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BEHAVIOUR OF BEAM-TO-COLUMN JOINTS UNDER LARGE VERTICAL DISPLACEMENTS FOLLOWING THE LOSS OF A COLUMN

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Abstract

In past years terrorist attacks became more frequent especially in developed countries and the consequencess got more devastating. The aim of this disertation is to give better understanding of mechanisms which could improove overal safety in buildings, in means of preventing progressive collapse and preclude disasters. This so called robustness assessment gains attention also because the current standards do not include sufficient provisions for the cases of accidental loading and do not track the current trends in structure design, such a use of nescessary software.

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The thesis examines the catenary effect and role of connections by means of numerical simulations which precede planned experimental tests. The document will present the framework of the theme and the modeling process starting with its validation and finnishing with expected results and conclusions.

Content

1 Introduction, motivation of the subject

1.1 Failures due to accidental actions - progressive collapse

In 1968 a 22-storey block of flats called Ronan Point in east London partly collapsed due to a gas explosion. The gas explosion from a kitchen in the corner of 18th floor and led to loss of a bearing walls which caused the collapse of the floors above and than the collapse spreaded downwards as the floors were overstressed from the falling derbis. Four people were killed and 17 were injured. The block was build using Large Panel System and it was observed that the connections of the panels were insufficient. This was a milestone for engineers to think about unexpected loads which can occur durning the lifetime of a building. Because of that changes were consequently made in building regulations of several countries.

Figure 1 Ronan Point

In 1995 was identified progressive collapse of The Alfred P. Murrah Federal Building in Oklahoma caused by terrorist bombing. The explosion destroyed the three main columns and all the nine stories above fell down. 168 people were killed and over 800 others were injured. According to FEMA 277 if Special Moment Frame Detailing would have been used (spiral reinforcement in the columns for shear, continuity reinforcement in the transfer girder, which is very similar what is required for seismic design), two of the lost columns would have not been destroyed, and three other scenarios would have happend. The estimated losses would have been 50-85% reduced.

Figure 2 Murrah building collapse

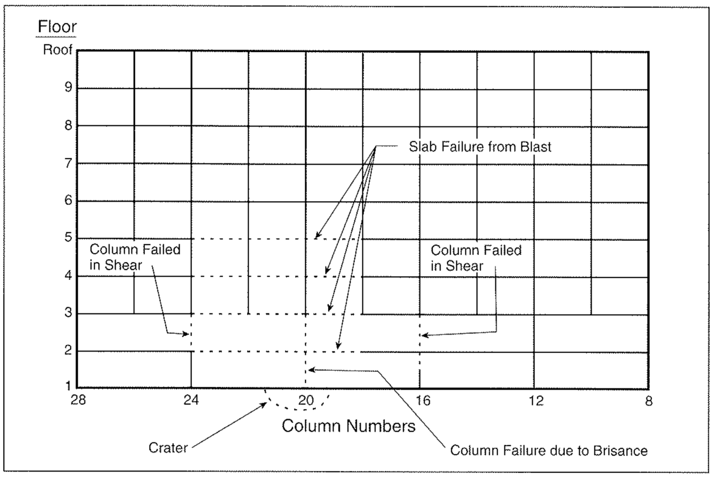


Figure 3 Damage of Murrah building directly after explosion

11th September 2001, the impact of airplane to each of the twin towers of the World Trade Center and induced fires caused the complete failure of three buildings of WTC complex and massive loss in lives (2,606 killed) and property. This event will remain in minds of people for many years as one of the biggest catastrophes in the modern history. There were several discussions if the collapse could be prevented or not, nevertheless the mechanism was again identified as progressive collapse.

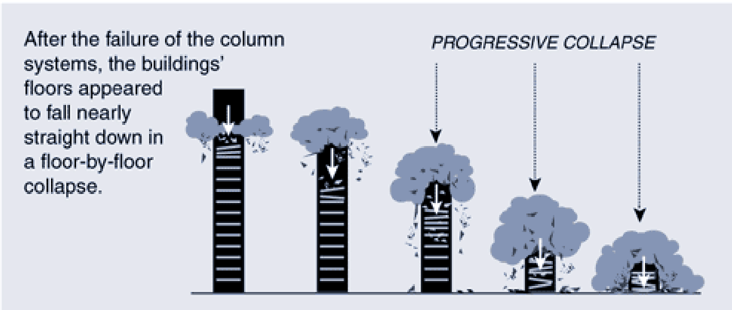


Figure 4

Figure 5 WTC collapse

Figure 6 WTC collapse

If we look just at gas explosions data on http://en.wikipedia.org/wiki/Gas_explosion we can count that after the Ronan Point disaster another major 24 gas accidents happened in which 1400 people died.

Figure 7 gas explosion in Astrakhan 2012

1.2 Basic explanations

For further understanding of the topic a few terms need to be explained

Accidental action is action with very low probability of occurrence, therefore it is not usual (and economic) to take it into account directly in the design. The possible risk can be natural accident such as exceptional wind, snow, flood, landslide, earthquake or man made accident such as impact, explosion, fire, human error and combinations of above. Due to its unpredictability it is very hard to deal with this actions, but once upon a time the disaster comes.

Eurocode package is covering this topic in rather general way, although Eurocode 1991-1-7 determines some cases such as impact and explosion, there is demand from authorities for further development of the standards.

Progressive collapse is a situation when initial local damage or failure of a member of structure triggers the failure of the adjecent members and lately spreads over the whole structure.

Most often we speak about gravity driven progressive collapse in buildings. The failure can be in the bottom of a structure causing loss of support for the upper floors and therefore falling down. Or the failure can spread from top to bottom when after failure the falling top part gets enough kinetic energy to smash the floors underneath, as we saw in case of WTC, so called *pancake collapse.*

Other progressive collapses can spread in horizontal direction like a row of *domino* destroying for example group of a buildings or group of a columns durning construction. Or in case of cable-stayed or suspension bridges, we speak about *zipper collapse* where a hanger is destroyed and the adjecent hangers are failing consequently due to increased load in them.

Structural integrity is ability of the structure to support or supplant damaged members of the structure.

Key element is structural member upon which the stability of the remainder of the structure depends.

Robustness is ability of the structure to withstand accidental actions and to prevent progressive collapse. It is related to strength and ductility of structure members as well as connections between them and finaly to the structural integrity.

Consequence class deteremines how important is to consider accidental loading in design.

	Description	**Examples of buildings and civil engineering works**	**Consideration of accidental loading**
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)	an examination of the specific case should be carried out to determine the level of reliability and the depth of structural analyses required. This may require a risk analysis to be carried out and the use of refined methods such as dynamic analyses, non-linear models and interaction between the load and the structure
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)	depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied
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	no specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in EN 1990 to EN1999, as applicable, are met

Table 1 Consequence classes according to Eurocode

Table 11 Building Classes

Class	**Building Type and Occupancy**
1	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 1.5 times the building height
2A	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 2000m² floor area in each storey Single storey educational buildings All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000m² at each storey
2B	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys Educational buildings greater than 1 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 members of the public are admitted which contain floor areas exceeding 2000m² but less than 5000m² at each storey Car parking not exceeding 6 storeys
3	All buildings defined above as Class 2A and 2B that exceed the limits on area and/or number of storeys Grandstands accommodating more than 5000 spectators Buildings containing hazardous substances and/or processes

Table 2 Consequence classes according to Approved document A 2004

Nature of Occupancy	Category
Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: Agricultural facilities Certain temporary facilities Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: Buildings and other structures where more than 300 people congregate in one area Buildings and other structures with day care facilities with capacity greater than 150 Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities Jails and detention facilities Power generating stations and other public utility facilities not included in Category IV Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the hazardous material does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities including, but not limited to: Hospitals and other health care facilities having surgery or emergency treatment facilities Fire, rescue, ambulance, and police stations and emergency vehicle garages Designated earthquake, hurricane, or other emergency shelters Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response Power generating stations and other public utility facilities required in an emergency Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency Aviation control towers, air traffic control centers, and emergency aircraft hangars Water storage facilities and pump structures required to maintain water pressure for fire suppression Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

Table 3 Consequence classes according to ASCE 7 2002

Catenary action is a mechanism that a member forms when vertical load (P) is resisted by means of tensile internal force. The specific requirements for finding equilibrium are rigid (stiff) horizontal supports at the ends and deflection (Δ) in the middle. The bigger the deflection the smaller axial force (T) needed to resist the load. This mechanism is used to build the longest span structures – suspension bridges.

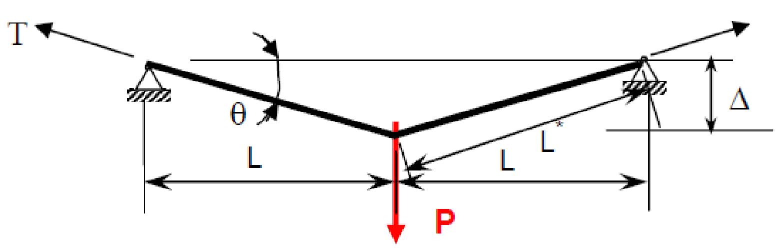


Figure 8 Catenary action

1.3 Local failure scenarios: Column loss and role of connections

Because of the variety of the accidental actions and demanding calculations (blast), structure engineers tend to rather considering the result of the action – local failure scenarios – than calculating the load impact on the whole structure.

If we are speaking particullary about buildings the possible local failure scenarios right after the accident can be possible loss of one or more vertical members (columns or walls) or horizontal members (beams, slabs).

This thesis was chosen to deal with loss of a column in musti-storey building with columns ordered in rectangural patterns focusing on the alternative load path approach. In the first stage of the analysis was considered loss of one of the columns in the base floor. When the column is not there, the load from the upper floors which it was supporting needs to be transfered to the surrouding columns. Depending on the strength, stiffness and ductility of the connections among the columns (beams,slab) and number of colums in proximity, the transfer can be successful. Otherwise the load finds its support on the ground.

We can distinguish three basic subscenarios according to the missing column location: in the middle of the building (B2), in the extremity (A3) and in the corner of a building (A1).

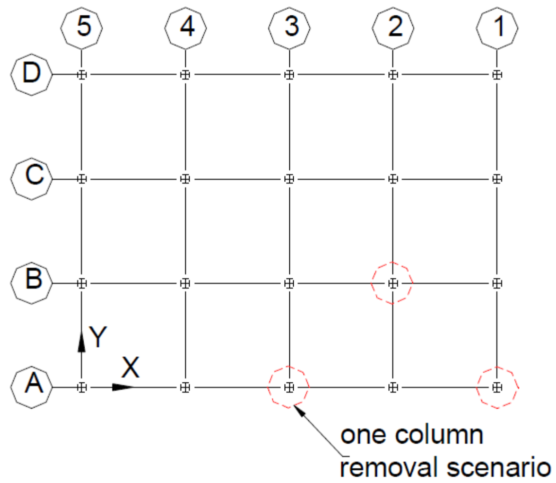


Figure 9 Column loss scenarios

If the lost column is in the corner (A1), there are potentially three columns which can support the load (A2,B1, B2). In this thesis the contribution of the slab is not considered, therefore just the connection (beams) to the columns A2 and B1 is present. The beams are perpendicular and there is no other horizontal support, therefore the catenary action can be never developped. The only mechanism to transfer the load is therefore bending. If the bending moment induced by the vertical load (dynamic) is lower than the bending resistance of the beam, connection to the column and the column resistance itself, the structure will stay stable. Because the floors are identical, in each of them there is the same bending stiffness and strength, the total loading vertical force can be devided by the number of floors due to Vierendeel action. In other words each of the floor will carry load of itself (but dynamically amplified).

If the lost column is located in the extremity (A3), the catenary action can be potentionally developped between the columns A4 and A2. The required conditions for that are: sufficient horizontal stiffness of the adjecent columns in x direction, sufficient rotation capacity of the column-beam connection for reaching the angle θ, when catenary force is significant (sufficient displacenent Δ, see **Chyba! Nenalezen zdroj odkazů.**) and sufficient strength of the connection in the axial loading. Before reaching this action the column is supported by moment resistance analogically as in previous scenario from columns A4, A2 and B3.

If the lost column is internal, in our case (neglecting slab) the catenary action can be formed in both x and y directions if the analogical criteria described above are met. If considering slab, another so called membrane action can be potentialy developped.

As explained, the middle column loss scenario has the biggest strength potential. But it should not be forgotten that the vertical load is higher – roughly twice as in extremity, fourtimes as in corner scenario. Because taking the catenary action into account is definetly not standard design routine, the investigations in that field are nescessary. Very important is to analyse the beam-column connections behaviour – ductility, strength in axial loading - because it is the prerequisite for developpment of the catenary action.

2 Code provisions and recent developments

2.1 Damage and Collapse Control Design

Damage Control Design is approach when all structural members are designed to withstand accidental actions. That means either making them stronger or protecting them. This approach is not economical for most of the cases, although there are structures (government buildings, powerplants), which are designed in this manner.

Collapse Control Design is approach which improves robustness of a stucture even when some structural members would be destroyed. The proggressive collapse is prevented due to structural integrity of the building and redundancy of the system. This approach put high demand on the engineers because the structural scheme of the building is changing. Some members plays different role and even the geometry of the structure may change. Therefore factors such as strength in tension and ductility of structural members gain importance. Special attention should be payed to connections of the members as they are often the weakest point of the chain. Similarly they may transfer tension, provide continuity and ductility in means of rotational capacity.

One approach, so called alternative load paths is using the redundancy of the members itself. It can be Vierendeel action which is using redundancy in moment resistance Figure 10 (a), catenary action, which is using redundancy in tension (b) and arch effect which is using redundancy in compression (c).

Another solution is so called key element method, when the structural integrity and overall stability is provided by the key elements. These are often big trusses across the storey as can be seen on the Figure 10 e) and d)

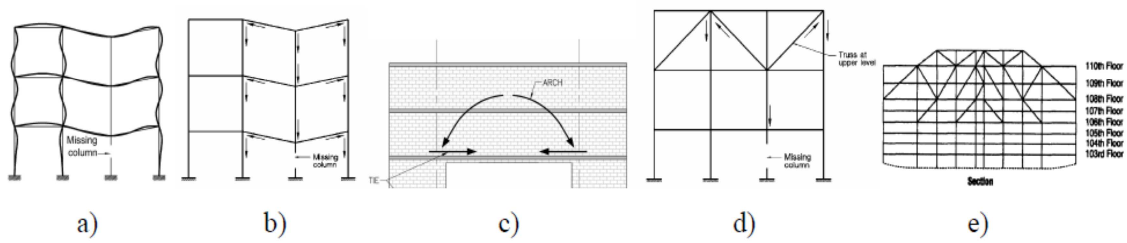


Figure 10 Supplementary load transfer routes (Marginean, 2013)

2.2 Design regulation (from NISTIR 7396)

2.2.1 British Standards

Since shortly after the Ronan Point collapse, British Standards have taken the lead in stating explicit design provisions against progressive collapse. British Standards emphasize general tying of various structural elements of a building together, to provide continuity and redundancy. Ties enhance the resistance of wall panels to being blown away in the event of an explosion, and also the ability of a structure to bridge over a lost support. In designing for this possibility, various structural elements are considered lost one at a time. In addition, structural elements deemed vital to a building stability should be designed as key elements, able to withstand accidental loads, e.g., a pressure of 34 kPa (5 psi). BS serves as basis for Eurocode.

2.2.2 The United Stated Approach (ASCE 7-10, 2010):

The ASCE document 7-10 includes a commentary, that provides the user with precautions in design to limit the effects of local collapse. The ASCE recommends design alternatives for multi-storey buildings to make them posses a level of structural integrity similar to that inherent in properly designed conventional frame structures. There are a number of ways to obtain resistance to progressive collapse and in the ASCE 7-10 two ways of design, **direct and indirect design**, are described.

The **direct design** considers the resistance to progressive collapse explicitly during the design process itself. This can be obtained by the alternative load path method which allows local failure to occur without major collapse, because the other load path(s) will allow the damage to be 'absorbed'. The structural integrity of a structure may be tested by analysis to ascertain whether alternative paths around hypothetically collapsed regions exist. In addition the Standard recommends the specific load resistance method. This method seeks to provide sufficient strength to resist failure from accidents or misuse. This may be provided in regions of high risk since it may be necessary for some elements to have sufficient strength to resist abnormal loads in order for the structure as a whole to develop alternate paths. The design philosophy necessitates that accidental actions are treated in a special manner with respect to load factors and load combinations.

The **indirect design** considers the resistance of progressive collapse during the design process implicitly through the provision of minimum levels of strength, continuity, and ductility. Alternative path studies may be used as guides to develop rules for the minimum levels of these properties needed to apply the indirect design approach to enhance structural integrity. Furthermore the ASCE provides specific recommendations to achieve a resistance to progressive collapse, as described next:

Ties: Provide an integrated system of ties among the principal elements of the structural system. These ties may be designed specifically as components of secondary loadcarrying systems, which often must sustain very large deformations during catastrophic events.

Returns on walls: Returns on interior and exterior walls will make them more stable.

Changing direction of span of floor slab: Here, a single span floor can be reinforced also in the perpendicular direction such that, in the case of failure of a load-bearing wall, collapse of the slab can be prevented and the debris loading of other parts of the structure minimised. Often, shrinkage and temperature steel will be enough to enable the slab to span in an additional direction.

Load-bearing interior walls: The interior walls must be capable of carrying enough load to achieve the change of span direction in the floor slabs.

Catenary action of floor slab: Where the slab cannot change span direction, the span will increase if an intermediate supporting wall is removed. In this case, if there is enough reinforcement throughout the slab and enough continuity and restraint, the slab may be capable of carrying the loads by catenary action, though very large deflections will result.

Beam actions of walls: Walls may be assumed to be capable of spanning an opening if sufficient tying steel at the top and the bottom of the walls allows them to act as the web of a beam with the slabs above and below acting as flanges.

Redundant structural system: Provide a secondary load path (e.g., an upper level truss or transfer girder system that allows the lower floors of a multi-storey building to hang from the upper floors in emergency) that allows framing to survive removal of key support elements.

Ductile detailing: Avoid low ductility detailing in elements that might be subject to dynamic loads or very large distortions during localized failures (e.g., consider the implications of shear failures in beams or supported slabs under the influence of building weights falling from above).

Reinforcement: Provide additional reinforcement to resist blast and load reversal when blast loads are considered in design.

Compartmentalization: Consider the use of compartmentalized construction in combination with special moment-resisting frames in the design of new buildings when considering blast protection.

Additional: While not directly adding structural integrity for the prevention of progressive collapse, the use of special, non-frangible glass for fenestration can greatly reduce risk to occupants during exterior blasts. To the extend that non-fragible glass isolates a building's interior from blast shock waves, it can also reduce damage to interior framing elements (e.g., supported floor slabs could be made to be less likely to fail due to uplift forces) for exterior blasts.

2.2.3 National Building Code of Canada

The National Building Code of Canada contains a general statement about the need for structural integrity, but its Commentary provides an extensive discussion on means to achieve that goal. The extent of the discussion reflects the importance accorded to the topic at the time, e.g., the 1975 version is much longer than the 1995 version. The Commentary covers recommendation for good structural layout, continuity of reinforcement, and structural mechanisms that would mitigate progressive collapse after local loss of support. No specific values are given for tie forces or accidental loads for key structural elements.

2.2.4 Swedish Design Regulations

The Swedish Design Regulations BKR contains guidelines on the three safety classes of various buildings. Normally, requirements relating to accidental loads and progressive collapse only apply to Safety Class 3. These requirements are detailed in a separate handbook and consist of: a) checking the stability of a damaged building under dead and live loads, and b) checking that falling debris do not cause successive failure of floors by ensuring load transfer capability within floor structure and between floor and bearing walls (tension and shear forces of 20 kN/m or about 1400 lb/ft).

2.2.5 ACI 318

The ACI 31-05 standard is an example of indirect design. It defines requirements for structural integrity such as continuity of reinforcement and use or ties in precast concrete construction.

2.2.6 New York City Building Code

The 1998 New York City Building Code is an example of direct design. It only mentions the alternate load path and the specific local resistance (34 kPa or 5 psi) methods.

2.2.7 Department of Defense Unified Facilities Criteria

Design for resistance to progressive collapse depends on the “level of protection” assigned to the building. For lower levels of protection the indirect design method is used by providing minimum tie forces. For higher levels of protection, the alternate load path method is used if sufficient ties cannot be provided.

2.2.8 Interagency Security Committee

The Interagency Security Committee (ISC) emphasizes the direct design methods (alternate load paths and specific local resistance) and makes no mention of the indirect method or structural ties.

2.3 Design regulation in Europe

The existing general approach to the design of robust buildings is either **deterministic** or, as allowed for by the design equations of codes, **semi-probabilistic**. Commonly the risks are considered implicitly and approximately by the use of various classifications of buildings. However, on certain occasions the involved risks are considered explicitly when designing for robustness, for example with Consequence Class 3 buildings of Eurocodes.

While ordinary limit states to common types of loads are given, for example, in EN 1991-1-1:2002, the robustness requirements are usually linked to accidental actions (EN 1991-1-7:2006) or other abnormal events. In other words, robustness is also the property of systems that enables them to withstand unforeseen or unusual circumstances without unacceptable levels of consequences or intolerable risks *(Gulvanessian et al. 2002)*.

The above mentioned explanation of the term "robustness" indicates that two types of circumstances may cause a failure of a structure or a structural system:

- Extreme but foreseen adverse combinations of actions and material properties. These extreme events include quantifiable abnormal events such as internal gas explosions or impact of vehicles.

Unforeseen events that may be hardly identified or whose intensity cannot be known in advance, such as bomb explosions, malicious impacts or the effects of unknown errors.

Whereas ordinary structural design is mainly orientated towards the design of structural elements or a structure, the robustness design is concerned also with"what if" scenarios in relation to component failure. While the main purpose of ordinary design is to avoid failure under foreseen circumstances, the aim of robustness verification is to limit consequences of a local failure due to foreseen and unforeseen circumstances. Here the use of the word "foreseen" itself is problematic because then it may be asked why a structure was not designed against such a "foreseen" action. However, a "foreseen action" should be interpreted as an intensity larger than the design value of that action. Therefore, the term "robustness" should be primarily considered as a property of a structure or structural system, and should not be limited to specific circumstances.

2.3.1 Europe – Robustness in the Eurocodes

The basic European document for structural design is the Eurocode EN 1990:2002, according to which sufficient structural reliability can be achieved by suitable measures, including with an appropriate degree of structural robustness. In EN 1991-1-7:2006 two strategies are presented for the accidental design condition, in general:

- The **first strategy** is based on identified extreme events (internal explosions, impact etc) and includes:
  a) design of the structure to have sufficient robustness;
  b) prevention and/or reduction of the intensity of the action (protective measures);
  c) design the structure to sustain the action.

- The **second strategy** is based on the limiting of the extent of local failure, i.e.:
  a) enhanced redundancy (alternative load paths);
  b) key element designed to sustain additional accidental load;
  c) prescriptive rules (integrity, ductility).

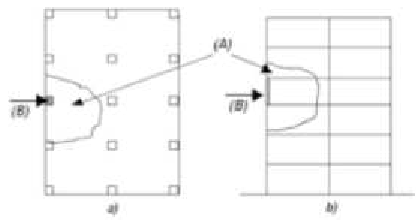
Enhanced redundancy measure

The Eurocode provides some structural measures to achieve robustness in buildings. These measures are mainly active vertical and horizontal ties (traction anchors). For main structural elements, that are designed to be capable of carrying an accidental action, the design verification is to be done using the actions that act on the main element and the adjacent components and their joints. It is thus necessary to consider the entire structure and not single elements in isolation.

The accidental design load according to EN 1990:2002 is to be applied as a concentrated load or a uniformly distributed load (when the accidental action may be considered as quasi-static one).

2.3.2 Key element design

A building should be checked to ensure that upon the notional removal of each supporting column, each beam supporting a column (i.e. a transfer beam/girder), or any nominal section of load-bearing wall, one at a time in each storey, the building remains stable and that any resulting damage does not exceed the limit given in Figure 11. Where the loss of a structural member causes more structural damage than allowed, that member should be designed as a Key Element to sustain a load of 34 kN/m2. For assessing damage, an analytical model of the structure can be used (EN 1991-1-7:2006).



Legend:
a) Floor plan
b) Elevation with vertical section
(A) Local damage less than 15 % of floor area but not more than 100 m^2 simultaneously in two adjacent floors
(B) Column, removed for analysis

Figure 11 Damage criteria

2.3.3 Risk-based design:

For structures in Consequence Class 3 (CC3) group, a systematic risk assessment is required under applicable hazards. However, there is no requirements related to risk prescribed in the code. It is the function of the authorities and/or stakeholders such as facility owners and users to prescribe these. Some information on this aspect and examples are available in *(ISO2394:1998, Vrouwenvelder et al. 2001, Canisius 2008, etc)*. General guidance for the planning and execution of risk assessment in the field of buildings and civil engineering structures is given in EN 1991-1-7:2006.

The three steps of the risk analysis can be based on the methodology of EN 1990:2002 as follows.

(a) Assessment of the probability of occurrence of various hazards, including their intensity

(b) Assessment of the probability of various states of damage and of the associated consequences of failure under the considered hazards

(c) Assessment of the probability of further failure of the damaged structure, together with the associated additional consequences of failure.

In the code, measures are also proposed to minimize the risk such as:

a) Prevent occurrence or decrease intensity of the hazard

b) Monitoring of the hazard in order to control it

c) Avoidance of collapse by changing the structural system

d) Overcoming of the hazard by enhanced strength and robustness, availability of alternative load paths by redundancies, and so on

e) Controlled failure of the structure, if the risks to human life is low.

2.4 Theoretical, experimental and numerical work

Following materials were studyied for better understanding of the topic. For each publication a small coment is present

2.4.1 Progressive collapse resistance of seismic resistant frames in case of column loss

Master thesis of Ioan-Mircea Mărginean. Uninversita Politechnica Timisoara, 2013

This master thesis is an ancestor of the thesis that you are reading. It has contributed to the chapter 3.

2.4.2 Computational Simulation of Gravity-Induced Progressive Collapse of Steel-Frame Buildings: Current Trends and Future Research Needs

DOI: 10.1061/(ASCE)ST.1943-541X.0000897

This publication describes the evolution of the robustness (progressive collapse) analysis. Differences between the modelling approaches – linerar/nonlinear, micro/macro modells, planar/solid, joints models, member models, floors and AEM are discussed. It also identifies trends and needs for future.

2.4.3 Modeling and Analysis of Single-Plate Shear Connections under Column Loss

DOI: 10.1061/(ASCE)ST.1943-541X.0000866

This paper is valuable because numerical simulation and experiments similar to this thesis are presented, however the software used was different (LS-Dyna) and the material properties were according to US standard. Reduced model is also presented. Author of this thesis is not fully satisfied how the numerical simulations corresponds to the experimental tests.

2.4.4 Numerical analyses of steel beam–column joints subjected to catenary action

DOI:10.1016/j.jcsr.2011.10.007

This paper presents also similar numerical and experimental tests totaly of 6 connection types, including extended end plate connection which is subject of this thesis. A small comment about this paper is made in chapter 5.1

2.4.5 Behavior of Composite Beam-Column Joints in a Middle-Column-Removal Scenario: Experimental Tests

DOI: 10.1061/(ASCE)ST.1943-541X.0000805

This paper is dealing with composite connections, but it gave anyway some inspiration about the experiments which this thesis is dealing with. A small comment about this paper is made in chapter 5.1

2.4.6 The Oklahoma City Bombing FEMA277

This 116 pages of investigation gives deep understanding what happened during the event, how the Murrah building behaved and how the building could be improved.

2.5 Research trends, gaps and potential contribution

Seismic detailing

Use of seismic detailing improoves generally robustness and progressive collapse prevention, as we can see in example of Murray building investigation.

Global analysis models

Without global analysis models no proper robustness assesment can be made. The software is in rapid development and the simulations are covering more and more aspects. That means nonlinearities, dynamics, large deformations and behaviour of the connections. The global models have to be validated by subassembly models or experimental tests.

Subassembly models

Subassembly models are used for validating the global models and mainly for getting the connection behaviour. Subassembly models have to be validated by experimental tests.

Experimental tests

Are nescessary as in every scientific field for validation theory, in our case numerical models. There were not many experiments done covering this topic of catenary mechanism, therefore there is big demand in this field.

Role of floor slabs

The slab behaviour and its influence in the global analysis in means of membrane action is not described sufficiently yet. The phenomena is complicated and recent studies (Li and El-Tawil 2013) are stating that the membrane action can have both positive and negative influence for the global robustness.

Probabilistic analysis

In spite that accidental load has probabilistic nature, most of the studies, researches, codes and guidlines are deterministic. This gives big untouched field for both load consideration and simulation nature.

Potential contribution

This thesis contributes directly to the field of subassembly models and experiments. It is based on the seismic philosophy and it is part of a project which is also covering the role of slabs. In further stage the results will be used for the global analysis models.

3 Selection and design of reference building structure

Steel moment-resisting frames (MRF) are considered a common type of building systems all over the world. The primary purpose of the moment frames is to withstand lateral loads, such as wind and seismic forces. In case of seismic design a big advantage is that the systems are ductile, giving behaviour factor q=6.5. The drawback are more expensive connections.

The structural design of the reference building was not the aim of this thesis. The selected reference structure design and analysis was taken from the Master Degree Project of Ing. Ioan-Mircea Marginean, presented at his Master Degree defence in June, 2013 at Universitatea Politehnica Timisoara. The building designed by him was part also of the "Structural conception and COllapse control performance based DEsign of multistory structures under aCcidental actions" CODEC (2012-2015) research programme (partially funded by the Executive Agency for Higher Education, Research, Development and Innovation Funding (UEFISCDI), Romania, (under grant PN II PCCA 55/2012) (the project within this thesis was also elaborated).

3.1 Design of structure for current design situations

3.1.1 Structural description

The geometry of the multi-storey frame building is presented in the Figure 3.1:

- 6 levels with 3.5 m height each;
- 4 openings of 8 m (in transveral direction);
- 4 bays of 8 m (in longitudinal direction);
    - Secondary beams parallel to transversal frames at a spacing of 2.66 m;

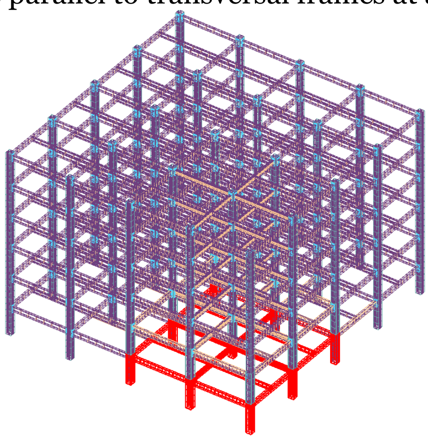


Figure 12 Geometry of the reference multi-storey frame building

3.1.2 Load evaluation

For conventional structural design, the following load cases were considered:

- Self-weight of the structure;
- Live load due to exploitation;
- Pressure induced by wind;
- Seismic action.

The considered dead load was 4.0 kN/m^2 , taking into account the 12cm thick concrete slab and its finishings. An uniform distributed load of 4.0 kN/m^2 was considered on all floors. The gravitational loads have been assigned on secondary beams during the analysis. For the determination of variable loads corresponding to the building's location, the following norms have been consulted CR 1-1-4/2012 (for evaluation of wind action) and P100-1/2012 (for evaluation of the seismic action).

For the wind load evaluation, transversal and longitudinal loading hypotheses have been considered with the following values:

$w_z = q_{ref} * C_{e(z)} * C_{pe}$

Where q_{ref} is the reference wind pressure (for Cluj Napoca is 0.4 kN/m^2), $C_{e(z)}$ is the exposure coefficient (for a medium density constructed area, at the specified height. Our case is $C_{e(24)}$=1.57 at 24 m above the ground) and C_{pe} is the building form coefficient.

The seismic response spectrum is dependent on the seismic area. For Cluj Napoca a_g=0.08 g and T_c=0.7 s.

The behaviour factors for high ductility class moment resisting frame structures is q=6.5.

For elements that are not dissipative (columns), the amplitude of the seismic action was multiplied with respect to the dissipative seismic action by $1.1 * \gamma_{ov} * \Omega$, equal to 3 on each direction for these particular cases (see Annex F of P100-1/2012).

3.1.3 Structural analysis

Loading hypotheses for plausible scenarios have been considered for fundamental loading cases, quasi-permanent and seismic loading cases.

The software SAP2000 was used to create a numerical model of the building structure. Columns and beams have been modelled as bar elements with the cross-sectional properties of the actual elements. Diaphragms have been assigned to each floor to simulate the constraints induced by the reinforced concrete slab presence and restraints were attributed to base joints to simulate the effect of foundations. The element material propertied were modeled defining the nominal characteristics of steel grade S355. Loads and masses have been assigned to the structure's elements in conformity with the ones evaluated, after which defining the response spectrums for the building. Analysis cases and finally combinations were introduced. Other inputs in the analysis software were design codes, material safety factors, buckling length coefficients, etc. To check if a first order analysis can be performed on the structure the value of α_{cr} , factor for which the design loading would have to be increased to cause elastic instability in a global mode, was checked to exceed 10 (α_{cr} = 11.45>10).

In the seismic combination (dissipative one), the inter-storey horizontal drift was computed as follows: $q * d_{max} * \gamma$, and comparing to the allowable deflection which has the value of $d_{allowable}$=0.008*h_{storey}=0.008*4000=32 mm (according to P100-1/2012).

Where γ=0.5 is the reduction factor according to the seismic return interval associated with SLS.

The resistance checks were performed as well with the data results obtained from SAP2000. For each member in part, specific checks and requirements have been verified to obtain the following results.

Columns have cruciform (maltese) sectios, made up of two HEB 450 hot rolled profiles. Main girders are made of IPE 400 hot rolled profiles. Secondary beams are made of IPE 330 for the non-composite floor structure and of IPE 270 in the case of concrete-steel composite structure case.

A reinforced concrete slab of 12cm was considered with a 2.67m span between the floor beams. The slab reinforcement includes welded wire mesh of $\Phi 6/166$ mm x $\Phi 6/166$ mm.

The design of the composite beam was performed using the software ABC Beam Calculator, resulting Nelson shear headed studs of 16mm diameter at a distance of 200 mm (Figure 3.2). For constructive reasons, for the main beams, the shear connectors were welded to top flange on one row at 200mm intervals, except at the ends, where a free zone of 2 x h_b , or 800 mm has been considered.

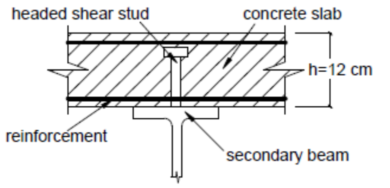


Figure 13 Cross-section of composite secondary beam

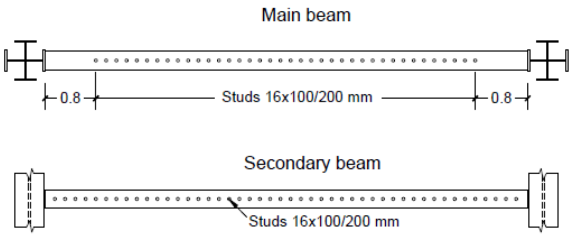


Figure 14 Spacing of connectors for main and secondary beams

For the connections, two types of extended end plate bolted connections were used. The difference consists of end plate thickness and bolt diameter.

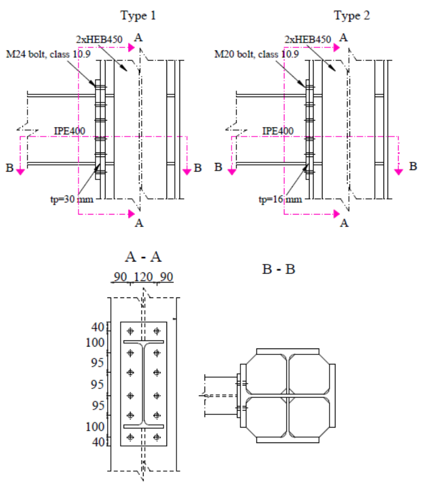


Figure 15 Connection details of beam-to-column

According to EN 1993-1-8/2005, there are three possible failure modes for bolted end plate connections. Mode 1 is characterized by a complete yielding of the flange, Mode 2 is characterized by bolt failure with yielding of the flange while in case of Mode 3 the connection fails due to the failure of the bolt. Type 1 connection has a beam strength ratio of 1.0 and Mode 2 of failure, while Type 2 has a beam strength ratio of 0.8 and Mode 1 of failure.

According to EN1993-1-8/2005, first connection is classified as full strength and full rigid while the second one is classified as partial strength and semi-rigid (Figure 3.5). However, according to EN 1998-1/2004, both connections are classified as partially restrained.

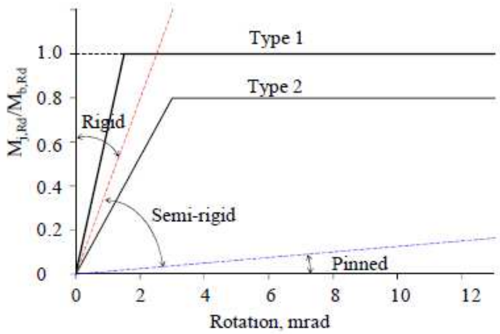


Figure 16 Moment-rotation curve for connections

3.2 Progressive collapse assessment using AP method

Structural analysis are nowadays made exclusively in softwares. Most of the comercial softwares offers some finite element solver. The difference is how sophisticated they are. More sophisticated models can better reproduce local behaviour, but the drawback is computation time.

First big division is to **linear** and **nonlinear** solvers. One nonlinearity is in material behaviour (nonlinear stress/strain curve) which is very useful if we want to catch some redistributions due to plasticity (such as in AP). Another nonlinearity is geometrical, which means that the internal forces (or stresses) are calulated on deformed structure, also called second order or Δ-P efects. This can give us a lot more precise solution when a member is loaded in combination of bending and axial force (or combinations of stresses), and also to catch for example catenary effect.

Another criteria is a **type of elements**. We start from the most primitive element types such as 1D beam elements, than 2D shells to 3D solids. There are differences after how well the software can generate a finite element mesh. More information about that can be found in chapter 0

Last division is to **dynamic** and **static** solvers. In static there is no account for kinetic energy, that means without motion. The time does not play any role - what a world. In dynamic the kinetic energy is accounted, therefore masses need to be evaluated and the time needs to be set to define velocity – derivation of position in time.

It can be seen that the best is to use nonlinear dynamic analysis with 3D elements everytime, but we would discover that a model of ordinary building would not be solved for ages. Therefore some techniques were developped to simplify the models. For example to include dynamic nature of column loss removal in static analysis Dynamic Increase Factor (DIF) is widely used. Evaluation of DIF for structures can be subject for whole publication.

In case of this project, for the assessment another type of analysis was used, so called Applied Element Method (AEM) in Extreme Loadind for Structures (ELS) software. This relatively new mathematical approach allows taking into consideration separation of elements and kinematic element interaction (contact at impact) for a reduced computational cost. In the AEM, the elements are not linked through common nodes, but with springs which are generated at the level of the common surface of the elements. These springs have the property of the volume of material represented by the tributary surface of the spring on the interface and distance between the centroids of the elements. If in FEM rigidity matrix, it is necessary for elements to share the same nodes in order to be connected, the AEM allows connectivity by sharing only surfaces. The discretisation in AEM is not limited just to the number of elements, but also allows controlling the number of springs generated on the surface, thus effectively simulating moment effects even between just two elements.

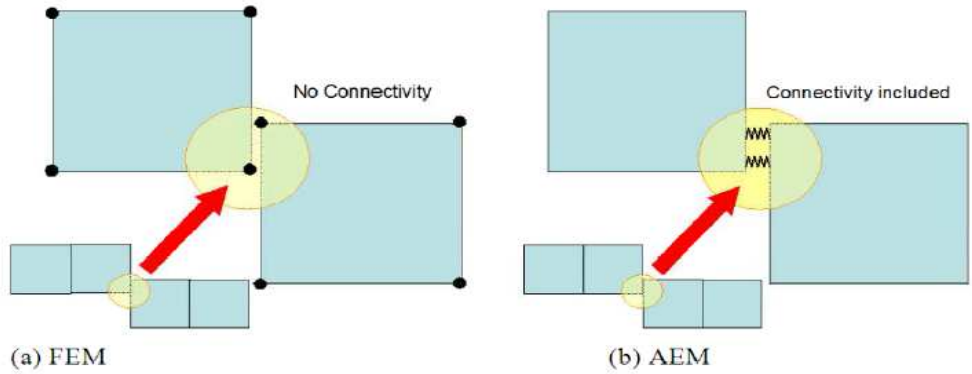


Figure 17 Partial element connectivity (Source: ELS Theoretical Manual)

The reference structure was analysed for several column loss scenarios by both nonlinear static and nonlinear dynamic analysis in ELS. In following figure and table we can identify them.

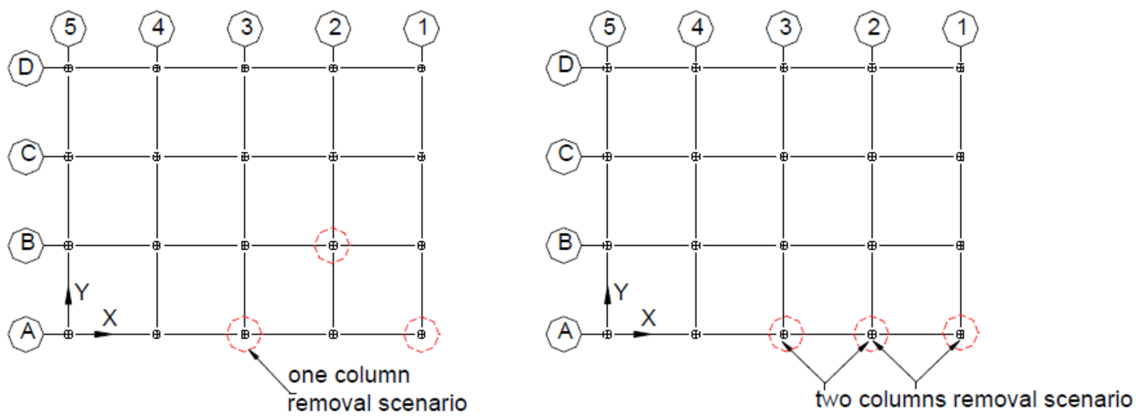


Figure 18 Column removal scenarios

Scenario	Member removed	Type of structure	Type of connection
S-I-A1	A1	Steel structure with non-composite floor beams (*S*)	Rigid connection-Type 1 (I)
S-I-A3	A3		
S-I-B2	B2		
S-I-A12	A1 + A2		
S-I-A23	A2 + A3		
C-I-A1	A1	Steel structure with composite floor beams (*C*)	
C-I-A3	A3		
C-I-B2	B2		
C-I-A12	A1 + A2		
C-I-A23	A2 + A3		
S-II-A1	A1	Steel structure with non-composite floor beams (*S*)	Semi-rigid connection Type 2 (II)
S-II-A3	A3		
S-II-B2	B2		
S-II-A12	A1 + A2		
S-II-A23	A2 + A3		
C-II-A1	A1	Steel structure with composite floor beams (*C*)	
C-II-A3	A3		
C-II-B2	B2		
C-II-A12	A1 + A2		
C-II-A23	A2 + A3		

Table 4

The assessment of the robustness of structures implies quantifying the capacity of the structure to endure additional deformation and loading in case of accidental actions. To attain this goal, one critical state is investigated for each scenario, state defined by propagation of the progressive collapse to the entire adjacent bays of the damaged columns. The gravitational forces applied on the adjacent bays are incremented, in static or dynamic nonlinear analysis, until the disproportionate collapse phenomena appears. The ratio of the value of gravitational forces that trigger the progressive collapse related to nominal gravity loads *(D+ 0.5L)* is actually a robustness index, also known as the overload factor (Ω), (Khandelwala and El-Tawil, 2011):

$$\text{Overload factor } \Omega = \frac{\text{Failure load}}{\text{Nominal gravity load}}$$

Progressive collapse static nonlinear analysis presumes removing the damaged column and increasing the gravitational loads on the floor adjacent bays to the removed element, on all floors above, without any dynamic effect. This step by step load increment takes into consideration the second order effects. The progressive collapse state was considered to be reached when the base reaction started to decrease in relation with the input gravitational load.

The dynamic nonlinear analysis approach to estimate the gravity load corresponding to failure requires performing different analyses for the same column loss scenario, with different initial gravitational loading. In the first stage, all columns have their full bearing capacity and gravitation loads are applied in a static procedure. The damaged column is removed instantaneously in a time history analysis which has a time step of 0.001 seconds, this way taking into account the inertia of the floors and assigned masses through loads. The results indicate if for the gravitational load, the structure finds stable alternate paths to redistribute the forces, or if this amount of gravitational loads will trigger the progressive collapse in that scenario. The highest value of gravitational loads that will not induce the initiation of progressive collapse is considered to be the critical load value.

3.2.1 Results

In all the scenarios the progressive collapse was prevented and the structure remained stable.

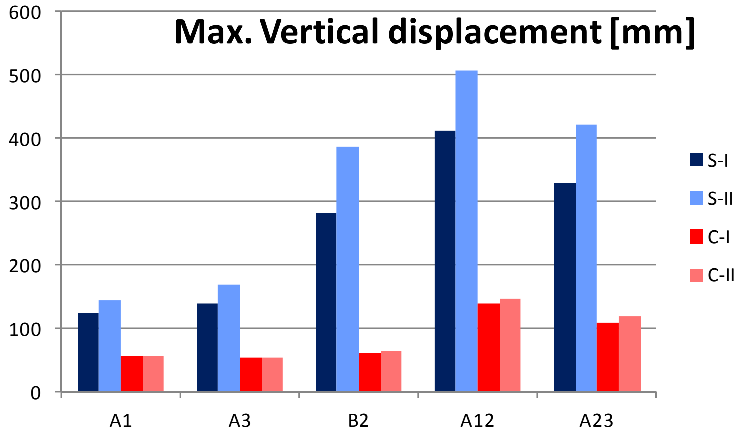


Figure 19 Vertical displacement

Scenario	Overload factor, Ω		Dynamic increase factor DIF=ΩS / ΩD
	Static analysis, ΩS	Dynamic analysis, ΩD	
S-I-A1	2.88	2.3	1.25
S-I-A3	2.35	1.8	1.31
S-I-B2	1.55	1.2	1.29
S-I-A12	1.5	1.2	1.25
S-I-A23	1.58	1.15	1.37
C-I-A1	3.82	2.83	1.34
C-I-A3	3.95	2.83	1.39
C-I-B2	3.81	2.91	1.31
C-I-A12	2.28	1.58	1.44
C-I-A23	2.95	1.92	1.54
S-II-A1	2.7	2.05	1.32
S-II-A3	2.2	1.6	1.38
S-II-B2	1.4	1.05	1.33
S-II-A12	1.45	1.1	1.32
S-II-A23	1.5	1.15	1.3
C-II-A1	3.5	2.66	1.32
C-II-A3	3.78	2.75	1.37
C-II-B2	3.65	2.58	1.41
C-II-A12	2.11	1.58	1.34
C-II-A23	2.51	1.91	1.31

Table 5 Overload and dynamic factors

Altohugh the value of classical theoretical DIF for sudden loss of support is 2. This study showed that because of the material nonlinearity the value is mostly below 1.5 which corresponds with recent studies - Ruth et al. 2006, Foley et al. 2008, Dinu et al. 2010, Khandelwala & El-Tawil 2011

3.3 Identification of role of connections

Connections play very important role as they are most often the weakest point of the chain. This can be avoided by designing them with overstrength as it is practiced in seismic design. As we can see from previous chapters the specific demand for the connections in case of column loss are rotation capacity (ductility) and axial strength. The brittle connections such using fully penetrated weldeds on site should be avoided. Even that pinned connections can give good rotation capacity, the axial strength is usually poor and therefore they are not recomended.

If we speak about the demand more specificaly the highest rotation achieved in the case study was

$tan\varphi = \frac{\Delta}{L} = \frac{500}{8000}$; φ = 63 mrad

Or we can just express it with ratio deflection over span which makes 6.3%

Because of some uncapabilities of ELS software and for verification, more refined model was made and experimeltal tests are planned.

4 Selection of the joint subassembly: pre-test evaluation validation

4.1 Subassembly testing model

For experimental tests investigating the connection behaviour a subassembly with following scheme was selected.

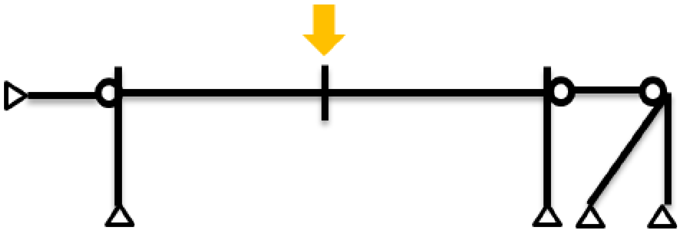


Figure 20 Subassembly static scheme

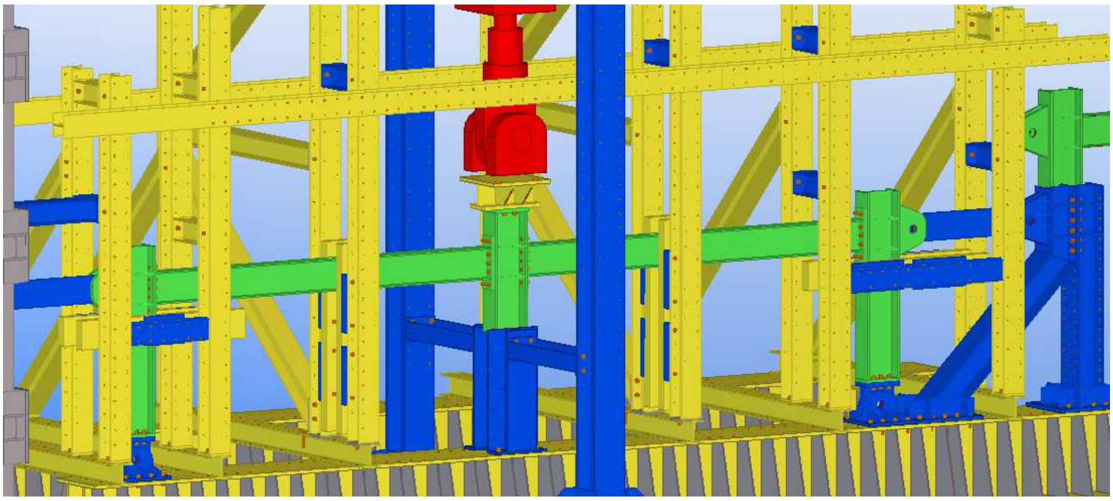


Figure 21 Subassembly 3D model

It represents one frame with missing column. The experiment is „2D“, so redundant resistance comes from beams just in one direction (located in the reality on the edge of a building). The beams are made of IPE 220 section, the columns of HEB 260 with cutted flanges to width of 160 mm. The test should investigate four different types of connections. The catenary action can be potentially developped, because the present columns are horizontally supported by ties on each extremity of the subassembly (simulating that the building is horizontally continuous and braced). The other support of the column is in the bottom (can represent missing column of a one storey building or a top floor) as a pinned connection. On top of the missing column vertical load will be applied as displacement controlled kvasi-static test. The head of the actuator has a spherical bearing, so in the actuator only axial force is transmitted. The head can also move in horizontal direction. Because of that there is a risk of lateral torsional instability of the beam and dolumn. Therefore pairs of sliding plates were placed 1.2 m on both sides from the lost column. Later from numerical simulations it was discovered that these plates are not sufficient for preventing the twist of a column and another pair of plates were added to guide the column directly. Report from the numerical simulation of this problem is in chapter 5.1.

4.1.1 Extended end plate connection (No. 1)

This very comon type of connection in MFR buildings, was designed according to EC3 as rigid, partial strength connection. It was design to fail in mode 1, plastification of the end plate due to bending in order to give good ductile behaviour. The endplate is welded to the beam in shop and than bolted on site to the column.

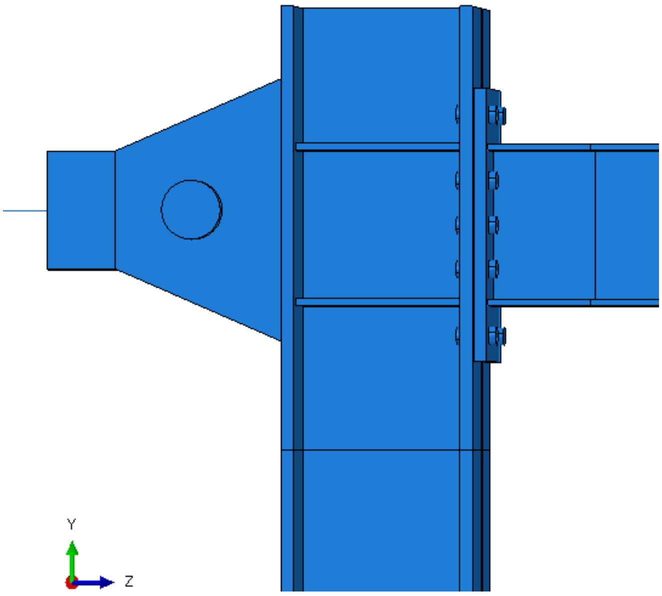
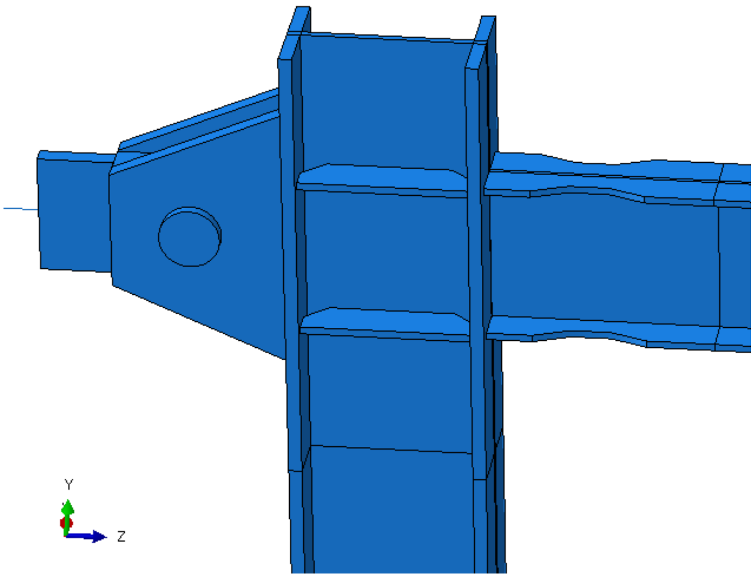


Figure 22

4.1.2 Dogbone connection (No. 2)

This type of connection can be called also Reduced Beam Section. The beam is directly welded to the column and has specific cuts which are forming the zone for the future plastic hinge. Because of the reduction full strength of the connection can not be expected, but it should give again good ductile behaviour. The negative aspect of this connection is welding on site.

Figure 23



4.1.3 Coverplate connection (No. 3)

This type of connection consists of coverplates welded directly to the column (in shop) and to the beam (on site). The plates are more strong than the the flanges of the beam, so the potencial place for plastic hinge is in the beam. The welds made on site have sufficient length and do not penetrate the materials, therefore there is no risk of weak point in them. Another coverplate welded to the column and bolted (can be also welded) to the web of the beam is transfering shear.

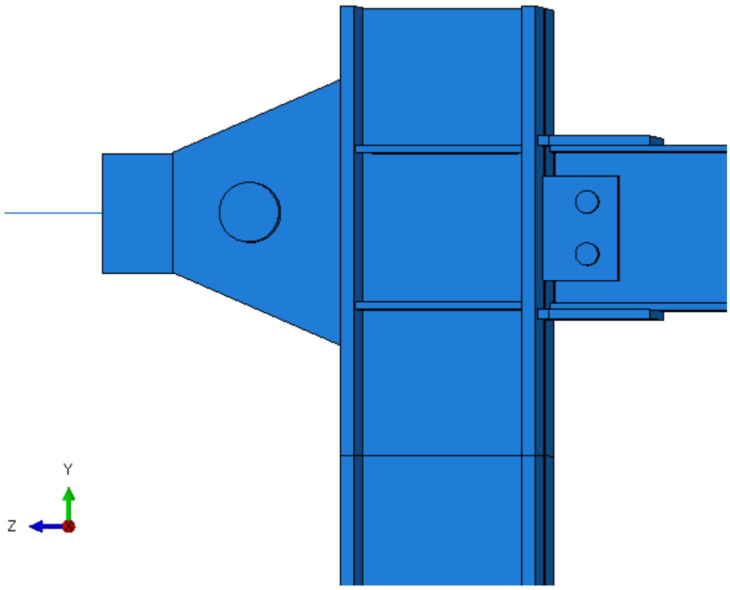
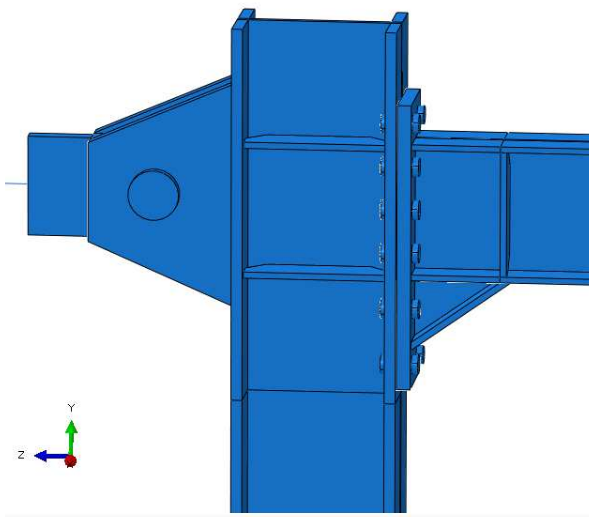


Figure 24

4.1.4 Haunch connection (No. 4)

This is very comon connection in MRF because it is sometimes not possible to design strong enough endplate connection. All welds are made in shop and the parts are just bolted on site. The possible plastic hinge is expected in the beam, giving good ductile behaviour.

Figure 25



4.2 Numerical modelling techique description

All the numerical simulations were run in software Abaqus FEA 6.11 as nonlinear dynamic analysis in explicit solver. The reasons for need of nonlinear analysis are straight forward, because it is needed to investigate the post elastic behaviour with large deformations.

For understanding the choice of the solver which Abaqus offers, following table is present.

Feature	Common	ABAQUS/Standard only	ABAQUS/Explicit only
Element library	Comprehensive	no limits	only elements appropriate for explicit solutions
Material models	Comprehensive	only yield models	yield and fracture models
Solution methods		Implicit Integration needs solve multiple coupled equation Using the K Matrix (F=Ku) Stable	explicit integration step by step using small time steps sometimes not stable
Required Disk Space		repetitive calculations likely takes a lot of space	no repetitive calculation normal
Types of Problems	Linear: non-linear: Contact*: usual systems**	Can solve Can solve Can solve if simple Optimal under steady*** loads	Can solve Optimal. even if highly non-linear Optimal. even for complex and varying conditions Optimal under transient*** loads like Impact, Pulse and Explosion

Figure 26 Abaqus solvers (www.wikipwdia.org)

So, as the structure was rather complex with lot of contacts (bolts) Explicit solver was chosen. This choice can also make a solid base for further investigations of the problem such as direct analysis of an impact or explosion.

4.2.1 Elements

The naming conventions for solid elements depend on the element dimensionality (Figure 4.16): e.q. one- , two-, three-dimensional or axisymmetric elements.

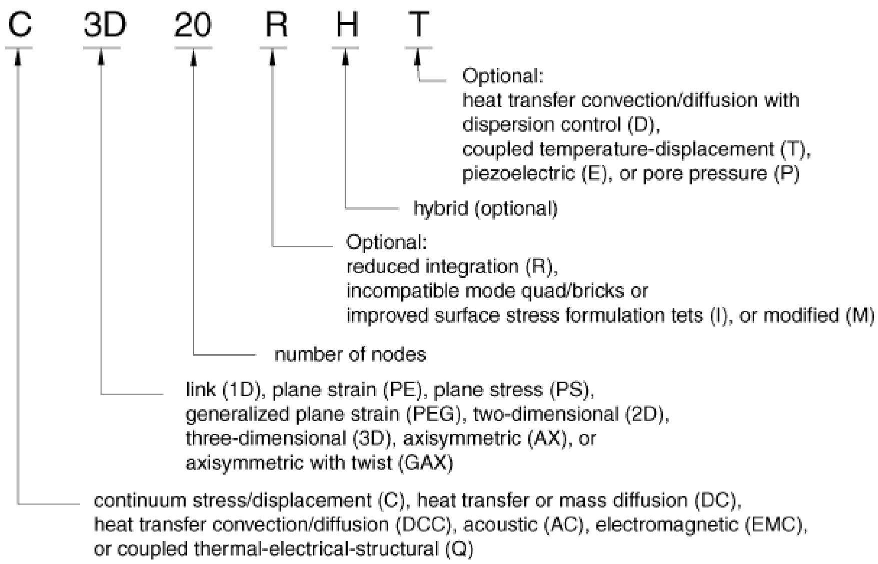


Figure 27 Name convention of solid elements in Abaqus

Mesh element shapes

Most elements correspond to one of the shapes shown on Figure 28; that is, they are topologically equivalent to these shapes. For example, although the elements CPE4, CAX4R, and S4R are used for stress analysis, DC2D4 is used for heat transfer analysis, and AC2D4 is used for acoustic analysis, all five elements are topologically equivalent to a linear quadrilateral. As you can see on Figure 29, a typicall "Hex" (Hexahedra or brick) element shape is presented for the meshing of an element.

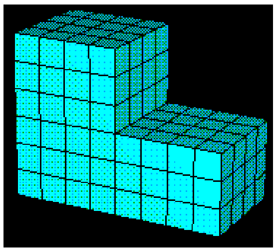


Figure 29 Hex

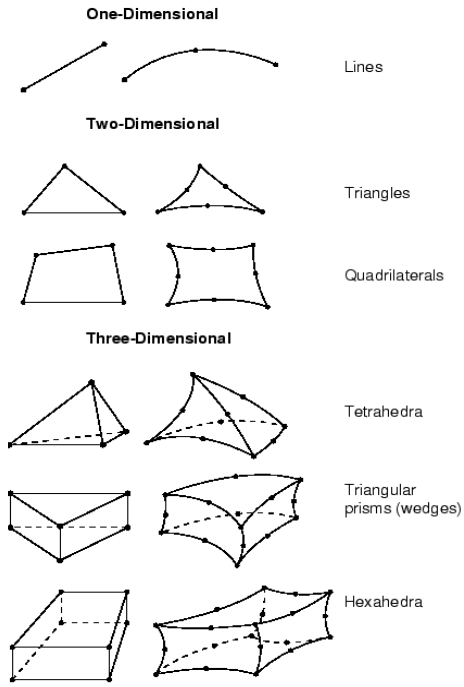


Figure 28 Element shapes

Choosing between bricks/quadrilaterals and tetrahedra/triangles

Triangular and tetrahedral elements are geometrically versatile and are used in many automatic meshing algorithms. It is very convenient to mesh a complex shape with triangles or tetrahedra, and the second-order and modified triangular and tetrahedral elements (CPE6, CPE6M, C3D10, C3D10M, etc.) in Abaqus, thus they are suitable for general usage. However, a good mesh of hexahedral elements usually provides a solution of equivalent accuracy at less cost. Quadrilaterals and hexahedra have a better convergence rate than triangles and tetrahedra, and sensitivity to mesh orientation in regular meshes is not an issue. However, triangles and tetrahedra are less sensitive to initial element shape, whereas first-order quadrilaterals and hexahedra perform better if their shape is approximately rectangular.

Choosing between first- and second-order elements

In first-order plane strain, generalized plane strain, axisymmetric quadrilateral, hexahedral solid elements, and cylindrical elements, the strain operator provides constant volumetric strain throughout the element. This constant strain prevents mesh "locking" when the material response is approximately incompressible.

Second-order elements provide higher accuracy in Abaqus/Standard than first-order elements for "smooth" problems that do not involve complex contact conditions, impact, or severe element distortions. They capture stress concentrations more effectively and are better for modeling geometric features: they can model a curved surface with fewer elements. Finally, second-order elements are very effective in bending-dominated problems.

First-order triangular and tetrahedral elements should be avoided as much as possible in stress analysis problems; the elements are overly stiff and exhibit slow convergence with mesh refinement, which is especially a problem with first-order tetrahedral elements.

In Abaqus/Standard the “modified” triangular and tetrahedral elements should be used in contact problems with the default “hard” contact relationship because the contact forces are consistent with the direction of contact. These elements also perform better in analyses involving impact (because they have a lumped mass matrix), in analyses involving nearly incompressible material response, and in analyses requiring large element distortions, such as the simulation of certain manufacturing processes or the response of rubber components.

Choosing between full- and reduced-integration elements

Reduced integration uses a lower-order integration to form the element stiffness. The mass matrix and distributed loadings use full integration. Reduced integration reduces running time, especially in 3D. For example, element type C3D20 has 27 integration points, while C3D20R has only 8; therefore, element assembly is roughly 3.5 times more costly for C3D20 than for C3D20R.

In Abaqus/Standard you can choose between full or reduced integration for quadrilateral and hexahedral (brick) elements. In Abaqus/Explicit you can choose between full or reduced integration for hexahedral (brick) elements. Only reduced-integration first-order elements are available for quadrilateral elements in Abaqus/Explicit; the elements with reduced integration are also referred to as uniform strain or centroid strain elements with hourglass control.

Second-order reduced-integration elements in Abaqus/Standard generally yield more accurate results than the corresponding fully integrated elements. However, for first-order elements the accuracy achieved with full versus reduced integration is largely dependent on the nature of the problem.

4.2.2 Steel S235 material model

Structural steel is an isotropic material which has good ductility and strength. It generates significant deformation prior to failure. Structural steel grade S235 was asked from the manufacturer, but the strength of delivered material was higher (f_y around 300 MPa). Because the analysis is dynamic, density (ρ=7850 kg/m^3) was needed to input. The isotropic elastic properties are defined by giving Young’s modulus (E=210000 GPa) and Poisson’s ratio (ν=0.3). The shear modulus (G) can be expressed by these two terms. For defining the plastic behaviour property of the material, isotropic hardening model was used by defining yield stress and plastic strain data To simulate the failure ductile damage was used, it gives nice necking behaviour before loosing strength completly (breakage). This material model was calibrated as described in chapter 4.3.1

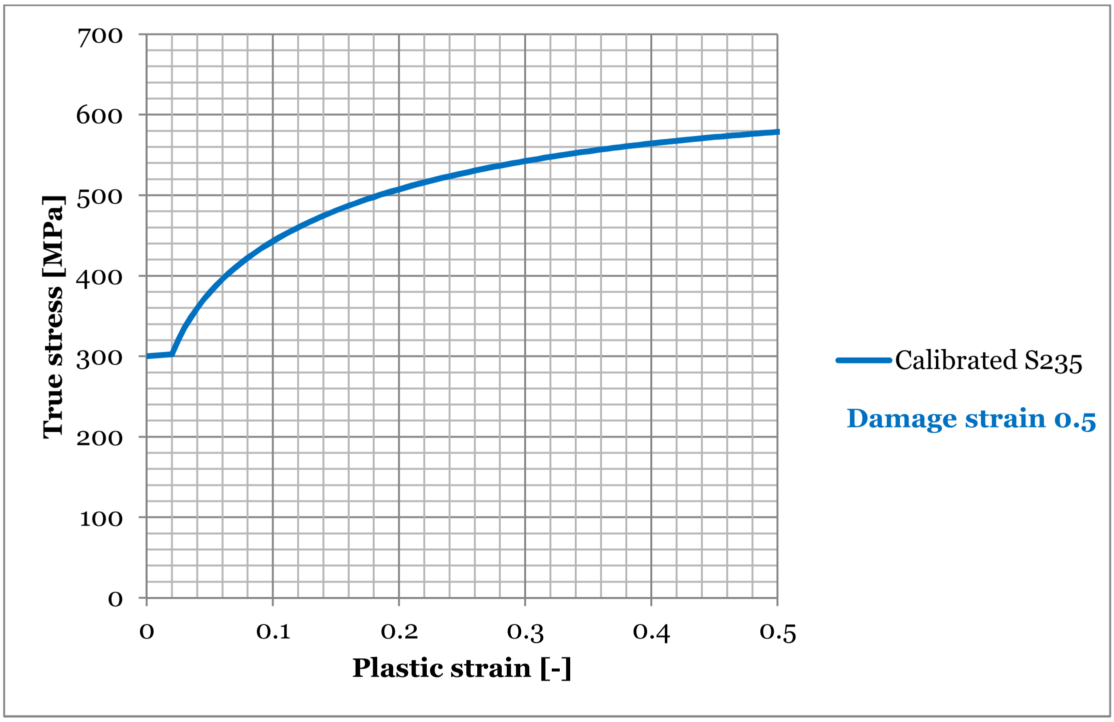


Figure 30 S235 plastic curve

4.2.3 Bolt grade 10.9 material model

Basic properties of bolts as density and elastic behaviour were the same as stuctural steel. The significant difference is in plastic region, because the high strength steel si much less ductile and with no significant yielding (proof stress is standardly used instedad). The true stress strain curve in plastic region is presented below. In reality there is no clear border between elastic and plastic region, so the model is behaving. The introduced properties carefully makes the smooth transition. Failure was again defined by introducing ductile damage. This material model was calibrated as described in chapter 0

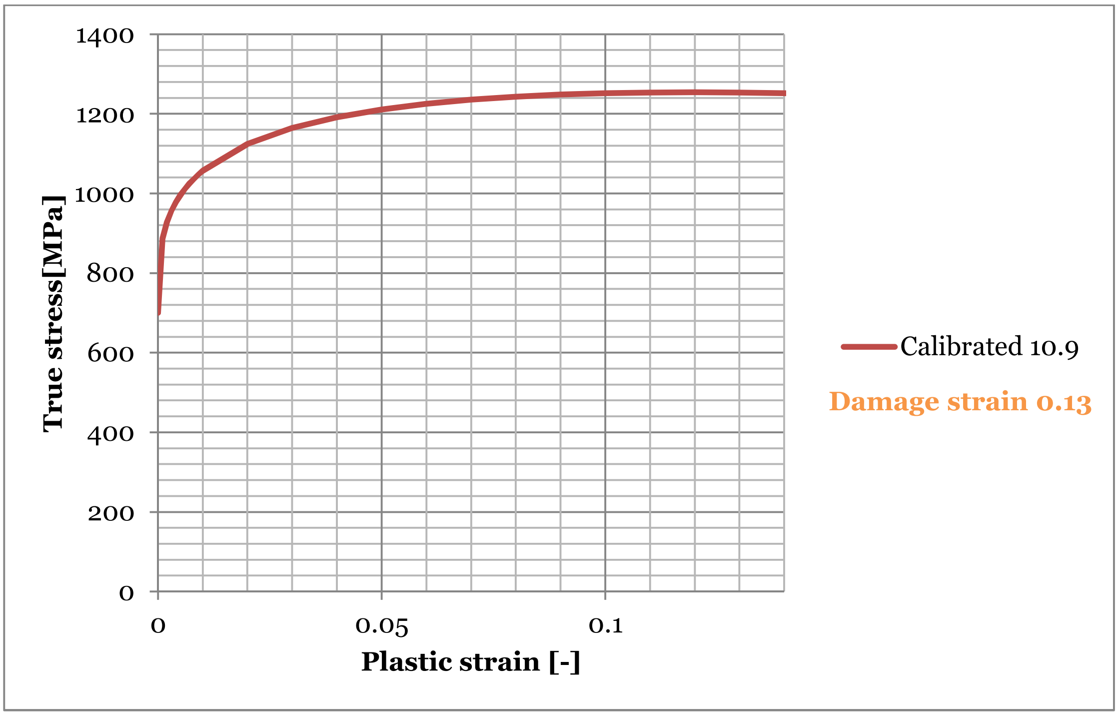


Figure 31 Bolt 10.9 plastic curve

4.2.4 Interactions, contacts

After the parts are meshed and assembled into required geometry, contacts between them need to be defined.

General contact

Is a contact which is making the parts interact with eachother in compression. Without this contact if two parts are meeting, they just intersect without creation of any stresses. This type of contact is forming also the connection when bolts are used. Minor property is tangential friction coefficient which can be also defined, in this case was used value of 0.8.

Tie

For contact between parts which are welded together forming one piece (beam-endplate) we define ties. That will link directly nodes which are in proximity together. This contact is working in both tension and compression.

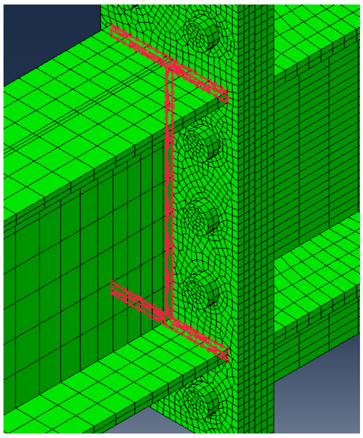


Figure 32 Tie between beam and endplate

Coupling

Another interaction is linking surfaces to points. This can be useful if, we want to define some point boundary condition, such as support or point load. It is similar interaction as tying and can work in both tension and compression. Advantage of using point boundary conditions is that basic deformations and forces can be easily monitored and interpreted there.

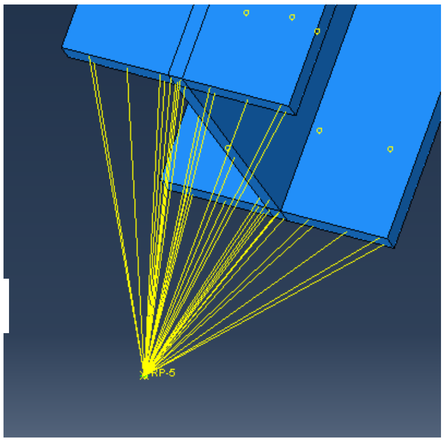


Figure 33 Coupling of base of a column

4.2.5 Mass scaling

Mass scaling is often used in Abaqus/Explicit for computational efficiency in quasi-static analyses and in some dynamic analyses that contain a few very small elements that control the stable time increment. Because the explicit central difference method is used to integrate the equations in time, the discrete mass matrix used in the equilibrium equations plays a crucial role in both computational efficiency and accuracy for both classes of problems. When used appropriately, mass scaling can often improve the computational efficiency while retaining the necessary degree of accuracy required for a particular problem.

Mass scaling for quasi-static analysis is usually performed on the entire model. However, when different parts of a model have different stiffness and mass properties, it may be useful to scale only selected parts of the model or to scale each of the parts independently. In any case, it is never necessary to reduce the mass of the model from its physical value, and it is generally not possible to increase the mass arbitrarily without degrading the accuracy. A limited amount of mass scaling is usually possible for most quasi-static cases and will result in a corresponding increase in the time increment and a corresponding reduction in computational time. However, we must ensure that changes in the mass and consequent increases in the inertial forces do not alter the solution significantly. Choosing the target time increment too high will not produce quasi-static results. Choosing too low, while conservative, will result in long run times.

Two types of mass scaling are available in Abaqus/Explicit: fixed mass scaling (performed once at the beginning) and variable mass scaling (performed in the beginning and periodically during the analysis). These two types of mass scaling can be applied separately, or they can be applied together to define an overall mass scaling strategy.

There are several methods to perform it, the chosen one for our analysis was the semi-automatic mass scaling for the whole model at the beginning of the analysis. A scale to target time increment method was used by defining target time increment equal to 5E-006. The option of scale element mass if below minimum target was enabled, in order to scale the masses of only the elements whose element stable time increments are less than the target value chosen.

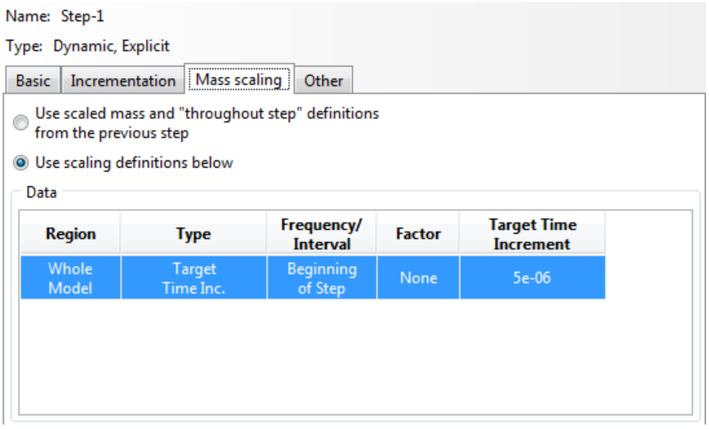


Figure 34 Mass scaling of the model

The target increment should be as big as possible to give shorter computational time, but on the other hand the global energies of the system had to be monitored (from history output) and if the kinetic energy was rising from small numbers, smaller target inrcement had to be set in ordet to stay in range of quasi-static analysis.

4.2.6 Interpretation of the results

In addition to the default history output (energy monitoring), a history output request was defined by the so called ”set” domain in point of interests. It can be for example for the load application point and requesting as output data the vertical displacement (U2) and vertical reaction (RF2) at this node. It was used this at the level of results, obtaining the force-displacement curve automatically at this point.

Another result is the **Von Mises Stress**. Von Mises Stress is widely used by designers, to check whether their design will withstand given load condition. Using this information an engineer can say his design will fail, if maximum value of Von Mises stress induced in the material is more than strength of the material. It works well for most of the cases, especially when material is ductile in nature. Concept of Von mises stress arises from distortion energy failure theory. According to distortion energy theory failure occurs, when distortion energy in actual case is more than distortion energy in simple tension case at the time of failure. Distortion energy is the energy required for shape deformation of a material. During pure distortion shape of the material changes, but volume does not change.

The objective is to develop a yield criterion for ductile metals that works for any complex 3-D loading condition, regardless of the mix of normal and shear stresses. The von Mises stress does this by reducing the complex stress state down into a single scalar number that is compared to a metal's yield strength, also a single scalar numerical value determined from a uniaxial tension test (because that's the easiest) on the material in a laboratory. The condition of failure is as follows:

$$\left[\frac{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}{2}\right]^{1/2} \geq \sigma_y$$

If the left hand side is denoted as Von Mises stress:

$$\left[\frac{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}{2}\right]^{1/2}=\sigma_v$$

More general representation can be:

$$\sigma=\sqrt{0.5\left[\left(\sigma_x-\sigma_y\right)^2+\left(\sigma_y-\sigma_z\right)^2+\left(\sigma_z-\sigma_x\right)^2\right]}$$
$$+\sqrt{+3\left(\tau_{xy}{}^2+\tau_{yz}{}^2+\tau_{zx}{}^2\right)}$$

Analogical to Von Mieses stress is plastic equivalent strain PEEQ. Its a scalar from the components of plastic strains. The only difference is that it is cumulative. PEEQ is telling directly to the user how much of plastic strain the material experienced and can predict the point of failure which is very usefull.

4.2.7 Bolts

All bolts were simulated as a rod made of solid elements (C3D8R) with an equivalent diameter based on the effective cross-sectional area (threaded part) of the shank, with cylinders at the ends representing the head and nut. Bolt thread was not modelled directly because its modeling would be extremely time consuming. Due to difficulty of measuring the initial stress in bolts, no pre-stress was applied to the bolts. The space between the head and nut was exactly the same as the thickness of the plates which it was holding, therefore no influence of not tightening or pre-stressing of the bolts was used. The diameters of the holes for the bolts were bigger than the shank as the code says for non-fitted bolts (typically 2 mm). The material was 10.9 grade as defined in chapter **Chyba! Nenalezen zdroj odkazů.**

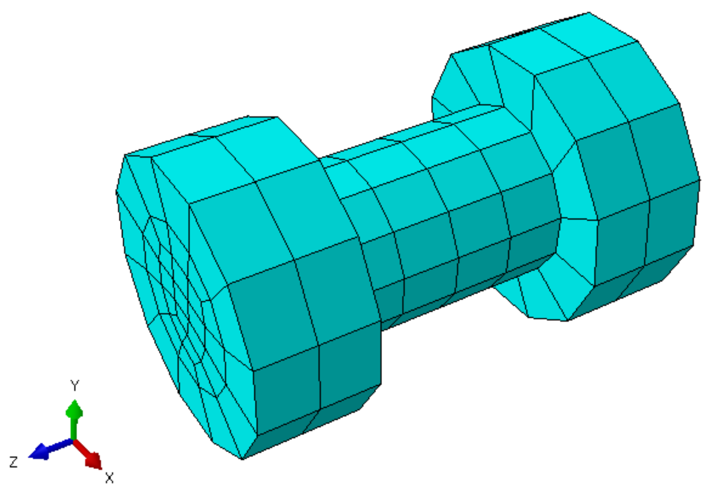


Figure 35 Numerical model of bolt

4.3 Pretest numerical evaluation, identification of macro-component tests (welded, bolted)

4.3.1 Coupon test

For calibration of the steel material model a tension coupon tests were made. The geometry was defined according to Figure 36 with thickness 10mm. was introduced to Abaqus with properties described in chapter **Chyba! Nenalezen zdroj odkazů.**. The used elements were C3D8R. The speciment was coupled in the ends with reference points which serves a support. The test was displacement controlled quasi-static, same as the simulation. The experimental results were not precise in the elastic phase due to the slippage, which was unfortunately not monitored. Results shows good aproximation of the real behaviour.

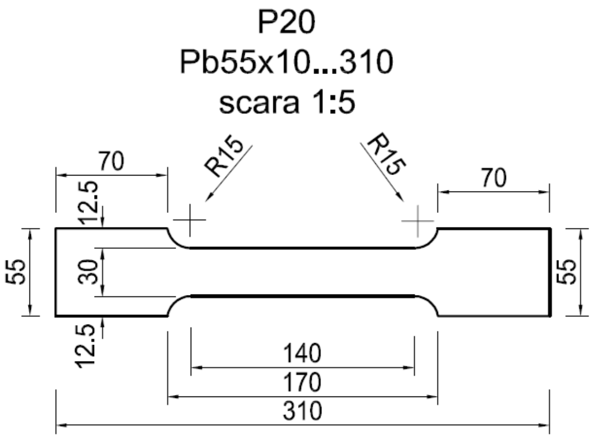


Figure 36 Dimensions of the coupon

Figure 37 Photo of coupons

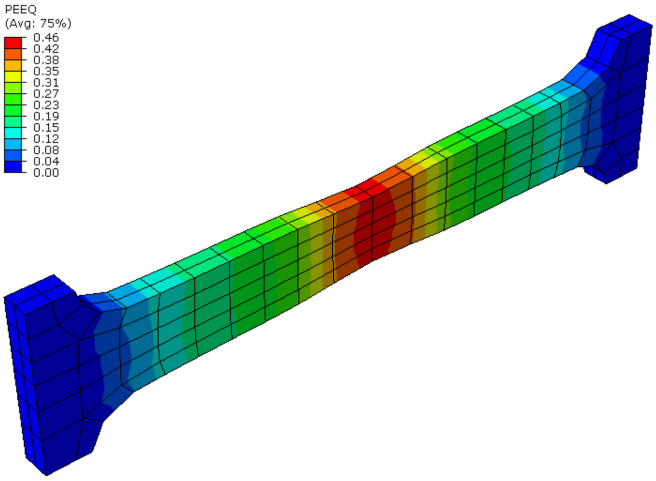


Figure 38 PEEQ before failure

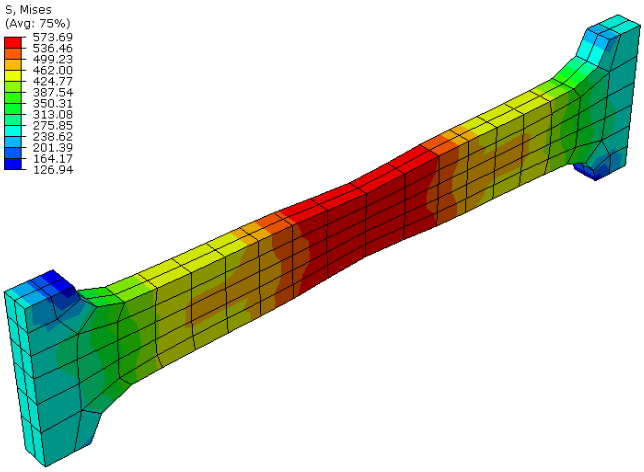


Figure 39 Stress before failure

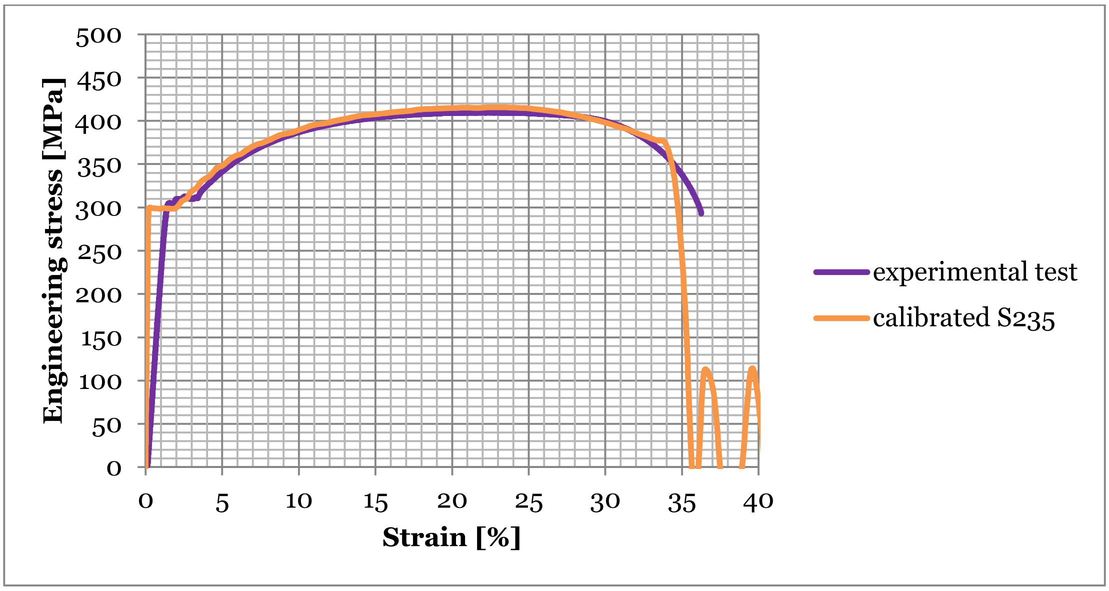


Figure 40 Coupon curve

4.3.2 Bolt test

For calibration of the bolt material properties again a tension tests were made. The geometry introduced to Abaqus was a cylinder with diameter corresponding to effective area of the threaded part of shank, with properties described in chapter **Chyba! Nenalezen zdroj odkazů.**. The used elements were C3D8R. The speciment was coupled in the ends with reference points which serves a support. The test was displacement controlled quasi-static, same as the simulation. The results of the experiment were not precise in the elastic phase due to the slippage, which was unfortunately not monitored. The results shows good aproximation of the real behaviour.

Figure 41 Photo of bolt test

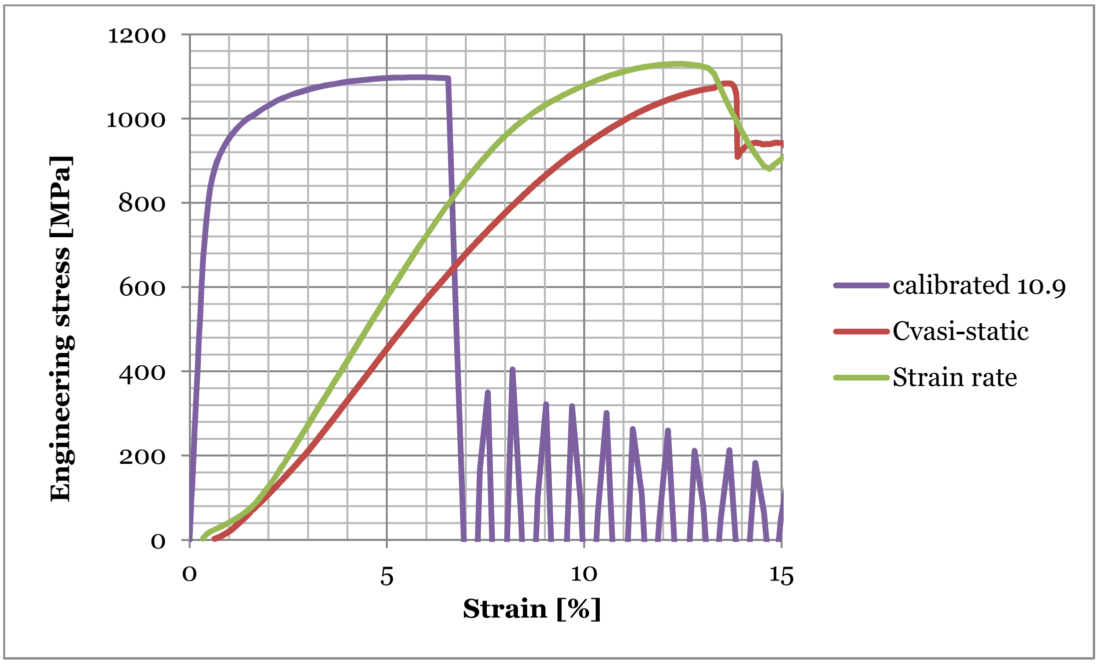


Figure 42 Bolt curve

4.3.3 T-stub test

Finaly for calibration of the bolted joint behavior a tension tests of a T-stubs were made. The geometry introduced to Abaqus was corresponding to precise measurements of the speciment made before the experiment. Material properties of the steel are described in chapter **Chyba! Nenalezen zdroj odkazů.**. The used elements were C3D8R. Bolt models were according to chapter 4.2.7. The speciment was coupled in the ends with reference points which serves a support. The test was displacement controlled quasi-static, same as the simulation. The results shows good aproximation of the real behaviour. The ultimate failure was fracture of the bolt with the endplate slightly plastified.

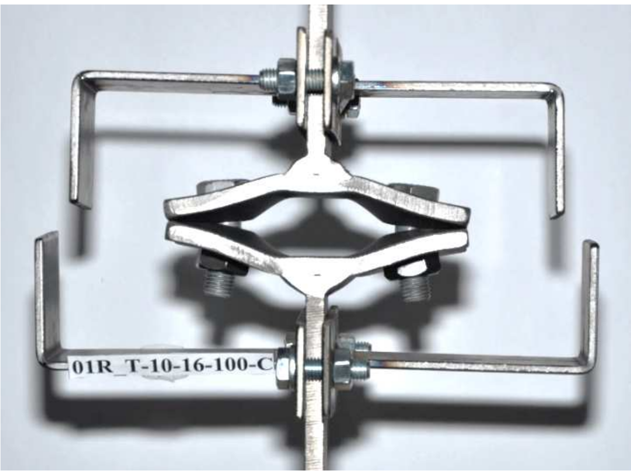


Figure 43 Photo of T-stub just before failure

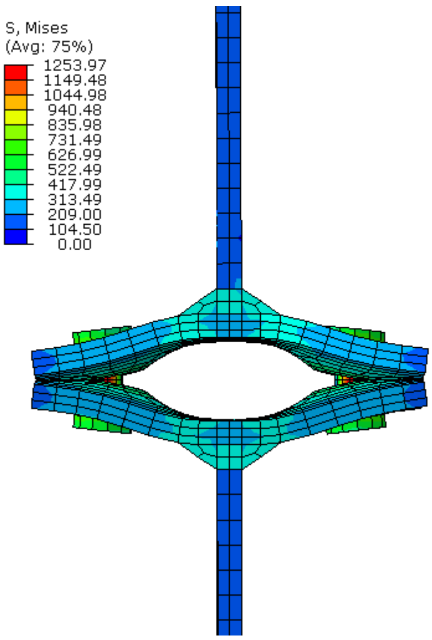


Figure 45 Stresses before failure

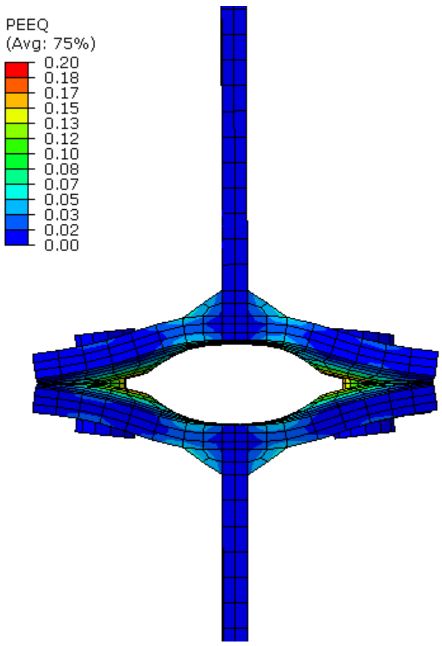


Figure 44 PEEQ before failure

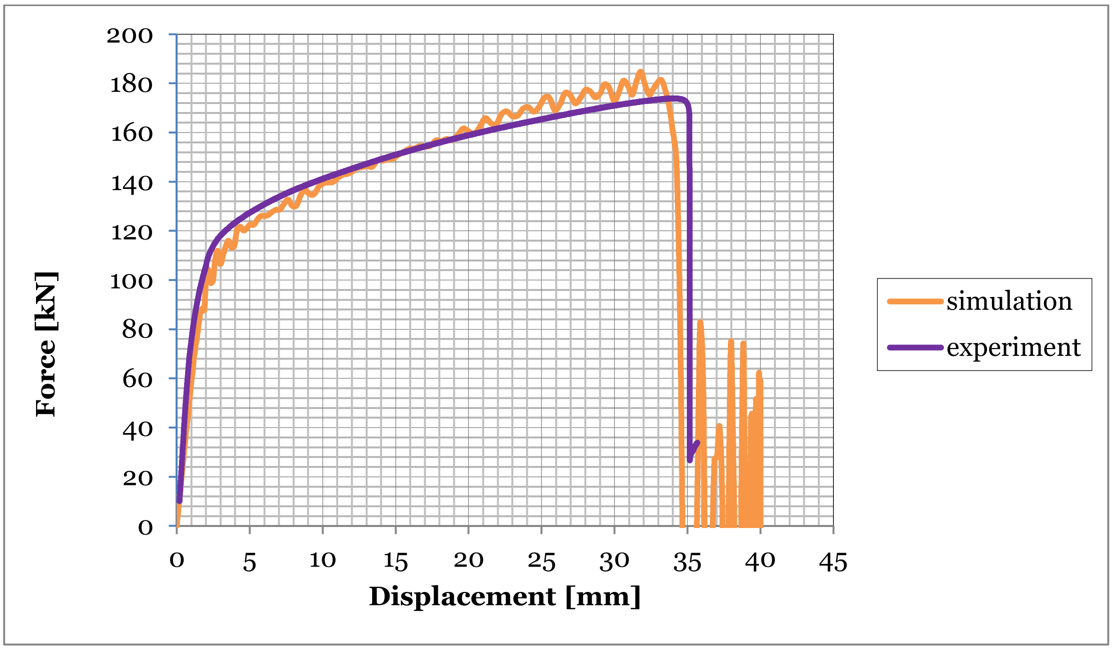


Figure 46 Force displacement curve of T-stub

5 Numerical simulations of beam-to-column joint behaviour under large vertical displacements following loss of a column

The numerical simulations of the beam-to-column joints were made again in software Abaqus with the same approach as the macro-component tests, detailed description can be found in chapter 0. Geometry is described in chapter 4.1.

The only difference are members which are providing the horizontal support in the level of the beams, which are modelled as beam members. The sections have equivalent area as a section of the ties in the experiment in order to get the same stiffness. We can see also plates guiding the column to prevent lateral instability is it is discussed in following chapter.

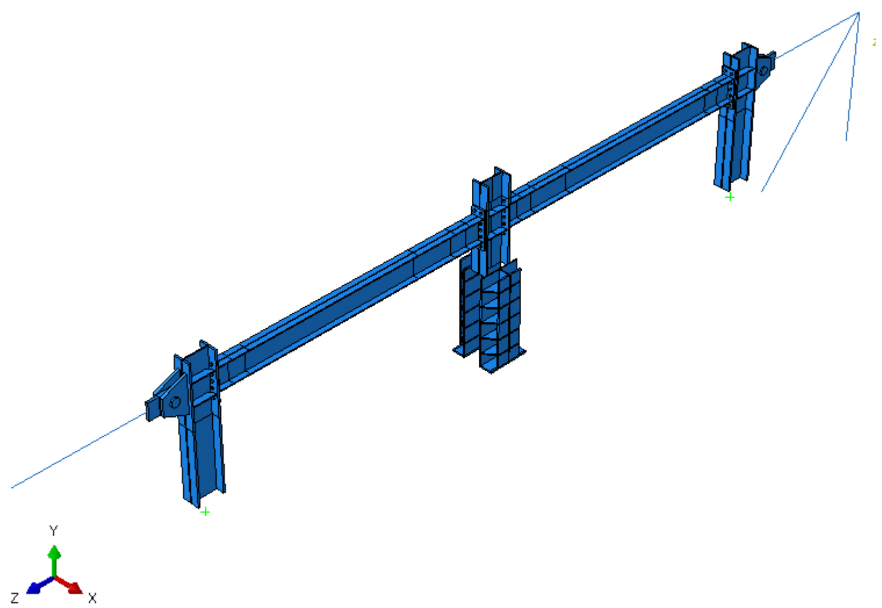


Figure 47 Typical arangement of the simulation

5.1 Study of lateral stability of the experiment and catenary

In every bending problem there is a risk of lateral instability. In the case of a column loss lets say in basement, the column in first floor which is loading the structure is providing the latteral support at the same time. In the experimental setup the actuator is manufactured to transmit only axial force, and therefore is not providing any lateral support. To provide the lateral supports, pairs of sliding plates were designed and put on the lateral part of the beam in 1.2m distance from the missing column on both sides. Later it was observed from numerical models that these supports are insufficient. To verify this statement, special numerical models with changing of the distance between the lateral supports and column were made. Unfortunately any other way (for example from standards) how to calculate this phenomena – lateral instability of beam in catenary action was not discovered.

All the models were created in the same style (and software) as the other numerical simulations. The statical scheme is analogical to the subassembly, but more simple. The sections and material models are identical.

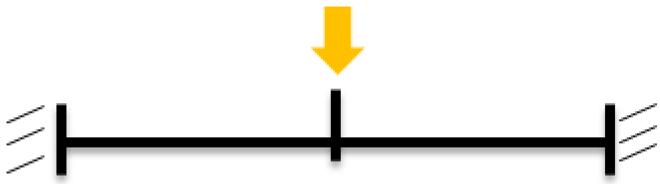


Figure 48 Scheme of lat. stability study

There were four outputs which were monitored in every case: vertical force and vertical displacement on the top of the column, horizontal reaction at the end support (catenary) and moment reaction at the end support. This little simulation is also demonstrating the influence of catenary effect. The graphical representation of the simulation is force/displacement curve which is plotted directly from the output (which is the same place as input :)), point of load application.

Horizontal reaction was monitored to get the catenary action. Catenary action is simply calculated by multiplying the horizontal reaction by vertical displacement in the middle and deviding by horizontal length.

Moment contribution is calculated by use of classical formula for the moment in the end. We presume, that the moments at the middle and at the end are equal to Loading Force times length devided by four. Therefore moment contribution to the loading force (reaction) is equal to Moment multiplied by 4 and devided by length of the beam.

As a proof of right, if we sum up the moment contribution and catenary we should obtain loading force. This is used when its not that easy to monitor the moment reaction - in our subassembly.

Thats why two lines are plotted for the moment contribution in this study. Green one is calculated from the moment reaction and violet (mom2) by substracting catenary action from the total load. As we can see they are almost identical and therefore this procedure is validated.

The curve of moment action in these simulation is a bit different than in reference materials 2.4.4 and 2.4.5.

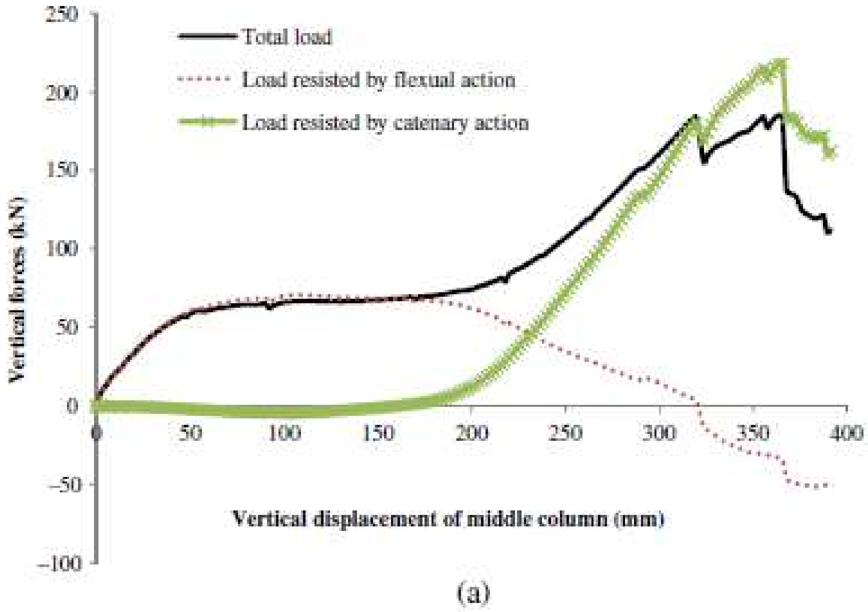


Figure 49 (Bo Yang, Kang Hai Tan 2013)

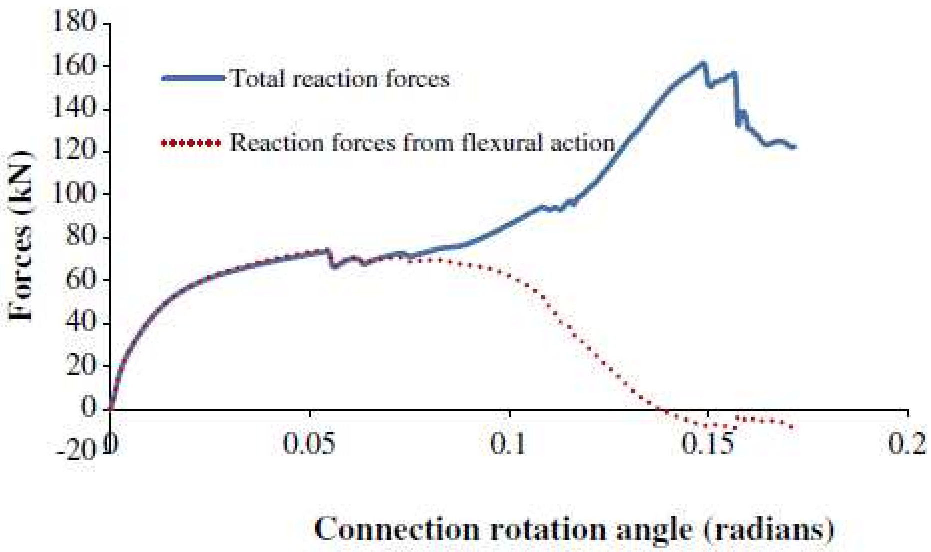


Figure 50 (Bo Yang, Kang Hai Tan 2009)

The difference is in the flexural action which starts to drop, when the catenary action is gaining importance (0.1 rad resp. 200 mm) and it drops even to negative values! In authors opinion the moment reaction can not go to negative values. The author would be glad to get any comments of this topic on his e-mail: lyznicek@seznam.cz

5.1.1 No lateral supports

It is no surprise that the phenomena is pronounced when no lateral supports are present.

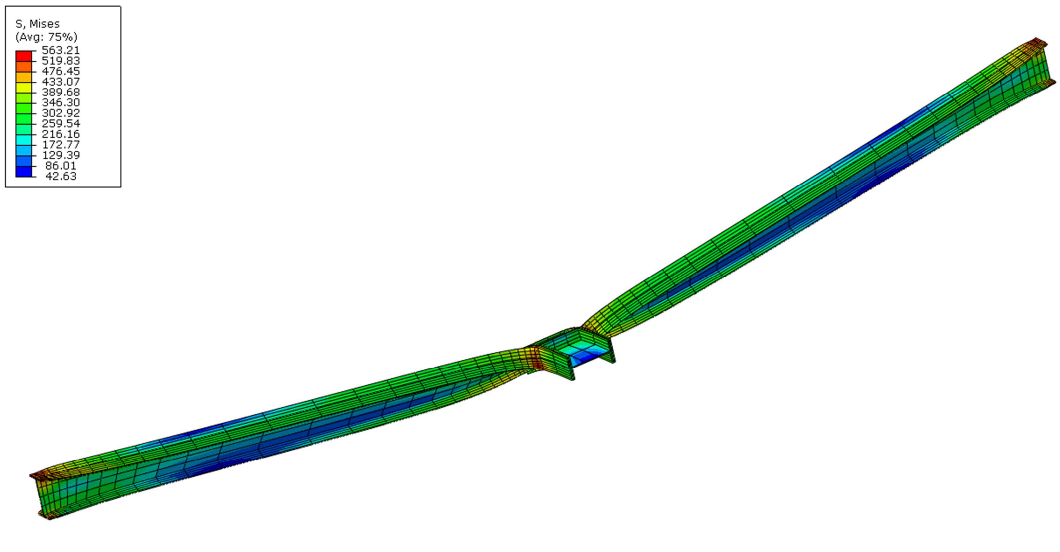


Figure 51

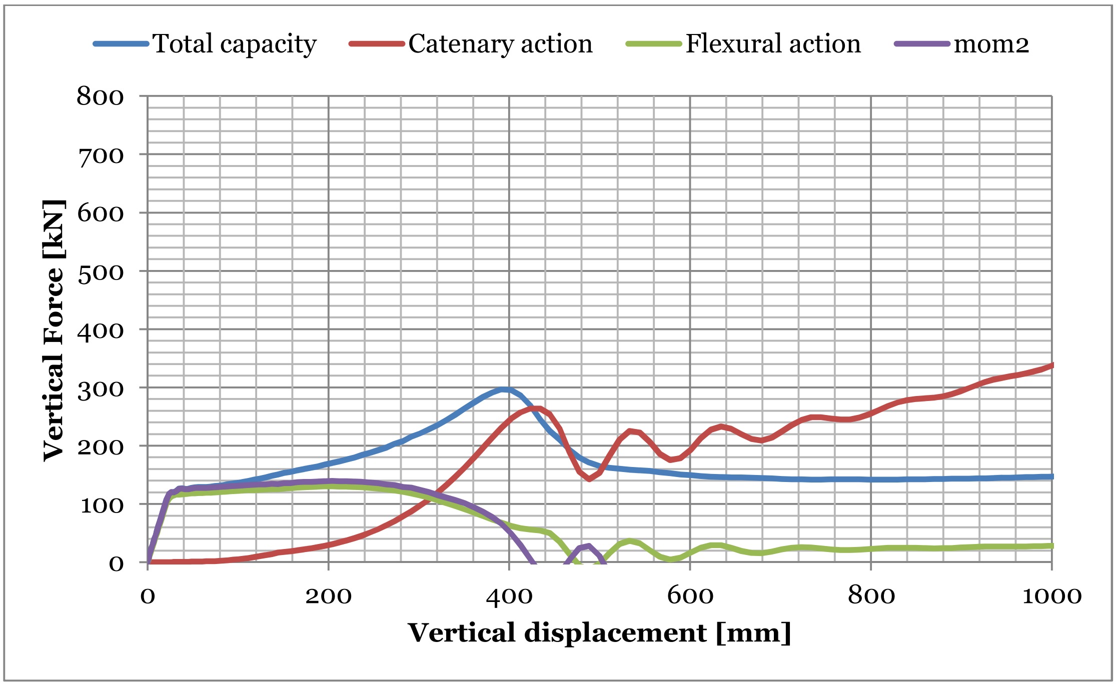


Figure 52

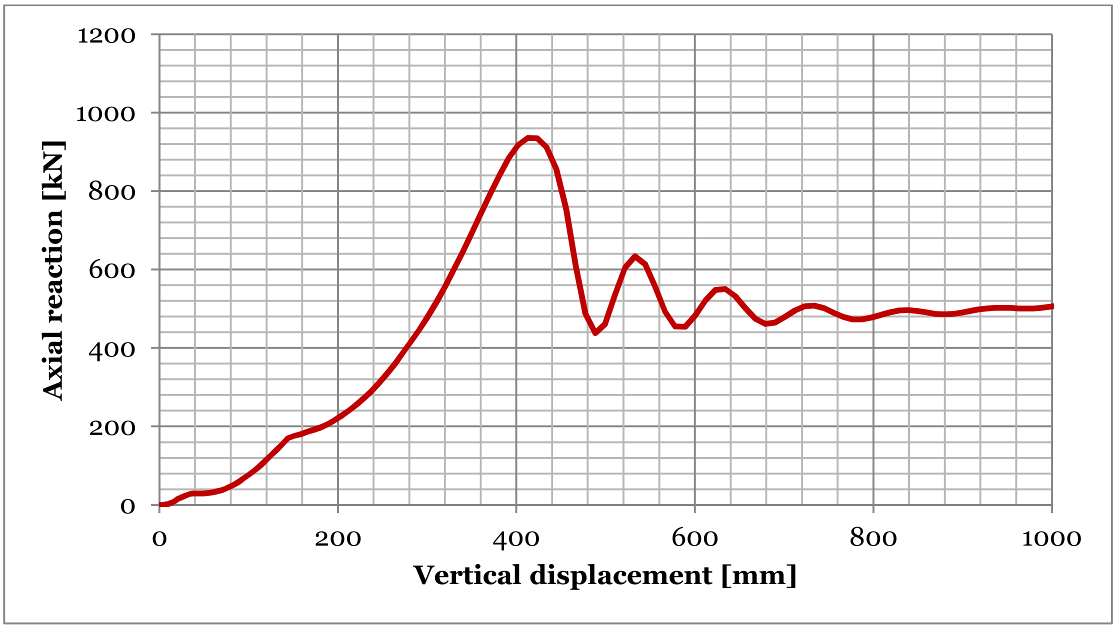


Figure 53

What is more surprising that the lateral instability phenomena is starting at deflection of 380 mm where the axial force has already got the power and flexural action is decreasing.

5.1.2 Lateral supports 2.4 m apart

When the lateral supports are placed 2.4 meters apart as it was originally proposed, the instability phenomena was significally reduced starting at deflection around 800 mm which is almost the ultimate capacity. Anyway the breakage of the material was not reached.

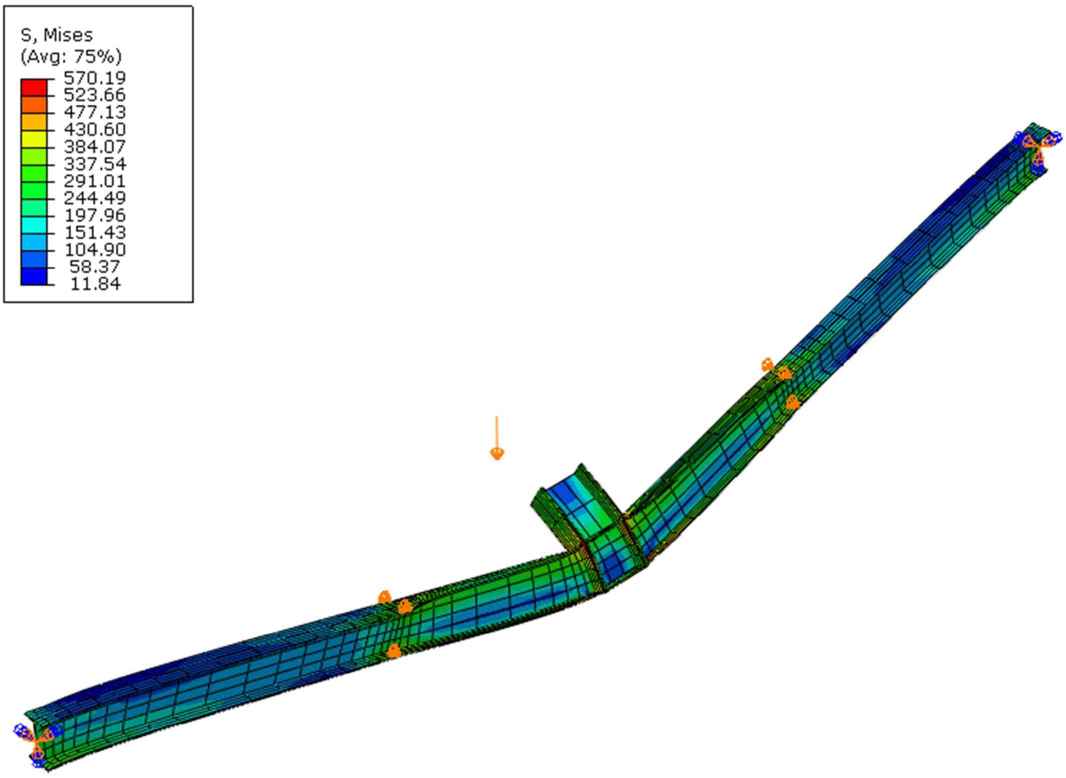


Figure 54

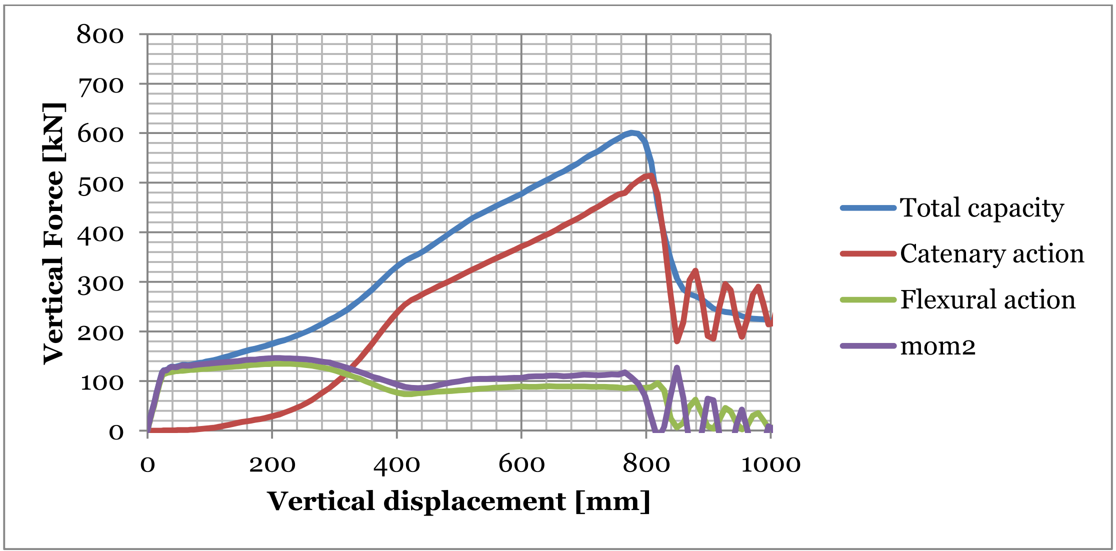


Figure 55

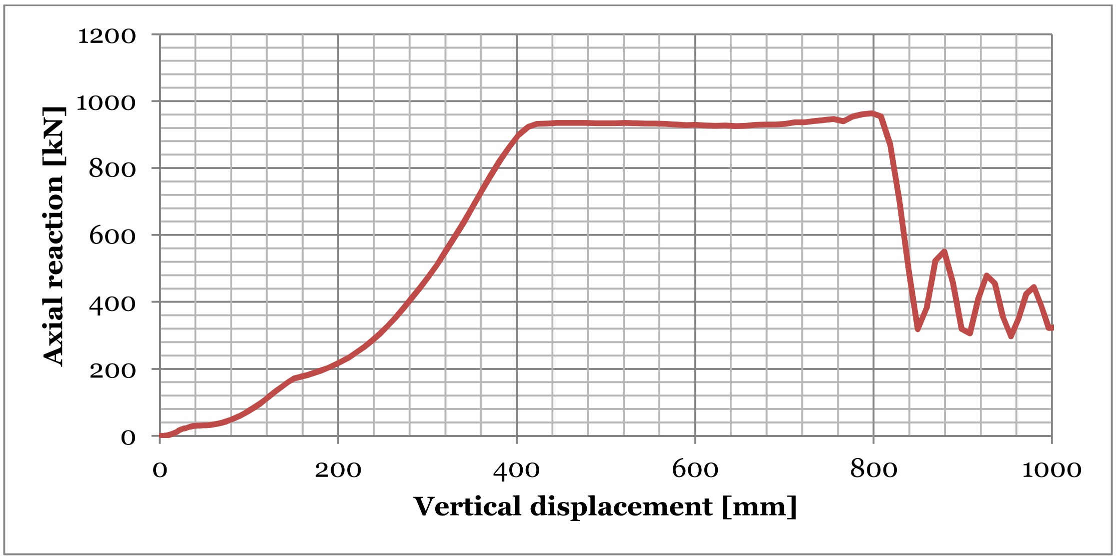


Figure 56

5.1.3 Lateral supports 0.7 m apart

Even when the lateral supports are placed just 0.7 m apart the breakage is not achieved. But as we can see comparing with next simulation the ultimate capacity of the material is almost reached.

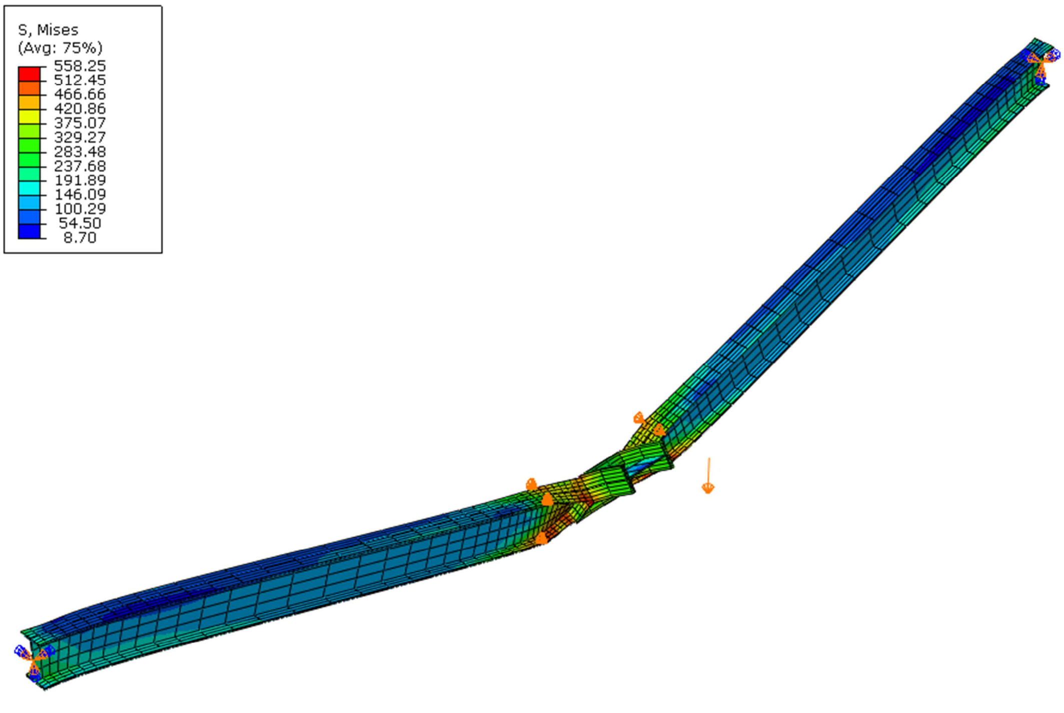


Figure 57

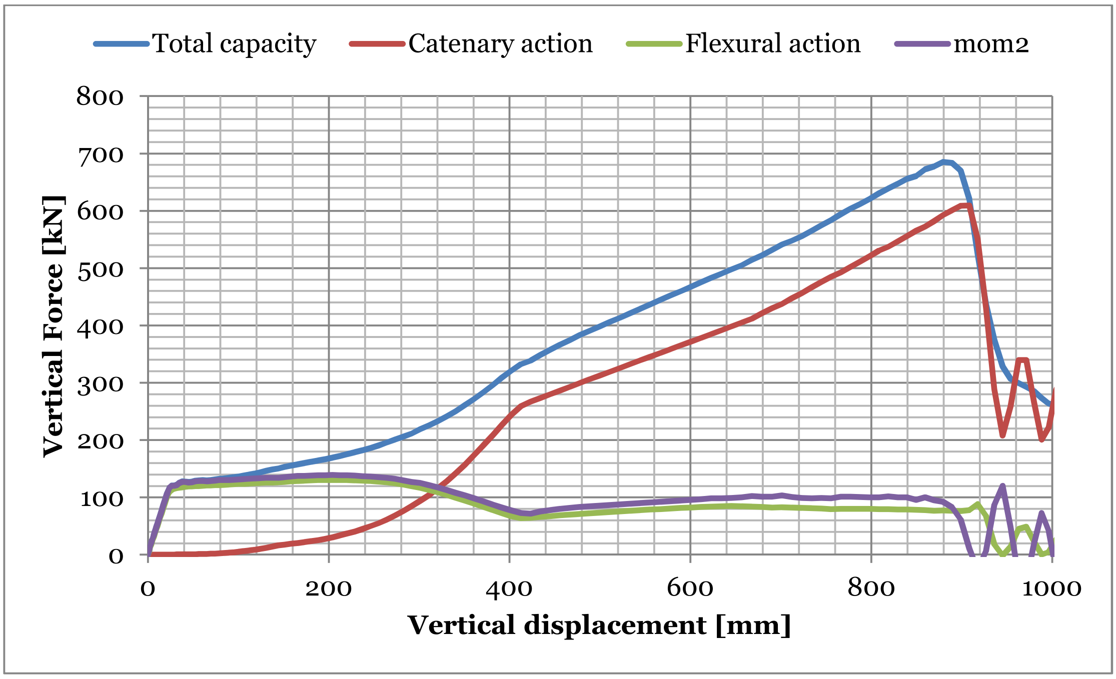


Figure 58

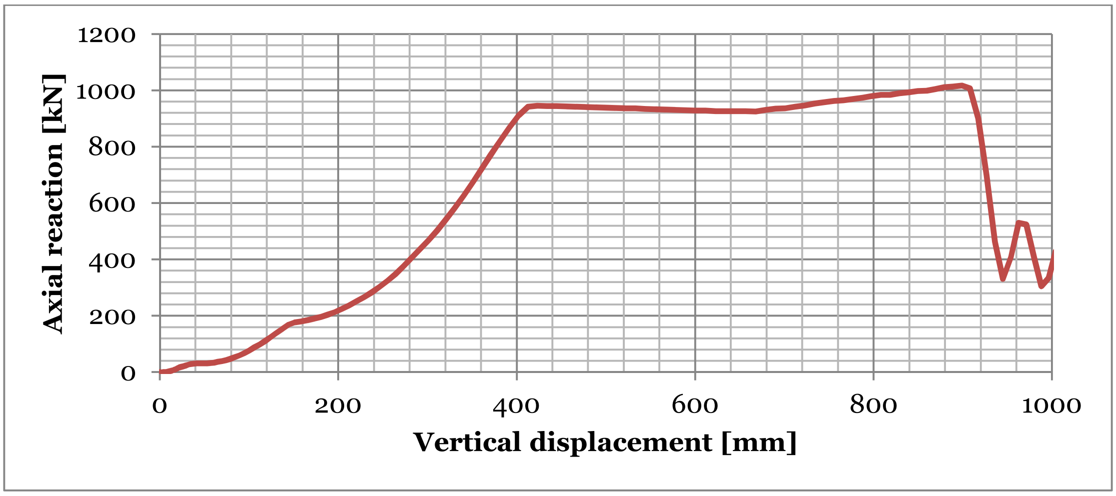


Figure 59

5.1.4 Lateraly fixed point of load application

Finally fully restrained test was simulated. The ultimate capacity was achieved with breakage of the material.

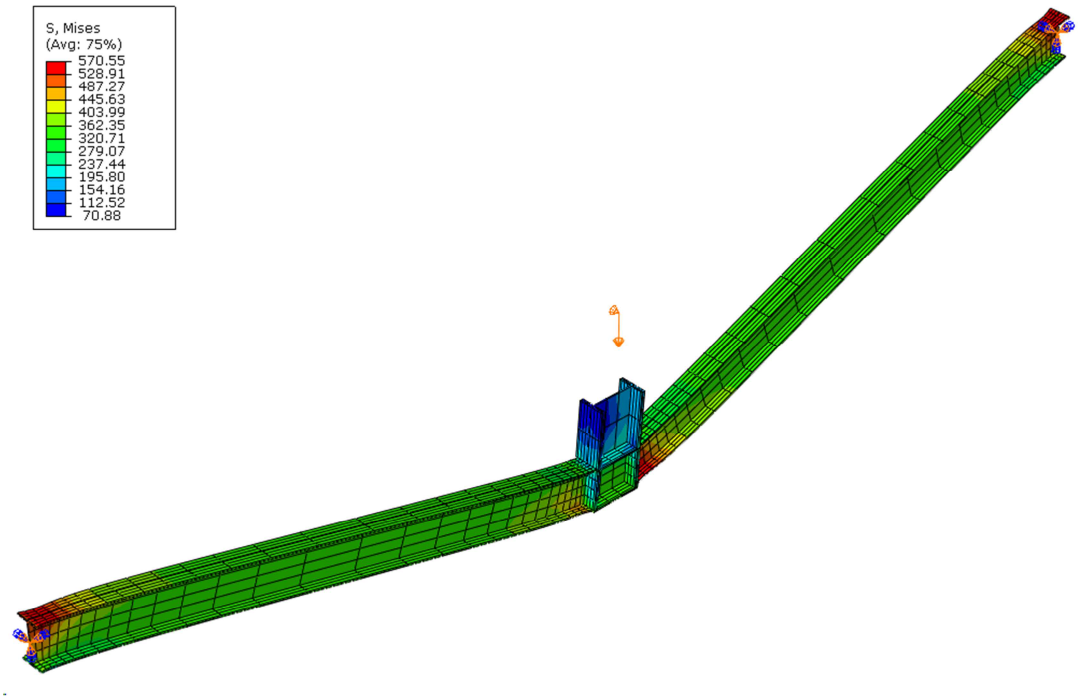


Figure 60

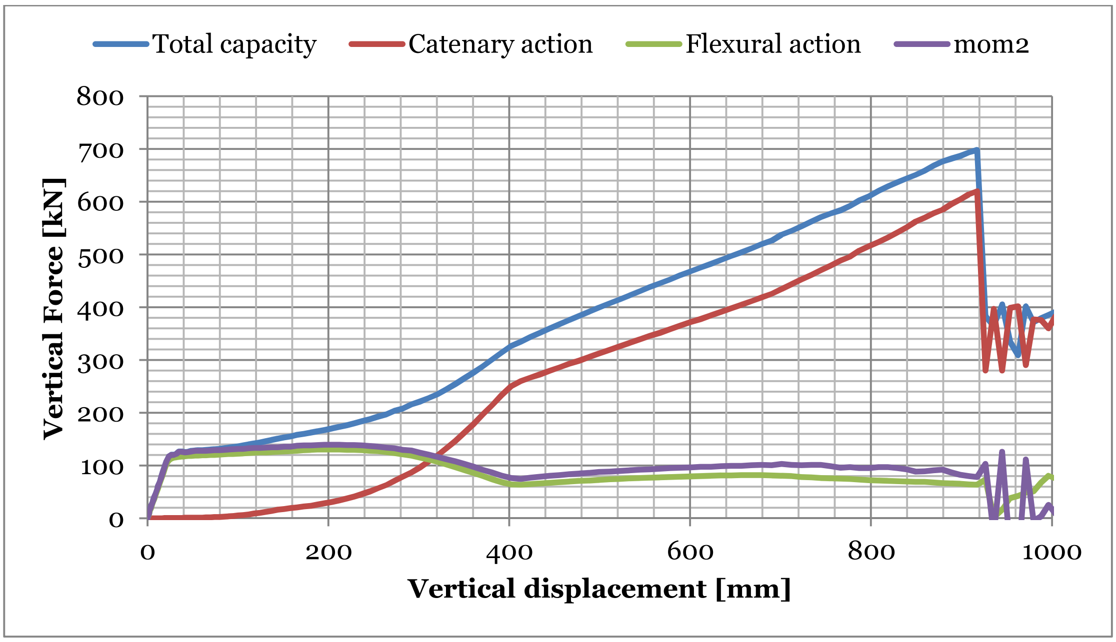
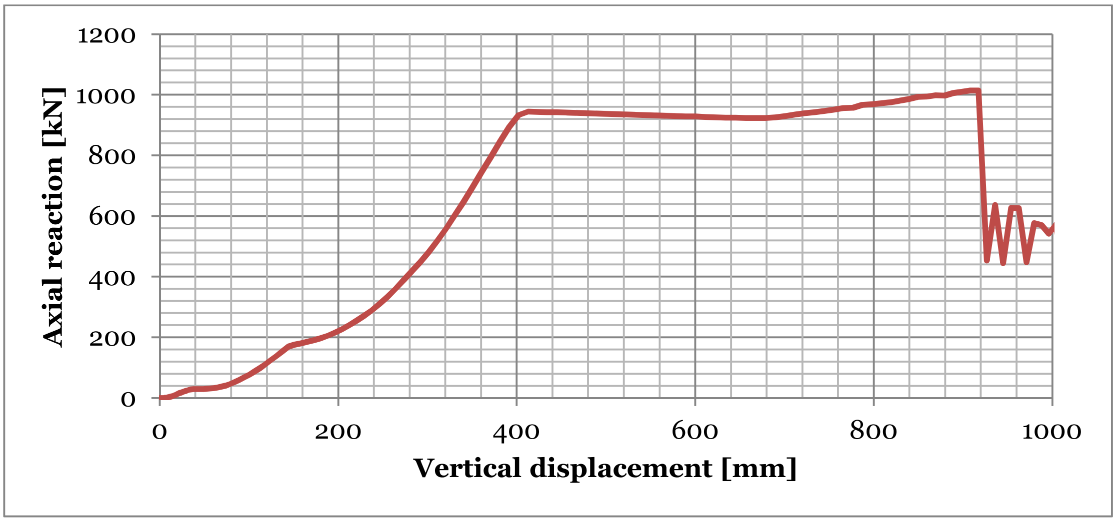


Figure 61



1200
1000
800
600
400
200
0
0
200
400
600
800
1000
Axial reaction [kN]
Vertical displacement [mm]

Figure 62

5.2 Results of numerical simulations

5.2.1 Extended end plate connection (No. 1)

The connection did not behave as it was expected, failing in brittle mode, fracture of the bolts. This simulation showed that the rotational capacity is not big enough for activation as much catenatry action as in the other types of connections.

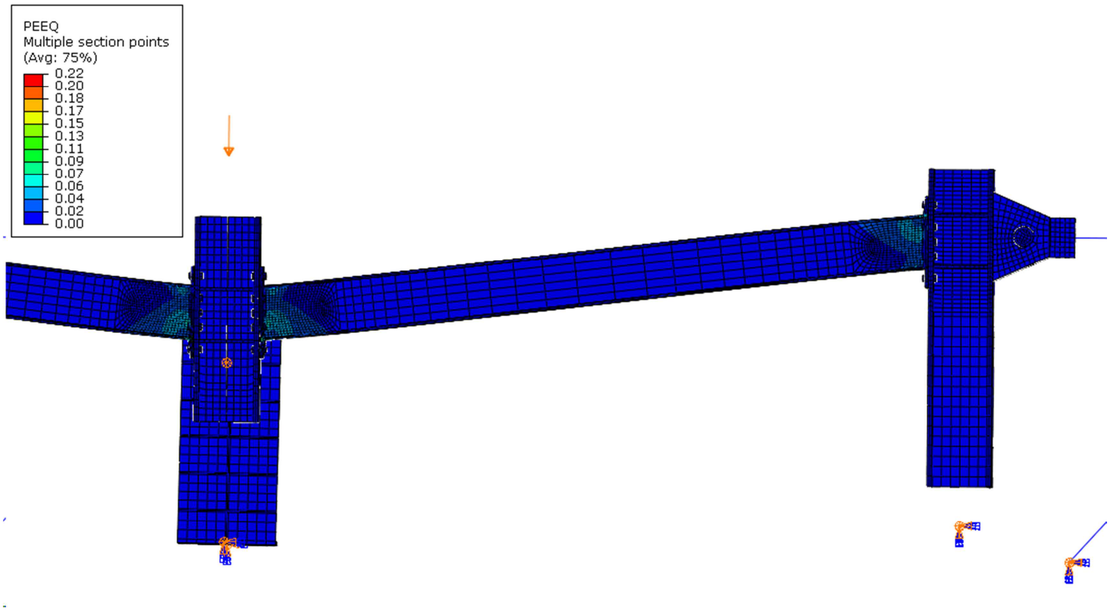


Figure 63

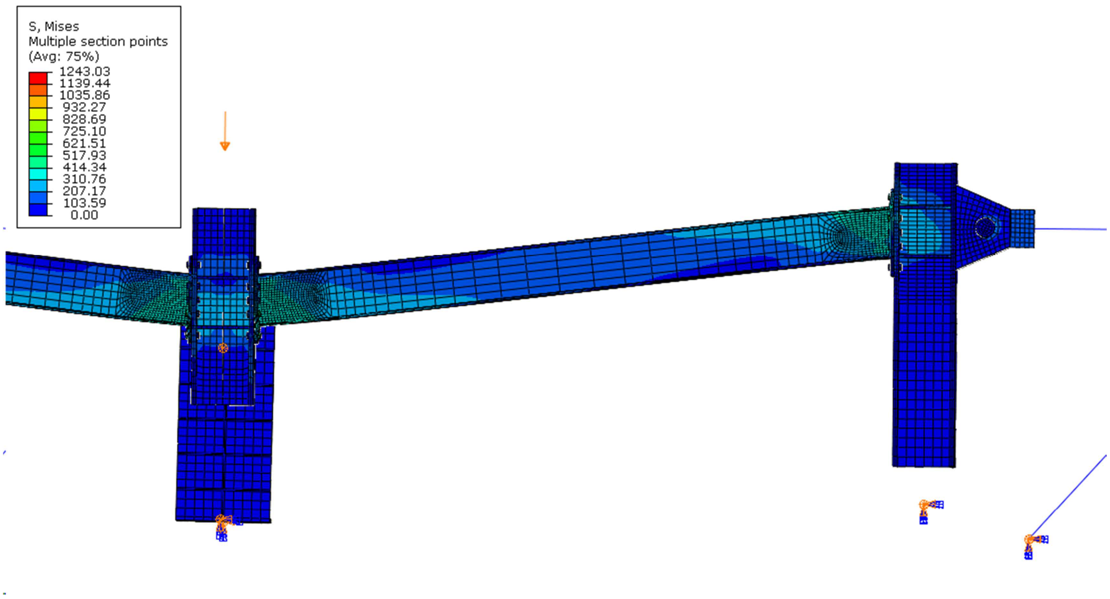


Figure 64

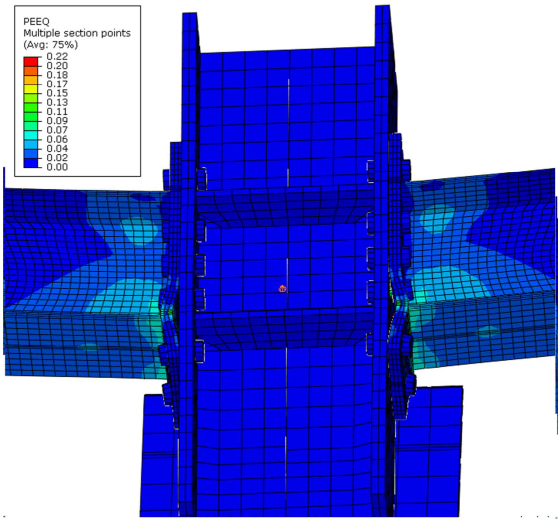


Figure 65

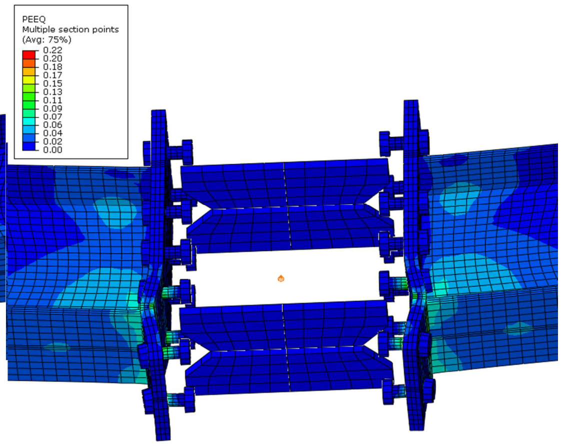


Figure 66 PEEQ in the bolts before failure

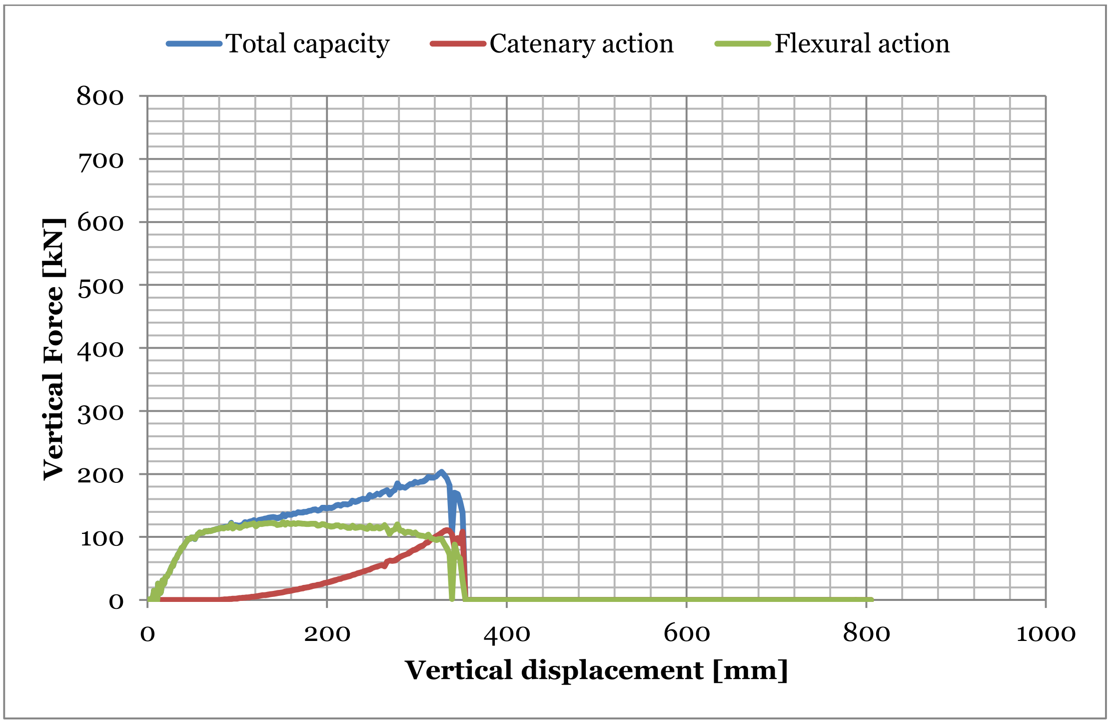


Figure 67

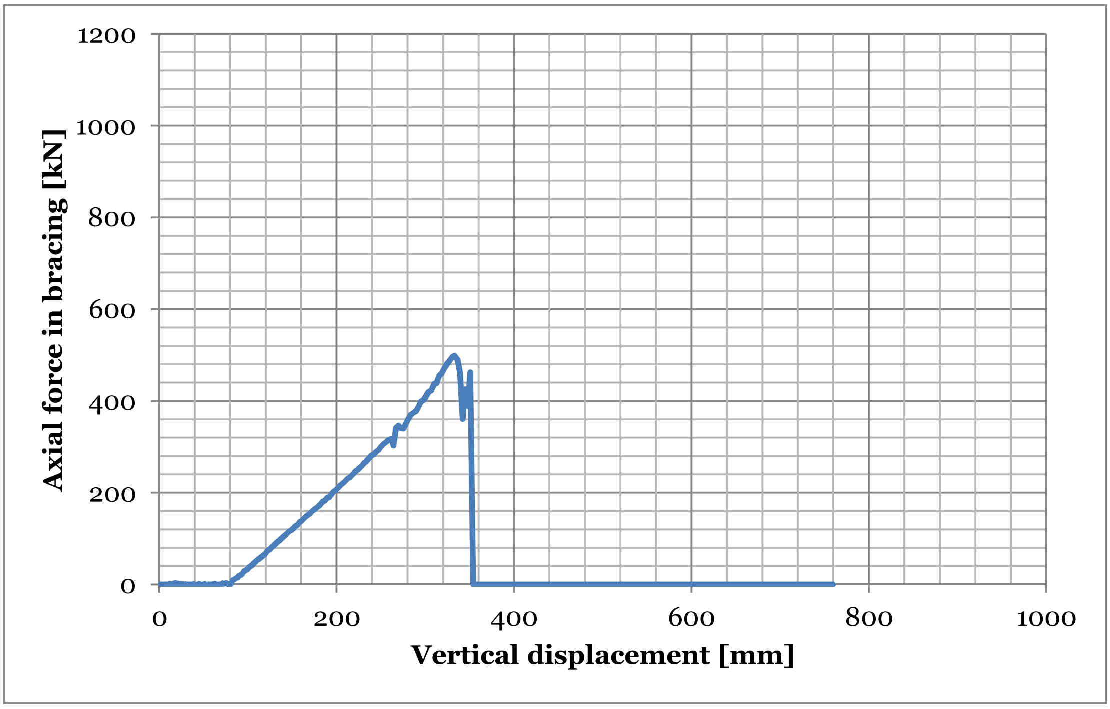


Figure 68

5.2.2 Dogbone connection (No. 2)

The connection behaved as expected forming plactic hinges in the reduced areas. This means ductile behaviour, but as the section area is reduced, the overall capacity is lower compared to full strength connections.

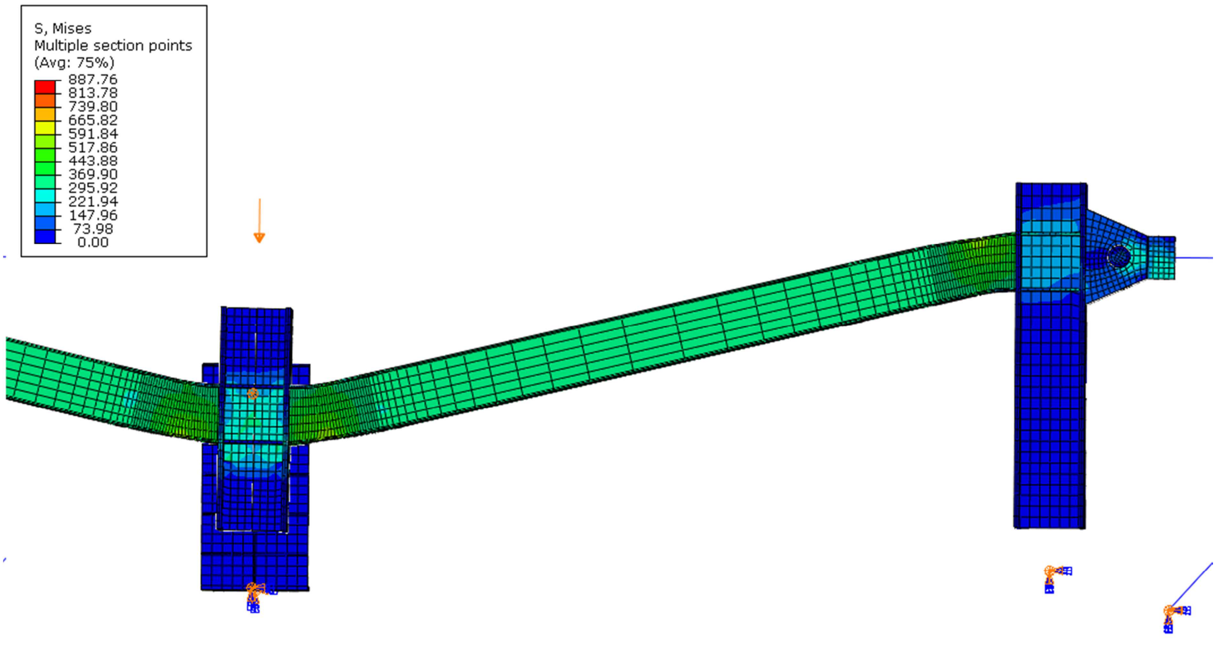


Figure 69

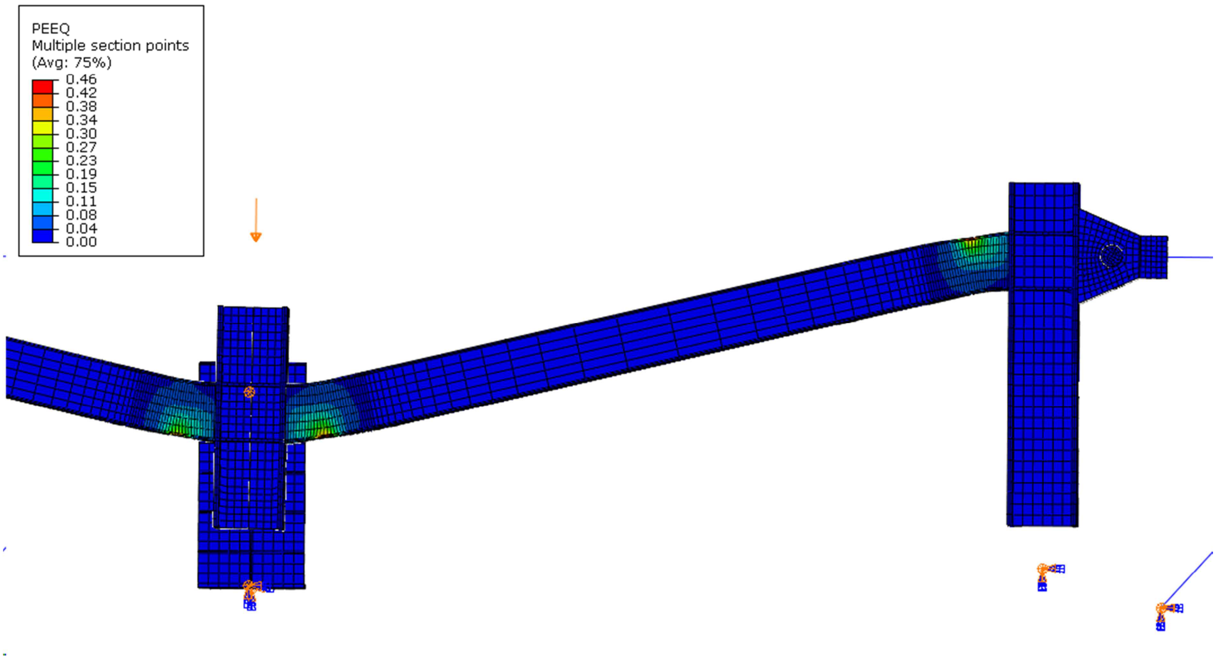


Figure 70

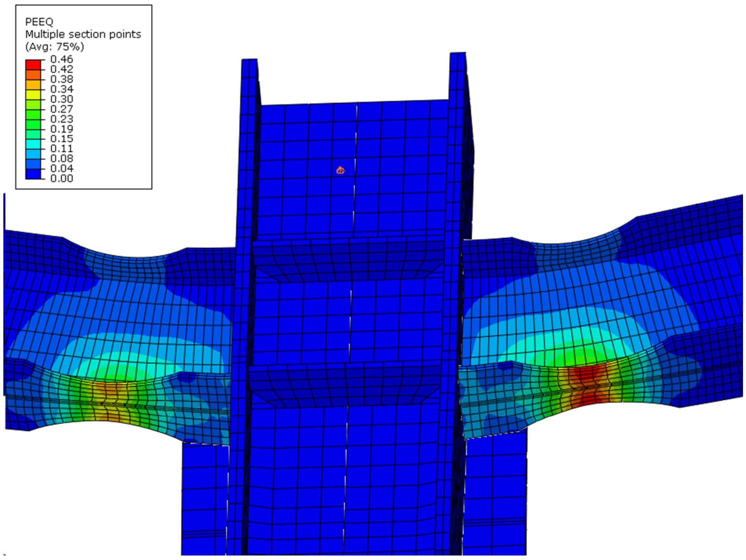


Figure 71

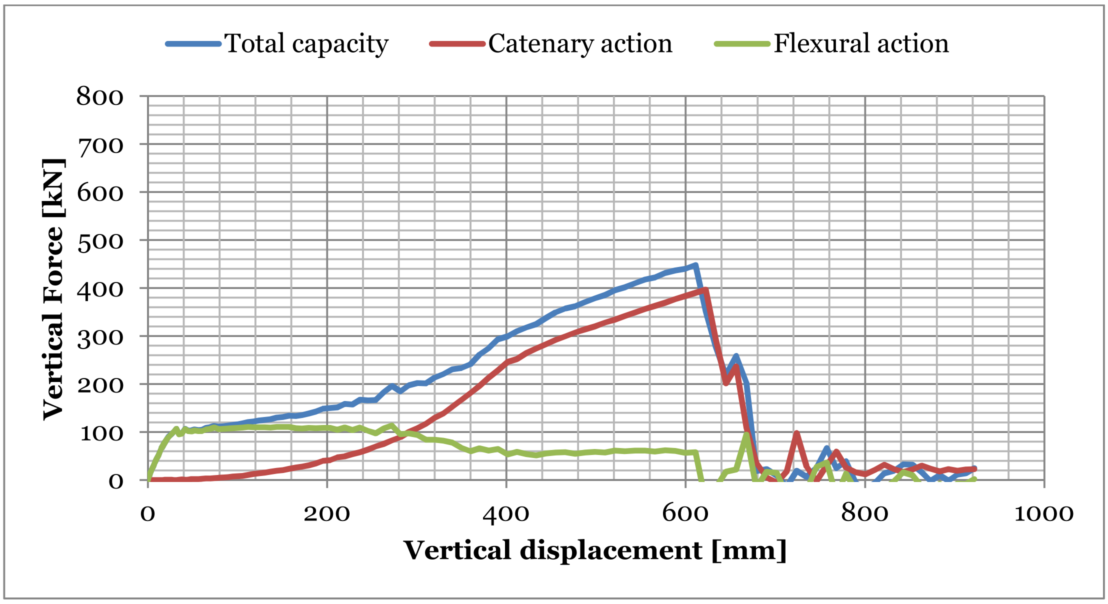


Figure 72

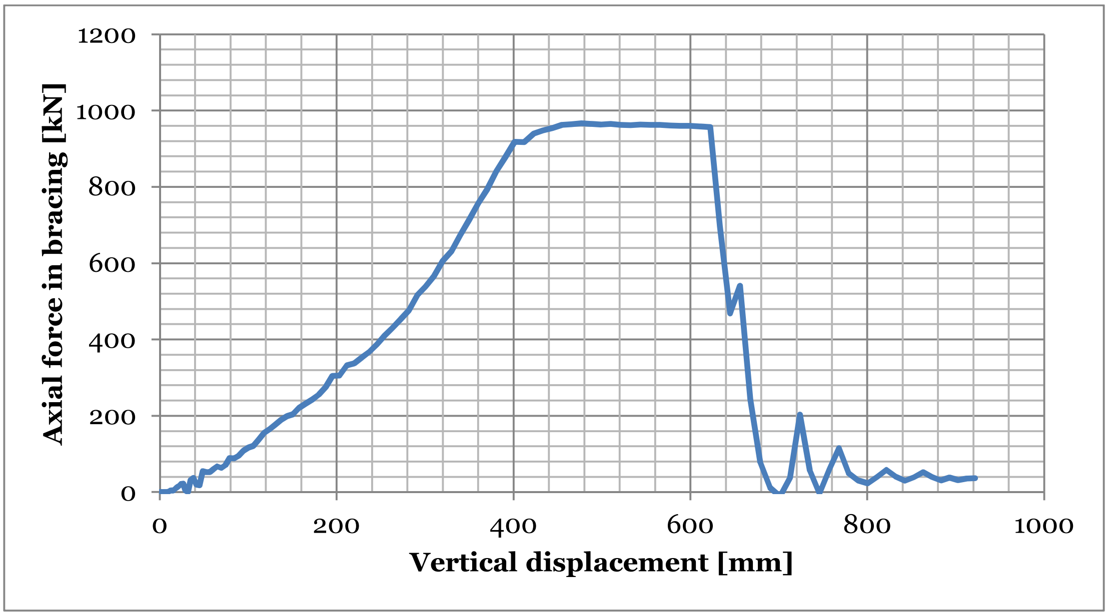


Figure 73

5.2.3 Coverplate connection (No. 3)

This type of connection shows the best performance from the selected types. Plastic hinges are formed in the beam, which is very ductile mode.

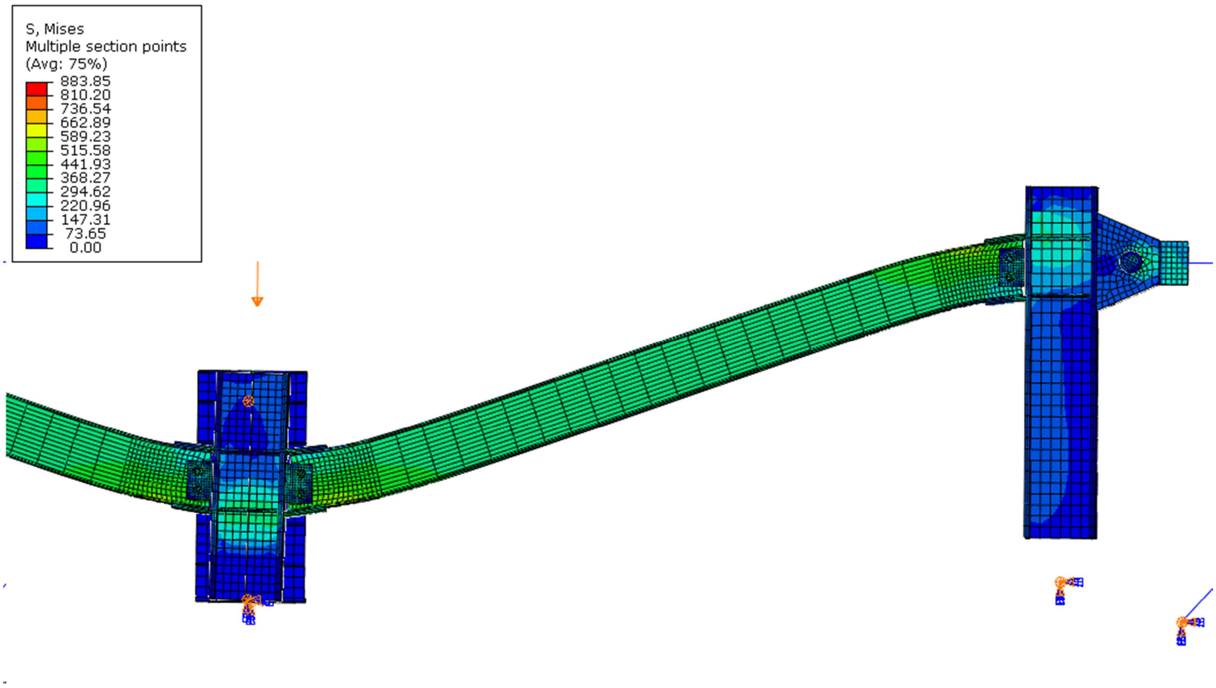


Figure 74

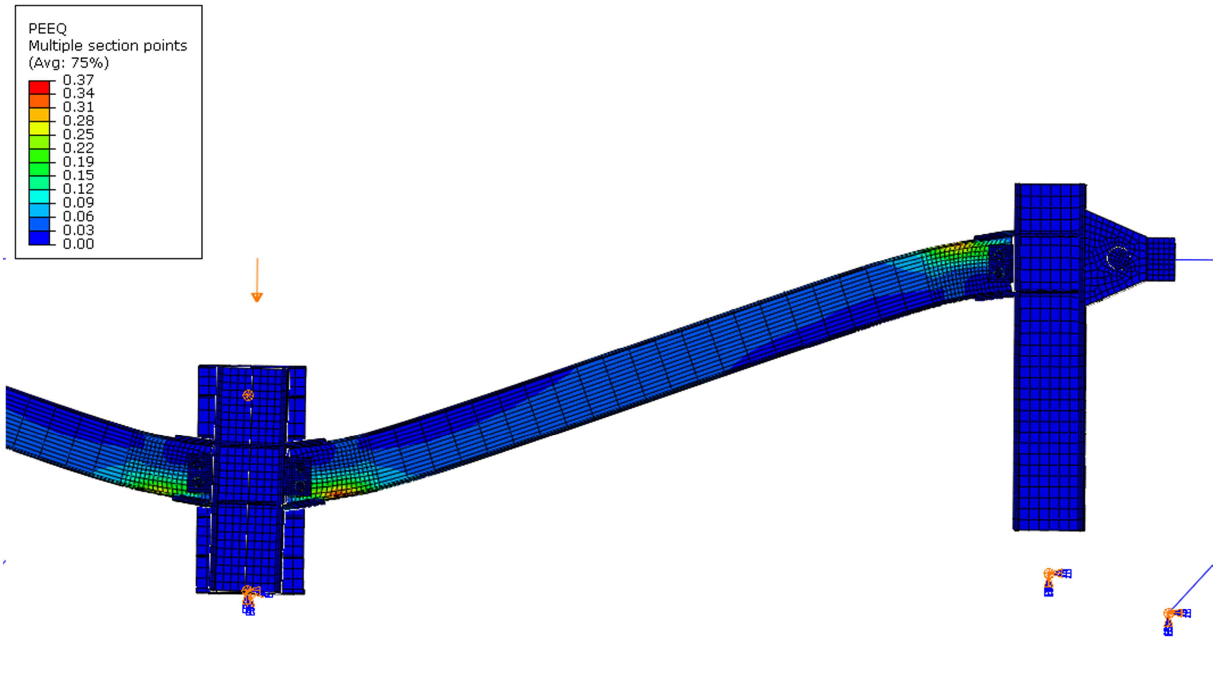


Figure 75

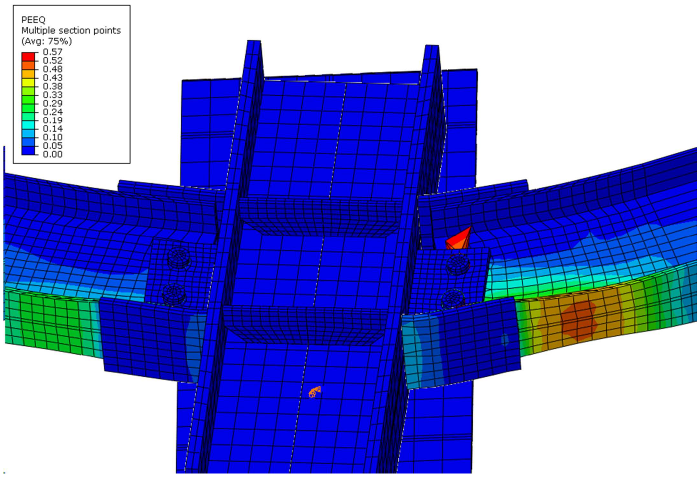


Figure 76

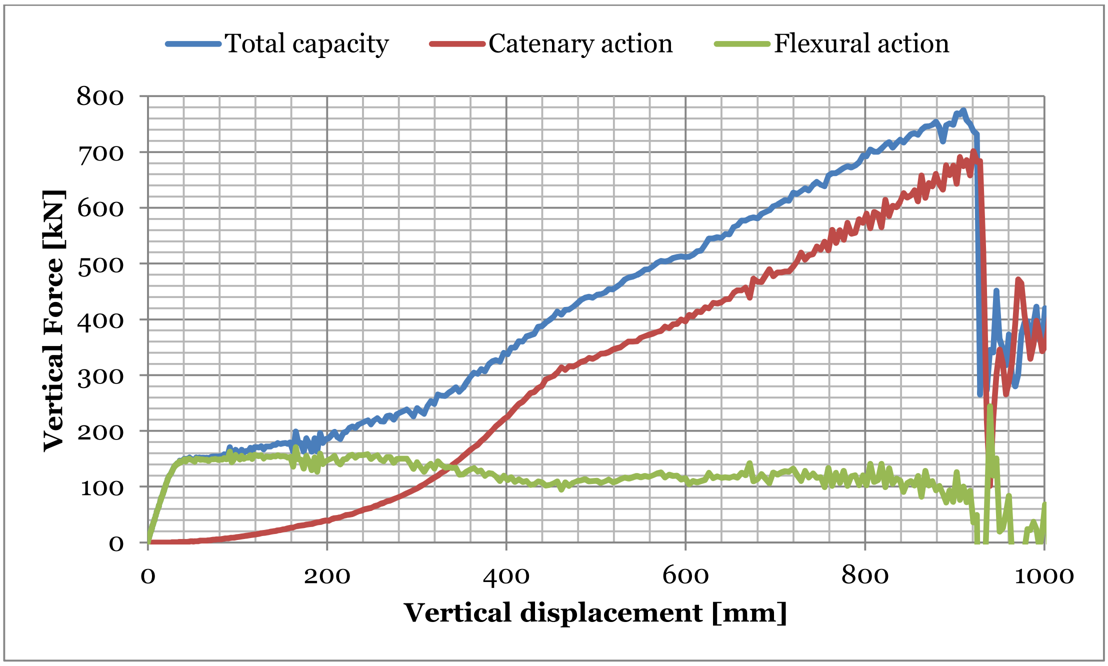
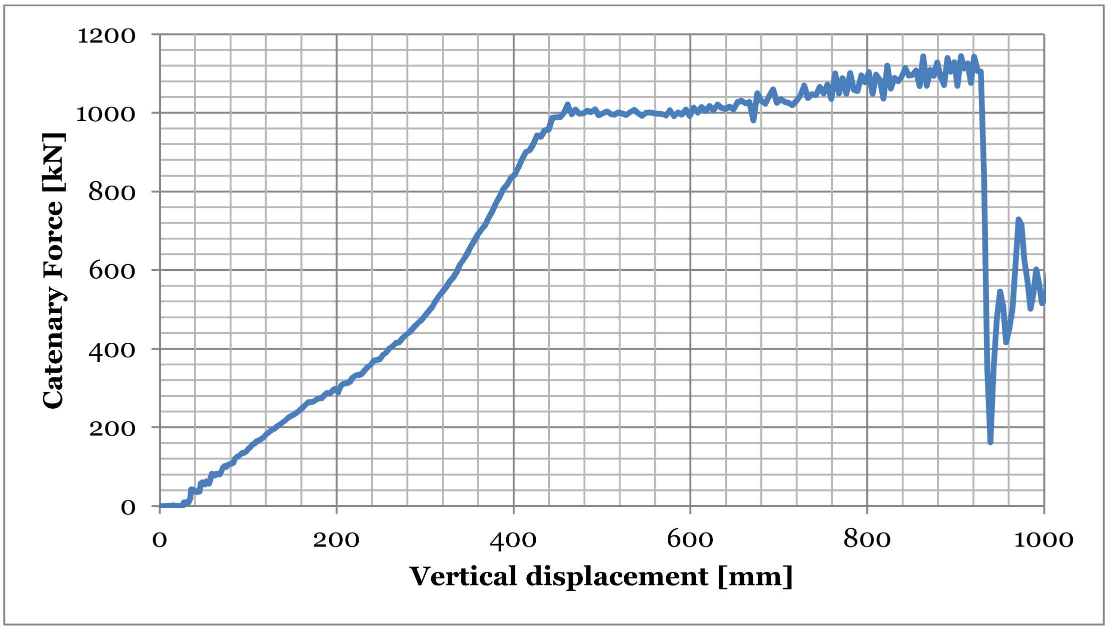


Figure 77



1200
1000
800
600
400
200
0
Catenary Force [kN]
0
200
400
600
800
1000
Vertical displacement [mm]

Figure 78

5.2.4 Haunch connection (No. 4)

This type of connection behaved as expected, forming the plastic hinges in the beam. In no other locations plastic deformations were observed.

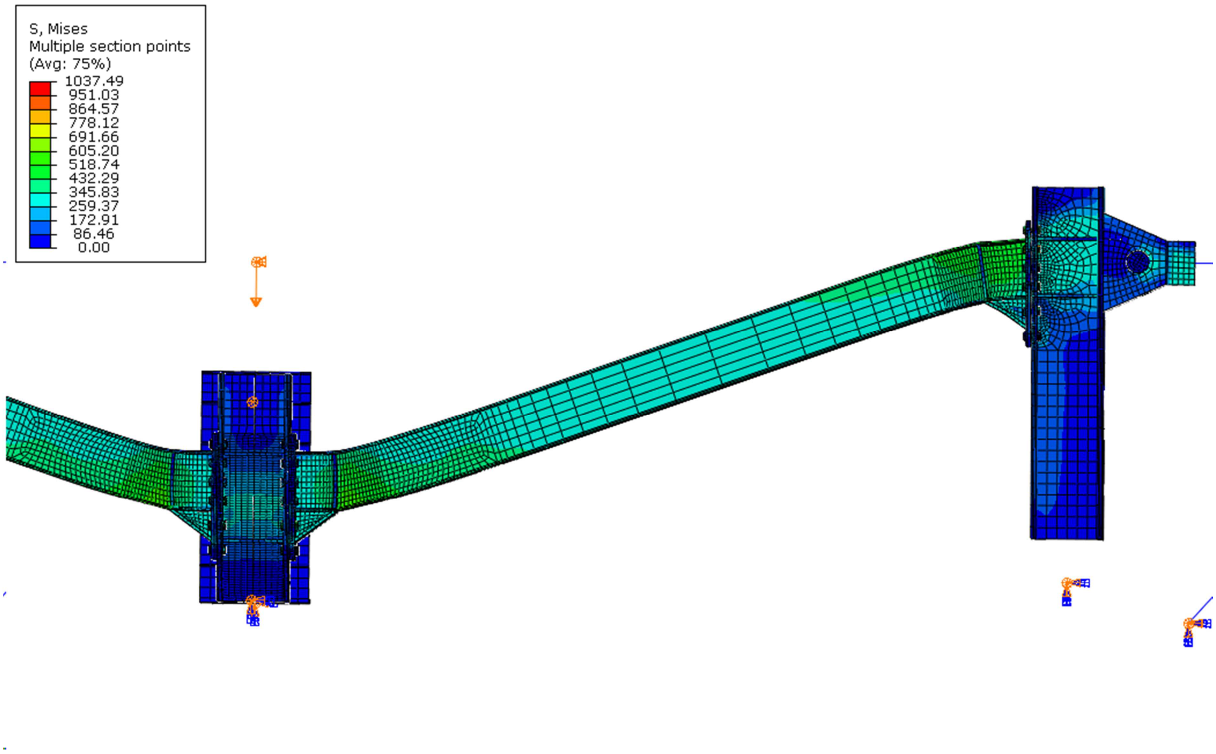


Figure 79

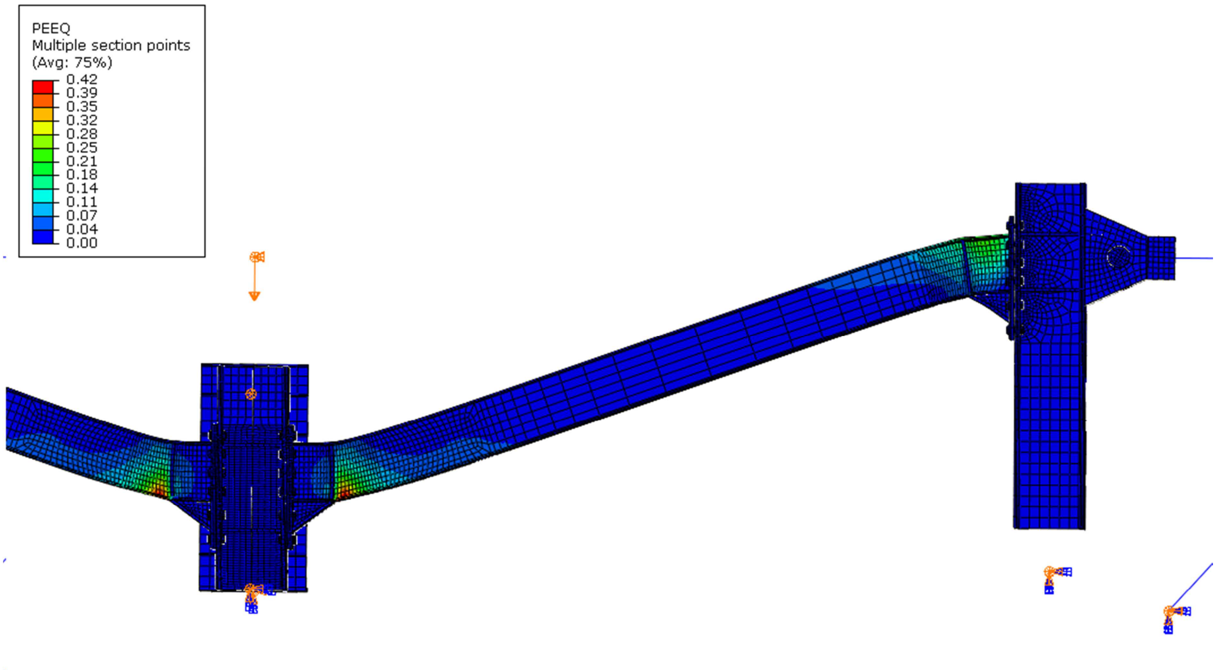


Figure 80

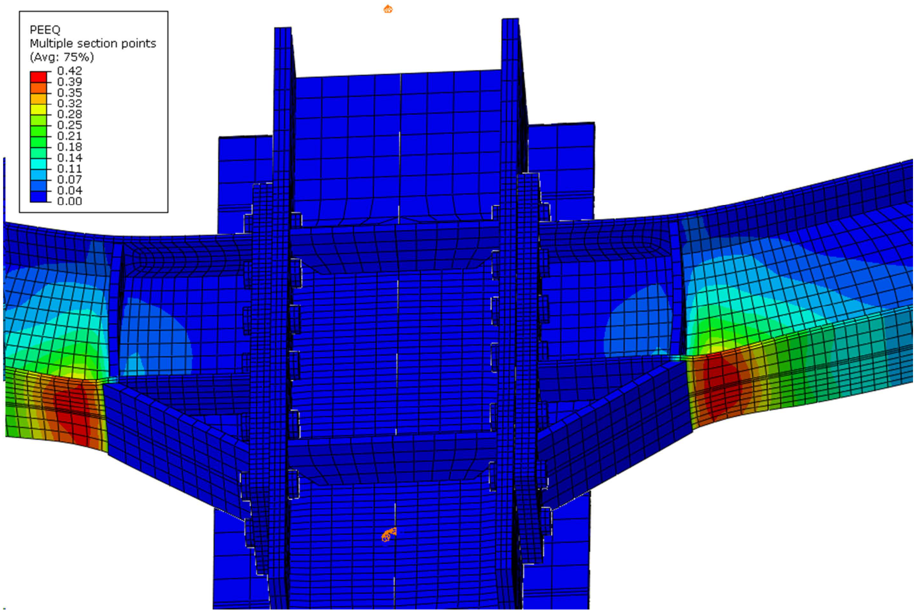


Figure 81

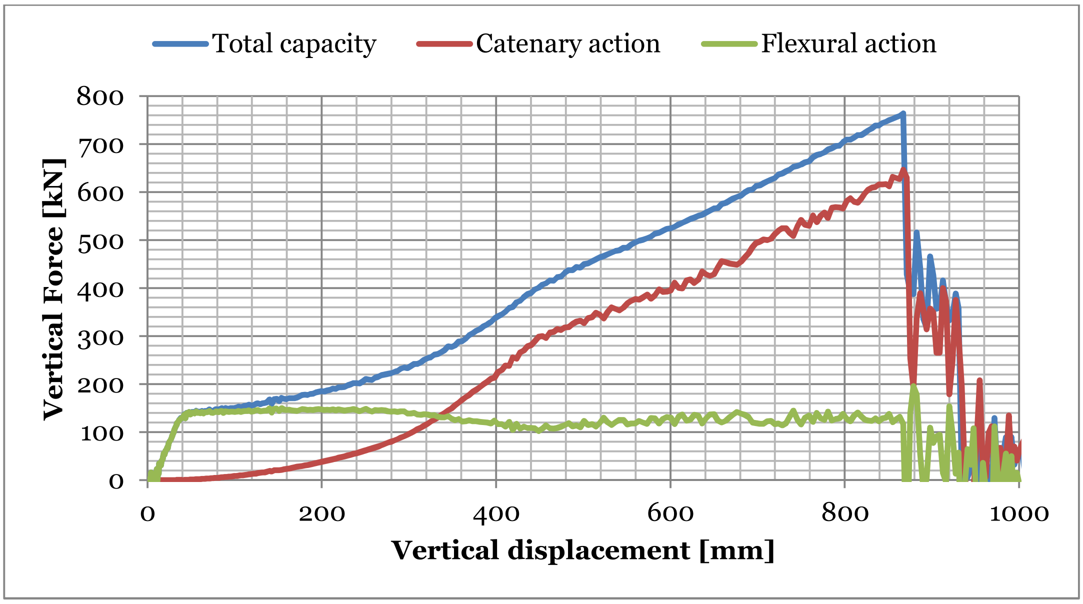


Figure 82

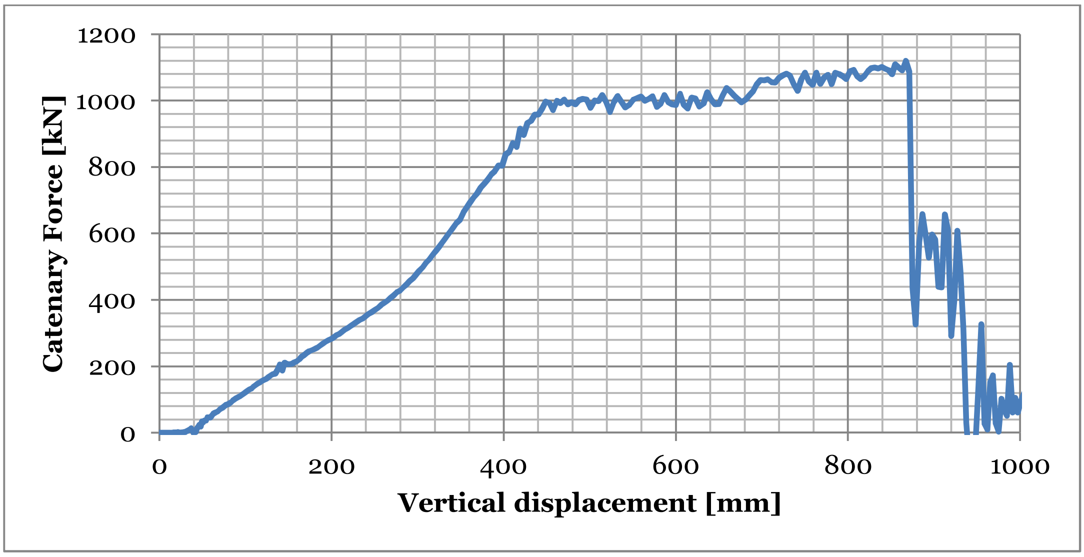

1200
1000
800
600
400
200
0
Catenary Force [kN]
0
200
400
600
800
1000
Vertical displacement [mm]

Figure 83

5.2.5 Comparision

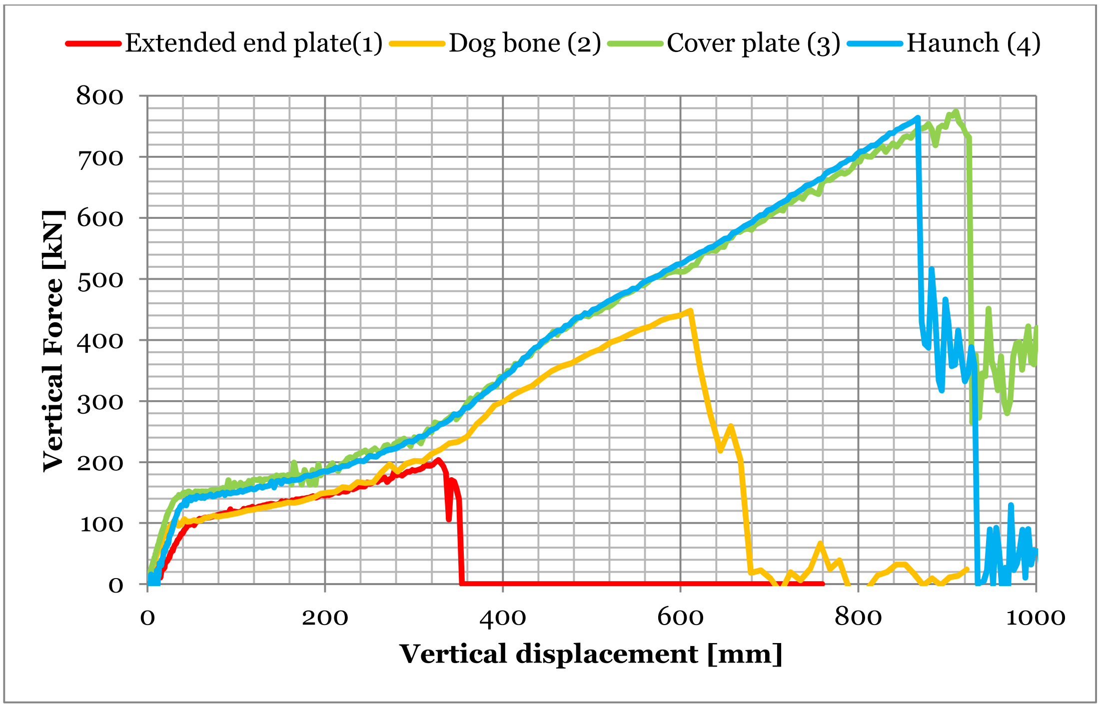


Figure 84

6 Conclusion

The main purpouse of this thesis was to explore how different types of connections behave in case of column loss. The results shows similarities with those from available publications related to this topic.

It was planned that this thesis will include also direct experimental results of the connections behaviour. Unfortunately the experiments were delayed and therefore direct validations of the models could not be performed. Anyway the autor thinks that a valuable work was performed and hopes for successful developement in the future.

From the four connections author recommends coverplate connection because of its low cost and best performance. Author is a bit surprised that the endplate connection was not ductile and therefore the catenary effect could not be fully exploited. If the failure mode in experimental test would not correspond to the numerical simulations, author recommends to revalidate the bolt material model and maybe to consider some tolerances, spacing or even modelling of the washers in the geometry.

The expected rotation capacities of these connections are varying from 0.1 rad till 0.3 rad. The demand from the global assessment was 63 mrad. That means all the connections were satisfactory.

The numerical simulations shows that catenary effect can increase very significantly the resistance, and therefore can be exploited in case of accident to improve robustness of the whole structure.