PROGRESSIVE COLLAPSE ANALYSIS OF STEEL FRAMED BUILDINGS

With
Autodesk Robot Structural Analysis

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ABSTRACT

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ABSTRACT

This thesis was commissioned by Pöyry Finland ÖY to study the process of progressive collapse in steel framed buildings.

The phenomenon occurs when a critical member such as a beam or column suddenly fails, resulting in redistribution of load paths to surrounding members. If the adjacent members are not capable of supporting the new loads, they begin to fail successively. This in turn leads to a chain reaction that may result in partial or total collapse of the building.

The study determined under what conditions such collapse occurs, how to control or mitigate it. Methods used were study of existing literature on the subject, lectures, as well as a case study in Robot Structural Analysis of a steel structure.

The results obtained from the analysis model showed that the structure was robust and collapsed once in more than 25 different accidentally loading situations. Results showed that the building was most susceptible to notional corner column removal.

Keywords Progressive Collapse, Steel frame, Robot Structural Analysis, Nonlinear

Pages 66 pages

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1 INTRODUCTION

After the partial collapse of the 21-storey Ronan Point apartments (1968) in Canning Town, East London from an accidental gas explosion, engineers and regulators alike began to consider Structural Robustness more seriously, or as referred to by Eurocode, Structural Integrity. Structural Robustness has been defined as;

"The ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause" (EN 1990)

Engineers thus set out to study and predict the conditions under which progressive collapse occurs and the measures to prevent it.

Progressive collapse is charactarised by a disproportionately undesired outcome to the event that initiated it. A local failure of a primary structural component results in the failure of neighbouring members successively until either partial or total collapse of the structure has occurred. It is an inelastic nonlinear dynamic behaviour charactarised by large deformations and contact impact. The structure will continue to collapse until a new global static equilibrium is reached. (El-Tawil n.d,).
Figure *1* shows the phenomenon in progress.

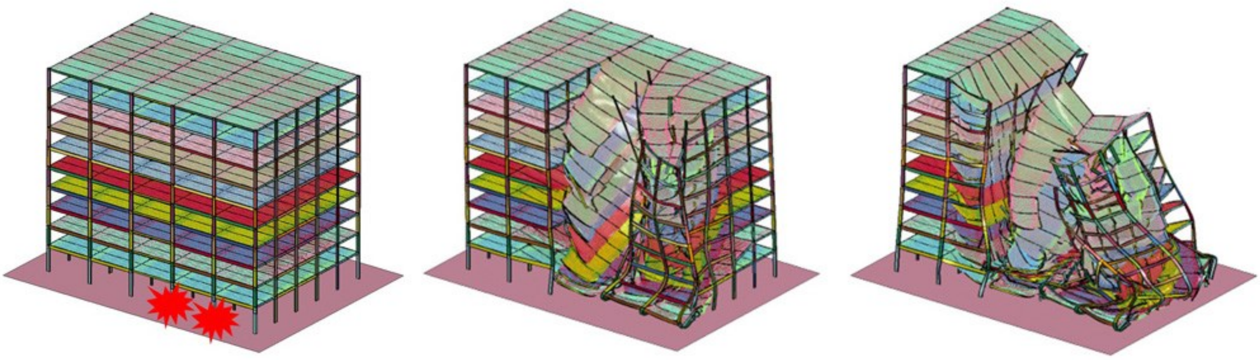


Figure 1. Progressive collapse simulated (El-Tawil, University of Michigan)

There is an important need to design with progressive collapse in mind due to an increasing risk of predetermined attacks on public buildings, as well as accidental factors. With more and more people being drawn into cities that are getting bigger and bigger, the cost of human life in collapse events is too high.

It has been seen that structures from the past were able to resist loads for longer and generations of people have used the same structures without the threat of progressive collapse. This can be seen from the numerous Roman and Greek buildings still standing today and still accessible to tourists. However, according to the Naval Facilities Engineering Command, recent developments in optimisation, innovative framing systems and refinement of analysis techniques have resulted in structures with a considerably smaller margin of safety.

Framing systems designed for easing the construction process, reducing time and costs have led to less structural continuity and less resistance to abnormal loading situations. (Crowder, 2005, p.6)

Figure 2 shows the difference in column size between ancient Greek columns vs modern columns. It is possible to say that the Greeks and Romans over-designed to err on the side of caution because they did not fully understand the principles of mechanics nor had the mathematical computational tools we have today. Reinforced concrete had not been invented yet restricting spans to tensile strengths of stone and mortar used. Today's engineering delivers only the needed capacity reducing material waste and costs but sensitive to unplanned abnormal loads.

Figure 2. The Leslie Dan Pharmacy Building, (Toronto) compares to The Temple of Olympian Zeus (Athens)

2 STANDARDS AND GUIDELINES

2.1 Historical Timeline

The Ronan Point Apartment collapse shown in Figure 3 below was an important event because it showed how lacking guidelines and standards were regarding the phenomenon.

Figure 3. Ronan Point Apartments (The Guardian 2018)

According to the forensic inquiry into the collapse by the Ministry of Housing and Local Government (Griffiths, Pugsley, Saunders. 1968), it was found that:

- There were no building standards violated in the design and construction of Ronan Point
- Poor workmanship was not a factor that led to the collapse of the building
- The building standards at the time gave clear guidelines on design of elements but did not give any guidelines on global stability

- Connection resistance was based only on friction, and gravity loading
- Upon notional removal of walls, the connections were incapable of redistributing loads since they were designed only for compression

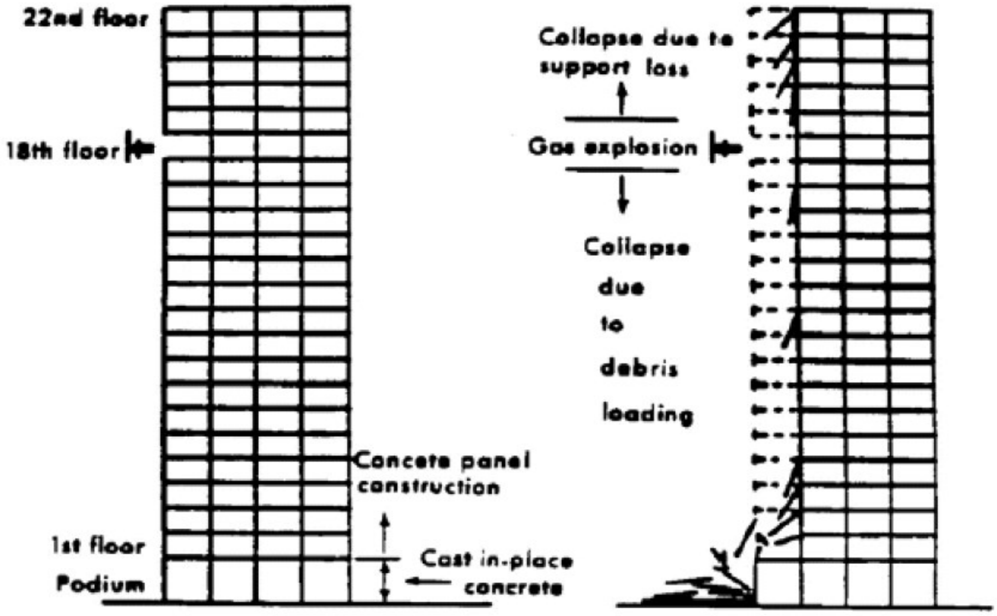


Figure 4. Ronan Point location of failure and ensuing collapse (Crowder, 2005, p.7)

Interest in progressive collapse was created and later that year, November 1968, The UK Ministry of Housing and Local Government issued standards to avoid progressive collapse. (Crowder, 2005)

In 1970 the standards recommended by the ministry became mandatory building regulations. Four years later provisions of structural ties entered the British Standard. (Crowder, 2005).

By far the two most extensive and exhaustive codes that cover the subject of progressive collapse are the British national standards and the American District of Defence (DoD) recommendations.

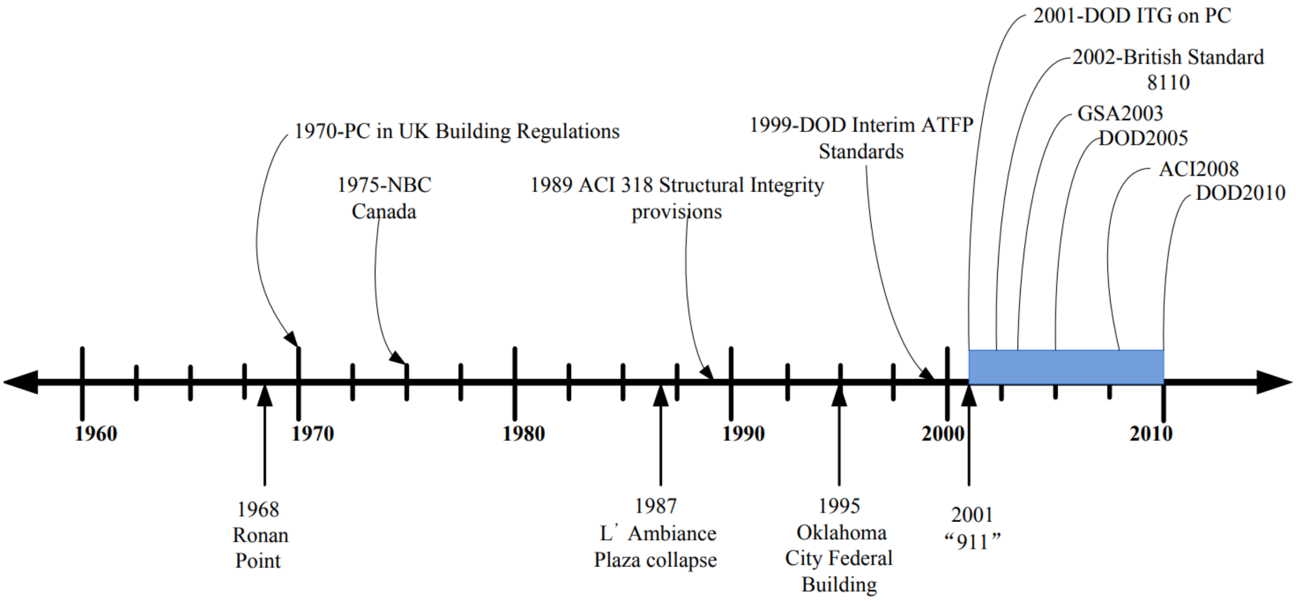


Figure 5. Significant Progreesive collapse events (Wang, Zhang, Li, Yan, 2014, p.2,)

2.2 Eurocode

The Eurocode stipulates that:

"A structure shall be designed and executed in such a way that it will not collapse to an extent disproportionate to the original cause when damaged by events like fire, explosion, impact or consequences of human error," (EN 1990-2-1 p.23)

It goes on to list choices engineers can use to limit damage from the aforementioned events. Engineers should:

1. Avoid or eliminate the hazard the structure is subject to
2. Select structural forms that have low sensitivity to the hazard
3. Select structural forms and designs that can survive adequately the removal of a member
4. Avoid structural systems that can collapse without warning
5. When possible, tie structural members together

2.3 British Standards

The British standards have no mention of progressive collapse. Instead they use the term Disproportionate Collapse. It is stated:

"A building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause." (BS, Approved document A, 2010, p.41)

Local collapse is limited to 15% of floor or roof area or $70m^2$ whichever is less at the relevant level and on the immediate adjacent level, below or above. (Crowder, 2005, p.3)

The British Standard, Approved Document A (AD-A) is very similar to Eurocode EN 1991-1-7 Annex A, Design for consequence of localised failure in buildings from an unspecified cause.

Table 1 below shows the classification of buildings according to their degree of importance or damage caused if collapsed. There are also buildings which do not come into Class 3 by virtue of their size but still merit Class 3 treatment because of their value or vulnerability. Examples of these are animal research centres, banks, control centres, embassies, religious buildings and museums.

Table 1. Building classes (The Building Regulations, nd)

Consequence Classes	Building type and occupancy
1	Houses not exceeding 4 storeys
	Agricultural buildings
	Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2a Lower Risk Group	5 storey single occupancy houses
	Hotels not exceeding 4 storeys
	Flats, apartments and other residential buildings not exceeding 4 storeys
	Offices not exceeding 4 storeys
	Industrial buildings not exceeding 3 storeys
	Retailing premises not exceeding 3 storeys of less than $2000m^2$ floor area in each storey
	Single-storey educational buildings
	All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding $2000m^2$ at each storey
2b Upper Risk Group	Hotels, blocks of flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys
	Educational buildings greater than 1 storey but not exceeding 15 storeys
	Retailing premises greater than 3 storeys but not exceeding 15 storeys
	Hospitals not exceeding 3 storeys
	Offices greater than 4 storeys but not exceeding 15 storeys
	All buildings to which members of the public are admitted which contain floor areas exceeding $2000m^2$ but less than $5000m^2$ at each storey
	Car parking not exceeding 6 storeys
3	All buildings defined above as Consequence Class 2a and 2b that exceed the limits on area and/or number of storeys
	Grandstands accommodating more than 5000 spectators
	Buildings containing hazardous substances and/or processes

2.4 American Standards

Design requirements in the Unified Facilities Criteria (UFC 4-023-03) published by the United States Department of Defence (DoD) highlights two approaches to designing for Progressive Collapse of new buildings and retrofit of existing ones. Table 2 below shows the level of protection given by the various methods.

I. Direct Design Method: This is use of the Alternate Path Method which gives explicit criteria for designing when a loss of a critical member occurs. This method takes Flexural Action into consideration through bending in beams and membrane action of large slabs

II. Indirect Design Method: This approach gives general recommendations for how structures should be in order to improve their ductility and connection stiffness. This method makes use of Catenary action.

Table 2. Level of protection and design requirements (Deneke, 2005, p.19)

Level of Protection	PC Design Requirement
Very Low	Provide horizontal Tie Forces.
Low	Provide horizontal and vertical Tie Forces.
Medium	Satisfy the following three requirements: A) Provide horizontal and vertical Tie Forces. B) Apply the Alternate Path method. C) Meet additional ductility requirements that effectively “harden” the perimeter, ground-floor load-bearing elements
High	

Details of these methods are covered in chapter 4: Design for Progressive Collapse.

3 MECHANICS OF PROGRESSIVE COLLAPSE

Developing multi-scale models is crucial for progressive collapse analysis. The process is a challenge that is faced in any field that utilises computational modelling for predicting behaviour where experimental data is not available. A material in a multi-scale model can behave differently depending on the scale being viewed. The information from lower length scales is used to derive the models at larger lengths and scales (Khandelwal, 2008, p.55).

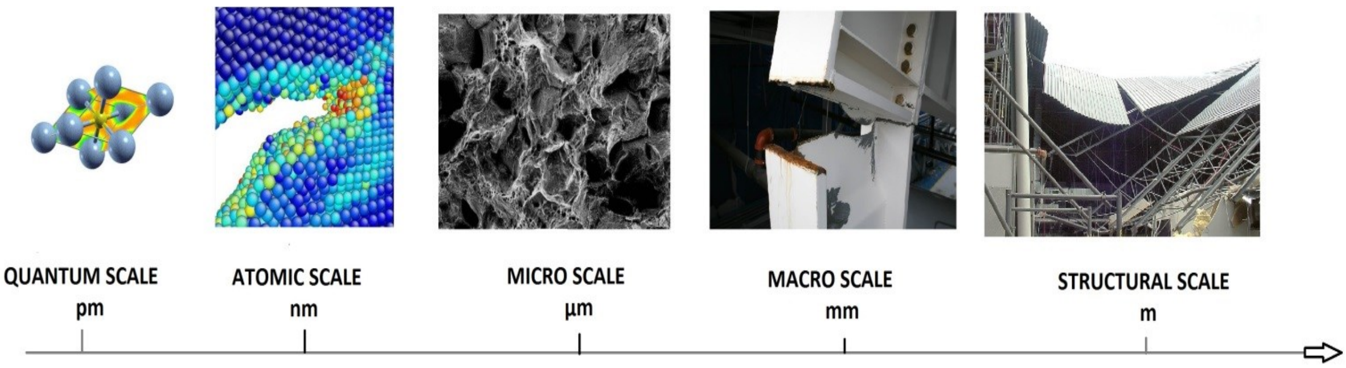


Figure 6. Structural Simulation Scales (Echeverri-Restrepo S. et al, 2017)

Multi-scale modelling involves developing a working model at different lengths of scale and time, bridging their behaviour in an effort to understand a process, from the electronic and molecular structure to how grains form and materials fail when stressed.

On the larger structural scale buckling of entire walls or progressive collapse of a building occurs, which is known as global behaviour. At a lower level on the macro scale local buckling and local fractures occur (El-Tawil, 2010). The micro level shows voids forming that move in the metal lattice like bubbles elongating and connecting to form fractures. On the atomic scale behaviours such as slip planes and atomic skipping are observed.

3.1 Atomic & Micro Scale Structure

3.1.1 Slippage and Dislocation

It has been found that theoretical strengths of perfect metal crystals are higher than those measured in lab settings. It was determined that the differences in strengths were due to atomic dislocations. Plastic deformations happen when the net movement of a large number of atoms respond to an applied stress. (Academic Resource Centre, n.d)

The process by which plastic deformation is produced by dislocation is called Slip. When metals are plastically deformed a fraction of the energy, about 5% of it, is retained internally as strain energy. The remainder is dissipated as heat. Lattice distortions will exist around dislocation lines. (Academic Resource Centre, n.d)

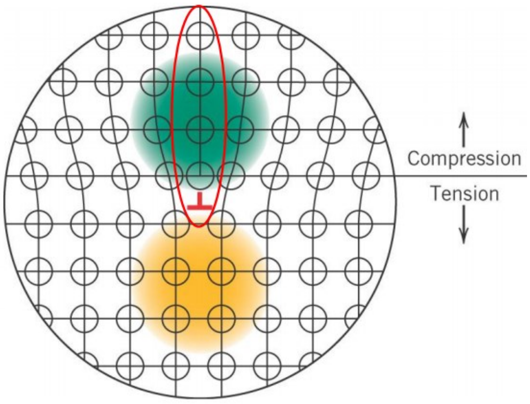


Figure 7. Strain energy around dislocation (Princeton.edu, n.d)

3.1.2 Ductile Fracture

Ductile Fracture is preceded by plastic deformation. No material is perfectly homogenous. When an external force acts on a material, hard regions in the lattice called inclusions do not deform at the same rate as the rest of the matrix. Voids are nucleated to accommodate for the incompatibility. The nucleation event may involve, for example, the fracture of the inclusion, or decohesion at the inclusion-matrix interface. (Bohemen et al, n,d) Once these voids have grown sufficiently and began to coalesce, then fractures visible at a macroscopic scale can form as shown in figure 8 below.

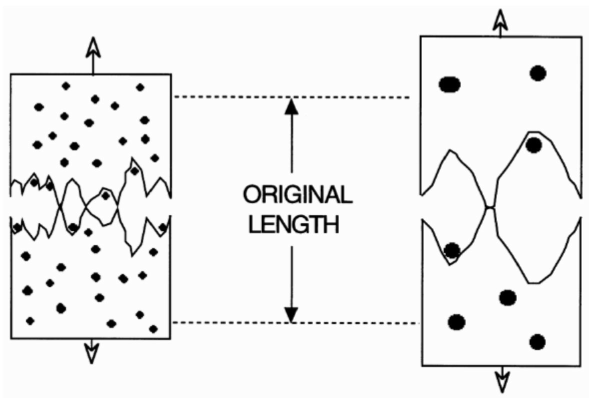


Figure 8. Coalescence of voids to create a fracture (Bohemen et al, n,d)

When fractographs of a ductile fracture on a tensile test specimen are viewed, they reveal hollow voids on the surface. It is possible to observe the inclusion responsible for nucleating the void.

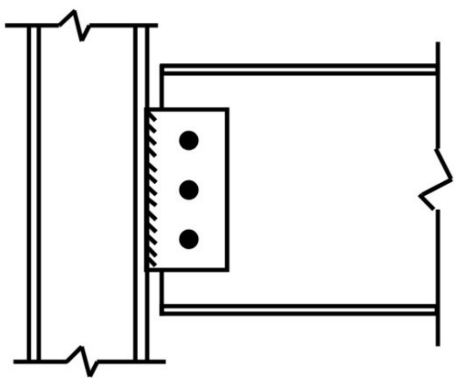
A.L. Gurson theorised in what is known as the Gurson model, a way to mathematically predict the behaviour of a material to ductile fracture. In the Gurson model, there exists a metallic matrix with spheres that grow once stress is applied until cohesion, followed by fracture. His model assumes spherical voids which in reality are sometimes elongated. This has resulted in modifications to his Porous Plasticity equations along the years to accommodate for the difference seen in experimental results.

3.2 Macro and Structural Scale Structure

Classical structural analysis in Ultimate Limit State, of beams, columns, slabs, trusses, connections etc. cover the scope of the macro and structural scale.

Analysis of progressive collapse on this scale is a combination of beam-column and discrete spring finite elements used to simulate the overall behaviour of the structure (Li, 2013, p.3). These types of models, commonly used in earthquake engineering, are known as Subassemblages. As an example we will look at a simple beam-column shear tab connection.

In a single plate shear connection subassemblage, transfer of forces is done through springs. Each spring can be assigned bolt properties or binding effects at the point where the beam bears against the column's flange. Binding springs are simple contact springs while bolt springs can be made to simulate the nonlinear behaviour of a plastically deformed bolt
(Li, 2013, p.14).



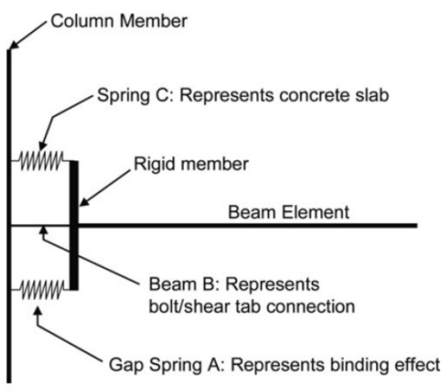


Figure 9. Simple shear tab and corresponding subassemblage (Khandelwal et al, 2008, p.15)

The resistance associated with flexural action is modelled by members A, B and C. A represents Binding effect from the contact of the beam and column, B represents the Bolt and shear tab connection and C represents the overlaid concrete slab. Spring A represents the contact condition that prevents the beam from penetrating the column's flange. It acts as a loaded spring pushing back on the force through Hook's law. Spring C has no resistance in tension and can crush in compression. Element B is a point with the same resistance as the bolts (Khandelwal et al, 2008, p.106).

According to Honghao Li's dissertation (2013), it is explained that the biggest line of defence against progressive collapse is catenary action. This mechanism also happens to be what shear connections are most vulnerable to.

Another example of a more straight-forward connection modelled is a simple moment connection with a reduced beam section (RBS). In the moment connection shown in Figure 10, pure shear deformations take place as is consistent with results obtained from experimental tests where shear stresses are distributed uniformly along the column web and the panel zone (column connection region) deforms in only shear as well (Li, 2013, p.16).

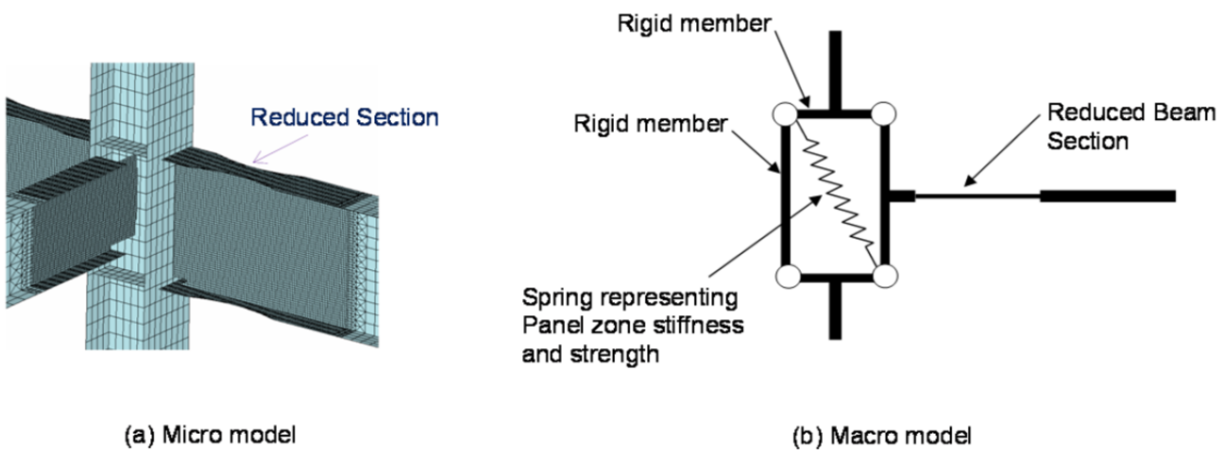


Figure 10. Subassemblage model of a simple moment connection (Khandelwal et al, 2008, p.16)

A sudden column failure will lead to a temporary loss of static equilibrium, which leads to acceleration of masses due to gravity. Elements exposed to an impact or explosion respond differently compared to the same magnitude of static loads due to a high strain rate. Uniform static loads have flexural mode failures while dynamic loads almost always result in shear failure modes. (Kjellman, 2017, p.8)

4 DESIGN FOR PROGRESSIVE COLLAPSE

For a structure to resist disproportionate collapse against a suddenly failed member, the structure must have adequate strength, ductility and stiffness. The structure must also be redundant enough for adequate load redistribution to members that still have capacity. After the collapse of port of Ramsgate (1994), it became mandatory in the UK to provide a catch ledge or chains to support hanging link bridges in case the main support failed (The Institute of Structural Engineers, 2010) discouraging single point of failure designs. This would provide alternate load paths for structural loads.

In addition to the above measures there needs to be mechanisms that contribute to a typical steel frame's ability to resist collapse. As highlighted by Khandelwal (2008, p.2):

1. Catenary action of beams and slabs allowing gravity loads to span adjacent elements
2. Frame action surrounding the location of damage
3. Support by non-structural elements such as in-fills and partition walls

Of all these mechanisms catenary action provides the last line of defence. Once this fails the structure is likely to continue collapsing. Steel structures are more susceptible to stability issues than concrete structures, and for pinned steel frames, the method of stability whether bracing or propping for global stability such as elevator shafts and shear walls, attention needs to be paid to make sure that it is robust enough. There are no specific EC3 requirements for progressive collapse in steel structures.

In summary, the fastest way of achieving robustness according to The Institute of Structural Engineers (2010), is to comply with EN 1991-1-7. The document summarises the criteria in the following building classes:

Class 1 - No additional measures. Other than designing for Ultimate Limit State and Serviceability Limit State
Class 2A - Provide effective horizontal ties or effective anchorage of suspended floors and roofs to walls.
Class 2B – Provide effective horizontal ties to floors and roofs plus effective vertical ties or apply notional column/wall removal or design as key element.

Structural form is the first step in attaining robustness. Structures should be designed such that there is no single point of failure as shown by the Port of Ramsgate failure. Single main girders and Central columns are to be avoided if possible in designs ensuring redundancy in the structure in

case member loss. If it is not possible to avoid single transfer beams in design, then other measures discussed below should be employed.

All structures should be designed such that they resist horizontal loads arising from wind, global imperfection from expected installation errors or member imperfections from column casting in concrete and manufacturing for steel. In seismically active zones, engineers design structures to withstand bigger horizontal loads and thus can be deemed fit to withstand accidental horizontal loading situations provided there is no loss of supporting members.

Many of the computational models for progressive collapse are actual direct extensions of models originally developed for earthquake engineering. The general consensus is that a building designed for seismically active regions is ductile enough to minimise progressive collapse. However, according to Khabdelwal et al. (2008, p.2) choosing a better layout of moment resisting frames is more influential than using strict seismic detailing to ensure good collapse resistance (Li, 2013, p.2). Research in the area has yet to show how seismic detailing protects against progressive collapse.

Modern design codes and verification methods fall short when protecting against progressive collapse. These design methods are a result of data obtained from observation and measurements and are represented by their probability density functions in the equations engineers use. Designs using these values provide a uniform level of safety. However, they do not protect against progressive collapse. The reason is because these methods focus on local failure as opposed to global failure. Design equations and safety checks such as checks for stresses, sectional forces and stability of load bearing elements, are usually applied on local levels only. How the global system behaves to progressive collapse can be different. (Starossek, 2009)

The probability, P(C), of progressive collapse, C, due to an event, E, is:

$$P(C) = P(C/L)*P(L/E)*P(E) \qquad \text{(Equation 4.1)}$$

Where P(E) is the probability of occurrence of (E), P(L/E) is the probability of local failure, L, given the occurrence of E, and P(C/L) represents the probability of progressive collapse given the occurrence of L. The factor P(C) is not reflected in the verification formulas. (Starossek, 2009)

There are four approaches (Chapter 4.1-4.4) to design as described by The Institute of Structural Engineers (2010). The approaches can be mixed as is often seen that columns are regarded as key elements.

4.1 **Indirect Design Method: Tying**

To add redundancy and increase continuity in precast structures, elements are usually connected by ties (Kjellman, 2017, p.9). Ties help constrain elements and limit displacement and can make possible the formation of alternate load paths by the help of catenary action and vierendeel action. Ties have been proved to provide robust structures preventing progressive collapse.

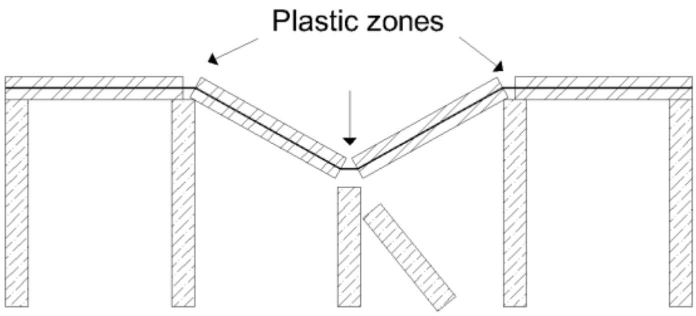


Figure 11. Function of the tying system (Kjellman,2017, p.9)

4.1.1 Horizontal Ties

Horizontal ties are generally split into two types i.e. ties around the structure are called Peripheral ties while ties found within the structure are called internal ties in the transverse or longitudinal directions.

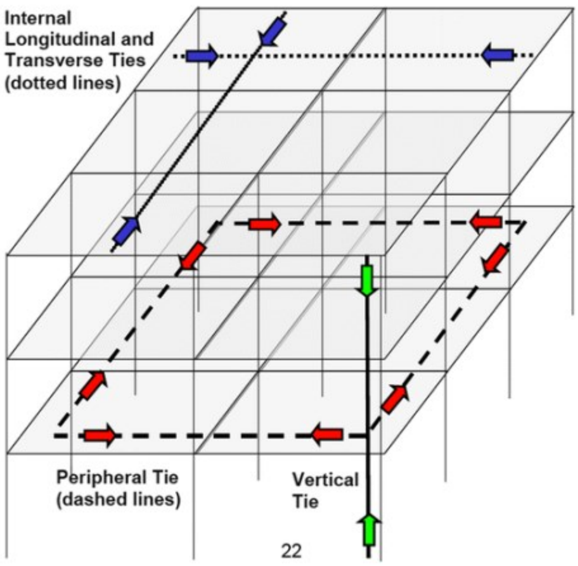


Figure 12. Types of ties (Kjellman,2017, p.10)

All tie force paths should be geometrically straight. Changes in tie direction to accommodate for openings or other types of discontinuities should be avoided where possible. It should be noted that ties tend to straighten under tensile force and that should be taken into account when designing. Ties also need to be continuous in nature either by lapping or connected across from edge to edge or around the structure. Horizontal ties are a regulatory requirement for Class 2A and Class 2B structures. In Class 1 Steel structures however, standards stipulate that columns should be tied in two directions at right angles to each other on the principal floor level. (The Institute of Structural Engineers, 2010). As highlighted by EN-1991-1-7, all perimeter ties should also be able to resist a factored tensile force the larger of (Kjellman, 2017):

$$T_p = 0.4(g_k + \psi q_k)a_1 L$$
$$T_p = 75\mu \text{ [kN]}$$

(Equation 4.2)

Where g_k is the charactaristic permanent load
q_k is the charactaristic live load
ψ_1 is the relevant load combination factor
a_1 is the distance to the inner tie
L is the distance presented in Figure 13 below
μ is a factor usually equal to 1

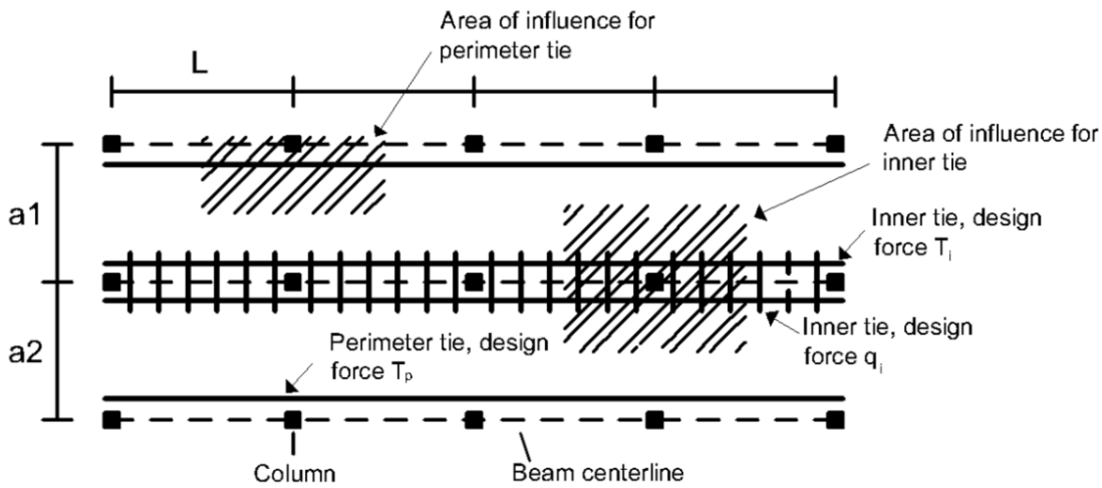


Figure 13. Design forces for horizontal ties (Kjellman, 2017, p.20)

Internal design tie force qi is the maximum of:

$$q_i = 0.8(g_k + \psi_1 q_k)a_m$$
$$q_i = 20\mu \quad [\mathrm{kN/m}]$$

(Equation 4.3)

Where

$$a_m = \frac{(a_1 + a_2)}{2}.$$

(Equation 4.4)

If as shown by Kjellman (2017) the ties are concentrated on beam lines as in Figure 13 above, then it must be able to hold a load that is the maximum of:

$$T_i = 0.8(g_k + \psi_1 q_k)a_m L$$
$$T_i = 75\mu \quad [\mathrm{kN/m}].$$

(Equation 4.5)

The distance L is column spacing centre-to-centre. Where columns are irregularly placed, the spacing that gives the maximum design load should be chosen.

Ties, however, create horizontal forces on the columns and if not accounted for in design might lead to pulling-in and eventual collapse of the surrounding columns. As described by Kjellman (2017, p.12), this resultant horizontal force is more detrimental to a loss of column near the edge as shown in the diagram below. The resultant force is carried entirely by the single line of columns on the left in one direction. To the right of the collapsed element, the horizontal force is carried by the adjacent column row and the other rows after that each participating in redistribution of the forces.

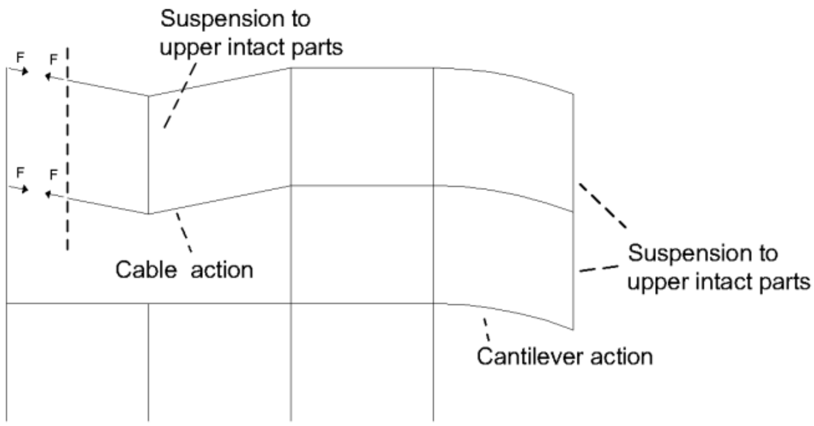


Figure 14. Redistribution of forces to remaining members after loss of an element (Kjellman, 2017, p.12)

When a corner column is lost, cantilever action redistributes the load if continuous beams are used in construction. If simply supported beams are used, redistribution of loads is achieved by cantilever action from horizontal ties placed on the top part of the beam. (Kjellman, 2017). The effect is shown in Figure 14 above.

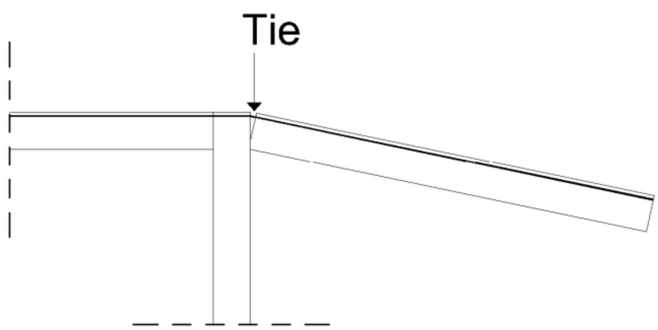


Figure 15. Cantilever action from ties (Kjellman, 2017, p.13)

In the case of steel, the most widely used system is where the steel frame provides stability to the structure while the flooring system, usually precast concrete or in situ, is laid on the beams. Eurocode provides recommendations to anchorage of floor systems to steel frames restricting dislodgment by upheaval as would be in an explosion or falling through when large enough forces displace members of the frame.

Guidelines require that stairs, precast concrete, floors and roofs be anchored in the direction of their span to each other over their support or directly to the support. Note that anchorage is only required in the direction of the span since steel beams already act as ties in the orthogonal direction. (The Institute of Structural Engineers, 2010)

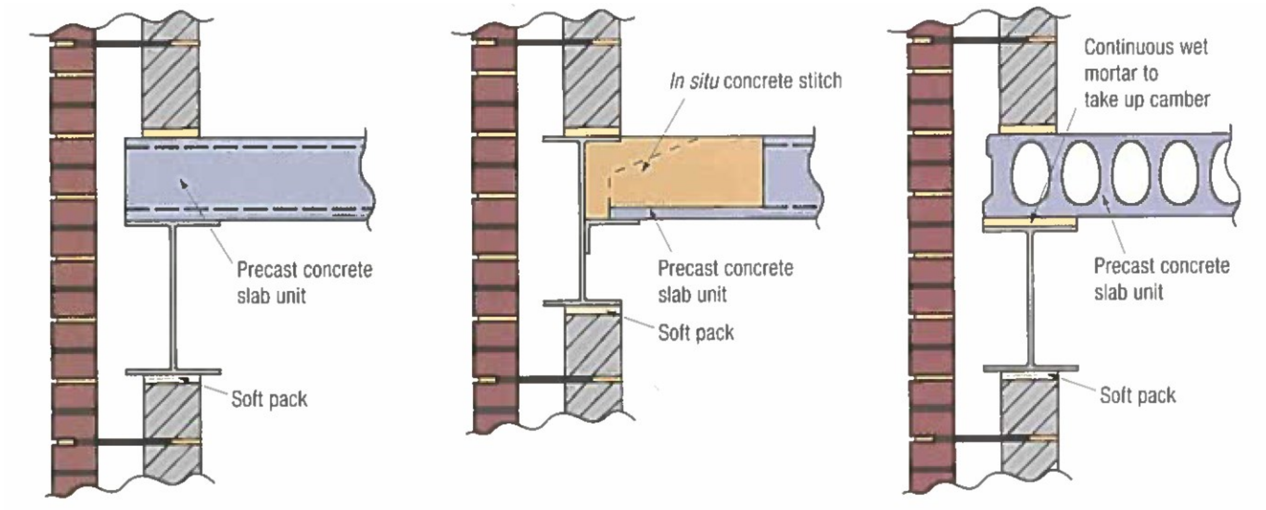


Figure 16: Anchorage of precast units to external steelwork by friction or embedment (The Institute of Structural Engineers, 2010)

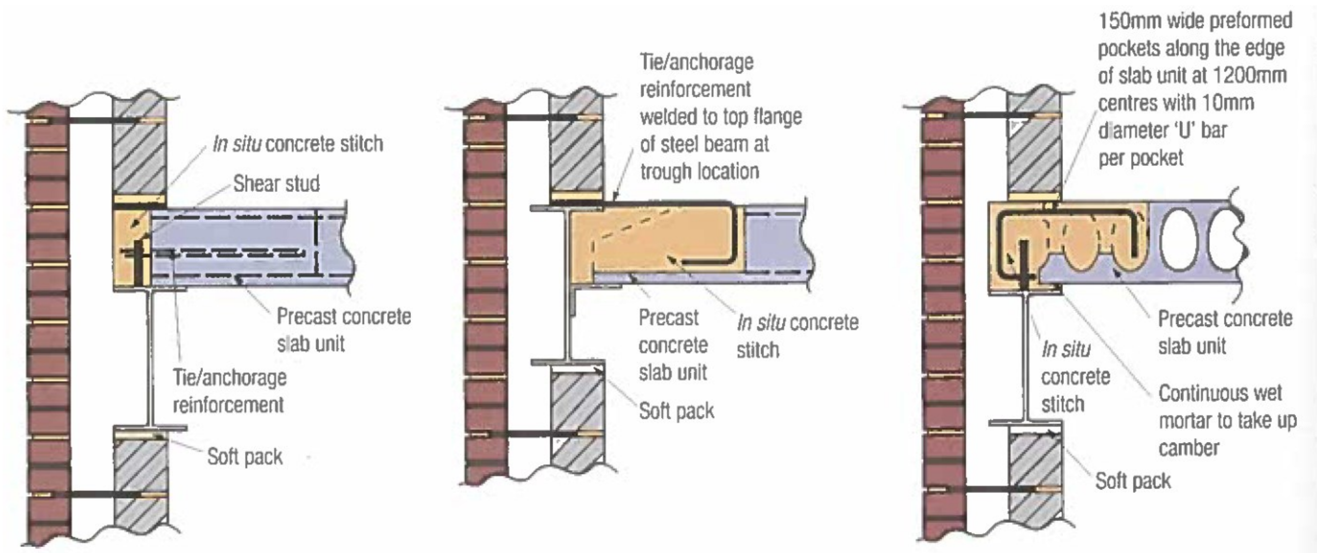


Figure 17. Anchorage of precast units to external steelwork by ties (The Institute of Structural Engineers, 2010)

4.1.2 Vertical Ties

The two roles vertical ties play are to provide support in case of vertical element removal (walls and columns) and to enable load sharing between the floors above a damaged vertical element by developing Vierendeel action.

The rule for vertical ties is that the column or wall should be able to support the largest load applied to the column or load above or below it. Vertical ties must also be continuous from anchorage of the footing to

the top floor. As explained by The Institute of Structural Engineers, (2010) this helps reduce the possibility of the lowest column removal.

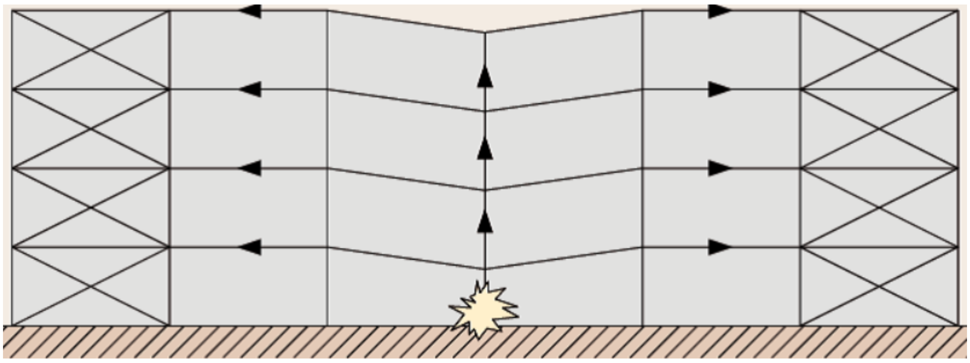


Figure 18. Centre vertical ties redistributing load to the top floors and outward (Steel Construction Info, 2012)

Vertical ties can be considered to be effective if,

1. The clear height of the wall, H, (measured in meters between faces of floors) does not exceed 20t, where t is the thickness of the wall in meters
2. If the ties are designed to carry vertical force T where;

$T = \frac{34\text{A}}{8000}(H/t)^2$ [Newtons] or 100kN/m of wall. Whichever of the two is greater.

(Equation 4.6)

Where A is the cross-section area in mm^2 of the wall measured on plan.

3. The vertical ties are grouped at 5m maximum centers along the wall and occur no greater than 2.5m from an unrestrained end of the wall.

4.2 Alternate Load Path Method: Notional Element Removal

Alternate Load Path method involves theoretically removing a column or part of wall from your designed structure and observing the behaviour afterwards. The structure must redistribute the loads effectively and remain stable. If the notional removal of any element would result in the

collapse of a greater area than allowed by Eurocode, the structural member should then be designed as a Key Element. EN-1991-1-7 specifies the elements that should be removed as:

1. Each supporting column and each beam that supports more than one column (transfer beams)
2. Any nominal length of loadbearing wall

The length of a concrete wall removed as specified by Eurocode is 2.25H, where H is storey height. In the case of cladding or timber walls, the area between vertical supports should be notionally removed. For corner walls the length of wall to be removed is equal to the height of wall. In the case of closely spaced columns, it is recommended that all columns within a plan diameter of 2.25H be removed simultaneously. The maximum area of collapse permitted when these elements are removed are described in section 2.3 above.

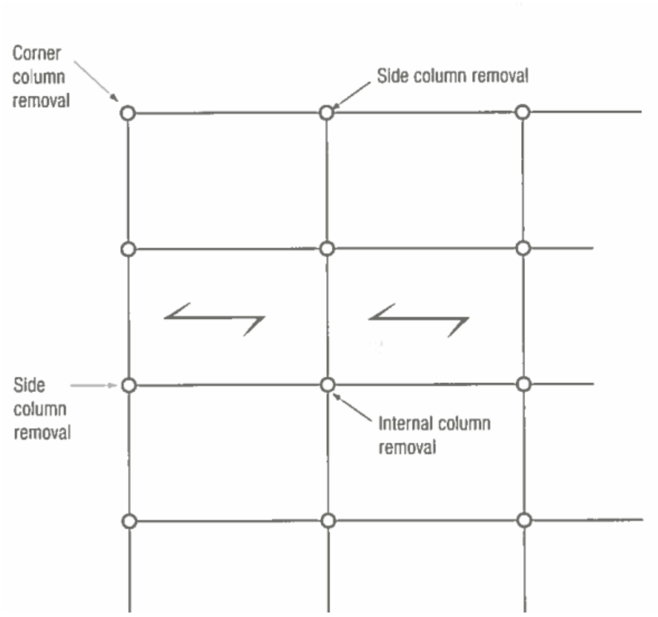


Figure 19. Location of stipulated column removal (The Institute of Structural Engineers, 2010)

Other design recommendations given are:

1. Spanning the floor slabs at right angles to their normal design case or utilising two-way slabs.
2. Making use of beam and slab reinforcement to ensure two bay spans.
3. Using the ability of a wall to cantilever as a deep beam over a notional opening.

In steel structures, the greatest amount of risk to initiate disproportionate collapse is the removal of the lateral stabilising system. If the building falls into Class 2A or 2B, the bracing system must also be

notionally removed, and the building be expected to stand. Redundancy ensures that there is a second bracing system in place to maintain stability. If the structure is stabilised using a steel moment resisting frame, then notional removal of each moment resisting joint should not result in an unreasonable degree of collapse. If the structure is stabilised by an internal concrete core, each storey segment of wall is notionally removed for the stability check. (The Institute of Structural Engineers, 2010)

4.3 Specific Load Resistance Method: Key Element

In Key Element method, members that are important in maintaining the stability of a structure are designed to withstand greater loads than prescribed by normal load combinations. These elements are regarded as key elements. Examples of key elements are transfer beams as shown in Figure 20 below or a column supporting a transfer beam.

As described by Steel constructions Info (2012), the key element must be able to sustain accidental design loading from any direction, in each orthogonal direction at a time, to any connection and component of the assembly. The prescribed accidental loading value for key elements is 34kN/m^2. This value was derived from the Ronan point collapse explosion.

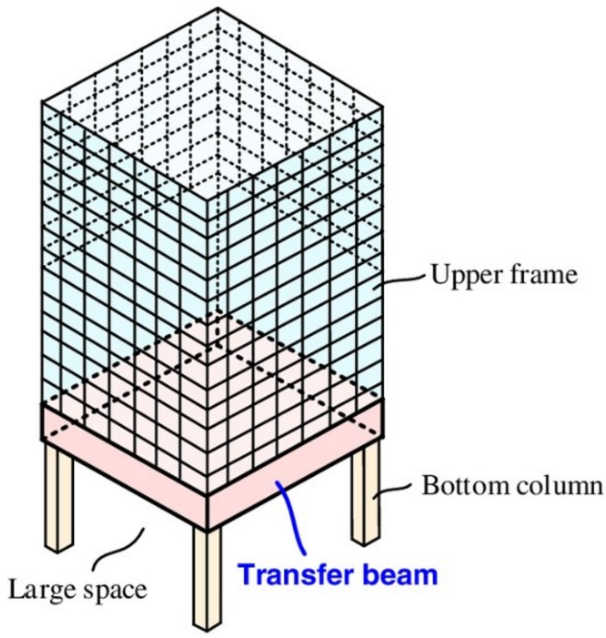


Figure 20. An example of a key element, transfer beam (ASCE Library, 2017)

It must be remembered that the relationship between accidental actions or any action is not linear to its effect. The general format for accidental actions combinations according to EN-1990 should be:

$$E_d = E\{G_{k,j} \,;\, P \,;\, A_d \,;\, (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} \,;\, \psi_{2,i}Q_{k,i}\} \quad j \geq 1 \,;\, i > 1$$

(Equation 4.7)

The combination of actions in brackets { } can be expressed as:

$$\sum_{j\geq 1} G_{k,j}\text{"+"}P\text{"+"}A_d\text{"+"}(\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}Q_{k,i}$$

(Equation 4.8)

The choice between $\psi_{1,1}Q_{k,1}$ and $\psi_{2,1}Q_{k,1}$ should be related to the relevant accidental design situation as highlighted by Eurocode. Combinations of actions for accidental design situations should define an explicit accidental action, A, called “impact” in Eurocode. Impact is described in EN-1991-1-7 in detail.

4.4 **Risk Assessment Method**

Risk management involves systematic measures undertaken in order to maintain a level of safety. The risk assessment method is applied to class 3 buildings as described by Eurocode. The purpose of risk assessment is to determine whether there are any hazard scenarios that have an acceptable level of risk, and if so, suggestions of their mitigations are proposed (Steel Constructions Info, 2012). A class 3 building is any kind of building in which members of the public are admitted in significant numbers, for example stadia able to accommodate 5000 people fall into this building class. Buildings that house hazardous substances or process them also fall into this category. EN 1991-1-7 Annex B outlines the procedures undertaken in risk assessment. The flow chart given Figure 21 below shows the steps undertaken in assessment.

Risk assessment is mainly qualitative, meaning it is descriptive in nature. It, however, can also be numerically quantitative where relevant. Examples of risk mitigation once identified include elimination or reduction of the hazard, controlling the hazard by providing warning systems and checks, overcoming the hazard by increasing reserve strength or robustness, or even permitting controlled collapse of a structure where the probability of injury may be reduced.

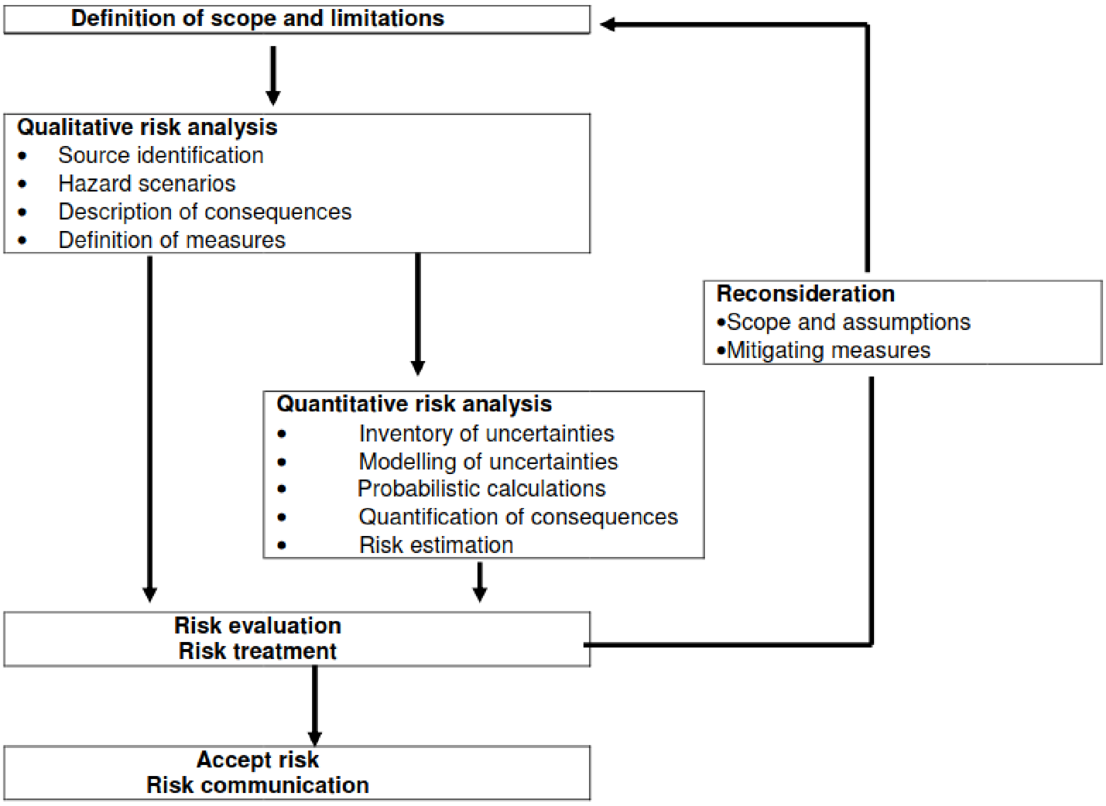


Figure 21. Overview of risk analysis (EN 1991-1-7, Annex B)

4.5 Segmentation

It is not in all cases that the prevention of maximum damaged is achieved by making the structure as robust as possible and establishing alternative load paths. In some cases, for example bridge design, the protection of the structure from total collapse during a progressive collapse event can be achieved by segmentation.

In bridges such as The Confederation Bridge that crosses the Northumberland Strait in Canada, an initial local failure could lead to total damage of the bridge. In its design, a sudden failure of the main bridge pier was assumed. Isolated collapsing sections were chosen to limit any initial failure thus protecting the entire structure from collapse. (Starossek, 2009) The partial collapse of Charles de Gaulle Airport Terminal 2E was initiated by the failure of a portion of the roof. The collapse of the structure came to a halt at an unintentionally at a joint that isolated that section of the structure. (Starossek, 2009) Alternative load paths as pointed out by Starossek (2009) require an increase in continuity and structural resistance. Segmentation on the other hand is achieved by less continuity and more resistance.

5 ANALYSIS METHODS IN FRAMED STRUCTURES

When analysing for progressive collapse there are four approaches at increasing levels of complexity as highlighted by the General Services Administration (GSA, 2003). They are linear Static, Nonlinear Static, Linear Dynamic and Nonlinear Dynamic analysis. The more complex the method of analysis is the more accurate the results tend to be. This however comes with a cost of time and computing power which can result in longer design times. However due to Moore's Law that states every 24 months, computing power doubles, this fact will become less of a problem as computing power becomes more accessible and affordable.

The lack of experimental results of progressive collapses suggests an approach based to developing models and simulating the effects (Masoero et al, 2015, p.2). Collapse models can be 2-D or 3-D, where 3-D models give behaviour that more closely resembles reality than planar models do. Linear models are simpler to analyse and develop, but do not mimic the actual progression of collapse, so when computing power and time resources are available, nonlinear models should be favoured (Li, 2013). Below will be discussed the four approaches in more detail as highlighted by Janssens & O'Dwyer (2010).

5.1 Linear Static Analysis

Linear static analysis is the simplest form of analysis that involves applying a factored gravity load to a damaged structure with the proceeding analysis being based on the assumption of small deformations. Dynamic effects are indirectly considered by assuming equivalent static loads based on a constant amplification factor equal to 2.0 (GSA, 2003) The analysis being static implies that it is not dependent on time. The loads and constraints do not change value or direction. The geometry of the structure does not change during analysis which implies that catenary and membrane effects cannot be taken into account.

A solver, which is a mathematical algorithm run by software, used in linear static analysis is a simple derivation of Hook's law. You provide the loading data, constraints, and geometry and material data. The solver uses this information to generate the stiffness matrix and boundary conditions that will help solve for the system. The unknown is the strain which leads to displacement in the model (Cyprien, 2017). In this analysis, actions are divided into force-controlled or deformation-controlled. An example of which is a moment resistant frame, the moment would be defined as a deformation-controlled action while shear and axial force are force-controlled actions as further explained in section 5.4. Load combinations for the different actions are (Kjellman, 2017):

$$G_{LD} = \Omega_{LD}(1.2D + (0.5L \textbf{ or } 0.2S))$$
$$G_{LF} = \Omega_{LF}(1.2D + (0.5L \textbf{ or } 0.2S))$$
$$G = 1.2D + (0.5L \textbf{ or } 0.2S)$$

(Equation 5.1)

Where:

G_{LD} is increased gravity loads for deformation controlled actions
G_{LF} is increased gravity loads for force controlled actions
G is gravity load
D is dead load
L is Live load
S is snow load
Ω_{LD} is dynamic load factor for deformation controlled actions
Ω_{LF} is dynamic load factor for force controlled actions

G_{LD} and G_{LF} are applied only to the areas shown in Figure 22. The rest of the structure is loaded with G according to the equation above. Also, as shown by Kjellman (2017), the magnitude of the dynamic load factors Ω_{LD} and Ω_{LF} is dependent on the material and type of structural element and is between the value 1 and 2.

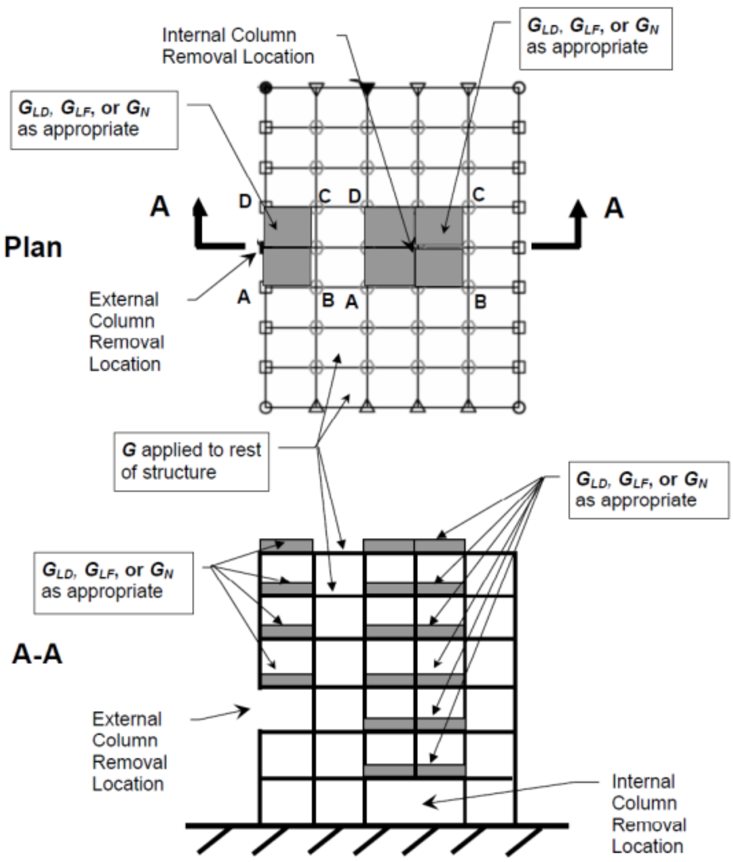


Figure 22. Application of dynamic load factor in linear static analysis (UFC 3-2.11.5)

5.2 **Nonlinear Static Analysis**

Nonlinear static analysis improves on linear static by including both geometric and material nonlinearities in the analysis. The inclusion of this is needed to account for catenary and membrane action as well as to allow for an accurate representation of inelastic response and P-Δ effects. This analysis also includes a dynamic amplification factor to account for time-dependent effects. Geometry change with time is also accounted for. In this analysis however, the load is not applied once as in linear static, instead loads are applied incrementally until maximum loads are attained or collapse occurs, this improves accuracy in the results. (Janssens & O'Dwyer, 2010, p.6). Both primary and secondary elements are included but the stiffness of secondary elements is set to zero. If secondary elements are not included in the model they must be checked separately if they can withstand the large deformations induced when loading. Load combinations are considered in linear static analysis but with a dynamic load factor included (Kjellman, 2017, p.27). All structural elements and components that provide capacity to the structure to resist collapse are primary elements. All others are defined as secondary elements.

$$G_N = \Omega_N(1.2D + (0.5L \textbf{ or } 0.2S))$$

(Equation 5.2)

Where:

G_N is increased gravity load
D is dead load
L is Live load
S is snow load
Ω_N is dynamic load factor also called the dynamic increase factor

The magnitude of dynamic load factor Ω_N is dependent on the ductility of structural elements. A factor of 2 is used if the structure should remain elastic; it is less than 2 if plasticity is allowed for. (Kjellman, 2017, p.28). Results have also shown that a constant dynamic load factor of 2 applied in nonlinear static analysis leads to inconsistencies in results from nonlinear dynamic analysis. (Zhu et al., 2018, p.209)

After vigorous simulated tests an equation for determining the dynamic load of steel structures was arrived on (Marchand et al., 2014)

$$\Omega_N = 1.08 + \frac{0.76}{\frac{\theta_{pra}}{\theta_y} + 0.83}$$

(Equation 5.3)

Where:

θ_{pra} is allowable plastic rotation angle
θ_y is rotation at which the section yields

A graphical representation of the dynamic increase factor from simulated experimental data obtained by McKay et al (2017). Figure 23 below shows the relationship between Ω_N and the ratio between θ_{pra} and θ_y. When the θ_{pra} - θ_y ratio approaches zero for low ductility elements, the dynamic increase factor value reaches 2 (Kjellman, 2017, p.28).

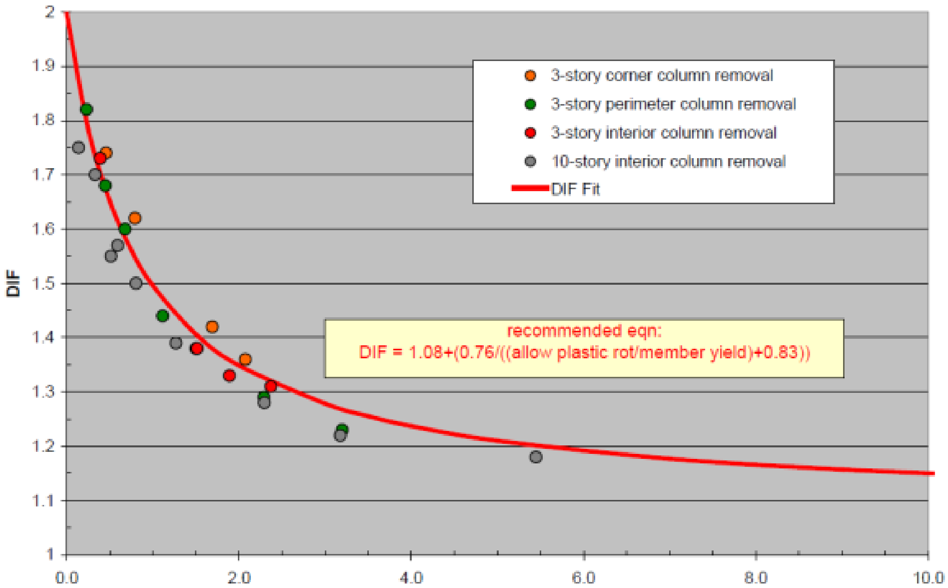


Figure 23. Dynamic Increase Factor as a function of θ_{pra} / θ_y in steel structures (McKay et al, 2017)

In conventional analysis it is assumed that beam-column connections are perfectly rigid or perfectly pinned for simplicity. Shown in Figure 24 is the range of rigid to flexible connection types in steel structures.

Connection deformation contributes a substantial proportion to overall deflection of the structure and has a non-dismissible bearing on the internal force redistribution. The basic joint types are rigid, semi-rigid and pinned. As a rule of thumb in basic beam-column connections, the stiffer the connection, the shorter the equivalent reference length of the beam (Zhu et al., 2018, p.211). Figure 25 below shows the moment rotation curves of different connection types starting from the most ductile to ideally rigid.

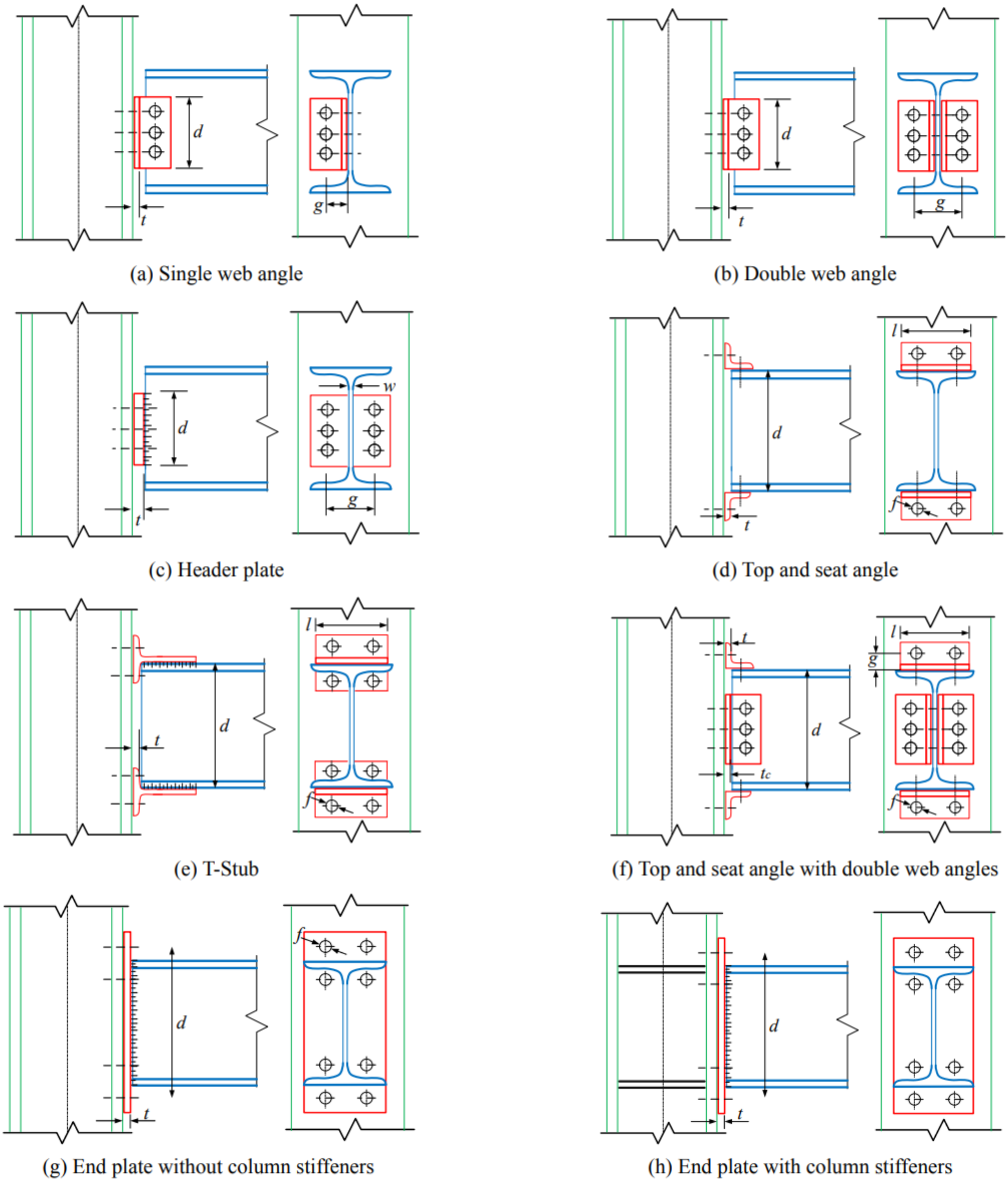


Figure 24. Types of beam-column connections (Zhu et al., 2018, p.211)

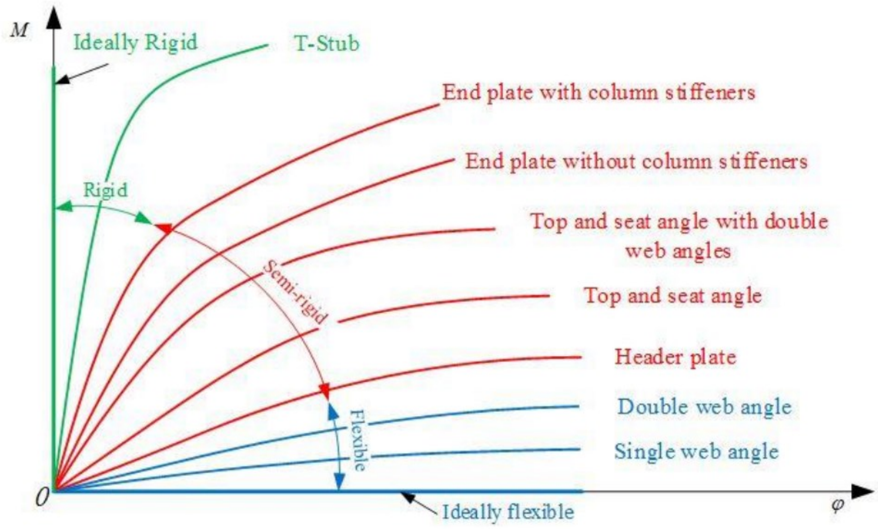
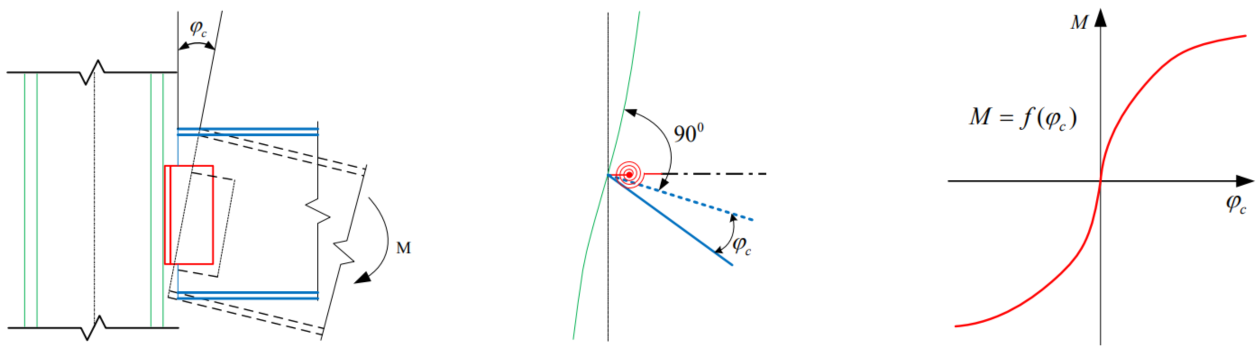


Figure 25. Connection moment-rotation curves (Zhu et al., 2018, p.211)



(a) The rotational deformation of a connection

(b) Simplified semi-rigid connection model

(c) Moment-rotation relationship of the nonlinear spring element

Figure 26. Design moment-rotation characteristics (Zhu et al., 2018, p.211)

5.3 Linear Dynamic Analysis

In linear dynamic analysis the notional removal of an element results in a sudden change in geometry. This results in changes in internal strain energy and kinetic energy of the structure. Many structures in real-life situations are not loaded statically. Building towers are subjected to wind which changes in intensity with time, airplane wings are subjected to ever-changing loading conditions in flight and offshore drilling platforms are subjected to water turbulence. As a result, the structures are in a constant state of motion due to the dynamic effects. This is solved with linear dynamic analysis. In a dynamic analysis, in addition to structural elasticity force, structural inertia and dissipative forces (or damping) are also considered in the equation of motion to equilibrate the dynamic

forces. (Sharcnet.ca, n.d). This method of analysis is, however, unable to account for any nonlinear behaviour associated with collapse (Janssens & O'Dwyer, 2010 p.6).

5.4 Nonlinear Dynamic Analysis

Nonlinear dynamic method of analysis allows for large deformations and energy dissipation through material yielding, fracture and cracking. Plastic hinges are allowed for. This method also takes into account energy from failed members. Failed members fall onto other structural elements causing forces that may lead to further collapse (Janssens & O'Dwyer, 2010, p.6). The procedure and equation used is similar to that of nonlinear static analysis. Since dynamic loading takes place in this approach, dynamic increase factor of 1 is used, $\Omega_N = 1$. The time period undertaken in the analysis is the period from initial static stability to maximum vertical displacement or a full cycle of vertical motion has occurred (Kjellman, 2017, p.30). Inclusion of secondary components is optional, but if omitted, should be verified separately for capacity against large deformations.

Columns under high axial loads of $P/P_{cl} > 0.5$ shall be considered force controlled. Classification of all actions as being force-controlled or deformation-controlled according to the curves shown in Figure 27 should be done. Separate structural models are required in verification of either deformation-controlled actions or force-controlled actions as mentioned in section 5.1. (UFC 3-2.5, 2013)

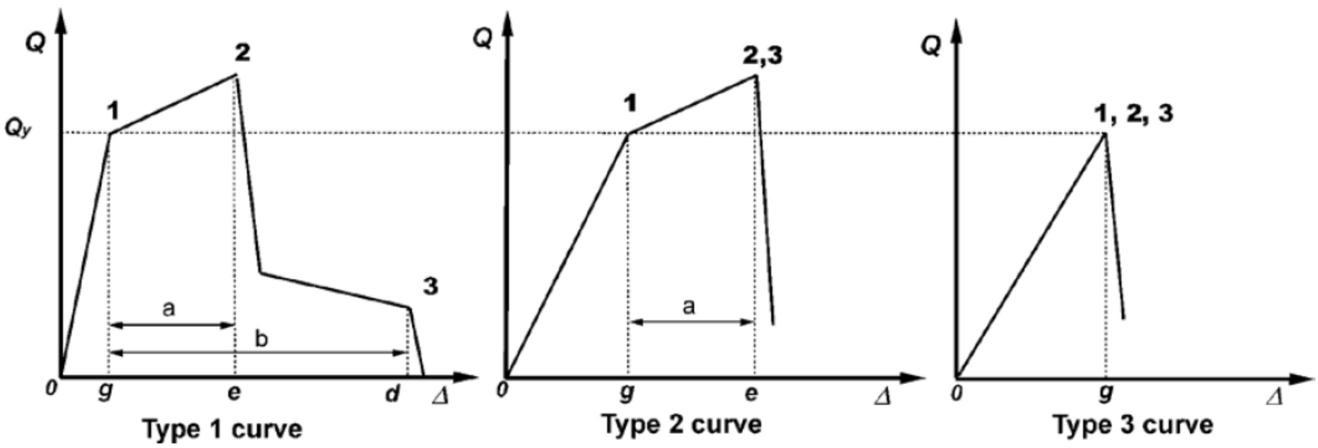


Figure 27. Component force vs Deformation curves (UFC 3-2.5)

A component might have both force- or deformation-controlled actions but it is not to up to the designer's discretion, the information in Table 3 below is used to specify the category. (UFC 3-2.5, 2013)

Primary component action is defined as deformation-controlled with a Type 1 or Type 2 curve and e ≥ 2g. Define a primary component as force-controlled if it is a Type 1 or Type 2 curve and e < 2g.

Secondary component action is defined as deformation-controlled that has a Type 1 curve of any e/g ratio or if Type 2 curve and e ≥ 2g. Define a secondary component as force-controlled if it is a Type 1 curve and e < 2g or Type 3 curve. (UFC 3-2.5, 2013)

Table 3. Deformation-controlled and Force-controlled actions examples

Component	Deformation-Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

6 CASE STUDY: MODELLING AND ANALYSIS

6.1 Existing Building Description

Geometry based on Finavia Helsinki Airport, Terminal 2 (Vantaa, Finland) was chosen as the model of this case study. At the time of this writing, Pöyry Finland currently has a number of different design projects (both renovation and new structures) in progress for Finavia. Blueprints for the case study were provided by Pöyry Finland. Finavia is a state-owned company whose core business is maintenance and development of 21 airports across the country. Figure 28 below shows the location of the area under consideration (marked in red) whose geometry will be covered in this case study.

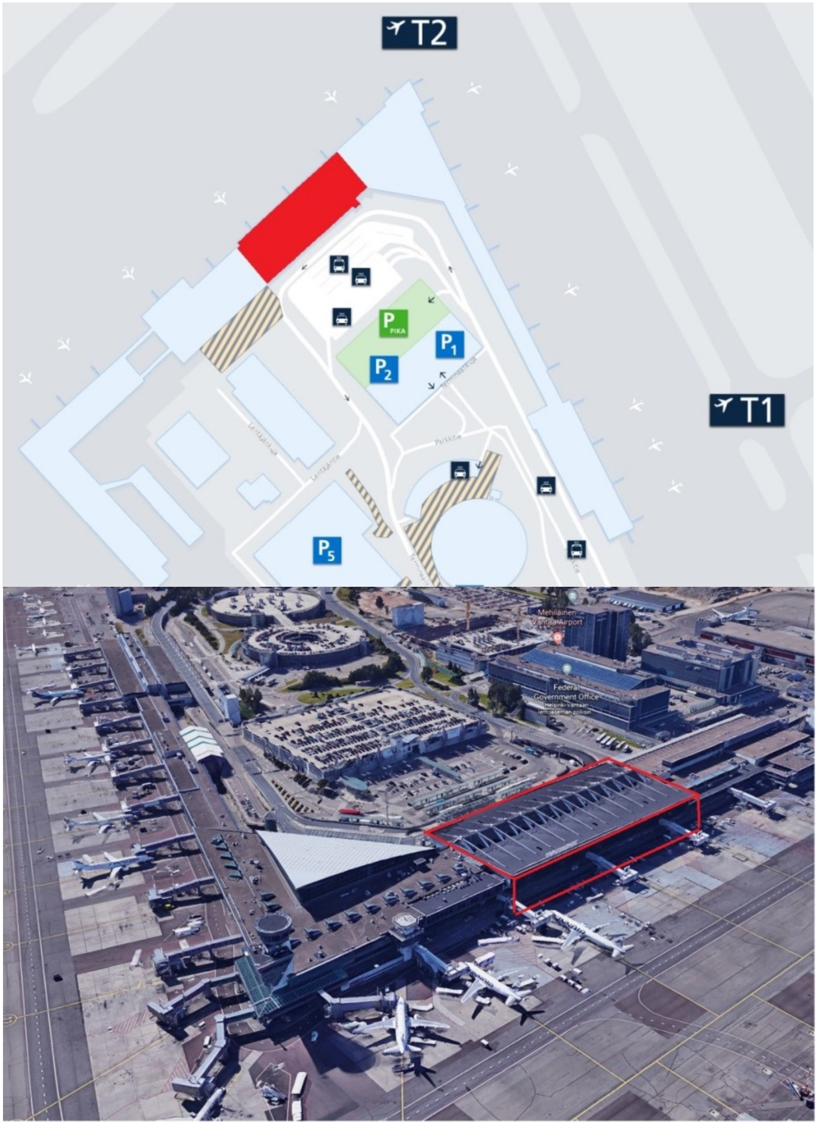


Figure 28. Terminal 2 focus area (Finavia & Google images)

Terminal 2, commonly referred to as T2, is the airport's international arrivals and departure section. This section of the terminal is a concrete structure with large open spaces with 70mm ribbed slabs on the 3rd floor and 160mm slabs on the 1st and 2nd floor. Its roof is cable suspended from 13 masts. It is designed this way to accommodate for large spaces with no columns for crowds of people waiting to board their flights. This design also allows for unobstructed natural light from the large windows.

Since the focus of this case study is on steel structures, the existing structure will be replaced with a robust steel structure and concrete floor system. The structure shall then be analysed using Robot Structural Analysis version 2018 software, according to prescribed standards and norms on progressive collapse.

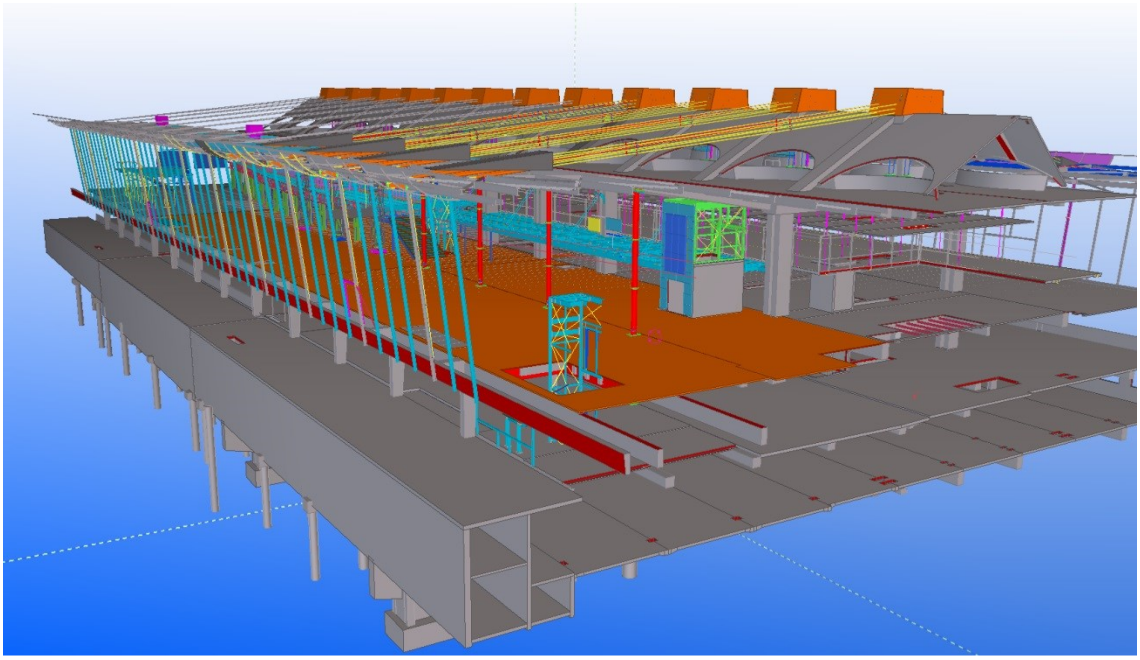


Figure 29. Tekla structures model of the real structure

The foundations rest on bedrock with foundation columns rising from anchored bedrock. Lateral stability against sway is provided by the elevator shaft and a shear wall. There is no bracing system except for minor steel structures within the building. The building has three main floors. Two expansion joints separate the structure into three sections. The roof and snow loads are entirely carried by a series of steel cables that lead to the masts. The studied section of the building is a *cast-in-situ* structure built originally in the 1970s and expanded in the 1980s.

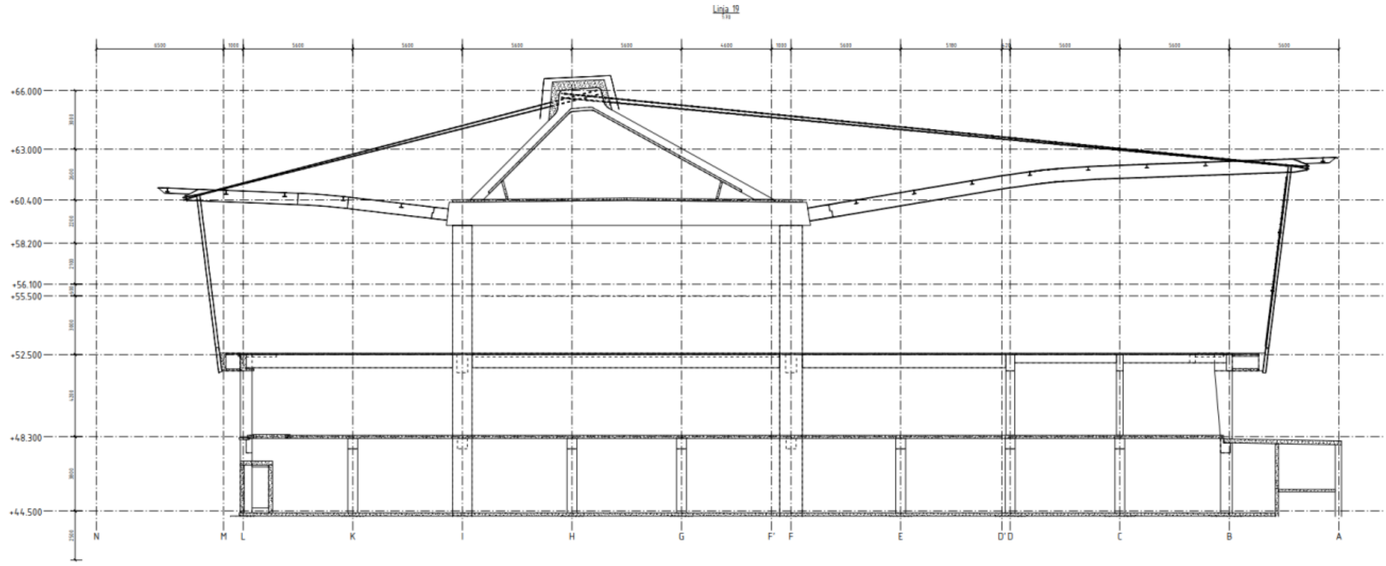


Figure 30. Cross-section of the structure

6.2 **Model Description and Process**

The aim of this thesis was to study progressive collapse in multi-storey steel structures. Therefore, the building in this study despite being a concrete structure was first converted to a robust steel structure in the first phase of this thesis and verified. The second phase involved analysing the structure against progressive collapse. The geometry of the building was followed with utmost strictness.

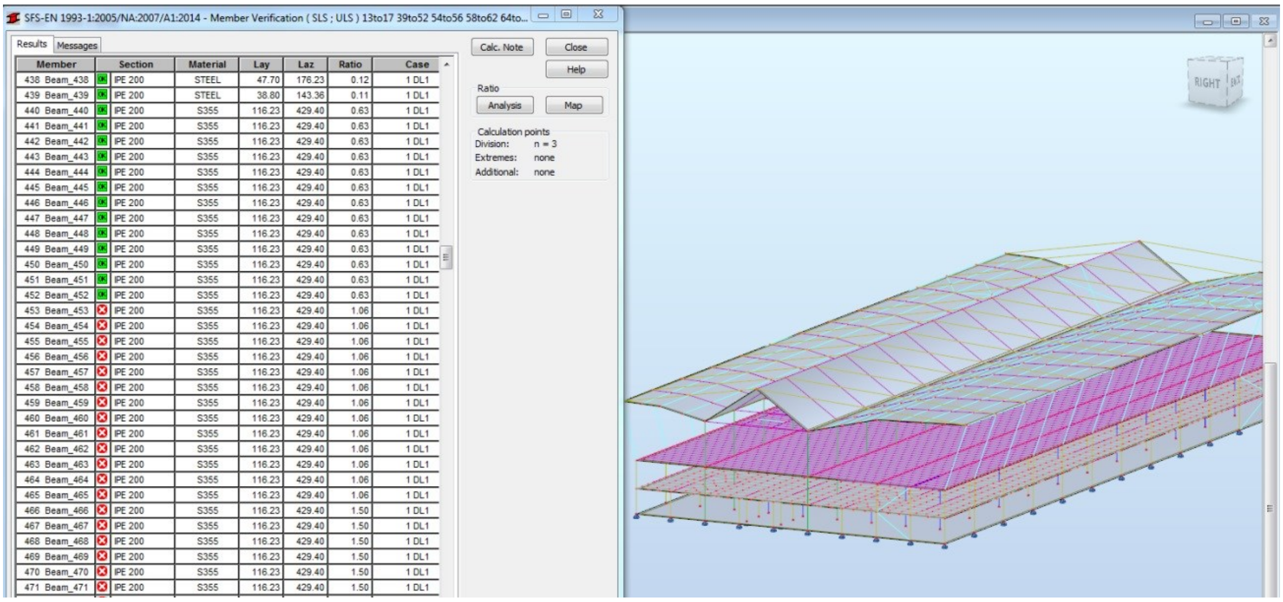
Assumptions made in the analysis model were:

1. The elevator shaft is not the primary source of lateral stability. A steel bracing system was introduced to provide lateral stability
2. One cable of cross-sectional size 65mm^2 was used for every mast as opposed to the combination of cables and steel rods used in the original building as designed by Leonhardt-Bauer Fritz in Berlin 1964. Cable pre-tension was set to 100kN
3. Deformation joints were not covered in the scope of this thesis.
4. Lateral stability arising from floor slabs on the 2nd floor, 3rd floor, and roof were omitted.
5. Supports were assumed to be pinned.
6. Lateral stability from the glass panes in the original structure was omitted.
7. Hanging floors in the centre of the 3rd floor were omitted.

The workflow chosen was to first choose steel sections most compatible with a building of this size and then verify their capacity under given loads. When found to be lacking, section profiles were increased. Loads were added progressively from dead loads to accidental combinations as prescribed by Eurocode.

6.2.1 Loading cases

Dead loads: Dead loads of the structure are added automatically by the program. Figure 31 shows the model built to the point of adding dead loads. It also shows a section of the report generated. Some beams and braces did not meet the needed capacity (marked in red).



Member	Section	Material	Lay	Laz	Ratio	Case
438 Beam_438	IPE 200	STEEL	47.70	176.23	0.12	1 DL1
439 Beam_439	IPE 200	STEEL	38.80	143.36	0.11	1 DL1
440 Beam_440	IPE 200	S355	116.23	429.40	0.63	1 DL1
441 Beam_441	IPE 200	S355	116.23	429.40	0.63	1 DL1
442 Beam_442	IPE 200	S355	116.23	429.40	0.63	1 DL1
443 Beam_443	IPE 200	S355	116.23	429.40	0.63	1 DL1
444 Beam_444	IPE 200	S355	116.23	429.40	0.63	1 DL1
445 Beam_445	IPE 200	S355	116.23	429.40	0.63	1 DL1
446 Beam_446	IPE 200	S355	116.23	429.40	0.63	1 DL1
447 Beam_447	IPE 200	S355	116.23	429.40	0.63	1 DL1
448 Beam_448	IPE 200	S355	116.23	429.40	0.63	1 DL1
449 Beam_449	IPE 200	S355	116.23	429.40	0.63	1 DL1
450 Beam_450	IPE 200	S355	116.23	429.40	0.63	1 DL1
451 Beam_451	IPE 200	S355	116.23	429.40	0.63	1 DL1
452 Beam_452	IPE 200	S355	116.23	429.40	0.63	1 DL1
453 Beam_453	IPE 200	S355	116.23	429.40	1.06	1 DL1
454 Beam_454	IPE 200	S355	116.23	429.40	1.06	1 DL1
455 Beam_455	IPE 200	S355	116.23	429.40	1.06	1 DL1
456 Beam_456	IPE 200	S355	116.23	429.40	1.06	1 DL1
457 Beam_457	IPE 200	S355	116.23	429.40	1.06	1 DL1
458 Beam_458	IPE 200	S355	116.23	429.40	1.06	1 DL1
459 Beam_459	IPE 200	S355	116.23	429.40	1.06	1 DL1
460 Beam_460	IPE 200	S355	116.23	429.40	1.06	1 DL1
461 Beam_461	IPE 200	S355	116.23	429.40	1.06	1 DL1
462 Beam_462	IPE 200	S355	116.23	429.40	1.06	1 DL1
463 Beam_463	IPE 200	S355	116.23	429.40	1.06	1 DL1
464 Beam_464	IPE 200	S355	116.23	429.40	1.06	1 DL1
465 Beam_465	IPE 200	S355	116.23	429.40	1.06	1 DL1
466 Beam_466	IPE 200	S355	116.23	429.40	1.50	1 DL1
467 Beam_467	IPE 200	S355	116.23	429.40	1.50	1 DL1
468 Beam_468	IPE 200	S355	116.23	429.40	1.50	1 DL1
469 Beam_469	IPE 200	S355	116.23	429.40	1.50	1 DL1
470 Beam_470	IPE 200	S355	116.23	429.40	1.50	1 DL1
471 Beam_471	IPE 200	S355	116.23	429.40	1.50	1 DL1

Figure 31. Building of the analysis model

An example of member 453 shown in Figure 32 below shows that the member failed by a margin of 6% on the stability check.

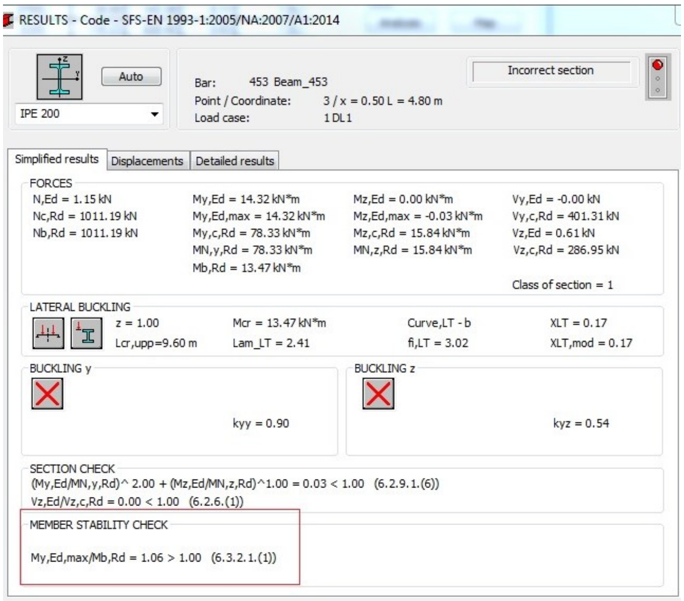


Figure 32. Failed member

All the failed members were replaced with the correct sections and analysed again until capacity was reached.

An occurring error was “Instability type 3” nodal rotational and translational error as shown in Figure 33 below.

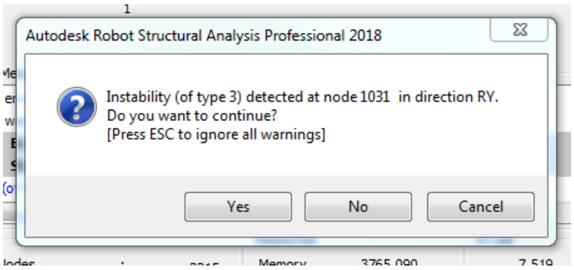


Figure 33. Type 3 error

When the model was tested for deformations, they were seen to be under normal limits. Further investigation in Autodesk Robot forums showed that if the model behaves as expected, type 3 errors should be ignored.

Snow Loads, Live Loads and Imposed loads: When analysed for snow loads, the structure gave a maximum deformation of 1117cm thereby failing the test. Affected sections found on the roof were replaced with more appropriate ones. Figure 34 shows the results. When the IPE 100 bars were replaced with IPE 300, the maximum deformation found was 30cm at the cables.

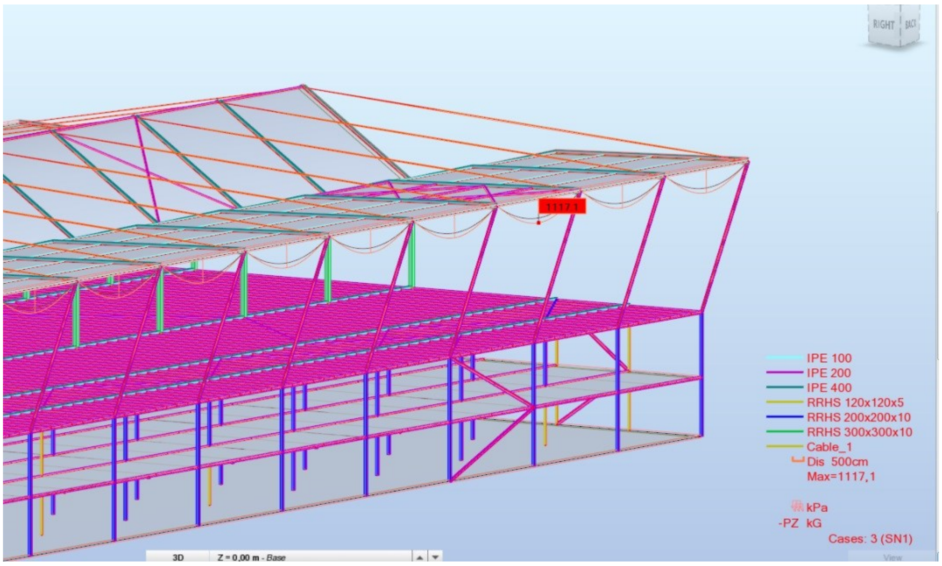


Figure 34. Deformation results

Wind Loads: Cladding panels were afterwards added to act as a surface for wind force. Claddings instead of concrete panels (with thickness) were chosen specifically for the reason that they offer no stability to the structure. The floor on the second floor was modelled as a concrete panel so as to attach the column row on module line B, D, E and G; otherwise the rows were regarded as separate structures.

Wind simulations pressure maps were generated on the panel surfaces. Figuere 37 shows the loading results for wind case 6. The existing building is not wind loaded axis +X and –X because of neighbouring structures.

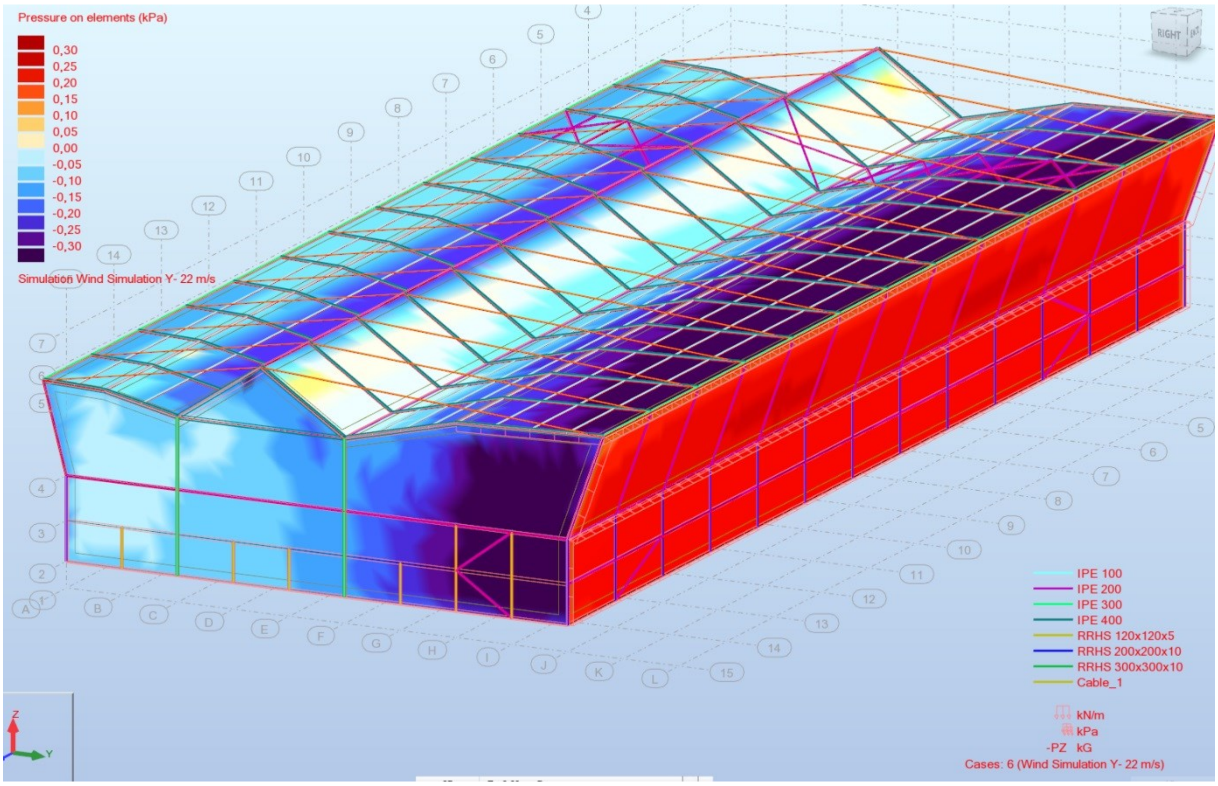


Figure 35. Wind pressure map on the structure

Parameters used were wind velocity of $22m^{2}$ and after a deviation greater than 5%, a new load case to be generated. For load case 6, which is wind in the –Y axis as shown above, a maximum pressure of 0.3kPa was generated. This created deformations of up to 317 cm centre 3rd floor beams and failing limit state design as shown in Figure 36. Member verification showed that most of the failed members were the bracings. They were subsequently replaced with stronger sections and the structure recalculated, passing the test. As a note, the bracing IPE 200 were replaced with IPE 400 sections. Roof cables also sagged with an 80cm displacement. Their pre-tensioning was increased from 150kN to 230kN. In the real concrete structure, the pre-tensioning on the cables is set to *Σ=300kN*.

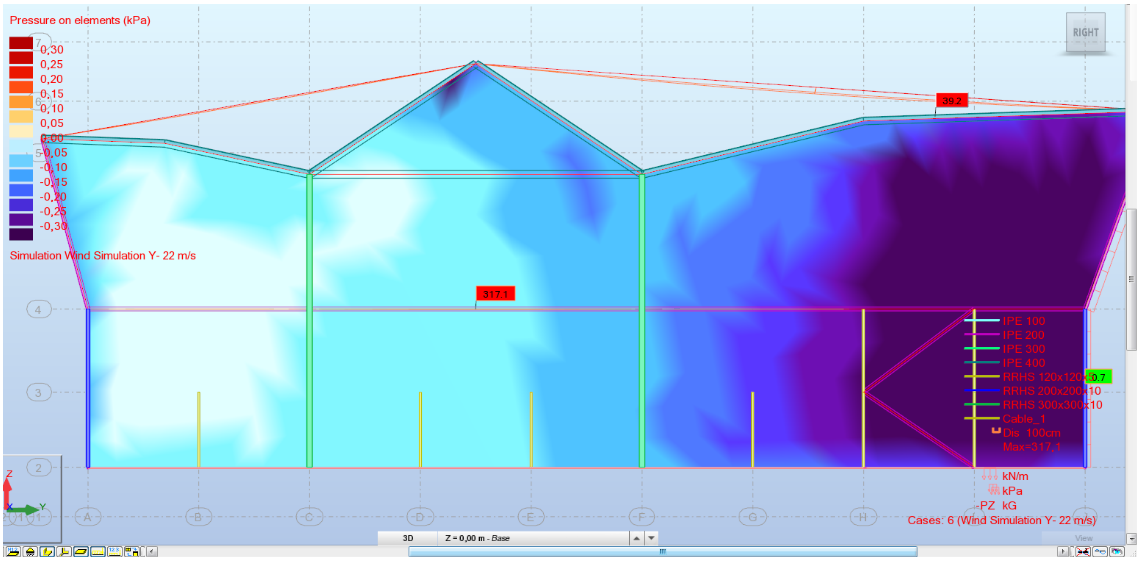


Figure 36. Limit state failure

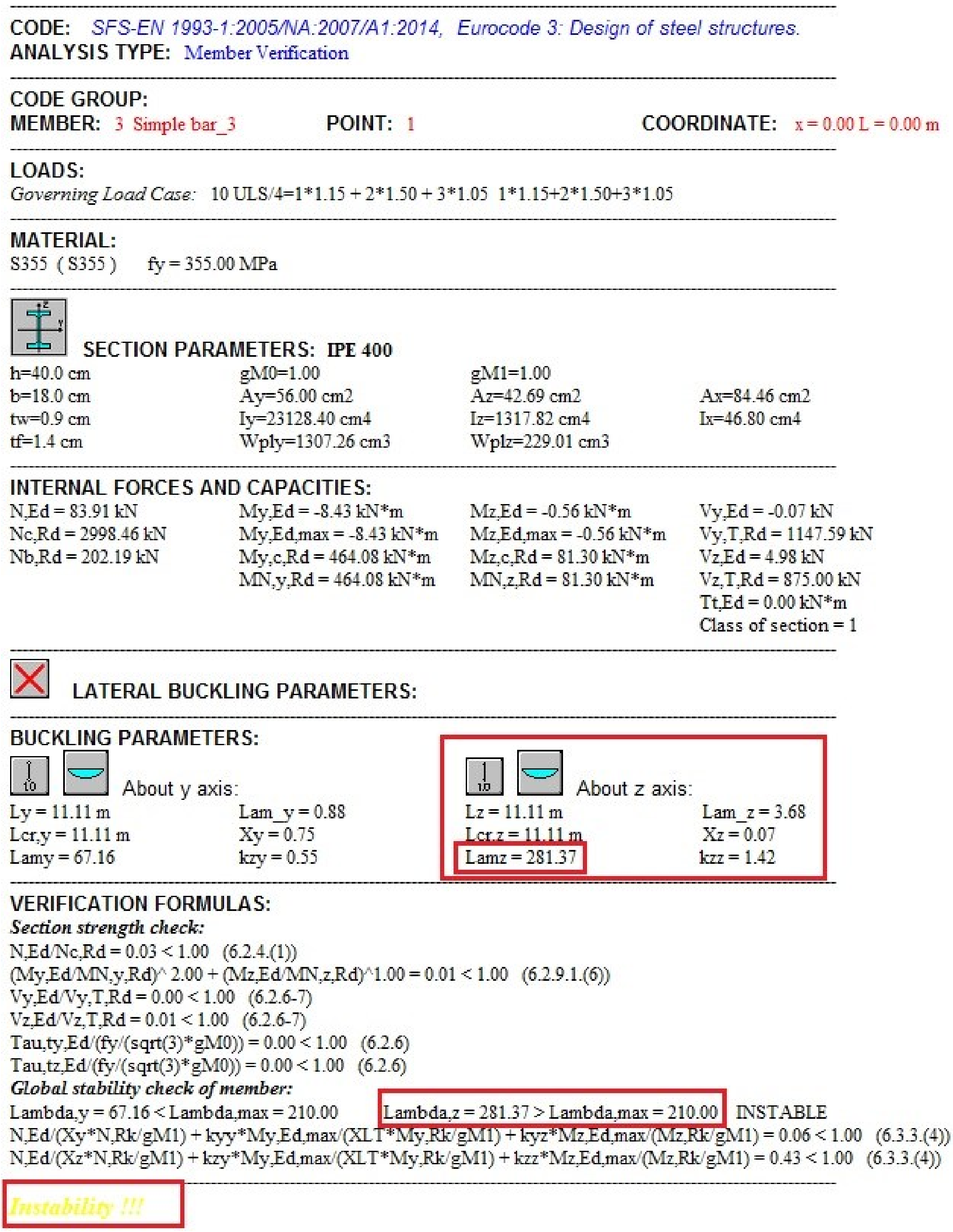
Combinations: Automatic load combinations were generated using dead loads, snow loads and loads like wind load cases 1-6 from above. The combinations generated are shown in Table 4 below, emphasis placed on the factors highlighted in red. A total of 44 combinations were generated.

Table 4. Load combinations

Combinations	Name	Analysis type	Combination	Case nature	Definition
7	ULS/1=1*1.35	Nonlin. Combination		dead	1*1.35
8	ULS/2=1*0.90	Nonlin. Combination		dead	1*0.90
9	ULS/3=1*1.15	Nonlin. Combination		dead	1*1.15
10	ULS/4=1*1.15 +	Nonlin. Combination		dead	1*1.15+2*1.50+3*1.05
11	ULS/5=1*1.15 +	Nonlin. Combination		dead	1*1.15+2*1.50
12	ULS/6=1*1.15 +	Nonlin. Combination		dead	1*1.15+4*1.50+3*1.05
13	ULS/7=1*1.15 +	Nonlin. Combination		dead	1*1.15+4*1.50
14	ULS/8=1*1.15 +	Nonlin. Combination		dead	1*1.15+5*1.50+3*1.05
15	ULS/9=1*1.15 +	Nonlin. Combination		dead	1*1.15+5*1.50
16	ULS/10=1*1.15	Nonlin. Combination		dead	1*1.15+6*1.50+3*1.05
17	ULS/11=1*1.15	Nonlin. Combination		dead	1*1.15+6*1.50
18	ULS/12=1*0.90	Nonlin. Combination		dead	1*0.90
19	ULS/13=1*0.90	Nonlin. Combination		dead	1*0.90+2*1.50+3*1.05
20	ULS/14=1*0.90	Nonlin. Combination		dead	1*0.90+2*1.50
21	ULS/15=1*0.90	Nonlin. Combination		dead	1*0.90+4*1.50+3*1.05
22	ULS/16=1*0.90	Nonlin. Combination		dead	1*0.90+4*1.50
23	ULS/17=1*0.90	Nonlin. Combination		dead	1*0.90+5*1.50+3*1.05
24	ULS/18=1*0.90	Nonlin. Combination		dead	1*0.90+5*1.50
25	ULS/19=1*0.90	Nonlin. Combination		dead	1*0.90+6*1.50+3*1.05
26	ULS/20=1*0.90	Nonlin. Combination		dead	1*0.90+6*1.50
27	ULS/21=1*1.15	Nonlin. Combination		dead	1*1.15+3*1.50
28	ULS/22=1*1.15	Nonlin. Combination		dead	1*1.15+2*0.90+3*1.50
29	ULS/23=1*1.15	Nonlin. Combination		dead	1*1.15+4*0.90+3*1.50
30	ULS/24=1*1.15	Nonlin. Combination		dead	1*1.15+5*0.90+3*1.50
31	ULS/25=1*1.15	Nonlin. Combination		dead	1*1.15+6*0.90+3*1.50
32	ULS/26=1*0.90	Nonlin. Combination		dead	1*0.90+3*1.50
33	ULS/27=1*0.90	Nonlin. Combination		dead	(1+2)*0.90+3*1.50
34	ULS/28=1*0.90	Nonlin. Combination		dead	(1+4)*0.90+3*1.50
35	ULS/29=1*0.90	Nonlin. Combination		dead	(1+5)*0.90+3*1.50
36	ULS/30=1*0.90	Nonlin. Combination		dead	(1+6)*0.90+3*1.50
37	SLS:CHR/1=1*1.	Nonlin. Combination	SLS:CH	dead	1*1.00
38	SLS:CHR/2=1*1.	Nonlin. Combination	SLS:CH	dead	(1+2)*1.00+3*0.70
39	SLS:CHR/3=1*1.	Nonlin. Combination	SLS:CH	dead	(1+2)*1.00
40	SLS:CHR/4=1*1.	Nonlin. Combination	SLS:CH	dead	(1+4)*1.00+3*0.70
41	SLS:CHR/5=1*1.	Nonlin. Combination	SLS:CH	dead	(1+4)*1.00
42	SLS:CHR/6=1*1.	Nonlin. Combination	SLS:CH	dead	(1+5)*1.00+3*0.70
43	SLS:CHR/7=1*1.	Nonlin. Combination	SLS:CH	dead	(1+5)*1.00
44	SLS:CHR/8=1*1.	Nonlin. Combination	SLS:CH	dead	(1+6)*1.00+3*0.70
45	SLS:CHR/9=1*1.	Nonlin. Combination	SLS:CH	dead	(1+6)*1.00
46	SLS:CHR/10=1*	Nonlin. Combination	SLS:CH	dead	(1+3)*1.00
47	SLS:CHR/11=1*	Nonlin. Combination	SLS:CH	dead	(1+3)*1.00+2*0.60
48	SLS:CHR/12=1*	Nonlin. Combination	SLS:CH	dead	(1+3)*1.00+4*0.60
49	SLS:CHR/13=1*	Nonlin. Combination	SLS:CH	dead	(1+3)*1.00+5*0.60
50	SLS:CHR/14=1*	Nonlin. Combination	SLS:CH	dead	(1+3)*1.00+6*0.60

A deformation check was made by selecting all load combinations and running the calculation. The highest amount of deflection was cable sagging with a total of 25cm with pre-tension taken into account.

Member verification showed that no member failed the check except for instability error. Figure 37 shows an example of a calculation note generated for a bracing member that was flagged for instability.



CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 3 Simple bar_3 POINT: 1 COORDINATE: x = 0.00 L = 0.00 m

LOADS:
Governing Load Case: 10 ULS/4=1*1.15 + 2*1.50 + 3*1.05 1*1.15+2*1.50+3*1.05

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: **IPE 400**

h=40.0 cm	gM0=1.00	gM1=1.00	
b=18.0 cm	Ay=56.00 cm2	Az=42.69 cm2	Ax=84.46 cm2
tw=0.9 cm	Iy=23128.40 cm4	Iz=1317.82 cm4	Ix=46.80 cm4
tf=1.4 cm	Wply=1307.26 cm3	Wplz=229.01 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 83.91 kN	My,Ed = -8.43 kN*m	Mz,Ed = -0.56 kN*m	Vy,Ed = -0.07 kN
Nc,Rd = 2998.46 kN	My,Ed,max = -8.43 kN*m	Mz,Ed,max = -0.56 kN*m	Vy,T,Rd = 1147.59 kN
Nb,Rd = 202.19 kN	My,c,Rd = 464.08 kN*m	Mz,c,Rd = 81.30 kN*m	Vz,Ed = 4.98 kN
	MN,y,Rd = 464.08 kN*m	MN,z,Rd = 81.30 kN*m	Vz,T,Rd = 875.00 kN
			Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 11.11 m	Lam_y = 0.88	Lz = 11.11 m	Lam_z = 3.68
Lcr,y = 11.11 m	Xy = 0.75	Lcr,z = 11.11 m	Xz = 0.07
Lamy = 67.16	kzy = 0.55	Lamz = 281.37	kzz = 1.42

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.03 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.01 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.01 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
Lambda,y = 67.16 < Lambda,max = 210.00 Lambda,z = 281.37 > Lambda,max = 210.00 INSTABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.06 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.43 < 1.00 (6.3.3.(4))

Instability !!!

Figure 37. Calculation Note

The highlighted regions in red show the cause of instability. In this case, that part of the bracing system is under compression. The I-profile lacks the capacity to bear the load (Case 10 combination) about the z-z local axis resulting in buckling due to a slenderness being 34% beyond capacity. As can be seen the member itself is quite slender with an effective buckling length of 11.11m and cross-sectional area of 84cm^2. To avoid design situation of having over-strengthened members, the solution to reduce the buckling length was chosen with replacing the current bracing system with an X-shaped configuration. Figure 38 below shows the moment diagram, load diagrams as well as a maximum axial force of 83.91kN on the cross-section.

Results:Bar no. 3

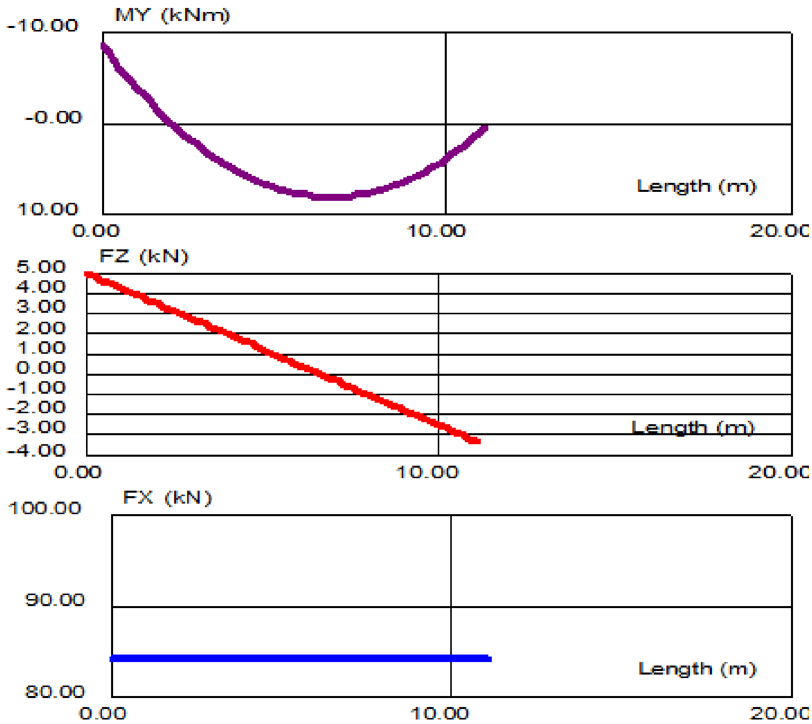
Case 10 ULS/4=1*1.15 + 2*1.50 + 3*1.05

Displacements [cm]:

519	UX=-0,2	UY=-0,0	UZ=-0,1
506	UX=-0,2	UY=0,1	UZ=-0,1

Maximum deflections [cm]:

	UX=0,0	UY=-0,1	UZ=-0,2
at point	X=0,55	X=0,35	X=0,55



	Fx (kN)	Fz (kN)	My (kN*m)
MAX for bar 3	83,91	4,98	8,14
at point:	0,0	0,0	6,67
MIN for bar 3	83,91	-3,33	-8,43
at point:	0,0	11,11	0,0

Figure 38. Results generated

The structure after several iterations was stable with all members passing verification checks. Table 5 shows member types and the section properties used in the model. The structure was now ready for progressive collapse analysis.

Table 5. Member section properties

Section name	Bar list	AX (cm2)	AY (cm2)	AZ (cm2)	IX (cm4)	IY (cm4)	IZ (cm4)
HEA 300	329 2379 2	112,53	81,21	26,32	75,30	18263,50	6309,56
HEA 600	302to314 4	226,46	143,87	77,58	440,00	141208,00	11271,30
HEB 600	54to56 58to	269,96	172,45	94,50	759,00	171041,00	13530,20
IPE 200	5 315to328	28,48	17,22	11,29	6,46	1943,17	142,37
IPE 300	682to784 9	53,81	31,63	21,51	19,47	8356,11	603,78
IPE 400	453to478 2	84,46	49,01	34,77	46,80	23128,40	1317,82
IPE 500	479to504 1	115,52	63,49	50,99	89,00	48198,50	2141,69
RRHS 200x200x10	13to17 39to	72,60	40,00	40,00	7070,00	4250,00	4250,00

6.3 **Progressive Collapse Analysis**

Since this is a Class 3 structure, EN-1991-1-7 outlines an approach as discussed in section 4.3 and 4.4. The Risk Assessment Method and Key-Element Approach are stipulated as the appropriate robustness checks. Risk mitigation factors like PA systems, fire truck access and barricades to protect key elements from collisions by vehicles have already been taken into account when designing the airport, and since risk assessment methods are mainly qualitative in nature, the more quantitative approach of Key Element removal; the focus point for this case study.

In addition to this approach, the Notional Removal approach was applied to selected members of the structure and the shortcomings of Autodesk Robot Structures was discussed.

The existing concrete structure building should have adequate horizontal and vertical ties (embedded rebar joining columns to beams with adequate anchoring lengths) in accordance with the norms. Since the analysis model is a steel structure, beams and columns are considered already tied due to the unavoidable use of welds and bolts in steel connections. If one end of a steel beam fails, the other end's bolts may hold it from falling. This applies to both hinged and stiff connections.

A Key Element as defined is Eurocodes as any supporting column, and any beam supporting a column, however, in the model there exists such columns but no such beams. However, the model has a row of transfer beams on the third floor supporting secondary beams. The transfer beams shown in Figure 39 are important enough to be regarded as a key element in this case study. Figure 19 (p.20) above shows the columns to be notionally removed in addition to the bracing system in Section 2 of this analysis.

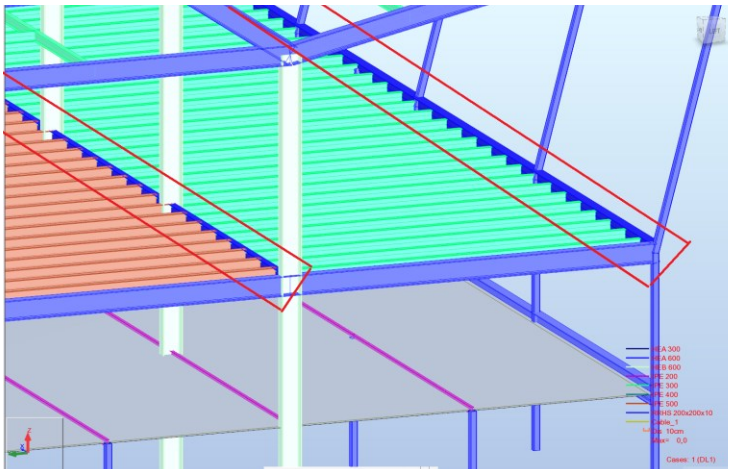


Figure 39. Transfer beams on the third floor

6.3.1 Robot Structures Analysis Method

The Robot Structures software assigns the load type “Linear-static” to models which contain members that are tension or compression-only. If tension members like cables or bracing are added to the model, “Nonlinear static” load type is assigned to the loads and analysed as such. This, however, does not take into account geometric nonlinearities in the structure. Nonlinear analysis takes into account Second-Order effects like bending rigidity depending on lateral forces that arise from eccentricity.

To allow for geometric nonlinearities such as stresses resulting from deformations and additional lateral rigidity, P-Delta analysis must be enabled. Selecting geometric non-linearity takes the actual higher-order effects into consideration. The method of solving equations was set to “Skyline” as was recommended by Autodesk.

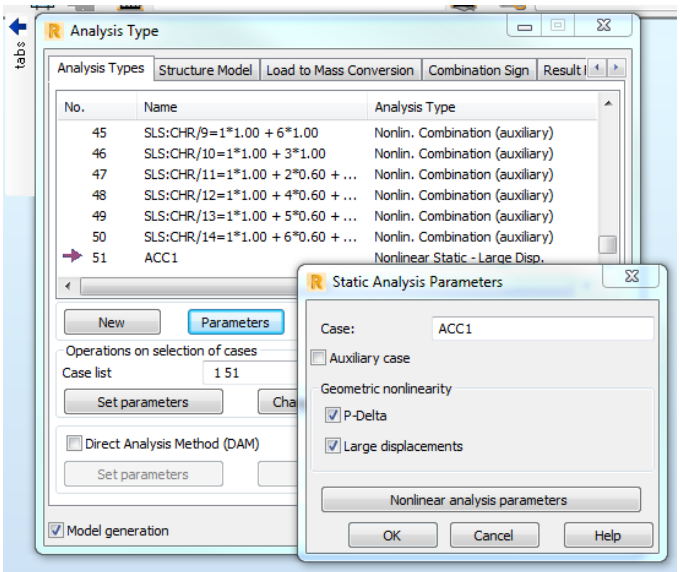


Figure 40. Geometric nonlinear analysis selection

There are two methods to solve a nonlinear equation in Autodesk Robot Structures Analysis i.e. the Arc-length method and the Incremental method.

The Arc-length method is a displacement steering method that applies a load incrementally with the intention of reaching divergence in the nonlinear calculation, that is, until a state of static instability is arrived at. The Arc-length method is defined mainly by deformations. Figure 41 below shows the process of solving for nonlinearities. The vertical axis represents Load while the horizontal axis represents Displacement.

Point 1 and 2 are critical points. A critical point is the point at which the loaded body cannot support an increase in force and loses stability. Point 1 is where the member or structure loses stability with less load still resulting in larger displacements. When stability is regained (a new geometric static equilibrium is achieved) the load is again incrementally added until a predefined boundary condition is reached. Both load and/or displacement boundary condition can be defined in the menu settings.

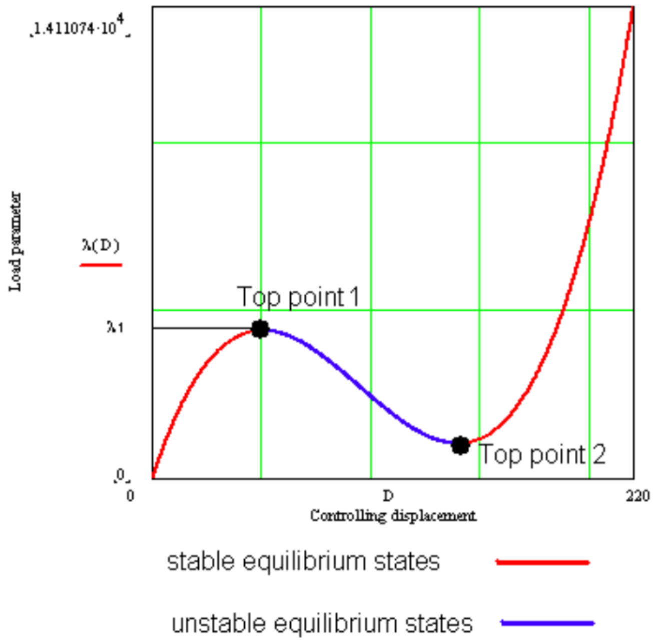


Figure 41. Arc-length method of solving nonlinear equation

This shortcoming of this method is that it tracks only one node in the structure. A node is selected and the axis to which the deformation will be tracked is highlighted from the six drop-down options. The extent the node must deform to, in the selected direction for the calculation to come to a stop, whether reaching convergence or divergence is input as shown below. In the example 10cm was defined.

This method was not used in the analysis of this case study for the reason that it is deformation-steered and that only a single node can be selected. Also, since it is meant for collapsing structures, if there is convergence in the calculation the analysis will run indefinitely. During testing of this method, the analysis had to be manually stopped because the loads were not large enough to collapse the structure. If there is a possibility that the analysis might be convergent then this method is not suitable. Figure 42 below shows the dialogue-box with the described parameters.

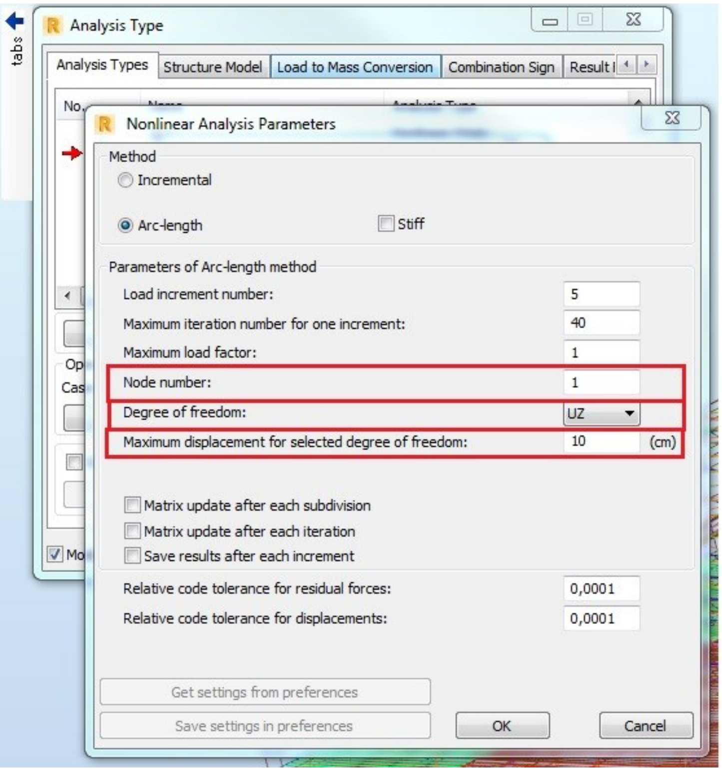


Figure 42. Arc-length method parameters

The Incremental method is based on incrementally applying loads to a member or structure. The loads are increased according to the equation below. Once the state of equilibrium for the previous increment is reached, the next load increment is applied accordingly. In the standard non-linear analysis, the process follows the following equation:

$$d\lambda = 1.0 / X$$

(Equation 6.1)

Where:

X = number of load increments (X can be seen as the percentage of load)

λ = load factor

Equation 6.1 shows the maximum possible load factor (λ) which may be reached for convergent calculations, where λ max =1.0. The point where a nonlinear equation is successfully solved is called Convergence. If the nonlinear equation cannot be solved, the analysis is Divergent which means stability was never achieved. Figure 43 below shows the process of solving for a nonlinear equation. The number of load increments influences the number of calculation iterations.

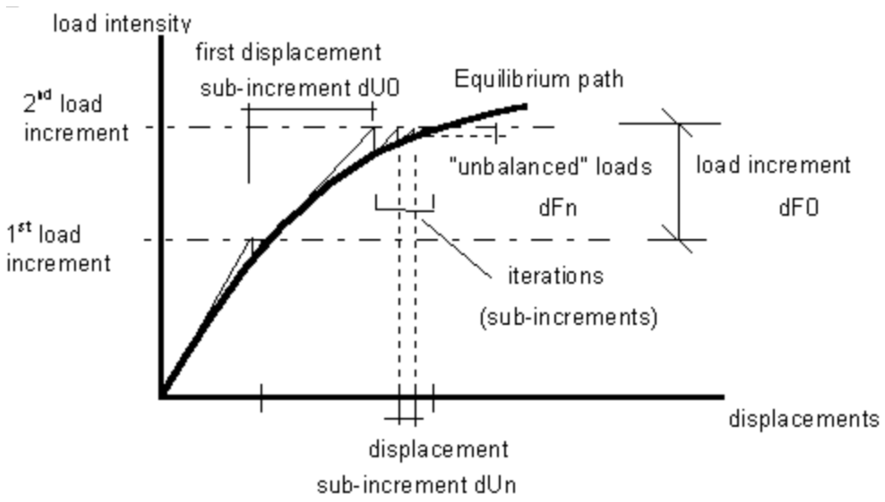


Figure 43. Incremental method Robot uses when solving a nonlinear equation

Below (Figure 44 and 45) are the dialogue boxes encountered when defining the analysis parameters for the Incremental Method.

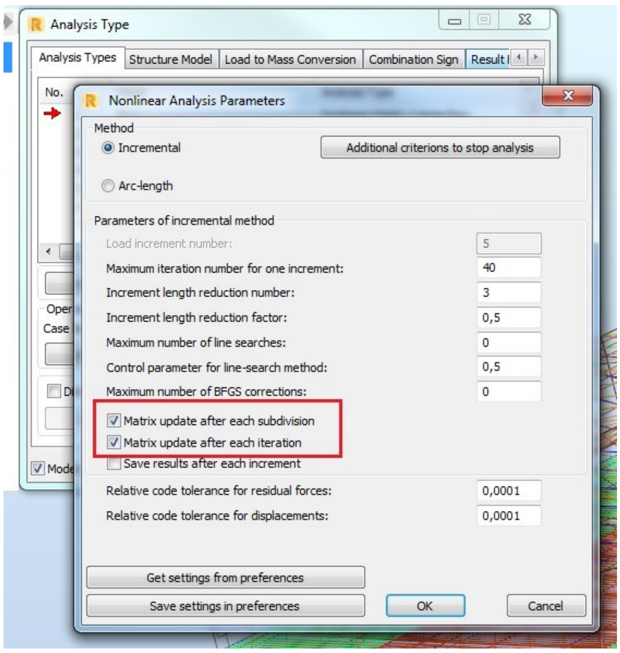


Figure 44. Incremental method parameters

By checking both the "Matrix update after each subdivision" and "Matric update after each iteration" the Full Newton-Raphson algorithm is automatically chosen to run the nonlinear calculations. However, this method is the most time consuming but allows for most accurate results. Unchecking both boxes automatically selects the Initial Stress Method but probability of attaining convergence is greatly reduced.

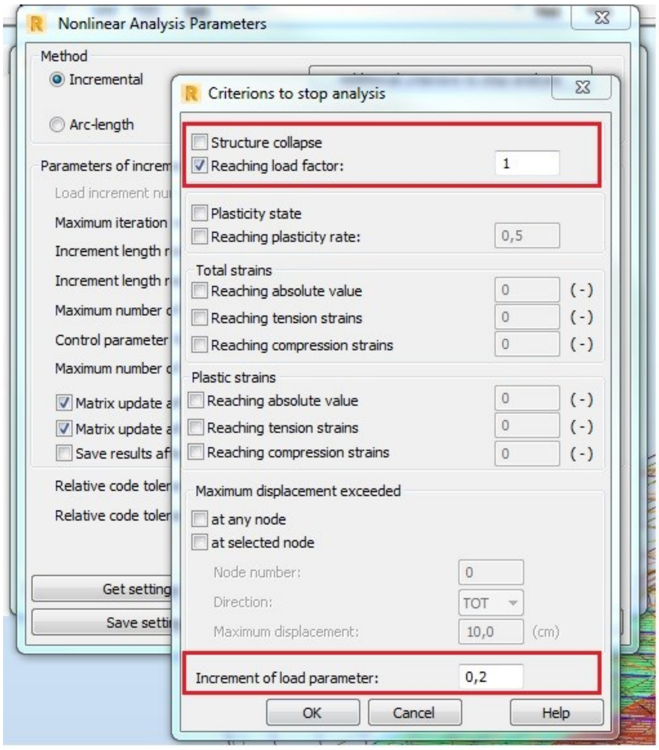


Figure 45. Stop-Calculation parameters

The above criteria can be used to stop the calculations when they are met. 'Structure collapse' option runs the calculation applying the load incrementally until the structure collapses. It is therefore seen that depending on how small your load is compared to the robustness of your structure, and how small your load increment criteria is, it is possible that when selected, the calculation can run almost indefinitely.

'Reaching Load Factor' was chosen in the analysis of this case study since the idea was to increase the load to the value in the load case tables.

The increase load parameter as discussed earlier is the percentage of load applied each time nearing the load factor defined.

6.3.2 How Robot Displays Results

To understand how Robot Structures Analysis displays results of an analysis, a simple structure was built. This was necessary since Robot Structures, unlike SAP2000 or Abaqus FEM software, Robot does not graphically display failed members or structures. Neither does it plot Stress-Strain curves for static nonlinear loads as of the time of writing of this thesis. It, however, displays well detailed Time-History graphs for seismically loaded structures.

This reason necessitated the modelling of a simple multi-storey steel structures braced frame for testing. Supports are pinned but restrained from rotating in the RZ direction. Beams and bracing were released in a Pinned-Pinned configuration. Not all beams were released because Robot tends to give an Instability type 3 warning for over-released structures. The results of the tests are highlighted below.

Test 1: A robust structure

For the first test, a robust stable structure was the aim. All members were RRHS 260x260x10 S355 square sections for simplicity in analysis. All loads except Self-weight, DL1 and Accidental load ACC1 (1kN/m^2 on each floor) were set to 'Auxiliary' loads so that they would not be analysed in the nonlinear calculations as shown in Figure 40. Parameters outlined in section 6.3.1 were used in the analysis. The calculation was run and all members passed verification as well as convergence in the nonlinear analysis as shown below in Figure 47. No errors or warnings were received. Capacity ratios are considered only under ULS.

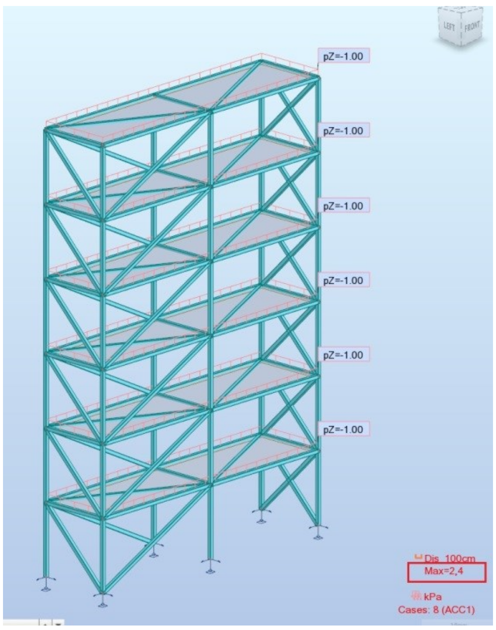


Figure 46. Test structure geometry and deformations

Case 1 : DL1
Analysis type: Static - Large displacements

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 0

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each subdivision
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 10
Precision obtained for non-convergent increment : 5.09267e-07
Relative displacement norm : 5.09267e-07
Relative residual forces norm : 6.92852e-05

Case 8 : ACC1
Analysis type: Static - Large displacements

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 0

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each subdivision
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 10
Precision obtained for non-convergent increment : 7.57523e-07
Relative displacement norm : 7.57523e-07
Relative residual forces norm : 9.14722e-05

Member	Section	Material	Lay	Laz	Ratio	Case
1 Simple bar_1	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
2 Simple bar_2	OK RRHS 260x26	S355	98.98	98.98	0.03	1 DL1
3 Column_3	OK RRHS 260x26	S355	49.49	49.49	0.02	1 DL1
4 Column_4	OK RRHS 260x26	S355	49.49	49.49	0.06	8 ACC1
5 Column_5	OK RRHS 260x26	S355	49.49	49.49	0.04	1 DL1
6 Simple bar_6	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
7 Simple bar_7	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
8 Column_8	OK RRHS 260x26	S355	49.49	49.49	0.04	1 DL1
9 Column_9	OK RRHS 260x26	S355	49.49	49.49	0.05	1 DL1
10 Column_10	OK RRHS 260x26	S355	49.49	49.49	0.04	1 DL1
11 Beam_11	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
12 Beam_12	OK RRHS 260x26	S355	49.49	49.49	0.02	8 ACC1
13 Beam_13	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
14 Beam_14	OK RRHS 260x26	S355	69.99	69.99	0.02	1 DL1
15 Simple bar_15	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
16 Simple bar_16	OK RRHS 260x26	S355	110.66	110.66	0.11	8 ACC1
17 Simple bar_17	OK RRHS 260x26	S355	110.66	110.66	0.03	8 ACC1
18 Simple bar_18	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
19 Simple bar_19	OK RRHS 260x26	S355	110.66	110.66	0.05	1 DL1
20 Simple bar_20	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
21 Column_21	OK RRHS 260x26	S355	49.49	49.49	0.02	1 DL1
23 Simple bar_23	OK RRHS 260x26	S355	110.66	110.66	0.03	1 DL1
24 Simple bar_24	OK RRHS 260x26	S355	69.99	69.99	0.02	1 DL1
25 Column_25	OK RRHS 260x26	S355	49.49	49.49	0.05	8 ACC1
26 Column_26	OK RRHS 260x26	S355	49.49	49.49	0.03	1 DL1
27 Simple bar_27	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
28 Simple bar_28	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
29 Column_29	OK RRHS 260x26	S355	49.49	49.49	0.03	1 DL1
30 Column_30	OK RRHS 260x26	S355	49.49	49.49	0.04	1 DL1
31 Column_31	OK RRHS 260x26	S355	49.49	49.49	0.04	1 DL1
32 Beam_32	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
33 Beam_33	OK RRHS 260x26	S355	49.49	49.49	0.02	8 ACC1
34 Beam_34	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
35 Beam_35	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
36 Simple bar_36	OK RRHS 260x26	S355	110.66	110.66	0.10	8 ACC1
37 Simple bar_37	OK RRHS 260x26	S355	110.66	110.66	0.03	8 ACC1
38 Simple bar_38	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
39 Simple bar_39	OK RRHS 260x26	S355	110.66	110.66	0.04	1 DL1
40 Simple bar_40	OK RRHS 260x26	S355	110.66	110.66	0.03	1 DL1
41 Simple bar_41	OK RRHS 260x26	S355	69.99	69.99	0.02	1 DL1
43 Simple bar_43	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
44 Simple bar_44	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
45 Column_45	OK RRHS 260x26	S355	49.49	49.49	0.02	1 DL1
46 Column_46	OK RRHS 260x26	S355	49.49	49.49	0.04	8 ACC1
47 Column_47	OK RRHS 260x26	S355	49.49	49.49	0.02	1 DL1
48 Simple bar_48	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
49 Simple bar_49	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
50 Column_50	OK RRHS 260x26	S355	49.49	49.49	0.03	1 DL1
51 Column_51	OK RRHS 260x26	S355	49.49	49.49	0.03	1 DL1
52 Column_52	OK RRHS 260x26	S355	49.49	49.49	0.03	1 DL1
53 Beam_53	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
54 Beam_54	OK RRHS 260x26	S355	49.49	49.49	0.02	8 ACC1
55 Beam_55	OK RRHS 260x26	S355	49.49	49.49	0.01	8 ACC1
56 Beam_56	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
57 Simple bar_57	OK RRHS 260x26	S355	110.66	110.66	0.10	8 ACC1
58 Simple bar_58	OK RRHS 260x26	S355	110.66	110.66	0.03	8 ACC1
59 Simple bar_59	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
60 Simple bar_60	OK RRHS 260x26	S355	110.66	110.66	0.04	1 DL1
61 Simple bar_61	OK RRHS 260x26	S355	110.66	110.66	0.03	1 DL1
62 Simple bar_62	OK RRHS 260x26	S355	69.99	69.99	0.01	1 DL1
64 Simple bar_64	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1
65 Simple bar_65	OK RRHS 260x26	S355	98.98	98.98	0.01	8 ACC1

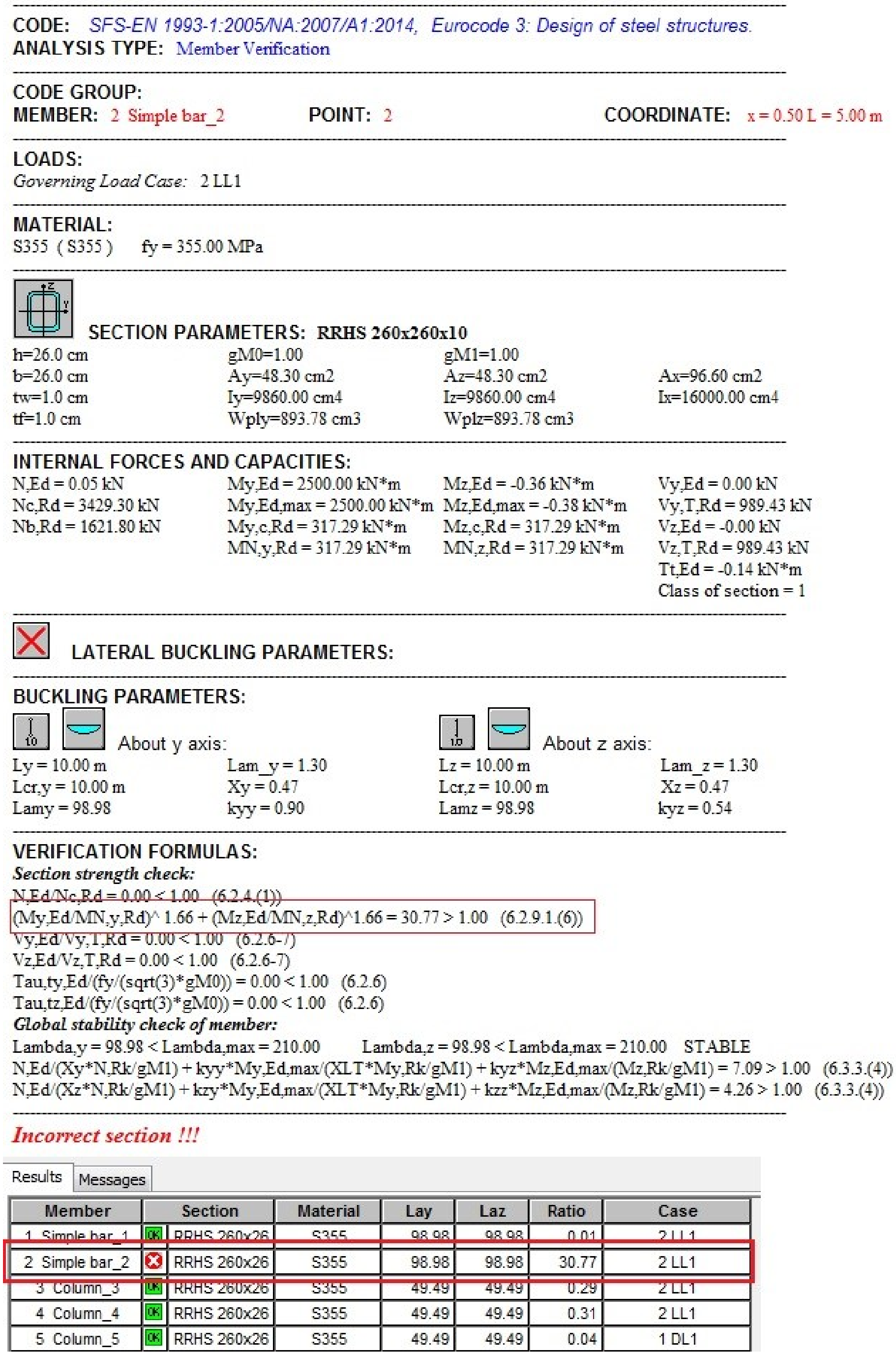
Figure 47. Calculation-Note showing results of the analysis

From Figure 47 above it can be seen that the result of the analysis was convergent on all the load cases highlighted. The calculation note also shows that the method of nonlinear analysis was iterative as indicated and the stop calculation parameter was set to maximum load factor of 1. The load increment parameter was set to 0.2 as indicated.

Test 2: Convergence with a failed member

The aim of the second test was to get a robust stable structure but with one failed member as can occur in real life situations. Failure of members does not necessarily lead to progressive collapse and the aim of this test was to see if that can be shown by the program. This was vital since Class 3 buildings are analysed under Key Element design method.

Member 2 was loaded with an extra live load case exceeding its capacity. The calculation-note was produced as shown in Figure 48.



CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 2 Simple bar_2 POINT: 2 COORDINATE: x = 0.50 L = 5.00 m

LOADS:
Governing Load Case: 2 LL1

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: RRHS 260x260x10

h=26.0 cm	gM0=1.00	gM1=1.00	
b=26.0 cm	Ay=48.30 cm2	Az=48.30 cm2	Ax=96.60 cm2
tw=1.0 cm	Iy=9860.00 cm4	Iz=9860.00 cm4	Ix=16000.00 cm4
tf=1.0 cm	Wply=893.78 cm3	Wplz=893.78 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 0.05 kN	My,Ed = 2500.00 kN*m	Mz,Ed = -0.36 kN*m	Vy,Ed = 0.00 kN
Nc,Rd = 3429.30 kN	My,Ed,max = 2500.00 kN*m	Mz,Ed,max = -0.38 kN*m	Vy,T,Rd = 989.43 kN
Nb,Rd = 1621.80 kN	My,c,Rd = 317.29 kN*m	Mz,c,Rd = 317.29 kN*m	Vz,Ed = -0.00 kN
	MN,y,Rd = 317.29 kN*m	MN,z,Rd = 317.29 kN*m	Vz,T,Rd = 989.43 kN
			Tt,Ed = -0.14 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 10.00 m	Lam_y = 1.30	Lz = 10.00 m	Lam_z = 1.30
Lcr,y = 10.00 m	Xy = 0.47	Lcr,z = 10.00 m	Xz = 0.47
Lamy = 98.98	kyy = 0.90	Lamz = 98.98	kyz = 0.54

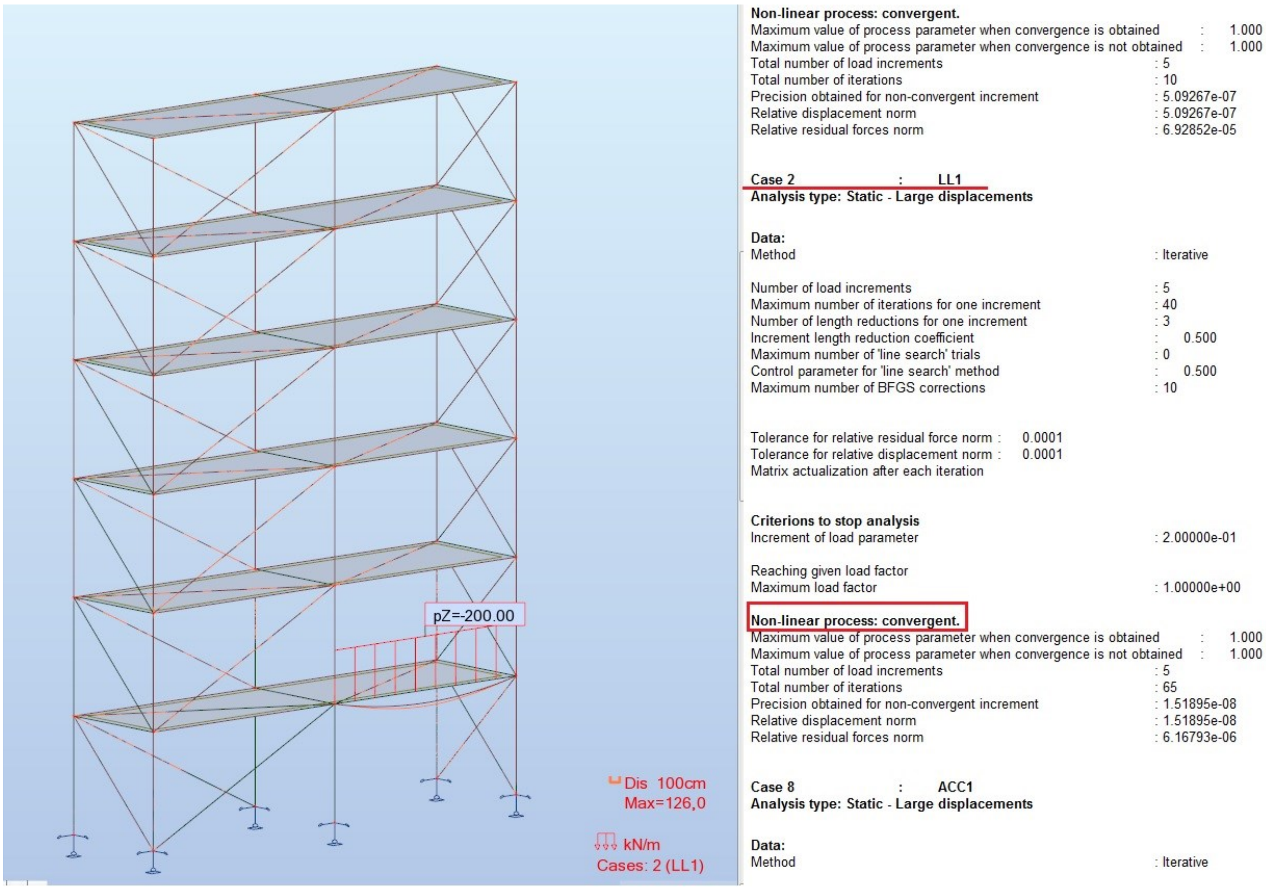
VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nc,Rd = 0.00 < 1.00 (6.2.4.(1))
(My,Ed/MN,y,Rd)^ 1.66 + (Mz,Ed/MN,z,Rd)^1.66 = 30.77 > 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
Lambda,y = 98.98 < Lambda,max = 210.00 Lambda,z = 98.98 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 7.09 > 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 4.26 > 1.00 (6.3.3.(4))

Incorrect section !!!

Results Messages

Member	Section	Material	Lay	Laz	Ratio	Case
1 Simple bar_1	RRHS 260x26	S355	98.98	98.98	0.01	2 LL1
2 Simple bar_2	RRHS 260x26	S355	98.98	98.98	30.77	2 LL1
3 Column_3	RRHS 260x26	S355	49.49	49.49	0.29	2 LL1
4 Column_4	RRHS 260x26	S355	49.49	49.49	0.31	2 LL1
5 Column_5	RRHS 260x26	S355	49.49	49.49	0.04	1 DL1

Figure 48. Member 2 failure



Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 10
Precision obtained for non-convergent increment : 5.09267e-07
Relative displacement norm : 5.09267e-07
Relative residual forces norm : 6.92852e-05

Case 2 : LL1
Analysis type: Static - Large displacements

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 10

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 65
Precision obtained for non-convergent increment : 1.51895e-08
Relative displacement norm : 1.51895e-08
Relative residual forces norm : 6.16793e-06

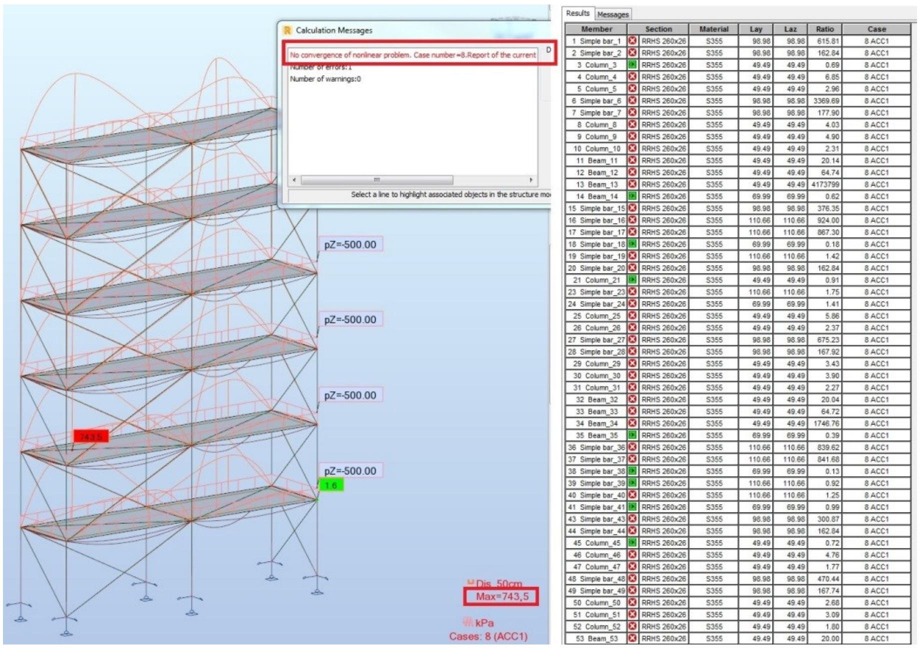
Case 8 : ACC1
Analysis type: Static - Large displacements

Data:
Method : Iterative

Figure 49. A successful convergent result

Test 3: Divergence under extreme load

The last test was to see how divergence is represented by Robot structures and the calculation note produced. For this analysis, an extreme vertical load was applied to all members on all floors. The results



Member	Section	Material	Lay	Laz	Ratio	Case
1 Simple bar_1	RRHS 260x26	S355	98.98	98.98	615.81	8 ACC1
2 Simple bar_2	RRHS 260x26	S355	98.98	98.98	162.84	8 ACC1
3 Column_3	RRHS 260x26	S355	49.49	49.49	0.69	8 ACC1
4 Column_4	RRHS 260x26	S355	49.49	49.49	6.85	8 ACC1
5 Column_5	RRHS 260x26	S355	49.49	49.49	2.96	8 ACC1
6 Simple bar_6	RRHS 260x26	S355	98.98	98.98	3369.69	8 ACC1
7 Simple bar_7	RRHS 260x26	S355	98.98	98.98	177.90	8 ACC1
8 Column_8	RRHS 260x26	S355	49.49	49.49	4.03	8 ACC1
9 Column_9	RRHS 260x26	S355	49.49	49.49	4.90	8 ACC1
10 Column_10	RRHS 260x26	S355	49.49	49.49	2.31	8 ACC1
11 Beam_11	RRHS 260x26	S355	49.49	49.49	20.14	8 ACC1
12 Beam_12	RRHS 260x26	S355	49.49	49.49	64.74	8 ACC1
13 Beam_13	RRHS 260x26	S355	49.49	49.49	4173799	8 ACC1
14 Beam_14	RRHS 260x26	S355	69.99	69.99	0.62	8 ACC1
15 Simple bar_15	RRHS 260x26	S355	98.98	98.98	376.35	8 ACC1
16 Simple bar_16	RRHS 260x26	S355	110.66	110.66	924.00	8 ACC1
17 Simple bar_17	RRHS 260x26	S355	110.66	110.66	867.30	8 ACC1
18 Simple bar_18	RRHS 260x26	S355	69.99	69.99	0.18	8 ACC1
19 Simple bar_19	RRHS 260x26	S355	110.66	110.66	1.42	8 ACC1
20 Simple bar_20	RRHS 260x26	S355	98.98	98.98	162.84	8 ACC1
21 Column_21	RRHS 260x26	S355	49.49	49.49	0.91	8 ACC1
23 Simple bar_23	RRHS 260x26	S355	110.66	110.66	1.75	8 ACC1
24 Simple bar_24	RRHS 260x26	S355	69.99	69.99	1.41	8 ACC1
25 Column_25	RRHS 260x26	S355	49.49	49.49	5.86	8 ACC1
26 Column_26	RRHS 260x26	S355	49.49	49.49	2.37	8 ACC1
27 Simple bar_27	RRHS 260x26	S355	98.98	98.98	675.23	8 ACC1
28 Simple bar_28	RRHS 260x26	S355	98.98	98.98	167.92	8 ACC1
29 Column_29	RRHS 260x26	S355	49.49	49.49	3.43	8 ACC1
30 Column_30	RRHS 260x26	S355	49.49	49.49	3.90	8 ACC1
31 Column_31	RRHS 260x26	S355	49.49	49.49	2.27	8 ACC1
32 Beam_32	RRHS 260x26	S355	49.49	49.49	20.04	8 ACC1
33 Beam_33	RRHS 260x26	S355	49.49	49.49	64.72	8 ACC1
34 Beam_34	RRHS 260x26	S355	49.49	49.49	1746.76	8 ACC1
35 Beam_35	RRHS 260x26	S355	69.99	69.99	0.39	8 ACC1
36 Simple bar_36	RRHS 260x26	S355	110.66	110.66	839.62	8 ACC1
37 Simple bar_37	RRHS 260x26	S355	110.66	110.66	841.68	8 ACC1
38 Simple bar_38	RRHS 260x26	S355	69.99	69.99	0.13	8 ACC1
39 Simple bar_39	RRHS 260x26	S355	110.66	110.66	0.92	8 ACC1
40 Simple bar_40	RRHS 260x26	S355	110.66	110.66	1.25	8 ACC1
41 Simple bar_41	RRHS 260x26	S355	69.99	69.99	0.99	8 ACC1
43 Simple bar_43	RRHS 260x26	S355	98.98	98.98	300.87	8 ACC1
44 Simple bar_44	RRHS 260x26	S355	98.98	98.98	162.84	8 ACC1
45 Column_45	RRHS 260x26	S355	49.49	49.49	0.72	8 ACC1
46 Column_46	RRHS 260x26	S355	49.49	49.49	4.76	8 ACC1
47 Column_47	RRHS 260x26	S355	49.49	49.49	1.77	8 ACC1
48 Simple bar_48	RRHS 260x26	S355	98.98	98.98	470.44	8 ACC1
49 Simple bar_49	RRHS 260x26	S355	98.98	98.98	167.74	8 ACC1
50 Column_50	RRHS 260x26	S355	49.49	49.49	2.68	8 ACC1
51 Column_51	RRHS 260x26	S355	49.49	49.49	3.09	8 ACC1
52 Column_52	RRHS 260x26	S355	49.49	49.49	1.80	8 ACC1
53 Beam_53	RRHS 260x26	S355	49.49	49.49	20.00	8 ACC1

Figure 50. A collapsed structure in Autodesk Robot Structural Analysis

are shown in Figure 50. Typically when collapse of the structure occurs, deformation diagrams that do not make sense can be observed. For example, Figure 50 shows deformations on the opposite side of where they should occur. Generally results obtained after the critical force has been exceeded should be disregarded. What Robot did in this example was to find another point of stability after collapse, on the opposite side of the initially deformed member as shown on point 2 in the graph in Figure 41. Since the stiffness matrix is negative, the further increase in load reduces the opposite side of the deformation resulting in a deformation on the wrong side. (Kosakowski. A, 2015)

Figure 50 below shows divergence in the calculation.

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 10
Precision obtained for non-convergent increment : 5.09267e-07
Relative displacement norm : 5.09267e-07
Relative residual forces norm : 6.92852e-05

Case 8 : ACC1
Analysis type: Static - Large displacements

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 0

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each subdivision
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: divergent.
Maximum value of process parameter when convergence is obtained : 0.950
Maximum value of process parameter when convergence is not obtained : 0.975
Total number of load increments : 7
Total number of iterations : 279
Precision obtained for non-convergent increment : 2.84906e-07
Relative displacement norm : 2.84906e-07
Relative residual forces norm : 1.43558e-03

Figure 51. Calculation Note of a collapsed structure

6.4 Case Study Results

6.4.1 Key Element Method

With the understanding obtained from section 6.3.2, the results obtained from the case study model could be correctly interpreted.

According to the Key Element design method for CC3 structures, a recommended design load should be applied to all orthogonal directions to an identified key element. For the analysis of this thesis, a total of 10 columns from the 144 found in the model were selected for testing. The area selected were columns next to corners, supporting bracings and in the centre of the building. The columns were loaded in five directions. Figure 52 shows a typical example of the results obtained. The accidental load case of $34kN/m^2$ as described in chapter 4.3 was applied while all other loads were set as “Auxiliary Cases” in the Static Analysis Parameter dialogue-box to exclude their effects on the structure.

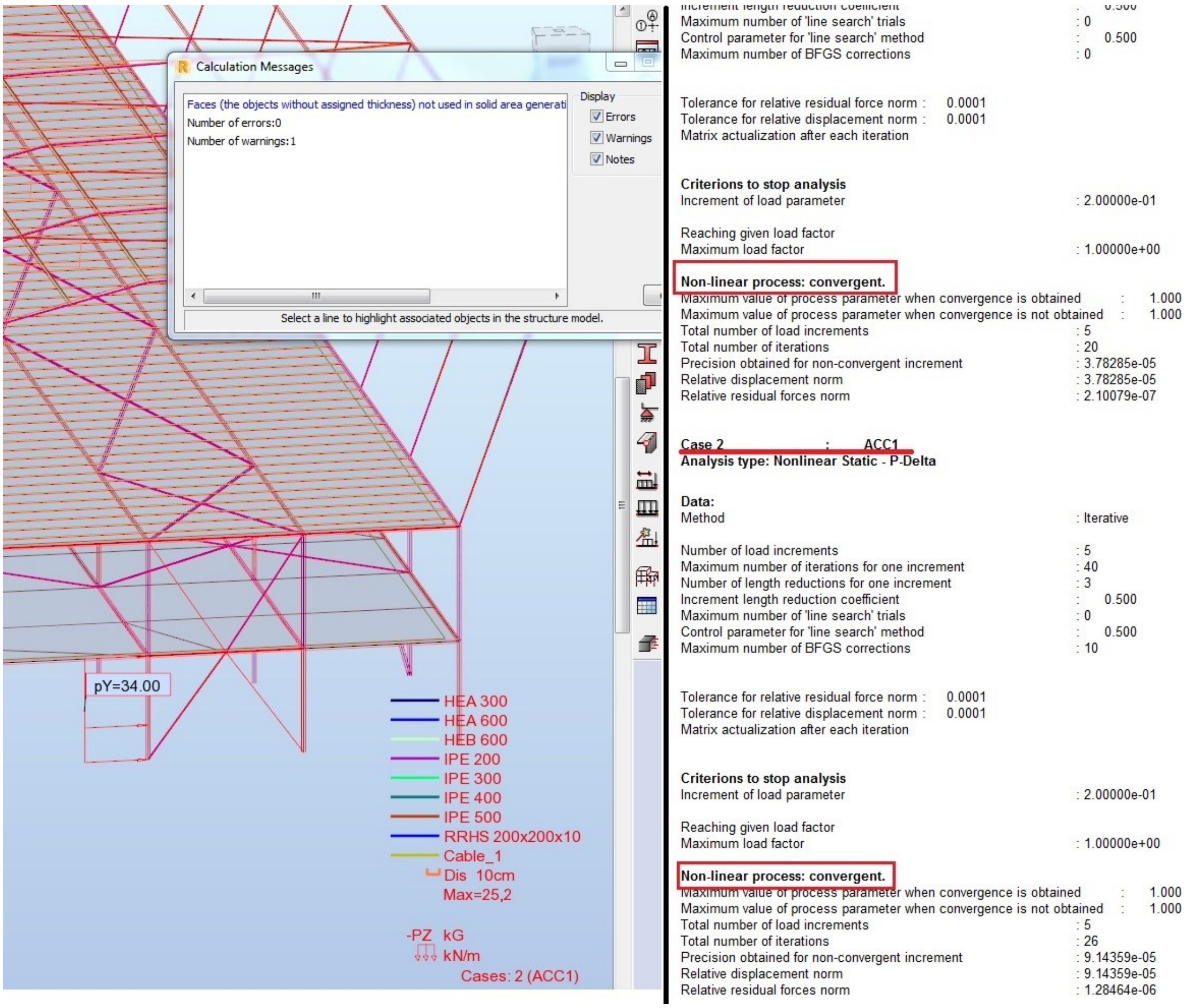
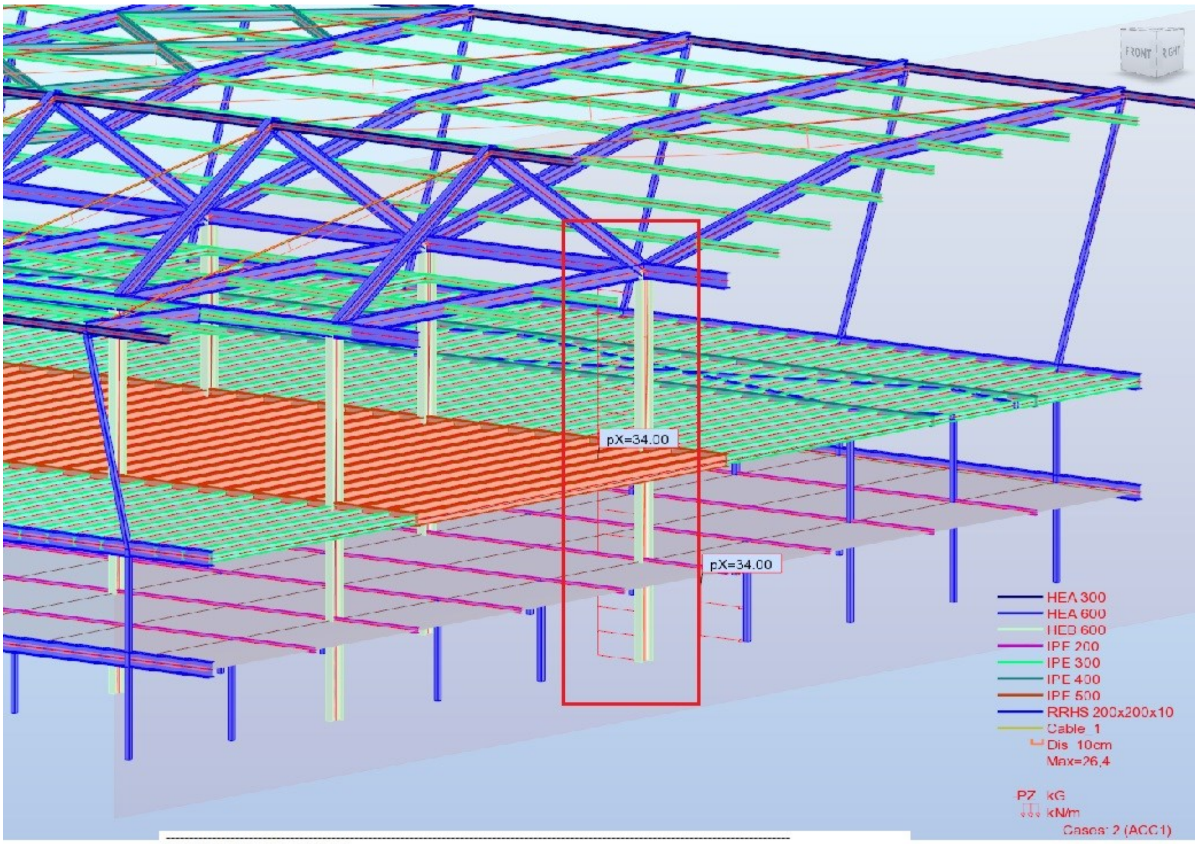


Figure 52. Typical results obtained from a column loaded with the Accidental load case

Progressive collapse was not initiated in any of the loaded columns, however, column shown in Figure 53 below failed.



BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 14.80 m	Lam_y = 0.77	Lz = 14.80 m	Lam_z = 2.74
Lcr,y = 14.80 m	Xy = 0.81	Lcr,z = 14.80 m	Xz = 0.12
Lamy = 58.80	kyz = 1.83	Lamz = 209.05	kzz = 1.83

VERIFICATION FORMULAS:

Section strength check:

Mz,Ed/Mz,c,Rd = 0.00 < 1.00 (6.2.5.(1))
N,Ed/Nc,Rd + Mz,Ed/Mz,c,Rd = 0.07 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed^2 + 3*(Tau,y,Ed+Tau,ty,Ed)^2)/(fy/gM0) = 0.08 < 1.00 (6.2.1.(5))
Vy,Ed/Vy,T,Rd = 0.01 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

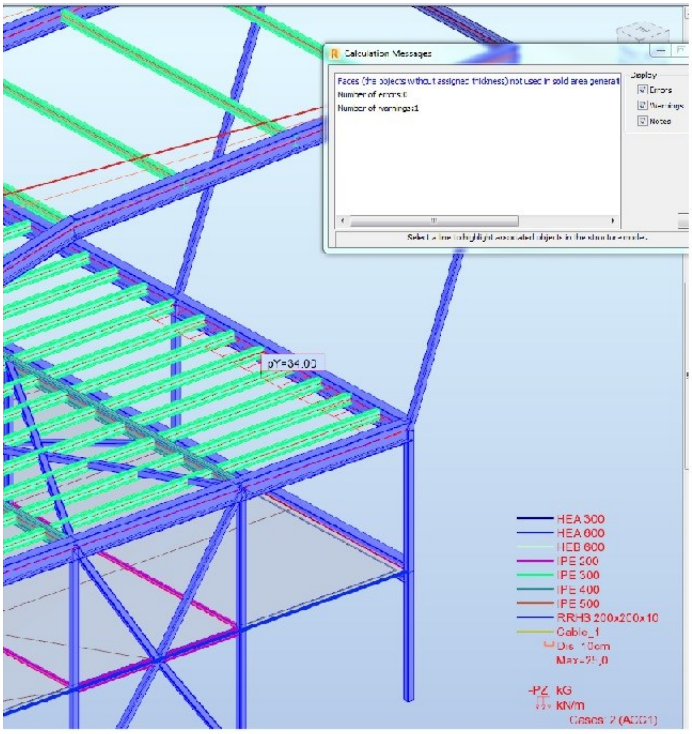
Global stability check of member:

Lambda,y = 58.80 < Lambda,max = 210.00 Lambda,z = 209.05 < Lambda,max = 210.00 STABLE
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.74 < 1.00 (6.3.3.(4))
$\boxed{\text{N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 1.27 > 1.00 (6.3.3.(4))}}$

Incorrect section !!!

Figure 53. Failed column loaded in the weakest direction

The next key elements to be loaded were the non-continuous transfer beams located on the second floor. Progressive collapse was not initiated in any direction of loading, neither did the members fail. Figure 54 shows the result of the analysis.



Case 2 : ACC1
Analysis type: Nonlinear Static - P-Delta

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 10

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: convergent.
Maximum value of process parameter when convergence is obtained : 1.000
Maximum value of process parameter when convergence is not obtained : 1.000
Total number of load increments : 5
Total number of iterations : 26
Precision obtained for non-convergent increment : 9.07878e-05
Relative displacement norm : 9.07878e-05
Relative residual forces norm : 1.32187e-06

Figure 54. Accidental Load on a transfer beam in the weakest axis

6.4.2 Notional Element Removal Method

In addition to the Element removal method, it was decided that Notional Removal Method of members in areas prescribed by Eurocode was to be used in hope of attaining progressive collapse in the structure.

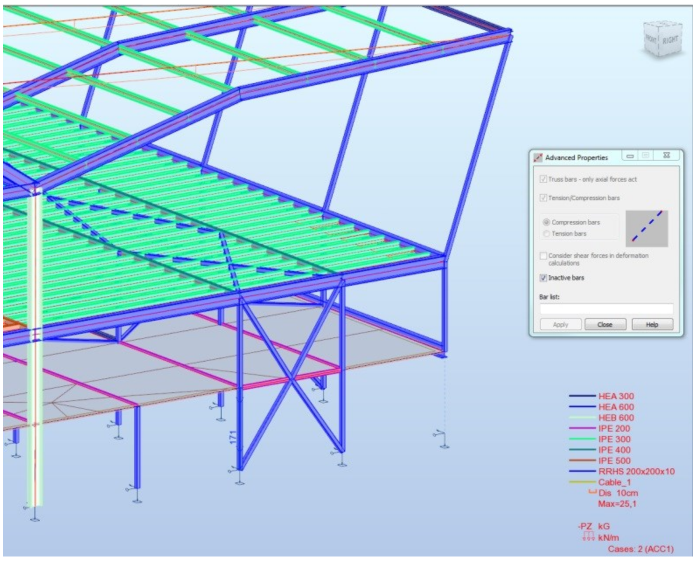


Figure 55. Inactivation of members for Notional Removal Method

It is not necessary to delete a member when analysing for progressive collapse. Robot Structures provides a method for inactivating a member, Figure 55 shows notional removal of a corner column (member 212). The dialogue box shown in the figure can be found in *Geometry > Additional Attributes > Advanced bar Properties.*

The model was analysed with the corner column notionally removed under Load combination 7 which was the most unfavourable load case. The structure collapsed shown in Figure 56 below.

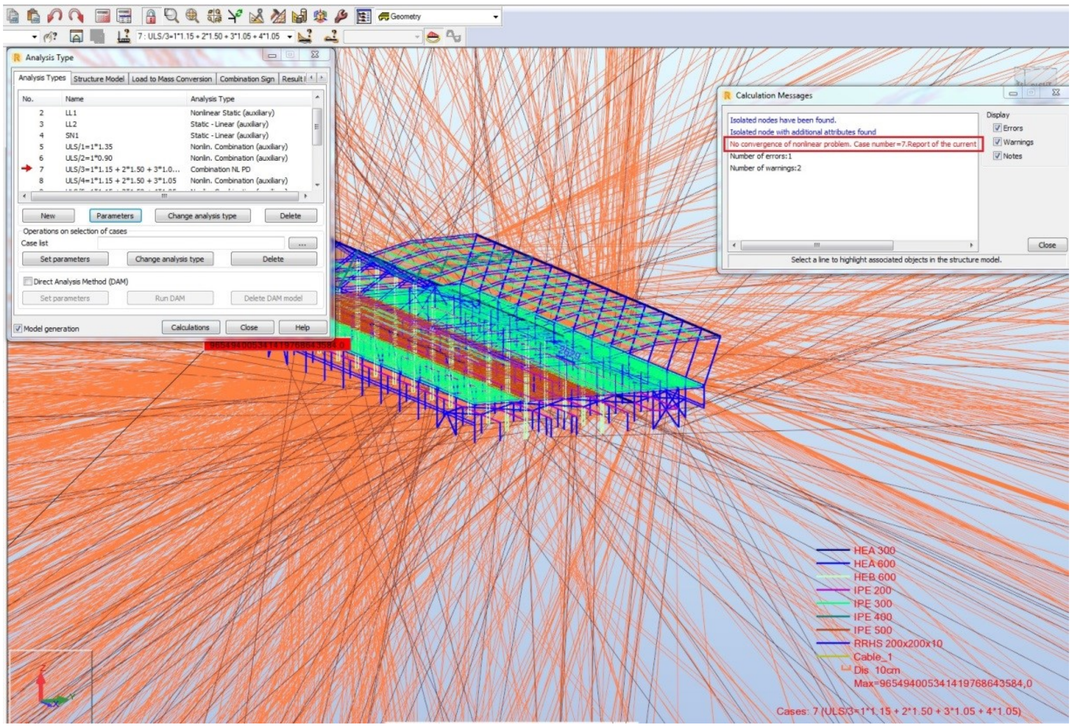


Figure 56. Collapsed structure

The Calculation Note below in Figure 57 shows Divergence in the result. Only one load increment was made with an increment factor of 0.2 as shown in Figure 45 (p.46) above.

Case 7 : ULS/3=1*1.15 + 2*1.50 + 3*1.05 + 4*1.05
Analysis type: Nonlin. Combination

Data:
Method : Iterative

Number of load increments : 5
Maximum number of iterations for one increment : 40
Number of length reductions for one increment : 3
Increment length reduction coefficient : 0.500
Maximum number of 'line search' trials : 0
Control parameter for 'line search' method : 0.500
Maximum number of BFGS corrections : 10

Tolerance for relative residual force norm : 0.0001
Tolerance for relative displacement norm : 0.0001
Matrix actualization after each iteration

Criterions to stop analysis
Increment of load parameter : 2.00000e-01

Reaching given load factor
Maximum load factor : 1.00000e+00

Non-linear process: divergent.
Maximum value of process parameter when convergence is obtained : 0.000
Maximum value of process parameter when convergence is not obtained : 0.025
Total number of load increments : 1
Total number of iterations : 84
Precision obtained for non-convergent increment : 6.06678e+22
Relative displacement norm : 6.06678e+22
Relative residual forces norm : 3.18317e+28

Figure 57. Divergence shown in the Calculation-Note

Members of the bracing system were notionally removed as is recommended (The Institute of Structural Engineers, 2010) and the structure analysed. The predominant load case was wind pressure in the Y+ and Y- axes.

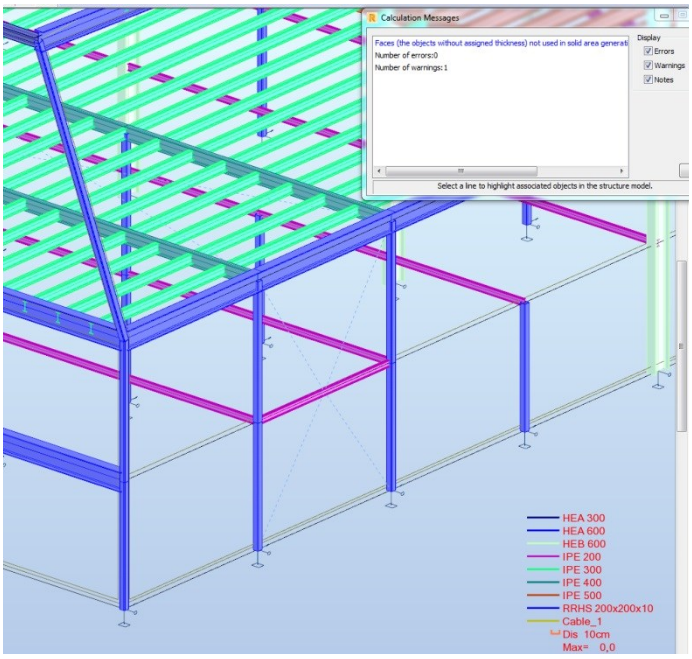


Figure 58. Bracing system notionally removed

The calculation note was convergent and no members failed the ULS verification. There were three bracing systems designed on the structure to handle lateral loads applied in different directions.

A centre column (column 186) was also notionally removed to observe catenary action of the surrounding beams. The location of the removed column is shown in Figure 59 below.

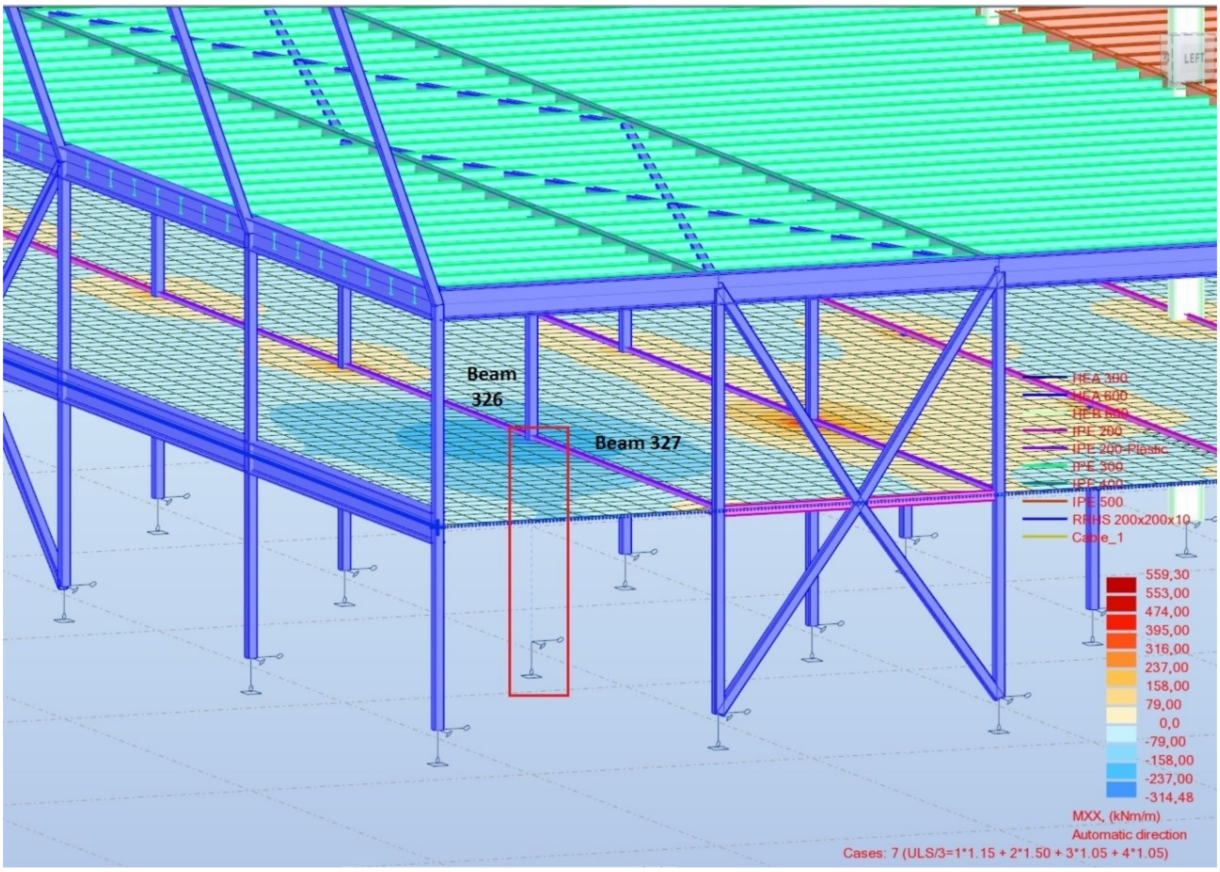


Figure 59. Location of removed column marked in red

Before removal the internal forces of the surrounding beams (326 and 327) were obtained and shown in Figure 60 below. The figure shows that the members passed verification checks and also highlights internal tensile forces “N_{ED}” and shear forces “V_Z” in red. These are the forces expected to rise when the supporting column fails underneath and catenary action takes effect.

CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 326 Beam_326 POINT: 3 COORDINATE: x = 0.50 L = 4.80 m

LOADS:
Governing Load Case: 7 ULS/3=1*1.15 + 2*1.50 + 3*1.05 + 4*1.05 1*1.15+2*1.50+(3+4)*1.05

MATERIAL:
STEEL fy = 235.00 MPa

SECTION PARAMETERS: IPE 200

h=20.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=19.58 cm2	Az=14.00 cm2	Ax=28.48 cm2
tw=0.6 cm	Iy=1943.17 cm4	Iz=142.37 cm4	Ix=6.46 cm4
tf=0.9 cm	Wply=220.66 cm3	Wplz=44.61 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -0.08 kN	My,Ed = 2.43 kN*m	Mz,Ed = -0.00 kN*m	Vy,Ed = 0.00 kN
Nt,Rd = 669.38 kN	My,pl,Rd = 51.85 kN*m	Mz,pl,Rd = 10.48 kN*m	Vy,T,Rd = 265.64 kN
	My,c,Rd = 51.85 kN*m	Mz,c,Rd = 10.48 kN*m	Vz,Ed = -0.10 kN
	MN,y,Rd = 51.85 kN*m	MN,z,Rd = 10.48 kN*m	Vz,T,Rd = 189.94 kN
	Mb,Rd = 13.47 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 13.47 kN*m	Curve,LT - b	XLT = 0.26
Lcr,upp=9.60 m	Lam_LT = 1.96	fi,LT = 2.21	XLT,mod = 0.26

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.00 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.18 < 1.00 (6.3.2.1.(1))

Section OK !!!

CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 327 Beam_327 POINT: 1 COORDINATE: x = 0.50 L =

LOADS:
Governing Load Case: 7 ULS/3=1*1.15 + 2*1.50 + 3*1.05 + 4*1.05 1*1.15+2*1.50+(3+4)*1.05

MATERIAL:
STEEL fy = 235.00 MPa

SECTION PARAMETERS: IPE 200

h=20.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=19.58 cm2	Az=14.00 cm2	Ax=28.48 cm2
tw=0.6 cm	Iy=1943.17 cm4	Iz=142.37 cm4	Ix=6.46 cm4
tf=0.9 cm	Wply=220.66 cm3	Wplz=44.61 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -0.05 kN	My,Ed = 6.02 kN*m	Mz,Ed = 0.00 kN*m	Vy,Ed = 0.00 kN
Nt,Rd = 669.38 kN	My,pl,Rd = 51.85 kN*m	Mz,pl,Rd = 10.48 kN*m	Vy,T,Rd = 265.60 kN
	My,c,Rd = 51.85 kN*m	Mz,c,Rd = 10.48 kN*m	Vz,Ed = -0.65 kN
	MN,y,Rd = 51.85 kN*m	MN,z,Rd = 10.48 kN*m	Vz,T,Rd = 189.92 kN
	Mb,Rd = 13.47 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 13.47 kN*m	Curve,LT - b	XLT = 0.26
Lcr,upp=9.60 m	Lam_LT = 1.96	fi,LT = 2.21	XLT,mod = 0.26

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.01 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.45 < 1.00 (6.3.2.1.(1))

Section OK !!!

Figure 60. Internal forces in beams 326 and 327 before column removal

CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 326 Beam_326 POINT: 1 COORDINATE: x = 0.01 L = 0.10 m

LOADS:
Governing Load Case: 7 ULS/3=1*1.15 + 2*1.50 + 3*1.05 + 4*1.05 1*1.15+2*1.50+(3+4)*1.05

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: IPE 200-Plastic

h=20.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=19.58 cm2	Az=14.00 cm2	Ax=28.48 cm2
tw=0.6 cm	Iy=1943.17 cm4	Iz=142.37 cm4	Ix=6.46 cm4
tf=0.9 cm	Wply=220.66 cm3	Wplz=44.61 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -0.22 kN	My,Ed = -109.37 kN*m	Mz,Ed = -0.00 kN*m	Vy,Ed = -0.00 kN
Nt,Rd = 1011.19 kN	My,pl,Rd = 78.33 kN*m	Mz,pl,Rd = 15.84 kN*m	Vy,T,Rd = 401.29 kN
	My,c,Rd = 78.33 kN*m	Mz,c,Rd = 15.84 kN*m	Vz,Ed = 2039.70 kN
	My,V,Rd = 61.71 kN*m	MN,z,Rd = 15.84 kN*m	Vz,T,Rd = 286.93 kN
	Mb,Rd = 13.47 kN*m		Tt,Ed = -0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 13.47 kN*m	Curve,LT - b	XLT = 0.17
Lcr,low=9.60 m	Lam_LT = 2.41	fi,LT = 3.02	XLT,mod = 0.17

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/My,V,Rd + Mz,Ed/Mz,c,Rd = 1.77 > 1.00 (6.2.8)
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 1.95 > 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 7.11 > 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 8.12 > 1.00 (6.3.2.1.(1))

Incorrect section !!!

CODE: SFS-EN 1993-1:2005/NA:2007/A1:2014, Eurocode 3: Design of steel structures.
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 327 Beam_327 POINT: 3 COORDINATE: x = 0.05 L = 0.51 m

LOADS:
Governing Load Case: 7 ULS/3=1*1.15 + 2*1.50 + 3*1.05 + 4*1.05 1*1.15+2*1.50+(3+4)*1.05

MATERIAL:
S355 (S355) fy = 355.00 MPa

SECTION PARAMETERS: IPE 200-Plastic

h=20.0 cm	gM0=1.00	gM1=1.00	
b=10.0 cm	Ay=19.58 cm2	Az=14.00 cm2	Ax=28.48 cm2
tw=0.6 cm	Iy=1943.17 cm4	Iz=142.37 cm4	Ix=6.46 cm4
tf=0.9 cm	Wply=220.66 cm3	Wplz=44.61 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = -0.15 kN	My,Ed = 70.04 kN*m	Mz,Ed = -0.00 kN*m	Vy,Ed = 0.01 kN
Nt,Rd = 1011.19 kN	My,pl,Rd = 78.33 kN*m	Mz,pl,Rd = 15.84 kN*m	Vy,T,Rd = 401.28 kN
	My,c,Rd = 78.33 kN*m	Mz,c,Rd = 15.84 kN*m	Vz,Ed = 1249.07 kN
	My,V,Rd = 61.71 kN*m	MN,z,Rd = 15.84 kN*m	Vz,T,Rd = 286.93 kN
	Mb,Rd = 13.47 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 13.47 kN*m	Curve,LT - b	XLT = 0.17
Lcr,upp=9.60 m	Lam_LT = 2.41	fi,LT = 3.02	XLT,mod = 0.17

BUCKLING PARAMETERS:
About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
N,Ed/Nt,Rd = 0.00 < 1.00 (6.2.3.(1))
My,Ed/My,V,Rd + Mz,Ed/Mz,c,Rd = 1.14 > 1.00 (6.2.8)
(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.80 < 1.00 (6.2.9.1.(6))
Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)
Vz,Ed/Vz,T,Rd = 4.35 > 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 5.20 > 1.00 (6.3.2.1.(1))

Incorrect section !!!

Figure 61. Internal forces in beams 326 and 327 after column removal

Results from figure 60 show the internal forces in the members. The highlighted figures in red show the current tensile and shear forces in the beams.

As expected, since the beam is acting in bending, the predominant forces are a combination of bending moments (from the concrete slab and live loads) and the resulting shear forces arising from the bending moments and supports with the beams being in static equilibrium. Tensile forces are near zero due to no axial action on the member.

Figure 61 shows what happens when the underlying column is notionally removed. Tensile forces $N_{ED,}$ increase by a factor of 2.75, from 0.08kN to 0.22kN. The increase is significant but not substantial as was expected. Upon further inspection, it was seen that the second floor above consisting of longitudinal IPE 400 beams and IPE 300 transverse beams was rigid enough for the unsupported column to be suspended from the roof.

The exponential increase in tensile capacity N_{ED} (from 669.38 kN in beam 326 to 1011.19 kN) was due to capacity check into the material's plastic mode. N_{ED} is highlighted in red on figure 61.

Shear forces in the beam 326 also exponentially increased from 0.10 kN to 2039.70kN as was expected with the notional removal of the column. The calculation note was convergent and the structure did not collapse due to a central column removal.

7 CONCLUSION

The main focus of this thesis was to study progressive collapse with Autodesk Robot Structural Analysis. The mechanics of how progressive collapse occurs as well as the background of how it came to be a field of study. The different governing codes around the world with an emphasis on Eurocode and their specifications were outlined in the thesis.

A case study of a robust multi-storey steel structure was analysed. This final part was the focus of this thesis with extra attention given to the process of modelling, testing and analysing the structure for progressive collapse using the methods specified by governing code.

Material on analysing for progressive collapse is scant and hard to come by. Little detailed documentation for nonlinear analysis is provided by Autodesk on the subject. Information obtained for this thesis was based on trial-and-error, Autodesk forums, and material found in various similar subjects from previous studies and research conducted in the past.

Eurocode gives guidance on the allowable floor area loss during partial collapse of a structure. Robot Structures is not able to show partial or localised collapse in structures, however, it shows when a structure fully collapses and whether a given member fails or passes verification checks. I have no doubt in future versions of the software their graphical representation will improve as and such analysis will be possible.

The results from the analysis model showed that the structure was robust and collapsed once in more than 25 different accidentally loading situations. The results showed that the building was most susceptible to notional corner column removal. However, it would be increasingly difficult to attain this given that 34kN/m exceeds typical explosive or vehicle impact dynamic loads.

Progressive collapse is a phenomenon that keen attention should be paid to by engineers. There is a wave of interest in the topic that rises and wanes depending on the political climate at a given time. It was shown in this thesis that analysis using available methods and software is possible with a reasonable amount of computing resources and general expertise.

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