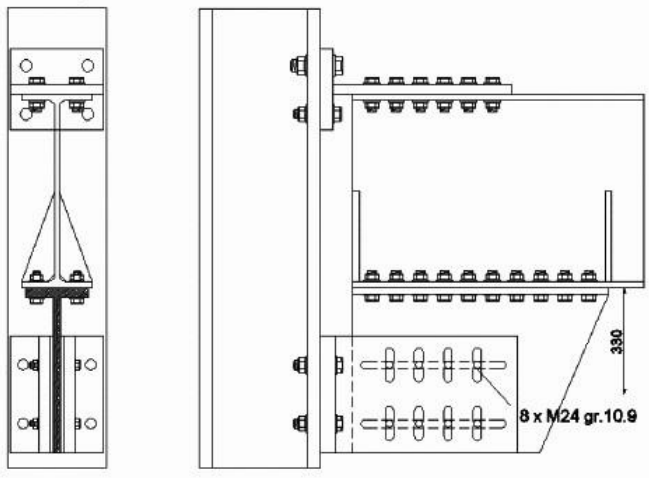
DEVICE D4

Beam range: IPE 500 - IPE 750x185



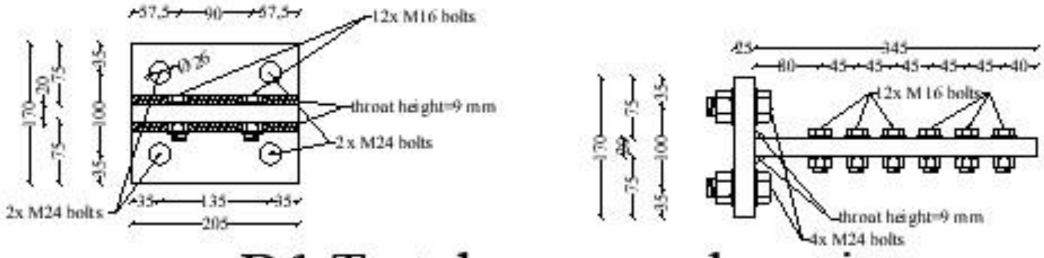
DEVICE D5

Beam range: IPE 750x147 - IPE 750x185

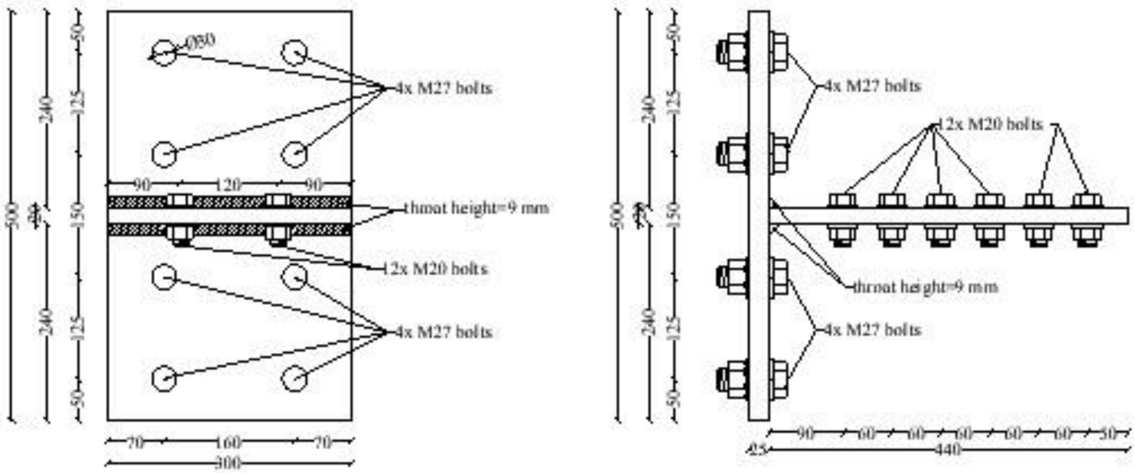


ANNEX B

D1 T-stub current version

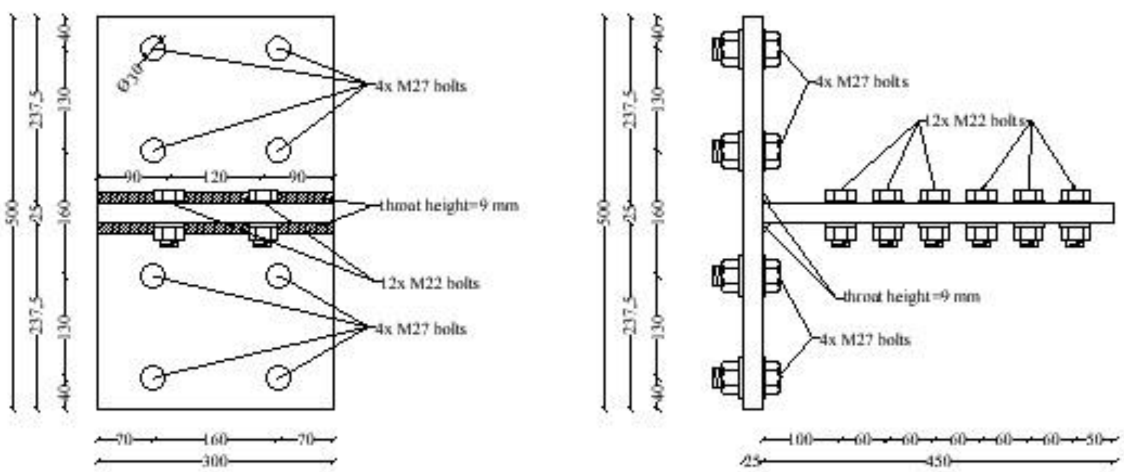


D1 T-stub proposed version



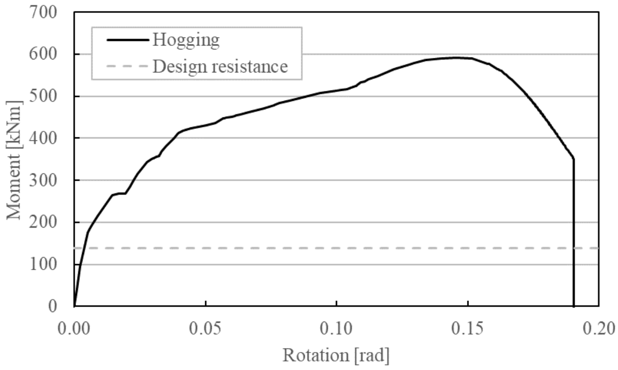
D2A T-stub current version

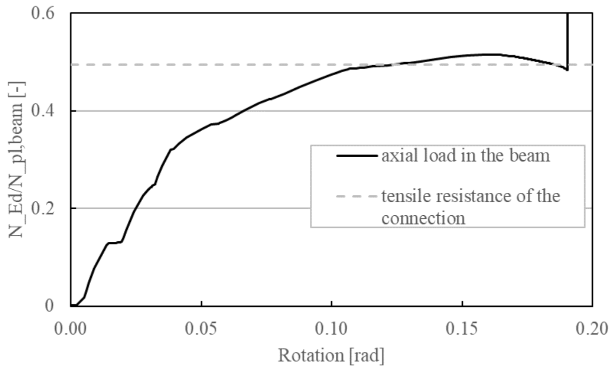
D2A T-stub proposed version

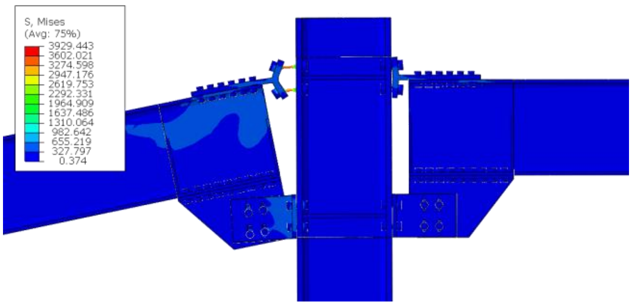


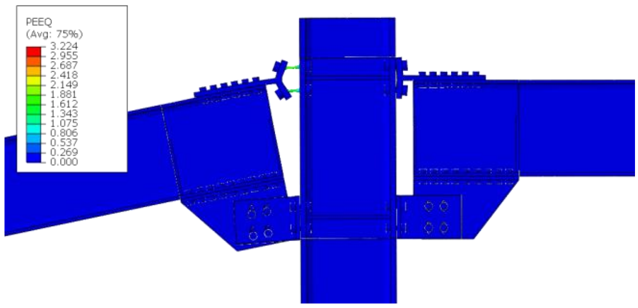
ANNEX C

a) X-III-XJ – “Hogging” Column Loss

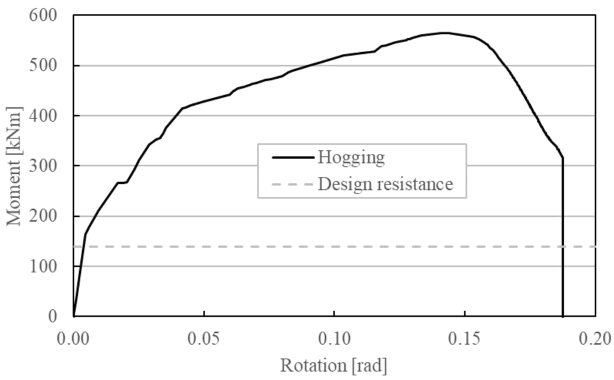


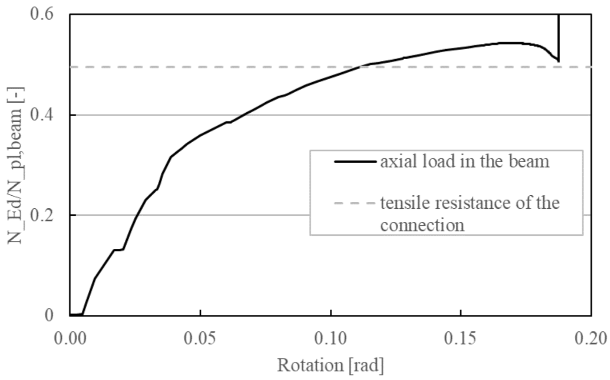


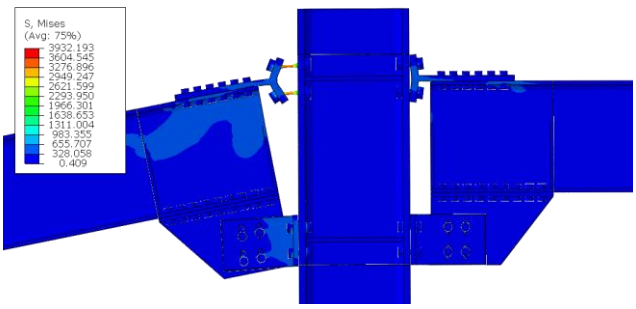


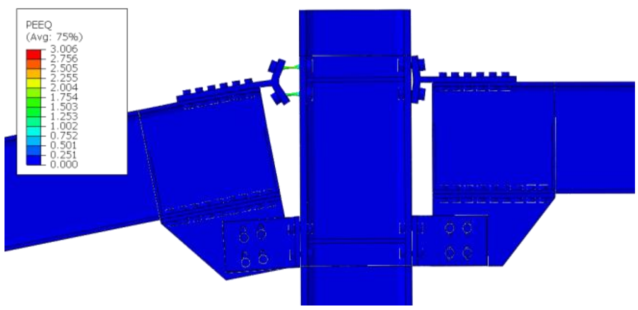


b) Y-III-XJ – “Hogging” Column Loss

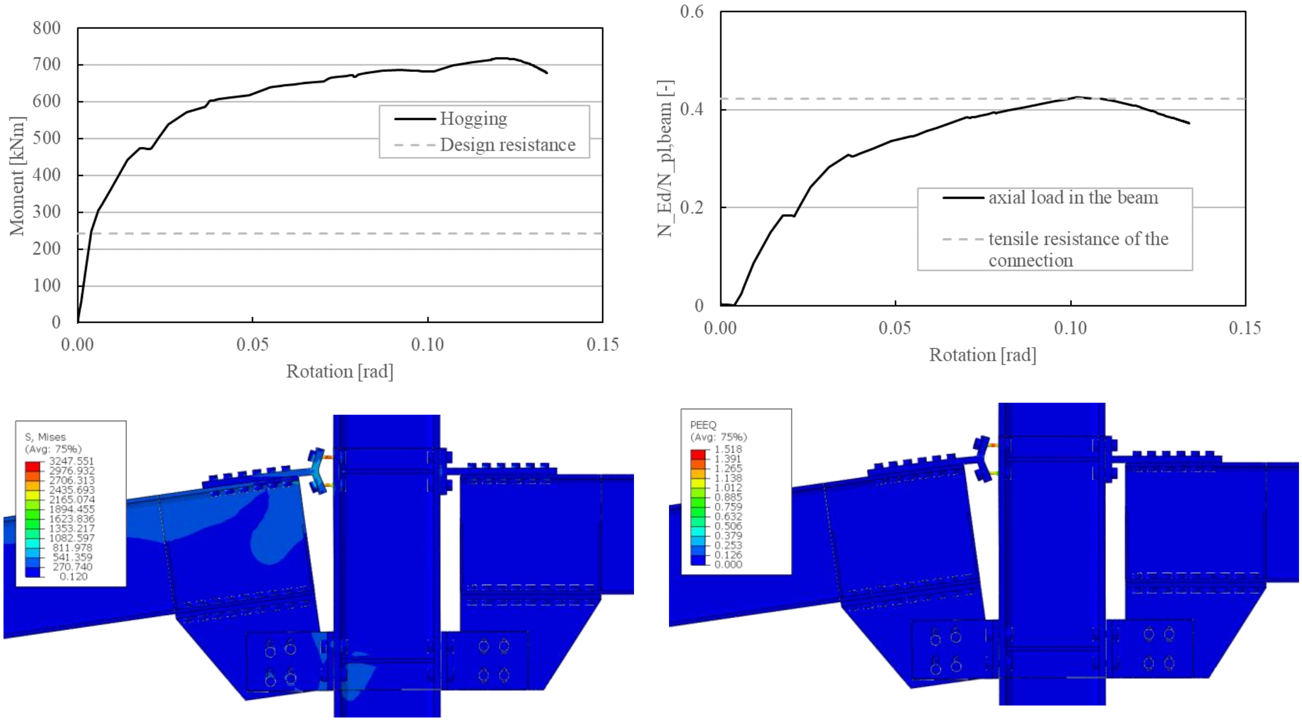




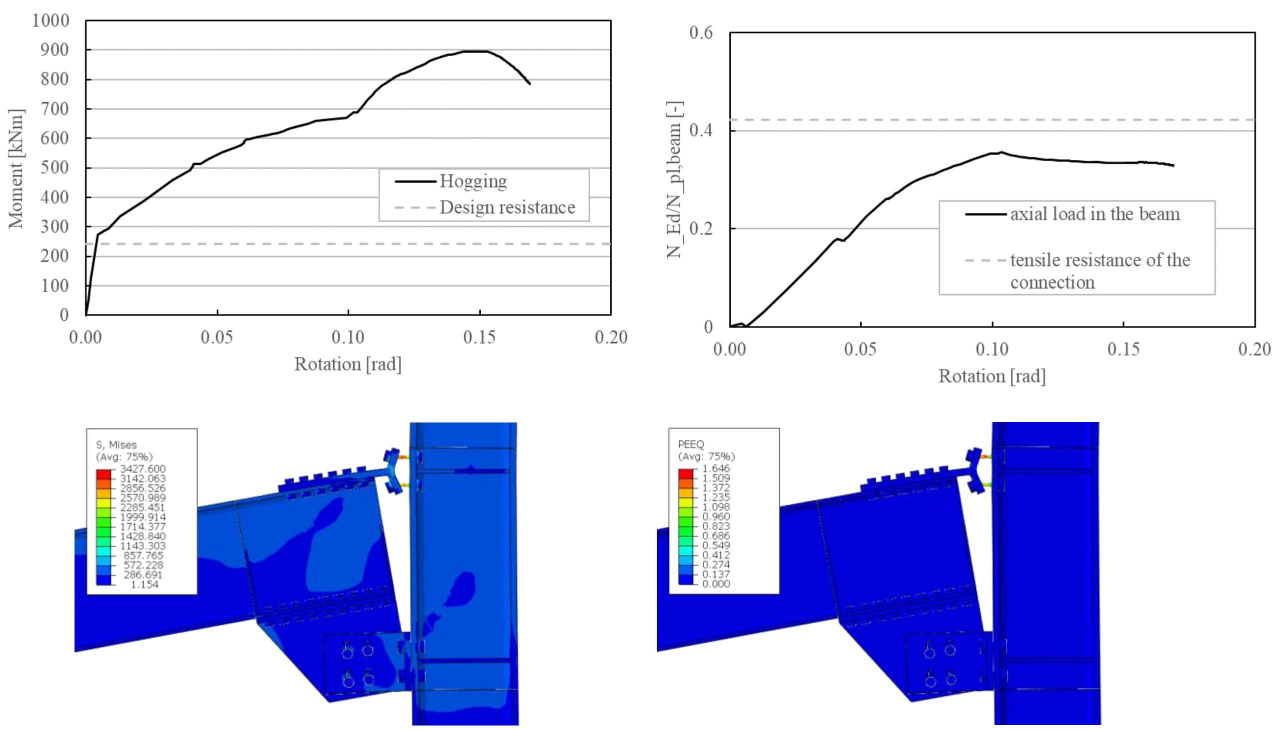




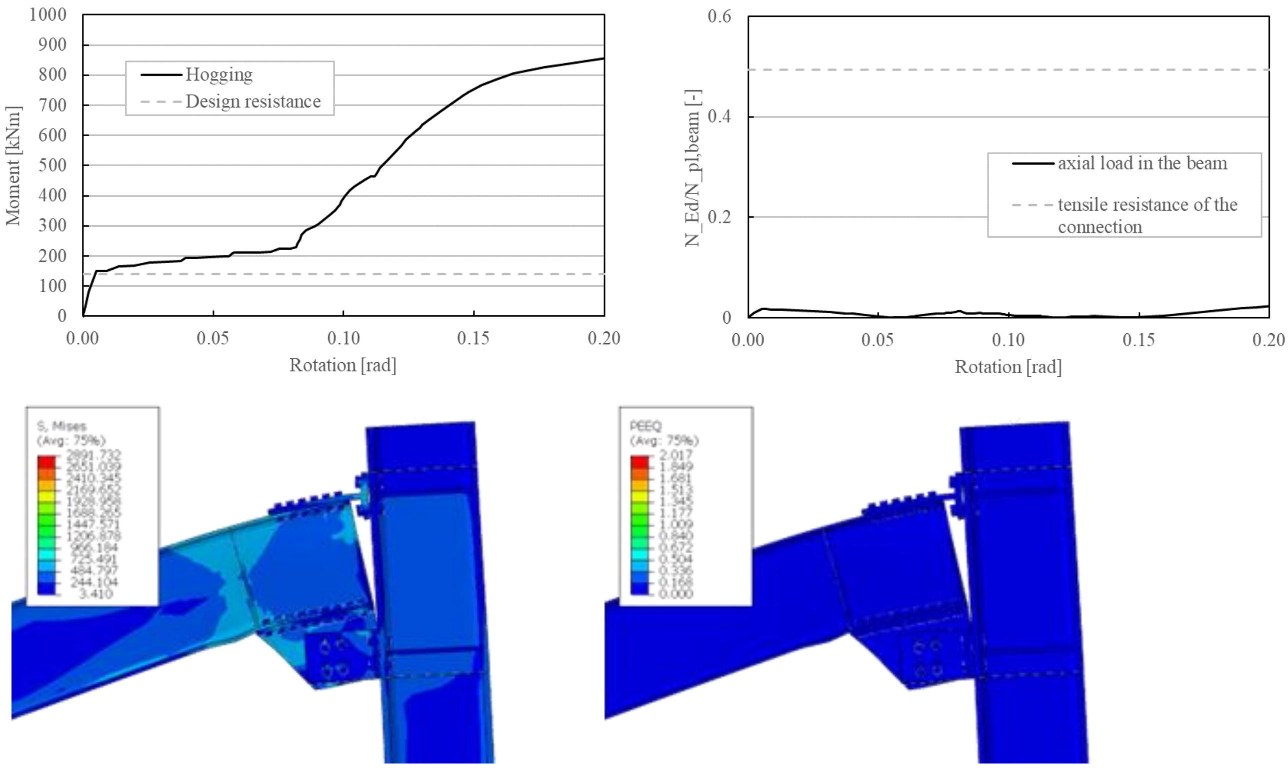
c) Y-II-XJ – "Hogging" Column Loss



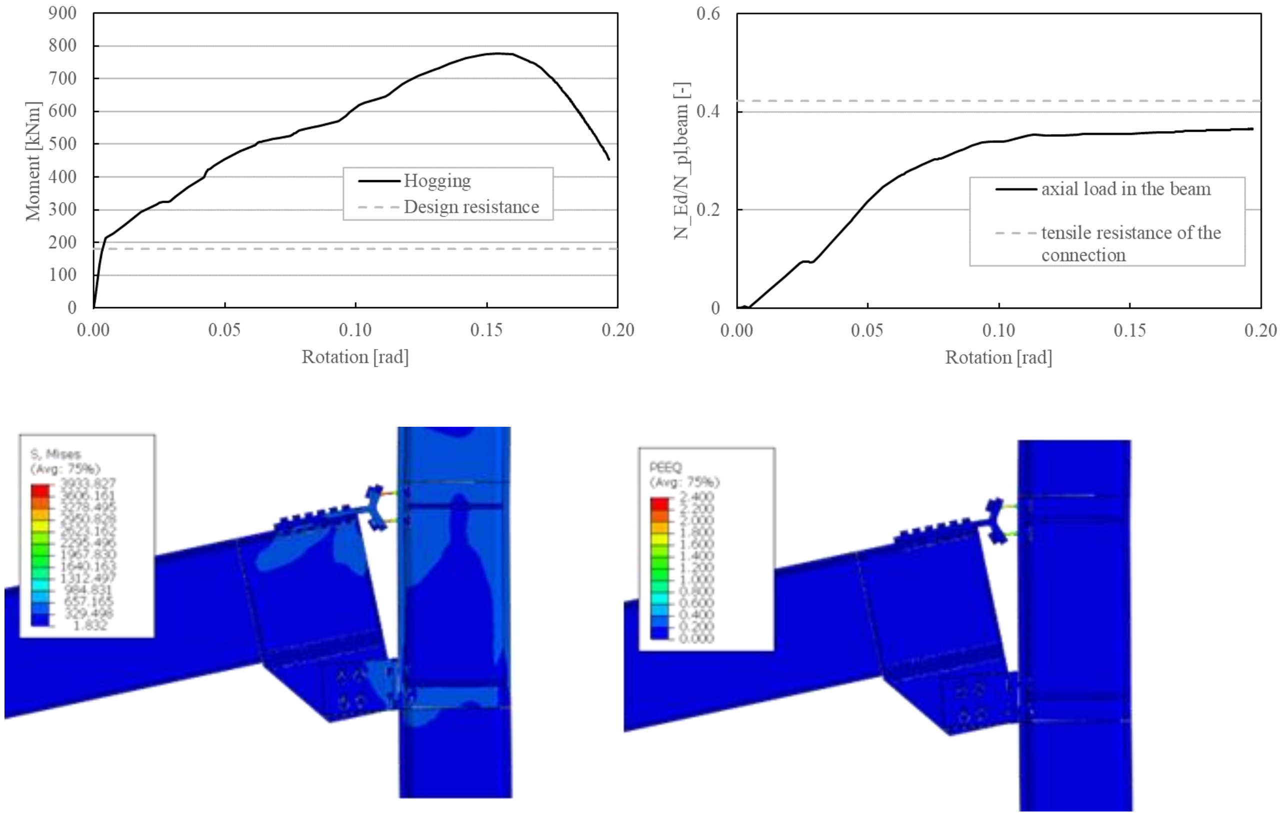
d) Y-II-TJ – "Hogging" Column Loss



e) X-III-TJ – “Hogging” Column Loss



f) X-II-TJ – “Hogging” Column Loss



g) Y-III-TJ – “Hogging” Column Loss