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MEMBRANE ACTION OF SLABS IN FRAMED STRUCTURES IN CASE OF ACCIDENTAL COLUMN LOSS

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Abstract

Recent disasters, emphasized the necessity to ensure the structural integrity of buildings under exceptional events. Accidental events may cause unforseen actions that lead to the loss of critical load bearing members. The loss of these load carrying members may initiate the phenomena of progressive collapse. Buildings should be provided with adequate robustness level in order to avoid progressive collapse. The term structural robustness is used to describe a quality in a structure of being indifferent to local failure, in which modest damage causes only a similarly modest change in the structural behavior. Significant area of research in structural engineering, particularly in the last years has been robustness of structures. The need for the concept of robustness lies in the fact that structural design codes are based mainly on the design of structural members or the consideration of the member failure modes and especially due to the difficulties in evaluation of the ultimate capacity under very large deformations. The alternate load path method provides the possibility to consider the effects of these accidental actions, without considering the nature of the action explicitly.

The study is part 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), that aims to provide guidelines for a proper design in preventing and mitigating progressive collapse. The activity within the present dissertation work has been supported in the framework of the CODEC project and represents namely the preliminary numerical simulation of the sub-assemblies. The main objective of this work is to perform full 3D Nonlinear Static Analysis for the loss of the central column of the sub-assembly in the case of only steel structure and in the case of composite concrete-steel structure, using Finite Element Analysis software, Abaqus. The document will present the modeling process starting from simple 2D steel models to complex and sophisticated 3D steel-concrete composite ones, considering composite action of the slab. Analysis were carried out to evaluate the vertical displacement and plastic deformations and to study the redistribution of forces, especially the development of catenary action (steel structure only) in beams and membrane action (composite structure) in the slab. Parametrical studies were developed in order to investigate the factors influencing the development of catenary and membrane forces in the case of large displacements. The contribution of the catenary and flexural action to the total capacity of the structure was studied, concluding the potential catenary action of structural members under extreme deformations, which can potentially increase the load-resisting capacity of beams compared to flexural capacity alone. By the evaluation

the potential catenary response, the actual behavior under extreme events can be more accurately considered and designed for. Results shown an increased performance of the structure if composite action between steel beams and reinforced concrete slab was considered.

As conclusion of the work, these results shown that structures are able to utilize catenary and membrane behavior to resist additional applied load past the flexural capacity, which are activated in the case of very large deformations. It is suggested that consideration of these forces be incorporated in future codes and design procedures, aimed to mitigate the probability of progressive collapse of building structures.

Organization of the present thesis

CHAPTER 1 presents a general introduction into robustness of structures, the methods available in the literature for measuring the robustness and the risk acceptance criteria for hazardous actions, the basic approaches to design for robustness in structural engineering, robustness aspects in forms of constructions and during the construction phase, effects of quality control with some examples from the history, modeling and type of hazards, progressive collapse analysis of buildings and progressive collapse typologies, examples to progressive collapse through the history and method preventing disproportionate collapse, collapse control design methodology, column loss scenarios and implications. This chapter is entirely dedicated for a deep understand of the topic and overview of the available literature about this. This chapter is not representing the aim of this thesis and its content can not be considered the originality of the author of this thesis, but it was necessary to give a global and complex overview of the related topic.

CHAPTER 2 introduces the evaluation of codes and standards related to robustness in Europe and around the world, giving details about robustness present in the Eurocodes and the US codes as well. In addition is presented the current research trends, gaps and potential contribution of the present thesis to the state of art.

CHAPTER 3 presents the definition of the refrence building structure and the design of the structure under conventional loading situation with brief description of the system, load evaluation and structural analysis. The design of the refrence building was not the aim of the thesis. Here is presented also the translation from the refrence building to the numerical sub-assemblies, which are the target for the numerical simulations present within this work.

CHAPTER 4 discusses the phenomena of catenary action in detail and the numerical and analysis procedure used within this thesis. After these, the modeling of the materials in the FEM is presented, followed by the detailed description of the modeling process and the achieved results for the scenarios and structures selected. This chapter is representing the aim of the present thesis.

CHAPTER 5 presents a parametrical study regarding the behavior of the structures related to parameters like slab thickness, shear connector location and number and reinforcement ratio.

CHAPTER 6 concludes the content of this thesis and the results obtained throughout this research, the effect of the parameters studied in the parametrical study on the behavior of the structure and the progressive collapse resistance. In addition future research activities and suggestions in progressive collapse mitigation are also proposed within this chapter.

CHAPTER 7 summarize the references used in the elaboration of this work.

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Chapter 1: Introduction

1.1 General introduction into robustness of structures

Recent incidents, such as natural catastrophes or terrorism attacks have emphasized the necessity to ensure the structural integrity of buildings under accidental events. Accidental events (e.g. fire, earthquake, blast, impact) may cause unforseen actions that lead to the loss of critical load carrying members. The loss of these load carrying members may lead up the phenomena of progressive collapse.

Buildings should be provided with decent robustness level in order to avoid progressive collapse. As the phenomena, **robustness** can be defined according to EN 1991-1-1, Part 1-7, to be *"... the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause."*

The term structural robustness is used to indentify the quality of a structure of being indifferent to local failure, in which small damage causes only a similarly small change in the structural behaviour. A robust structure has the ability to redistribute load when a load carrying member experience a loss of strength or stiffness and exhibits ductile rather than brittle global failure modes. A robust structure does not mean one which is over-sized. The ability to resist damage is attained through consideration of the global structural behaviour and failure modes so that the effects of a localised structural failure can be decreased by the ability of the structure to redistribute the load elsewhere, and so that the effects of the initial failure are limited.

Significant area of research in structural engineering in the last years has been structural robustness. The need for robustness rest in the fact that structural design codes are based mainly on the design of structural members or the consideration of the member failure modes. Furthermore, design codes and engineers may not always include all relevant design situations or relevance for the integrity of the overall structural performance. Robustness is a property which is not only associated with the structure itself, but needs to be considered as a product of several factors: risk, redundancy, ductility, consequences of structural component and system failures, variability of loads and resistances, dependency of failure modes, performance of structural joints, occurence frequency of extraordinary loads and enviromental exposures, strategies for monitoring and maintenance, emergency preparedness and evacuation plans.

In order to minimize the possibility of failures, as those mentioned above, many building codes consider the need for robustness in buildings and provide methods to obtain robustness. In fact, in all modern building codes, it is possible to recognize the statement (in this or a slightly different form): *„total damage resulting from an action should not be disproportional to the initial damage caused by this action."*

During the last decade, there have been considerable efforts to measure aspects of robustness. When modelling robustness, system effects are very important. However, the primary criteria in building codes are related to design and verification of sufficient reliability of components. It should be also noted that redundancy in systems is closely related to robustness. In principle, redundant systems are believed to be more robust than non-redundant systems; but this is not always the case as illustrated by the failures of the Siemens Arena and the Bad Reichenhall Ice Arena; for further information see *(Frühwald, Serrano et al. 2007), (Hansson and Larsen, 2005)* and *(Winter and Kreuzinger, 2008).*

Figure 1.1 Failure of the Bad Reichenhall Ice Arena, Germany, 2 January, 2006 (www.worldofkj.com)

The division of structures according to risk is an essential basis for further decisions about the level of design required, e.g., a hospital is required to survive significant events with lower probabilities than a residential building. It is well-known in the definition of the risk classification that the real issue is the relation between occupancy, evacuation time, the associated risk to occupants and factors such as societal expectations, rather than the size of the structure defined as the number of storeys or floor area. A method that considers only the structure size/use but neglects the consideration of occupancy levels may not have a logical meaning.

Enhancing structures robustness through the design and construction process requires consideration of numerous uncertainties. Some of these uncertainties are inherent; these would include material strengths, occupant and environmental loads or actions. Other uncertainties are knowledge-based and arise from limitations in modelling and insufficient databases. These uncertainties give rise to risk. Building risk cannot be eliminated and it must be managed through both technical and non-technical means. Risk management often involves difficult choices. The annual probabilities or frequencies of events that threaten most buildings and other structures in the built environment are very low. The fact that the probabilities are so small makes communicating risks to project owners especially difficult, even if the potential for human or economic losses are substantial. For further information about risk analysis and assessment see *(Ellingwood, 2001; 2007).*

1.1.1 Measurement of robustness

Robustness can be defined in several ways and on distinct levels of complexity or applicability. On a general level, robustness can be measured by the basis of decision analysis theory, by estimating both, **direct risk** and **indirect risk**. Indirect risk can be defined as risk from consequences disproportionate to the cause of the damage. Robustness of a structure can be defined therefore by the contribution of the indirect risk to the total risk. Such a risk based definition of robustness is proposed by *(Baker et al. 2008)* and the JCSS document "Risk assessment in engineering" *(JCSS, 2008)*. The basic framework for risk analysis is based on the following equation in which risk contributions from **local damages** (**direct consequences**) and **comprehensive damages** (follow-up/**indirect consequences**), are added:

$$R = \sum_i \sum_j C_{dir,ij} P(D_j | E_i) P(E_i) + \sum_k \sum_i \sum_j C_{ind,ijk} P(S_k | D_j \cap E_i) P(D_j | E_i) P(E_i) \quad (1)$$

where

$C_{dir,ij}$ consequence (cost) of damage (local failure) D_j due to exposure E_i

$C_{ind,ij}$ consequence (cost) of comprehensive damages (follow-up/indirect) S_k given local damage D_j due to exposure E_i

$P(E_i)$ probability of exposure E_i

$P(D_j|E_i)$ probability of damage D_j given exposure E_i

$P(S_k|\ldots)$ probability of comprehensive damages S_k given local damage D_j due to exposure E_i

The optimal design is the one minimizing the sum of costs of mitigating measures and the total risk *R*. A detailed description of the theoretical basis for risk analysis can be found in *(Baker et al. 2008)* and *(JCSS, 2008)*.

Three measures of robustness are described in the following: **risk-based**, **reliability-based** and **deterministic robustness index**. Two of three general definitions of robustness are based on stochastic modelling of the uncertainties (loads, strengths and models).

Risk-based robustness index

(Baker et al. 2008) proposed a definition of the robustness index based on risk measures. The approach divides consequences into direct consequences associated with local component damage (that might be considered proportional to the initiating damage) and indirect consequences associated with subsequent system failure (that might be considered disproportional to the initiating damage). An index is formulated by comparing the risk associated with direct and indirect consequences. The index takes values between zero and one, with larger values indicating larger robustness. The index is not always applicable with a full risk analysis, but can be considered as a useful indicator based on risk analysis principles. Since the direct

risks typically are related to code based limit states, they can generally be estimated with higher accuracy than the indirect risks.
The index accounts not only for the characteristics of the structural performance but also for the performance of the system after damage and all relevant consequences.
Furthermore, all measures which can be implemented either to improve structural performance with respect to robustness or to decrease the vulnerability (increasing component reliability), are accounted for by the index.

Reliability-based robustness index

(Frangopol and Curley, 1987) and *(Fu and Frangopol, 1990)* proposed some probabilistic measures related to structural redundancy, which also indicates the level of robustness. The index takes values between zero and infinity, with smaller values indicating larger robustness. They also considered a redundancy factor which takes values between zero and infinity, with larger values indicating larger robustness.

Deterministic robustness index

A simple and practical measure of structural redundancy and robustness, used in the offshore industry, is based on the so-called RIF-value (Residual Influence Factor), see *(ISO, 2008)*. In order to measure the effect of damage (or loss of functionality) of structural member i on the structural capacity, the so-called RIF-value can be defined sometimes referred to as the Damaged Strength Ratio. The RIF takes values between zero and one, with larger values indicating larger robustness.

Other simple measures of robustness have been proposed based on e.g. the determinant of the stiffness matrix of structure with and without removal of elements.

1.1.2 Risk acceptance criteria

The problem of definition of acceptance criteria for robustness is always related to the problem of **risk acceptance criteria**, and may be considered as a particular aspect of it. Methodology and procedures applied in order to assess the risk are also applicable to evaluate the robustness. A complete evaluation is an important factor for risk acceptance and consequently for robustness assessment. Although the entire problem is very complex, in order to reduce to a simple measure of societal expectations, a scalar function defined as **"Life** Quality Index" has been adopted. This allows to connect the various economic aspects and life-saving design criteria on the basis of economical indicators at national level. The Life Quality Index includes various types of risks in a unified manner. This allows for expressing the broad variety of possible factors as a single scalar value. The final acceptance criteria can finally be expressed in terms of a positive variation of the index itself. The acceptance criteria applied for robustness problems in existing structural regulations are defined as deterministic checks performed on the structural level. This is to ease the implementation of the adequate requirements in standard designs. The bases of such deterministic formulations are still empirical.

1.1.3 Approaches to design for robustness in structural engineering

The basic design methods that exist for designing for robustness in structures are relatively small in number, each authority implementing variations of one or more of these. Since the basic principles are common, the key methods will be discussed in more detail. The four basic approaches in design codes around the world are as follows according to *(Cormie, 2009):*

Tie-force based design methods: prescriptive (rule-based) approaches by which the structure is usually considered to meet the robustness requirements through minimum levels of ductility, continuity and tying.

Alternative loadpath methods: quantitative approaches where the structure is shown to have an adequate resistance to collapse to satisfy the code requirements.

Key element design: typically used as the method of last resort, a quantitative design approach for designing elements, the removal of which would lead to a collapse defined as disproportionate, for an accidental loadcase.

Risk-based methods: are used where the design fall outside normal limits.

Probability-based approaches: are more complex, in which some sort of quantitative or semi-quantitative model is built are now being developed. These typically desire to set up a given level of reliability in the structure, i.e., to demonstrate that the probability of failure is less than some defined threshold. These methods are also sometimes called **uncertainty-based methods**. The basic concept is that rather than taking a single value for a yield strength, a probability distribution for yield strength is carried through the analysis. When compounded with similar probability distributions for all other variables (including the magnitude of the applied load), a probability distribution is calculated for the failure of the structure. The uncertainty in this calculation is expressed by the standard deviation of this final probability function. Such methods are not currently implemented in codes and standards, although BS EN 1991-1-7 does contain an annexe which sets out a probability-based framework which may be used if required.

Tie-force based design methods

The main assumptions in tie-force methods are that they are a design method for low-risk structures and that for higher-risk structures, a more quantitative method of assessing robustness is needed. Tie-force based design, or tying, only requires the designer to detail the structure such that members are tied together in accordance with certain specified requirements. This is assumed to result in an increased degree of continuity, ductility, and load transfer to other parts of a structure such that the overall robustness of the structure is assumed to be enhanced. Tying is can be:

- in the horizontal members only (transverse and peripheral ties); or
- in both the horizontal and the vertical members.

Vertical tying was conceived as a method of dealing with large-panel structures, but is also helpful in framed structures in helping to develop vertical continuity in columns through which the load can be redistributed in the event of loss of strength or stiffness due to damage of the structure. It is, obvious that such tying must be adequate to develop sufficient tension to hang floors from the column above in the event of a column loss. Some researchers found that tying is inadequate in developing sufficient resistance against progressive collapse.

Tie-force based design is a prescriptive approach which explicitly assesses neither the robustness of the structure prior to application of the tying requirements, nor the level additional robustness that results from the application of these requirements. Tie-force based design methods incorporate the specification of which members are to be tied, the forces that the ties are required to resist and prescriptive detailing rules, compliance with which is assumed to provide sufficient robustness.

Tie force methods originated in the UK following the Ronan Point collapse in 1968. In the current Building Regulations Approved Document, Eurocode BS EN 1991-1-7 and the UFC criteria UFC 4-023-03 July 2009, the levels of tie force to be resisted are consistent. Tie force methods are a prescriptive, rather than deterministic or quantitative, approach, i.e. an approach in which compliance with the prescriptive rules is assumed to be sufficient for the structure to meet the requirement. In the US, prescriptive tie-force methods are classed as "Indirect Design" because the actual effects on the structure of member loss are not explicitly considered.

Several researchers have considered the efficacy of tying from the general concept to tying as it applies to specific forms of construction.

There is a general conclusion in most of the published literature that tie-force methods provide a minimum level of robustness, but that the level of robustness given to the building is unquantifiable. This originate the idea that methods are suitable for low-risk structures but that deterministic methods are necessary as an addition to qualitative methods for buildings which are represented by higher-risk.

Alternative load path methods

Alternative load path analysis is a deterministic or quantitative method by which robustness is demonstrated, rather than prescriptive or rule-based approaches. Alternative load path analysis is the analytical assessment of the structure under damaged conditions such as the partial or total loss of load-carrying capacity of a beam or a column, calculating whether the alternative load paths available in the structure are capable of redistributing the additional applied loads. The loss of a column will cause the gravitational load previously carried by it to be redistributed through the floor beams to the adjacent columns. If the residual load-carrying capacity in these columns (or the beam-column connections) is insufficient to sustain this additional load, failure will result in those elements and the collapse will be propagated.

Five mechanisms are fundamental to the robustness problem *(Cormie, 2009)* and are illustrated below; namely (a) catenary action in the structural frame, (b) shear deformation of transfer structures, (c) membrane action in structural slabs, (d) Vierendeel action, and (e) compressive arching in the beams and/or floor slabs. For most structures, the successful redistribution of load through alternative loadpaths it is based on the successful mobilisation of these behaviours.

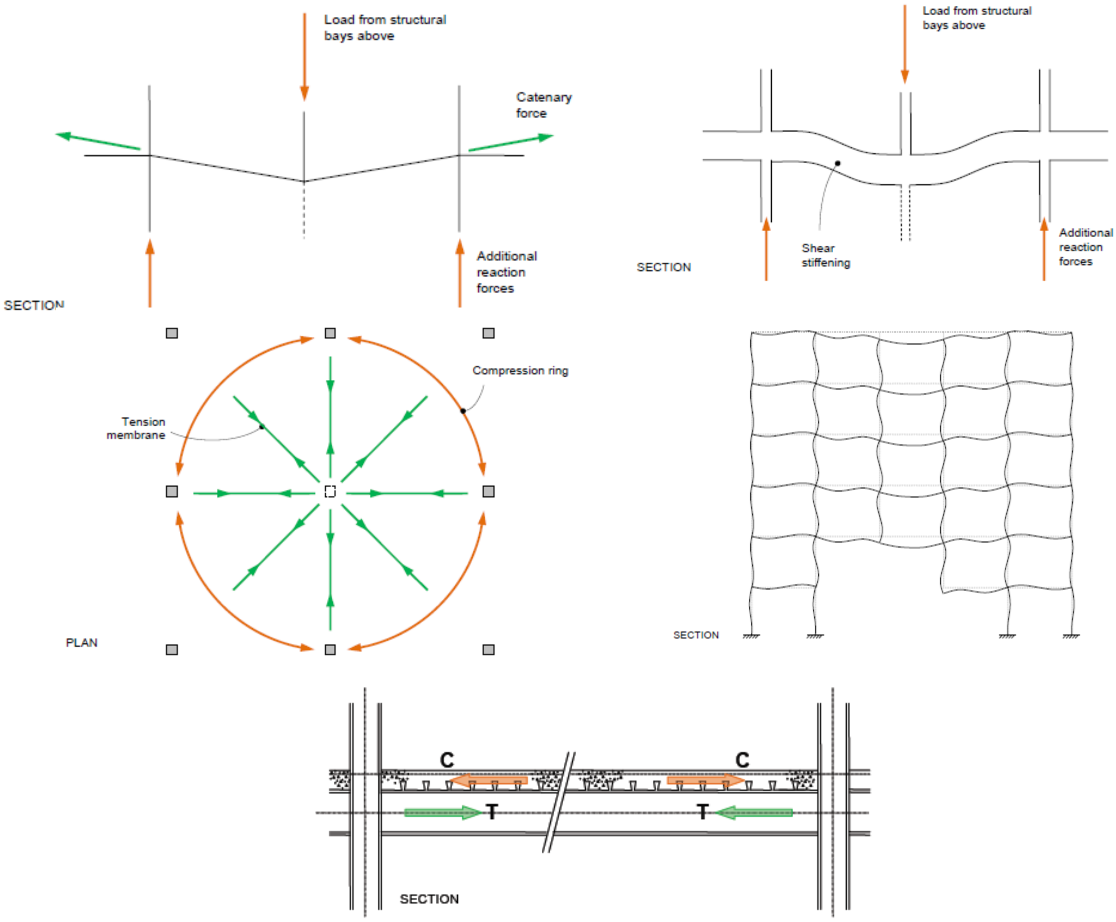


Figure 1.20 The five mechanisms fundamental to robustness problem (Cormie, 2009)

The beam-to-column connections above the damaged column are initially designed for hogging moments but after damage they are subjected to large sagging moments. At the distant connections, the hogging moments, may increase significantly to their initial values. If the system is not able to resist the increased bending moments, an alternative load path is needed to prevent progressive collapse *(Kang Hai Tan, 2012).*

Key Element design and Enhanced/Specific Local Resistance Methods

Key Element design can be implemented independently as a method for disproportionate collapse, but should preferably be used only if alternative load path analysis is not feasible. In other words, if the structure is not possible to be designed to ensure that the effects of the loss of a column are not disproportionate, the element must be designed to withstand the applied loads without its failure.

Key Element design is a scenario-specific design approach. The disadvantage of Key Element design is that exceedance of the capacity of the element results in a collapse which has under normal design practice been intolerable and otherwise the element would not have been designated as *Key*. Key Element design, usually represents a cliff-edge in the capacity of a structure, beyond which there is a decrease in structural stiffness or strength (and consequently a sudden increase in the damage) rather than there being a gradual reduction in stiffness or strength with increasing displacement/damage, i.e. a ductile response. Consequently, key element design should only be used as a method of last resort.

In the US, Key Element design is referred to as either the **Enhanced Local Resistance Method** or the Specific Local Resistance Method, depending on whether the resistance of the section is enhanced (by a multiple of the basic strength required by analysis), or designed to withstand a specified design event. In both the UK and US, Key Element design and Enhanced/Specific Local Resistance are intended only where robustness cannot be achieved by other measures. There is the possibility to define load criteria other than 34 kPa in the design of Key Elements for Enhanced Local Resistance, in which case in US terminology such an approach becomes the Specific Local Resistance Method. In principle, specific client requirements can be set to design Key Elements for loads other than the notional static load of 34 kPa.

Probability-based approaches

A number of authors proposed probabilistic (i.e. risk- and/or consequence-based) approaches as alternatives to deterministic methods. These papers propose uncertainty-based or statistical design approaches given the low frequency/high consequence nature of exceptional events such as terrorism, and some aim to account for the uncertainty associated with the producing event, the structure, the hazards and the consequences. Such approaches are useful in that they recognise the uncertainty in the basic variables and perform uncertainty analysis on a range of values with an assumed statistical distribution, rather than attempting to prescribe a single deterministic value to each variable. A probabilistic approach in respect of progressive collapse is therefore more difficult, particularly if terrorism-related. Uncertainty-based approaches have only been developed in relatively recent years and therefore they are not present in codes and standards. Uncertainty-based methods found in the literature are given below:

- Ellingwood - Load and Resistance Factor criteria for progressive collapse design' (2003);
- Faber - On the quantification of robustness of structures (2006);
- Maes - Structural robustness in the light of risk and consequence analysis (2006);
- Ellingwood - Structural reliability and performance-based engineering (2008).

1.1.4 Robustness in forms of construction

It must be mentioned that robustness is simply not achievable in all types of structures, or rather that each form of construction has safe and economic limits in terms of the robustness. For example, just because it is possible to span a gap by building a bridge out of paper and spaghetti does not mean that such a solution is as suitable one of steel bridge to span the same gap. The relative value of each material must be recognised and accepted; rather than trying to maintain a reasonable status by achieving the same level of robustness in all construction materials, the interest should be in maximising the robustness of each material to its own benefits.

Structural steelwork constructions

Structural steelwork is perhaps the most easily idealised in terms of the development of the mechanisms necessary for the development of resistance against collaps. However, there are concerns that the inherent level of robustness has been substantially decreased by the advances in structural steelwork over recent years due to greater efficiencies in the design of connections, lighter floor constructions permitting longer spans, the drive for efficiencies permitting quicker and cheaper erection and crane-led erection on site. There is, a considerable uncertainty about whether the connections in steel-framed buildings are capable to stop the collapse.

Steel-concrete composite constructions

In composite construction, the composite metal decking is usually shot-fixed to the top flange of the secondary beams. Shear strains along this interface, the effects of local stress concentrations around the shot fixings, the non-linear material response of the concrete and the effective transfer of flexural stresses require consideration when developing a model to describe the membrane and compressive arching effects of the floor slab when acting compositely. The robustness of composite floor slab construction is better understood thanks to experimental testing of composite systems by *(Jaspart, Demonceau et al. 2007, 2008)* which has been undertaken at the

University of Liege. *(Astaneh-Asl, 2001)* concluded from experimental studies that the floor system would be able to resist the effects of column loss including its dynamic effects, whereas *(Foley, 2006)* conducted analytical studies and found that the floor system would be able to carry the characteristic dead and imposed loads following a column removal, but without the full dynamic load effects. *(Sadek, 2008)* showed that the capacity of the composite floor system is marginal compared with the static characteristic dead and imposed load, and that the floor system is only able to develop 50% of the required capacity to absorb the loss of a column when dynamic effects are considered, in spite of the development of significant membrane action in the floor components and the consideration of moment-rotation parameters for the composite shear tab connections. These conclusions were confirmed by *(Alashker, 2010)*, who as well as extending the analytical studies and undertaking parametric studies to investigate the effect of steel deck thickness, quantity of reinforcing steel and shear tab connection strength, reports validation studies against tests carried out by *(Kim, 2001)*, *(Foster, 2004*) and *(Nie, 2008)*. The steel deck is shown to be the main source of the floor's capacity, carrying as much as 60% of the load at failure.

Reinforced concrete constructions

From all construction materials, in situ reinforced concrete construction allow the development of alternative loadpaths and of tying resistances given the monolithic nature of the material, although is highly dependent on the design and detailing of reinforcement. The ease with which this overall conclusion is drawn has arguably led to a tendency to assume robustness is implicit in reinforced concrete: as such relatively little fundamental research has been undertaken on the design and detailing necessary to ensure the potential robustness of the material is realised.

Both, BS 8110 and Eurocode 2 tend towards tying rather than alternative loadpath analysis as a means of satisfying the requirements, supplemented by key element design where necessary, since tying is more easily achieved in reinforced concrete due to the monolithic nature of the material.

BS 8110 and Eurocode 2 does not specify where the ties should be placed in the section, but that ties are most effective when placed in the bottom part. When placed in the top steel, there is insufficient ductility available in the bar to develop the tie forces. Reinforcement should be of at least ductility grade B, and minimum links are required to prevent the bars being ripped out of the structure and resulting in a non-ductile failure, particularly at laps between bars associated with tie rebar.

Consequently Eurocode 2 is insufficient to meet the requirements of the Building Regulations and Approved Document A, and those requirements of BS 8110 that are not covered by BS EN 1992-1-1 have been incorporated into PD 6687-1:2010 as non-contradictory complementary information.

1.1.5 Robustness during construction process

An objective of the design process is to prevent failures during the permanent structure stage, not giving importance to its temporary phase; unfortunately, these resulting sometimes with disaster. This is not acceptable, because a structure which is being built is as much a person's workplace as a permanent structure and avoiding failure of the temporary structure serves the same considerations as does the avoidance of failure at the permanent structure stage. Over the years, there have been many examples of structures that collapsed during their construction stage. Intuitively, it is understood that when a structure collapses, even partially, then it has failed. Another definition can be that a structure can be considered failed when it is not accomplish its purpose both functionally and structurally.

There are three **types of failure**, as follows:

a). **Overload failure** is usually characterized by the overloading of a component. In the absence of alternative load paths, this local failure may causes overloading of adjacent components and progressive collapse.

b). **Serviceability failure** prevents a structure being used to its full potential. For example, when a component in a building exhibits deflections under load that are enough to prevent things like doors opening properly, it is said to have suffered a serviceability failure. The problem can be remedied, so it is not catastrophic.

c). **Functional failures** prevents a structure from fulfilling completely the reason for it being built. The problem can be remedied, so it is not catastrophic.

In the construction phase, the **functional** and **serviceability failures** are usually not considered because, they are related to end use. Therefore, the only type of failure considered here is **overload failure** where the effect of a component failure could have an effect disproportionate to the cause, i.e., the structure in its temporary state is not sufficiently robust. Currently, there is a little guidance on how engineers should provide the necessary stability during the construction phase. To make the construction case the governing design is not always a feasible approach. However, some design resources needs to be expended when a designer recognises that a structure in its temporary (construction) phase is vulnerable.

The first step in preventing disproportionate collapse is to recongnize the way in which the structure being designed can be built. This could allow situations where the construction phase could govern the design. It is necessary to take into consideration how vulnerabilities in it could be increased during construction, by: a).Excessive loads applied during construction; b).Incorrect sequencing of construction; c).Temporary weaknesses in a system; d).Temporary instability.

Each of the initiating actions listed above could lead, eventually, to "overload" of a component which could, in turn, lead to failure of that component. Where is insufficient alternative load path, the transfer of load to adjacent elements could lead to their overload, setting up a progressive collapse or loss of overall stability.

1.1.6 Effects of quality control and deterioration

Engineers are generally give importance to have good quality design, materials, construction, maintenance and durability of a structure. However, whether this knowledge is always practised is questionable because nearly 90% of structural failures have been caused by poor quality or human error; see *(Allan, 1992).* Considering the quality in preventing structural failure, many publications such as that *by (Ellingwood, 1987), (Blockley, 1992), (Thorburn & MacArthur, 1993), (Canisius, 2000) and (Ellingwood and Kanda, 2006)* on related topics have appeared at various times.
In recognition of their importance to good structural performance, codes of practice, such as the Eurocodes, have given importance to quality and durability. For example, in Annex B, EN 1990:1992 has provided a **method of adjustment of partial safety factors on materials** to reflect the level of **Quality Control** (QC) and **supervision**. A lower partial factor is recommended when the quality and supervision are better than "normal" and a higher factor is recommended when these aspects are lower than normal. Supervision and checking are some activities that help to control the quality. Deterioration, reversed or repaired, can be considered as a form of poor quality.
If a disproportionate failure did not result from poor quality construction or maintenance, or improper use, then it could be related to poor quality design, provided the cause of failure was not beyond the state of the art.

Examples of poor quality that affected robustness:

By *(Canisius, 2000),* there are two main issues to be addressed when controlling the quality of structures:
- "Human errors"; and
- Quality errors of materials and fabrication process.

As mentioned in the previous section, any robustness reducing effect can be considered as due to poor quality. Some examples of these are shown below:

a). Ronan Point, UK – Poor concept

Figure 1.2 Ronan Point collapse (www.wikipedia.org)

The Ronan Point failure (see Figure 1.2) was caused by a design that considered the friction between wall panels and floors as sufficient to resist lateral loads on the former. In the case of an internal explosion that lifted the ceiling floor of a storey, the inter-component friction became almost negligible, allowing a wall to slide off its "support". This collapse is due to poor quality design, although it may be argued that it was beyond the state of the art of the time. Following this disaster the world's first disproportionate collapse regulations came into being in the UK.

b). Officers' Mess, Aldershot, UK – Poor construction quality and instability under temporary conditions

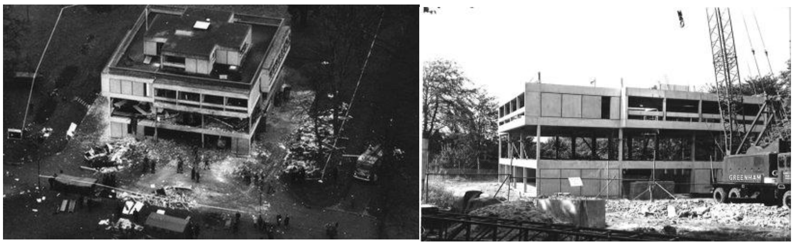


Figure 1.3 Aldershot building prior and after collapse (www.bbc.co.uk)

The progressive collapses during the construction phase have usually been attributed to construction errors. A significant pre-Ronan Point disproportionate collapse that occurred during construction was that of the Officers' Mess at Aldershot in the UK. On the 21st July, 1963 one of four identical buildings being constructed in the UK collapsed. These buildings consisted of three storeys and a penthouse, with a total height of 40 ft (see Figure 1.3). Each building had a concrete frame built with precast beams and columns, using in-situ concrete joints, and clad with precast concrete panels. At the time of the collapse, the building frame had been erected to its full height and many of the cladding panels had been attached. After the collapse it was decided to demolish the other three buildings but one of them collapsed before this was done. The initiation of the collapse of the structure was attributed to the local failure of a beam to beam or beam to column joint, due to a poor quality connection between them. Then, the general instability of the building which was under construction, due to the absence of any wall panels or bracings in the middle storey at the time, had contributed to the disproportionate collapse.

c). Pipers row car park, UK – Poor design and maintenance

Figure 1.4 Pipers row car park, Wolverhampton, England (www.corrosionengineering.co.uk)

The horizontal progressive collapse of a floor of this structure is attributed to both poor concrete area repair and poor structural design; for further information see *(Wood, 2003).* However, the unintended non-continuation of reinforcement between different areas of the roof slab prevented a more significant progressive collapse.

d). Collapse of wedding hall floor, Jerusalem, Israel – Bad structural modifications

Figure 1.5 Versailles wedding hall disaster, Jerusalem, Israel (www.bbc.co.uk)

This is an example of poor quality structural changes. The building had been extended to provide an additional storey on the existing flat roof. The new "floor", which was not strong enough for the imposed new live load, had survived by also resting on the "non-structural" partitions of the storey below. However, once the partitions had been removed later to create more unobtrusive space, the floor had sagged and then collapsed during a wedding celebration on 24 May, 2001. The floor collapsed through another floor, making it a progressive collapse via overloading of a region.

1.2 Failures due to accidental actions – Progressive collapse

The robustness design can be carried out against **unidentified hazards** and **identified hazards**. Whether a hazard was considered in a design or not would determine whether it is an identified or unidentified one.

1.2.1 Type of hazards

In the terms of robustness, hazard is an important threat to the integrity of a structure and the safety of people. Hazards may have natural or human origins.
The characteristics of a hazardous event (the point in time of the occurrence, its intensity and distribution) are usually unknown to the engineer. Explicit design values and requirements are given in Eurocode for only a limited number of accidental hazardous events that a structure can experience. These hazards include fire, earthquake, impact, internal gas explosions and dust explosions. The other hazards, the so-called unidentified actions, are addressed for lower consequence class structures by prescribing more or less general measures such the tying of parts of a structure together. However, it is the responsibility of the designer to provide a sufficiently safe structure by using the freedom he has been given by the code.

In Table 1.1 there are three **categories of hazards** shown:

- The first category is the type of hazards that are more or less given rise to by nature or general human activities. **Natural hazard** are those such as strong winds and earthquake. (Unintentional) man-made hazards include explosions. However, the difference between them is hardly relevant for structural design.

- The second category includes the type of **man-made** actions, such as vandalism and malicious attacks. To some extent it may not help to make a structure stronger to resist them, because it could generate more (re)action on the loading side. In such cases, design efforts to limit damage propagation can be more efficient. This type of hazard has become more important after the events of 9 September, 2001 at the World Trade Center attack.

- The third category includes **errors and negligence**. There is a direct link of this type of hazards with quality control and supervision. These hazards are best controlled by good supervision and quality control during all stages of the life of structure and by including general robustness measures suitable to deal with unidentified actions.

Hazard	Considered in Eurocodes	Category
Internal gas explosion	x	1
Internal dust explosion	x	1
Internal bomb explosion		1
External bomb explosion		1
Internal fire	x	1
External fire		1
Impact by vechicle	x	1
Impact of aircraft, ships		1
Earthquakes	x	1
Landslide		1
Mining subsidence		1
Tornado and Typhoons/Hurricanes/Cyclones		1
Avalanche		1
Rock fail		1
High groundwater		1
Flood		1
Storm surge		1
Volcanic eruption		1
Enviromental attack		1
Tsunami		1
Vandalism		2
Public disorder effects		2
Design or assessment error		3
Material error		3
Construction error		3
User error		3
Lack of maintenance (deterioration)		3
Errors in communication		3

Table 1.1 Overview of relevant hazards for structural safety

There are numerous hazards that could trigger the progressive collapse, each with different probability of occurrence: intentional or accidental explosion (e.g. bomb detonation, gas explosion), aircraft, car or debris impact, fire, natural hazard (earthquakes, extreme snow, extreme wind, extreme rain - flood),design/construction error. The different nature of the actions makes difficult to give a general model to interpret their effects on the structural behavior. Excepting the snow, which is a static load and the fire, which modifies the mechanical properties of the material, all other actions specified before are dynamic.

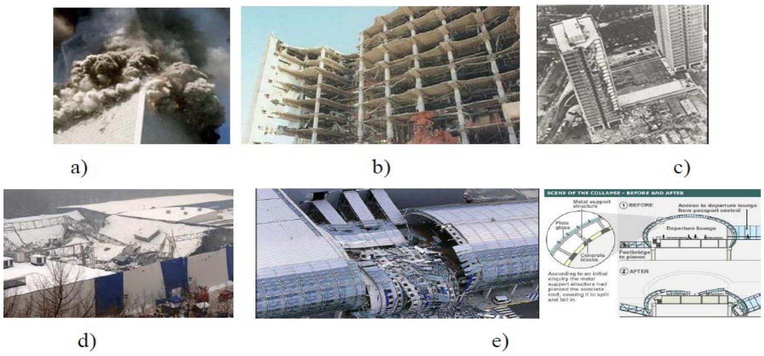


Figure 1.6 Progressive collapse: a) aircraft impact at WTC, 2001; b) Oklahoma City bombing, 1995; c) gas explosion at Ronan Point building, 1968; d) snow accumulation at Katowice/Poland, 2006; e) wrong steel-concrete connection details at Charles-de-Gaulle Airport new terminal, 2004 (www.wikipedia.org)

The dynamic features of the loads are different and, therefore, the consequences can be very different. Floods and landslides are characterized by the flow of a moving mass. Impacts, blasts and explosions are dynamic, and release a big amount of energy in a very short time. Figure 1.7 shows a typical time history of the pressure resulting from a gas explosion.

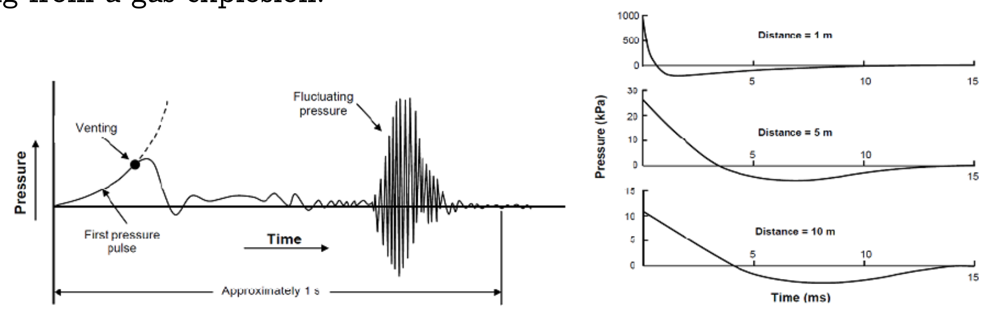
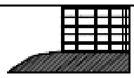
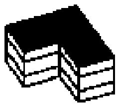
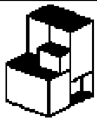
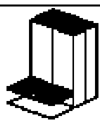
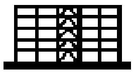
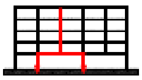
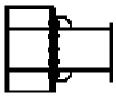


Figure 1.7 Typical time history of the pressure resulting from a gas explosion (left) and from detonation of explosives (right) (Marginean et al, 2013)

The bomb detonation creates a shock wave pressure that can be approximated by a triangular impulse load. The features of the loads are important when the design is of concern. Thus, in case of a gas explosion, the venting is important and can drastically limit the overpressure. On the other hand, the pressure released by a bomb blast decays very rapidly with the distance from the source of blast, and therefore the most efficient way to reduce the risk is creating a large safe perimeter or a large "stand off" distance. Considering the features of different loads, one can say the design of a building structure for a specific load can be ineffective for other loads. However, the seismic design or the adoption of the seismic design principles can prepare the structure for many other hazards. Past experience have demonstrated the resistance to progressive collapse is a multihazard issue (see WTC 2001 collapse, combination of impact of explosion and fire).

In the following, a multi-hazard design matrix (see Table 1.2) is presented by *(FEMA 577)* but adapted and restricted to site and building characteristics. A new type of hazard, i.e. explosion (including bomb and gas explosion) was also added.

One can say there are some structural characteristics that can efficiently improve the expected performance of the building and mitigate the risks of collapse associated with several hazards: type of structural system, avoidance of indirect supports (column on beam), ductile connections. Obviously, there is a strong need for systematic vulnerability studies and analyses in order to evaluate the performances of multi-story frame buildings to various natural and man-made hazards.

	Site and building characteristics		Hazard: Seismic	Flood	Wind	Fire	Explosion	Interaction
1	Elevated building site		-	+	0	0	+	Highly beneficial for floods and external bomb explosion, not significant for wind or fire
2	Re-entrant corner plan forms		-	0	-	0	-	Stress concentration at corners, irregular behavior in case of earthquakes; localized wind pressures, amplification of shock wave in case of external blast
3	Very irregular buildings		-	0	-	-	-	Indirect load paths, stress concentrations in earthquakes, explosions. Localized high wind pressure, aggravates evacuation in case of fire
4	Large roof overhangs		-	0	-	-	-	Vulnerable to earthquakes (vertical motion), wind and also adjacent external blast. Mai pose risk also in case of fire evacuation
5	Steel structural frame		+	+	+	-	+	When properly detailed, is recommended in seismic and high-wind zones. Good in flood with proper detailing. Vulnerable to fire if is not protected or well detailed and designed. Low vulnerability in case of blast and explosion, offers multiple paths.
6	Indirect load path		-	0	-	-	-	Very vulnerable for seismic, wind and explosion hazards because poor structural integrity increases likelihood of collapse. Fire may further weaken structure.
7	Ductile detailing of structure and connections		+	0	+	+	+	Provides good plastic response. The structure has large ductility and is more resistant to collapse in case of extreme loading

Note: The following convention has been used in the table:
+ indicates a desirable condition or beneficial interaction between the designated component/system and hazard
0 indicates little or no significant interaction between the designated component/system and hazard
- indicates an undesirable condition or the increased vulnerability of a designated component/system to a hazard

Table 1.2 Multi-hazard design matrix (Marginean et al, 2013)

1.2.2 Modelling of hazards

The principles of modelling the three types of hazard mentioned above are discussed below, individually, because the method of modelling is different for each of them. For most **natural hazards** the following set of models will be needed. (those for an earthquake selected as an example are presented within parenthesis):

- the occurrence model (average number of events per year or design life);
- a model to describe (the intensity of) the event (e.g. ground acceleration timehistory);
- a model to describe the effect of distance (attenuation law);
- a model to describe the effect of mitigating measures (e.g. base isolation).

Based on such models the exposure of the structure and corresponding occupants and contents can be estimated. In combination with other loads and events, a complete hazard scenario can be formulated as a basis for the estimation of the consequences. Within such an analysis, the effect of mitigating or preventive measurers can be incorporated.

In principle, the modelling of **malicious attack and vandalism** is more difficult. As already stated, the intention of the terrorist is destruction and the strength of a structure is the starting point. Past statistics may not be of much use here, except to evaluate what sort of intensity of action is to be expected. Of course, by good design, it is possible to make it more difficult to achieve the intended destruction of the structure. A key word for such good design is robustness.

Errors and quality related robustness is best controlled by general measures of robustness because, usually, gross errors could have very significant effects. If their potential occurrence is expected, then the best option is to control them before they materialise. These aspects were considered in the previous sections.

1.2.3 Consequences of failure

Failure means the total loss of functionality, both for components and systems. Failure of a system is given by the collapse of one or more components, triggered by an initiating event (exposure).

By **exposure** we understand an event during which the state of the component/ system deviates significantly from the ”normal” state (average state in probabilistic terms) in a direction such that it gets closer to the failure surface. Usually the deviation occurs with respect to a single state variable (or a group of correlated state variables), but this does not apply always.

Magnitude of the exposure can be defined as the **magnitude of the deviation**.

Consequences are possible outcomes of a desired or undesired event and must be considered during a risk assessment. Where consequences result from ‘hazards’, as in the case of robustness design, the event is undesirable. In the case of a repair of a damaged structure, the described event is a desirable one with positive consequences.

Consequence analysis:

The systematic procedure to describe and calculate consequences is called **Consequence Analysis**. Consequences are generally multidimensional. However, in specific cases they may be simplified and described with a limited number of components such as, for example, human fatalities, property damage, environmental damage and costs of disruption due to the unavailability of a facility.

A consequence analysis should start with a technical and functional description of the system considered. Important for this are the type of the structure, its intended use and foreseen activities and the number of people expected to be affected by a failure. If the structural response is significant, then it is necessary to distinguish between the direct response of the exposed elements and the subsequent behaviour of the rest of the structure. If the direct response of a structural element is "inadequate", then that element is considered as vulnerable. If the failure of vulnerable elements is followed by inadequate behaviour of the remaining part of the structure, then the latter is said to lack sufficient robustness. Both the assessment of direct and indirect structural behaviour may require quite advanced structural analysis which considers, non-linear effects, dynamic effects and temperature effects. The properties of the structural system (e.g. parallel plan arrangement) are expected to play a very important role in providing robustness.

Consequence classes:

Design for accidental situations or by designing for robustness, needs to be included only for structures the collapse of which, either in part or on whole, may cause particularly large undesirable consequences.

A convenient and easy way to decide whether to design against hazards such as accidental situations is to categorise structures or their structural components according to the consequences of an undesirable event (i.e. an "accident"). In Eurocode EN 1991-1-7, although the categorisation of structures is based on consequences of failure, they are not quantified and are given only qualitatively as follows:

- Consequences Class 1 Limited consequences;
- Consequences Class 2 Medium consequences;
- Consequences Class 3 Large consequences.

The appropriate robustness measures and the appropriate method of analysis to use for a situation may depend on its safety category in the following manner:

- Consequences Class 1: no specific consideration of accidental actions.
- Consequences Class 2: depending on the specific circumstances of the structure in question: a simplified analysis by static equivalent load models for identified accidental loads and/or by applying prescriptive design/detailing rules.
- Consequences Class 3: extensive study of accident scenarios and using scenario approach, risk analysis, dynamic analysis and non-linear analysis, if appropriate.

1.2.4 Progressive collapse

A **progressive collapse** is one which develops in a progressive manner related to the collapse of a row of dominos. A collapse may be progressive horizontally, successively from one structural bay to those adjacent to it and propagating through the structural frame. A collapse may also be progressive vertically, successive collapse of the columns supporting a number of floors due to the dynamic shock load. The term "progressive" refers to a characteristic of the behaviour of the structural collapse.

A **disproportionate collapse** is one which is estimated (by some measure defined by the observer) to be disproportionate to the initial cause. This is more a judgement made on observations of the consequences of the damage which results from the initiating events and does not describe the characteristics of the structural behaviour.

A collapse may be progressive in nature but not necessarily disproportionate in its extents, or vice-versa, a collapse may be disproportionate but not necessarily progressive. An example is the collapse of a statically determinate structure from the failure of a single member. In the case of a progressive collapse, different members of a statically indeterminate structure fail one after the other as they get overloaded with an accompanying redistribution of load.

The phenomena refers to the failure of one or a group of key structural load-carrying members (local failure) that give rise to a more widespread failure of the surrounding members and partial or complete structural collapse. Progressive collapse of building structures might be induced by a series of accidental and intentional events, such as false construction order, local failure due to accidental overload, damage of critical component by explosion, impact and earthquake.

Because of the catastrophic nature of progressive collapse and the potentially high cost of constructing or retrofitting buildings to resist it, it is compulsory that the progressive collapse analysis methods be reliable. For engineers, their methodology to carry out progressive collapse evaluation need not only to be accurate and concise, but also be easily used and works fast. Thus, many researchers have been spending effort in developing reliable, efficient and straightforward progressive collapse analysis methods in the past years.

1.2.4.1 Progressive collapse analysis methods for building structures

In this section, current methods for structural progressive collapse available in the literature are presented in two major categories, namely the **Direct Design** and **Indirect Design**. The design approach is primarily a function of the importance of the building (consequence class according to EN 1990). The probability of collapse, $P(C)$ due to the extreme load, H, can be defined using the following equation:

$$P(C) = P(C|LD)\, P(LD|H)\, \lambda_H \quad (1)$$

where

λ_H = rate of occurrence of the extreme load or hazard,

$P(LD|H)$ = probability of local damage given that the extreme load occurs, and

$P(C|LD)$ = probability of collapse given that local damage occurs.

The basic strategies for reducing the probability of (progressive) collapse are based on the minimization of the terms in the equation above. First measure refers to reducing the hazard or minimizing its intensity - λ_H. For blast event, for example, this can be done by creating a large stand off distance. Active fire protections can put out or slow the progress of a fire. In other cases, like earthquakes, engineers can do nothing to prevent or limit the seismic hazard. If the other two terms are referred, one can say the Specific local resistance method can be employed for reducing *P(LD/H)* while Alternate path method for reducing *P(C/LD).*

Indirect Design Method

It is a prescriptive approach and requires a minimum level of connectivity for structural members (see Figure 1.8). This can be achieved by the use of specific structural systems, the arrangement of the members, the ratio between their capacities and the capacity of connections in terms of strength and ductility. Seismic details for continuity and tying, that apply primarily lateral load resisting system, can be used as reference but extended also to gravity load resisting system.

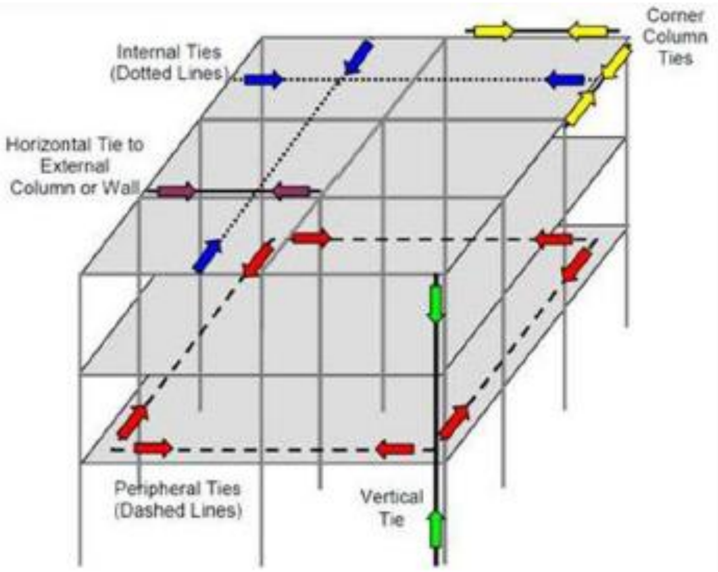


Figure 1.8 Different types of ties incorporated to provide structural integrity (DOD, 2005)

Direct Design Method

Considers the resistance of the structure and includes two methods:

- **Specific local resistance method**: first, critical members are identified and then their capacity to resist a specific load (such as the direct effect of an impact or explosion) is verified. This method is "threat specific method", as it does require the characterization of the hazard. This method is also referred to as **key element design**. Key elements are defined as structural elements whose notional removal would cause collapse of an unacceptable extent. They should therefore be designed for accidental loads, which are specified in several standards as 34 kPa. Such accidental design loading should be assumed to act simultaneously with 1/3 of all normal characteristic loading: $D+L/3+W_n/3$ (where D=dead load, L=live or imposed load and W_n=wind load).

Wind load simulates the effect of overall stability and can be replaced by global imperfection. The specific local resistance design is often the only rational approach when retrofitting an existing building. For blast and explosions but also for impact, which are dynamic load events by their nature, although the Finite Element Method (FEM) can be used in modeling the structural response, a more accurate prediction of the structural performance under extreme loading may be necesary. The Applied Element Method (AEM) is a new method that can investigate the structural collapse behavior passing through all stages of the application of loads: elastic stage, crack initiation and propagation in tension-weak materials, strain hardening effect in post-elastic range, element physical discontinuity, element collision (dynamic contact), and collision with the ground and with adjacent structures; see *(Dinu et al., 2010).*

- **Alternate path method**: first, it is assumed the loss of a structural member due to an extreme load event and then the capacity of the structure to gap over the missing member is investigated. This is a "threat independent method", as it does not require the characterization of the hazard. This method reduces the risk of progressive collapse by ensuring structural redundancy. Another advantage of this approach is that it improves the systems with ductility, continuity, and energy absorbing properties that are desirable in preventing progressive collapse. This kind of method is a way to ensure that alternative load paths exist to redistribute loads around a failed member and encourages redundant and ductile behavior. They usually start progressive analysis by removing one or more key vertical load carrying elements. Dynamic simulation for the collapse of World Trade Center is the earliest case of using nonlinear dynamic alternative load path method to carry out progressive collapse analysis of building structures available in the literature, which is even earlier than the publications of the two guidelines in progressive collapse analysis, i.e., GAS and DoD guidelines.

The alternative load path method defined in the two guidelines allows for four type of analyses, namely **linear static analysis (LS), nonlinear static analysis (NS), linear dynamic (LD)** and **nonlinear dynamic analysis (ND)**. Different levels of analyses have different procedures, and could achieve results with different levels of accuracy.

Linear static analysis (LS) is based on small deformations, in which the material is treated as linear elastic. When carrying out the progressive collapse analysis with linear static analysis method, the procedure is as follows:

1) Establish the finite element model; 2) Remove the key element;

3) Apply the combination of dead loads and live loads as defined in GSA and DoD guidelines and perform the analysis;

4) Analysis is completed if none of the structural elements fails. If there is a failed element, remove it and redistribute its loads, redo the analysis;

5) Evaluate the results.

From the above procedure one could easily see that this method is very simple and easy to be performed. However, it does not consider dynamic effects and material nonlinear behavior; therefore, it can only be used for analyzing simple structures with predictable behavior.

In the **nonlinear static analysis (NS**) method, both the material and geometry are treated as nonlinear. A load history from zero load to the full factored load is applied to the structure with a removed vertical load-bearing element. The procedure is as follows:

1) Establish the finite element model; 2) Remove the key element;

3) Apply the loads using a load history that starts at zero and is increased to the final values defined in GSA and DoD guidelines and perform the analysis;

4) As the analysis is performed, evaluate the damage of the structural components, if an element is shown to fail, redistribute the element's loads and restart the analysis; The analysis is completed when the structure collapses or after the full apply of the load;

5) Evaluate the results. The advantage of this method is its consideration of material nonlinear behavior. However, it still does not consider the dynamic effects such as amplification factors, inertia, and damping forces. Moreover, it is relatively complex and time consuming. Thus, it cannot be effectively used for progressive collapse analysis.

Nonlinear dynamic analysis (ND) assumes nonlinear behavior material and geometry. A dynamic analysis is performed by instantaneously removing a vertical loadbearing element from the fully loaded structure and analyzing the resulting motion. It is believed that a 3D ND analysis will yield the most accurate results for an alternative path load analysis. Its procedure is as follows:

1) Establish the finite element model; 2) Prior to the removal of the key element, bring the model to static equilibrium under the combination of the dead loads and

live loads as defined in GSA and DoD guidelines and perform the analysis; 3) With the model stabilized, remove the appropriate key element instantaneously; 4) Continue the dynamic analysis until the structure reaches a steady and stable condition or collapses; 5) Evaluate the results. Obviously, this method not only includes dynamic effects, but also includes material and geometry nonlinear behavior. Thus, it can provide the most realistic results. It also has some disadvantages, such as its complexity and time consuming. It is also very sensitive to the accuracy of the numerical model.

There are different accidental loads, lateral loads and combinations of loads for which the building stability should be checked within the design codes; see below:

Standards	Load combinations after notional member removal	Accidental load
BS	(1±0.5)D+L/3+W_e/3	34 kPa
Eurocode 2003		20 kPa
DOD UFC 4-023-03	D+0.5L net floor uplift	
DOD UFC 4-010-01	D+0.5L net floor uplift (0.9 or 1.2)D+(0.5L or 0.2S)+0.2W_e (nonlinear dynamic analysis) 2.0((0.9 or 1.2)D+(0.5L or 0.2S))+0.2W (static analysis)	
NYC 1998, 2003	2D+0.25L+0.2W_e	
GSA	2(D+0.25L) static analysis D+0.25L dynamic anaylis	
D, L, W_e, S = dead, live, wind and snow loads; *Q_{sk} = Characteristic value of accidental action;* *G_k, Q_k = Characteristic dead, imposed loads per unit area of the floor or roof;* *Ψ is a load reduction factor which, when multiplied with Q_k, gives the fequent value of a variable action.*		

Table 1.3 Loads and combination of loads for alternate load path method

Because the transition from the original structural configuration to the damaged state is assumed to be instantaneous, the structure is exposed to a dynamic effect. For static analysis (LS and NS), the dynamic effect is employed by the amplification of the loads on the bays above the failed elements by means of a **Dynamic Increase Factor** (DIF). Previous results from the literature suggested that a DIF of 2 would be appropriate. However, recent results *(DOD UFC, 2010; Dinu et all., 2010; Stevens et all., 2011)* showed two main issues are of concern. First, the same level of DIF is used for LS and NS. However, for extreme events such as progressive collapse, it is more economical to design structures to respond in the nonlinear range. Second, DIF does not vary with level of performance and allowable ductility (plastic deformation). The results obtained so far suggests values of DIF limited to 1.5 rather than 2.0 for NS analysis.

To sum up, for different levels of alternative load path methods, more complicated and comprehensive methods provide more accurate results. This accuracy, however, is obtained with costs such as enormous computer resources, waiting some weeks before examining the output. Moreover, even for the nonlinear dynamic analysis, it does not necessarily yield reliable predictions of structural progressive collapse. This is due to one primary drawback of the alternative load path approach, which is its neglect of the “initial damage”, or damage in adjacent structural members caused by blast load or impact, and the nonzero initial condition assumption.

1.2.4.2 Progressive collapse typology (Types of progressive collapse)

Progressive collapse of structures is characterized by a disproportion in size between a release event and the resulting collapse. Although the disproportion between cause and effect is a defining aspect, there are different mechanisms that produce such an output. The approaches for quantifying indices, and possible or preferable countermeasures can vary accordingly. Propagating action, used in the following, has the meaning of an action that results from the failure of one element and leads to the failure of further elements.

Pancake-type collapse

The collapse of the World Trade Center (WTC) towers can be an example. The impact of the airplanes and the subsequent fires initiated local failures in the areas of impact. The ensuing loss in vertical bearing capacity was limited to a few stories but extended over the entire cross-section of the respective tower. The upper part of the structure started to move downwards and accumulated kinetic energy. The subsequent collision with the lower part of the structure, which was still intact, caused large impact forces which were far beyond the reserve capacities of the structure. This resulted in the complete loss of vertical bearing capacity in the area of impact. The pancake-type collapse is represented by the following features:

- initial failure of vertical load-bearing elements;
- partial or complete separation and fall, in a vertical rigidbody motion, of components;
- transformation of potential energy into kinetic energy;
- impact of separated and falling structural components on the remaining structure;
- failure of other vertical load-bearing elements due to the impact loading;
- collapse progression in the vertical direction.

Characteristic features are the separation of structural components, the release of potential energy, and the occurrence of impact forces.

Zipper-type collapse

For the design of cable-stayed bridges, the PTI Recommendations require that the sudden rupture of one cable shall not lead to structural instability and specify a corresponding load case “loss of cable”. Such requirement is intended, among other things, to prevent a zipper-like collapse initiated by the rupture of one cable and propagating by overloading and rupture of adjacent cables. Some of the main characteristics of this kind of progressive collapse:

- initial failure of one or a few structural elements;
- redistribution of forces carried by these elements in the remaining structure;

- impulsive loading due to the suddenness of the initial failure;
- dynamic response of the remaining structure to that impulsive loading;
- due to the combined static and dynamic effects, a force concentration in and failure of elements which are similar in type and function to and adjacent to or in the vicinity of the initially failing elements;
- collapse progression in a direction transverse to the principal forces in the failing elements.

Characteristic features are the redistribution of forces into alternative paths, impulsive loading due to sudden element failure, and static and dynamic force concentration in the elements to fail next.

Domino-type collapse

The mechanism behind this type of collapse is as follows:
- initial overturning of one element (i.e., of one domino block);
- fall of that element in an angular rigid-body motion around a bottom edge;
- transformation of potential energy into kinetic energy;
- lateral impact of the upper edge of that element on the side face of an adjacent element; the horizontal pushing force transmitted by that impact is of both static and dynamic origin because it results from both the tilting and the motion of the impacting element;
- overturning of the adjacent element due to the horizontal loading from the impacting element;
- collapse progression in the overturning direction.

The list of features characterizing a domino-type collapse is:
- initial overturning of one element;
- fall of that element in an angular rigid-body motion around a bottom edge;
- transformation of potential energy into kinetic energy;
- abrupt deceleration of the element's motion through sudden activation of discrete other elements; the horizontal force induced by that event is of both static and dynamic origin because it results from both the tilting and the motion of the decelerated element;
- overturning of other elements due to the horizontal loading from the decelerated element;
- collapse progression in horizontal direction.

Section-type collapse

A beam under a bending moment or a bar under axial tension is considered. When a part of the respective cross section is cut, the internal forces transmitted by that part are redistributed into the remaining cross section. The corresponding increase in stress at some locations can cause the rupture of further cross-sectional parts, and, in the same manner, a failure progression throughout the entire cross-section. While this kind of failure is usually not called a progressive collapse (but fast fracture), it is useful to include it in this description in order to possibly exploit similarities and analogies. When comparing to the previously discussed types of collapse, a section-type collapse appears similar to a zipper-type collapse. Indeed, the same list of features applies when the terms "cross-section" and "part of cross-section" are substituted for the terms "structure" and "element", respectively.

Instability-type collapse

Instability of structures is characterized by small perturbations (imperfections, transverse loading) leading to large deformations or collapse. Structures are designed such that instability will not normally occur. The failure of a bracing element due to some small triggering event, however, can make a system unstable and result in collapse. This could apply to truss or beam structures where bracing elements are used to stabilize bars or cross-sectional elements in compression.
An instability-type collapse exhibits the following features:

- initial failure of elements which stabilize load-carrying elements in compression;
- instability of the elements in compression that cease to be stabilized;
- sudden failure of these destabilized elements due to small perturbations;
- failure progression.

Mixed-type collapse

The types of collapse considered so far are relatively easily discerned and described. Some collapses that have occurred in the past do not neatly fit into these categories, however: The partial collapse of the Murrah Federal Building (Oklahoma City, 1995) seems to have involved features of both a pancake-type and a domino-type scenario. The collapse of Haeng-Ju Grand Bridge (Seoul, 1992) possibly involved features of the zipper-type and the domino-type categories.
Such mixed-type collapses are less docile to generalization because the relative importance of the contributing basic categories of collapse can, in principle, vary.

Classification of collapse types

The preceding discussion of types of collapse and their respective features allows further generalization and classification. Both zipper-type and section-type collapses are most strongly characterized by the redistribution of forces carried by failing elements in the remaining structure. They are thus subsumed under one class of collapse which is called the redistribution class. Pancake-type collapse and domino-type collapse, in comparison, have fewer features in common, but in some important respects they are similar. In both, a substantial amount of potential energy is transformed into kinetic energy during the fall or overturning of elements and subsequently reintroduced into the structure. The latter two types of collapse are thus combined in one class of collapse which is called the impact class. This term is chosen for convenience and also refers to the abrupt deceleration of overturning elements in a domino-type collapse. The instability-type collapse forms one class on its own. It is characterized by destabilization of load-carrying elements in compression through discontinuance of stabilizing elements. The transformation of potential energy plays a role but in a different way than for an impact-class collapse. Finally, mixed-type collapses also form one class, for which, however, it is difficult to identify general properties other than the fact that features of various types of collapse interact and combine to produce a collapse.

1.2.4.3 Examples to progressive collapse through the history

The Ronan Point Building, 16 May, 1968

Figure 1.9 Ronan Point building after the collapse in 16 May, 1968 (www.911research.wtc7.net)

On 16 May, 1968, the partial failure of the 22-storey precast concrete residential flats building occurred in Newham, east London. In the early morning, a domestic gas explosion within the kitchen of a flat in the eighteenth storey blew out concrete panels forming part of the loadbearing flank wall at a corner of the building. The removal of this part of the loadbearing wall precipitated the collapse of the corner of the block above the eighteenth floor. The weight of this part of the building as it fell set off a chain reaction of collapses of the remainder of the south-east corner down to the level of the in-situ concrete podium. The result was a progressive collapse that gave rise to spectacular pictures such as that shown in Figure 1.9.
While the failure of the Ronan Point structure was not one of the larger building disasters of recent years, the magnitude of the collapse was completely out of proportion to the triggering event. This type of sequential, domino-effect failure was labeled “progressive collapse.” Since then, the engineering community and public regulatory agencies resolved to change the practice of building design to prevent the recurrence of such tragedies. It has also been suggested that the degree of “progressivity” in a collapse be defined as the ratio of total collapsed area or volume to the area or volume damaged or destroyed directly by the triggering event. In the case of the Ronan Point collapse, this ratio was of the order of 20. By any definition, the Ronan Point disaster would qualify as a progressive collapse. It was also

disproportionate: A corner of a 22-story building collapsed over its entire height as a result of a fairly modest explosion that did not take the life of a person within a few feet of it. The scale of the collapse clearly was disproportionate to the cause.

Alfred P. Murrah Federal Office Building, 19 April, 1995

Figure 1.10 Murrah Federal Office Building collapse on 19 April, 1995 (www.wikipedia.org)

The Murrah Federal Office Building in Oklahoma City was destroyed by a bomb on April 19, 1995. The bomb, in a truck at the base of the building, destroyed or badly damaged three columns. Loss of support from these columns led to failure of a transfer girder. Failure of the transfer girder caused the collapse of columns supported by the girder and floor areas supported by those columns. The result was the collapse. The Murrah Building disaster was progressive collapse by all definitions of that term. Collapse of a large part of the building was precipitated by destruction of a small part of it (a few columns). The collapse also involved a clear sequence or progression of events: column destruction; transfer girder failure; collapse of structure above. But was the Murrah Building collapse disproportional? The answer is not nearly as clear as in the case of the Ronan Point collapse. The Murrah collapse was large, but the cause of the collapse, the bomb, was very large too—large enough to cause damage over an area of several city blocks. Ultimately, we must judge the Murrah Building collapse "possibly disproportional" only because we know now that with some fairly modest changes in the structural design, the damage from the bomb could have been reduced significantly.

World Trade Center 1 and 2, 11 September, 2001

The twin towers of World Trade Center 1 and 2 collapsed on 11 Sept, 2001 following this sequence of events: A Boeing 767 jetliner crashed into each tower at high speed; the crash caused structural damage at and near the point of impact, and set off an intense fire within the building; the structure near the impact zone lost its ability to support the load above it as a result of some combination of impact damage and fire damage; the structure above collapsed, having lost its support. The weight and impact of the collapsing upper part of the tower caused a progression of failures extending downward all the way to the ground. Clearly, this was a "progressive collapse" by any definition. But it cannot be labeled a "disproportionate collapse."

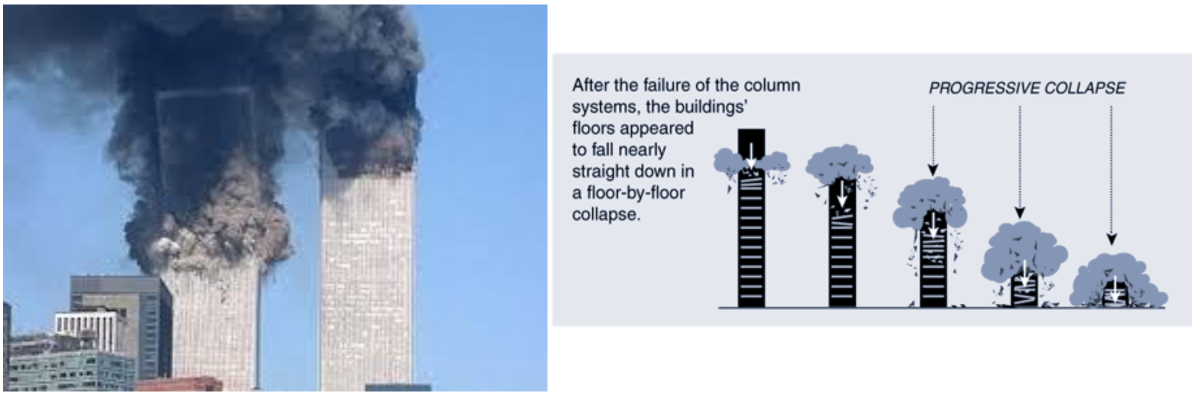


Figure 1.11 World Trade Center 1 and 2 collapse on 11 September, 2001 (www.wikipedia.org and www.911research.wtc7.net)

It was a very large collapse caused by a very large impact and fire. And unlike the case with the Murrah Building, simple changes in the structural design that might have greatly reduced the scale of the collapse have not yet been identified.

1.2.4.4 Methods of preventing disproportionate collapse

In general, there are three alternative approaches to designing structures to reduce susceptibility to disproportionate collapse:

Redundancy or Alternate load paths

In this approach, the structure is designed such that if any one component fails, alternate paths are available for the load in that component, preventing a general collapse from occurring. This approach has the benefit of simplicity and directness. Design for redundancy requires that a building structure be able to tolerate loss of any one column without collapse. The problem with the redundancy approach, is that it does not account for differences in vulnerability.

Local resistance

In this approach, critical components that are potential subjects for attack and that are susceptible to progressive/disproportionate collapse are provided with additional resistance. This requires some knowledge of the nature of potential attacks and is difficult to codify in a simple and objective way.

Interconnection or Continuity

This is not a third approach separate from redundancy and local resistance, but a means of improving them. Studies shown that failure could have been avoided or at least reduced in scale at little additional cost if structural components had been interconnected more effectively. This is the basis of the "structural integrity" requirements in the ACI 318 specification *(ACI, 2002).*

1.3 Damage and Collapse Control Design

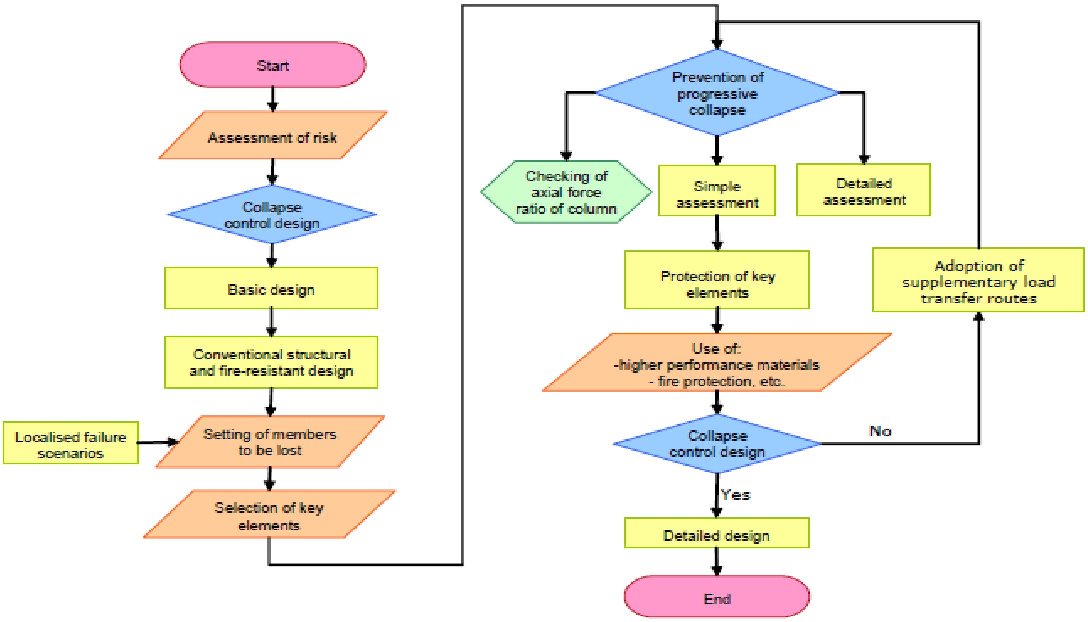


Figure 1.12 Collapse control design flowchart (Marginean et al, 2013)

The development of design guides for collapse control of the multi-story buildings started in 1968 with the collapse of the Ronan Point high-rise building.

The concept of **collapse control design** (Figure 1.12) can be considered the most appropriate and recent approach for preventing the progressive collapse in case of extreme load events. Generally, it is difficult and uneconomical to conduct structural design by assuming accidental loads due to extreme events. In contrast to conventional methods, the present design method assesses and improves the redundancy of buildings by assuming the loss of structural members such as columns and beams due to accidents and assessing how many members might be lost and the probability of entire collapse of the building occurring. We can say that all buildings are susceptible to progressive collapse in varying degrees. Continuous, highly redundant structures with ductility tend to absorb local damage well.

By *(Corley et al., 1998) "Redundancy is a key design feature for the prevention of progressive collapse. There should be no single critical element whose failure would start a chain reaction of successive failures that would take down a building. Each critical element should have one or more redundant counterparts that can take over the critical load in case the first should fail."*

Redundancy is seen in terms of the process whereby collapse advances from local to progressive, and buildings with redundancy are defined as those in which it is difficult for local collapse to lead to progressive collapse. In practical terms, the assessment of a building's redundancy is an evaluation of the relation between degree of loss and collapse when the structural members assumed to be lost are floor slabs, beams,columns and walls.

When collapse control design is used, the key-element members are specified in the frame design according to an assessment flow. Priority is given to protecting the key-element members so as to improve building redundancy.

The assessment flow starts with assessing the risk and judjing whether or not to use collapse control design. In the present design method, the effect of unexpected loads caused by terrorist explosions and aircraft crashes is not assessed directly. Rather, losses or declines in the yield strength of vertical load supporting members that are brought about by the application of unexpected loads are assessed and are reflected in the design work. Based on the concept that improving the redundancy of buildings minimizes the risk of a progressive collapse, the present design method aims to compensate for loss or decline in the yield strength of members that support vertical loads. Further at this stage of design, the potential scale of column member loss is assumed by taking into account the degree of risk involved and the importance of the building, i.e. the effect it would have in the case of collapse. In cases when the design of a building requires more appropriate redundancy, it is desirable to determine the number of columns to be lost in the design. More practical determination of the members to be lost can be made after fixing the sectional dimensions of the members by means of conventional structural and fire-resistant design methods. After these, a basic design process comes. It is desirable to select a frame system that will have a high load redistribution capacity after the functional loss of vertical load supporting members.

After completion of the basic design, the cross section of the members is decided in conformity with conventional structural and fire-resistant design. The members to be lost are determined and the key elements are selected. The members to be lost are determined taking into account the scale of a potential accidental action and the risks involved. At this stage, the key elements can be excluded from the members to be lost on the premise that they will be reasonably safe because they are protected with every available measure. In the present collapse control design method, the determination of key elements is cited as an important requirement. The key elements are those members whose loss directly affects the risk of a chain-reaction collapse; the specifications of fire protection etc. of the key element are to be determined so as to secure the greatest possible safety against extreme actions.

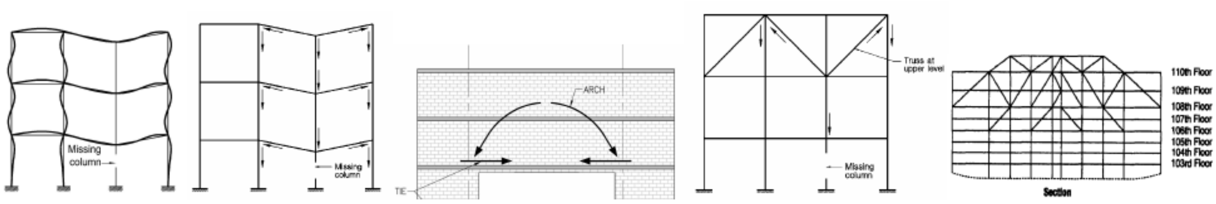
It is known that the loss of corner columns is the greatest cause of reducing vertical load supporting capacity. Accordingly, it is desirable to set the corner columns as key elements and to adopt for them methods and materials conducive to improving redundancy, such as FR steel (fire-rated steel), CFTs (concrete filled tubes) and the blanket-type fire protection. In selecting the key elements, they are to be arranged in a concentrated manner, such as selecting only corner columns, providing the chosen columns with sufficient excess strength (lower axial force ratio of columns) so that they alone could support the loads on all floors, or possibly selecting every third column as a key element.

After setting the key elements, an assessment regarding the prevention of chain-reaction collapse is made. There are three assessment methods: assessment using only the axial force ratio of columns, simple assessment and detailed assessment.

The next step is the protection and the detail design of the key elements. The detail design stage includes the design of beam-column connections, the design of floor systems, the design of fire separations and connection details, and the determination of fire protection specifications. As stated above, in order to meet emergency conditions that arise because of the loss of structural members, adopting connections with sufficient load-carrying capacity for joining beams-columns and columns-columns is important in securing the deformation capacity of members, realizing the integration of floor systems and ensuring the fire resistance of key elements.

1.3.1 Design requirements or methods effective in increasing progressive collapse resistance

- **Supplementary load transfer routes**: beneficial effects like Vierendeel, Catenary, Arch or Suspension effects (Figure 1.13). For example braces installed at intermediate levels or on top of the structure, associated with vertical bracings for increasing the wind and seismic resistance are also effective in redistributing gravity loads in case some critical members are lost.



a) b) c) d) e)

Figure 1.13 Supplementary load transfer routes: a) Vierendeel action; b) Catenary action; c) Arch effect; d) Suspension effect; e) Hat trusses on top of WTC (suspension effect) (Marginean, 2013)

- **Increase of load redistribution capacity**: e.g. the continuity of beams at the intersection with columns allows the redistribution of loads after the column is lost.
- **Increase of connection strength**: e.g. frames with full strength connections joints as in case of seismic design can bridge over the missing column and can secure the resistance against progressive collapse.
- **Increase of connection ductility**: when some critical members are lost, e.g. columns, the vertical displacements in the affected area are large, and this requires large deformation capacity of members and their connections. For beam-to-column connections, the demand can be several order of magnitude larger than in case of earthquake.

- **Proper design of fire resistance of members**: this can be done by adoption of fire protection materials (e.g. concrete-filled steel tubes CFT or partially/fully encased columns) or of fire resistant materials (eg. fire resistant steel, with similar material characteristics at room temperature and elevated temperatures).
- **Proper design of the structural system**: Moment resistant frames and moment resistant frames with seismic resistant elements with installation in core sections of braces, steel-plate shear walls and other sesimic resistant elements with vertical load-carrying capacity is effective in securing redundancy. Tube structures provided with lattice beams (hat trusses) in the upper and middle sections of the building is highly effective in redistributing vertical load after column loss. The use of super MRF structures, where two to four columns are connected by the use of diagonal members and rigid beams to form single super columns, which thus formed are connected by means of girders having large depth, are as well good solution if the key element (”super columns”) are well protected.

According to EN 1990 (Basis of Structural Design) requirements, robustness can be ensured by one of the following measures:

- Avoiding, eliminating or reducing the hazards to which the structure can be subjected;
- Selecting a structural form which has low sensitivity to the hazards considered;
- Selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localized damage;
- Avoiding as far as possible structural systems that can collapse without warning;
- Tying the structural members together.

All these measures are intended to provide the structure with enough capacity to survive every type of loading condition it was designed for and beyond. The main problems arise from the difficulty in verifying the efficacy of the measures listed above with a reasonable degree of confidence.
Good design involves looking beyond the minimum design requirements in a code or standard. Building codes and standards have yet to incorporate specific universal requirements, either in approaches or detailing, that effectively quantify the design goal for increased robustness. Furthermore, performance expectations are not well stated. The state of practice is to acknowledge the general need for robustness and to identify structural characteristics that generally are responsive to that need. Much of the guidance is anecdotal, and of only general use to the design engineer. Those references that do prescribe specific goals employ performance-based approaches.

1.4 Local failure scenarios – Column loss and implications

In the case of building structures, the possible local failure scenarios after an exceptional event can be the loss of one or more vertical members (i.e. columns, walls) or horizontal members (i.e. beams, slabs).

In the present work, the loss of a column is considered to be an exceptional case, i.e., an event which is not taken into account in the design process. The event of column loss can been studied for a variety of scenarios, either for one column loss or for two columns loss situations, etc.

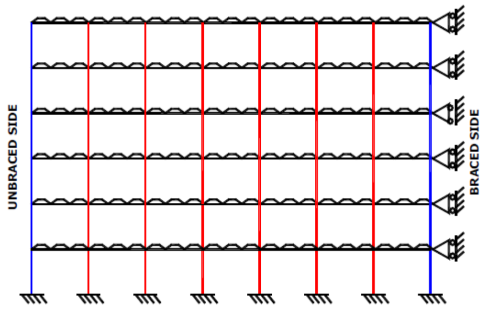


Figure 1.14 Column loss scenario (Hai, 2009)

The lost column can be at different positions in the frame and different height; therefore, two specific positions of the damaged column are identified, as illustrated in Figure 1.14. The internal columns are in red while the external columns are in blue in Figure 1.14. The loss of a column can be explained by different types of exceptional events, such as explosions or vehicle impacts. In some of these exceptional events, dynamic effects may play an important role. When a column is lost in a frame, the frame can be divided into two parts (illustrated in Figure 1.15, where a column on the 2nd floor is lost for example):

- the **directly affected part**, which represents the part of the building which is directly affected by the loss of the column, i.e., the beams and the columns which are just above the lost column (with red in Figure 1.15);
- the **indirectly affected part**, which represents the part of the building which is affected by the forces developing within and influenced by the directly affected part (with blue in Figure 1.15).

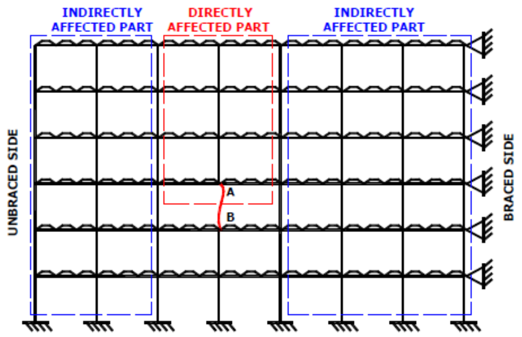


Figure 1.15 Directly and indirectly affected part (Hai, 2009)

Studies in the literature has been shown that the membrane forces developing in the beams of the floor just above the column lost are significantly higher than the ones developing in the other beams (Figure 1.16). The columns from the both sides of the directly affected part, bend and produce compression on the top beams.

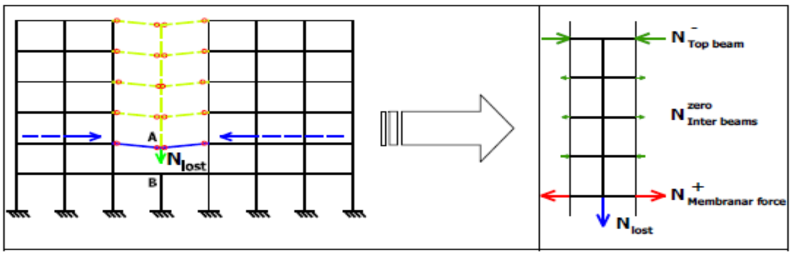


Figure 1.16 Distribution of the membrane forces in the directly affected part (Hai, 2009)

The internal/central column loss scenario has the highest strength potential. But it should be kept in mind that in this case the vertical loading is roughly twice higher than in extrimity and four times higher as in the corner case scenario.

For the numerical simulations and experimental testing a framed structure was selected and designed according the current design requirements having 4 bays, 4 spans and 6 stories (Figure 1.17).

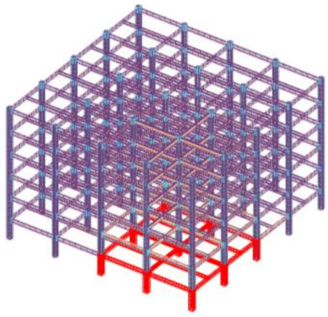


Figure 1.17 Structural system set-up (Marginean, 2013)

Due to the limitation in the testing facility at the host institution, the specimens were scaled to cover two-bay two-span configuration. The largest specimens are the sub-assemblies and they are two-bay two-span models (in red in Figure 1.17). In order to obtain an exact behavior under large vertical displacements two column loss scenarios were considered: loss of central column and loss of corner column. To study both the catenary effect in beams and the catenary effect (membrane effect) in the slab and to study the influence of composite actions on the performance of the building two cases were considered: only steel structure and composite concrete-steel structure. In total 4 sub-assemblies will be studied in the CODEC (PN-II-PT-PCCA-2011-3) National Research Project:

1. Loss of the central column in the case of steel structure only
2. Loss of the central column in the case of composite concrete-steel structure
3. Loss of the corner column in the case of steel structure only
4. Loss of the corner column in the case of composite concrete-steel structure

The aim of this work is to perform two out the four cases presented above, namely the loss of the central column of the sub-assembly in the case of only steel structure and in the case of composite concrete-steel structure.

Chapter 2: Theoretical background

2.1 Theoretical, experimental and numerical background

Europe

The first studies about progressive collapse appeared after the collapse of the Ronan Point building, in 1968, London, UK. Shortly after the collapse, first standards have been approved in 1970. Other provisions appeared in 1974, the British Standards. In 1976 a new design provision have been accepted (Building Regulations, HMSO, 1976). The benefit of these provisions was confirmed by the performance of the structures subjected to accidental actions including impact, explosion, blast, etc.. One of the examples could be the Exchequer Court, St. Mary's Axe in London, a modern steel construction at that time with concrete flooring acting compositely and designed to resist lateral wind loads by a system of braced steel bays. In April, 1992, a bomb exploded in the vicinity of the building producing damage in several buildings. Altought the building suffered serious damage to both its non-structural and structural parts, the building remained intact. Furthermore, the type of the explosion was different by nature to the internal gas explosion stated in the rules given in the Building Regulations, Approved Document A and the material Codes. The continuous research led to the upgrading of the design codes and the new version of the Guidelines has been released in 1991 (Approved Documents, 1991) and 2004, respectively in 2010. According to the requirements of these standards, the structures are designed to be more robust which are more resistant to disproportionate collapse due to various causes like impact, gas explosion, etc. The most recent version of the Approved Documents, released in 2010, has fourteen technical "Parts" and refers, among other to progressive collapse and fire safety. Beside from the provisions from UK, early studies about progressive collapse appeared in Sweden (Ganstrom, S.,1970) and Denmark (Hanson and Olesen, 1969), but also in Germany, Netherlands and France.

United States of America

The first studies on progressive collapse were developed in the 1970s, following the collapse of Ronan Point building and concentrated on quantifying the risk of collapse for precast concrete structures mainly in case of gas explosions. Firts requirements for general structural integrity to provide resistance for progressive collapse have been incorporated in 1972 (ASCE 7-05). The studies have been revied by terrorist attacks against US facilities (WTC in 1993, Murah Building in 1995, WTC in 2001). Several federal agencies developed their own design regulations for mitigation of the risk under extreme loading events. The most important design requirements have been developed by General Service Administration (GSA, 2003) and the Department of Defence (DOD UFC, 2010). GSA progressive collapse Guidelines is used by the US General Service Administration for the design of new federal buildings and for evaluation of existing federal facilities. The main method is based on the evaluation of the structural integrity after some primary members are lost, also called "alternate load path" method. The UFC applies for buildings belonging to Department of Defense but can be used as well by other federal agencies or organizations with role in creating or implementing design codes for constructions. UFC can be used for new or existing buildings as well. The latest release (2010) provides, apart from general rules and detailed design procedures, specific requirements for designing reinforced concrete buildings, steel buildings, masonry and timber buildings to resist progressive collapse.

Japan

In the country of earthquake events, the progressive collapse resistant design became less important. The collapse of WTC in 2001 gave a warning all over the world for design engineers on the dramatic consequences of progressive collapse in case of high-rise buildings. Also became clear that in the case of such a situation not only one extreme loading is present. For instance, in the case of the Great Hanshin Earthquake (Kobe, 1995) most of the injuries and damage was not caused directly by the earth motion but by the hundreds of fires that followed it. Another example can be the earthquake in 2011 that affected Japan and the following tshunami that struck off the country and brought almost a nuclear disaster. As a response, The Japan Iron and Steel Federation established the Commitee to Study the Redundancy of High-Rose Steel Buildings within the Japanese Society of Steel Construction (JSSC) and in cooperation with the Council on Tall Buildings and Urban Habitat (CTBUH). The first results were published in 2005, in two parts. First part addresses the recommendations to improve redundancy and progressive collapse propertied under extreme loads (JSSC, 2005a). The second part presents in detail these steel materials with example of application (JSSC, 2005b). The investigations also shown that the most appropiate approach for preventing progressive collapse under extreme load events is the collapse control design (mentioned in the above chapters), based on the seismic and fire resistant design.

2.1.1 Design codes and standards in the world

The existing general approach to the design of robust buildings is either **deterministic** or, 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 (e.g. Class 3 buildings of Eurocodes).

2.1.1.1 Europe – Robustness in the Eurocodes

While ordinary limit states to common types of loads are given in EN 1991-1-1:2002, the robustness requirements are usually linked to accidental actions (EN 1991-1-7:2006) or other abnormal events.

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:

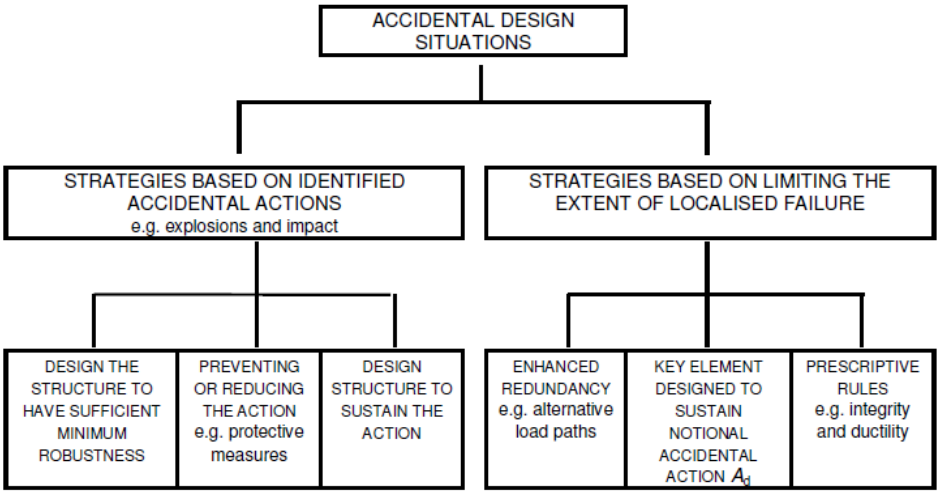


Table 2.0 Accidental design situation presented in Eurocode EN 1991-1-7:2006

For these strategies the Eurocode EN 1991-1-7:2006 provides three consequence categories for the design of structures under extraordinary events as shown in Table 2.1 and Chapter 1, Section 1.2.3.

	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 2.1 Consequence Classes of Eurocode EN 1991-1-7:2006

Enhanced redundancy measure

The Eurocode provides some measures to achieve robustness in buildings. These measures are by active vertical and horizontal ties. 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 isolated single elements. 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).

Key element design

A building should be checked to ensure that in the case of 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 2.1. 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).

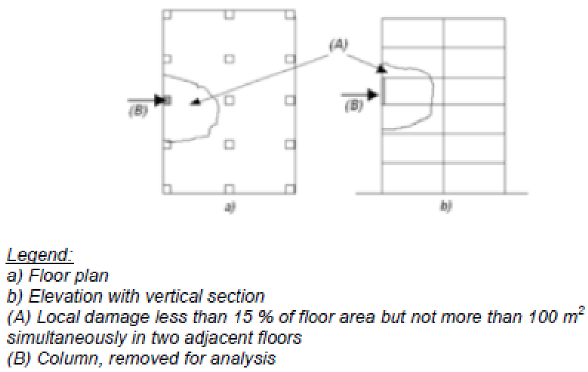


Figure 2.1 Recommended limit of acceptable damage in EN 1991-1-7:2006

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).* 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.

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

The ASCE document 7-10 includes a remark, 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. The US approach and also the British Standard provides three consequence categories for the design of structures under extraordinary events, but the second one can be devided into two sub-categories compared to the Eurocode.

Consequences class	Example structures
Class 1	low rise buildings where only few people are present
Class 2, lower group	most buildings up to 4 storeys
Class 2, upper group	most buildings up to 15 storeys
Class 3	high rise building, grandstands etc.

Table 2.2 Consequence Classes of the US approach and British Standard

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. 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 mitigated. In addition the code 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 **indirect design** considers the resistance of progressive collapse during the design process implicitly by 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 presented 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.

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.

2.2 Research trends, gaps and potential contribution to the state of art

As mentioned in the beginning, significant area of research in structural engineering, particularly in the last decade has been robustness of structures. The need for the concept of robustness lies in the fact that structural design codes are based mainly on the design of structural members or the consideration of the member failure modes. On the international scale, the research on structural robustness and developtment of methodologies to ensure an appropriate level of structural robustness in building structures has so far dominated by North America and Europe.

It was shown in the recent studies that the use of seismic detailing improves robustness and progressive collapse resistance of building structures, see (Dinu et al. 2013). The development of the numerical simulation tools provide more and more accurate results covering many aspects (e.g. material nonlinearities, large deformations, etc). However, both global and local analysis are needed in order to properly assess the phenomena. In order to validate the global behavior of the structure, sub-assembly models are used in numerical and experimental testings. As in each field of engineering, experimental test are needed. However, there are just a few similar experimental tests done covering the topic presented within this thesis, thus there is a big demand for the current research field.

Robustness is not only of extreme importance but that the present situation with regard to ensuring sufficient structural robustness through codes and standards is highly unsatisfactory. Many projects and research work started in the last decade to provide basic framework, methods and strategies necessary to ensure that the level of robustness of structural systems is adequate and sufficient in relation to their function and exposure over their life time.

Some of the **research gaps** in the field of structural robustness are:

- Risk assessment;
- Robustness assessment and methodological framework for robustness;
- Exposures acting on structures;
- Failure/collapse models for structures/systems;
- Minimum requirements to the acceptable robustness of structures;
- Codification of design process, assessment, monitoring and condition control for robustness of structures;

- Guideline for practicing engineers on how to asses and enhance the robustness of structures;
- Improving the European impact on international codification on robustness of structures;
- Development of guidelines for minimum requirements for horizontal and vertical tying;
- Structural design method by considering the removal of a critical member;
- Slab contribution by the means of membrane action to the global behavior;
- Tying contribution of the concrete floors in the case of critical member removal (contribution of catenary effect of concrete floors);
- Database with extreme load events
- Studies about the specific mechanics by which a moment resisting frame devolves from a flexure dominant system to a tensile membrane or catenary dominant system and what are the rotation demands on connections
- The reserve axial tension capacity of steel beam-to-column connections after reaching significant inelastic rotations
- Importance and consequences of analysis approaches chosen;

Potential contribution of the work to the state of art:

- Behavior of building structures under extreme loads;
- Advanced numerical investigations, aiming of evaluating the main parameters that contribute to the robustness of building structures;
- Advanced numerical analysis in the case of 2-3D steel/steel-concrete composite structures considering column loss scenario;
- Finite element numerical modeling of membrane action of composite slabs in the case of column loss;
- Performance criteria for developing the catenary action in the structure after column loss scenario;
- Parametrical study regarding the membrane effect of composite slabs in the case of column loss scenario;
- Parametrical study regarding the effect of the variation of slab thickness to progressive collapse resistance;
- Parametrical study regarding the effect of the variation of the reinforcement ration to progressive collapse resistance;
- Parametrical study regarding the effect of the variation of the number and location of the shear connectors to progressive collapse resistance;
- Study of the contribution of the catenary action and flexural action into the global behavior of the structure;
- Proposals of progressive collapse mitigation measures.

Chapter 3: Selection and design of the reference building structure

3.1 Selection of the reference building structure

Steel moment-resisting frames (MRF) are considered a common type of building systems all over the world. These are typically compromised of composite floors, where steel beams are composite with concrete deck and gravity columns to which the the floor beams are attached through shear connections. The primary purpose of the moment frames is to transmit lateral loads, such as wind and seismic forces. They are considered structural systems with good behavior what concerns extreme events, having in mind the inherent ductility of steel and its reduced weight. Beside these, steel has an increased strength compared to concrete, which is an advantage in the progressive collapse resistance.

The structural design of the reference building was not the aim of this thesis. The selected reference structure was taken from the Master Degree Project of Ing. Ioan-Mircea Marginean, presented at his Master Degree defence in Iune, 2013 at Universitatea "Politehnica" din Timişoara. 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).

According to the work of Ing. Ioan-Mircea Marginean (2013), the event of column loss has been studied by him for several scenarios, either for one column loss or for two column loss situation. In all of the cases mentioned above, implied the damage/loss of the columns from the ground floor. Results obtained by studying the structural response in the case of column loss shown that the structural system have an adequate robustness, whether it is the case of corner, side or central column loss.
A major role in the collapse prevention has the capacity of the connections to allow the development of the so called "catenary effect" in the beams. The aim of the project was to achieve fully rigid connections to allow the development of plastic hinges in the beams. Another major role in the progressive collapse prevention has the connection between the concrete slab and steel beams and the reinforcing of the slab. For instance, in the case of a structures with reduced redundancy, the two-directional reinforcing of the slab combined with an adequate connection of the concrete slab to the steel part may lead to the redistribution of the stresses in the affected zones, inclusively the development of the "catenary effect" (membrane effect) in the concrete slab.

3.2 Design of the structure under conventional loading situation

The reference building structure (moment-resisting frame) was designed according to his work, which is presented in the following chapter, in accordance to existing design codes, characterized by low seismicity zone in the city Cluj Napoca, Romania, considering rigid connections. The structural design did not account for any accidental loss of structural members.

3.2.1 Structural description

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

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

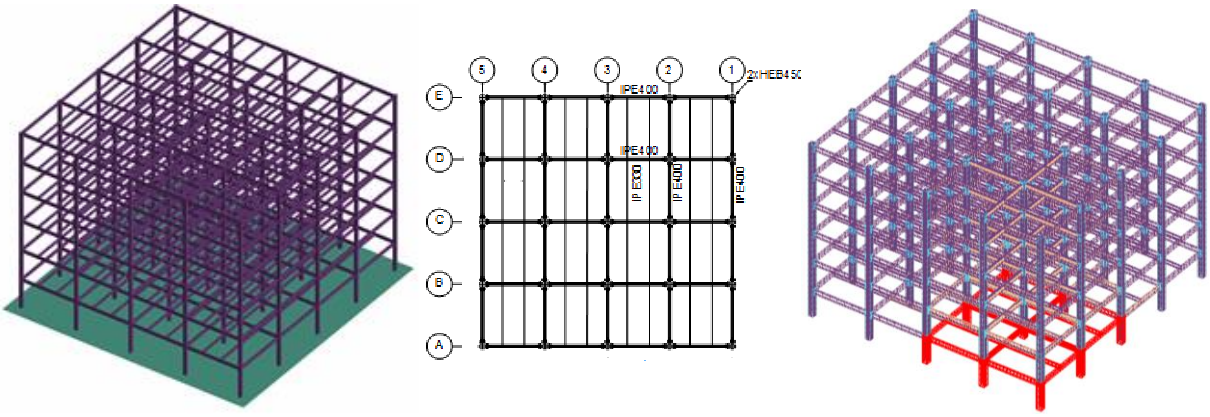


Figure 3.1 Geometry of the reference multi-storey frame building (Marginean et al, 2013)

3.2.2 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 12 cm 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 24m above the ground) and C_{pe} is the building shape coefficient.
The seismic response spectrum is dependent on the seismic area. For Cluj Napoca $a_g=0.08g$ and $T_c=0.7s$.
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).

3.2.3 Structural analysis

Loading hypotheses for potential 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.
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 ($\alpha_{cr} = 11.45 > 10$).
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 $d_{allowable}=0.008*h_{storey}=0.008*4000=32mm$ (according to P100-1/2012). Where $\gamma=0.5$ 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 12 cm was considered with a 2.66 m span between the floor beams. The slab reinforcement includes welded wire mesh of Φ6/166mm X Φ6/166mm.

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 200mm (Figure 3.2). For constructive reasons, for the main beams, the shear connectors were welded to top flange on one row at 200 mm intervals, except at the ends, where a free zone of 2 X h_b, or 800 mm has been considered (Figure 3.3).

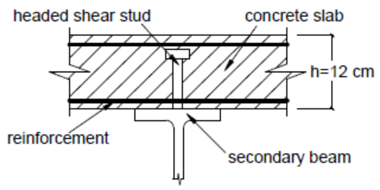


Figure 3.2 Cross-section of composite secondary beam (Marginean et al, 2013)

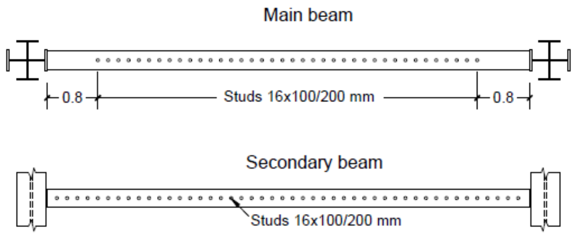


Figure 3.3 Spacing of connectors for main and secondary beams (Marginean et al, 2013)

For the connections, two types of extended end plate bolted connections were used. The difference consists of end plate thickness and bolt diameter (Figure 3.4).

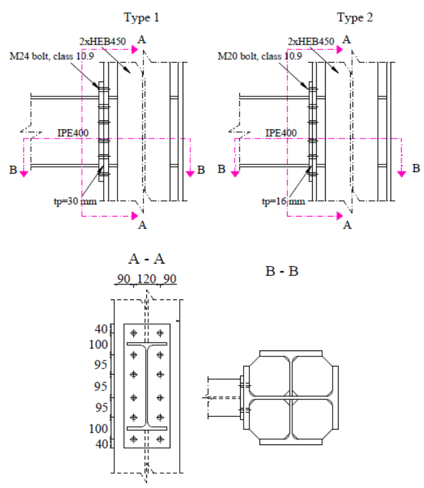


Figure 3.4 Connection details of beam-to-column (Marginean et al, 2013)

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.

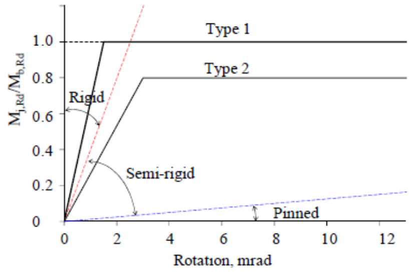


Figure 3.5 Moment-rotation curve for connections (Marginean et al, 2013)

3.3 From the reference building structure to the sub-assembly specimens

After the selection of the reference building for the numerical simulations and further experimental testing, due to the limitation of the testing facility at the host institution, sub-assemblies were extracted from the designed reference building and scaled to 1:2.5, ensuring that the support and connection conditions are equivalent to those in the refrence building (Figure 3.6). The largest specimens are the sub-assemblies and they are two-bay two-span structures having the following dimensions 3.0m X 3.0m X 1.5m (Figure 3.6). The design checks of the sub-assemblies were not part of this work and they are not presented in this thesis.

In order to obtain the behaviour of the full scale structure and simulate the effect of the above stories and adjacent bays, during the large vertical displacement testings, it was necessary to block the lateral movement possibility at the level of the beams as well the excessive rotation capacity at the superior end of the columns. To provide stability for the sub-assembly it was designed a framing system at the top of the columns (Figure 3.6), made up from circular hollow section steel bars.

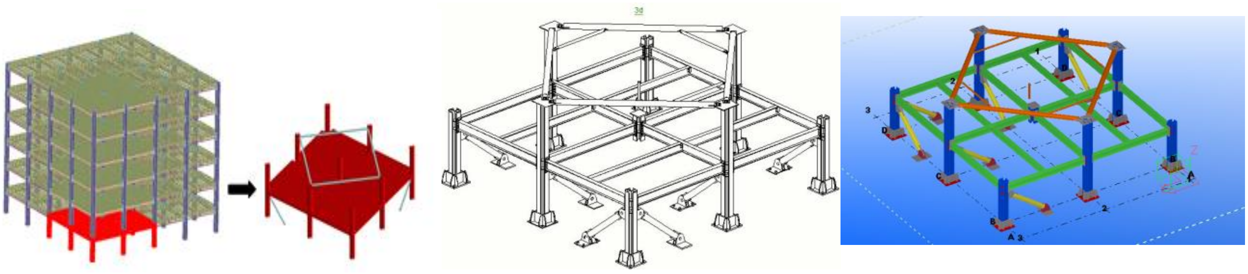


Figure 3.6 Sub-assembly specimen

The CHS steel bars have the following cross-sectional characteristics for the two elements (Figure 3.7) and the joint between them is considered to be pinned:

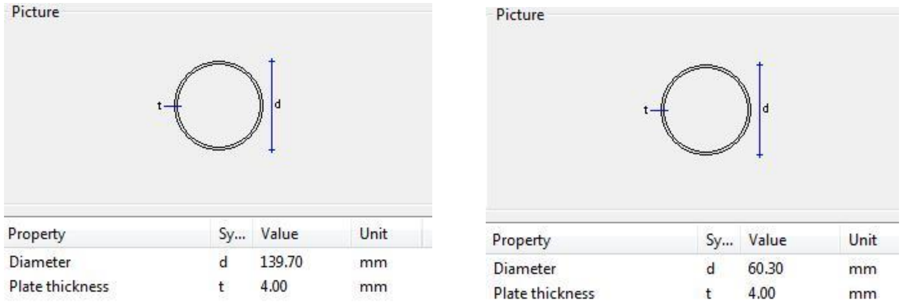


Figure 3.7 CHS cross-sectional characteristics

In order to provide the effect of the real structure, a bracing system was designed using steel grade S500 (analysis shown that its necessary) and the following cross-sectional characteristics (Figure 3.8):

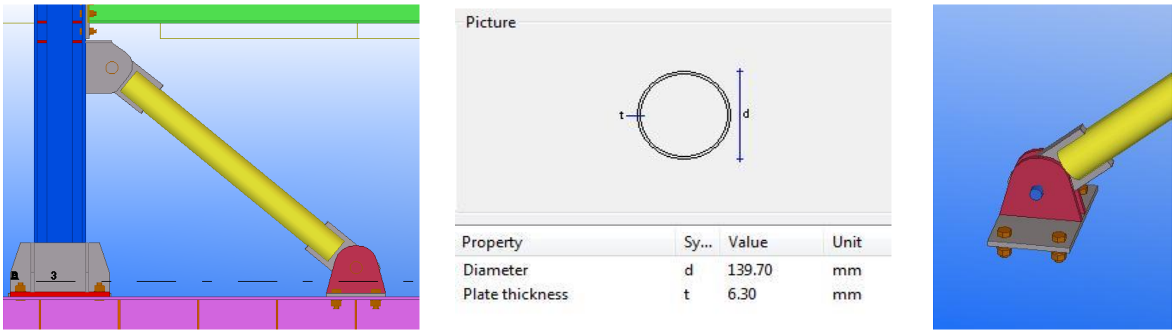


Figure 3.8 Bracing system cross-sectional characteristics

The bracing system was designed to be pinned-pinned. The distance between the axis is 3m in both directions and the height of the column is 3m (1.5 m + 1.5 m). The columns were designed having HEB 260 cross-section type maltese (with reduced flange width to 130 mm) with the following characteristics (Figure 3.9).

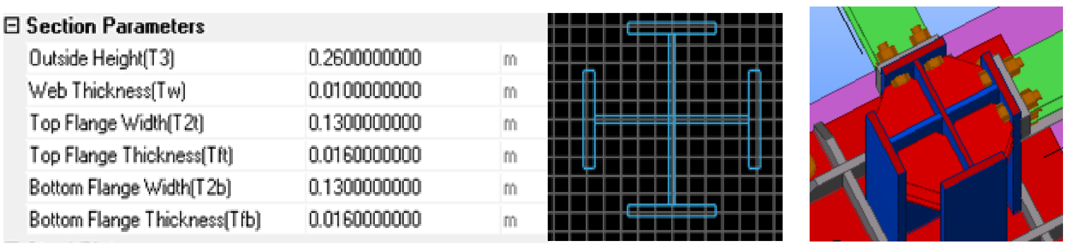
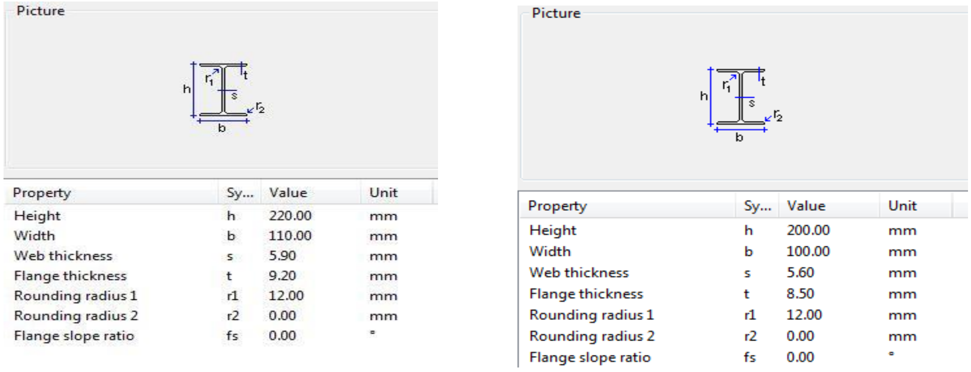


Figure 3.9 Column cross-sectional characteristics

The primary girders are made of IPE 220 and the secondary beams of IPE 200 (Figure 3.10).

The column base connection was designed to be fixed with the following configuration: 4 X M30 10.9 grade bolts and stiffening plates of 16mm in both directions (Figure 3.11).



Property	Sy...	Value	Unit
Height	h	220.00	mm
Width	b	110.00	mm
Web thickness	s	5.90	mm
Flange thickness	t	9.20	mm
Rounding radius 1	r1	12.00	mm
Rounding radius 2	r2	0.00	mm
Flange slope ratio	fs	0.00	°

Property	Sy...	Value	Unit
Height	h	200.00	mm
Width	b	100.00	mm
Web thickness	s	5.60	mm
Flange thickness	t	8.50	mm
Rounding radius 1	r1	12.00	mm
Rounding radius 2	r2	0.00	mm
Flange slope ratio	fs	0.00	°

Figure 3.10 Primary beam (left) and secondary beam (right) cross-sectional characteristics

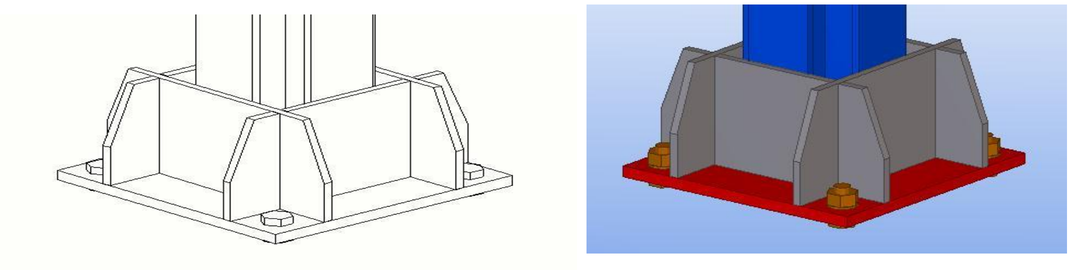


Figure 3.11 Column base connection configuration

The connection between the primary girder and column was designed to be fully rigid (like in the reference structure case) with the following set-up (Figure 3.12): the end-plate has a thickness of 20mm and dimensions of 130 mm X 370 mm. To connect the primary beams to the columns, 10 (2 X 5) M20 10.9 grade bolts were used (Figure 3.12).

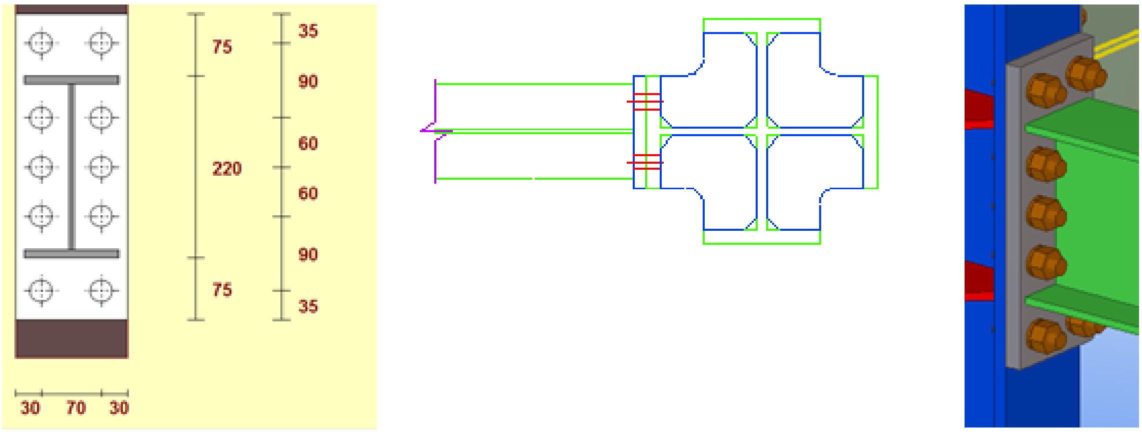


Figure 3.12 Column-to-beam connection configuration

What concerns the connection between the primary girder to the secondary beam, a simple shear plate connection was used with two bolts M16 10.9 grade and a steel plate with 8 mm thickness.

Chapter 4: Numerical simulations. Catenary effect due to the loss of a column

4.1 Catenary effect due to the loss of a column

In the case of a hazardous event (e.g. loss of a column), the so-called ”catenary effect” describes a phenomena where two connected beams lose their middle support (column). Non-linear behavior develops and increased axial forces which appear within the beams are called ”catenary forces”. This phenomenon helps in increasing the structure’s ability to maintain its stability. According to Hamburger and Whitaker (2002), this is the key in providing robustness of building structures.
The membrane effect or catenary action is broadly accepted as the solution to increase redundancy in frames, in the case of abnormal loads. According to Hamburger and Whitaker (2002), for an abnormal loading, the structure should be designed to be able to withstand the large tensile forces and large inelastic flexural deformations. As mentioned in Chapter 1, when a column is removed, a plastic mechanism is formed in the directly affected part and the vertical displacement rapidly increases. As consequence of this, second order effects develop in the directly affected part. Catenary forces develop in the bottom beams of the directly affected part. This catenary action is shown in the Figure 4.1 as follows:

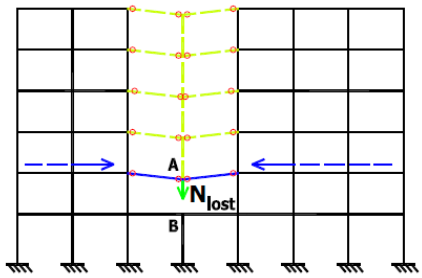


Figure 4.1 Catenary action in the case of $N_{lost}^{PL.Rd} < N_{design}$ (Hai, 2009)

When the directly affected part reaches its plastic limit, the additional load N_{lost} reaches the critical value of $N_{lost}^{PL.Rd}$. It is possible that the plastic limit is never reached by the additional load. This happens when: $N_{design} < N_{lost}^{PL.Rd}$
There are some conditions to be respected in order for the development of the catenary action within the directly affected part. As mentioned in previous chapters, when the alternative load path cannot be realized, progressive collapse occurs. For this reason, the **first condition** for the development of significant catenary actions, the directly affected part has to be capable to support the additional loads (N_{lost}) from the direclty affected part. By other terms, the columns just around the damaged column have to remain stable when subjected to additional compression loads coming from the directly affected part.
The **second condition** is related to the possibility of development a plastic mechanism within the directly affected part. The connections (in the case of partial-strength joints) or the beam extremities included in the direclty affected part have to have sufficient ductility to develop plastic hinges (this means that for the beam extremities, they have to be Class 1, according to the Eurocode).

According to Demonceau's work, there are three important parameters influencing the phenomena of catenary action.When all three conditions (the adequate value of $N_{lost}^{PL.Rd}$, the surrounding element stability and the full yielding of the directly affected part) are fulfilled, the special non-linear behavior called catenary action is activated in the bottom beams.
For further information regarding the load-carrying behavior of the bottom beam or lateral translational stiffnesses, etc, see *(Demonceau, 2008)* and *(N. Hai, 2009)*.
As a conclusion, the words stated by Hamburger and Whittaker (2004), regarding the advantage of catenary action were: "*firstly, it may not be necessary to provide moment resisting framing at each floor of the structure in order to provide progressivle collapse resistance. Secondly, it is not necessary to have substantial flexural capacity in the horizontal framing, either in the beam section itself or connections. Thirdly, it may not be necessary to provide full moment resistance in the horizontal framing, and conventional steel framing may be able to provide progressive collapse resistance as long as connections with sufficient tensile capacity to develop catenary action are provided".*

4.2 Numerical simulation and Analysis procedure

Numerical simulation

Collapse models can be **linear** or **nonlinear**. The first are simpler to develop and more convenient for usage, especially considering that most of existing commercial analysis software can solve linear models. However, linear models have several disadvantages: they cannot capture the progressive behavior that occurs during collapse, specially, the redistribution of forces that occurs as a result of local nonlinear behavior of the phenomenon. In despite of this limitation, linear models are use commonly to provide an idea of the structure's ability to resist collapse through so-called "demand-to-capacity ratios" (DCRs). This is defined as the ratio of computed internal force (bending moment, axial force or shear force) determined in a component or connection divided by the expected strength of the component or connection (if DCRs > 1 means that the capacity of the member will be exceeded in analysis).
The nonlinear dynamic procedures are considered the most sophisticated and accurate structural assessment techniques. However, due to the complexity and inherent challenges, these procedures have been used less frequently for
progressive analysis. The main difficulties are related to the numerical convergence problems, material models capturing inelastic properties and damage, modeling component disintegration caused by failure, requirements for mesh resolution capturing local effects, and the size of the FE models for large structures (e.g., multistory buildings).

Structural models that can be utilized for studying progressive collapse can be classified as **macromodels** and **micromodels**. Continuum finite-element models are examples of micromodels. Macromodels utilize a combination of shell, beam-column, and discrete spring finite elements to simulate the overall response of the structure. The success of the macromodels lie in the ability to adequately simulate the local and global response of the phenomena.
As mentioned in Chapter 1.2.4.1 and bearing in mind the previously related subject regarding collapse models, we can build up four levels of analyses, namely **linear static analysis (LS), nonlinear static analysis (NS), linear dynamic (LD)** and **nonlinear dynamic analysis (ND)**. Because the transition from the original structural configuration to the damaged state is assumed to be instantaneous, the structure is exposed to a dynamic effect. For static analysis (LS and NS), the dynamic effect is employed by the amplification of the loads on the bays above the failed elements by means of a Dynamic Increase Factor (DIF).

The **Finite Element Method** (FEM) is a numerical technique for finding approximate solutions to boundary value problems of differential equations. It uses variational methods to minimize an error function and produce a stable solution. The method involves:

- dividing the domain of the problem into a collection of subdomains (each subdomain represented by a set of element equations to the original problem);
- analysis of subdomains;
- systematically recombining all sets of element equations into a global system of equations for the final calculation.

The subdivision of a whole domain into simpler parts has several advantages:

- accurate representation of complex geometry;
- inclusion of dissimilar material properties;
- easy representation of the total solution;
- capture of local effects.

Finite Element Analysis (FEA) is widely used in engineering applications and uses the mesh generation techniques for dividing a complex problem into small elements. For the analysis, of the column loss scenario presented within this thesis, have been selected the Finite Element Analysis (FEA) software **ABAQUS FEA 6.11** from SIMULIA. The product used was namely Abaqus/Explicit, a special-purpose finite element analyzer that employs explicit integration scheme to solve highly non-linear systems with many complex contacts under transient loads (e.q. impact, impulse).
The reason for the need of non-linear analysis was straigth forward, because it is needed to investigate the post-elastic behavior of the structure in the case of large deformations.

Explicit dynamics procedure (analysis method used in this project) is a mathematical technique for integrating the equations of motion through time and performing a large number of small time increments efficiently. The explicit dynamic integration method is also known as the Forward Euler or Central Difference Algorithm. An explicit central-difference time integration rule is used; each increment is relatively inexpensive (compared to the direct-integration dynamic analysis procedure available in Abaqus/Standard) because there is no solution for a set of simultaneous equations. The explicit central-difference operator satisfies the dynamic equilibrium equations at the beginning of the increment, *t*; the accelerations calculated at time *t* are used to advance the velocity solution to time t+Δt/2 and the displacement solution to time t+Δt. Combining the explicit dynamic integration rule with elements that use a lumped mass matrix (*M* allows the program to calculate the nodal accelerations easiliy at any given time, *t*) is what makes an explicit finite element program work. Abaqus/Explicit provides the following advantages:

- solves highly discontinuous, high-speed dynamic problems;
- very robust contact algorithm that does not add additional degrees of freedom to the mode;
- less disk space than Abaqus/Standard for large problems and more efficient solution;
- easy to simulate quasi-static problems;
- material failure with element deletion for elastic-plastic materials;
- efficiency for large models;
- possibility of impact loading simulation.

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

Table 4.0 Comparison between Abaqus/Standard and Abaqus/Explicit (www.wikipedia.org)

Analysis procedure

For the real scale experiment (Figure 4.2), the testing method chosen was the quasi-static testing method.

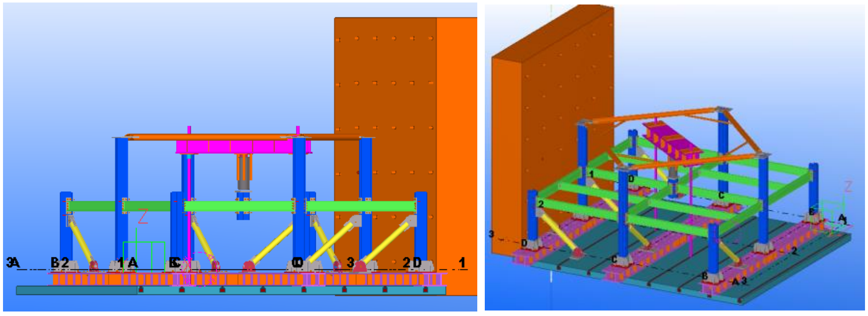


Figure 4.2 Test set-up of the sub-assembly for the experiment

In quasi-static tests, loads and/or displacements are applied at slow rates. Such type of tests are carried out to study structural performance of structures and members such as the rate of propagation of cracks, hierarchy of collapse and associated level of damage, etc. Quasi-static tests are performed by imposing predefined displacement or force histories on the testing specimen.

Different type of displacement histories are shown below (Figure 4.3):

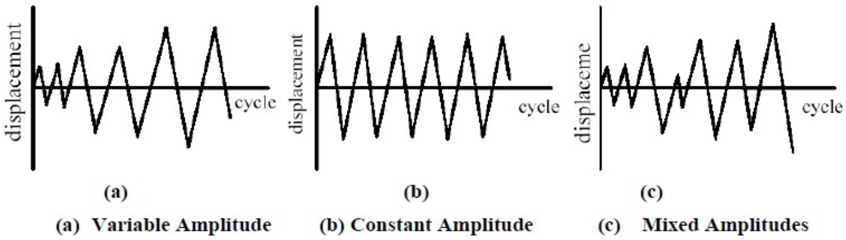


Figure 4.3 Various types of loading histories in quasi-static cyclic tests (www.engpedia.com)

The slow loading rate during the test has the advantage of providing an insight regarding the behaviour of structure/structural member in the post-yielding regime. However, the associated disadvantage is that the effects of acceleration-dependent inertial forces and velocity-dependent damping forces are neglected, which can be significant for some structural types.

A **3D Nonlinear Quasi-Static Analysis** has been chosen as the method for the column loss scenario numerical simulation presented within this paper. Static Nonlinear Analysis presumes removing the damaged column and increasing the applied gravitational loads on the floor adjacent bays to the removed element, on all floors above, without considering the dynamic effect influence. The step by step load increment take into account the second order effects. The progressive collapse assessment is considered to end when the base reaction started to decrease in relation with the input gravitational load. The highest value of the gravitational load, that will not trigger the progressive collapse is considered to be the critical load value. For static analysis (LS and NS), the dynamic effect is employed by the amplification of the loads on the bays above the failed elements by means of a Dynamic Increase

Factor (DIF). Recent studies shown that the value of DIF is between 2.0-1.5. The dynamic increase factor (DIF) usually relates to the increase of gravitational load values in the static analysis in order to obtain the same vertical displacement as in the case of dynamic analysis. DIF will represent the ratio of static over dynamic analysis gravitational loading values for initiatig progressive collapse (the ratio of gravitational forces that trigger out the progressive collapse related to nominal gravity loads (D+0.5L), known as robustness index also or overload factor Ω where Ω=Failure load/Nominal gravity load).

For the slab areas situated above the affected zones, the gravitational forces will be applied by DIF and the load combination will be described by the following equation: DIF x [D+0.5L] where D = dead load, L = live load, DIF = dynamic increase factor. For the remaining areas of slab, the current gravity loading will be applied according to the following equation: [D+0.5L]. It is also necessary to take into account equivalent lateral loadings described by the following equation: 0.002 x (sum of gravity forces (D+L)). In the case of dynamic analysis, the second and third equation is used simultaneously. In order to measure the robustness index, the gravity loading is artifically increased until the failure occurs. By the ratio of the failure load to the nominal load, it is possible to obtain the robustness index or overload factor Ω.

To sum up, by **Static Nonlinear Analysis**, both the material and geometry are treated as nonlinear. A load history from zero load to the full factored load is applied to the structure with a removed vertical load-bearing element. The procedure is presented in more detail in Chapter 1, Section 1.2.4.1. The advantage of this method is its consideration of material nonlinear behavior. However, it still does not consider the dynamic effects such as amplification factors, inertia, and damping forces.

As mentioned in the above parts, for the numerical simulation of column loss scenario, a static nonlinear analysis were carried out considering material nonlinearities of the elements. First, a 3D finite element model of the sub-assembly was built up followed by the removing of the key element (in our case the column from the ground floor). The removed column was modeled by ”cutting” the column at a certain level providing in this manner with rotational and translational degrees of freedom (Figure 4.4).

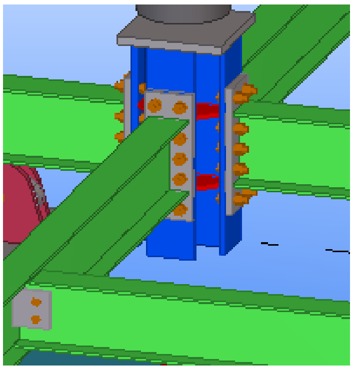
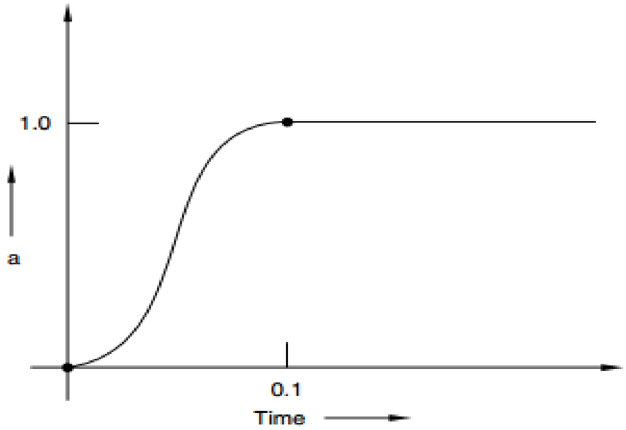


Figure 4.4 Removal of the key element (column)

For the loading of the system, the method of displacement was used. A displacement in the vertical direction was applied at the top plate of the missing column using smooth step data for defining the amplitude curve of the loading (displacement). This method is used to define the amplitude between two points, *a*, between two consecutive data points (t_i,A_i) and (t_{i+1},A_{i+1}) (Figure 4.5).



$t_0 = 0.0 \qquad A_0 = 0.0 \qquad t_1 = 0.1 \qquad A_1 = 1.0$

$$
\begin{aligned}
a &= A_0 \quad \text{for} \quad t \le t_0 \\
&= A_0 + (A_1 - A_0)\,\xi^3\,(10 - 15\,\xi + 6\,\xi^2) \ \text{for}\ t_0 < t < t_1 \\
&= A_1 \quad \text{for}\ t \ge t_1
\end{aligned}
$$

$$\text{where}\ \xi = \frac{t - t_0}{t_1 - t_0}$$

Figure 4.5 Smooth step amplitude definition example with two data points (Abaqus users manual)

This type of definition is intended to ramp up or down smoothly from one amplitude value to another. In this manner the displacement is applied in low increments, from zero to the final values. The analysis is considered to be completed structure collapses or after the full apply of the displacement. After each analysis, a displacement-force curve is requested from the software as output in order to evaluate the behaviour of the system. The material properties of all the structural steel components were modelled using an elastic-plastic material model which incorporates the material's nonlinearity. The concrete material was modelled using a concrete damage plasticity model in Abaqus.

Alternative loadpath analysis necessitates an assessment of the capacity of a structure to dissipate the energy of collapse. Predominantly, this energy is dissipated through plastic strain which is developed by rotation of the connections.

In order to be sure that the models are performing in a static loading situation, the kinetic energy was monitored during the analysis procedure. If the value of the monitored kinetic energy during the analysis was observed to be small, the model was dissipating energy just in a static manner and it was possible the approximation of the analysis being static.

4.3 Numerical modeling of the material characteristics

It is common in structural analysis to use a **linear-elastic material model** for design (Figure 4.5c), and to examine the 'over-stressing' of members and connections. While there is some validity to this approach for minor levels of plasticity, this technique quickly becomes invalid where significant load shedding to alternative loadpaths occurs.

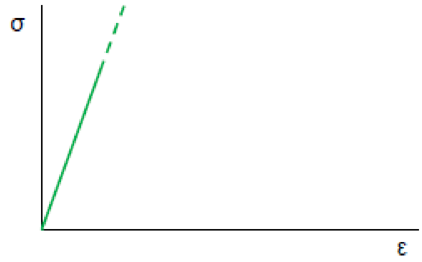


Figure 4.5c Typically linear-elastic material model (Arup)

If plasticity is expected to be significant, a simplistic **linear elastic-perfectly plastic material model** may be assumed (Figure 4.5d). An ideally elasto-plastic model typically allows the dominant effects of the response to be modelled to a sufficient degree of accuracy by capturing the shedding of load through alternative loadpaths.

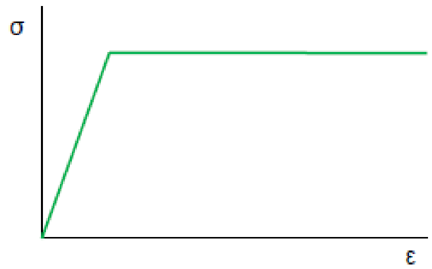


Figure 4.5d Simplistic linear elastic-perfectly plastic material response (Arup)

In some circumstances, modelling **strain hardening** may be desirable in the plastic phase. This is more rigorous than the elasto-plastic model and allows the gradual gain in resistance that results from strain hardening to be described. It therefore leads to a lower level of load shedding to alternative loadpaths than an elasto perfectly plastic model. Though strain hardening can be significant, an elasto perfectly plastic model (Figure 4.5e) is typically found to be adequate for predicting the nonlinear response. Thus, being the solution chosen for our analysis as well.

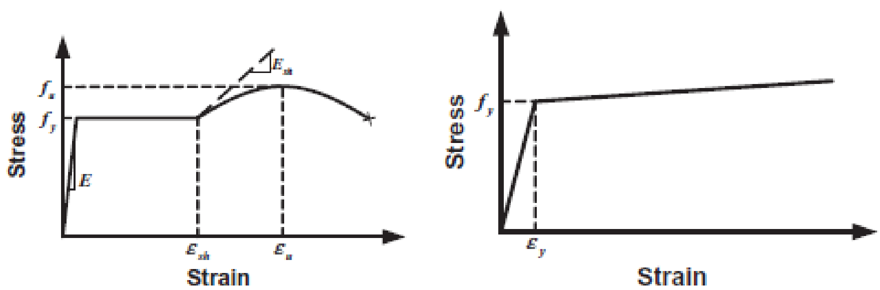


Figure 4.5e Elasto-perfectly plastic model with strain hardening (Eurocode)

4.3.1 Steel S355 material model

Steel is an isotropic material which has good ductility and strength. It generates significant deformation prior to failure. Structural steel grade S355 was used during the analysis for majority of the structural members (including shear connectors), with elastic-plastic characteristics presented in Figure 4.6. The data is obtained from prior experimental tests. Firstly, the density of the material was defined, introducing 7.85E-009 (i.e. 7850 kg/m^3) with uniform distribution. The isotropic elastic properties are completely defined by giving Young's modulus (E=210000 N/mm^2) and Poisson's ratio (ν=0.3). The shear modulus (G) can be expressed by these two terms. For defining the classical metal plasticity property of the material, isotropic hardening model was used by defining yield stress and plastic strain data (Figure 4.7). The failure of material was modeled by gradual decrease of its strength.

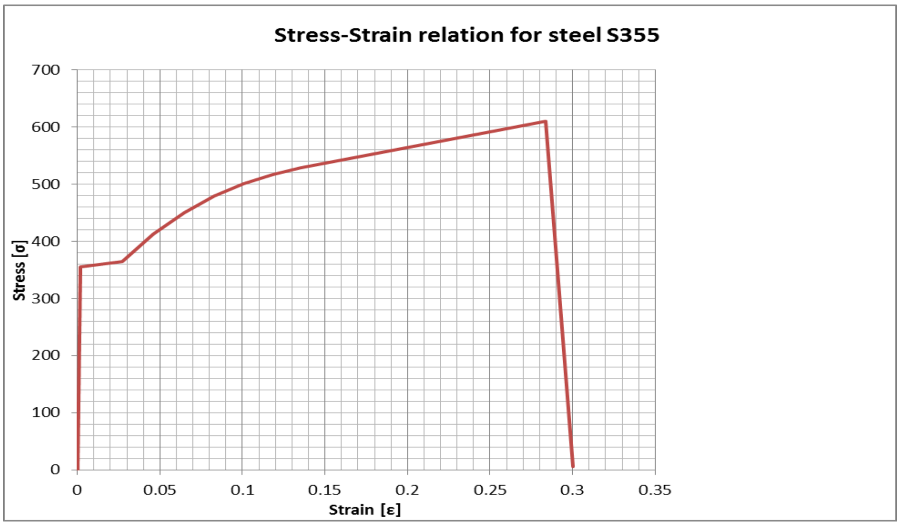


Figure 4.6 Stress-Strain relation of steel S355

Steel S355				
Density (ρ)	Young's modulus (E)	Poisson's ratio (μ)	Yield stress (f.y)	f.u/f.y
[kg/m^3]	*[N/mm^2]*		*[N/mm^2]*	
7850	210000	0.3	355	1.4

Table 4.1 Steel material characteristics

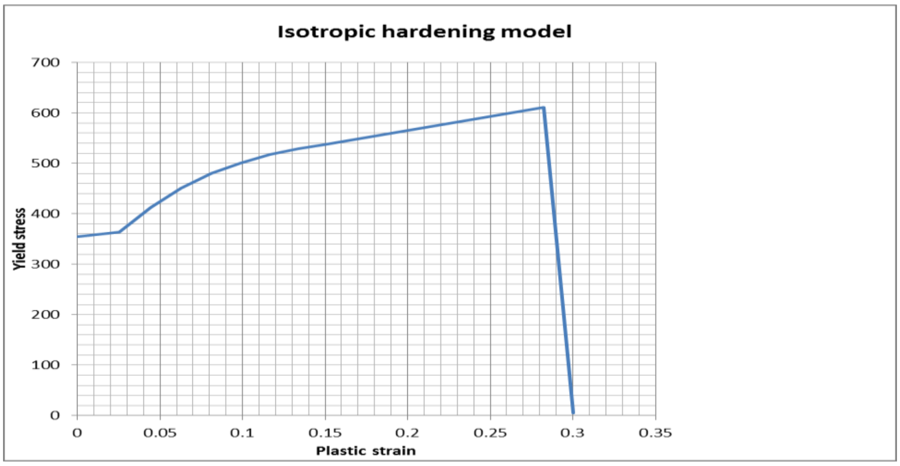


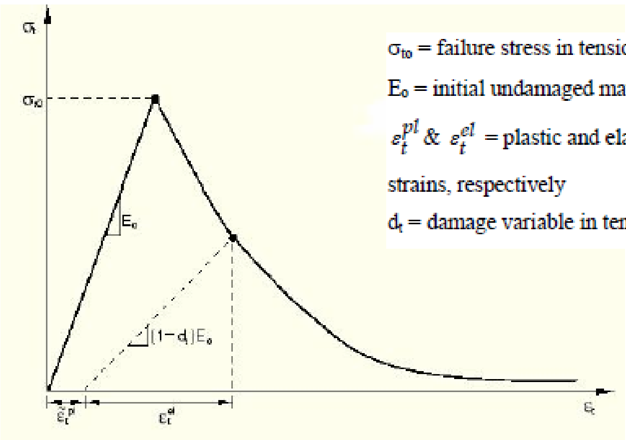
Figure 4.7 Isotropic hardening model data used for defining the nonlinear plastic behavior of steel

4.3.2 Concrete C25/30 material model

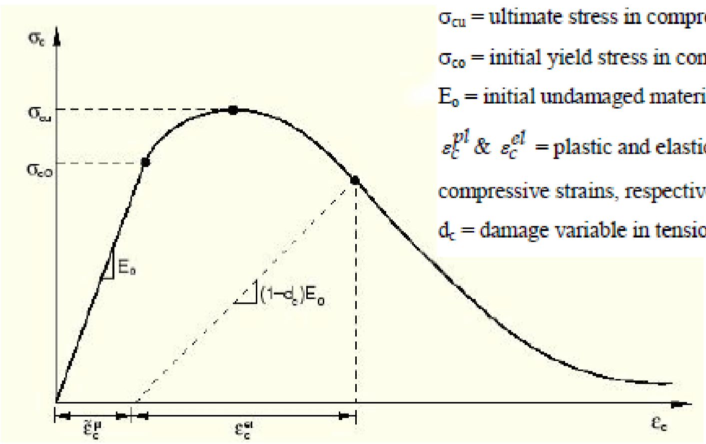
Concrete's mechanical behavior is complex due to the structure of the composite material. The stress-strain behavior is influenced by the development of micro- and macro-cracking of the material. Concrete class C25/30 was selected for the reinforced concrete slab. Firstly, the density of the material was defined, introducing 2.5E-009 (i.e. 2500 kg/m^3) with uniform distribution.

Elastic properties

The elastic properties of concrete mainly depend on its constituent materials, specially the aggregates. The elastic properties were defined by giving Young's modulus (E=32500 N/mm^2) and Poisson's ratio (ν=0.2).



(a) Uniaxial Tensile Loading



(b) Uniaxial compressive loading

Figure 4.2a Uniaxial tensile and compressive model for concrete (Abaqus users manual and EC2)

Plastic properties

Material failure refers to the complete loss of load carrying capacity that results from progressive degradation of the material stiffness. Stiffness degradation is modeled using damage mechanics. Progressive damage and failure can be modeled in bulk materials by continuum constitutive behavior (used in conjunction with the Mises, Johnson-Cook, Hill or Drucker-Prager plasticity models).

For defining the plastic behavior of concrete, **Conrete Damage Plasticity** model (CDP) was used. This model is the most advanced one. It is recommended for a variety of problems at monotonic, cyclic and dynamic loading with account of plastic strains and crack formations. The model allows for interaction with reinforcement and uses non-associated flow law as the base with use of Prager-Drucker (yield function) plasticity model. Prager-Drucker model is a two-parameter model, which takes into consideration the degradation of elastic stiffness induced by plastic straining both in tension and compression. Is often used for constitutive modeling of concrete. It also accounts for stiffnes recovery effects under cyclic loading.
The concrete damage plasticity model in Abaqus, uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. The model is a continuum, plasticity-based, damage model for concrete.
The CDP model follows nonassociated plasticity flow rule, whereby the plastic potential function and the yield surface do not coincide with each other.

Plasticity parameters

Concrete can show significant volume change, commonly referred to as dilation, when subjected to severe inelastic stress states. The dilation can be represented by appropriate plastic potential function. On the other hand, the yield surface can be defined by the hardening rule. In this study, the dilation angle (ψ) was taken as 38° while default values were assumed for all other plasticity parameters. For the flow potential eccentricity (eccentricity is a small positive number that defines the rate at which the hyperbolic flow potential approaches its asymptote) the default value was taken, 0.1. For the the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress (fb0/fc0) the default value was taken, 1.16. K or K_c is the ratio of the second stress invariant on the tensile meridian, to that on the compressive meridian, at initial yield for any given value of the pressure invariant p, such that the maximum principal stress is negative. It must satisfy the condition that $0.5<K<1.0$. The value $K=2/3=0.667$ (default one) was selected.
The material of concrete has the following characteristics (Table 4.2b):

Concrete C25/30		Poisson's ratio (μ)	Dilation angle (ψ)	Eccentricity	fb0/fc0	Invariant stress ratio (K)	Viscosity parameter	Ultimate compression strength (σ.cu)	Maximum tensile stress (σ.tu)
Density (ρ)	Young's modulus (E)								
[kg/m^3]	[N/mm^2]							[Mpa]	[Mpa]
2500	32500	0.2	38	0.1	1.16	0.667	0	22	1.85

Table 4.2b Concrete material characteristics

Concrete damage parameters

When the concrete specimen is unloaded from any point on the strain softening branch of the stress-strain curves, the unloading response is weakened: the elastic stiffness of the material appears to be damaged (or degraded). The degradation of the elastic stiffness is characterized by two damage variables, d_t and d_c , which are assumed to be function of the plastic strains. The damage variables can take values from zero, representing the undamaged material, to one, which represents total loss of strength (Table 4.3 a and b). The choice of the damage properties is important since, generally, excessive damage may have a critical effect on the rate of convergence. It is recommended to avoid using values of the damage variables above 0.99, which corresponds to a 99% reduction of the stiffness, in Abaqus.

Post-failure behaviour under compression is defined by a softening stress-strain response. The strain softening behaviour of cracked concrete in tension is specified by tension stiffening, which is also an indirect way to model interaction between reinforcement and concrete. Tension stiffening can be specified either by means of post-failure stressstrain behaviour in tension or by applying a fracture energy cracking criterion.

No tension and compression recovery was considered in the analysis (i.e. no stiffness recovery). It was assumed in the Table 4.3 that the value of damage is growing from zero to one in proportion to plastic strains.

Stress (MPa)	Strain		Damage
	Full	Plastic	
15.4	0.00047	0.00000	0.0000
19.8	0.00108	0.00047	0.0156
21.56	0.00142	0.00076	0.0253
22.0	0.00200	0.00132	0.0440
21.56	0.00266	0.00199	0.0663
19.8	0.00316	0.00256	0.0853
15.4	0.00358	0.00311	0.1036
12.76	0.00533	0.00486	0.1620
9.20	0.00683	0.00655	0.2180
8.8	0.00816	0.00789	0.2630
7.04	0.01083	0.01062	0.3540
5.72	0.01750	0.01733	0.5760
4.94	0.02516	0.02501	0.8336
4.84	0.02700	0.0269	0.9000

Table 4.3a Property of concrete in compression (V. Korotkov et al, 2004)

Stress (MPa)	Strain		Damage
	Full	Plastic	
1.85	0.000057	0.00000	0.0000
1.46	0.000107	0.00005	0.165
0.814	0.000132	0.000075	0.248
0.52	0.000156	0.0001	0.330
0.41	0.000172	0.00012	0.396
0.31	0.000209	0.00015	0.495
0.19	0.00036	0.0003	0.999

Table 4.3b Property of concrete in tension (V. Korotkov et al, 2004)

Compressive and tensile behavior data

A constitutive model for concrete is primarily based on two main failure mechanisms, namely tensile cracking and compressive crushing of concrete. The next parameters which has to be defined are the compressive behavior and tensile behavior (Table 4.3 a and b). The above shown tabulated values for concrete damage plasticity modeling of concrete C25/30 were taken from *(V. Korotkov et al., 2004*), based on experimental tests.

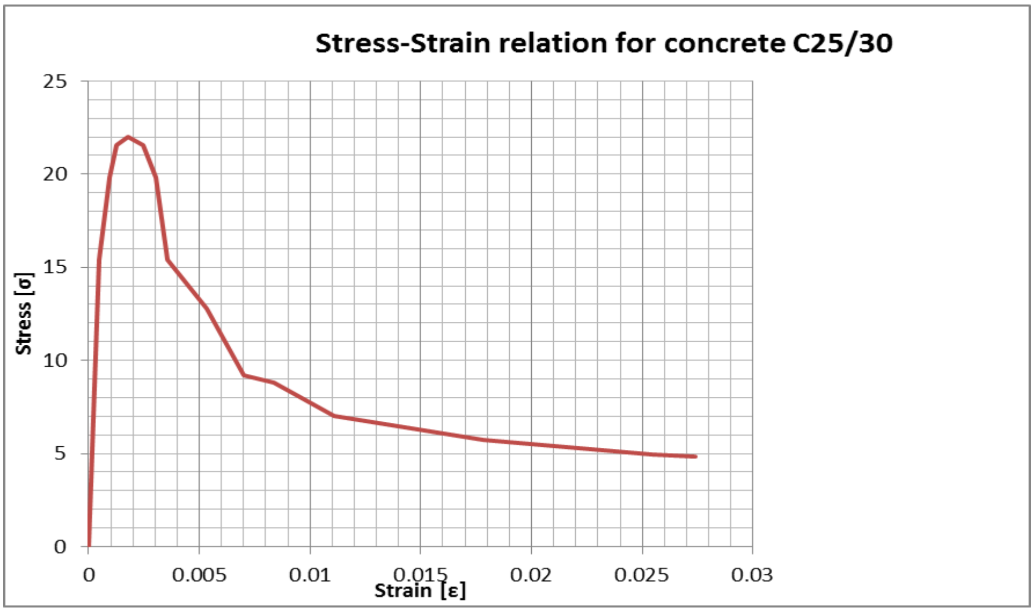


Figure 4.8 Stress-Strain relation of concrete C25/30 (compressive behavior)

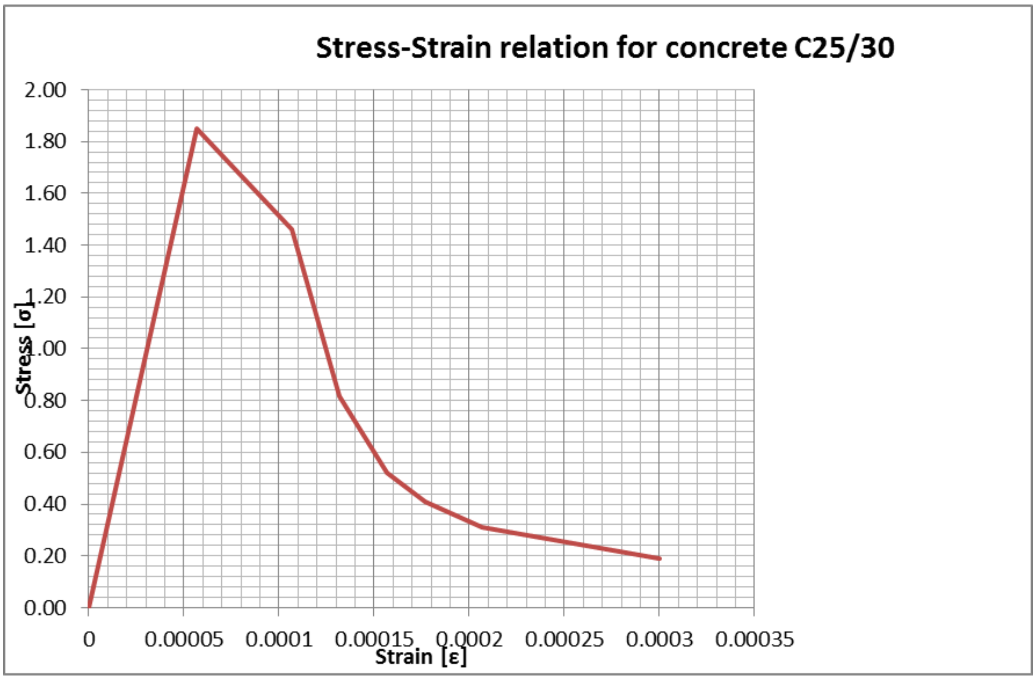


Figure 4.9 Stress-Strain relation of concrete C25/30 (tensile behavior)

4.3.3 Steel S500 material model

Structural steel grade S500 was used for the bracing system shown in Figure 3.8, with elastic-plastic characteristics presented in Figure 4.10. The data is obtained from prior experimental tests. The density of the material was defined, introducing 7.85E-009 (i.e. 7850 kg/m^3) with uniform distribution. The isotropic elastic properties are completely defined by giving Young's modulus ($E=210000\ N/mm^2$) and Poisson's ratio ($\nu=0.3$). The shear modulus (G) can be expressed by these two terms. For defining the classical metal plasticity property of the material, isotropic hardening model was used by defining yield stress and plastic strain data (Figure 4.11). The failure of material was modeled by gradual decrease of its strength.

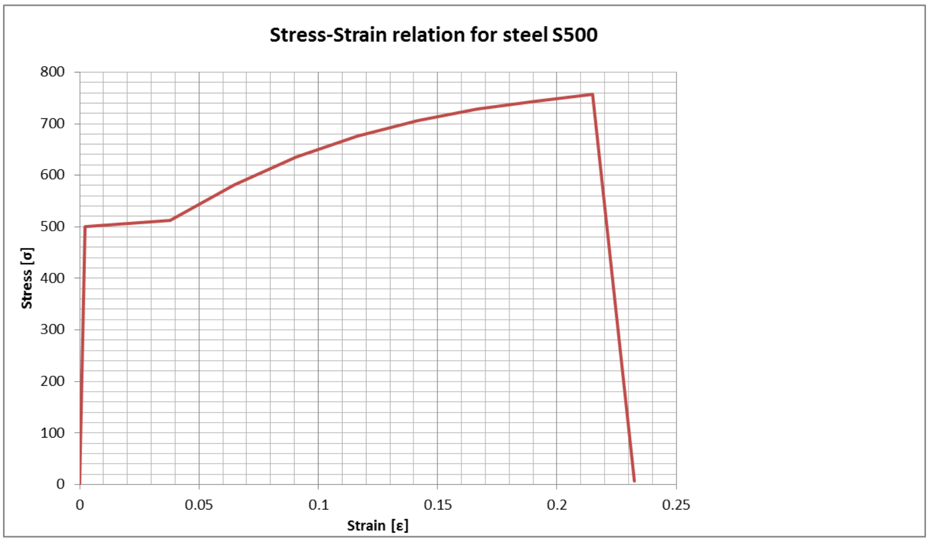


Figure 4.10 Stress-Strain relation of steel S500

Steel S500			
Density (ρ)	Young's modulus (E)	Poisson's ratio (μ)	Yield stress (f.y)
[kg/m^3]	*[N/mm^2]*		*[N/mm^2]*
7850	210000	0.3	500

Table 4.4 Steel material characteristics

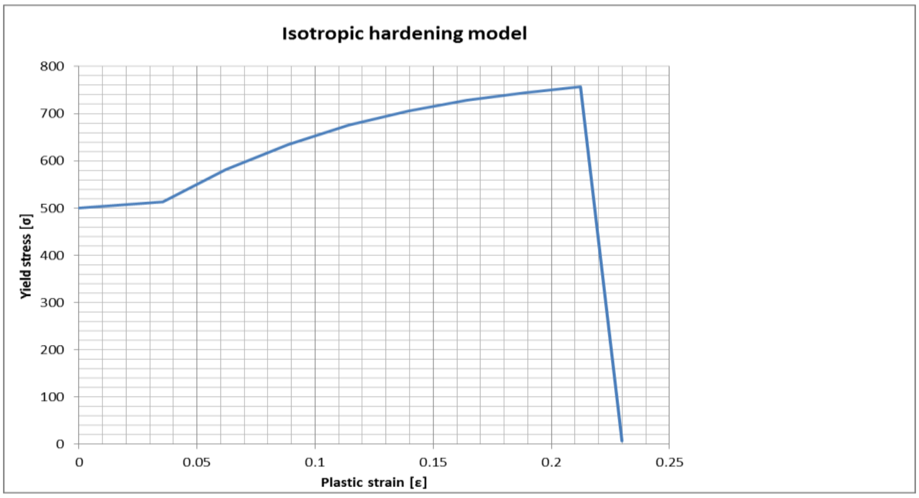


Figure 4.11 Isotropic hardening model data used for defining the nonlinear plastic behavior of steel

4.3.4 Bolt grade 10.9 material model

Structural bolt grade 10.9 was used for the colum-to-beam, beam-to-beam and column base connections, with elastic-plastic characteristics presented in Figure 4.12. The data is obtained from prior experimental tests. Firstly, the density of the material was defined, introducing 7.85E-009 (i.e. 7850 kg/m^3) with uniform distribution. The isotropic elastic properties are completely defined by giving Young's modulus (E=210000 N/mm^2) and Poisson's ratio (ν=0.3). The shear modulus (G) can be expressed by these two terms. For defining the classical metal plasticity property of the material, isotropic hardening model was used by defining yield stress and plastic strain data (Figure 4.13).

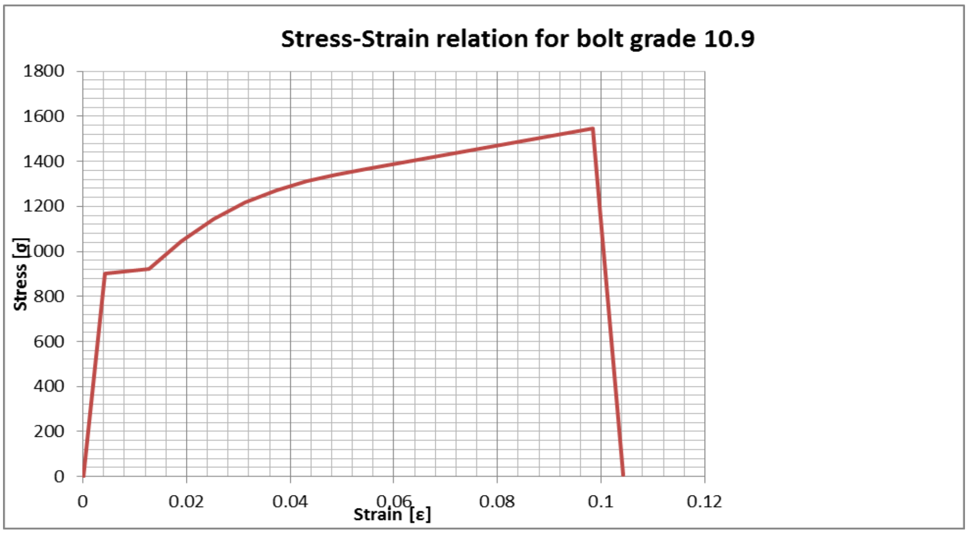


Figure 4.12 Stress-Strain relation of bolt grade 10.9

Bolt grade 10.9 material			
Density (ρ)	Young's modulus (E)	Poisson's ratio (μ)	Yield stress (f.y)
[kg/m^3]	*[N/mm^2]*		*[N/mm^2]*
7850	210000	0.3	900

Table 4.5 Steel material characteristics

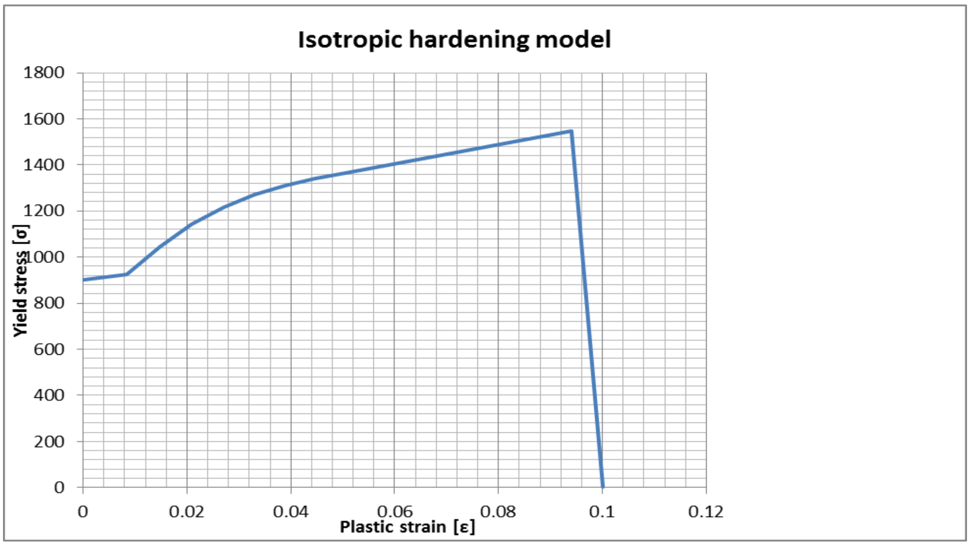


Figure 4.13 Isotropic hardening model data used for defining the nonlinear plastic behavior of steel

4.4 The numerical model

The proposed model is a full 3D finite element model to investigate the progressive collapse of multi-storey buildings. Based on this model, full scale experimental tests will be performed and parametric studies will be carried out to investigate the structural behavior of this type of buildings. To effectively model the selected structure and to reduce the computational time of the analysis, several simplifying assumptions had to be made, which are not affecting the accuracy of the analysis; these will be presented in later chapters.
The aim of this work is to perform two out the four cases presented above, namely the loss of the central column of the sub-assembly in the case of only steel structure and in the case of composite concrete-steel structure.
A modeling approach adopted in this study is to model all structural members and the connections between them, just like they are in the real case scenario. As in the case of a numerical modeling process, the modeling process starts from a smaller model in the beginning (2D model) and proceed to the complex problem (3D model). In the following chapters, this modeling process will be presented from a simple 2D steel frame to the complex and sophisticated 3D composite sub-assembly.

4.5 Catenary effect considering the loss of a column – 2D steel model

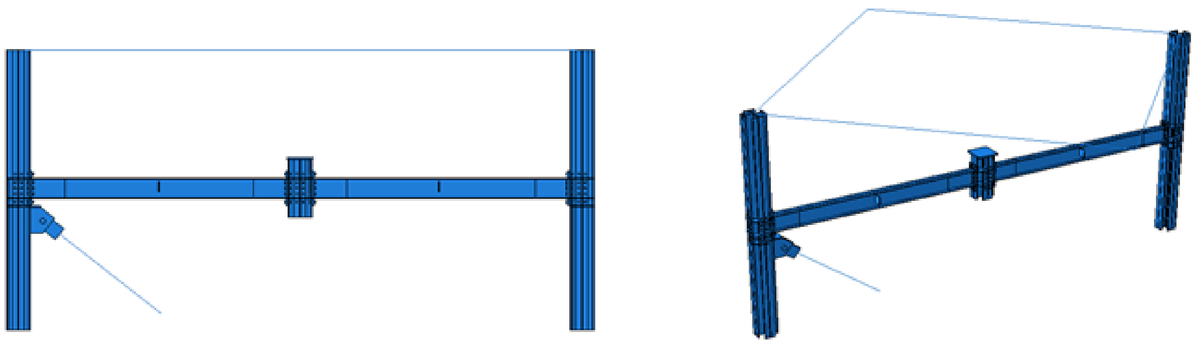


Figure 4.14 Finite element model of the central frame of the sub-assembly

As stated in the section above, the numerical modeling process started with a smaller and less complex 2D model, considering the loss of the central column in the case of steel structure only. After defining the materials in Chapter 4, Section 4.4, the modeling process could began. For security reasons, conservatively, structural steel S355 was assigned for the beam, column, end-plate, stiffening and truss elements (except the bracing system), however, for the full scale experimental tests steel grade S235 will be ordered. From experience, the material will have higher characteristics than the ordered S235. In order to model the exact material behavior, in the future, experimental tests will be carried out on the obtained steel grade and the material

properties will be corrected and calibrated in the modeling software.
For all connections present in the model, steel bolts having grade 10.9 were considered. For the modeling process a 2D system was selected, namely the central frame of the sub-assembly presented in Chapter 3, Section 3.3).
To effectively model the selected system, several simplifying assumptions had to be made. In order to simplify the model, to reduce the working time of the analysis and reduce the number of finite elements involved in analysis, the truss system from the top of the frame and the bracing system as well were modeled using wire element type in Abaqus (Figure 4.14). Another simplification was that the secondary-beams were not modeled in the analysis at this step. Lateral constraints were defined on the end-plates which connect them to the primary-beams, simulating thus the effect of lateral stability provided by the secondary-beams.

The exterior (side) columns and the interior (central) column as well, have been defined as 3D deformable solid vertical elements with constant cross-section of HEA 260 (Figure 4.15). The method of defining them was by extrusion. Solid homogeneous sections was assigned for the elements with the previously defined S355 material. The solid (or continuum) elements can be used for linear analysis and for complex nonlinear analyses involving contact, plasticity, and large deformations.

Figure 4.15 Model of the exterior column

It can be seen in Figure 4.15, the stiffeneres were extruded from the column cross-section in order to avoid the increase of contact surfaces between elements, as well the holes for the connection bolts were also modeled at this stage by cut extrude method. For an accurate mesh, the column was partitioned. Due to the fact that the column section was extruded from once and the stiffeners as well, it was no need to consider further interaction property for the column. Before going further to the meshing of the element, it is necessary to clarify some basic information about element types in Abaqus.
Given the wide variety of element types available, it is important to select the correct element for a particular application. Choosing an element for a particular analysis can be simplified by considering specific element characteristics: first- or second-order; full or reduced integration; hexahedra/quadrilaterals or tetrahedra/triangles; or normal, hybrid, or incompatible mode formulation. By considering each of these aspects carefully, the best element for a given analysis can be selected.

Name convention

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

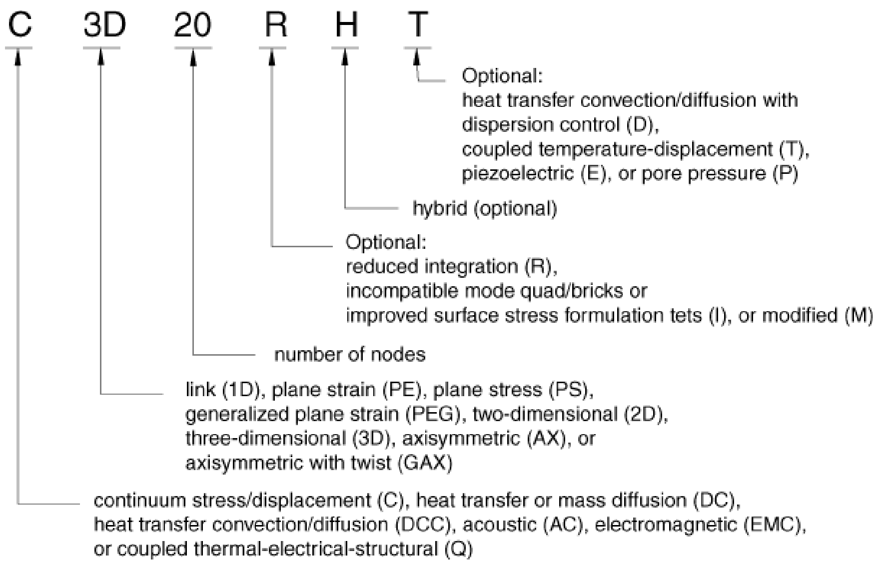


Figure 4.16 Name convention of solid elements (Abaqus user manual)

Mesh element shapes

Most elements correspond to one of the shapes shown in Figure 4.17; 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 in Figure 4.18, a typicall "Hex" (Hexahedra or brick) element shape is presented for the meshing of an element.

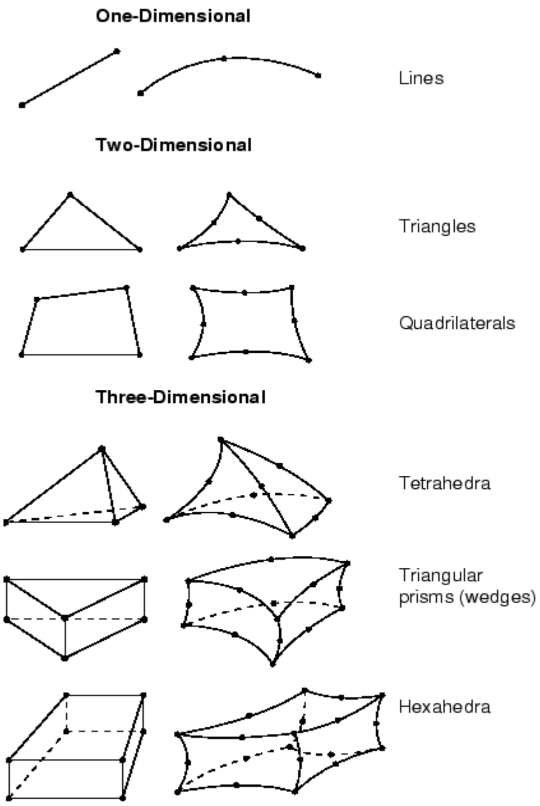


Figure 4.17 Mesh element shapes (Abaqus users manual)

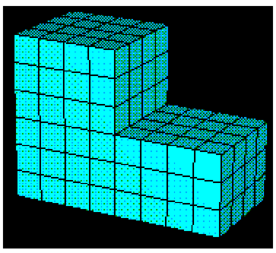


Figure 4.18 Hexahedra element shape

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.

Hourglassing or mesh instability

Hourglassing can be a problem with first-order, reduced-integration elements (CPS4R, CAX4R, C3D8R, etc.) in stress/displacement analyses. Since the elements have only one integration point, it is possible for them to distort in such a way that the strains calculated at the integration point are all zero, which, in turn, leads to uncontrolled distortion of the mesh. First-order, reduced-integration elements in Abaqus include hourglass control, but they should be used with reasonably fine meshes. Hourglassing can also be minimized by distributing point loads and boundary conditions over a number of adjacent nodes.

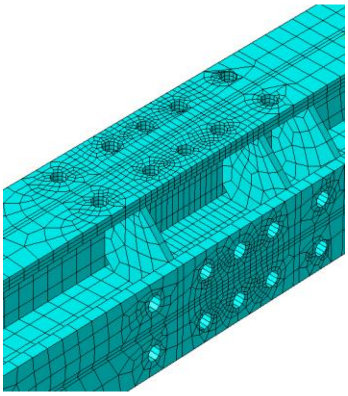


Figure 4.19 Exterior column mesh at the level of the connection

According to the theory presented above, an 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the member from standard element library. This type of element is a stress/displacement element. The family (the family has the meaning of the type of analysis that will be performed with the element) associated to the element was the 3D Stresses (i.e. 3D stress analysis). The column was meshed using structured mesh technique using hexahedral element shape (Figure 4.19). Instead of global seeding of the elements, local seeding was chosen by number method, because this method is more accurate regarding the complex 3D models. The mesh sizes were defined along selected edges by prescribing the number of elements to creat.

In the case of the middle column (damaged column) (Figure 4.20) the stiffeneres, holes for the bolts and top end-plate were created from the cross-section of the element. The material, element type and element characteristics used, were the same as in the case of exterior column, presented above. For meshing the element the same type of element was selected (C3D8R) as previously.

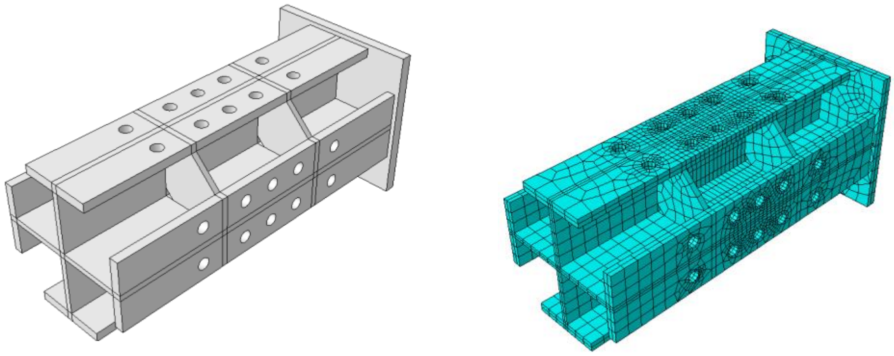


Figure 4.20 Central column model

The primary-girder (Figure 4.21) has been defined as 3D deformable solid horizontal element. The method of defining was by extrusion. Solid homogenous section was assigned for the element with S355 material having cross-section IPE 220. As it can be seen in Figure 4.20, the end-plates were extruded from the beam cross-section, as well the holes for the connection bolts were also modeled at this stage by cut extrude method. For an accurate mesh, the beam was partitioned. Due to the fact that the beam section was extruded from once and the end-plates as well, it was no need to consider further interaction property for the beam. An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the member. The beam was meshed using structured mesh technique and using hexahedral element shape (Figure 4.21).

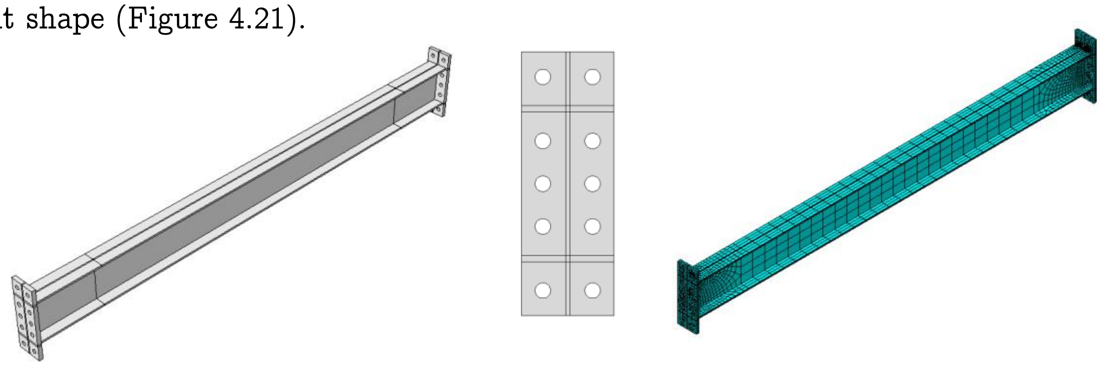


Figure 4.21 Primary-beam model

The bracing system and the truss system from the top of the model have been defined as 3D deformable planar wire elements. Beam type section was assigned for the element, with S500 material for the bracing system and S355 material for the truss system. Beam section integrated during analysis method was selected for both cases. This procedure allow the cross-sectional behavior to be calculated by numerical integration of the stress over the cross-section to define the beam's response as the analysis proceeds. The material behavior is evaluated independently at each point on the section. This type of beam section should be used when the section nonlinearity is caused only by nonlinear material response. For the beam shape, pipe cross-section (Figure 4.22) was selected from the standard beam section library. Geometric input data was r (outside radius), t (wall thickness).

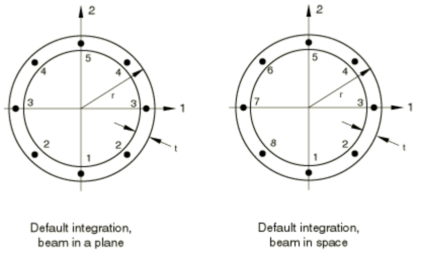


Figure 4.22 Pipe cross-section from standard beam section library (Abaqus user manual)

If it was assigned beam sections to the wire regions of a part or to its stringers, its necessary to assign an orientation to the beam sections by defining the approximate local 1-direction of the cross-section.

A 2-node linear beam in space (B31) element type was selected for the members from standard element library. The family associated to the element was the Beam one. The beam was meshed creating ten elements.

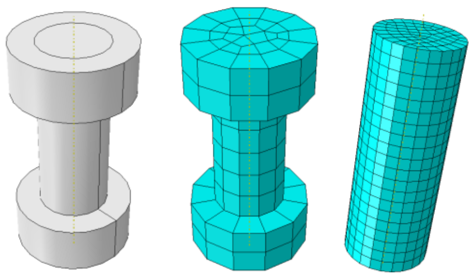


Figure 4.23 Bolt model

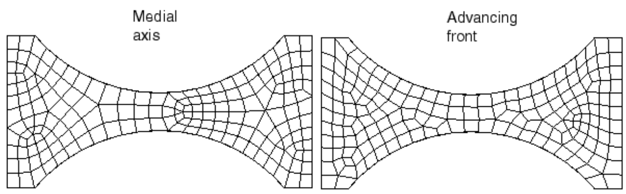


Figure 4.24 Sweep algorithms (Abaqus user manual)

The bolts for the connections (Figure 4.23) has been defined as 3D deformable solid element. The method of defining was by revolution. Solid homogenous section was assigned for the element with bolt grade 10.9 material. As it can be seen in Figure 4.23, the head and the nut of the bolt were created from the bolt cross-section by revolution. For an accurate mesh, the bolt was partitioned. Due to the fact that the bolt was defined from one element, it was no need to consider further interaction property for the bolt. The thread was not modelled directly to reduce the computational time. The diameters of the holes for the bolts was bigger than the shank for non-fitted bolts as the code of practice says (typically 2 mm).

An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the bolt. The bolt was meshed using sweep mesh technique (based on advancing front algorithm instead of medial axis) and using hexahedral element shape (Figure 4.23). Sweep meshing is used to mesh complex solid and surface regions. This involves two phases: a). creation of mesh on one side of the region, known as the source side; b). copies the nodes of that mesh, one element layer at a time, until the final side, known as the target side, is reached.

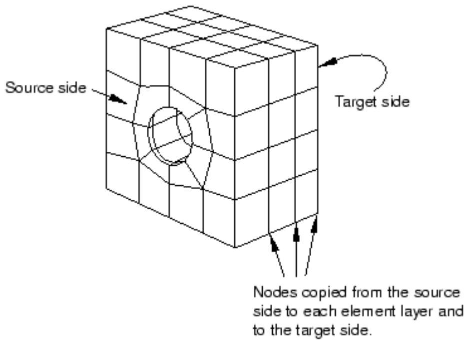


Figure 4.25 Sweep mesh technique (Abaqus user manual)

The advancing front algorithm generates quadrilateral elements at the boundary of the region and continues to generate these elements as it moves systematically to the interior of the region. Both algorithms generate acceptable meshes (Figure 4.24).

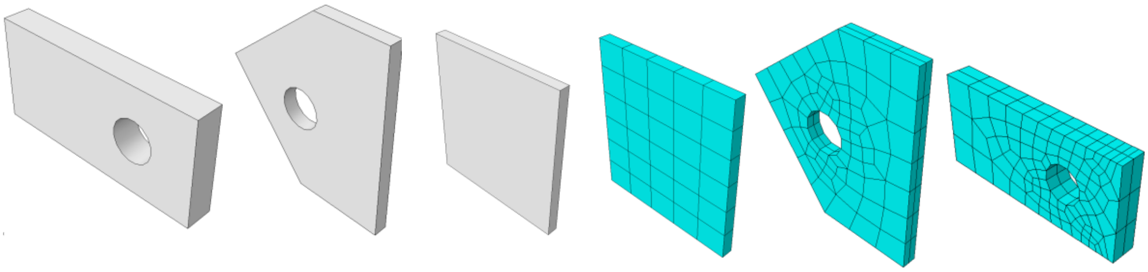


Figure 4.26 End-plate and stiffener models

All the other smaller parts, namely the end-plates and stiffeneres (Figure 4.26) used in the connections and not defined together with one of the main structural members, were defined as 3D deformable solid elements. The method of defining was by extrusion. Solid homogenous sections were assigned for the elements with S355 material. 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the members. They were meshed using both structured and sweep mesh technique (based on advancing front algorithm) and using hexahedral element shape in both cases (Figure 4.26).

After the proper modeling of the structural parts and the assembly of them, the interaction/contact between the parts had to be defined. One of the main reasons for using Abaqus/Explicit module is that, the general contact algorithm present just in the explicit module, provides the possibility of defining automatically many or all regions of the model with a single interaction. Surfaces used in general contact interactions can span many disconnected regions of the model, but allows the use just for 3D surfaces. This method is very fast and simple using sophisticated tracking algorithms to ensure that proper contact conditions are enforced efficiently, with very few restrictions on the types of surfaces involved. To sum up, general contact algorithm is an automatically generated single unified algorithm capable of handling all types of contact interactions. In this manner, a general contact (explicit) was defined for all allowable element faces and model entities, using "All*with self" method for the contact domain and global property assignment (contact property globally to the entire contact domain). Two mechanical contact properties were defined for the general contact. First was the "Normal behavior", with "Hard contact" (uses Penalty Algorithm in Abaqus/Explicit analysis) definition for pressure-overclosure option and default constraint enforcement method (i.e. enforcing constraints to use a contact pressure-overclosure relationship). The penalty contact algorithm results in less stringent enforcement of contact constraints than the kinematic contact algorithm, but the penalty algorithm allows for treatment of more general types of contact (for example, contact between two rigid bodies). The penalty contact method is well suited for very general contact modeling. The second contact property was the "Tangential behavior", with "Penalty" friction formulation (Stiffness or penalty method permits some relative motion of the surfaces (an "elastic slip") when they should be sticking) and using isotropic (uniform) friction coefficient of 0.7 (steel-steel). No shear stress limit was specified and for the elastic slip stiffness, infinite (no slip) option was defined. The general contact method was usefull in reducing the modeling process in the definition of contacts between the several modeled elements such as the bolts-holes, members-members, etc, instead of defining individually contacts manually.

Beside the general contact method (which is usefull for elements being in compression to each other), constraints have been defined for simulating plates and stiffeneres welded in the model, for modeling pinned connections and for simplifying some modeling problems. Constraints partially or fully eliminate degrees of freedom of a group of nodes and couple their motion to the motion of a master node (or nodes). Specially, they are used for modelling adhesive connections, pinned connections, welding, etc. Tie constraint (tie two separate sufraces together so that there is no relative motion between them) was used for the primary-beam and secondary-beam end-plate connection, stiffeneres and end-plates involved in the connection of the column-to-bracing, etc.

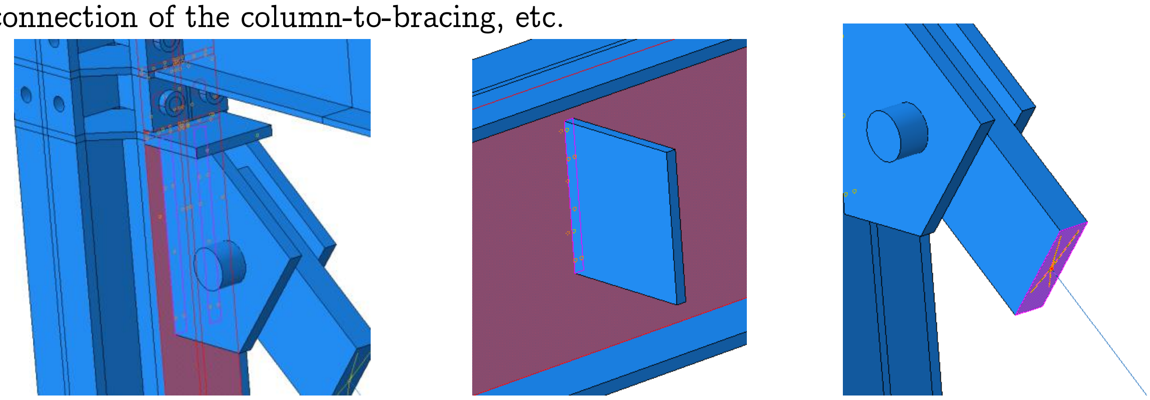
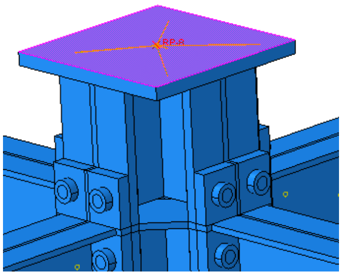


Figure 4.27 Tie constraint for the bracing, end-plate connection modeling

Coupling constraint (allow the constrain of the motion of a surface/constraint region to the motion of one or more control/reference points) was defined for pinned connection of the bracing system and truss system at the top of the frame, for coupling the motion of the column base sections into one control point and coupling the load application surface into one refrence point in order to apply the loading in that point. Constraints have been defined both manually by creating separately for each surface and by contact pair detection tool provided by the software. This provides a fast and easy way to define contact interactions and tie constraints in a three-dimensional model. Instead of individually selecting surfaces and defining the interactions between them, we can select automatically to locate all surfaces in a model.



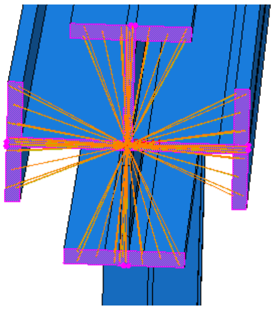


Figure 4.28 Coupling into control points the motion of the top-plate and column base

As stated in the Chapter 4.2, for the analysis procedure, general dynamic explicit type was defined. Analysis type during which the response can be either linear or nonlinear is called a general analysis type. An analysis type during which the response can be linear only is called linear perturbation analysis type. A general analysis type is one in which the effects of any nonlinearities present in the model can be included. Nonlinear stress analysis problems can contain up to three sources of nonlinearity: material nonlinearity, geometric nonlinearity, and boundary nonlinearity. During the definition of the analysis procedure, the "Nlgeom" or large-displacement formulation option was enabled (i.e. accounting for geometric nonlinearity during the analysis). Automatic incrementation with global stable increment estimator was chosen. In nonlinear problems (those with large-deformations and/or nonlinear material response) the highest frequency of the model will continually change, which consequently changes the stability limit. Abaqus/Explicit has two strategies for time incrementation control: fully automatic time incrementation (where the code accounts for changes in the stability limit) and fixed time incrementation. In automatic time incrementation two types of estimates are used to determine the stability limit: element-by-element and global.

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.

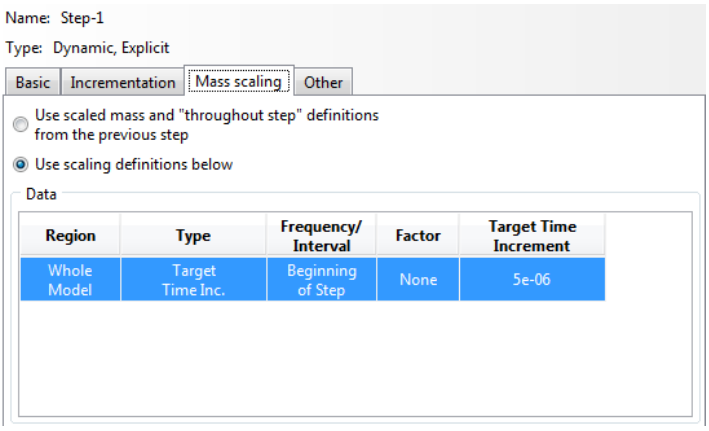


Figure 4.29 Mass scaling of the model

In addition to the default history output, a history output request was defined by the so called ”set” domain with 500 intervals for the load application point and requesting as output data the vertical displacement (U2) and vertical reaction (RF2) at this node. It will be usefull this at the level of results, obtaining the force-displacement curve automatically at this point.
The next phase of modeling was the application of boundary conditions and application of loading. Displacement/rotation type of boundary condition was assigned to constrain the movement of the inferior part of the bracing system

(pinned), the column base connection (fixed), end-plate connection of the secondary-to-primary beam (pinned), etc. For the loading of the structure the method of displacement was used. A displacement of 800mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column.

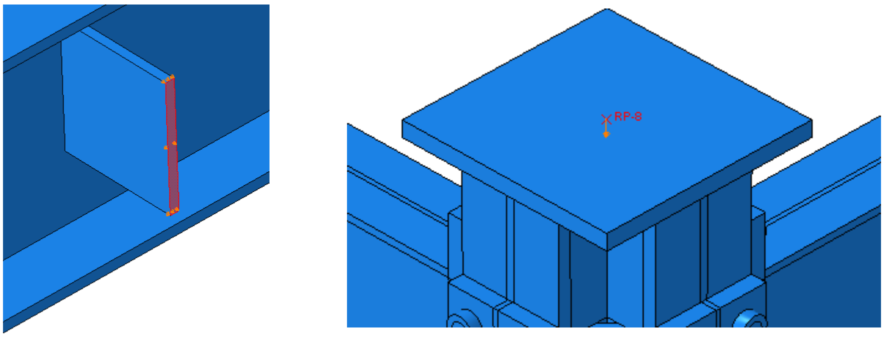


Figure 4.30 Boundary condition modeling and application of the displacement based loading

Full analysis was created, using multiple processors (4) and single precision.
In Figure 4.30 it can be observed the deformed shape of the frame and the PEEQ (equivalent plastic strain) distribution within the structure.

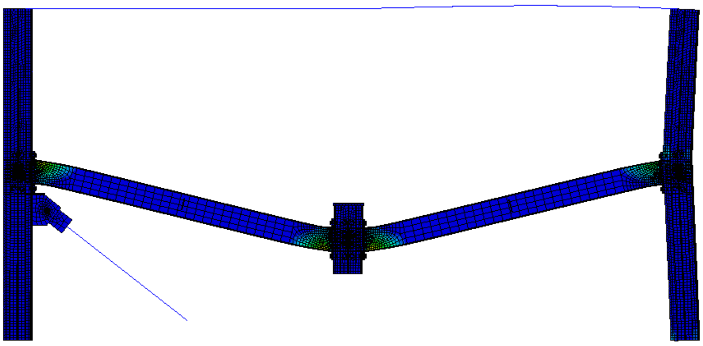


Figure 4.30 Deformed shape of the structure

As it can be seen in the Figure 4.31, the development of the plastic hinges occured at the most affected zones of the beams, namely at the end of the beams.

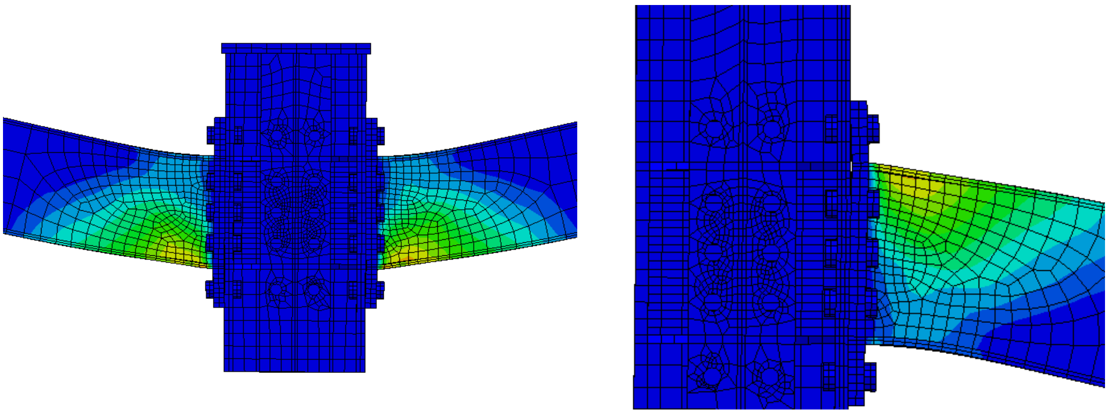


Figure 4.31 Development of plastic hinges

The **nonlinear static response** of a damaged structure is derived from the analysis of the system in its damaged condition. The gravitational load is applied proportionally in a static analysis and the nonlinear static response is thus derived. Typically, the static response comprises an initially linear phase (characterized by a state where external loads are resisted entirely by the bending action) followed by significant non-linearity due to geometric non-linearity (P-Δ effects).

At the end of this phase, the plastic hinges are developing at the end of the beams. This is followed by material non-linearity (plasticity) characterized by plastic rotations and increasing axial forces. The external loads are resisted by both flexure and axial tension. After which there may be either hardening (due to catenary or membrane action and it is called also catenary phase, characterized by a significant reduction of the flexural capacity at the plastic hinges while the catenary capacity starts to develop more and more) or softening (due to buckling or failure of subsequent structural elements) of the response prior to the ultimate failure of the structural system.

In the catenary phase, the external loads are resisted entirely by the catenary action until the capacity is achieved and the collapse is occured.

These different phases are illustrated schematically in Figure 4.32a:

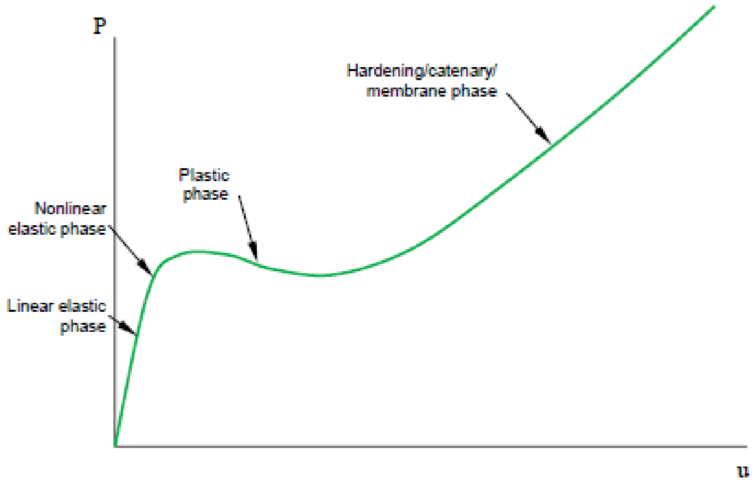


Figure 4.32a Typically nonlinear static response of the damaged structure (Arup)

The phases of the nonlinear statis response of a damaged structure can be seen in Figure 4.32, where the force-displacement curve was generated for the 2D steel model presented above. Its clearly visible that the response of the 2D steel model comprises an initially linear phase. At this phase the external load is resisted only by bending action. This phase is followed by significant nonlinear phase. At the end of this phase, the plastification of the end of the beams occuring. This is followed by material non-linearity or plasticity characterized by plastic rotations and increasing axial forces in the beams. At this phase the applied load is resisted by both flexure and axial tension. The last phase is called catenary or membrane phase and its characterized by significant reduction of the flexural action while the catenary forces are significantly increasing followed at the end by failure (see Figure 4.32).

For the output of the analysis, the main data was the force-displacement curve of the load application point (Figure 4.32).

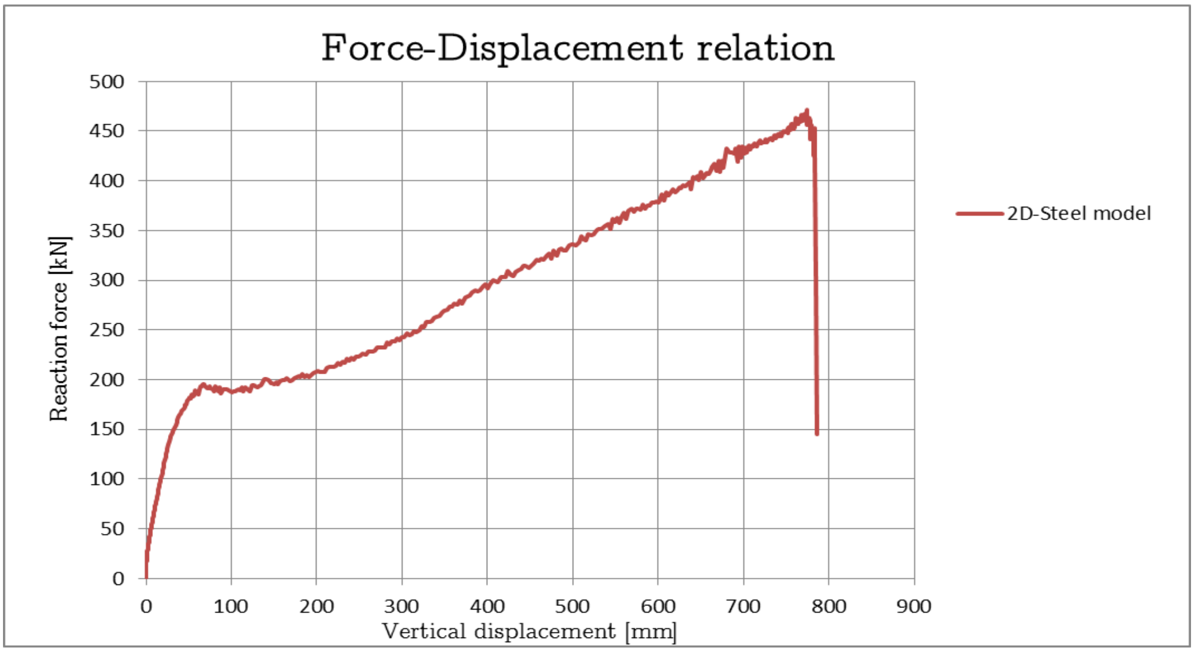


Figure 4.32 Force-Displacement curve for the 2D-model

The plastification of the beam started firstly at the superior flange, close to the connection with the exterior column (Figure 4.33) and than proceed to the inferior flange of the beam, near central beam-to-column connection (Figure 4.33).

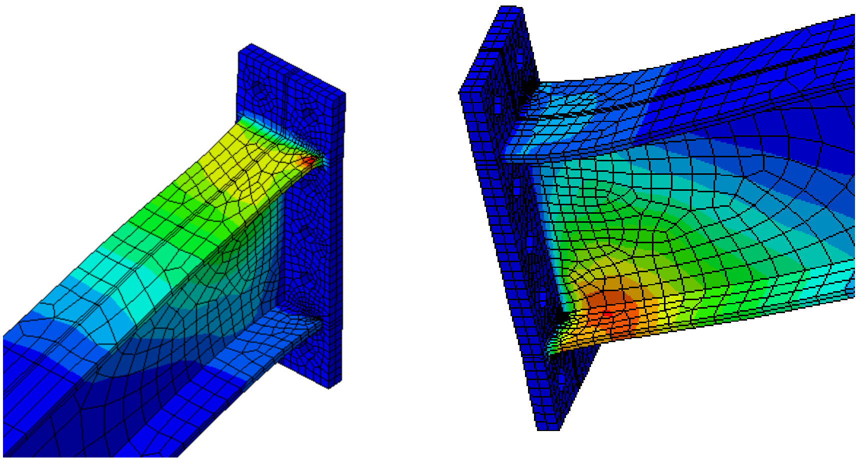


Figure 4.33 Development of plastic hinges at the end of beams

Locally, significant plastification of the beams occured at a displacement of 720 mm and an applied loading of 437 kN. At this level, the stresses in the beams reached the critical values inputed (ultimate capacity of the material) as it can be seen in the Figure 4.34. However, on a global level, the value of the force was still increasing instead of decreasing. This may happen due to some parts of the structure which are still able to undertake loading, due incorect material behavior definition at the level of modeling. Even the value of the force was still increasing, this increase was not significant. At a displacement of 770 mm, the failure of the structure occured with a maximum load of 470 kN (Figure 4.32). As it can be seen in Figure 4.35, the failure occured at the most affected zone, namely at end of the beam, near to the beam-to-column connection.

In the Figure 4.34, the unit of measure used is the S, Mises or **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:

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

Finally, the failure criterion, that the engineer can check, whether Von Mises stress induced in the material exceeds yield strength (for ductile) of the material, can be simplified as: $\sigma_v \geq \sigma_y$

More general representation can be:

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

Where σ and τ represents the normal and shear stresses respectively.

In the following Figures, the parts coloured with grey can be considered failed or exceeding their material limits. This was a sign convention of the software used.

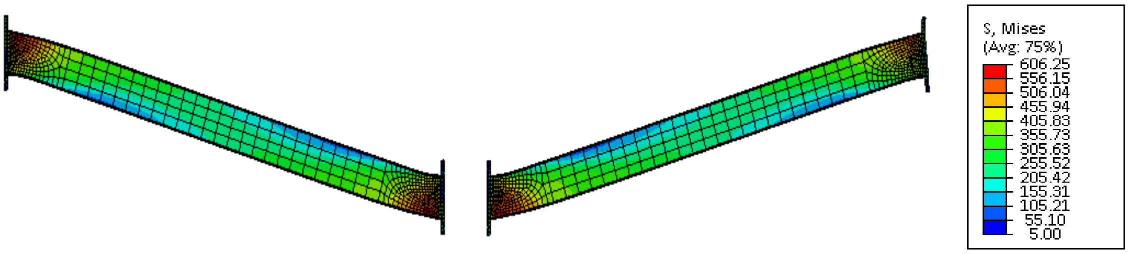


Figure 4.34 Stresses in the beams at a displacement of 720 mm, load of 437 kN

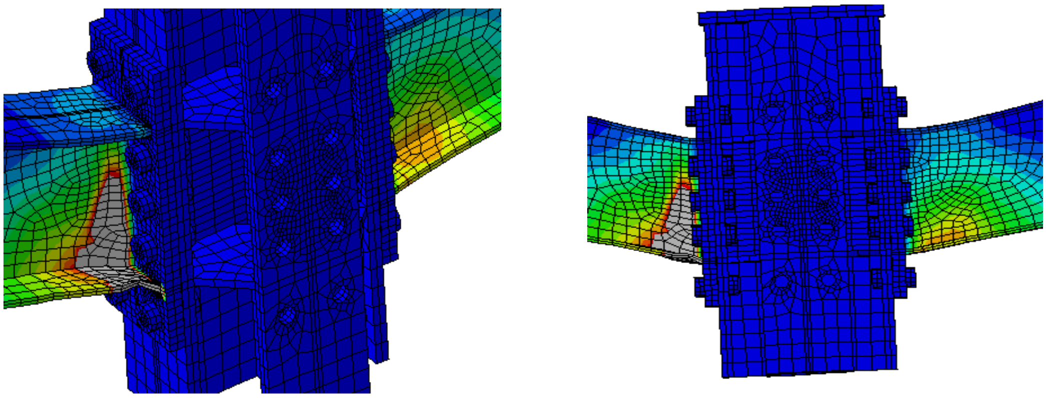


Figure 4.35 Failure of the structure at a displacement of 770 mm, load of 470 kN

During the analysis of the 2D frame, some problems occured. One of them being the tendency of rotation of the middle column during loading procedure. In reality, this phenomena is constrained mainly by the presence of next storeys of the building. In the Figure 4.36 its clearly visible this tendency of rotation.

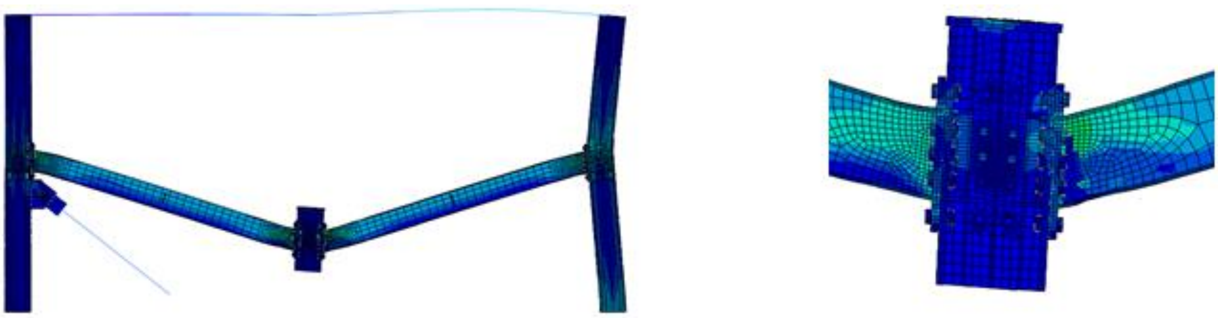


Figure 4.36 Rotation tendency of the loaded column (2D model)

However, this can be a result of the asymmetry of boundary conditions.
In order to decide whether the phenomena occurs due to the absence of the frame perpendicular to the analyzed frame (3D structure) or it is just a deficient simulation of the global behavior of the structure, it was decided to proceed with the modeling to the 3D model case and have a look over the behavior of a three dimensional sub-assembly. This problem of rotation tendency it will be analysed more in further chapters.

4.6 Catenary effect considering the loss of a column – 3D steel model

For modeling the three dimensional structure, the frame on the perpendicular direction on the previously studied one was selected (Figure 4.37). In order to effectively model the selected system, several simplifying assumptions were made in this case also. In order to simplify the model, to reduce the working time of the analysis and reduce the number of finite elements involved in analysis, the truss system from the top of the frame and the bracing system as well were modeled using wire elements (Figure 4.38). Another simplification, as in the previous case, was that the secondary-beams were not modeled in the analysis. Lateral constraints were defined on the end-plates which connect them to the primary-beams, simulating thus the effect of lateral stability provided by the secondary-beams (Figure 4.37).

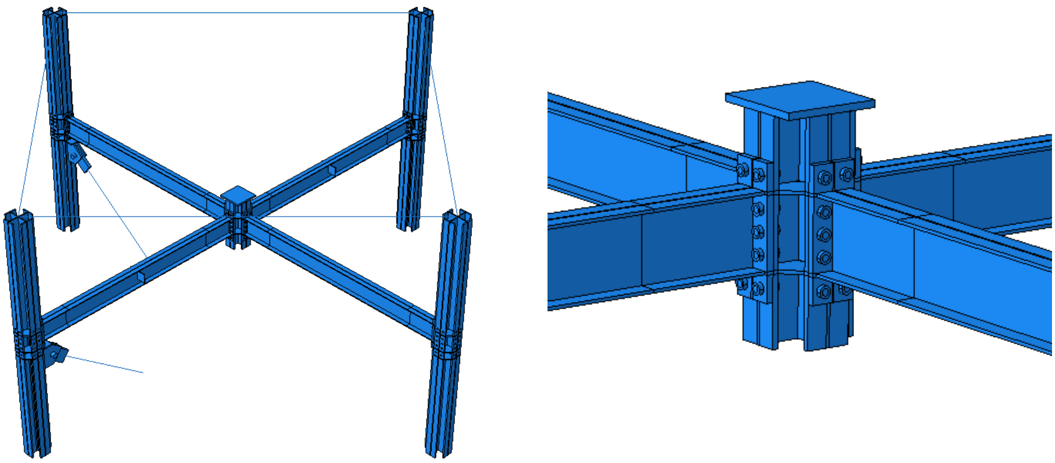


Figure 4.37 Finite element model of the 3D sub-assembly

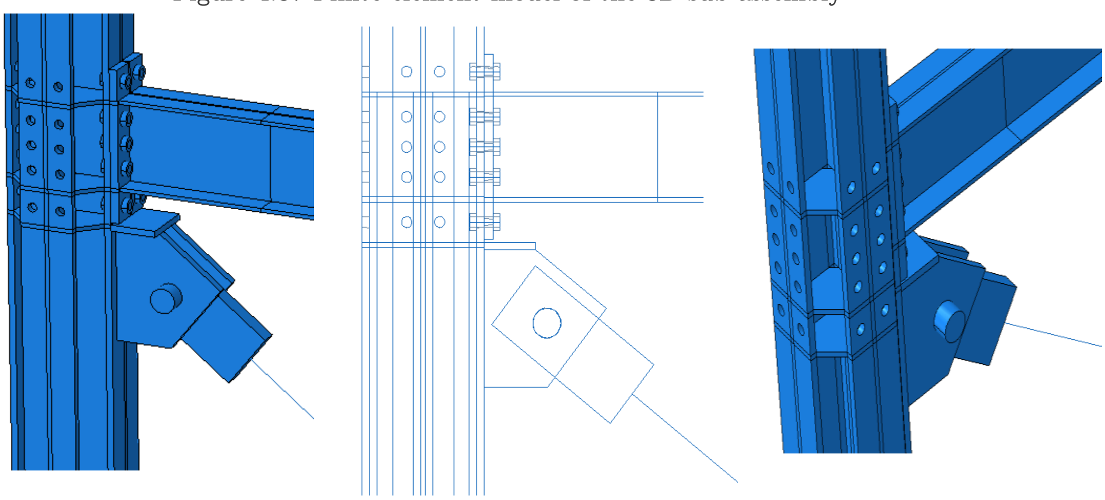


Figure 4.38 Bracing system connection detail in Abaqus

As from the point of view of structural members and connections modeled or from the point of view of element type chosen for the member modeling and interactions/constraints/boundary conditions applied to these, the modeling proces was performed in a similar manner to the 2D model case. In this manner, only the differences will be presented for this and for the future cases as well. Figure 4.39 illustrates the mesh on the assembly.

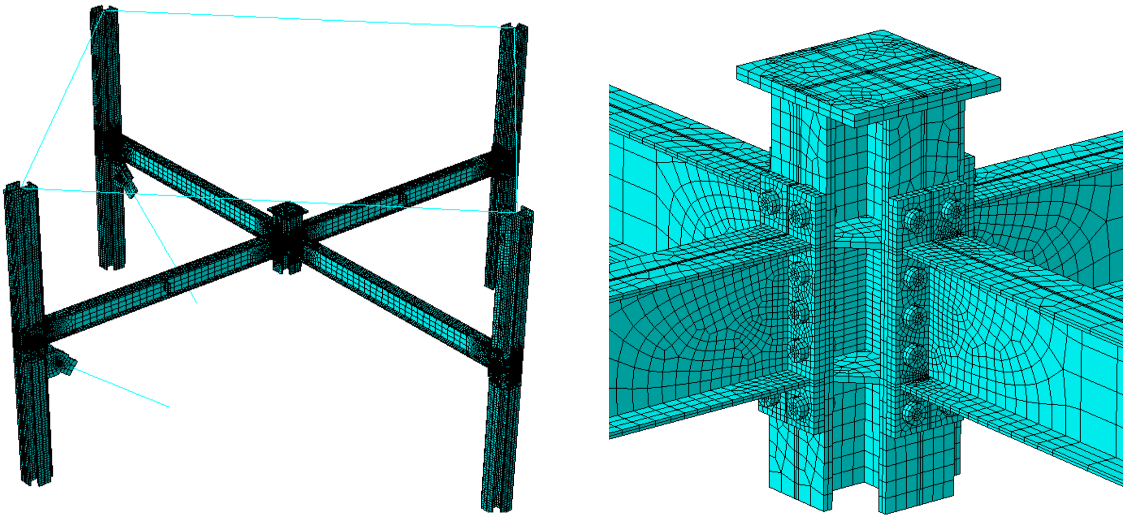


Figure 4.39 Mesh defined for the 3D model

It was not mentioned in the previous chapter that the primary-beam was partioned in three parts in order to have a refined mesh at the level where the plastic hinges may occur and a more global mesh at the middle of the beam (Figure 4.40).

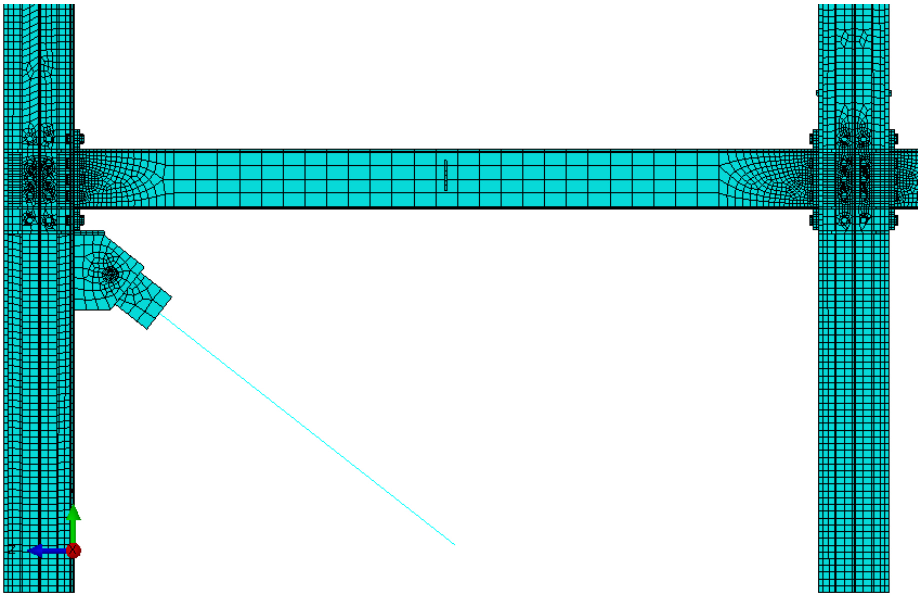


Figure 4.40 Mesh defined for the primary-beam

The mesh is refined usually around the connections to ensure that the large strain and stress gradients in such regions are correctly captured. Although the mesh is somewhow coarse in some locations, to reduce the modeling effort and associated computational demands. Extensive mesh size sensitivity studies confirmed that the model produces reasonable results in this manner.

For the loading of the structure the method of displacement was used once again. A displacement of 800mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step. The loading was applied to the control point initially created and constrained with the superior part of the top plate of the central column.

Full analysis was created, using multiple processors (4) and single precision degree.

In Figure 4.41 it can be observed the deformed shape of the frame and the PEEQ (equivalent plastic strain) (left), Mises (von Mises equivalent stress) (right) distribution within the structure.

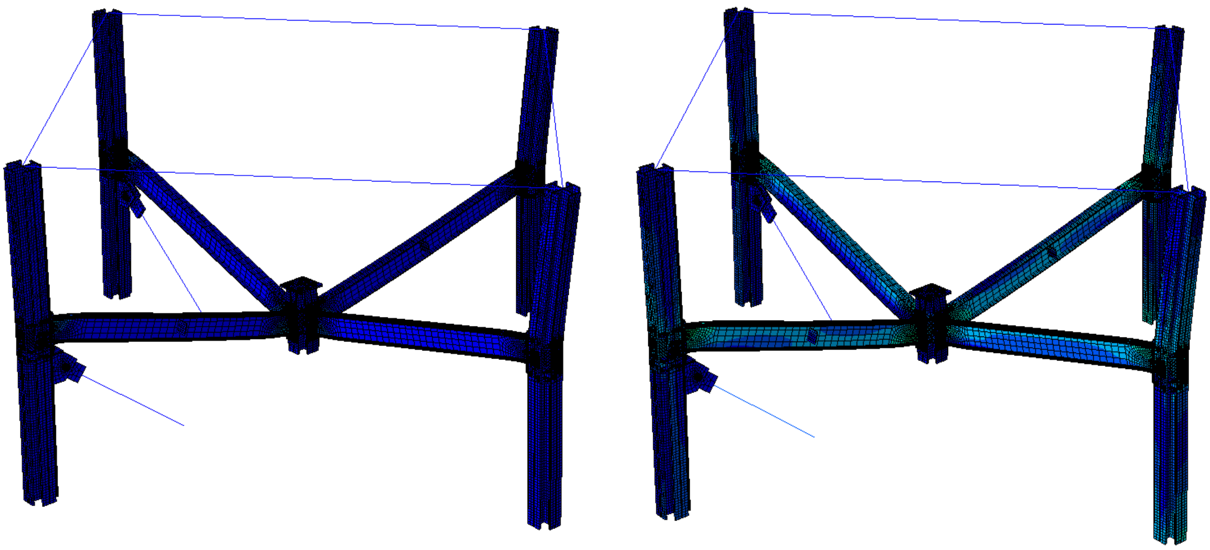


Figure 4.41 Deformed shape of the structure

As it can be seen in the Figure 4.42, the beams were plastified at the most affected zones, exactly where it was expected, namely at the end of the structural member.

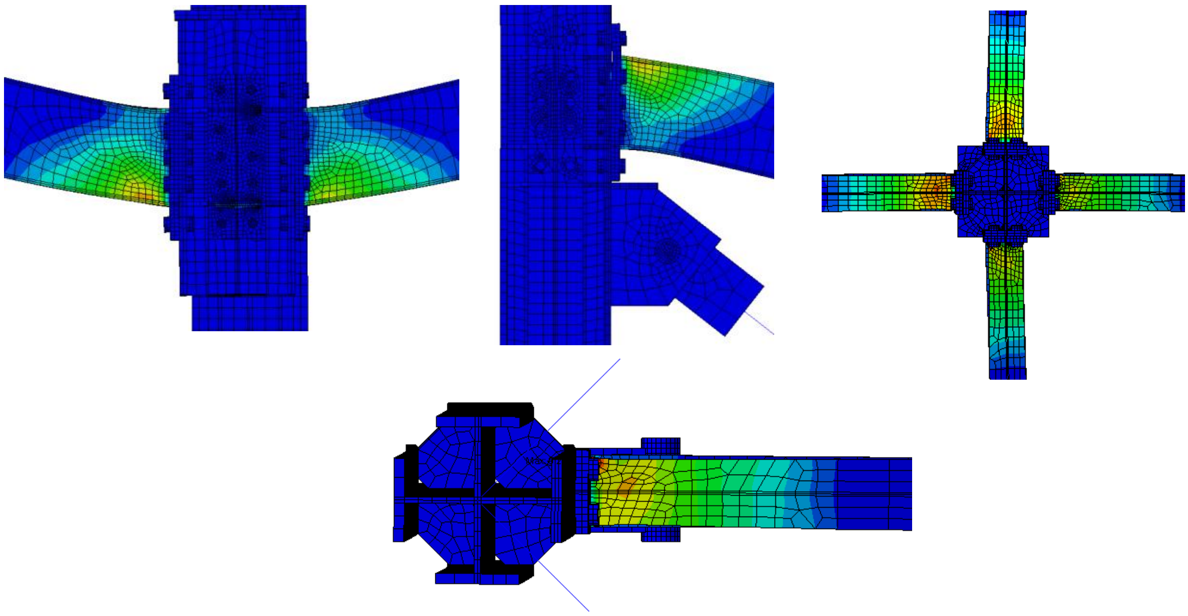


Figure 4.42 Development of plastic hinges in members

For the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.43). See below:

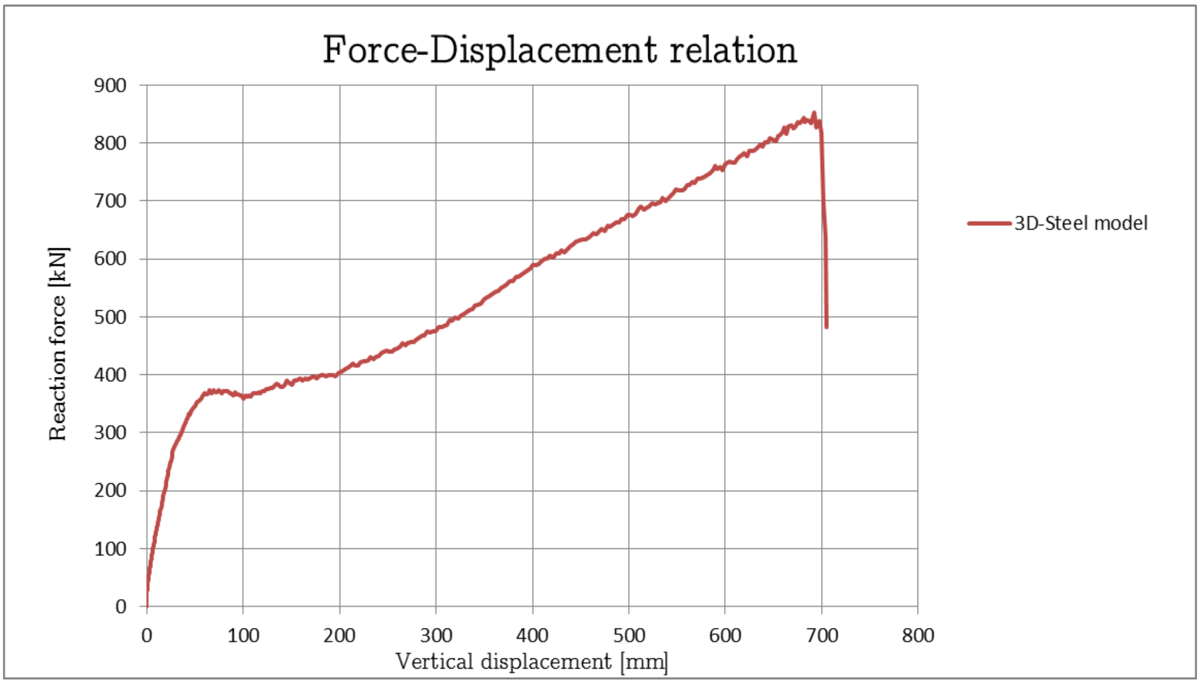


Figure 4.43 Force-Displacement curve for the 3D-model

The plastification of the beam started firstly at the superior flange (similar to the 2D case), close to the connections with the exterior columns (Figure 4.44) and than proceed to the inferior flange of the beam, near central beam-to-column connections (similar to the 2D case) (Figure 4.44). At the end, the failure of the beam was observed.

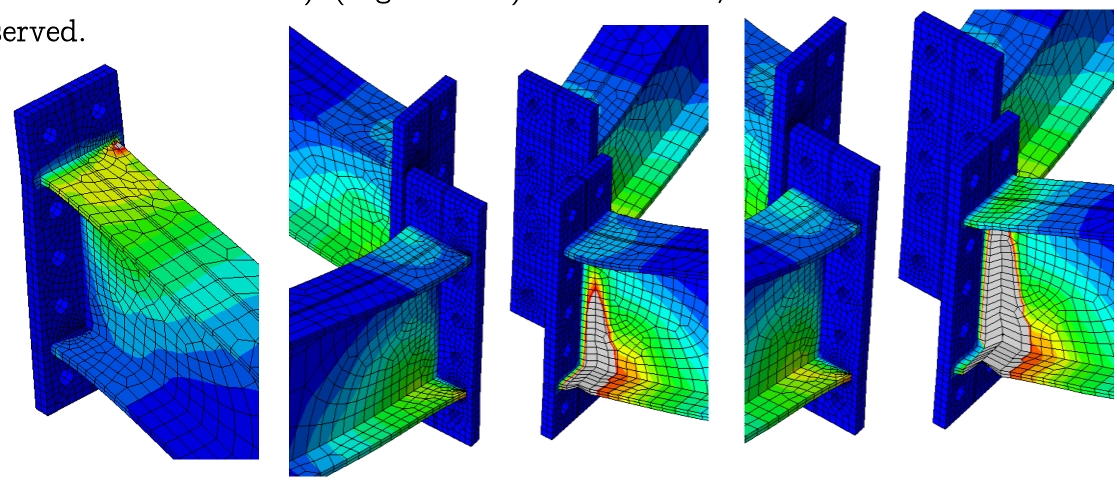


Figure 4.44 Development of plastic hinges at the end of beams

From local point of view, significant plastification of the beams occured at a displacement of 692 mm and an applied loading of 852 kN. At this level, the stresses in the beams reached the critical values inputed (ultimate capacity of the material) as it can be seen in the Figure 4.45. What concerns the global behavior, compared to the 2D model, the value of the forces started to decrease rapidly after the failure of the beam occured. So, it can be considered that at a displacement of 692 mm, the failure of the structure occured with a maximum load of 852 kN (Figure 4.43). As it can be seen in Figure 4.46, the failure occured at the most affected zone, namely at end of the beam, near to the beam-to-column connection.

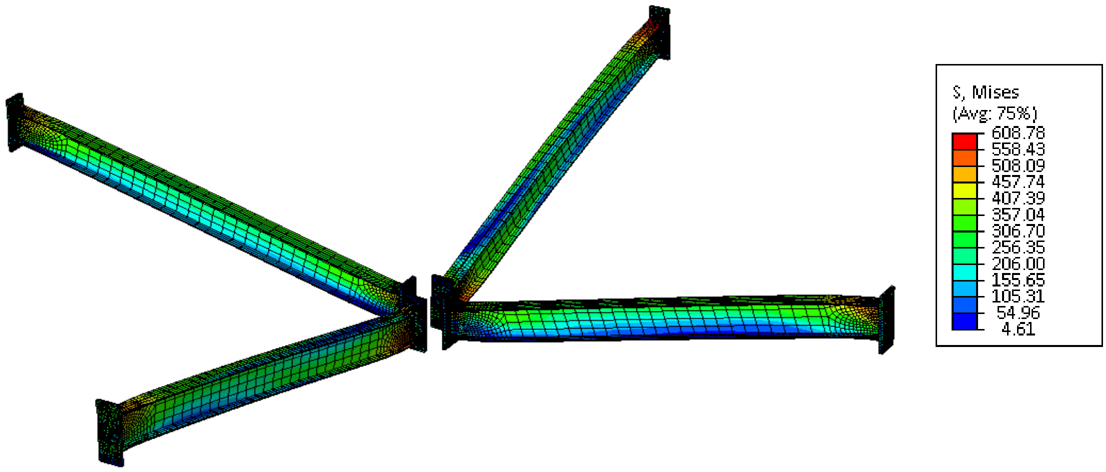


Figure 4.45 Stresses in the beams at a displacement of 692 mm, load of 852 kN

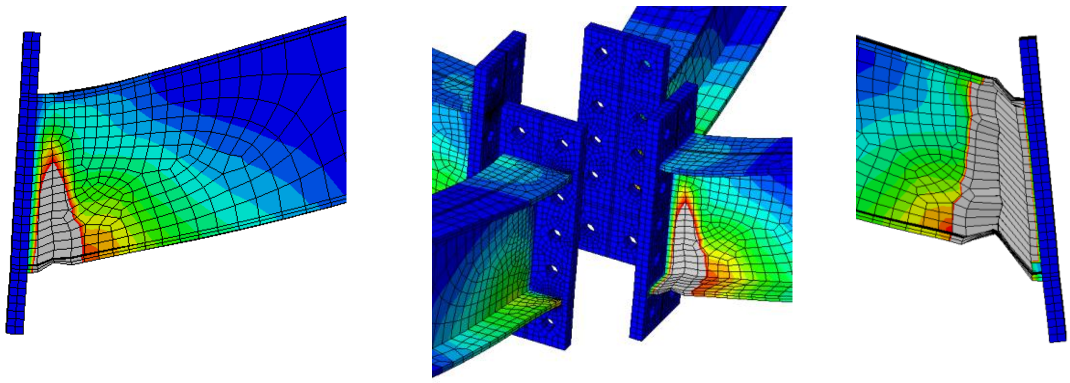


Figure 4.46 Failure of the structure at a displacement of 692 mm, load of 852 kN

Compared to the 2D model, the achieved failure force is aproximately double. This is due to the two directional framing system, resulting an increased (double in our case) bearing capacity of the structure. Following this approach, the expected failure force would be 840 kN (2 X 470 kN). The difference between the expected failure force for the 3D model and the actually obtained value is 1.43%. In the case of the 3D model, the plastification of the end of the beams occured in earlier stages, at a displacement of 692 mm compared to the first situation, when they started to plastify just at a displacement of 720 mm.

Regarding the rotation tendency of the damaged column during analysis, we noticed that the same problems occured as in the case of the 2D model. The column started to have exceesive translational (except the vertical direction) and rotational movements after a period of loading (Figure 4.47). The influence on results of this phenomena was discussed several times during the beginning of the modeling process. In order to understand better the behavior of the column and to be sure that this phenomena will not affect the results measure during the analysis, a decision was made. A case was modeled when the rotation tendency of the column different from the vertical direction (loading direction) was constrained. In what follows, the model with the restrained column will be presented.

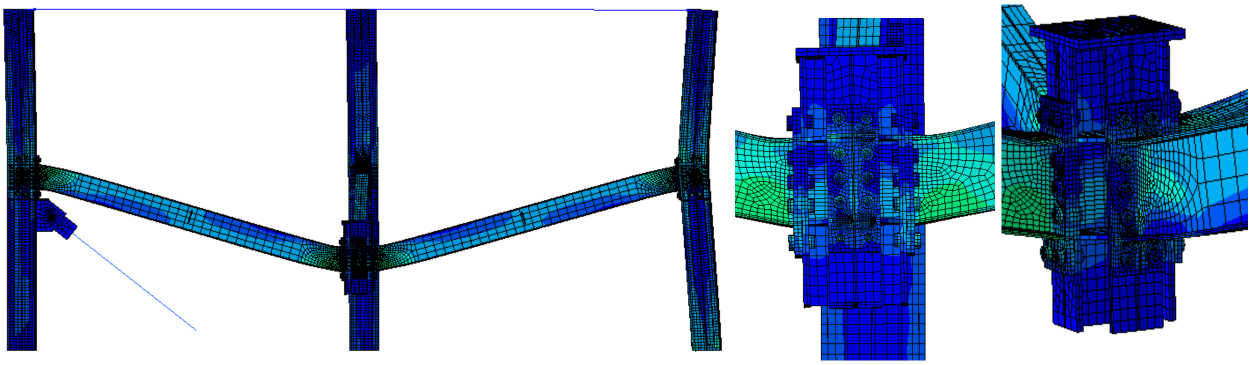


Figure 4.47 Rotation tendency of the loaded column (3D model)

In the Figure 4.48 below, it can be seen the 3D model of the structure when the rotation tendency of the middle (damaged) column is constrained.

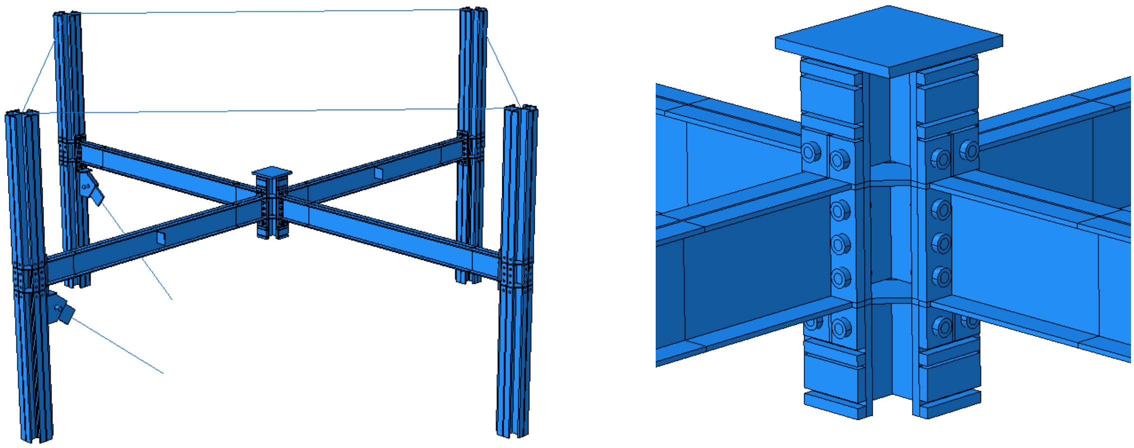


Figure 4.48 Constrained rotation tendency of the loaded column (3D model)

The constraints were modeled steel plate members defined by 3D deformable solid elements. The method of defining was by extrusion. Solid homogenous sections were assigned for the elements with S355 material. 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the members. They were meshed using structured mesh technique and using hexahedral element shape (Figure 4.49).

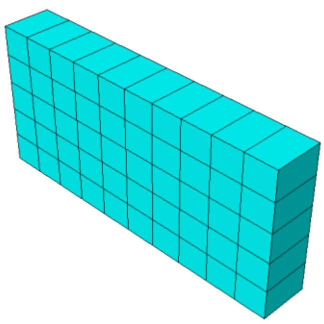
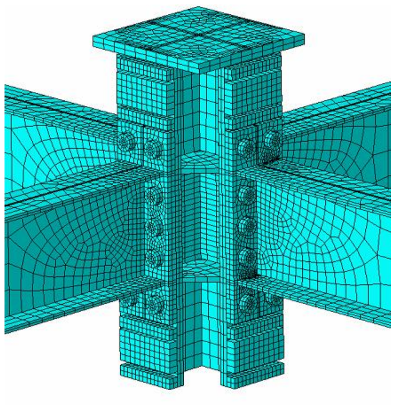


Figure 4.49 Meshing of the steel plates



Tie constraint (tie two separate sufraces together so that there is no relative motion between them) was used for the steel plate-to-column connection.

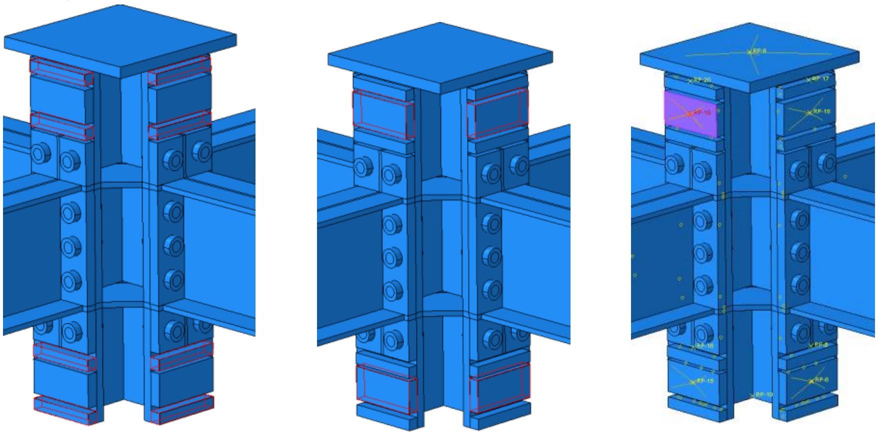


Figure 4.50 Constraints on the column-to-steel plate connection

As it can be observed on Figure 4.50 (on the left with red), the steel plates (stiffeneres) were tied to the column surface using surface-to-surface discretization method (generating the tie coefficients such that stress accuracy is optimized for the specified surface pairing). In this manner, the plates tied to the column flange surface will move to the column during the loading process. The steel plates with red in the Figure 4.50 (middle) were not tied to the column, thus they will move together/separetly with the column during the application of loading. As it can be see in Figure 4.50 (right), the steel plates were connected by coupling to refrence/control points by kinematic coupling type. Kinematic coupling, constrains the motion of the coupling nodes to the rigid body motion of the reference node. This was applied to all the 8 steel plates on the four surfaces of the column.

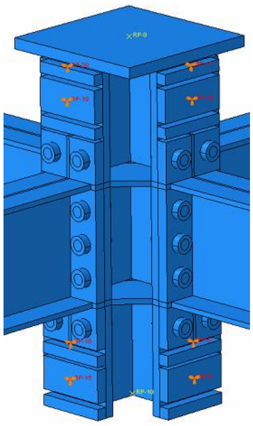


Figure 4.51 Boundary conditions on the steel plates

After coupling the control points with the surfaces of the steel plates, boundary conditions were applied. All the rotational and translational degrees of freedom, except the vertical displacement ability, of the steel plates which were supposed to move together with the column, were constrained by applying displacement/rotation type of boundary conditions. This method was supposed to improve the column displacement in the vertical direction, constraining the tendency of rotation or excessive translation. When the vertical displacement was applied to the column, the column would start to move downward and the steel plates defined in Figure 4.50 (middle) were supposed to move together with the column. Because we discussed previously and we agreed that the column might not have equal displacement with the steel plates, the solution presented above was accepted in which case the steel plates are moving together with the column until a time step, after which will be constrained by the upper/smaller steel plates to have excessive displacement related to the column and "fly" away (Figure 4.50, left).

In Figure 4.52 it can be observed the deformed shape of the frame and the PEEQ (equivalent plastic strain) (left), Mises (von Mises equivalent stress) (right) distribution within the structure.

Full analysis was created, using multiple processors (4) and single precision degree. For the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.53).

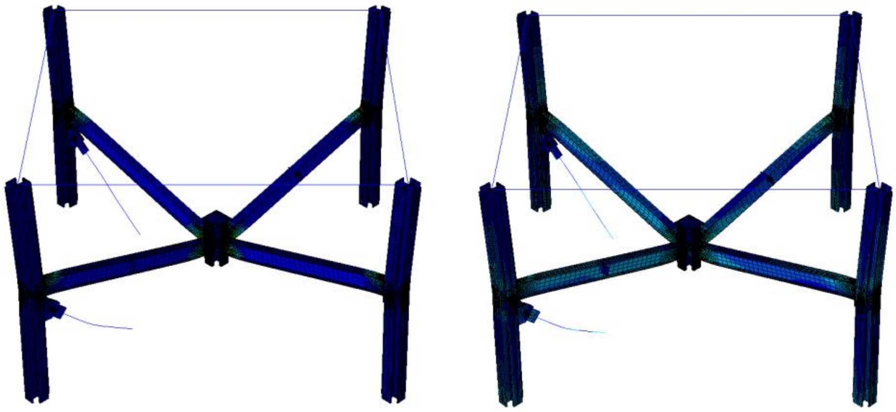


Figure 4.52 Deformed shape of the structure

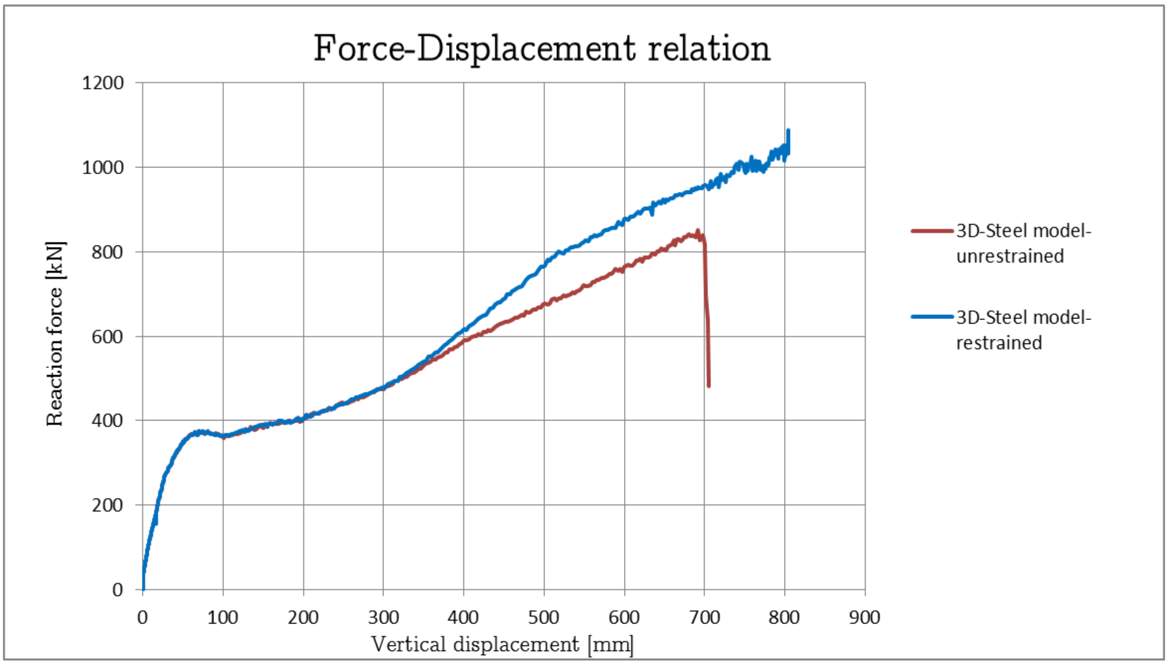


Figure 4.53 Force-Displacement curve for the 3D-model (restrained column case)

Regarding the development of plastic hinges of the structure, it was similar to the 3D case with rotation tendency of the column. Significant plastification of the beams occured at a displacement of 758 mm and an applied loading of 1025 kN.
At this level, the stresses in the beams reached values close to critical values inputed (ultimate capacity of the material) (Figure 4.54). What concerns the global behavior, compared to the 3D model (unrestrained column case), the value of the force was still increasing instead of decreasing (Figure 4.53). By restraining the column to have translational and rotational degrees of freedom, it was provided an additional stiffness to the structure and in this manner the load bearing capacity of the system increased significantly.

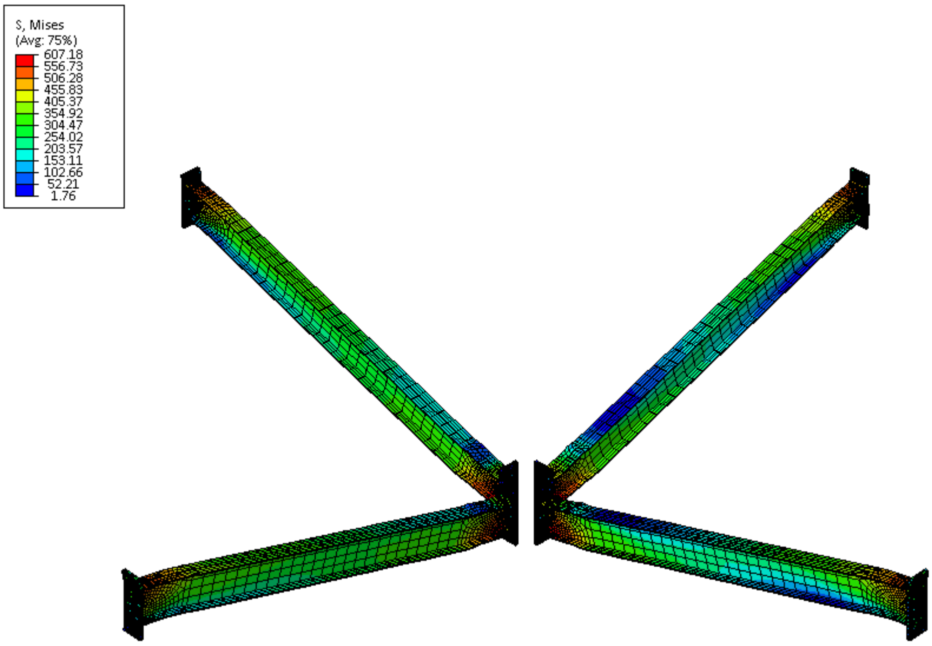


Figure 4.54 Stresses in the beams at a displacement of 800 mm, load of 1086 kN

Considering this increase of stiffness, it was not possible to achieve the failure of the structure with an imposed displacement of 800 mm. However, significant plastification started around 758 mm and an applied load of 1025 kN. As it can be seen in the Figure 4.53, the behavior of the two modes were similar until a displacement of 380 mm, after which due to the restrained column and additional stiffeness applied, the 3D model with restrained column could overtake higher applied loading and had an increased resistance which concerns the failure.

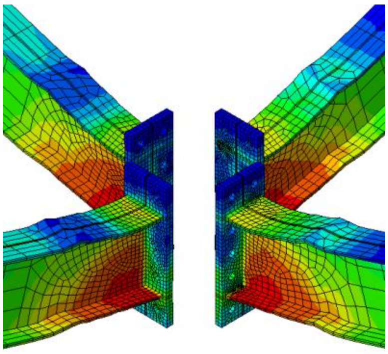
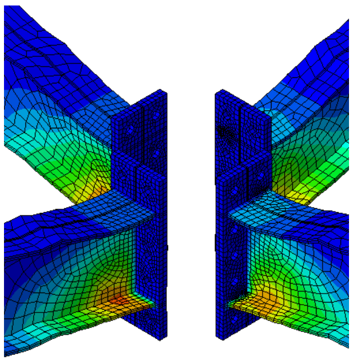


Figure 4.55 Von Mises stress (left) and equivalent plastic strain (right) distribution at the end of the beams

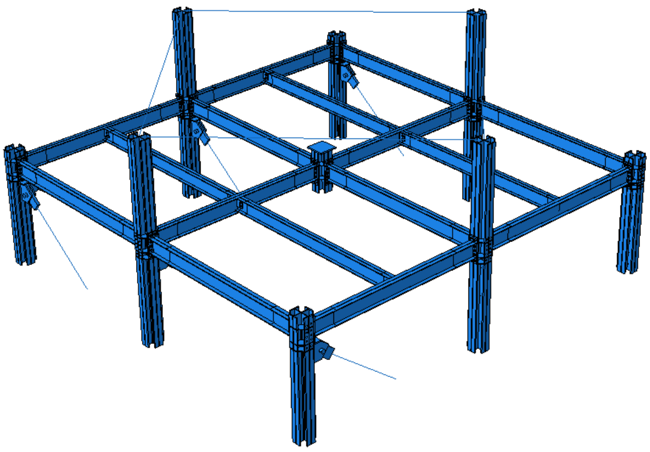


As its clearly visible in the Figure 4.55, the development of plastic hinges in the case of the 3D steel-restrained model is very close to the behavior of the 3D steel-unrestrained model presented in Figure 4.46.
Taking into account that during the experimental testing, it is almost impossible to ensure such a zero rotation of the column (due to the impossibility of constant fixing the column during its vertical movement) and due to the problems of data measurement and quantification of results/reactions during the experimental testing, the final decision was to enable the rotation of the column and continue the modeling with this type of models. Even it is a rotation/translation of the column before the failure mechanism is achieved, it was observed that the significant rotation tendency of the column arise after the failure of the main structural members and also in the real situation, the columns would have similar behavior. However, in the future it is need to turn back after the final results and check the influence of the rotation tendency on the behavior and accuracy of the unrestrained models.

4.7 Catenary effect considering the loss of a column – Full 3D steel model

As stated above, the modeling process started from a smaller model in the beginning (2D model) and proceed to the complex problem (3D model). From the point of view of materials, the modeling process started with only steel structure and proceed to the complex composite ones, considering reinforced concrete slab, reinforcement and headed shear stud connectors. As we reached the stage of the composite model, it was necessary to add several parts and elements to the 3D steel model in order to could be able to model the composite structure. For instance, in order to place the shear connectors in the model it was necessary to model the secondary-beams fully; in order to model the secondary-beams it was necessar to model the connection between the secondar-to-primary beam; to connect the secondary-beams in both ends it was necessary to model the primary-beams on the perimeter as well (Figure 4.56).

Figure 4.56 Full 3D steel model



All the structural members and parts were modeled just how they will be in the sub-assembly for the experimental testing (Figure 4.57).

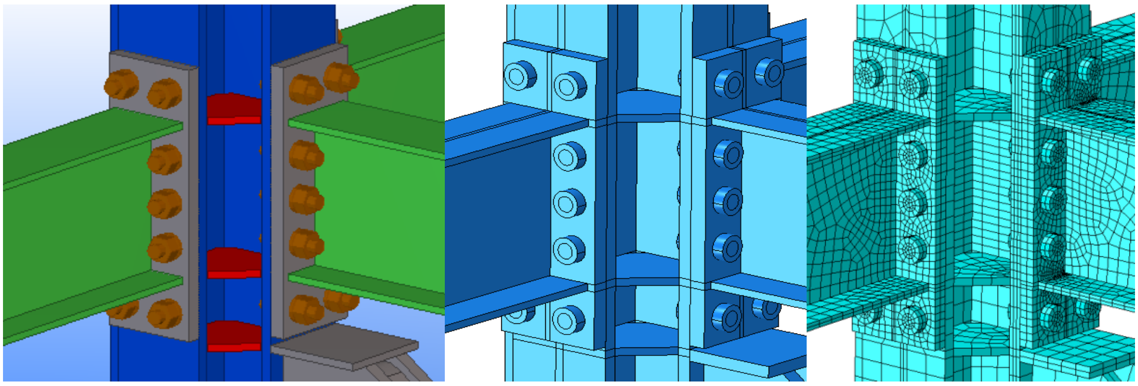


Figure 4.57 Experimental set-up and numerical modeling

The secondary-girder (Figure 4.58) has been defined as 3D deformable solid horizontal element. The method of defining was by extrusion. Solid homogenous section was assigned for the element with S355 material having cross-section IPE 200. The holes for the connection bolts were modeled at this stage by cut extrude method. For an accurate mesh, the beam was partitioned. An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the member. The beam was meshed using structured mesh technique and using hexahedral element shape (Figure 4.58).

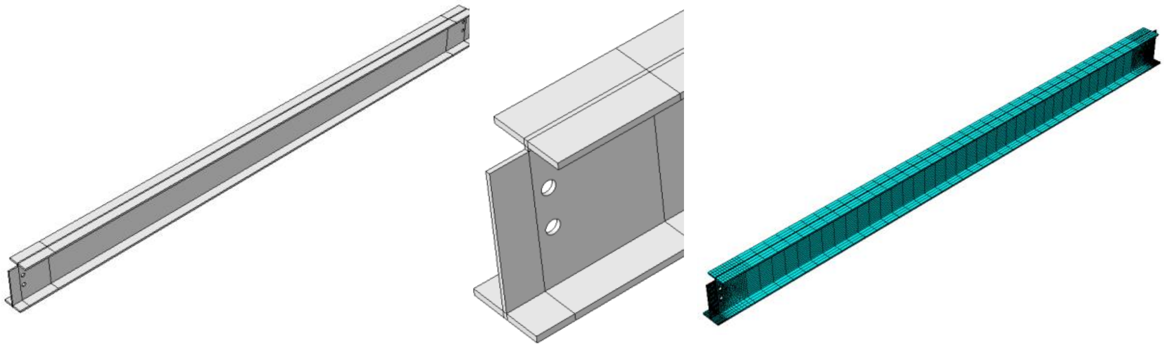


Figure 4.58 Secondary-beam modeling

As it is visible in Figure 4.57, the bolts for the secondary-to-primary beam connection have been defined as 3D deformable solid elements. The method of defining was by revolution (M16). Solid homogenous sections were assigned for the elements with bolt grade 10.9 material. As it can be seen in Figure 4.57, the head and the nut of the bolt were created from the bolt cross-section by revolution. For an accurate mesh, the bolt was partitioned. Due to the fact that the bolt was defined from one element, it was no need to consider further interaction property for the bolt.

An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the bolt. The bolt was meshed using sweep mesh technique (based on advancing front algorithm instead of medial axis) and using hexahedral element shape (Figure 4.57).

In Figure 4.58 it can be observed the deformed shape of the frame and the PEEQ (equivalent plastic strain) (left), Mises (von Mises equivalent stress) (right) distribution within the structure.

Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.59).

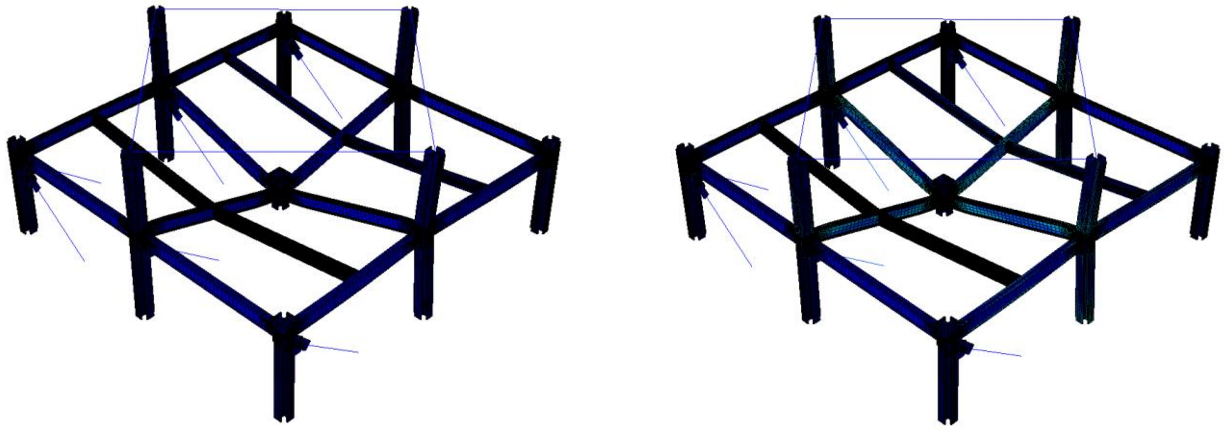


Figure 4.58 Deformed shape of the structure

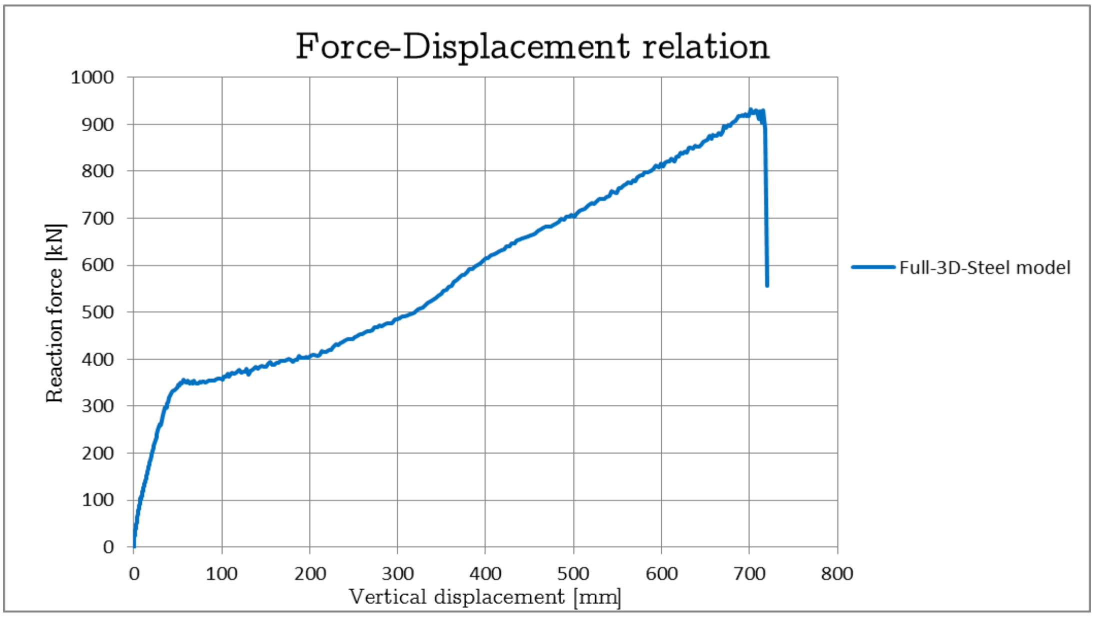


Figure 4.59 Force-Displacement curve for the full 3D-model

The development of plastic hinges of the structure, it was similar to the simple 3D model case. Significant plastification of the beams occured at a displacement of 701 mm and an applied loading of 932 kN, compared to the simple 3D model, when the plastification of the beams occured at a displacement of 692 mm and an applied load of 852 kN.

At this level, the stresses in the beams reached values close to critical values inputed (ultimate capacity of the material) (Figure 4.61). What concerns the global behavior, compared to the simple 3D, the behavior of the structure was similar. The forces started to decrease rapidly after the failure of the beam occured. It is possible to consider than that at a displacement of 701 mm the failure of the structure occured with the failure load of 932 kN (Figure 4.59). Once again, the failure occured at the most affected zone, namely at end of the beam, near to the beam-to-column connection. By modeling the secondary-beams into the structure, it was provided an additional stiffness to the structure and in this manner the load bearing capacity of the system increased from 852 kN to 932 kN. This means an increase of 9.39% in terms of the bearing capacity. See Figure 4.60, where is compared the force-displacement relation of the simple 3D model with the full 3D model. It is clearly visible the influence of the secondary-beams on the behavior of the structure.

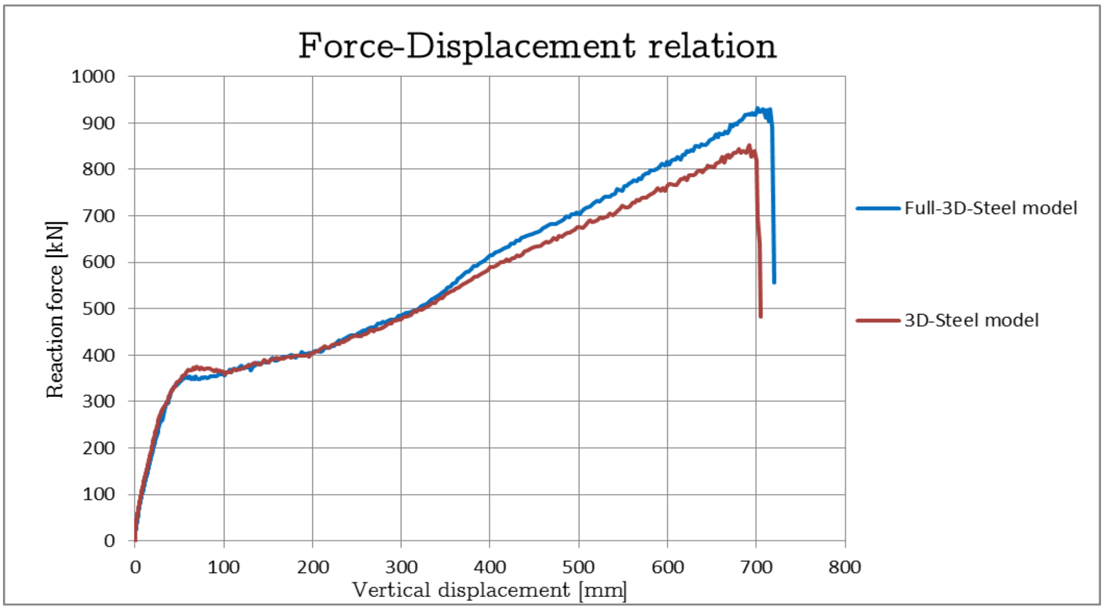


Figure 4.60 Comparison of Force-Displacement relation for the full 3D-model and simple 3D model

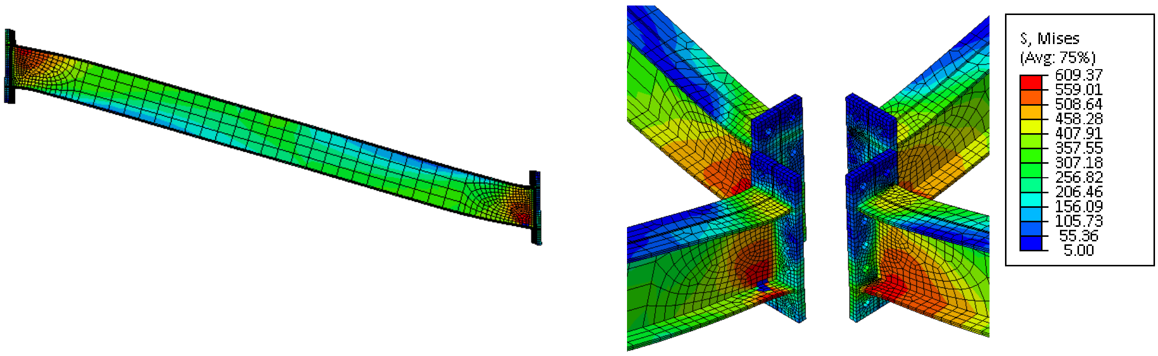


Figure 4.61 Stresses in the beams at a displacement of 701 mm, load of 932 kN

Regarding the rotation tendency of the damaged column during analysis, it was noticed that the same problems occured as in the case of the 2D and simple 3D models. However, significant rotation occured just after the failure of the structure. Thus, the rotation tendency is not influencing the behavior of the model and the results of the simulation in such an increased manner.

4.8 Catenary effect considering the loss of a column – 3D model considering composite action

As it was discussed in Chapter 4.7, in order to be able to model structure considering the composite action of the concrete slab it was necessary to add several parts and elements to the 3D steel model. After defining them (see Chapter 4.7) and with the materials defined in Chapter 4, Section 4.4, the modeling process for the composite structure could began. As in all cases before, several simplifying assumptions had to be made. In addition to the simplifications stated before, in the case of the composite model, the reinforcements for the concrete slab were modeled by wire element type. As from the point of view of structural members and connections modeled or from the point of view of element type chosen for the member modeling and interactions/constraints/boundary conditions applied to these, the modeling proces was performed in a similar manner to the full 3D model case. In this manner, only the differences between the full 3D model and the 3D composite model will be presented in the future. The exterior columns, the interior column, the primary- and secondary-beams, the bracing systems, the truss system, the end-plates and stiffeneres and all the connections presented in the above chapters are valid as well for the composite model too.

In Figure 4.62, it can be seen the initial model for the 3D composite structure in shaded (left) and wireframe (right) view.

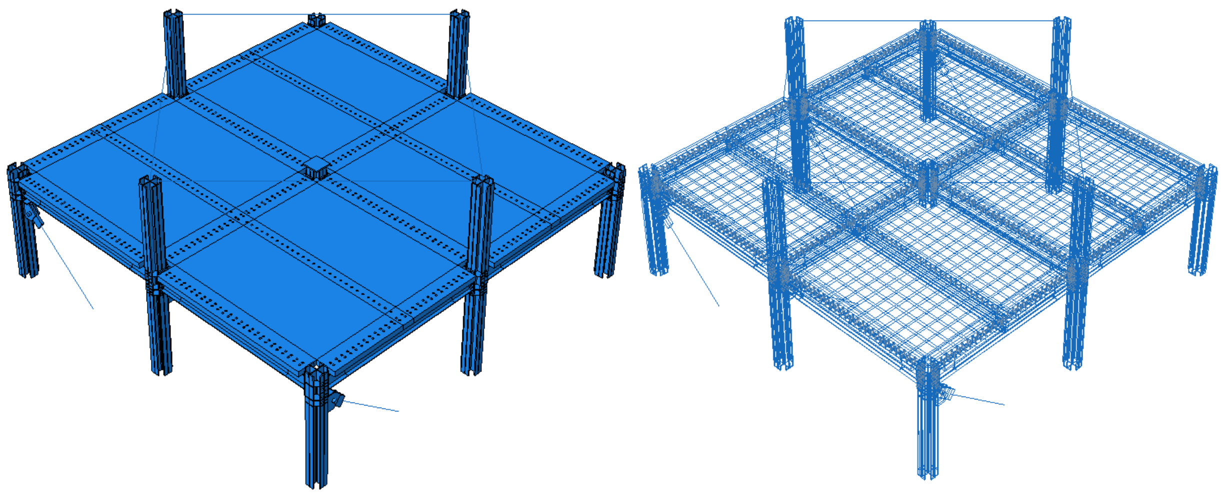


Figure 4.62 Shaded and wireframe view of the 3D composite model

Two directional reinforcing of the concrete slab with reinforcement Φ6/200 mm in x and y direction, top and bottom, was designed for this model. The length of the reinforcing bars varys; according to the location where they are placed, there are two type of reinforcements (type 1 and type 2).

Both type of the reinforcements were defined as 3D deformable planar wire elements. Beam type section was assigned for the elements, with S500 material. Beam section integrated during analysis method was selected. For the beam shape, circular cross-section (Figure 4.63) was selected from the standard beam section library. Geometric input data was *r* (radius).

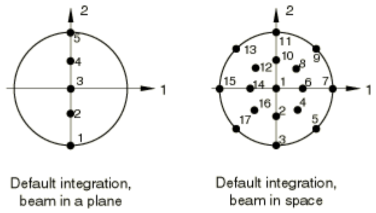


Figure 4.63 Circular cross-section from standard beam section library (Abaqus user manual)

Approximate local 1-direction of the cross-section was assigned as orientation to the beam section. A 2-node linear beam in space (B31) element type was selected for the members from standard element library. The family associated to the element was the Beam one. In Figure 4.64, the spacing of the reinforcements within the structure is presented.

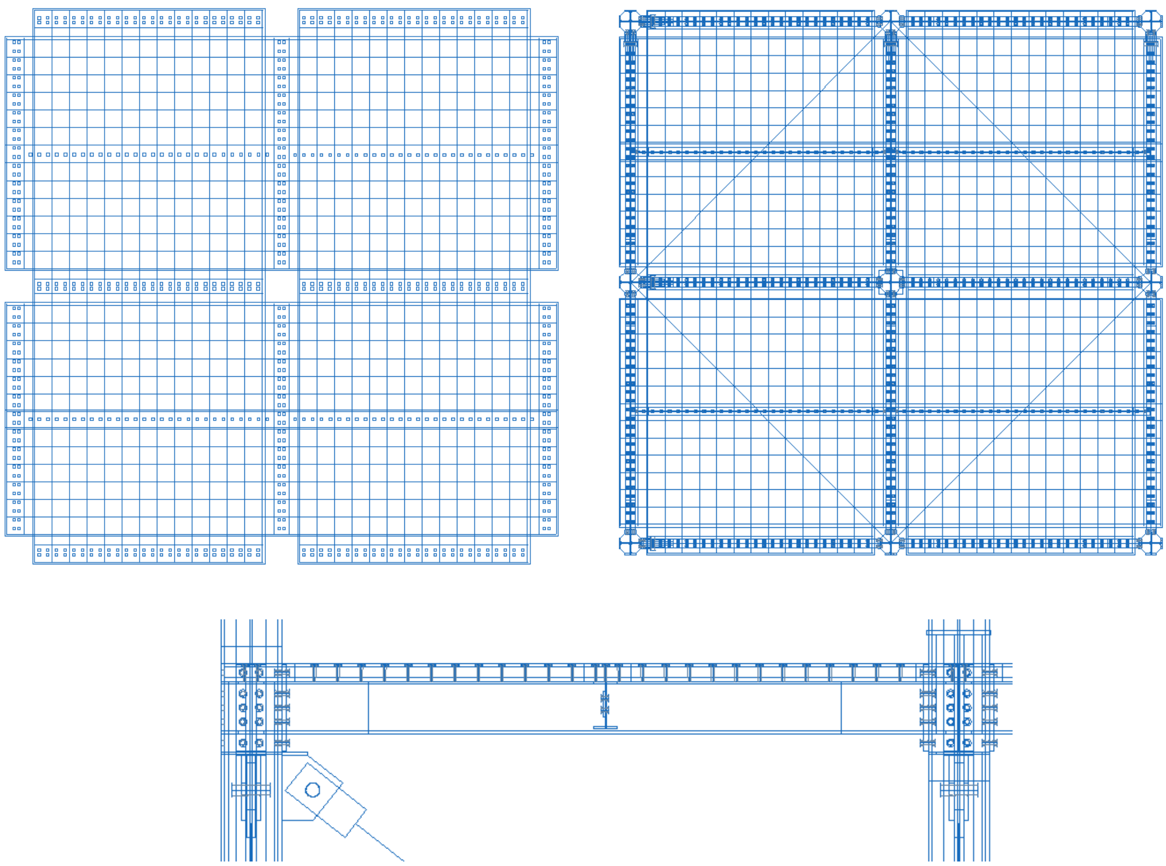


Figure 4.64 Spacing of the reinforcement within the concrete slab in Abaqus

In the case of the reinforcement, for defining the interaction/contact with the concrete slab, the technique of embedded element was used. This technique can be used in geometrically linear and nonlinear analysis to model a set of rebar reinforcement, shell, or surface elements that lie embedded in a set of three-dimensional solid (continuum) elements and will not constrain rotational degrees of freedom of the embedded nodes when shell or beam elements are embedded in solid elements. A beam-in-solid embedded definition was used, defining the reinforcement bars being the embedded elements and the concrete slab the host region. By this method it was not necessary to define separately for each reinforcement bar the contact with the concrete slab and it was not necessary to model the holes for the re-bars in the slab.

In the case of the sub-assembly, according to DIN EN ISO 13918 shear connectors were selected having diameter of 16 mm (d_1=16 mm) and height of 75 mm (I_2=75 mm) (Figure 4.66). The spacing of the shear connectors (Figure 4.67) is the following: 1 X Φ16/200 mm on the secondary-beams, 2 X Φ16/200 mm on the primary-beams. In the case of the primary-beams the distance between the two row of connectors is 54 mm. From initial analysis, it was necessary to provide such an increased connection at the level of the primary beams too, because near to the primary-beam to column connections the so called phenomena of "zipper" type of failure of the concrete slab occured during the missing column scenario. This means that the concrete slab, during the loading process, started to "disconnect" from the primary beam by the failure of the shear connectors each after the other (Figure 4.65). In the following figure, the failure is presented according to ELS (Extreme Loading for Structures software model) by *(Marginean et al, 2013):*

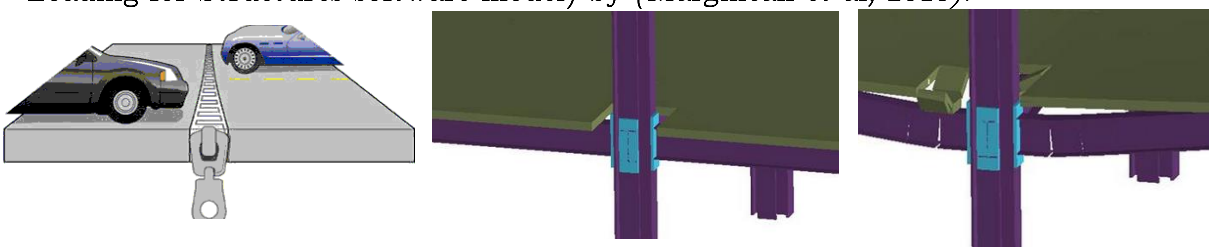


Figure 4.65 Zipper failure of shear connectors in concrete slab (Marginean et al, 2013)

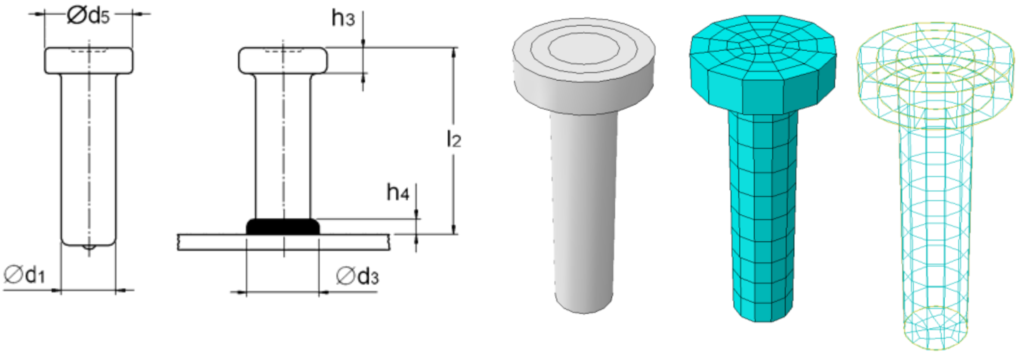


Figure 4.66 Shear connector modeled in Abaqus

The shear connectors have been defined as 3D deformable solid element. The method of defining was: revolution. Solid homogenous section was assigned for the element with S355 material. It can be seen in Figure 4.66, the head of the stud was created from the connector cross-section by revolution. The stud was partitioned.

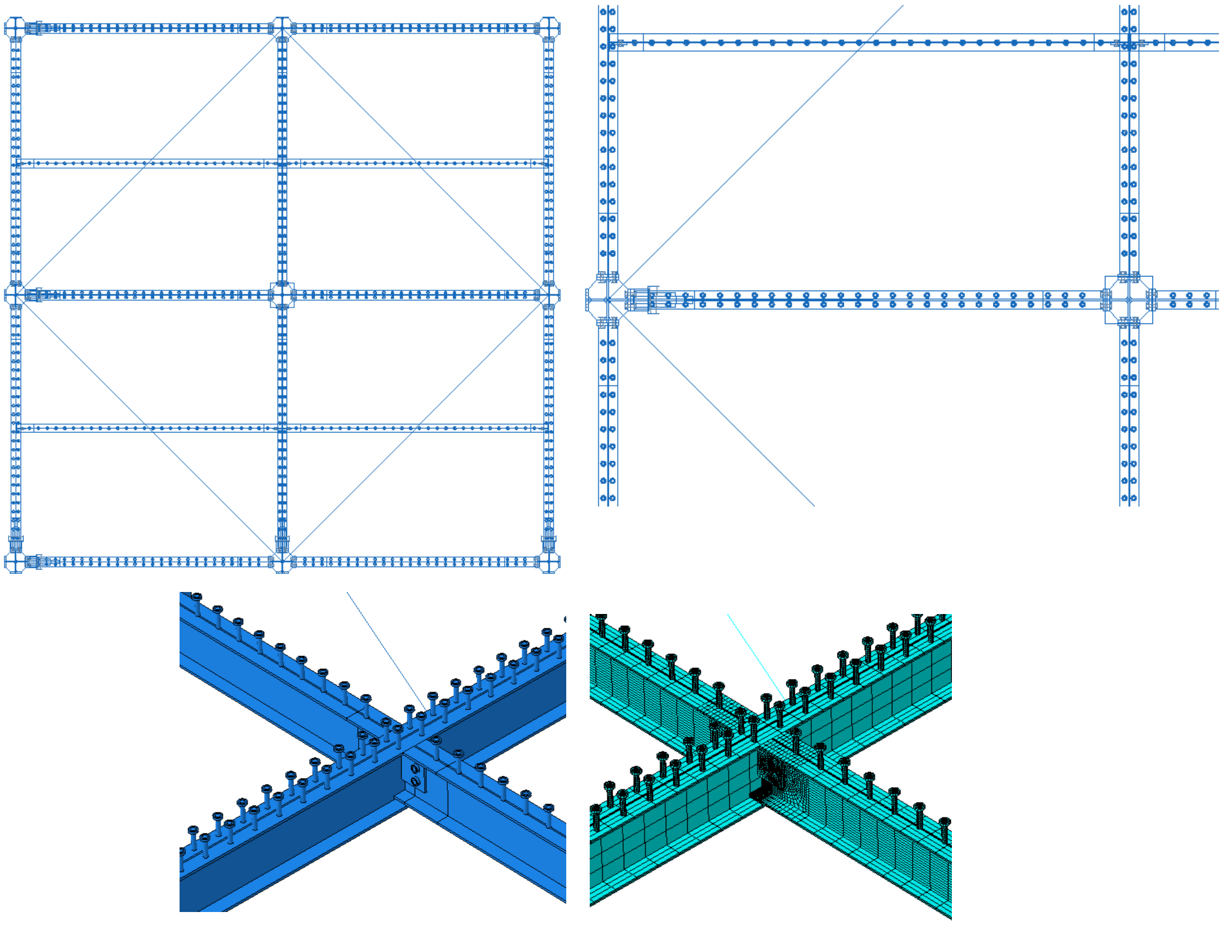


Figure 4.67 Spacing of shear connectors in the model

An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the stud. The stud was meshed using sweep mesh technique (based on advancing front algorithm instead of medial axis) and using hexahedral element shape (Figure 4.66). As in the case of the reinforcement, for defining the interaction/contact with the concrete slab, the technique of embedded element was used. A solid-in-solid embedded definition was used, defining the shear studs being the embedded elements and the concrete slab the host region. By this method it was not necessary to define separately for each shear connector the contact with the concrete slab and it was not necessary to model the holes for the studs in the slab.

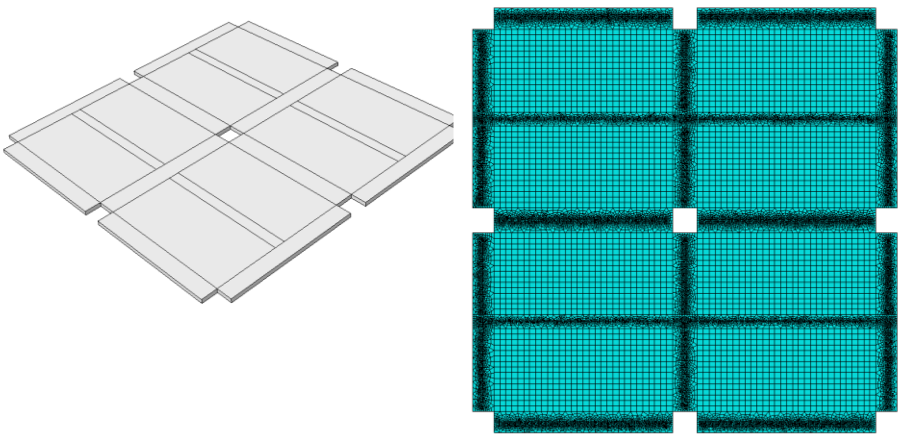


Figure 4.68 Modeling of the concrete slab in Abaqus

The concrete slab (Figure 4.68) has been defined as 3D deformable solid horizontal element. The method of defining was by extrusion. Solid homogenous section was assigned for the element with C25/30 material having thickness of 80 mm. The concrete slab will not intersect with the steel columns (neither the corner and perimetral, nor the central column) in order to be able to quantify (measure) the reactions and deformations at the level of joints, during the experimental testing.

As it can be seen in Figure 4.68, the eight perimetral+corner holes for the columns and the hole for the central column were also modeled at this stage by cut extrude method from the section of the slab. For an accurate mesh, the slab was partitioned. An 8-node linear brick, reduced integration, hourglass control (C3D8R) element type was selected for the member. The concrete slab was meshed using both structured (for the eight middle/central concrete panels) and sweep (for the areas of the shear connectors) mesh technique (based on advancing front algorithm) and using hexahedral element shape in both cases (Figure 4.68). As it can be seen, the mesh at the areas with the shear connector location was refined in order to the simulation accuray. However, this refinement produced enourmos time demand for the calculation, producing more than 88000 finite elements just for the concrete slab part. It was need to increase the mass scale, but after having some computational errors during the running of the analysis, it was necessary to decrease from 5E-006 to 4E-006, thus, transforming the hours needed for the calculation time into several days.

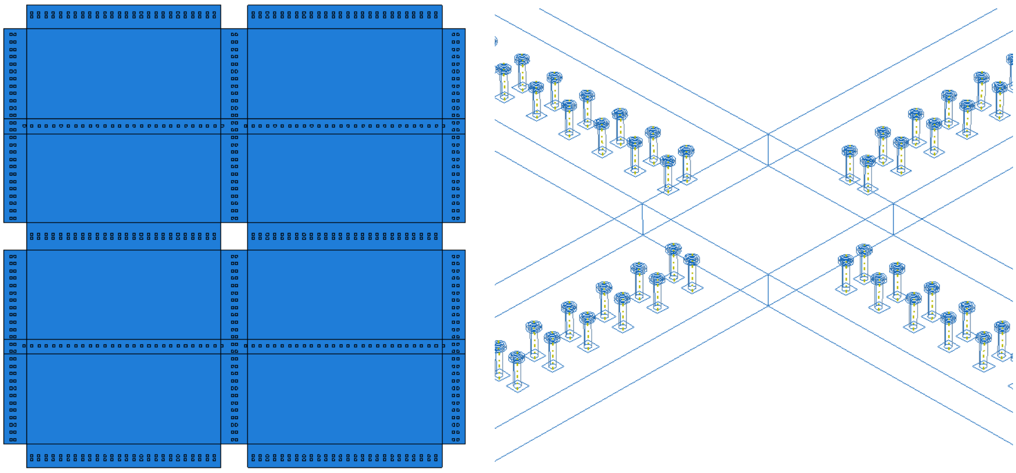


Figure 4.69 Divison of the concrete slab inferior surface into smaller surfaces

After modeling of the concrete slab, reinforcement and shear connectors, another problem occured. It is not possible in Abaqus to define embedded interaction and tie interaction at the same time for the same element. In this manner it was not possible to simulate the welding of shear connectors to the flange of the beam. The solution to model the welding of the studs to the beam was the following: on the concrete slab inferior surface, several rectangular surfaces (16 mm X 16 mm) where defined, which will simulate the contact between the shear studs welded to the beam flange. These surfaces were created at the position of the shear studs on the beam flange. After defining the surfaces on the concrete slab, these were tied to the beam flange.

In this manner the shear connectors could be embedded in the concrete slab and by the sufraces tied to the beam flange, the interaction between the connectors and the beam could be simulate simultaneously/at once.

For the loading of the structure the method of displacement was used, just like in the cases presented before. A displacement of 650mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column.

Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.71). In Figure 4.70 it can be seen the deformed shape of the frame and the PEEQ (equivalent plastic strain) distribution within the structure.

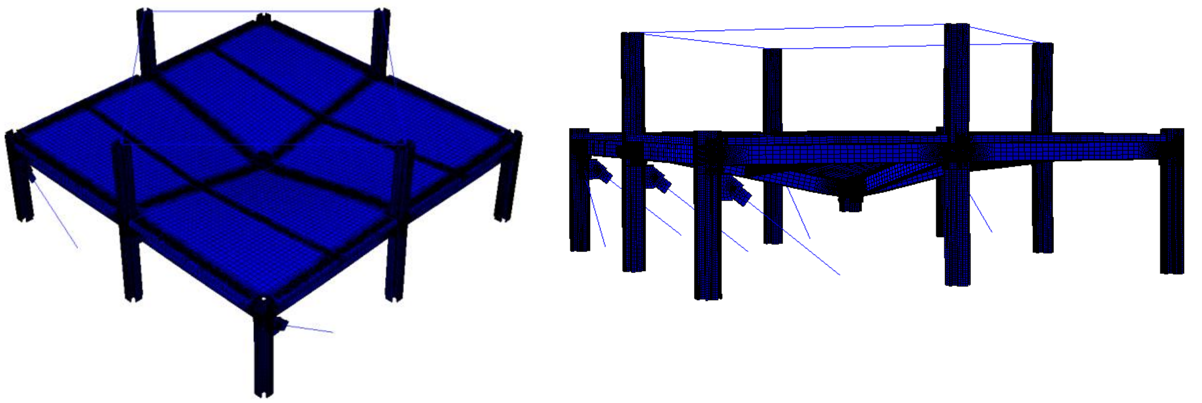


Figure 4.70 Deformed shape of the structure

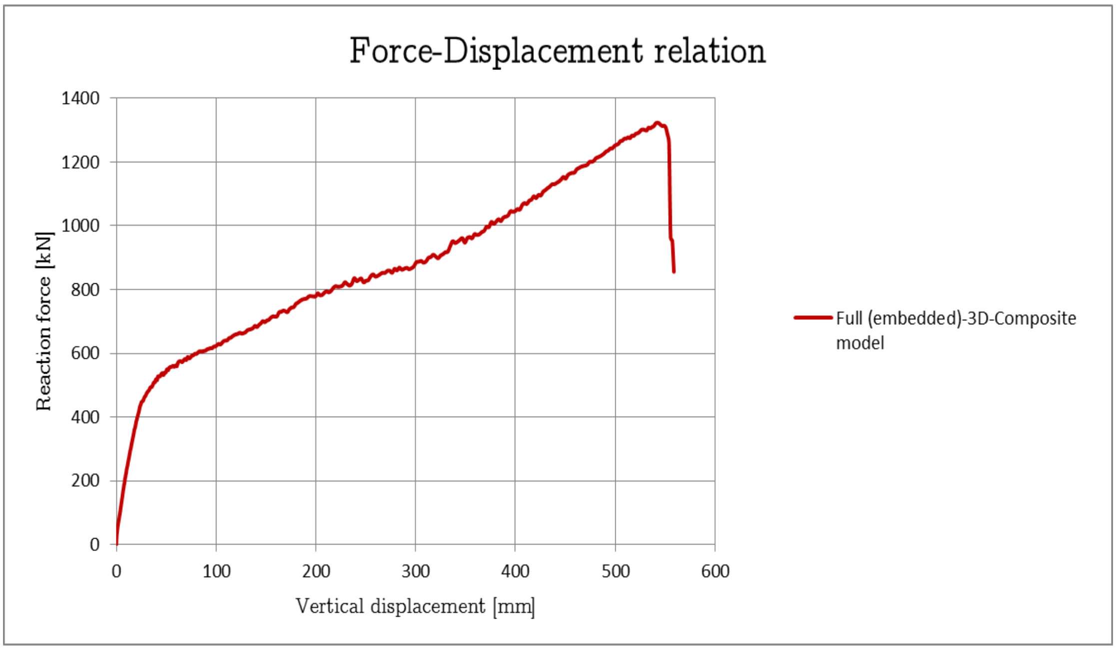


Figure 4.71 Force-Displacement curve for the 3D-model considering the composite action

Significant plastification of the beams occured at a displacement of 542 mm already and an applied loading of 1324 kN, compared to the full 3D steel model, when the plastification of the beams occured at a displacement of 701 mm and an applied load of 932 kN.

At this level, the stresses in the beams reached values close to critical values inputed (ultimate capacity of the material) (Figure 4.72). What concerns the global behavior, compared to the full 3D steel model, the behavior of the structure was similar. The forces started to decrease rapidly after the failure of the beam occured. It is possible to consider than that at a displacement of 542 mm the failure of the structure occured with the failure load of 1324 kN (Figure 4.71). Once again, the failure occured at the most affected zone, namely at end of the beam, near to the beam-to-column connection. By modeling the reinforced concrete slab and considering the composite action of the shear connectors with the beams in the structure, it was provided an additional stiffness to the structure and in this manner the load bearing capacity of the system increased from 932 kN to 1324 kN. This means an increase of 42.06% in terms of the bearing capacity. See Figure 4.73, where is compared the force-displacement relation of the full 3D steel model with the 3D composite model. It is clearly visible the influence of the composite action on the behavior of the structure.

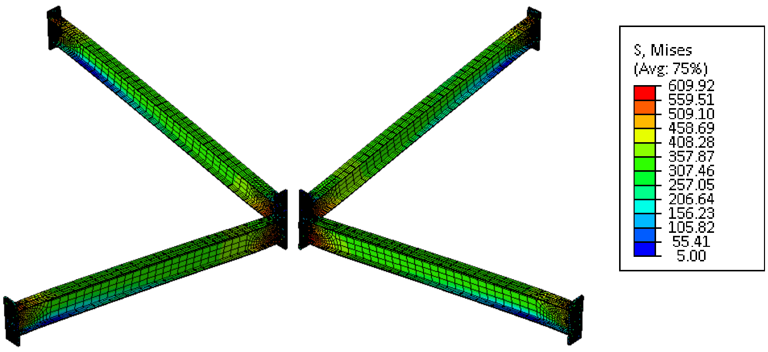


Figure 4.72 Stresses in the beams at a displacement of 542 mm, load of 1324

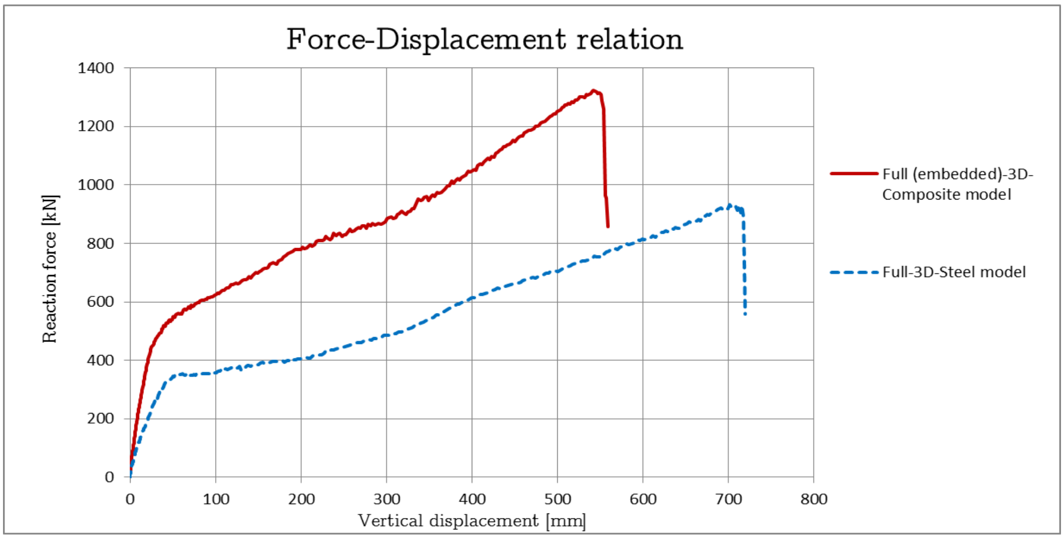


Figure 4.73 Comparison of Force-Displacement curve for full 3D steel model and 3D composite model

In order to better understand the behavior of the structure and the behavior of the composite action of the reinforced concrete slab, two more structures have been modeled. First scenario is considering one row of shear studs on the primary-beams (instead of the initial two row configuration) and the second scenario is considering just the secondary-beams acting compositely (i.e. no shear studs on the primary-beams). In Figure 4.74 it can be seen the spacing of the shear connectors for the two scenarios.

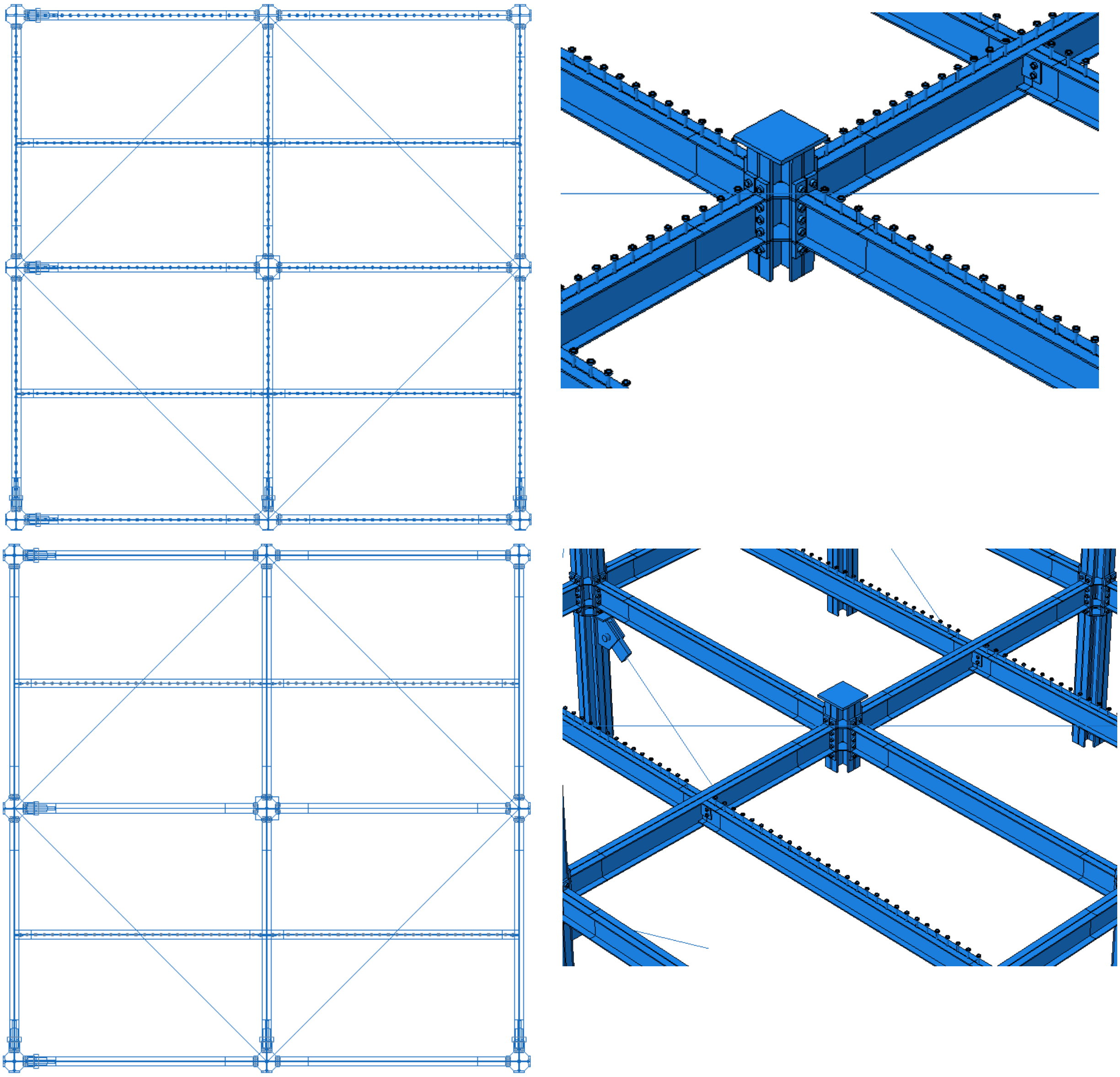


Figure 4.74 Spacing of the shear connectors for the two scenarios

The difference between these two scenarios and the previous 3D composite model is only within the terms of shear connectors. All the other members were defined in similar manner and for the loading of the structure, the method of displacement was used, just like in all the cases presented before. A displacement of 650mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column.

Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.75). As it was expected, the the behavior of the structure it was different from the case of the initial 3D composite model. In the case of scenario 1 3D composite model (both primary- and secondary-beams acting compositely; 1 row of studs on the primary-beam) the behavior of the model was very close to the behavior of the full 3D composite model. In the case of scenario 2 3D composite model (only secondary-beams acting compositely) the behavior of the structure was more likely in the case of the full 3D steel model or better say, it was between the full 3D steel model and the full 3D composite model. The results are better understable by the meaning of the chart below:

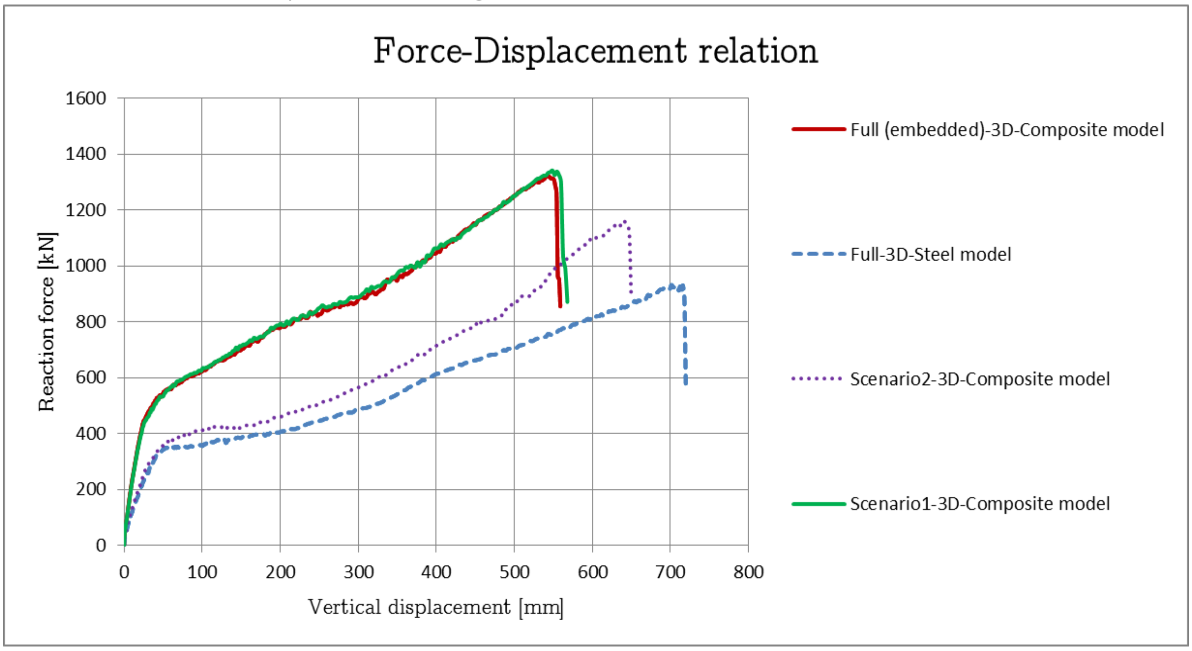


Figure 4.75 Comparison of Force-Displacement curve for full 3D steel model, full 3D composite model, Scenario 1 3D composite model and Scenario 2 3D composite model

However, the scenario 1 3D composite model seem to behave better than the full 3D composite model, an important issue need to be considered, namely the modeling of the interaction between the concrete slab and the shear connectors. Beside the embedded technique, as mentioned above, the interaction between the slab and the shear studs it is possible to model also by defining the holes for the connectors in the concrete slab and than connecting by tie constraint the shear connectors to the beam flange. In this manner the interaction between the shear studs and concrete slab will be defined by the general contact algorithm. It should be mentioned that the degree of reliability might be reduced in connecting indirectly the shear studs to the beam flange (by the meaning of the surfaces defined on the inferior surface of the slab and tied to the beam flange), due to the issue of modelling the complex behavior of the composite action. This method might not simulate the exact behavior of the connection, but reduced significantly the analysis time due to the reduced finite elements created on the concrete slab. In order to ensure that the obtained results and behavior are adequate, in the following section a full 3D composite model will be analysed, where the shear studs were connected directly to the beam flange by tying and instead of the embedded technique, the actual holes were defined for the shear connectors.

The only difference between the following model and the full 3D composite model presented above is the definition of the interaction between the shear studs and concrete slab/beam flange. In the Figure 4.76, the model of the new(not embedded) full 3D composite model is presented. As it can be seen, the initial shear connector distribution was used also for this case (2 rows of connectors on the primary-beams and 1 row of connectors on the secondary-beams).

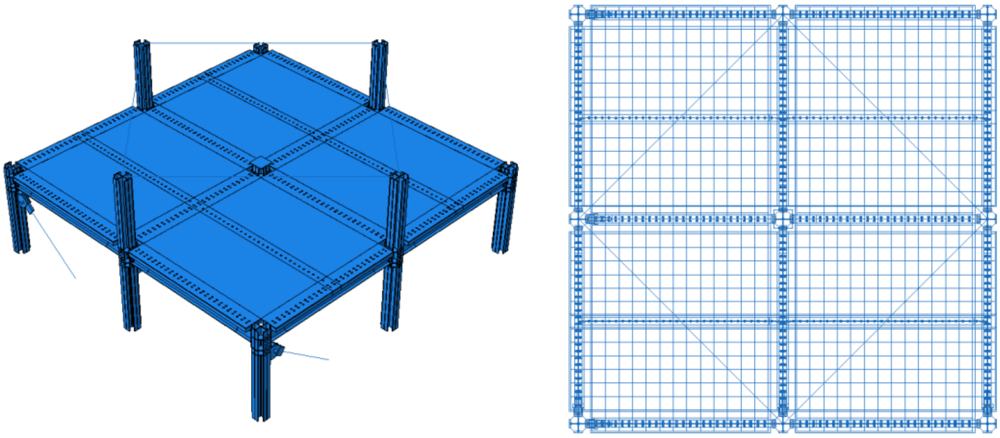


Figure 4.76 Model of the composite structure

The same element type were used for the definition of the reinforcement, shear connectors and concrete slab. In the concrete slab holes were defined by cut extrude technique at the location of each shear connector. The shape and the size of the hole was equal to the dimentions of the shear stud. For meshing the slab two techniques were used: structured mesh for the eight central slab panels and sweep mesh (using advancing front algorithm) for the parts where the holes have been defined (Figure 4.78). Before meshing the concrete slab it was necessary to partition the thickness of the slab into several parts, according to the shape of the hole (shear connector) (Figure 4.77).

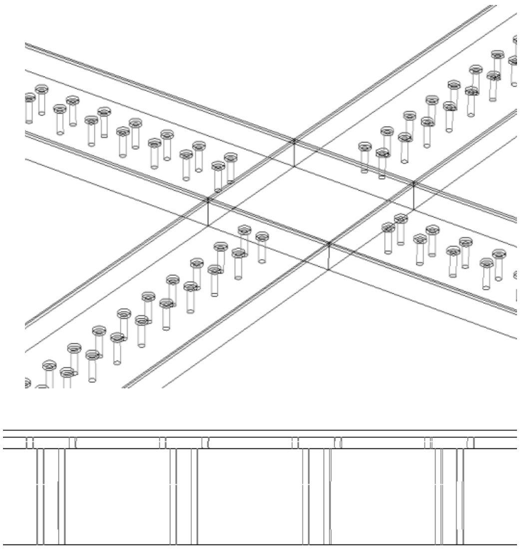


Figure 4.77 Definition of the holes within the section of the concrete slab

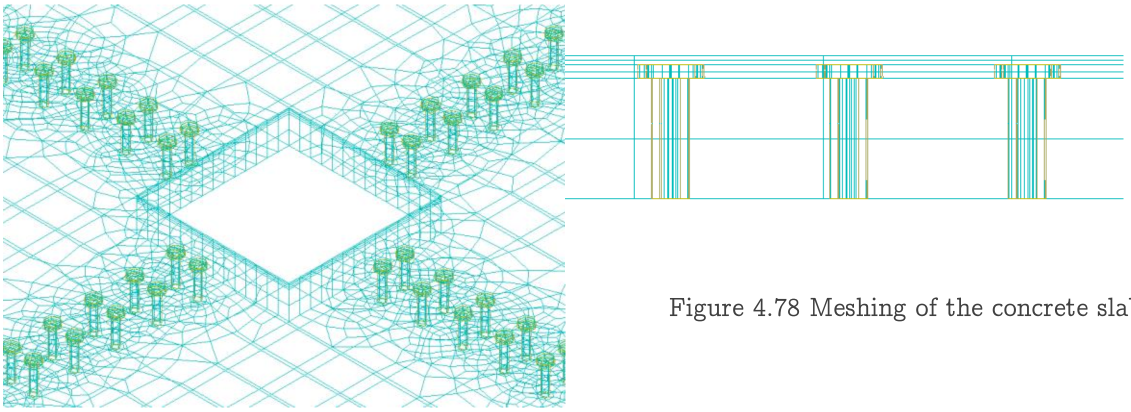


Figure 4.78 Meshing of the concrete slab

The shear studs were connected to the beam flange by the tie constraint type, using surface-to-surface discretization method (Figure 4.79). "Adjust slave surface initial position" option was disabled (do not move all the nodes of the slave surface onto the master surface in the initial configuration). "Tie rotational DOFs" option was enabled (Abaqus will constrain the rotational degrees of freedom that exist on both master and slave surfaces). In this manner the shear connectors were directly connected/tied to the surface of the beam without the use of additional surfaces which may be an inaccurate method of modeling for this situation. This method of modeling was a bit slower than the previous one and created an increased number of finite elements on the concrete slab, namely 153000 compared to the previous case when it was around 88000 finite elements on the slab. To increase the mass scale it was not possible due to errors occuring during the analysis. So with the previously defined mass scale of 4E-006 the analysis took several days.

A displacement of 650mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column.

Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 4.79).

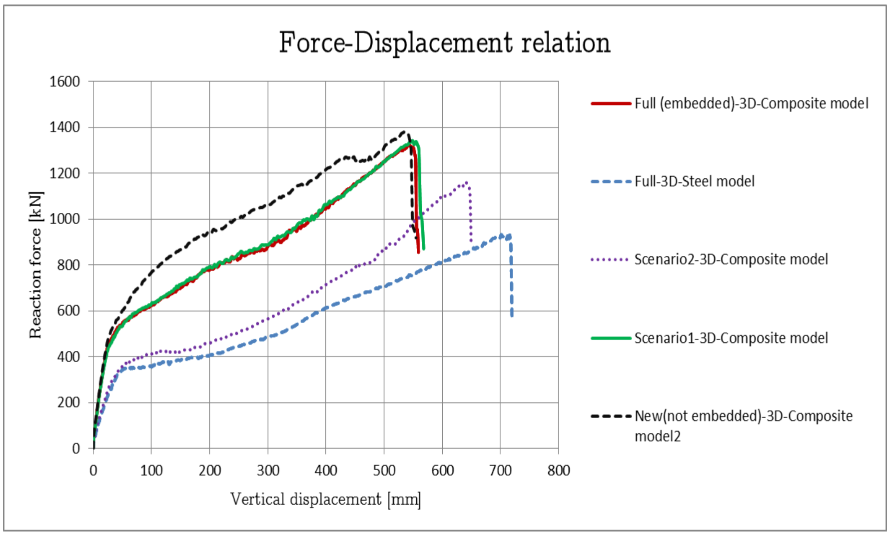


Figure 4.79 Comparison of Force-Displacement curve for full 3D steel model, full 3D composite model, Scenario 1 3D composite model, Scenario 2 3D composite model and the new(not embedded) 3D composite model

In Figure 4.80 it can be observed the deformed shape of the frame and the PEEQ (equivalent plastic strain) distribution within the structure.

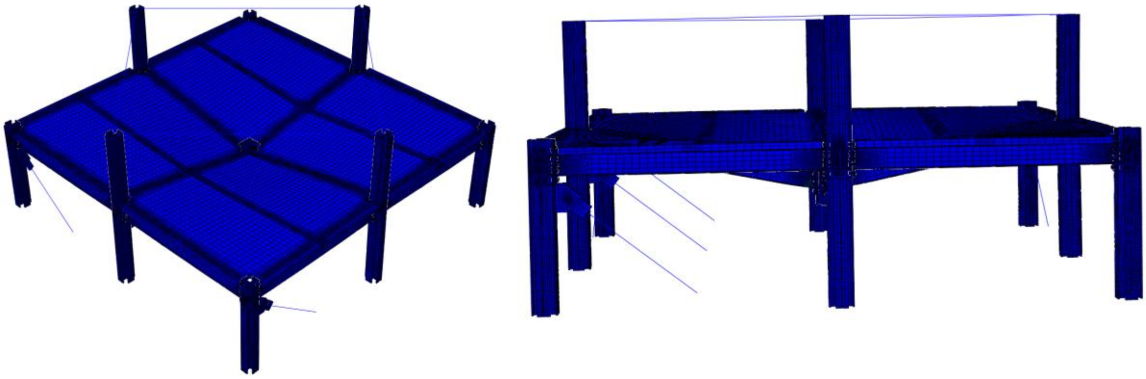


Figure 4.80 Deformed shape of the structure

Significant plastification of the beams occured at a displacement of 521 mm and an applied loading of 1369 kN, compared to the full 3D composite model, when the plastification of the beams occured at a displacement of 542 mm and an applied load of 1324 kN. At this level, the stresses in the beams reached values close to critical values inputed (ultimate capacity of the material). What concerns the global behavior, compared to the full 3D composite model, the behavior of the structure was similar. The forces started to decrease rapidly after the failure of the beam occured. It is possible to consider than that at a displacement of 521 mm the failure of the structure occured with the failure load of 1369 kN (Figure 4.79). Once again, the failure occured at the most affected zone, namely at end of the beam, near to the beam-to-column connection.

In both scenarios presented above, the failure load occured at the same moment aproximately, however the behavior is significantly different as it can be seen in Figure 4.79. By modeling the reinforced concrete slab and considering the composite action of the shear connectors directly with the beams in the structure, it was provided an additional stiffness to the structure and in this manner the load bearing capacity of the system increased from 1324 kN to 1369 kN. This means an increase of 4.0% in terms of the bearing capacity. See Figure 4.79, where is compared the force-displacement relation of the full 3D composite model with the new (not embedded) 3D composite model. It is clearly visible the influence of the direct interaction of the studs on the beam flange what concerns the behavior of the structure. Even the difference was not so high, for future modeling it is better to use the technique presented above instead of the embedment of the shear connectors in the concrete slab.

What concerns the future studies, in the Chapter 5, a parametrical study will be carried out on the behavior of the composite structure related to the number of shear connectors needed, the reinforcement ratio and the results will be presented.

4.9 Contribution of flexural action and catenary action into the total/global capacity of the structure

From the plastic theory, it can be seen that, when plasticity starts to develop, the equation below can be obtained:

$$\frac{M}{M_y} + \left(\frac{N}{N_y}\right)^2 = 1$$

From the above equation can be seen that, when the deflection is increased, gravity loads are mainly resisted by the vertical components of axial catenary forces that develop in the beams. It is apparent from equation that, when N approaching N_y, thus M will approximate to 0. This means that the beam bending stiffness will be greatly softened by the catenary axial force N. Consequently, when the catenary force is extremely large, the bending moment will almost disappear.

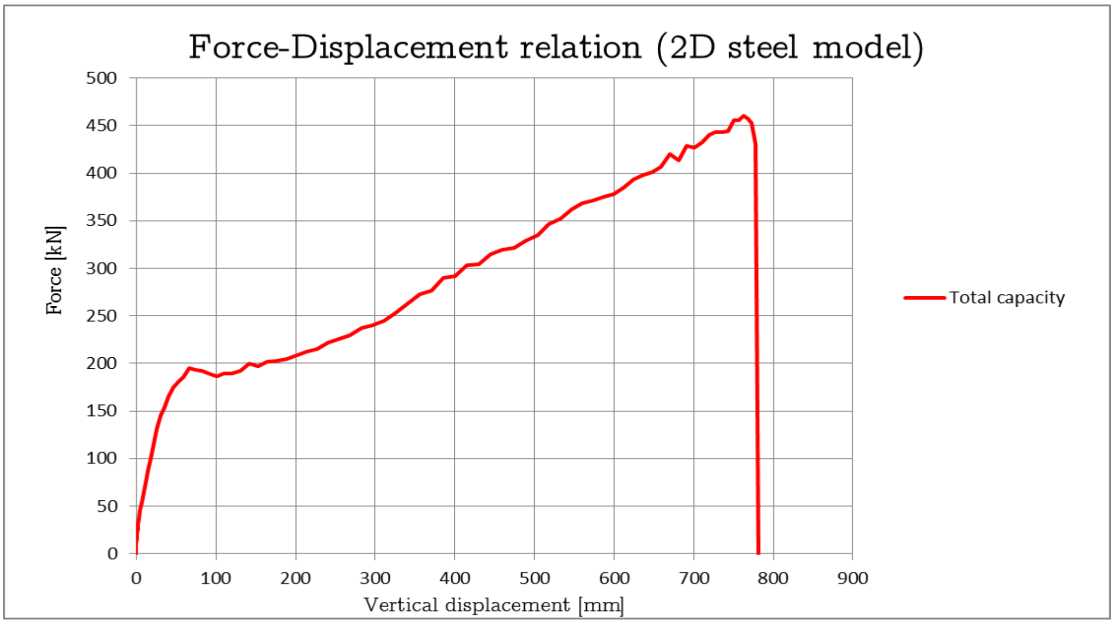


Figure 4.81 Force-Displacement relation for the total applied loading (2D steel model)

As it can bee seen in Figure 4.81, the force-displacement curve for the 2D steel model was plotted. In the following figures, the contribution of the flexural and catenary actions will be discussed separately, with respect to the total capacity of the structure. In Figure 4.82, the flexural or bending action is presented. The applied loading is resisted by the flexural action until a displacement of 150 mm after which the contribution of the moment starts to decrease significantly while the deformation becomes larger. In Figure 4.83 it can be seen that, when the deflection is increased enough, gravity loads are mainly resisted by the vertical components of axial catenary forces that develop in the beams. Before the large deformations stage, the catenary forces can be considered to be very small. When the catenary forces are extremely large, the bending moment will almost disappear. Figure 4.81, 4.82 and

4.83 is a clear proof for the above mentioned equation taken from the plastic theory and is valid after the plasticity starts to develop.

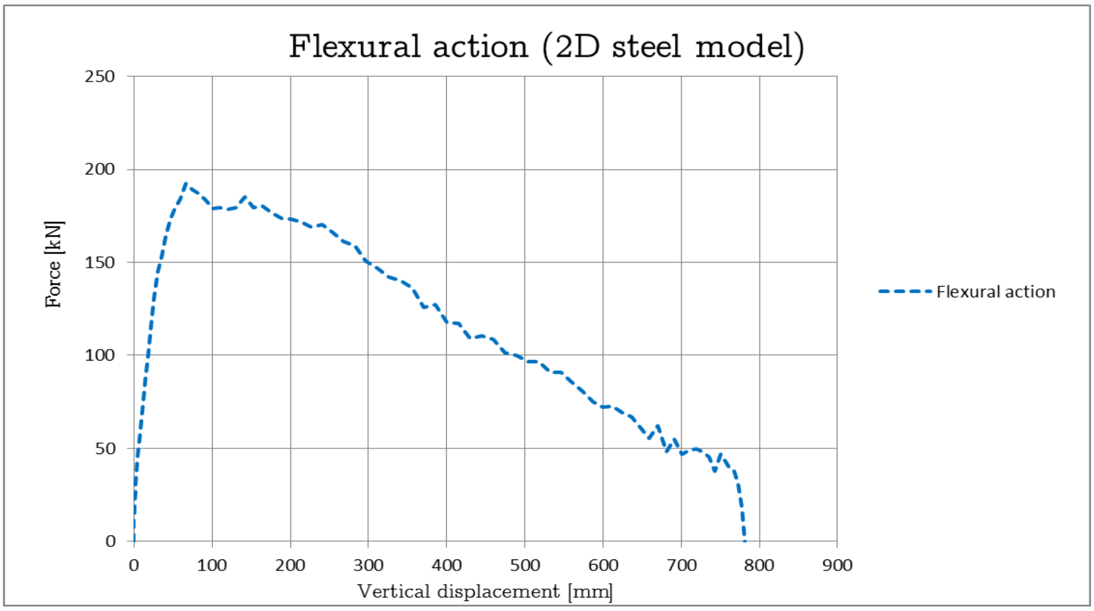


Figure 4.82 Contribution of the flexural action for the total applied loading (2D steel model)

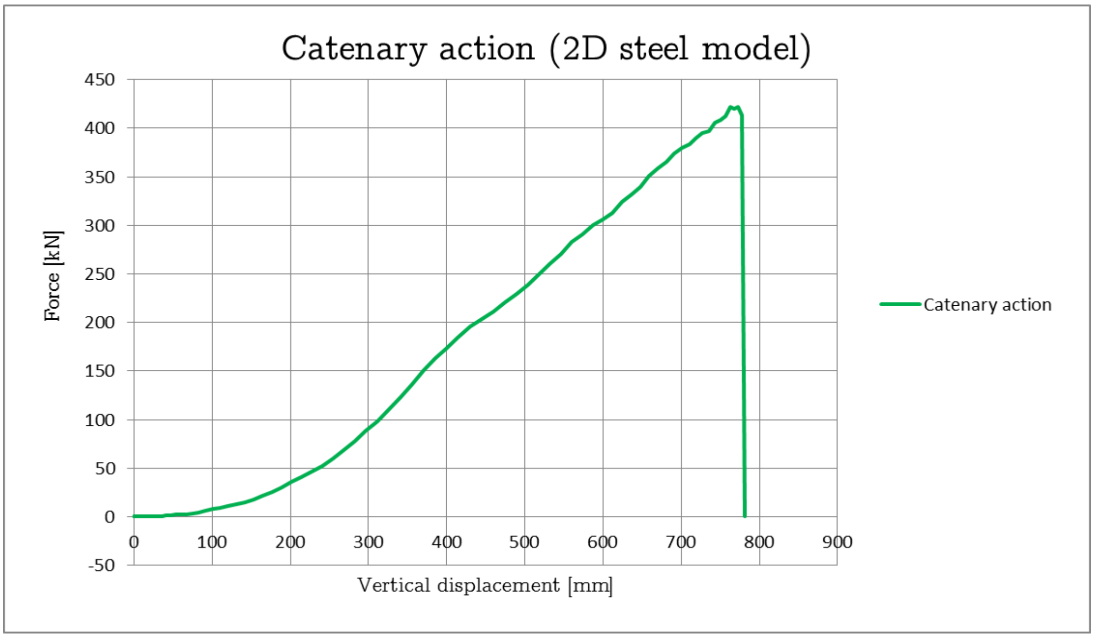


Figure 4.83 Contribution of the catenary action for the total applied loading (2D steel model)

If the beams are not sufficiently strong to resist the loading demand resulting from the instantaneous removal of the column in an elastic manner, plastic hinges will form at the two ends of the beams (see the study carried out in the previous chapters within this thesis). If the flexural strength of the beams is insufficient to accomplish stability, the beams will deflect mobilizing catenary tensile action that, if sufficient, may arrest the failure of the system. In order for the catenary action to be fully activated in a beam member, beside the phenomena of large deflections, the ends of the beams need to be strongly anchored to the joints, enough to resist the large axial force generated by the catenary action. These, have the meaning that in real structures, the contribution of catenary action on resisting progressive collapse will depend highly on the connection conditions.

Nevertheless, it should be noted that catenary action is effective in preventing progressive collapse, if and only if lateral restraint from adjacent boundary elements is adequate. When a mid-span column is removed, adjacent slabs may form a very strong in-plane diaphragm effect, which is able to sustain the catenary tension forces. When a penultimate external or internal column is removed, the catenary tension forces may pull inwards the perimeter columns, leading to a progressive collapse. In such a situation, catenary action may cause, rather than prevent the progressive collapse; see (Tan Kang Hai, 2012).

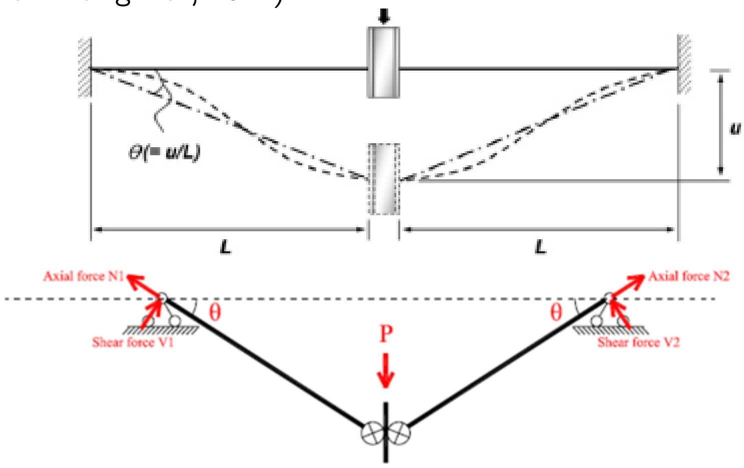


Figure 4.84 Representation of the catenary action mechanism (Lee et al, 2009)

If we approximate that the deformation levels are small enough such that the sine or tangent of an angle equals the angle itself in radians, than the rotation angle Θ (in radian) is obtained by dividing the vertical deflection with the beam length, L.

In the case of a deformed element, as shown in Figure 4.84, the equilibrium of forces gives the following equation: $P=(N_1+N_2)\sin\Theta+(V_1+V_2)\cos\Theta$, where P is the total applied load; the load component $(N_1+N_2)\sin\Theta$ is resisted by catenary action and the load component $(V_1+V_2)\cos\Theta$ is resisted by flexural action.

The total load will be resisted by both catenary and flexural action. For a better understand, see the Figure 4.84 above, however in the above figures, the case valid for our situation is with fixed end constraints only.

As mentioned above, the load in the initial phase is resisted by the flexural action, while at the large deformation and rotation stage, the load is substantially resisted by catenary action. The contribution of the catenary action and the flexural action in the resistance of the total applied load is clearly visible in the Figure 4.85, where the force-diplacement curve was plotted for the 2D steel model mentioned in the Chapter 4.5, together with the catenary and flexural action.

In the Figure 4.85 it can be observed that, in the initial phase the total applied load is resisted only by the flexural or bending action.

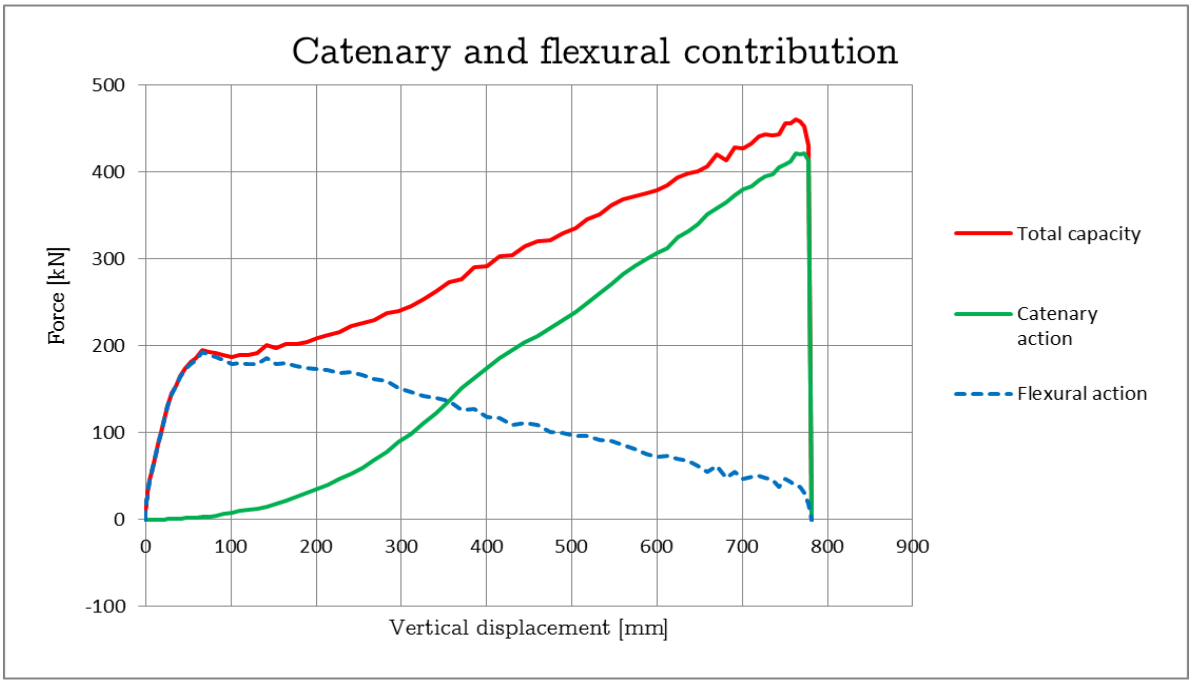


Figure 4.85 Catenary and flexural action for the 2D steel model

At this phase, the axial forces are close or equal to 0. If the connections are rigid enough and after large deformation stage, the flexural action contribution starts to decrease, while the contribution of the catenary action is activated. This drop of the flexural action is occuring after the rotation angle increases large enough and the deflection is large enough as well, in order to activate the catenary forces in the beams. It can be seen in the Figure 4.85 that the catenary action is activated just after a displacement of 150 mm and becomes the governing action just after a vertical displacement of 350 mm. Once the catenary forces are activated, the applied loading is resisted only by catenary action until the failure occures. When the catenary forces are extremely large, the bending moment will almost disappear. In Figure 4.85, the total load (red line) is the variable P in the equation above, and the catenary (green line) and flexural action (blue line) is the variable $(N_1+N_2)\sin\Theta$ and $(V_1+V_2)\cos\Theta$ respectively.

In the following, the contribution of the catenary action and flexural action to the behavior of the full 3D steel model will pe presented. As it can be seen in Figure 4.86, in the case of the full 3D steel model the contribution of the catenary and flexural action is similar to the 2D steel model case. The total applied load is resisted by both catenary and flexural action. In the initial loading phase, the behavior of the structure can be described as flexural behavior until a vertical displacement of 150 mm. At this phase, the axial force in the beams is relatively small or it can be considered equal to 0. Due to the fact that the connections in our structure were designed to be fully rigid and after reaching significant deformations level, the contribution of the flexural action starts to have less effect in resisting the applied loading. At a displacement of 150 mm the catenary forces are activated in the system and after a displacement of 350 mm, the loading is resisted only by catenary forces.

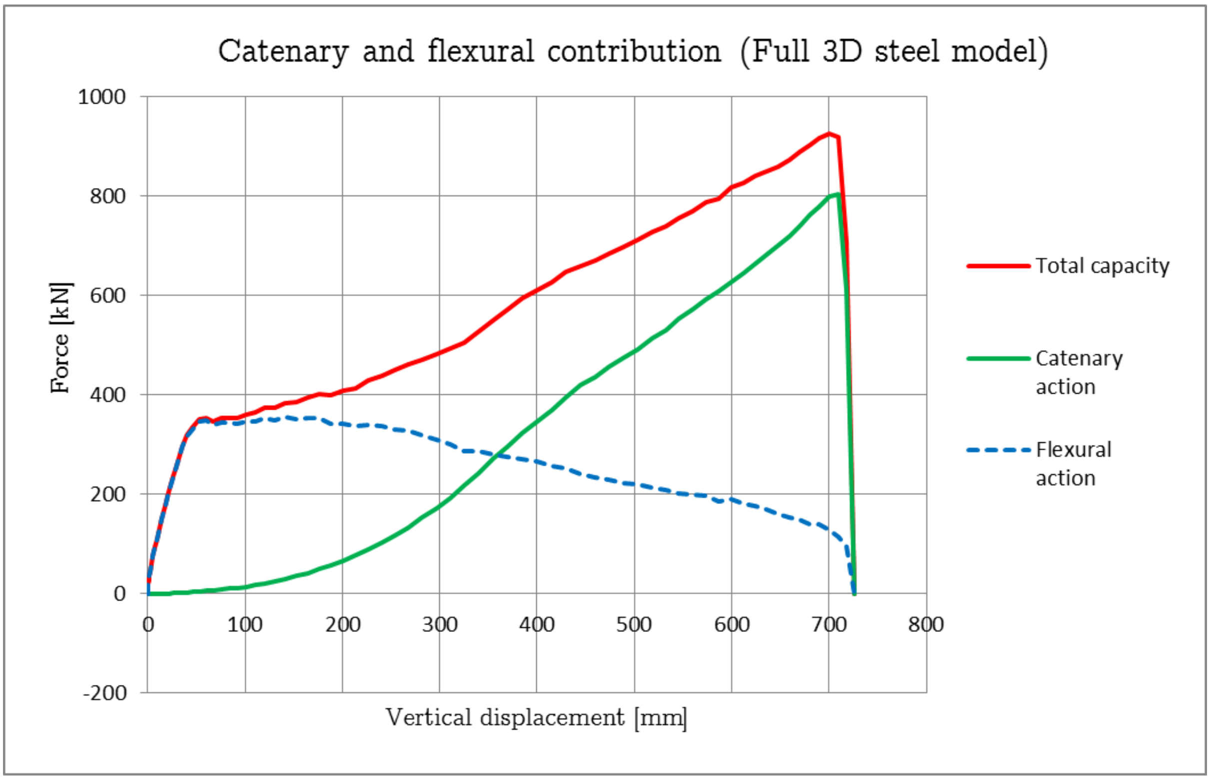


Figure 4.86 Catenary and flexural action for the Full 3D steel model

The above chart is clearly a proof of the beneficial effect of the catenary forces in the beams. This implies that when catenary action is fully activated the beams can resist significantly larger loadings, but as it can be observed in the Figure 4.85 and Figure 4.86, the catenary forces implies significantly large deformation levels to be activated.

Lee et at. (2009) noted that catenary action becomes dominant under very large rotations exceeding 0.1 radians. He concluded also that for moderate amounts of total rotation by the typical connections, between 0.03 and 0.05 radians, the benefit of catenary action is much more modest. In addition, Kim and An (2009) indicated that, in the case of beam total rotations larger than 0.075 radians, large catenary forces were observed as well as significant increase in the strength of a structure in a progressive collapse event. They shown that at 0.25 radians, the catenary force in beams increase to 100% of its yield strength. When catenary action is activated, the tensile forces in the beam increases as the rotation angle increases and the catenary behavior of the beam drastically increases as the flexural capacity of the beam begins to decrease (see Figure 4.85 and 4.86).

Regarding the studies carried out within this research, it was observed that the catenary force is activated at a rotation of 0.033 radians (which means a displacement of 100-150 mm) and becomes the dominant behavior at a rotation of 0.117 radians (which means a displacement of 350 mm). At the failure point, the rotation of 0.24 radians was observed (displacement of 726 mm). As it can be seen, the results of the investigations in this thesis shown similar behavior to those described by Lee et al. (2009) and Kim and An (2009).

Chapter 5: Parametrical study

As it was shown in the previous chapter, the degree of reliability regarding the behavior of the connection between the shear connectors with the beam flanges it is still reduced. However, the models on the basis of defining the exact holes in the slab and tying directly the connectors to the beam flanges shown a superior behavior over the embedded technique, used to define the interaction between the shear connectors with the concrete slab and than tying indirectly the studs to the beam flanges by the meaning of aproximate surfaces on the slab inferior part. For the refrence model, according to the above mentioned principles, in further analysis, the 3D composite model will be used, defined by the direct connection of the shear connectors to the beam flanges. In the following chapter a parametrical study will be carried out regarding several aspects:

- Parametrical study on the variation of the number and location of the shear connectors;
- Parametrical study on the variation of the slab thickness;
- Parametrical study on the variation of the slab reinforcement ratio.

The following table (Table 5.1) it is a summary of the parametrical study and the models defined for this:

Parametrical study summary				
Parametrical study	Model name	Description	Parameter	Value
Variation of the shear studs	Case 1	Shear studs-Double on primary- and 1 row on secondary-beams everywhere	Shear stud	-
	Case 2	Shear studs-3 rows on primary- and 1 row on secondary-beams at each column	Shear stud	-
	Case 3	Shear studs-3 rows on primary- and 1 row on secondary-beams at central column only	Shear stud	-
Variation of the slab thickness	Case 1	Initial/renfrence model	Slab thickness h_p	80 mm
	Case 2	Increase with 10 mm of the slab thickness compared to the initial model	Slab thickness h_p	90 mm
	Case 3	Increase with 20 mm of the slab thickness compared to the initial model	Slab thickness h_p	100 mm
	Case 4	Increase with 50 mm of the slab thickness compared to the initial model	Slab thickness h_p	130 mm
Variation of the reinforcement ratio	Case 1	Initial/renfrence model reinforcement of Φ6/200 mm	Reinforcement ratio ρ	0.178%
	Case 2	Decrease of the initial model reinforcement to Φ4/200 mm	Reinforcement ratio ρ	0.079%
	Case 3	Increase of the initial model to reinforcement to Φ8/200 mm	Reinforcement ratio ρ	0.314%
	Case 4	Increase of the initial model to reinforcement to Φ10/200 mm	Reinforcement ratio ρ	0.491%

Table 5.1 Parametrical study summary

5.1 Parametrical study on the variation of the number and location of the shear connectors

In the following chapter, the number and position/spacing of the shear connectors will be studied. As presented in Figure 4.79, the model with the direct tying of the studs had better behavior, however, further analysis will be carried out after experimental testing of the specimens in order to calibrate the material model in the software and to validate the exact behavior and solution/method. In order to see the behavior of the structure regarding the number and spacing of the shear connectors, two more scenarios were studied beside the case presented above, with two row of shear connectors on the primary- and one row on the secondary-beams. Firstly (case

2), a model was defined having three rows of double shear connectors (3x2) only at the central column (damaged column) on the primary-beam and everywhere else one row of shear studs (including the secondary-beams as well) (Figure 5.1, left). For the second scenario (case 3), the same configuration of three rows of double shear connectors was used, but for all columns present in the structure and one row of shear studs on the secondary-beams (Figure 5.1, right). This configuration it was useful to study the behavior of the structure in the case of column loss scenario. Due to the fact that in a real case scenario, the position of the damaged column in an accidental situation is unknown. During the structural design process, the position of the damaged element is usually unknown for the design engineer, compared to the given situation within this project, where the position of the damaged column is defined, namely the central column. For this reason it is necessary to study the spacing and number of connectors within the structure.

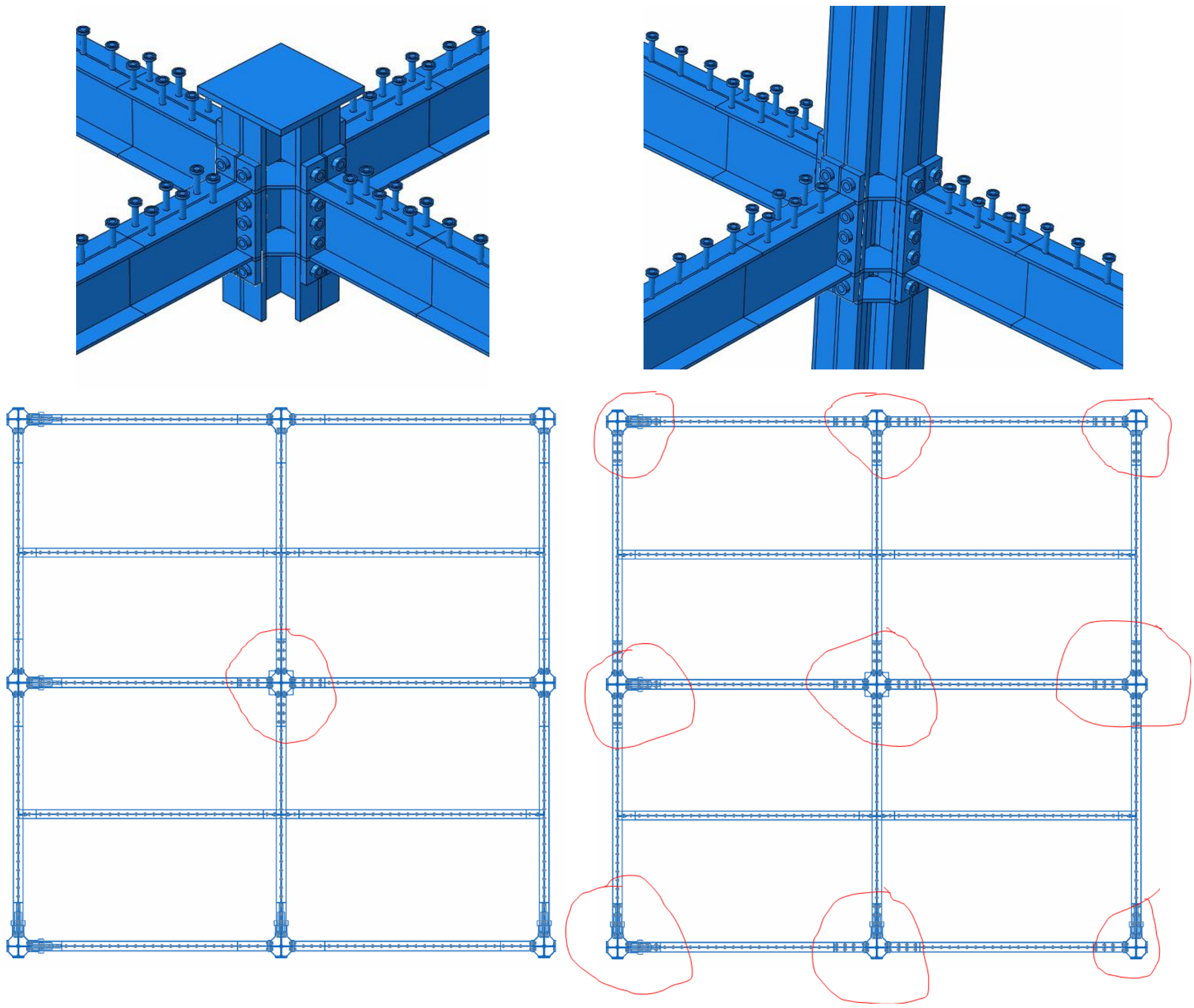


Figure 5.1 Number and spacing of shear connectors

All the other parameters and elements were defined similarly to the case before, the only differences being in the terms of the shear connectors. A displacement of 800 mm in the vertical direction with uniform distribution was applied, using amplitude

type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column. Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 5.2).
The failure occured at the most affected zones in all cases, namely at end of the beams, near to the beam-to-column connections, where the plastic hinges developed. In all scenarios presented above, the failure load occured at the same moment aproximatly (case 3 at 460 mm, case 2 at 495 mm and case 1 at 540 mm), however the behavior of the structures was significantly different as it can be seen in Figure 5.2. By reducing gradually the number of the shear connectors in the models, the load bearing capacity of the system increased from 1390 kN (case 1) to 1450 kN (case 3). This means an increase of 4.32% in terms of the bearing capacity. The reason of this increase is still uknown and the need for material model calibration became vital for an accurate behavior modeling. In case 3 scenario occured the failure mechanism at a displacement of 460 mm, compared to the case 1 scenario, where the failure occured at a displacement of 540 mm (Figure 5.2). However, the failure load in case 3 it was higher than in the case 1 noticed.
Due to the fact that in the real design situation, the exact position of the damaged or susceptible to damage column/member is relatively unknown, for the further analysis, the model with three rows of double shear connectors on the primary- and one row on the secondary-beams at each column (case 3) will be adopted. The reason for this is, that in a design process all the columns can be susceptible to damage and all of them need to be considered in an equal or symmetrical manner in the analysis.

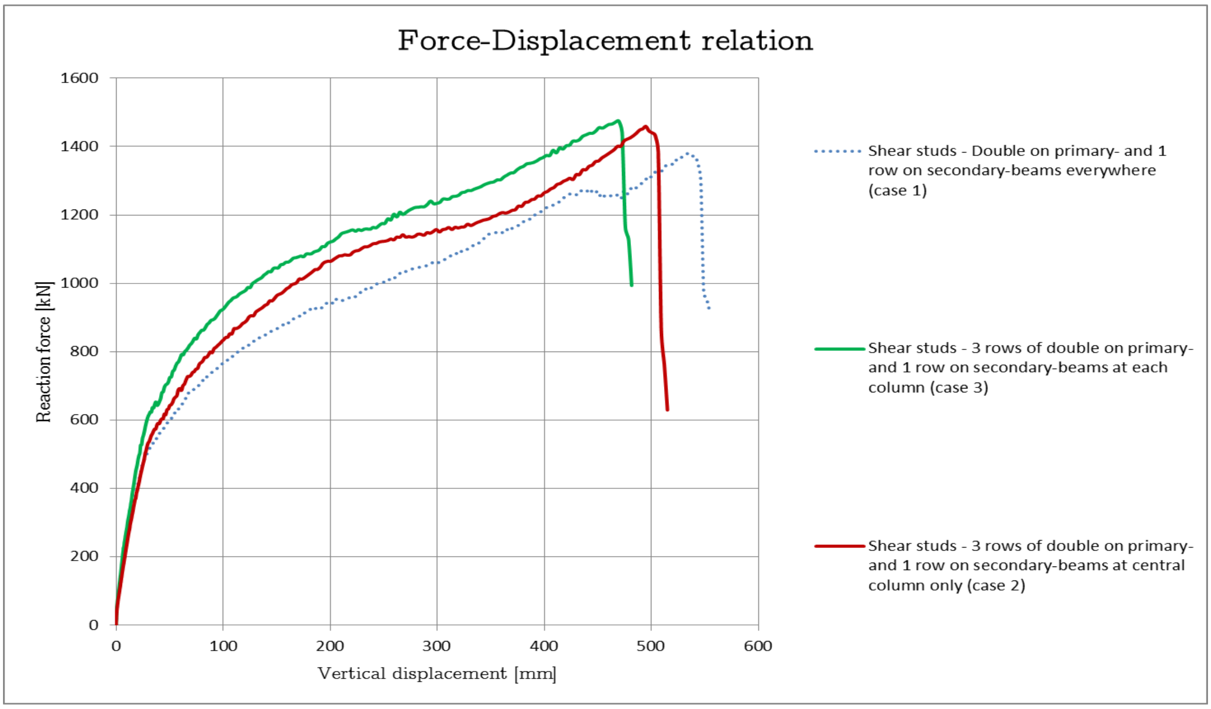


Figure 5.2 Force-Displacement relation for case 1, case 2 and case 3 (shear studs)

In the absence of calibrated material models, in order to validate the model and the method used for the definition of the interaction between the shear connectors and the beam flange, the analysis of the column loss scenario for the 3D composite model have been performed using also the Extreme Loading for Structures (ELS) software. The solver of this analysis software uses the so called Applied Element Method (AEM) instead of the conventional Finite Element Method (FEM). This is a relatively new mathematical approach which allows taking into consideration separation of elements and kinematic element interaction (contact at impact) for a reduced computational cost. 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. It was modeled approximately the same structure with similar materials in ELS and the force-displacement relation obtained is presented below:

Figure 5.3 Force-Displacement relation for case 1, case 2, case 3 (shear studs) and ELS Analysis

As it can be observed in Figure 5.3, the behavior of the structure in ELS was somehow similar to the one performed in Abaqus. In the next chapter, the parametrical study on the variation of the slab thickness will be presented on the case 3 model.

5.2 Parametrical study on the variation of the slab thickness

In the following chapter, the slab thickness contribution to the behavior of the structure will be studied. The parametrical study will be carried out on the case 3 model as mentioned in the previous chapter. In addition to the initial scenario, where the slab thickness was 800 mm, three models were defined having the following slab thickness:

- slab thickness of 90 mm;
- slab thickness of 100 mm;
- slab thickness of 130 mm.

In the present chapter the name convention of the models will be the following: model having slab thickness of 80 mm (case 1), model having slab thickness of 90 mm (case 2), model having slab thickness of 100 mm (case 3) and model having slab thickness of 130 mm (case 4). In all the cases, the only parameter modified was the slab thickness. All the other elements were similarly defined as in the initial/refrence model, respecting the similar concrete cover values as well. A displacement of 800 mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3, Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column. Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 5.4).

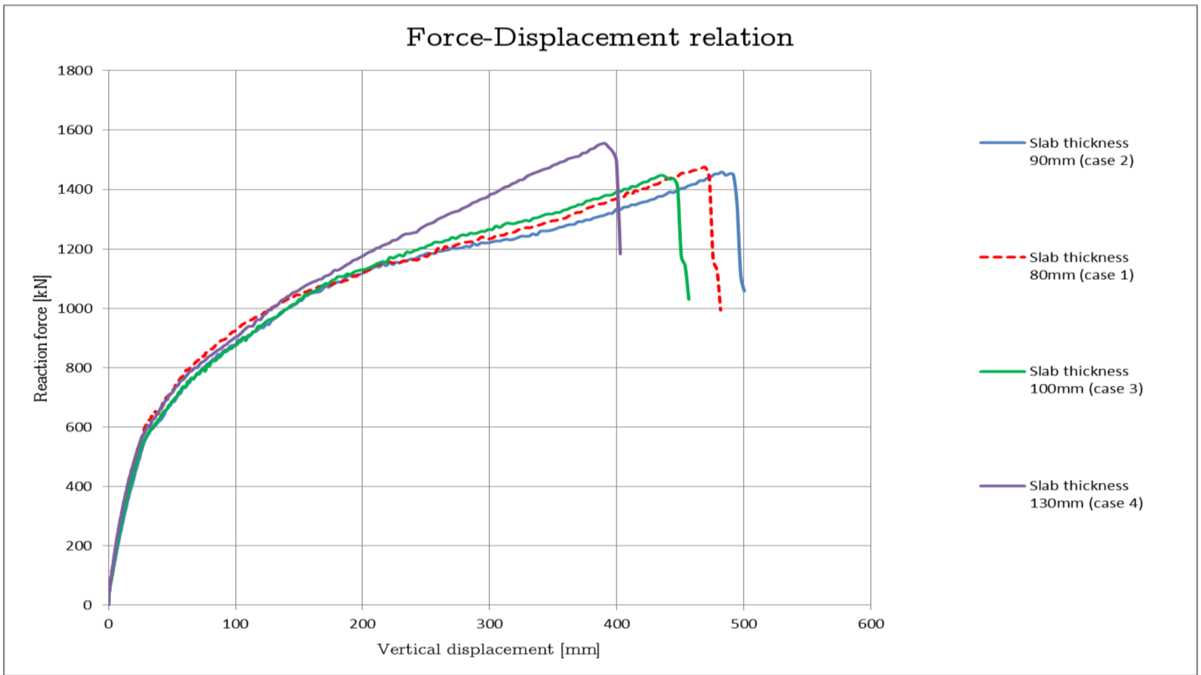


Figure 5.4 Force-Displacement relation for case 1, case 2, case 3 and case 4 (slab thickness)

The failure occured at the most affected zones in all cases, namely at end of the beams, near to the beam-to-column connections, where the plastic hinges developed. In all scenarios presented above, the failure load occured at the same moment aproximately (case 1 at 480 mm, case 2 at 490 mm and case 3 at 440 mm), just for case 4, the failure occured at a relatively earlier stage (390 mm). However the behavior of the structures was significantly different for case 1, 2, 3 and case 4, as it can be seen in Figure 5.4. By increasing gradually the concrete slab thickness in the models, the load bearing capacity of the system changed accordingly. While for a slab thickness of 80 mm, 90 mm and 100 mm, the maximum load that the structure could undertake was similar in all cases (approximately 1450 kN), by increasing the slab thickness to a larger value (130 mm) the behavior changed significantly (1580 kN and earlier stage failure 390 mm). This means an increase of 9.0% in terms of the bearing capacity due to the additional stiffness provided to the system by the slab thickness increase. Even the increase is less than 10%, the behavior of the structure is significantly changed by increasing in a large manner the slab thickness. In this manner the structure is able to undertake higher loads but the failure occurs at smaller displacements than in the case of reduced slab thicknesses. According to Figure 5.4 it is clear that the significant increase of the slab thickness may have a beneficial effect on the behavior of the structure in the case of column loss scenario.

5.3 Parametrical study on the variation of the slab reinforcement ratio

In the following chapter, the slab reinforcement ratio contribution to the behavior of the structure will be studied. The parametrical study will be carried out on the case 3 model. In addition to the initial scenario, where the slab reinforcement ratio was 0.178% (Φ6/200 mm), three models were defined having the following slab reinforcement ratio:

- slab reinforcement ratio of 0.079% (Φ4/200 mm);
- slab reinforcement ratio of 0.314% (Φ8/200 mm);
- slab reinforcement ratio of 0.491% (Φ10/200 mm).

In the present chapter the name convention of the models will be the following: model having slab reinforcement ratio of 0.178% (case 1), model having slab reinforcement ratio of 0.079% (case 2), model having slab reinforcement ratio of 0.314% (case 3) and model having slab reinforcement ratio of 0.491% (case 4). In all the cases the only parameter modified was the slab reinforcement ratio. All the other elements were similarly defined as in the initial/refrence model, respecting the concrete cover as well. A displacement of 800 mm in the vertical direction with uniform distribution was applied, using amplitude type smooth step (Chapter 4.3,

Figure 4.5). The loading was applied to the refrence point initially created and constrained (coupled) with the superior part of the top plate of the central column. Full analysis was created, using multiple processors (4) and single precision degree and for the output of the analysis, the main data was the force-displacement curve of the load application control point (Figure 5.5).

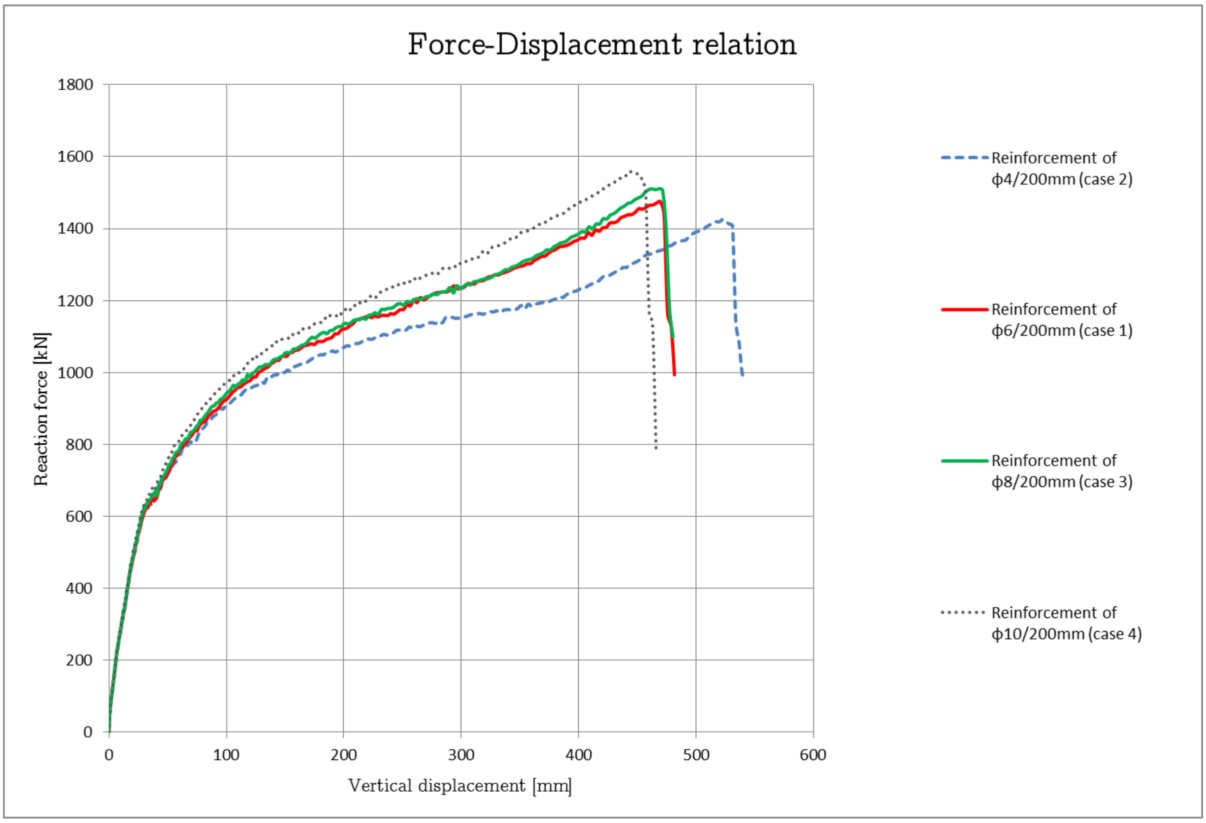


Figure 5.5 Force-Displacement relation for case 1, case 2, case 3 and case 4 (reinforcement ratio)

The failure occured at the most affected zones once again, at the end of the beams, where the plastic hinges developed asfter significant vertical displacement was applied. As it can be seen in Figure 5.5, for case 1, the failure was observed at a later stage (520 mm) and a failure load of 1410 kN. By decreasing the reinforcement ratio, the structure became more ductile, thus the failure occuring at larger vertical deflections compared to the refrence case 1, where the failure occured at a displacement of 480 mm with a failure load of 1440 kN. As it can be seen, by decreasing the reinforcement ratio and providing to the structure a higher flexibility, the system was able to undertake larger deformations and it was failing in later stages; even the failure load was with 2.12% higher in case 1 what the structre could undertake. It can be also noticed that by reducing the rebar ratio, the structural behavior is more close to the just steel one (having larger ductility) while increasing the rebar ratio is more stiff.

For case 3, by increasing the reinforcment ratio in a small manner, the difference observed was very small in the terms of failure load and failure deformation. What concerns case 4, by increasing almost double the reinforcement ratio compared to the intial case, the obtained behavior of the structure was the increased bearing capacity. The system was failing at a load of 1580 kN and a displacement of 460 kN. This means an increase of 9.8% in terms of the bearing capacity. In this manner the structure is able to undertake higher loads but the failure occurs at smaller displacements than in the initial case 1 model.

To conclude the studies carried out about the effect of the reinforcement ratio on the behavior of the structure, one can say that, the variation of the reinforcement bar ratio of this four type of members is small. Therefore, the difference in the results was not remarkably. However, by increasing the reinforcement ratio over the values used in a conventional design situation, the steel reinforcement may help the damaged slab for preventing progressive.

Chapter 6: Conclusions and Discussions

6. Conclusions and Discussions

It was shown in the literature through numerical investigations performed at the Univeristy of Liege by Demonceau, that is possible to extract a simplified sub-assembly able to reproduce and simulate the global response and behavior of a reference building structure. Within this thesis, a simplified sub-assembly has been defined and extracted from the full 3D structure with the objective to check the possibility of this sub-structure to simulate, with a sufficient accuracy, the behaviour of the real structure when significant membrane and catenary forces develop.

Although some research has been done so far in the field of robustness, they are all based on pure steel frames without considering the contribution of the floor systems. Most of them are 2D models. Therefore, it is unrelated to real structural performance. Without considering the contribution of the slabs, the beneficial effects of such as compressive arching and catenary actions are not clear. Therefore, more detailed research on the progressive collapse of multi-storey building is needed.

In addition, the simulation studies presented within this document shown that there are distinct and large differences between the results of the planar (2D) and 3D models. The 3D models are considered more realistic than the planar models in that they can capture the effects of the slab and its interaction with the other frame members. These conclusions suggest that a full 3D analysis, in spite of its computational cost, may be the only sure way to explicitly investigate structural robustness of buildings.

The proposed model in this thesis is a full 3D finite element model to investigate the progressive collapse of multi-storey buildings using nonlinear quasi-static analysis method. Based on models presented above, parametrical studies were carried out to investigate the resistance ability of this type of buildings under sudden column loss scenario. Through the parametric study, the measures to mitigate progressive collapse in future design can be also recommended.

The risk assessment of a multi-storey building shows that, one column removal scenario is the most frequently occurring scenario. Therefore, most of recent research has focused on the one column removal of multi-storey buildings. For most research done so far, the plasticity is presumed to develop in the steel member under the one column removal scenario, and a plastic hinge is formed in the beam, therefore most research are based on the plasticity theory. However, this is not always true, after removal, the beam may still be in the elastic phase as shown in the literature.

In what follows, several aspects will be discussed regarding their effect on robustness and progressive collapse mitigation in the case of building structures. The results of this thesis will be presented, concluded and compared to the relevant results available in the literature.

Effect of membrane action, slab thickness, variation of the shear connectors and reinforcement ratio on the progressive collapse resistance

The investigation within this research started with the following assumption: to mitigate progressive collapse, composite floor systems may play an important role in providing effective tying.

As it was shown in Chapter 4, Section 4.8 and also throughout the parametrical studies carried out in Chapter 5, Section 5.2, the composite action induced by the presence of the reinforced concrete slab in the model it was remarkably in the behavior of the structure.

Especially, considering the primary-beams composite too, the effect was large enough to resist large applied loadings. Considering only the secondary-beams acting compositely, the behavior of the structure was more likely in the case of the full 3D steel model or better say, it was between the full 3D steel model and the full 3D composite model. Membrane forces are inevitably developing in the plane of a loaded slabs and they may have a key role in its collapse resistance. After large deformations are reached, the slab will have a tendency of inward movement under the action of the increasing tensile forces at the central areas of the floor, but it is restrained from doing so by the adjacent parts, creating a peripheral ring of compression forces supporting the central area of tensile forces. The overall load carrying capacity of the slab therefore comprises tensile membrane capacity in the central deflected area of the slab and the enhanced flexural capacity in the outer ring where in-plane compressive stresses occur.

Compared to catenary action in the beams, membrane action in slabs do not require any lateral restraint from adjacent boundary elements.

Compared with the effect of the connections, where the connection stiffness improves the tying capability in one-dimensional, the effect of the slabs on the structure is two-dimensional quality. The top side of the slab is in compression and the bottom side is in tension. Membrane forces are mainly carried by the steel reinforcement in the slab. The effective tying can be improved if the bottom side of the concrete are reinforced for tension. Sufficient tension reinforcement is essential to increase progressive collapse resistance of the system.

In order to evaluate the effect of the steel reinforcement ratio used in the concrete slab, four types of steel reinforcement ratios were chosen to be studied in the parametrical study of this thesis (see Chapter 5, Section 5.3). The results shown that for the conventional steel reinforcement retios used in current design practice, the variation of the deformation is small. The variation of the reinforcement ratio led to the variation of the ultimate capacities of the structure and the variation of the deformations as well. By decreasing the reinforcement ratio, the structure became more ductile and the forces were mainly taken by the steel beam rather than the slab. When the reinforcement ratio was increased in a small manner, the difference observed was very small in the terms of failure load and failure deformation. What concerns case 4 model, by increasing almost double the reinforcement ratio compared to the refrence case, the obtained behavior of the structure was the increased bearing capacity. By this manner the structure was provided with increased stiffnes but less ductility.

It can be concluded that, just if the reinforcement ratio is increased significantly in the slab, a significant increases in the resistance of the structure can be achieved. The increasing ductility increases the energy absorption capacity of the joints. This is because the ductile joints allow for redistribution of internal forces within the structural system by enabling large deformations so that they are suitable for progressive collapse mitigation by transition from flexural loading to axial loading in the members and joints and initiating of a catenary action. When progressive collapse occurs, the connections are subjected to significant tension, which is different from their behavior in normal load condition. Tensile reinforcement in the vicinity of the connections may reduce the risk of progressive failure also.

Regarding the parametrical study developed on the location and number of shear connectors within Chapter 5, Section 5.1, it can be concluded that, according to the results obtained there is no clear evidence that the connectors were pulled out from the concrete part due to the limitation in the modeling process. However, it can be concluded that the arrangement of the shear studs over the limits of a conventional design may not necessary increase the resistance capacity of the structure or its ductility. Although, some differences in the results were observed, further improvements and corrections are necessary in the terms of the material model calibration and modeling technique validation.

The parametrical studies carried out on the slab thickness variation in Chapter 5, Section 5.3, demostrated clearly that only with a substantial increase in the concrete slab thickness it was possible to produce significant increase in the capacity of sub-assembly, accompanied with the reduction in ductility. It can be said that, by increasing the concrete's slab thickness, it will increase the resistance to the progressive collapse as the vertical deflection is reduced and the overall stiffness of the building is increased.

Studies developed within this thesis shown clearly that the use of the composite action of the slab may increase with 42.06% the capacity of the structure (see Figure 4.73) by the means of membrane actions developed in the structure. This means that by considering the composite action it was almost doubled the resistance of the system to failure after the column removal scenario.

Measures to mitigate progressive collapse in building structures and future research recommendations

Based the above results, one can conclude that the design of steel beams as composite, reduces very much the risk of progressive collapse for the column removal scenarios considered here, thus, increasing the robustness of the system. Especially, designing the primary-beams compositely too, can help in increasing the resistance of the structure. Compared to catenary action, which is effective in preventing progressive collapse if and only if lateral restraint from adjacent boundary conditions are adequate, membrane action in slabs do not require any lateral restraint from adjacent boundary elements. However, the composite action is more effective for internal spans or internal column removal scenario, where the membrane action may develop in the concrete slab. In the case of outer spans, the efficiency is much reduced when critical members are removed.

From above discussions and parametric studies carried out it can be seen that, for the multi-storey composite steel frame buildings, the possbility to mitigate the progressive collapse might be to increase the thickness of the concrete slab, as well as to vary the reinforcement ratio within the concrete slab or to design the connections to be fully rigid in order to enable the formation of plastic hinges in the beams, thus, allowing the development of catenary forces in beams. Although the role of the shear connectors within the composite structure is obvious, it was not found clear evidence that the variation of them would have a beneficial effect or not due to the model limitations.

Another effective way to resist progressive collapse can be to decrease the spacing of the grid or provide more redundancy in the structural scheme/shape.

The collapse mode and failure mechanism of the steel only structure involves the plastification of the end of the beams. Firstly, the top flange at the remote columns are plastified, followed by the bottom flange of the beam near to the internal (damaged) column connection. In the beginning of the loading process, the section of the beam is symetrically divided to a compressed and tensioned part, but after large deformations its clearly visible that the cross-section of the beam is heavily subjected to tension stresses only. Plastification of the bolts and end-plates were observed as well. The failure is initiated by the facture of the beam bottom flange at the location of the plastic zones, followed by increased plastification of the bolts and end-plates as well.

In the case of the composite structure, the detailed identification of the failure mechanism it was not clear. It should be mentioned that the degree of reliability of the results might be reduced due to the issues incured in modeling of the complex behavior of concrete and shear studs connection. However, it was noticed that the plastic hinges or plastification of the structural components occured in similar zones as in the only steel structure. The fracture of the beam and the plastification of several elements it was observed during the analysis. In order to predict the exact mechanism of failure, further analysis and corrections are needed.

Although membrane and catenary actions have been investigated for years, most research studies focused on composite slabs under fires and simple 2D models. So far, very limited numerical, analytical and experimental works on these mechanisms under column loss condition have been published, if at all.

In this thesis, by using numerical investigation tools, membrane and catenary actions have been shown to be a feasible solution for preventing progressive collapse of building structures under accidental column loss scenarios. The load-baring capacity of be sub-assemblies has been shown to be significantly enhanced by membrane actions in slabs and catenary action in the double-span beams when displacement becomes significantly large. Following the above idea, it was noticed a very large increase in the performance of the structure if composite action between beams and reinforced concrete slab was taken into account. From the perfomed analysis it was possible to determine separetely the contribution of the catenary action and flexural action into the total capacity of the structure, offering a clear proof of the presence and beneficial effect of the catenary forces in progressive collapse mitigation.

The present document represents a preliminary numerical investigation related to the catenary forces in beams and membrane forces in slabs after one column removal scenario.

Although the finite element analysis proceeds very well into the membrane and catenary stage in all cases, some limitations do still exist. These limitations address the need for further numerical model calibration, material model calibration and experimental testings as well. Due to the previously mentioned words, this numerical investigation it is just an approximation of the behavior of the structure; further studies and researches need to be done and model validation is compulsory for future assumptions.

The slab contribution by the means of membrane action to the global behavior of structures need to be further studied. Further studies about the specific mechanics by which a moment resisting frame devolves from a flexure dominant system to a tensile membrane or catenary dominant system need to be established.

As conclusion of the work, results shown that structures are able to utilize catenary and membrane behavior, which are activated only in the case of very large deformations, past their flexural capacity, to resist additional applied loadings.

It is suggested that, consideration of these forces be incorporated in future codes and design procedures, aimed to mitigate the probability of progressive collapse of building structures.

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