SUSCOS – Sustainable Constructions under Natural Hazards and Catastrophic Events
May 2013

# ROBUSTNESS OF STEEL STRUCTURES

## Exercise SOLUTION:

*Application of different robustness approaches to a frame building structure*

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# PART A:

# Application of simplified robustness methods

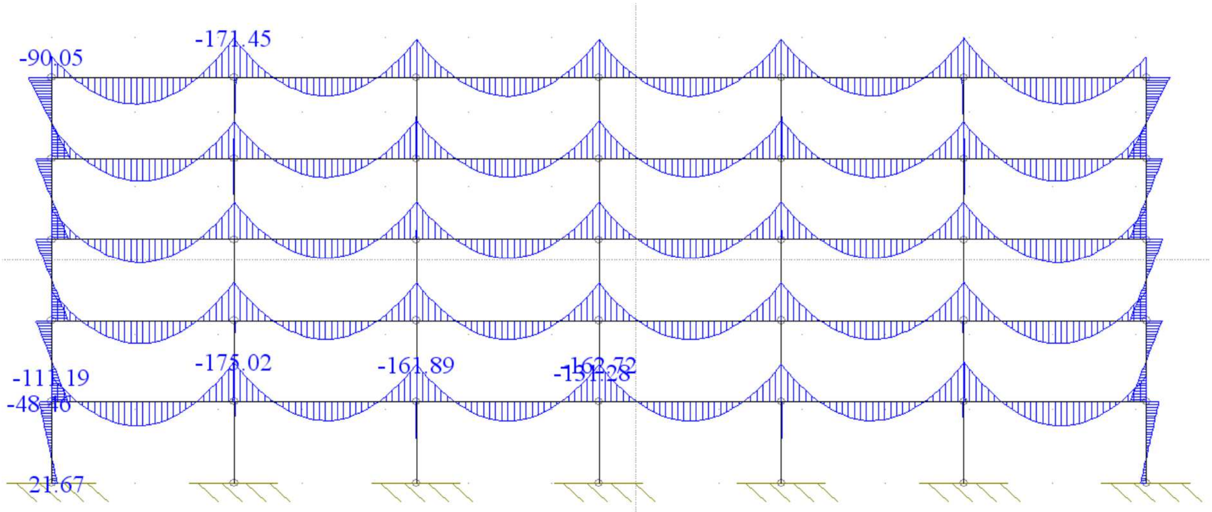
## 1. ANALYSIS OF THE STRUCTURE IN THE INITIAL SITUATION

The structure is first analysed in the initial situation, under the accidental load combination. As the floors are supported by the primary frames, the secondary beams only support their self-weight. Those secondary beams are pinned at both ends and consequently the columns of the structure are only bent about their major axis under the considered vertical loads (no horizontal actions are taken into account here).

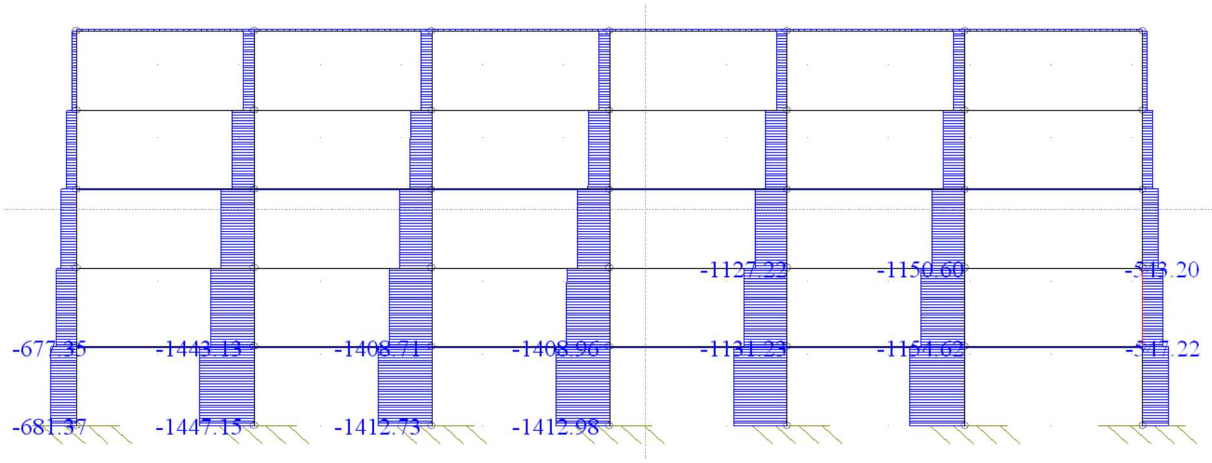
Consequently, a 2D analysis of the primary frames is performed. The beams are submitted to a uniformly distributed load coming from the floors: it is equal to $38{,}75\ kN/m$ for the internal frames and to $19{,}375\ kN/m$ for the external frames. The self-weight of the structural elements is equal to $77{,}00\ kN/m^3$. The beam-to-column joints are considered to be fully rigid.

The diagrams of the bending moment, the normal force and the shear force are represented in Fig. 1 and Fig. 2 for an internal and an external primary frame respectively.

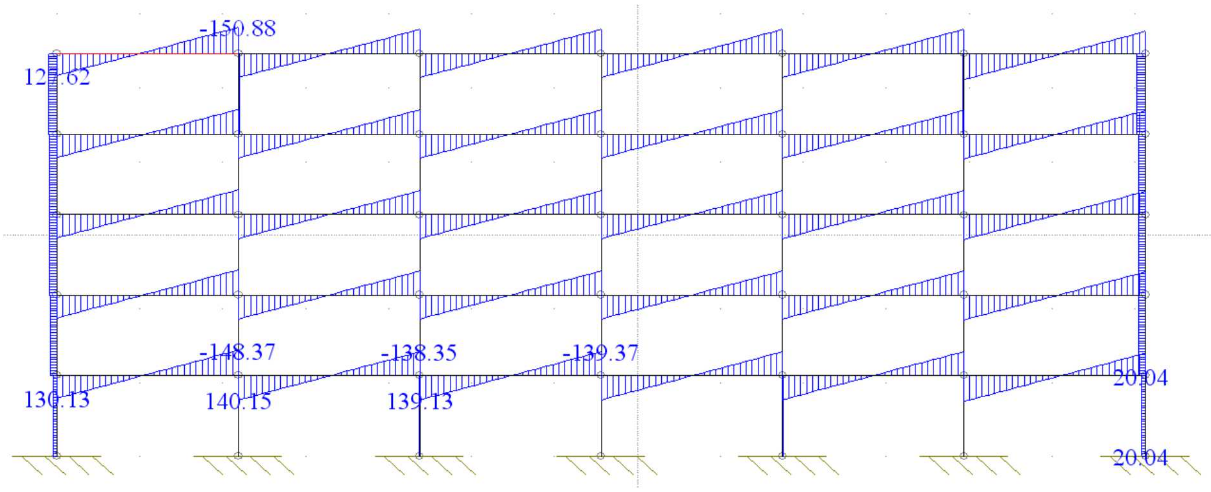
<u>Remark:</u> *The self weight of the secondary beams is neglected.*




*(a) Bending moment (kN.m)*

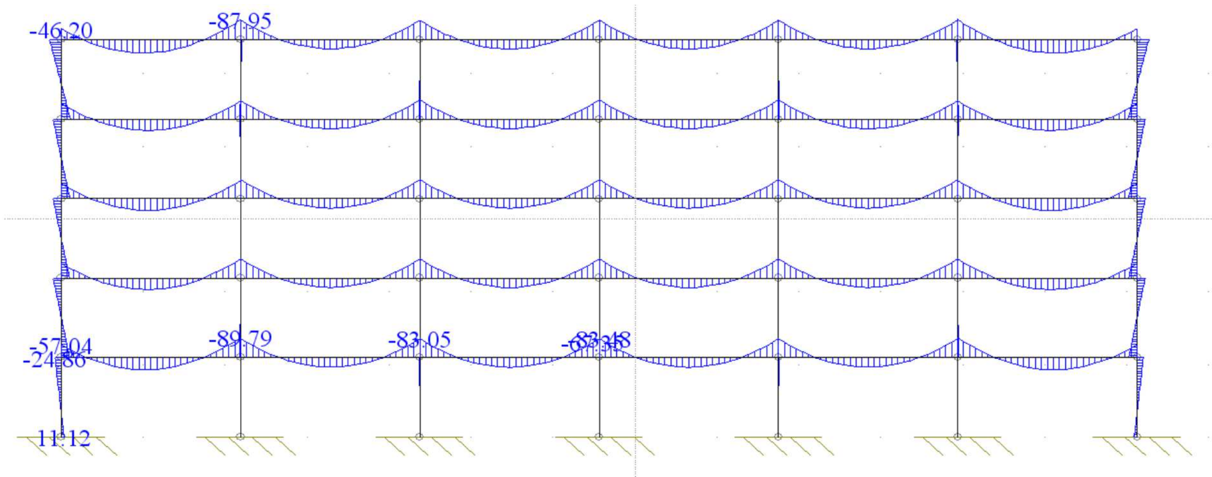



*(b) Normal force (kN)*

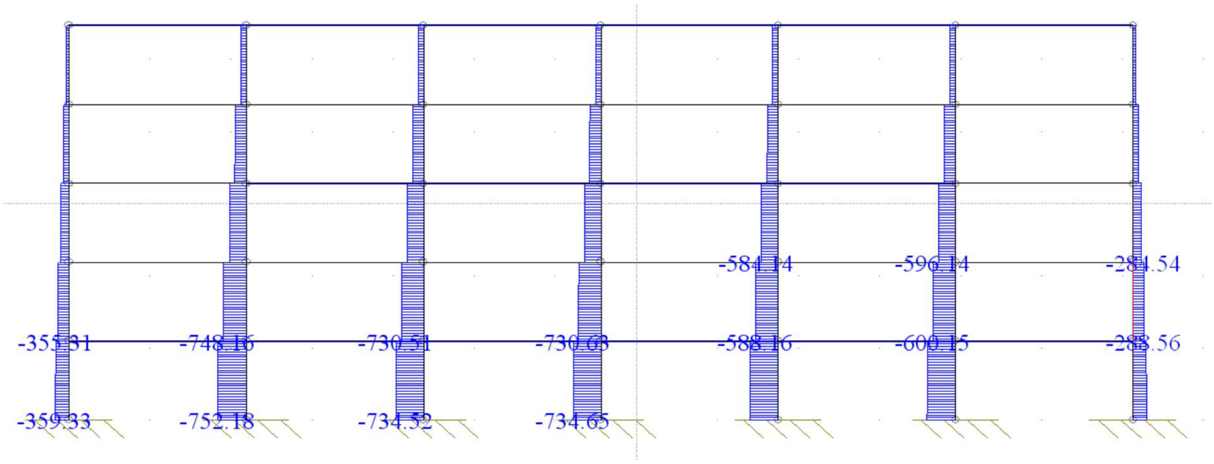



*(c) Shear force (kN)*

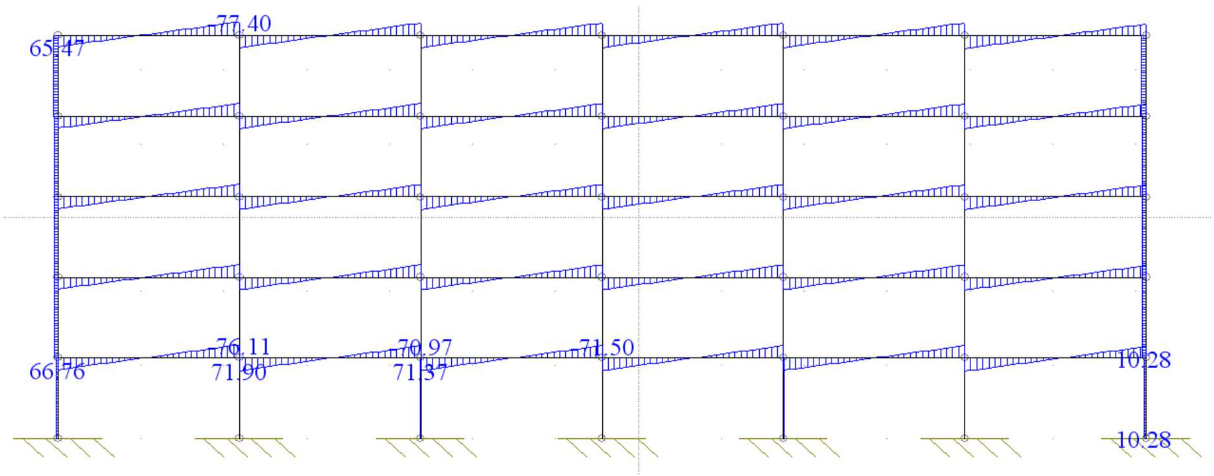
*Fig. 1: Diagrams of the internal forces in an internal primary frame*




*(a) Bending moment (kN.m)*




*(b) Normal force (kN)*




*(c) Shear force (kN)*

*Fig. 2: Diagrams of the internal forces in an external primary frame*

# 2. TYING METHOD

## 2.1. Tying force to be sustained by the horizontal ties

According to the tying method, horizontal ties should be provided around the perimeter of each floor and roof level and internally in two right angle directions. The primary and secondary beams of the structure can play this role on condition that they are able to sustain a sufficient tensile force. Obviously, the joints connecting the beams to the rest of the structure should also be able to transfer this force. The design tying force is given in prEN 1991-1-7:2004 (3) as follows:

- For internal ties : $T_i = max\ (0{,}8 \cdot (g_k + \Psi \cdot q_k) \cdot s \cdot L\,; 75\ kN)$
- For perimeter ties: $T_p = max\ (0{,}4 \cdot (g_k + \Psi \cdot q_k) \cdot s \cdot L\,; 75\ kN)$

Where $s$ is the spacing of ties (5 m for the primary beams and 7 m for the secondary beams); $L$ is the span of the tie (7 m for the primary beams and 5 m for the secondary beams); and

$g_k + \Psi \cdot q_k = 6{,}25 + 0{,}5 \cdot 3 = 7{,}75\ kN/m^2$ refers to the accidental load combination. So the same value of the design tying force is obtained for the primary and the secondary beams:

- For internal beams : $T_i = 217\ kN$
- For perimeter beams: $T_p = 108{,}5\ kN$

The beams and the beam-to-column joints should be able to sustain this force, without consideration of the combination of actions as given in EN 1990 [prEN 1991-1-7:2004 (4)].

## 2.2. Primary and secondary beams

The primary beams are IPE550 profiles and the secondary beams are IPE360 profiles, in S235 steel. The plastic resistance of the IPE360 section in tension is:

$$N_{pl} = 72{,}73 \cdot 100 \cdot 235\ N = 1709\ kN \quad > \quad T_i$$

So the resistance of the secondary beams is sufficient. The tensile resistance of the primary beams is higher and it is thus also sufficient.

## 2.3. Primary beam-to-column joints

### 2.3.1. Introduction

Only the joints at the end of the primary beams are considered in this exercise. From the computation of the joint bending resistance through CoP, it can be observed that no group effect develop including the three upper bolt rows. The resistance of the two upper bolt rows can be found from the CoP results. As the joint are symmetrical, the tensile resistance of the joint is simply equal to twice the resistance of the group including row 1 and row 2 (the group of rows 3 and 4 has the same resistance in tension as the group of rows 1 and 2).

### 2.3.2. Internal joint

For an internal joint, CoP gives the following results:

- Row 1: $N_{Rd,1} = 246{,}55\ kN$ (end-plate in bending – mode 1)
- Row 2: $N_{Rd,2} = 378{,}00\ kN$ (end-plate in bending – mode 1)

Finally, the resistance of the joint in tension is sufficient:

$$N_{Rd,jI1} = (246{,}55 + 378{,}00).\,2 = 1249{,}1\ kN \quad > \quad T_i$$

### 2.3.3. External joint

For an external joint, CoP gives the following results:

- Group of rows 1 and 2: $N_{Rd,1+2} = 845{,}78\ kN$ (column flange in bending – mode 1)

So the resistance of the joint in tension is sufficient:

$$N_{Rd,jE1} = 845{,}78.2 = 1691{,}6\ kN \quad > \quad T_i$$

# 3. KEY ELEMENT METHOD (VEHICLE IMPACT ON A COLUMN)

## 3.1. Introduction

The key element method is applied here considering the particular case of a vehicle impacting a column. The affected elements are designed to resist this accidental load.

Table 4.1 in prEN 1991-1-7:2004 4.3.1 gives equivalent static design forces to be used for different cases of vehicle impacts on members supporting structures over or adjacent to roadways. A force $F_{d,x}$ has to be considered in the direction of normal travel and a force $F_{d,y}$ perpendicular to the direction of normal travel. These two forces need not be considered simultaneously. The height of the resulting collision force (above the level of the carriageway) ranges from 0,50 m (cars) to 1,50 m (lorries).

In this exercise, the perimeter columns have to be designed to resist the accidental load corresponding to the collision of a lorry. The height of the impact point is taken equal to 1,5 meter. Considering the building is located in urban area, the equivalent static forces are:

- Parallel to travel: $F_{d,x} = 500\ kN$
- Perpendicular to travel: $F_{d,y} = 250\ kN$

Practically, columns 1 and 2 have to be checked. For column 1, $F_{d,x}$ causes the columns to bend about its major axis and $F_{d,y}$ is related to minor axis bending. It is the opposite for column 2, which means the higher force ($F_{d,x}$) causes minor axis bending. Consequently, this situation is more critical than an impact in the perpendicular direction ($F_{d,y}$) and column 2 will only be checked under $F_{d,x}$. However, column 1 has to be checked too, even though the minor axis bending moment will be smaller than in column 2 subject to $F_{d,x}$, because the normal compression force is bigger in column 1.

## 3.2. Column 1

### 3.2.1. Impact in the direction parallel to travel

- Analysis of the structure

  - Non-sway frame

When the force $F_{d,x}$ is applied to column 1, it causes a major axis bending of the considered column but also a global "bending" of the corresponding primary frame subject to horizontal loading in its plane. The primary frame is "non-sway" according to the Eurocode criterion (EN 1993-1-1: 2005 5.2.1 (5.1)), which means that although it is unbraced, the nodes are not likely to show significant horizontal displacements and thus global second order effects "$P - \Delta$" can be neglected. Consequently, the columns can be checked using the buckling length corresponding to the global non-sway buckling mode of the structure (assuming the extremities of the columns are fixed).

- Basic analysis for manual computation

As a first assumption, the column is extracted from the frame and is considered to be simply supported at its top end. The bending moment due to the impact load is much higher then the one due to the vertical loads, which is thus neglected. On the other hand, the compression force in the column due to the vertical loads obviously has to be taken into account. Consequently, the internal forces to be considered for the verification are the following:

- $N_{Ed} = 752\ kN$
- $M_{Ed,y} = 319\ kN.m$, with the moment distribution represented in Fig. 3

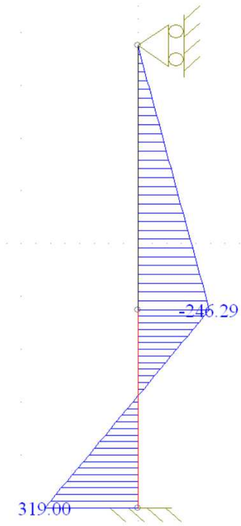
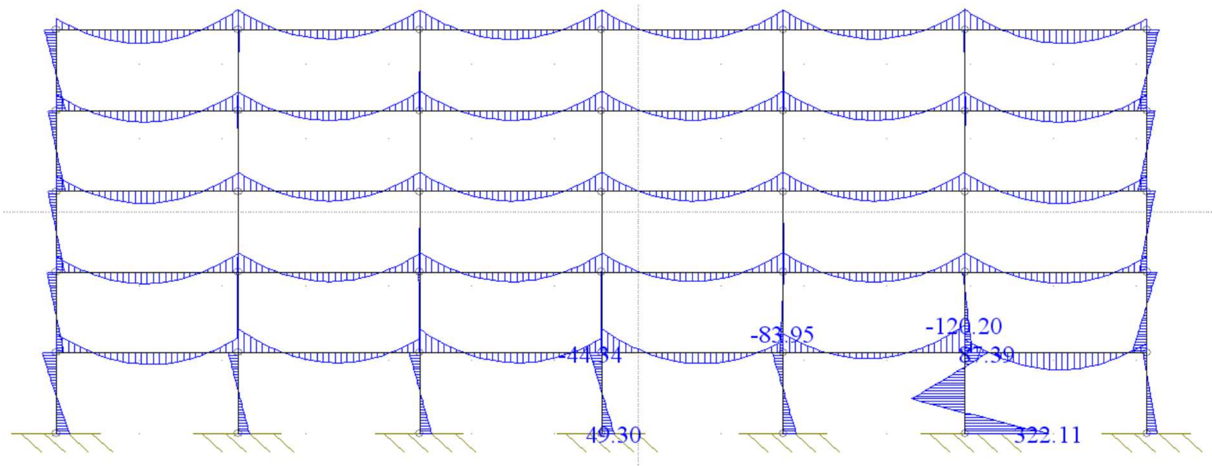


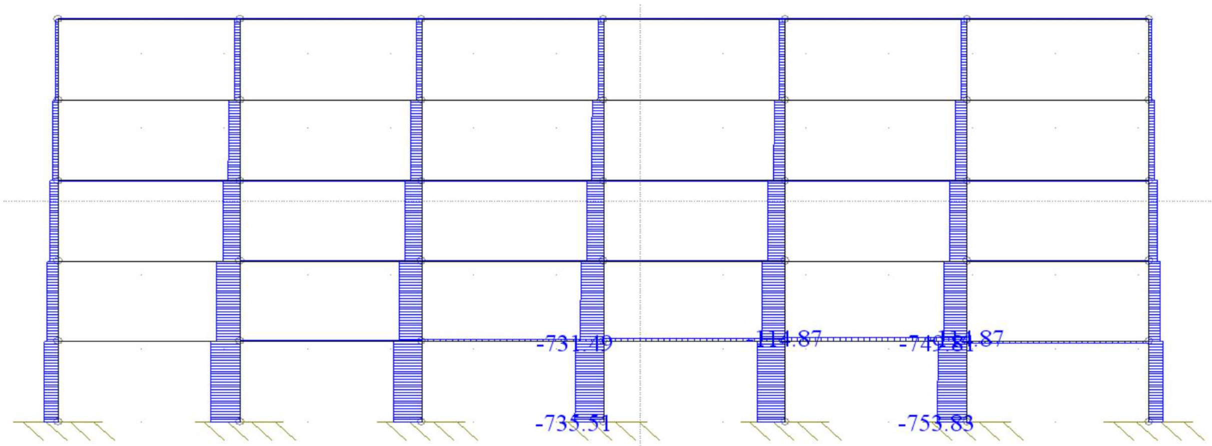

*Fig. 3: Bending moment diagram due to the impact load $F_{d,x}$ on column 1 – basic assumption*

- Analysis of the whole 2D frame

As the first order elastic analysis can easily be performed using the software OSSA2D, the whole 2D frame (primary frame in this case) can directly be analysed under the combination of the vertical loads corresponding to the accidental situation and the impact force. The obtained major axis moment and normal force diagrams are given in Fig. 4 for the case where the force is applied from the right to the left on the figure (worse solicitation).




*(a) Bending moment (kN.m)*




*(b) Normal force (kN)*

*Fig. 4: Internal forces under accidental combination including impact load $F_{d,x}$ on column 1*

➢ Verification of the structural elements

- Impacted column

The impacted column is checked using the provided Excel sheet and considering the following simplifications:

- $C_1 = 1{,}5$
- $C_2 \cdot z_g = 0$
- $k_c = 0{,}8$
- $C_{m,y,0} = 1$
- $L_{fl,y} = L_{fl,z} = 0{,}7.L = 2{,}45\ m$

The internal forces to be taken into account are:

- $N_{Ed} = 754\ kN$
- $M_{Ed,y} = 322\ kN.m$ (moment distribution according to Fig. 4)
- $M_{Ed,z} = 0\ kN.m$
- The shear forces can be neglected

*Conclusion*: HEB300 in S235 steel is OK:

- Y-Y: $0,95 < 1,0$
- Z-Z: $0,61 < 1,0$

- Supporting beam at column top

Due to the impact force, the primary beam that is behind the top of the column is subject to compression ($N_{Ed} = 115\ kN$). The bending moments are also a bit increased. However, the compression force is only 4% of the plastic resistance $N_{pl}$ of the beam and the moments are much smaller than the ones considered in the primary beams to design the structure at ULS under normal forces, especially in the internal frames. So the beam should be OK (the verification was rapidly made under safe assumptions but is not detailed here).

### 3.2.2. Impact in the direction perpendicular to travel

➢ Analysis of the structure

The force $F_{d,y}$ on column 1 is applied in the direction of the secondary frames, which are braced and non-sway. The vertical loads acting on the primary beams induce major axis bending and compression in the columns (see Fig. 2). The minor axis moments in the columns due to the impact load (Fig. 5) have to be considered simultaneously with these internal forces.

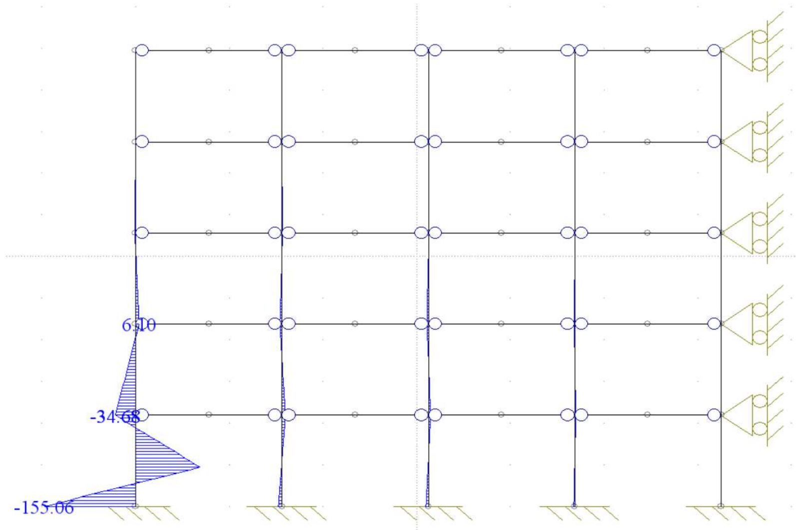



*Fig. 5: Bending moments under impact load $F_{d,y}$ on column 1*

➢ Verification of the structural elements

- Impacted column

The impacted column is checked considering:

- $C_2 \cdot z_g = 0$
- $C_{m,z} = 1$

- $L_{fl,y} = L_{fl,z} = 0{,}7.L = 2{,}45\ m$

*Remark*: $C_1$, $k_c$ *and* $C_{m,y}$ *have no influence here as* $M_{Ed,y} \approx 0\ kN.m$

The solicitations of the column are the following:

- $N_{Ed} = 752\ kN$
- $M_{Ed,y} \approx 0\ kN.m$
- $M_{Ed,z} = 155\ kN.m$ (moment distribution according to Fig. 5)
- The shear forces can be neglected

*Conclusion*: HEB300 in S235 steel is OK:

- Y-Y: $0{,}67 < 1{,}0$
- Z-Z: $0{,}90 < 1{,}0$

- Supporting beam at column top

Due to the impact force, the secondary beam that is behind the top of the column is subject to a compression force equal to $N_{Ed} = 84\ kN$. It can sustain this force.

## 3.3. Column 2

### 3.3.1. Impact in the direction parallel to travel

➢ Analysis of the structure

The force $F_{d,x}$ on column 2 is applied in the direction of the secondary frames, which are braced and non-sway. The vertical loads acting on the primary beams induce major axis bending and compression force in the columns (see Fig. 1). The minor axis moments in the columns due to the impact load (Fig. 6) have to be considered simultaneously with these internal forces.

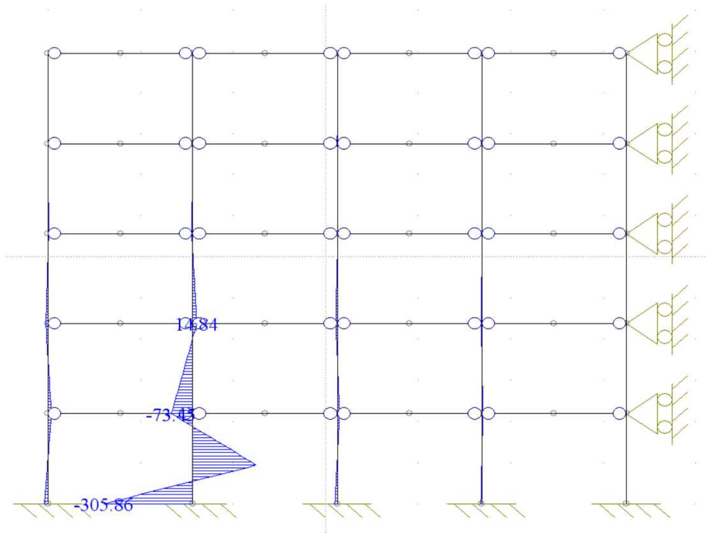



*Fig. 6: Bending moments under impact load $F_{d,x}$ on column 2*

- Verification of the structural elements

  - Impacted column

The internal forces in the impacted column are:

- $N_{Ed} = 681\ kN$
- $M_{Ed,y} = 48\ kN.m$ (linear distribution of moments with $\psi = -0,45$)
- $M_{Ed,z} = 305\ kN.m$ (moment distribution according to Fig. 6)
- The shear forces can be neglected

The following simplifications are considered:

- $C_2 \cdot z_g = 0$
- $C_{m,z} = 1$
- $L_{fl,y} = L_{fl,z} = 0,7.L = 2,45\ m$

The values of $C_1$, $k_c$ *and* $C_{m,y}$ are computed considering the distribution of major axis bending moments:

- $C_{m,y,0} = 0,69$
- $C_1 = 2,29$
- $k_c = 0,68$

*Conclusion*: HEB300 in S235 steel is not OK:

- Y-Y: $1,21 > 1,0$
- Z-Z: $1,69 > 1,0$

In S235 steel, a HEB650 profile would be needed. If we change the steel grade to S355, HEB340 is OK (Z-Z: $0,99 < 1,0$).

- Supporting beam at column top

Due to the impact force, the secondary beam that is behind the top of the column is subject to a compression force equal to $N_{Ed} = 161\ kN$. It can sustain this force.

3.3.2. Impact in the direction perpendicular to travel

As explained in 3.1, the verification of column 2 under impact loading $F_{d,y}$ need not be performed since the collision in the direction parallel to travel ($F_{d,x}$) governs the design.

# 4. BRIDGING METHOD (LOSS OF A COLUMN)

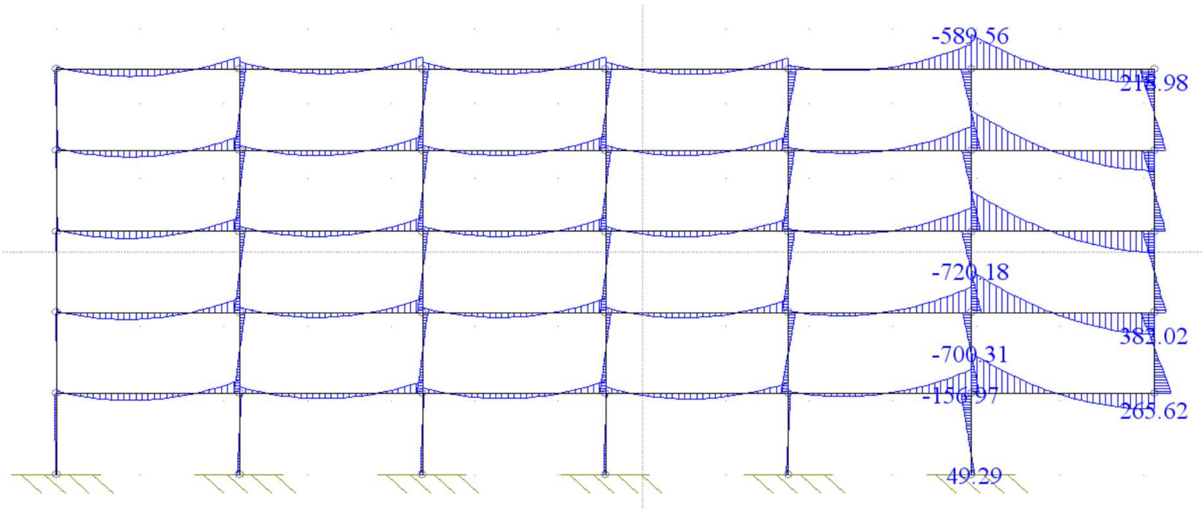
## 4.1. Introduction

The bridging method is applied here considering the loss of column 2 which is assumed to statically disappear due to an unspecified event. The structure has to sustain the loads corresponding to the accidental combination without the lost column. More specifically, the primary frame which the considered column is part of has to keep sustaining the loads after the column has gone.

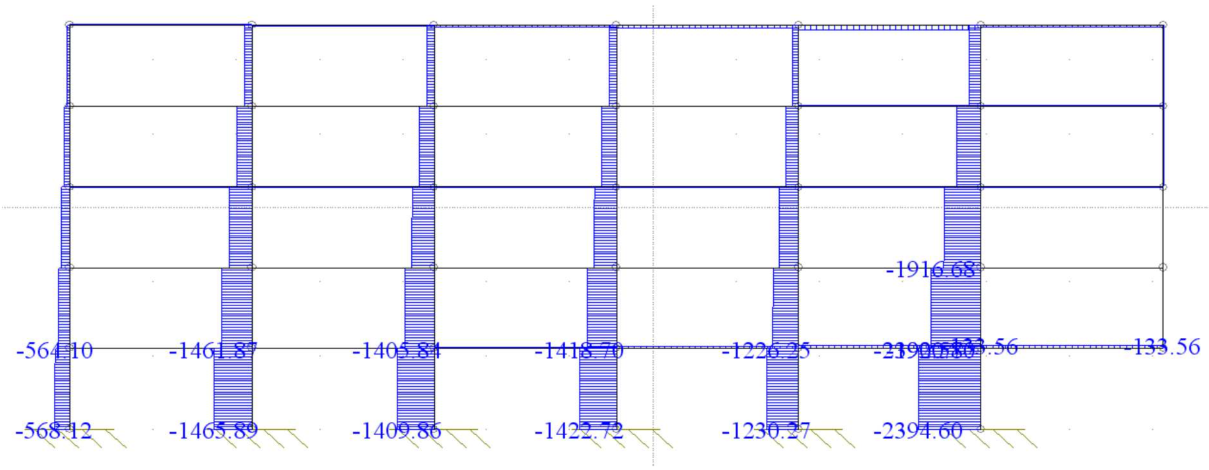
The bridging method is associated to an elastic analysis of the frame. Indeed, considering the bridging method based on a full non-linear analysis would amount to applying the so-called alternative load path method and this would not be a “simplified” approach any more… The alternative load path method is the subject of the second part of this exercise (see Part B).

## 4.2. Analysis of the structure

Column 2 is an external column of an internal primary frame. The diagrams representing the distribution of major axis bending moment and normal force in the primary frame from which column 2 has been removed are given in Fig. 7. These internal forces result from a first order elastic analysis of the structure submitted to the accidental combination of loads. A second order elastic analysis of the frame has also been performed and the second order effects have been shown to be negligible.




*(a) Bending moment (kN.m)*




*(b) Normal force (kN)*

*Fig. 7: Internal forces in the primary frame from which column 2 has been removed*

## 4.3. Verification of the structural elements

### 4.3.1. Members

➢ Beam just above the lost column

Once column 2 has gone, the beams of the directly affected part of the structure are not vertically supported any more at one end. It can be observed that the bending moments are higher in the second beam of the directly affected part (from the bottom), which is also subject to a very low tension force. The verification of this beam stability is made in the next section.

The beam which is just above the lost column is subject to compression. Consequently, it has to be checked under the interaction of bending and compression although the bending moments are smaller than in the upper beam. In order to see the influence of the compression force on the beam stability, the verification is first made under both bending and compression and the results are then compared to the case where the compression force is neglected.

The internal forces in the considered beam are:

- $N_{Ed} = 134\ kN$
- $M_{Ed,y} = 700\ kN.m$ (moment distribution according to Fig. 7)
- The shear forces can be neglected

The node at the top of the lost column is laterally fixed thanks to the braced secondary frame. The torsional rotation of this cross-section of the beam also remains prevented. The fact that the vertical displacement of the studied beam right end is not restrained any more has no influence on the lateral torsional buckling behaviour of the beam. However, it has a great influence on the buckling length of the beam about its major axis which is much increased.

Consequently, the LTB parameters can be considered as follows:

- $L_{LT} = 7\ m$
- $k_z = k_w = 1{,}0$
- $C_1 = 3{,}3$ and $k_c = 0{,}58$ are computed for the considered moment distribution (uniformly distributed load with $\psi = -0{,}38$, $\mu = -0{,}35$ and $M/M_0 = 1/\mu = -2{,}87$)

The minor axis and major axis buckling lengths are taken equal to:

- $L_{fl,z} = L = 7\ m$
- $L_{fl,y} = 1{,}473.L = 10{,}31\ m$ ($\eta_1 = 0{,}4706$ and $\eta_2 = 0{,}5161$ – for HEB300 columns and IPE550 beams)

The equivalence coefficient is taken equal to $C_{my,0} = 1{,}0$.

*Conclusion*: IPE550 in S235 steel is not OK:

- Y-Y: $1{,}25 < 1{,}0$
- Z-Z: $0{,}78 < 1{,}0$

Simply using S355 steel instead of S235 would solve the problem and a profile IPE550 could still be used ($0{,}91 < 1{,}0$). If the steel grade S235 is kept, a profile IPE600 is needed ($0{,}97 < 1{,}0$).

If the compression force is neglected and the beam checked for stability to LTB only, the following results are obtained:

- For IPE550 S235: $1{,}07 > 1{,}0$ → not OK
- For IPE550 S355: $0{,}78 < 1{,}0$ → OK
- For IPE600 S235: $0{,}85 < 1{,}0$ → OK

Comparing these results with the ones got above taking account of the compression force, it can be noticed that the latter is not negligible although it is rather small. The fact that the major axis buckling length is great because the right end of the beam is not vertically supported contributes to increase the influence of the compression force on the element stability.

➢ Beam of the directly affected part which is subject to the higher bending moments

The beam which is subject to the higher bending moments is also subject to a small tension force. However, this tension force is negligible and the member is thus checked under bending alone ($M_{Ed,y} = 720\ kN.m$, moment distribution according to Fig. 7).

The LTB parameters are considered as follows:

- $L_{LT} = 7\ m$
- $k_z = k_w = 1{,}0$
- $C_1 = 3{,}15$ and $k_c = 0{,}61$ are computed for the considered moment distribution (uniformly distributed load with $\psi = -0{,}53$, $\mu = -0{,}34$ and $M/M_0 = 1/\mu = -2{,}95$)

*Conclusion*: IPE550 in S235 steel is not OK:

- Y-Y: $1{,}12 < 1{,}0$
- Z-Z: $0{,}59 < 1{,}0$

Simply using S355 steel instead of S235 would solve the problem and a profile IPE550 could still be used ($0{,}83 < 1{,}0$). If the steel grade S235 is kept, a profile IPE600 is needed ($0{,}88 < 1{,}0$).

It is interesting to notice that the lower beam governs the design although it is subject to lower bending moments, due to the compression force it is subject to in addition (even though this force might seem rather low).

➢ Column adjacent to the lost one, at the first storey

The internal forces to be considered for the verification of the adjacent HEB300 column at the first storey are (see Fig. 7):

- $N_{Ed} = 2395\ kN$
- $M_{Ed,y} = 157\ kN.m$ (linear distribution of moments with $\psi = -0{,}31$)
- $M_{Ed,z} = 0\ kN.m$
- The shear forces can be neglected

For the considered linear distribution of moments, the following parameters can be computed:

- $C_1 = 2{,}118$ and $k_c = 0{,}698$
- $C_{my,0} = 0{,}719$

The buckling lengths are taken equal to:

- $L_{fl,y} = L_{fl,z} = 0{,}7.L = 2{,}45\ m$

*Conclusion*: HEB300 in steel S235 is OK:

- Y-Y: $0{,}96 < 1{,}0$
- Z-Z: $0{,}88 < 1{,}0$

➢ Column adjacent to the lost one, at the second storey

The column of the second storey has to be checked too although the compression force it sustains is smaller, because its buckling lengths are greater. They are taken equal to:

- $L_{fl,y} = L_{fl,z} = L = 3,5\ m$

The internal forces to be considered for the verification of the adjacent HEB300 column at the second storey are:

- $N_{Ed} = 1921\ kN$
- $M_{Ed,y} = 133\ kN.m$ (linear distribution of moments with $\psi = -0,94$)
- $M_{Ed,z} = 0\ kN.m$
- The shear forces can be neglected

For the considered linear distribution of moments, the following parameters can be computed:

- $C_1 = 2,6$ and $k_c = 0,61$
- $C_{my,0} = 0,572$

*Conclusion*: HEB300 in steel S235 is OK:

- Y-Y: $0,79 < 1,0$
- Z-Z: $0,76 < 1,0$

### 4.3.2. Joints

➢ External joints

The maximum bending moment an external joint has to sustain in the considered exceptional situation is $M_{Ed} = 382\ kN.m$ (sagging). It is associated with a negligible shear force ($V_{Ed} = 18\ kN$). The external joints that were initially designed to resist the loads corresponding to the "normal" combination and above all to be rigid are sufficiently resistant.

It is interesting to notice that the external joints of the directly affected part are subject to sagging bending further to the loss of column 2 while they were subject to hogging bending in the normal situation. This robustness consideration was already taken into account for the pre-design of the joints: that is the reason why they are symmetrical. Indeed, if they did not have to resist sagging bending, the lower bolt row would not be of any use and the joints would not have been designed with an end-plate which is extended at the bottom part.

➢ Internal joints

The maximum forces an internal joint is subject to further to the static loss of column 2, based on an elastic behaviour, are:

- $M_{Ed} = 720\ kN.m$ (hogging)
- $V_{Ed} = 297\ kN$ (downwards)

This is much higher than the resistance of the pre-designed internal joints ($M_{Rd,jI1} = 334\ kN.m$). For the structure to resist the loads in the considered situation, the joints have to be much reinforced. The re-design of the internal joints is made using the software CoP and considering an IPE550 S355 beam.

A joint fulfilling the resistance requirement is represented in Fig. 8. The geometrical and mechanical properties of the joint components are detailed in Fig. 9 and the joint main characteristics are given in Table 1. It is obvious that for the structure to be robust according to the bridging method applied to the loss of a column, the joints have to be very strong. This might lead to high costs.

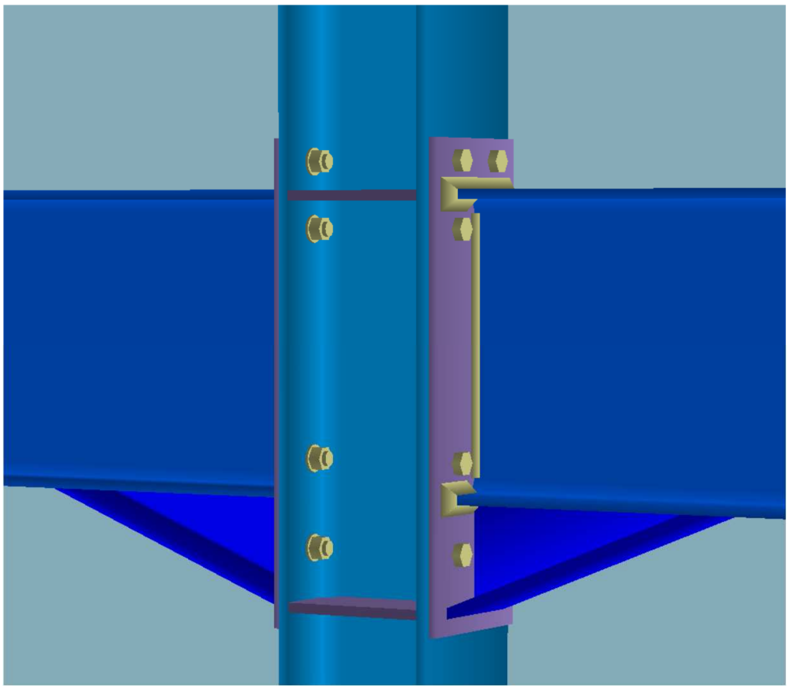


*Fig. 8: Proposed joint configuration for robustness (bridging method – loss of column 2)*

*Table 1: Joint main characteristics*

| | Moment resistance | Shear resistance | Initial stiffness |
|---|---|---|---|
| HOGGING | $M_{j,Rd} = 738{,}0\ kN.m$ | $V_{j,Rd} = 681{,}9\ kN$ | $S_{j,ini} = 721692\ kN.m/rad$ |
| SAGGING | $M_{j,Rd} = 510{,}1\ kN.m$ | $V_{j,Rd} = 681{,}9\ kN$ | $S_{j,ini} = 317370\ kN.m/rad$ |

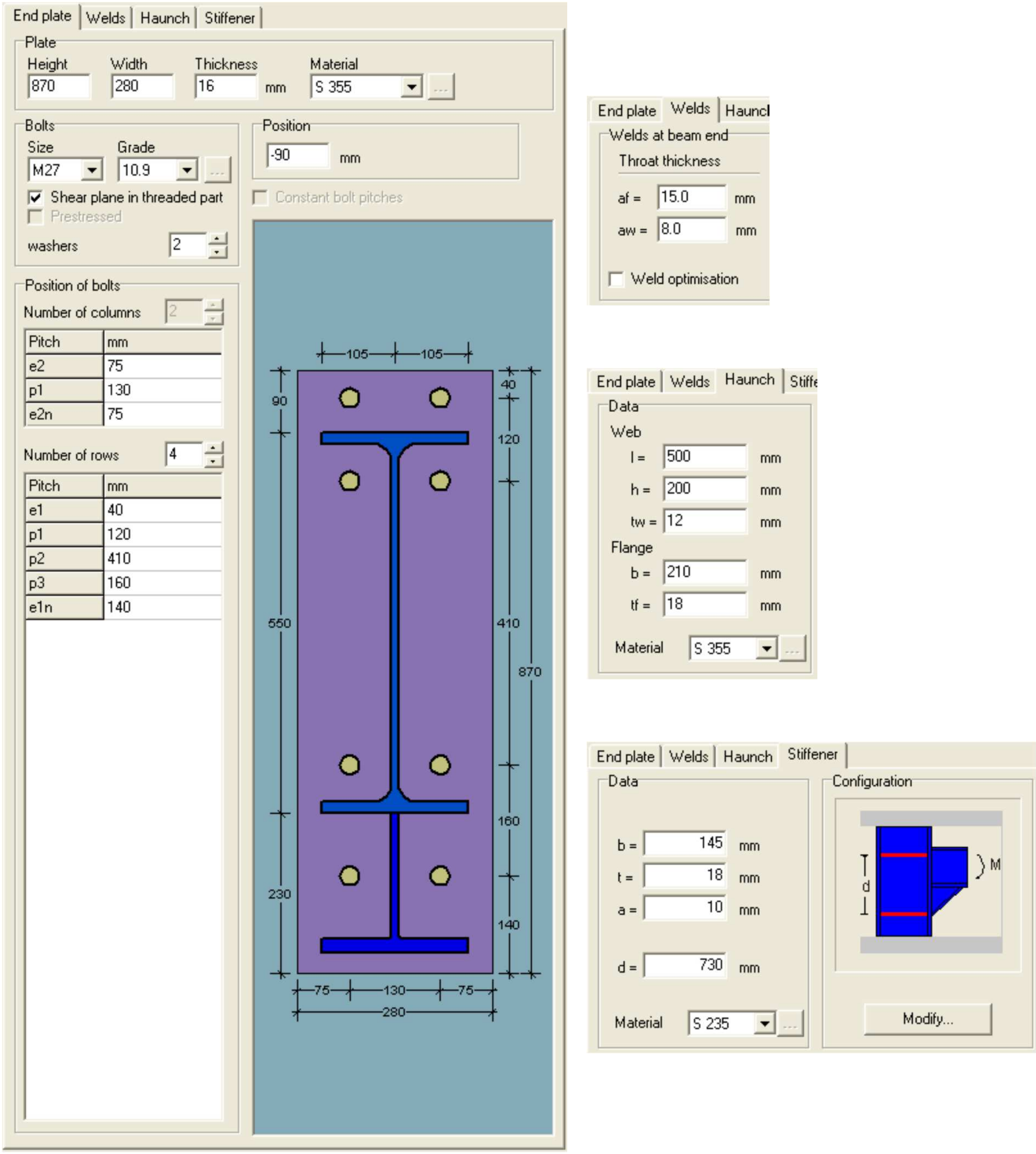



*Fig. 9: Joint detailing*

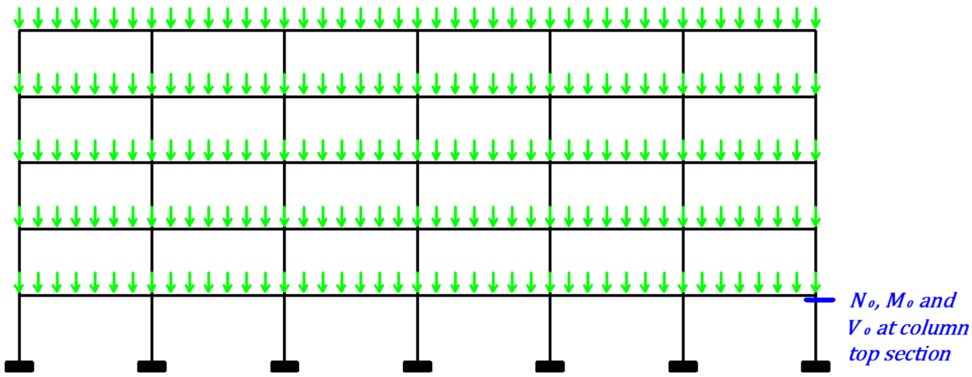
# PART B:

# Application of the alternative load path method

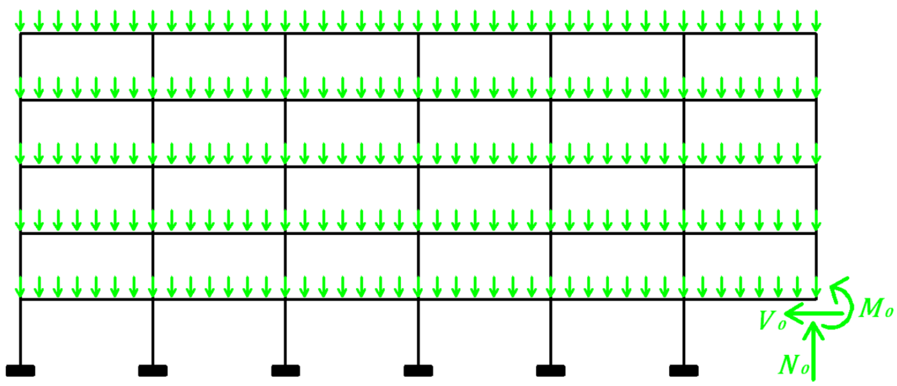
## 1. INTRODUCTION

In this part of the exercise, the alternative load path method is applied to investigate the behaviour of the structure statically losing a column, taking account of both the elasto-plastic behaviour and the second order effects. To study the redistribution of forces in the structure during the loss of a column, the procedure below is followed. It is illustrated in Fig. 10 for column 2 but the procedure is the same for any other column (except that $M_0$ and $V_0$ can be neglected for the internal columns – see Fig. 1).

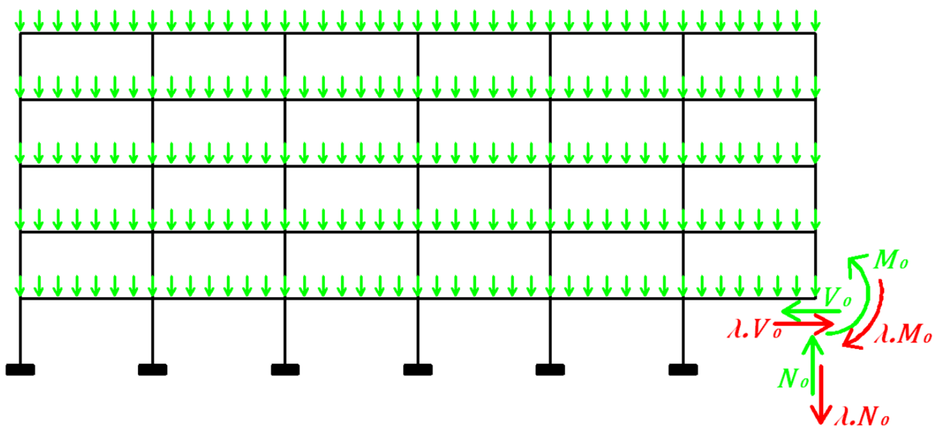
- Step 1: The undamaged structure is first studied in the initial situation and the internal forces at the top of the column which is meant to disappear are recorded.
- Step 2: The structure is then modelled without the damaged column and the initial situation is reproduced in this model by applying loads at the cut level equal to the internal forces that were found before (step 1) at the top of the considered column.
- Step 3: The static loss of the column is then simulated by applying static loads opposite to the forces applied at step 2. The removal of the damaged column is completed when the value of these loads reaches the value of the initial internal forces at the top of the considered column ($\lambda = 1$).




*(a) Step 1: analysis of the initial structure*




(b) Step 2: reproduction of the initial situation in the structure model from which the damaged column has been removed




(c) Step 3: simulation of the static removal of the column

*Fig. 10: Analysis procedure to simulate the static removal of column 2*

Simplified manual approaches will first be applied in section 2 with the aim to highlight the development of stabilising membrane effects in the beams of the directly affected part of the frame after the loss of a column. Indeed, these beneficial second order effects play an important role in the behaviour of a frame structure losing a column. This will also be shown at a second stage by investigating the redistribution of forces in the case of a realistic situation through numerical geometrically and materially non-linear computation.

# 2. SIMPLIFIED MANUAL APPROACH – TRANSVERSAL PLANE – COLUMN 2

## 2.1. Introduction

In this section, basic applications of the alternative load path method are performed based on hand computations. The loss of column 2 is considered and the redistribution of the vertical force $N_0$ this column was initially supported is investigated. This redistribution is assumed to develop in the transversal frame only while the redistribution of $M_0$ and $V_0$ is not taken into account (they act in the longitudinal plane and are assumed to redistribute through the primary frame). The secondary frame is thus investigated under the application of a downward force equal to $N_0$ at the top of the removed column.

### 2.2. Secondary frame braced at both extremities at each floor level

In this first example, the horizontal displacement of each floor level is assumed to be fully restrained at both sides. Consequently, the directly affected part can be extracted and studied as shown in Fig. 11.

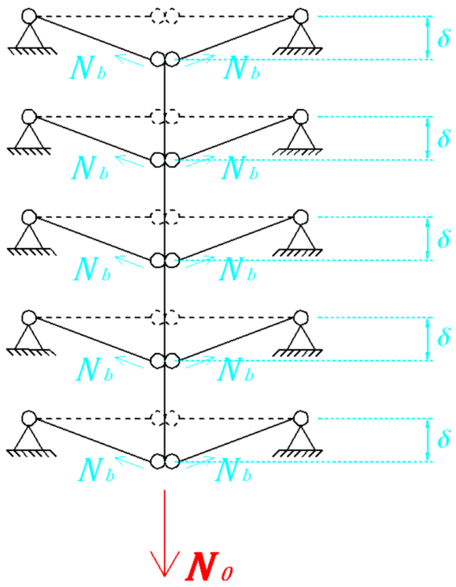



*Fig. 11: Fully braced directly affected part*

From the study of the directly affected part, it can easily be shown that the same tension force develops in all the beams of the directly affected part: each double-beam has the same contribution to sustain the force $N_0$. Consequently, the behaviour of the frame can finally be studied using the sub-system of Fig. 12 submitted to a force $P = N_0/5$.

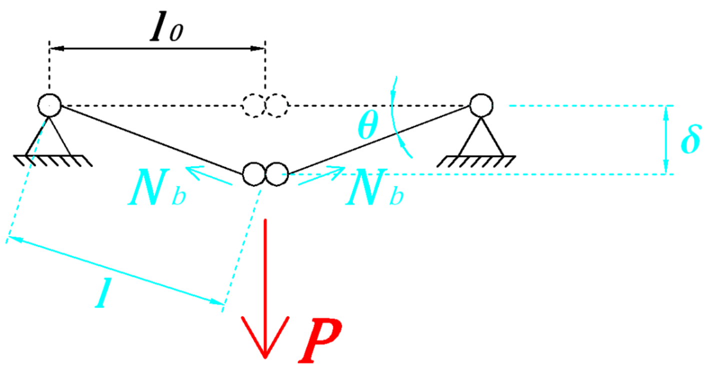



*Fig. 12: Simplified sub-system*

For the sub-system represented in Fig. 12, Eqs. (1) and (2) can be written based on equilibrium and geometrical considerations:

$$P = 2.N_b.\sin\theta \qquad (1)$$

$$l = l_0 / \cos\theta \tag{2}$$

In the elastic range, the elongation of the beams is related to the tension force they sustain as stated in Eq. (3):

$$\Delta l = N_b . \frac{l_0}{E.A} \tag{3}$$

From Eqs. (2) and (3), it comes:

$$N_b = \frac{1 - \cos\theta}{\cos\theta} \cdot E\ A \tag{4}$$

Using Eqs. (1) and (4) and considering $P = N_0/5 = 135{,}5\ kN$, the values of the tension force $N_b$ and the joint rotation $\theta$ can be found, provided the system remains elastic:

- $N_b = 1520\ kN$
- $\theta = 0{,}045\ rad = 2{,}6°$

The tension force in the beams $N_b$ is smaller than the plastic resistance of the IPE360 S235 section $N_{pl} = 1709\ kN$. The assumption of an elastic behaviour is thus valid. The rotation $\theta$ is associated to a vertical displacement $\delta = 0{,}22\ m$.

In conclusion, a final stable state can be reached in the elastic domain provided that the joints have a sufficient rotation capacity and tension resistance and that the columns are able to sustain the increased compression force.

### 2.3. Secondary frame braced at both extremities only at first floor

In this section, the secondary frame is considered to be braced at one side only, except for the first floor which is braced at both extremities. Consequently, the directly affected part can safely be modelled as represented in Fig. 13, where the stiffness of the left external column to transverse horizontal forces is neglected.

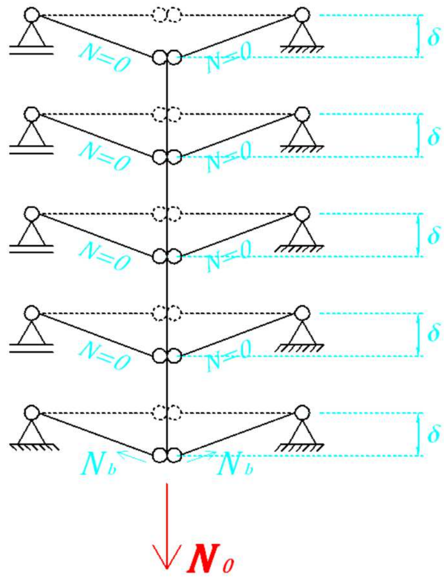



*Fig. 13: Directly affected part braced at both extremities of the first floor*

In such a case, only the lower beam contributes to the redistribution of the load $N_0$ through the secondary frame. The behaviour of the frame can still be investigated using the sub-system of Fig. 12 but it has now to sustain a force $P = N_0$, which is five times more than in the previous case.

Eqs. (1) and (2) are still valid. However, if an elastic behaviour is assumed, the value of the tension force in the beams when a stable state is reached is found to be higher than the beam plastic resistance and is thus incompatible with the elastic hypothesis. That means that Eq. (3) is not valid any more, and neither is thus Eq. (4).

Consequently, the system has to be solved in the plastic range considering that the beam sustains a tension force equal to its plastic resistance and that it can elongate indefinitely. Simply using Eq. (1) in which $N_b = N_{pl} = 1709\ kN$ and $P = N_0 = 677{,}4\ kN$, the value of the rotation $\theta$ can be computed. The associated vertical displacement is easily deduced from geometrical considerations; so is the elongation of the beams (including the axial deformation of the joints).

- $\theta = 0{,}200\ rad = 11{,}4°$
- $\delta = l_0 . \tan\theta = 1{,}01\ m$
- $\Delta l = l - l_0 = l_0 \cdot \frac{1-\cos\theta}{\cos\theta} = 0{,}10\ m$

In conclusion, a final stable state could be obtained in the plastic domain provided the system has a sufficient deformation capacity to reach the deformed configuration described above. In particular, the ductility of the joints has to be great enough.

## 2.4. Conclusions

### 2.4.1. Analysis

In the considered examples, it has been shown that a final stable state can be reached provided the deformation capacity of the system is sufficient. Obviously this stabilisation can only be found based on a second order analysis. Indeed, if a first order analysis of the system is made, it can easily be observed that it is not stable. When a second order analysis is performed, the vertical displacement of the application point of the load $P$ induces the development of membrane forces in the beams. These tension forces can equilibrate the load $P$ provided the deformation of the system and the associated membrane effects can increase in such a way that Eq. (1) is fulfilled. In the first studied example, this stable state could be reached in the elastic range. On the other hand, it has been observed that the development of plastic deformations was required in the second case.

From these basic examples, it is clear already that the robustness assessment of a structure using the alternative load path method should be carried out based on a second order analysis since the second order effects play a major role. Moreover, a materially non-linear analysis is usually required because a final stable state can only seldom be reached without local yielding of the system. Indeed the internal forces developing in a structure under exceptional events such as the loss of a column are very different from those existing in normal situations and the elastic capacity of the structure is usually exceeded. However, this is not necessarily a bad thing. Indeed, the development of local plastic deformations increases the global deformation of the system permitting the development of beneficial second order effects (membrane forces in the beams) which become significant only if the displacements are sufficient.

### 2.4.2. Resistance and ductility of the directly affected part

The design of a structure at ULS under “normal” loading is essentially governed by the resistance capacity of the structural elements (members and joints).

In case of exceptional events, the global stability of the system under a distribution of forces which is very different from the one observed in normal situations is not only linked to the resistance of the structural elements but also to their deformation capacity. Indeed, it is important for a final stable state to be reached that significant second order effects develop, which only occurs if significant displacements are observed. The global deformation of the system implying local deformations of the structural elements, it is thus required that they have a sufficient ductility.

This was illustrated for the considered examples. The global stability of a frame structure after the loss of a column implies a significant vertical displacement at the top of the damaged column, which requires an important rotation capacity of the joints at the beam ends. Besides, these joints have to sustain a significant tension force, which was not the case in the initial situation (before the column was removed).

### 2.4.3. Resistance of the indirectly affected part and bracing system

The tension forces that develop in the beams of the directly affected part have to be transferred to the indirectly affected part. In the considered basic examples, that was not a problem because the frame (or at least the directly affected double-beam) was supposed to be perfectly braced at both extremities. Consequently, the horizontal forces that were applied to the indirectly affected part of the structure by the directly affected beams were simply supported by the bracing systems (assuming they are able to support these loads).

If the frame is braced at only one side (or not braced at all), horizontal forces have to be sustained by the unbraced indirectly affected part. These forces will cause important bending moments in the columns of the unbraced indirectly affected part. In the present case, there is only one adjacent column to support the horizontal force. Besides, it is bent about its minor axis in this plane. Consequently, a plastic mechanism will rapidly form in the unbraced directly affected part (Fig. 14) and the force $P$ the frame is able to sustain is very low (much smaller than $N_0$).

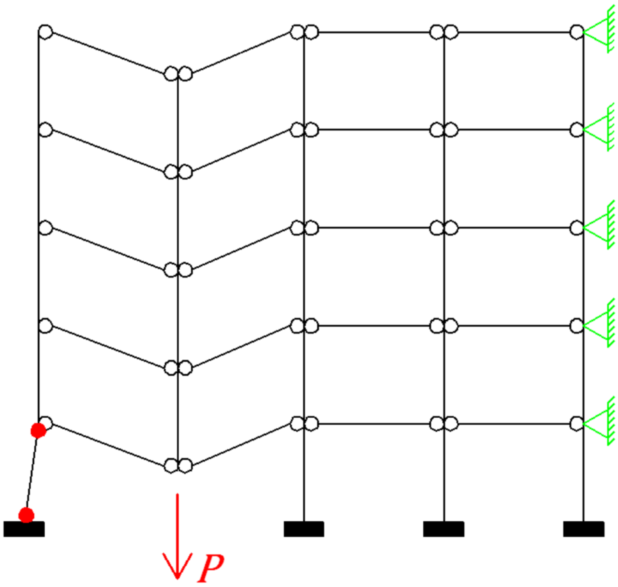



*Fig. 14: Plastic mechanism in the indirectly affected part*

In conclusion, if the secondary frames are braced only at one extremity or if they are not braced at all, their contribution to the redistribution of the forces in the 3D structure further to the loss of a column is negligible. That is the reason why the robustness of the structure is assessed considering only the contribution of the primary frame in the next sections.

# 3. NUMERICAL APPROACH – LONGITUDINAL PLANE – COLUMN 3

## 3.1. Introduction

In this section, the behaviour of the structure further to the loss of column 3 is investigated. The contribution of the secondary frame and 3D effects are neglected; so only the primary frame is studied. This investigation is based on a numerical geometrically and materially non-linear analysis. The procedure described in section 1 (of part B) is followed to study the redistribution of forces in the frame during the static loss of column 3. The particularity is just that the considered column (column 3) is not subject to bending (nor shear) in the initial situation.

First, the robustness of the pre-designed structure is assessed in 3.2. It will be shown that the frame do not remain globally stable further to the loss of column 3. That is why modifications of the structure are suggested in 3.3 to ensure the robustness of the structure under the considered scenario.

## 3.2. Study of the pre-designed structure

### 3.2.1. Robustness assessment

The redistribution of forces in the structure further to the loss of column 3 is investigated through a numerical second order elasto-platic analysis of the primary frame. In the initial situation, column 3 supports a compression force equal to $N_0 = 1409\ kN$ at its top section. This situation is reproduced in the model of the structure from which column 3 has been removed by applying an upwards vertical force equal to $N_0$ at the top of the lost column. Then, the static removal of the failing column is simulated by applying a statically increasing force $P = \lambda . N_0$ at the same point, in the opposite direction (downwards). If the frame remains globally stable until $\lambda = 1$, which corresponds to the complete loss of the column, the structure is considered as robust.

The graph of Fig. 15 represents the evolution of the vertical displacement $u$ at the top of the failing column versus the load factor $\lambda$. It can be noticed that the frame becomes unstable before the column has been completely removed (when column 3 has lost 82 % of the force it was initially sustaining). Consequently, the pre-designed structure can not be considered as robust.

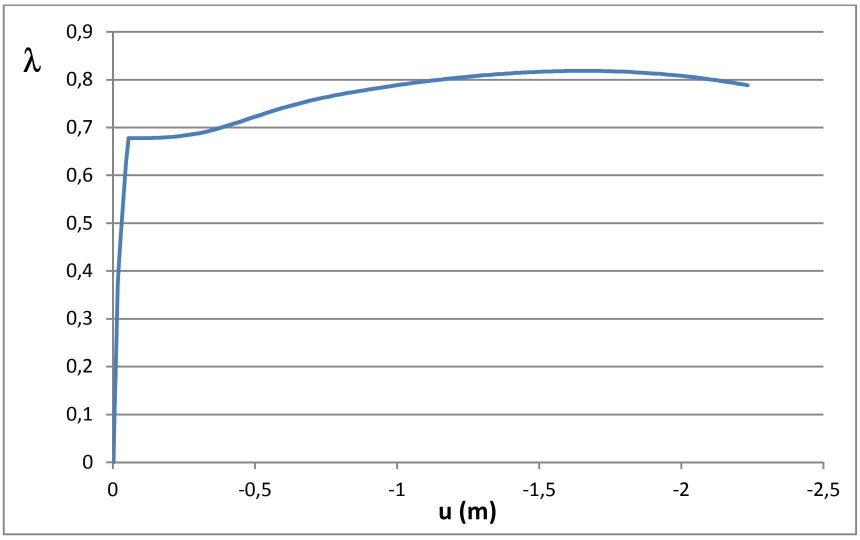



*Fig. 15: Vertical displacement at the top of the failing column versus load factor*

### 3.2.2. Behaviour of the frame during the column loss

The behaviour of the frame during the column loss can be decomposed in the following stages, which can easily be observed in Fig. 15. During the first phase, the frame behaves elastically. The bending moments at mid-length and at the extremities of the double-beams of the directly affected part progressively increase. When the moment at a beam end reaches the value of the resistant moment of the joint, which is smaller than the plastic moment of the beam, a plastic hinge forms in the joint.

The plastic hinges form first in the joints at the extremities of the double-beams of the directly affected part under hogging bending (the joints are designed to be ductile). These hinges appear quasi simultaneously at all floors, for $\lambda = 0{,}37$ (change of the slope in the curve of Fig. 15 due to the diminution of stiffness resulting from the formation of plastic hinges). This is the beginning of phase 2 during which the frames goes from a fully elastic behaviour (end of phase 1) to the formation of a global beam plastic mechanism in the directly affected part of the structure (start of phase 3). This mechanism is completed when plastic hinges appear at mid-length of the double-beams at each floor of the directly affected part under sagging bending. As it can be observed on the graph of Fig. 15, this happens when the force column 3 supports has decreased by 68 % ($\lambda = 0{,}68$), which corresponds to $P = 955\ kN$.

This value of $P$ causing the formation of the plastic mechanism in the directly affected part can easily be analytically predicted (see Eq. (5) and Fig. 16).

$$p \cdot l \cdot \frac{\delta}{2} \cdot 2 \cdot 5 + P \cdot \delta - N_0 \cdot \delta + PP_{col} \cdot \delta = 20 \cdot M_{pl,jI1} \cdot \theta \quad \Longrightarrow \quad P_{pl} = 955\ kN \qquad (5)$$

Where:

- $p = 39{,}79\ kN/m$ is the uniformly distributed load on the beams
- $PP_{col} = 16{,}07\ kN$ is the self weight of the columns above column 3
- $M_{pl,jI1} = 334{,}1\ kN.m$ is the resistant moment of the internal primary joints
- $l = 7\ m$ is the length of the primary beams
- The displacement $\delta$ and the rotation $\theta$ are defined in Fig. 16

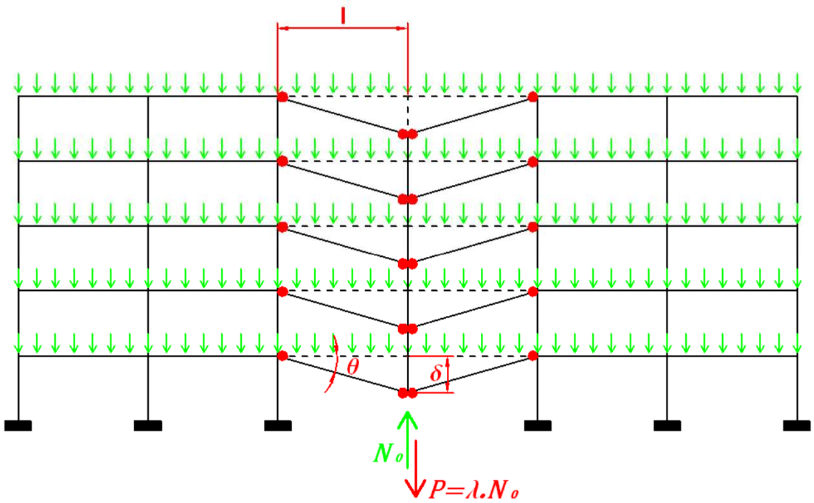



*Fig. 16: Beam plastic mechanism in the directly affected part of the frame*

When the plastic mechanism has formed in the directly affected part, the vertical displacement at the top of the failing column rapidly increases due to the loss of bending stiffness in the joints of the directly affected part. As the displacement increases, the second order effects become significant. In particular, tension forces develop in the bottom beams (see Fig. 18). The axial stiffness of the beams is activated due to these membrane effects and the deformation rate progressively decreases until yielding starts to develop in the indirectly affected part ($\lambda = 0{,}71$).

Indeed, due to the tension forces developing in the directly affected beams, horizontal loads are applied to the indirectly affected part. This causes plastic hinges to form at the top and at the bottom of the indirectly affected part first storey columns. As these plastic hinges form one after the other, the stiffness of the indirectly affected parts subject to horizontal forces progressively decreases, implying the increase of the deformation rate. The failure mode corresponds to the formation of a plastic mechanism in the indirectly affected parts for $\lambda =$

0,77 (Fig. 17); but the load $P$ can still increase a little as explained below. Finally, the frame becomes unstable before column 3 has been completely removed ($\lambda_{max} = 0{,}82 < 1$).

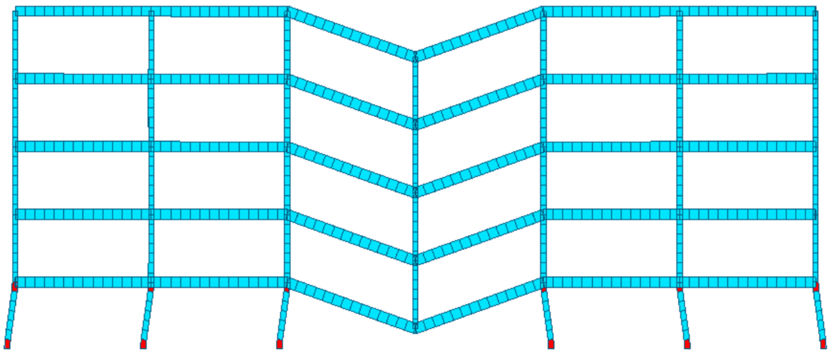


*Fig. 17: Plastic mechanism in the indirectly affected parts of the frame*

The graph of Fig. 18 shows the evolution of the normal load in the lower beams of the directly affected part. It can be observed that the tension force in the bottom beams is maximal when the plastic mechanism forms in the indirectly affected part ($\lambda = 0{,}77$). After that, the indirectly affected parts can not sustain additional horizontal forces and the structure starts to collapse. The normal force in the lower double-beam begins to decrease. However, the load $P$ continues to increase a little thanks to geometrical effects. Indeed, the vertical displacement at the top of the lost column and thus the rotation of the joints increases in such a way that the vertical projection of the tension loads in the bottom beams increases even though the value of $N$ is decreasing; and the horizontal projection of this tension load $N$ in the beams decreases so that it does not exceed the capacity of the indirectly affected part.

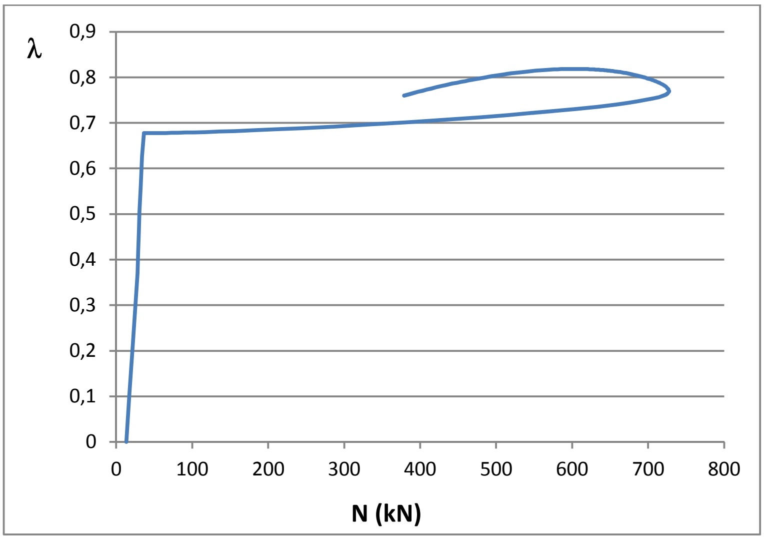



*Fig. 18: Tension force in the lower beams of the directly affected part versus load factor*

### 3.2.3. Flexural behaviour versus membrane behaviour

➢ Definition and description of the flexural and membrane behaviours

Investigating the behaviour of a frame during the static loss of a column, it has been highlighted that two main behaviour types can operate in the directly affected part for the structure to sustain the column removal and to redistribute the forces.

The first behaviour type is called flexural behaviour and is related to the bending of the directly affected part beams. These beams and the joints at their extremities contribute to the support of the load $P$ induced by the column loss thanks to their bending resistance. This is the only behaviour type which is activated during phases 1 and 2, i.e. before the global plastic mechanism has appeared in the directly affected part.

The second behaviour type (membrane behaviour) is related to the development of significant tension forces in the directly affected beams. These membrane effects constitute an additional contribution to sustain the column loss: the vertical projection of these tension loads equilibrate a portion of the force $P$. This membrane behaviour is a "second order" behaviour. Indeed, the activation of this behaviour type requires significant displacements for tension loads to develop in the directly affected beams. This behaviour permits the increase of the load $P$ above the plastic plateau corresponding to the formation of the plastic mechanism in the directly affected part (phase 3). This was also the only behaviour that permitted to reach a final stable state in the example of section 2 that considered the redistribution of forces in the secondary frame, the beams of which are pinned at both extremities so that no flexural behaviour could occur in the directly affected part of the frame.

- Influence of the M-N interaction in the plastic hinges of the directly affected part

In this section, the analysis of the frame losing column 3 has been performed assuming that the resistant moment of the joints remains the same when tension forces develop in the beams and thus in the joints at their extremities. Under this hypothesis the development of membrane effects in the directly affected beams does not influence the flexural contribution of these beams to sustain the load $P$.

To be correct, the joint plastic resistance curve for M-N interaction should have been considered. That means that the plastic moment in a joint should decrease as the horizontal force it is submitted to is increasing. Consequently, the load which is supported by flexural behaviour of the directly affected part should progressively decrease as the membrane behaviour develops, while it was considered to remain constant here despite the development of tension forces in the joints of the directly affected beams.

In the considered example, only the lower double-beam of the directly affected part is subject to significant tension loads. That means that only the four joints at these beam ends would have seen their plastic resistant moment significantly decrease after the formation of the mechanism in the directly affected part if the M-N interaction had been considered to define the joint plastic resistance. The bending moment supported by the joints at the other storeys of the directly affected part would have remained approximately constant and equal to their plastic resistance under bending only.

In other words, the load that is sustained by flexural behaviour of the directly affected part would have decreased as the load supported thanks to membrane effects was increasing, but the maximum decrease would have been 20 %. Indeed, if the tension force in the lower beams reached the tensile resistance of the joints, the bending moment supported by these joints would come to zero. But the bending moment in the joints at the four upper storeys would still contribute to support the load by flexural behaviour. Here, it was shown that the frame becomes unstable due to the formation of a plastic mechanism in the indirectly affected parts far before the tension force in the bottom beams reaches the plastic resistance of the joint under tension only.

## 3.3. Modifications to be provided to ensure the robustness of the structure

### 3.3.1. Possible modifications of the frame aiming at improving the robustness

In order to improve the behaviour of the frame in such a way that it remains globally stable further to the static loss of column 3, it is possible to act on two main characteristics: the resistance of the joints and the resistance of the columns. The joint bending resistance influences the flexural behaviour of the directly affected part while the column resistance influences its membrane behaviour.

If the bending resistance of the joints is increased, the directly affected part of the frame will be able to resist a higher force $P$ before the plastic mechanism forms. Consequently, the plateau of the curve representing the evolution of the displacement $u$ versus $\lambda$ (or $P$) will be situated at a higher level ($P_{pl}$ is increased). However, the membrane effects developing after the formation of the plastic mechanism in the directly affected part are not influenced by the modification of the joint resistance. Indeed, the analysis is performed here assuming the bending resistance of the joints remains constant despite the development of tensile forces (the M-N interaction is neglected).

As observed in the previous section (see 3.2.2), the development of tension loads in the directly affected beams is limited by the appearance of a plastic mechanism in the indirectly affected parts. It is thus possible to improve the membrane behaviour of the directly affected beams by increasing the column resistance so that the indirectly affected parts could sustain a higher horizontal force. Such a modification might also influence the flexural behaviour of the directly affected part because the joint resistance may be increased when more resistant columns are used (some joint components are part of the column).

Obviously, it is also possible to increase both the resistance of the joints and the resistance of the columns. Different options are investigated in the following paragraphs. Fig. 19 shows a graph comparing the force-displacement curves corresponding to different slightly modified frames which are studied in sections 3.3.2, 3.3.3 and 3.3.4.

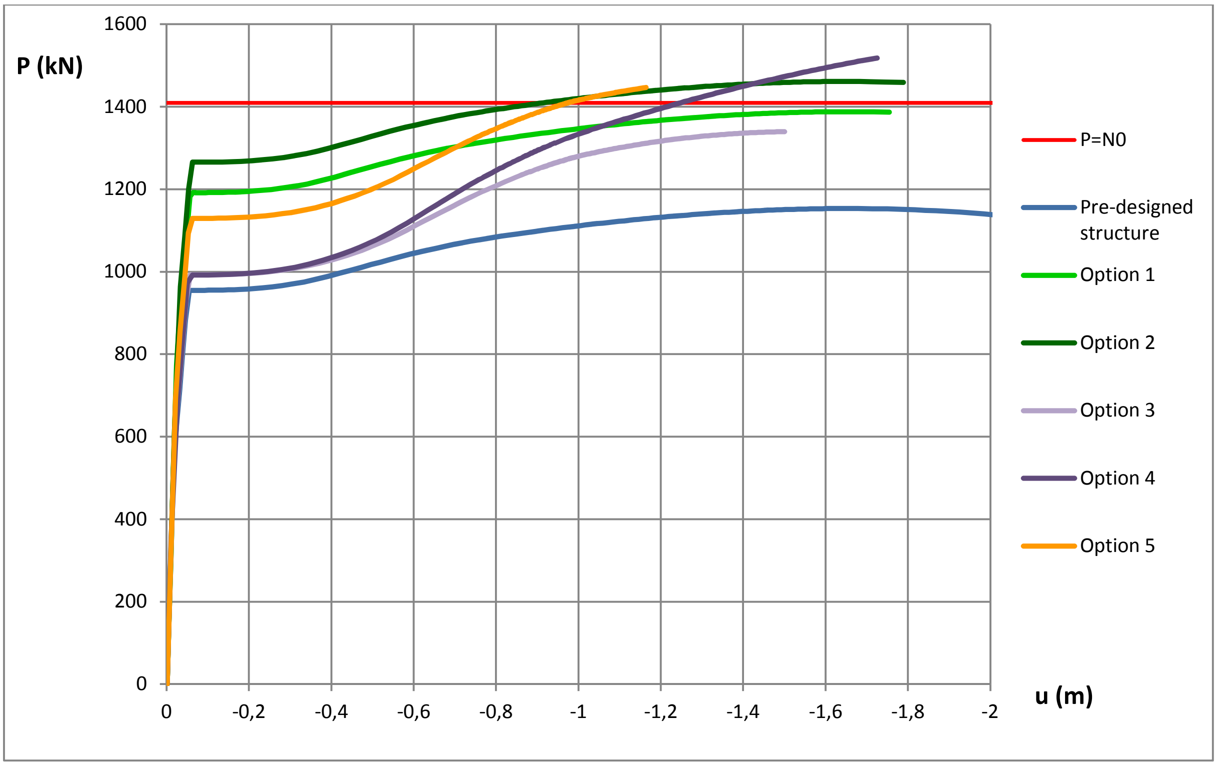



*Fig. 19: Comparison of the (u,P) curves corresponding to different options*

### 3.3.2. Reinforcement of the joints only

Only the internal joints need to be reinforced when the loss of column 3 is considered as the external ones are not part of the directly affected part of the structure. It has to be checked that the re-designed joints have a sufficient rotation capacity for plastic analysis (ductile failure mode) and are rigid.

- Option 1: internal joints "I-2a"

The first modification of the internal joints which is studied simply consists in using S355 steel instead of S235 steel for the end-plate. The resistant moment of the joint is then increased to $M_{Rd,jI2a} = 417{,}0\ kN.m$. It can be observed in Fig. 19 that it is not enough for the structure to be robust.

- Option 2: internal joints "I-2b"

In option 2, the joint end-plate is still made of S355 steel but its thickness is increased to 20 mm (instead of 16 mm) and bolts M30 are used (instead of M27). The joint bending resistance is now $M_{Rd,jI2b} = 442{,}9\ kN.m$. It is sufficient for the structure to remain globally stable after the static loss of column 3 ($P_{max} > N_0$).

#### 3.3.3. Reinforcement of the columns only

- Option 3: columns HEB300, S355

In option 3, the connections are not modified compared to the initially pre-designed structure. The only modification is that the columns are made of S355 steel instead of S235. The joint resistance is increased due to the fact that the column steel grade has been modified: $M_{Rd,jI3} = 347{,}2\ kN.m$. It can be observed in Fig. 19 that the resistance of the indirectly affected part is not increased enough for the structure to be robust.

- Option 4: columns HEB320, S355

If the columns are now made of HEB320 profiles in S355 steel, the flexural behaviour of the directly affected part remains unchanged compared to option 3 (in particular the value of $P_{pl}$ is the same) because the joint resistance is not modified as the components governing it are not part of the column any more. The membrane behaviour is improved: higher tension forces can develop in the directly affected beams before the mechanism forms in the indirectly affected part. The robustness of the frame is ensured as far as the static loss of column 3 is concerned.

#### 3.3.4. Reinforcement of the joints and the columns

- Option 5: columns HEB300, S355 and internal joints "I-5"

In option 5, both the columns and the joints are modified compared to the pre-designed structure. The column profile is still HEB300 but the steel grade is increased from S235 to S355. The joint modification consists in using a 14 mm thick end-plate in S355 steel instead of a 16 mm thick end-plate in S235 steel (the bolts are still M27). The frame is found to be robust with these simple changes (Fig. 19).

For this last option, the vertical displacement that is reached when the column has been completely removed is a bit less than 1 meter. This corresponds to a joint rotation of about 140 mrad (8°).

## 4. DISCUSSION – LONGITUDINAL PLANE – COLUMNS 5, 4, 2

### 4.1. Column 5

The bending moment $M_0$ and shear force $V_0$ at the top of column 5 in the initial situation are very low and can be neglected. The behaviour of the primary frame losing column 5 is qualitatively very similar to its behaviour during the removal of column 3.

Only internal joints are involved in the directly affected part of the structure, where plastic hinges will form. The flexural behaviour of the directly affected part is thus the same as for the loss of column 3 and the formation of the beam plastic mechanism corresponds to approximately the same value of $P_{pl}$ (see Eq. (5)). On the other hand, the membrane behaviour developing in phase 3 is limited by the appearance of a plastic mechanism in the indirectly affected part of the structure which is composed of two columns only (Fig. 20). This mechanism forms for a smaller value of the horizontal force applied to the indirectly affected part, which means smaller tension loads can develop in the directly affected beams. Fig. 21 below compares the behaviour of the pre-designed frame for the loss of column 3 and 5 respectively ($N_0$ is nearly the same in the two columns in the initial situation).

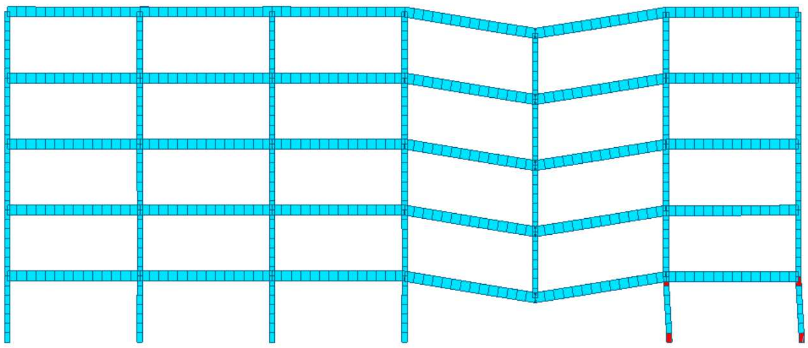


*Fig. 20: Failure mode of the structure due to the loss of column 5*

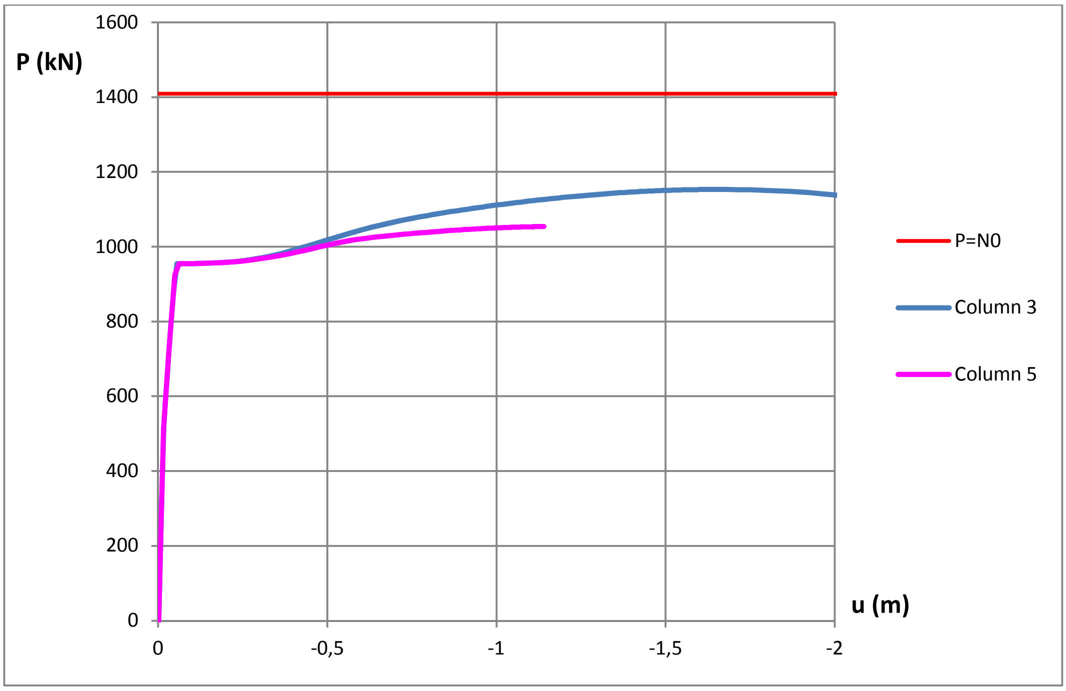



*Fig. 21: Comparison of the (u,P) curves for the loss of column 3 and column 5 in the pre-designed structure*

### 4.2. Column 4

The bending moment $M_0$ and shear force $V_0$ at the top of column 4 in the initial situation are very low and can be neglected. The behaviour of the primary frame losing column 4 is qualitatively similar to its behaviour during the removal of column 3 or column 5.

When the loss of column 4 is considered, the directly affected part of the structure involves 15 internal joints and 5 external joints. The flexural behaviour of the directly affected part is thus different from the two previous cases (removal of column 3 and column 5). In particular, the value of $P_{pl}$ causing the formation of the beam plastic mechanism is higher because the resistance of the external joints is higher than the resistance of the internal joints.

As far as the membrane behaviour is concerned, the tension loads that can develop in the directly affected beams are very limited because the indirectly affected part at one side of the structure is composed of one column only. Consequently, a plastic mechanism forms in this column for a low value of the horizontal force.

### 4.3. Column 2

The case of column 2 is particular because it is an external column. Consequently, the membrane behaviour can not develop as it did when the removal of an internal column was considered. Another difference is that the bending moment $M_0$ and shear force $V_0$ at the top of the column in the initial situation are not negligible.

# CONCLUSION:

# Comparison of the different methods

## 1. TYING METHOD

The tying method is an indirect method, which means it is not based on a particular scenario and thus does not require any structural analysis. It consists in applying simple design requirements. Basically, the beams and beam-to-column joints have to be able to sustain a given tension force in order to constitute efficient horizontal ties.

This method is very easy to apply and leads to increase the robustness of the structure by providing a better continuity. But this method is based on resistance aspects only while no ductility considerations are taken into account. However, it has been shown that the deformation capacity of the structure and thus the ductility of its structural elements (in particular the joints where plastic hinges form) are essential factors on which the robustness of the structure depends. Indeed, the achievement of a final stable state is usually associated with high displacements giving rise to significant second order effects.

Besides, the value of the tension resistance which is required by the tying method seems to be quite unsafe if it is compared to the membrane forces developing in the lower beams of the directly affected part after a column loss. These tensile forces can be estimated using the alternative load path method which is based on a full non-linear analysis and reproduces best the actual behaviour of the frame losing a column.

## 2. KEY ELEMENT METHOD (VEHICLE IMPACT ON A COLUMN)

The key element method is a direct method also known as specific load resistance method. It is based on a particular accidental event and requires the analysis of the structure under the considered scenario. The aim of this method is to design the elements of the structure that might be affected by the considered accidental event in order that they can resist this action.

This method has been applied here considering the collision of a lorry on an external column and using the static equivalent forces suggested in the Eurocode. A simple static first order elastic analysis was thus performed. It has been shown that a column suffering an impact inducing minor axis bending in the direction of normal travel could not resist. Consequently, the columns of the external secondary frames should be reinforced according to this method, in such a way that the considered event does not lead to the failure of the impacted column.

As explained above, the key element method has been applied in this exercise in a “simplified version”, i.e. considering static equivalent forces under which the analysis of the structure is easy to perform. This method could also be applied based on a more sophisticated analysis considering directly the dynamic loading and the possible non-linear behaviour of the structure and material. Obviously such an analysis would be much more difficult to perform.

## 3. BRIDGING METHOD (LOSS OF A COLUMN)

The bridging method is also a direct method. In this case, the considered particular scenario is the static loss of a column (due an unspecified event). The elements around the removed column are most affected by the column loss and have to be designed to sustain this exceptional event. In other words, the frame has to sustain the accidental combination of loads and remain globally stable without the lost column, which means the forces have to redistribute differently within the structure.

This principle is basically the same as for the so-called “alternative load path method”. The difference is that the method called here “bridging method” is a simplified method in the fact that it is based on a first order elastic analysis of the structure. This design procedure leads to stronger structural elements than the alternative load path method, which considers the same scenario (loss of a column) but is based on a full non-linear analysis (much more difficult to perform). In particular, it has been shown that very strong and thus expensive joints should be used for the structure to be robust according to the bridging method considerations (elastic behaviour).

Following this approach, the structure behaves elastically further to the column loss and its deformation remains rather limited (that is why the second order effects are negligible). The robustness is thus in this case essentially a matter of resistance and less of ductility.

## 4. ALTERNATIVE LOAD PATH METHOD

This last direct method has also been applied considering the loss of a column. The difference in comparison to the bridging method is that a geometrically and materially non-linear analysis is made, in order to investigate the real behaviour of the frame much more precisely.

First, an elasto-plastic analysis allows considering the development of successive plastic hinges in the joints until the formation of a plastic mechanism in the directly affected part, while the resistance of the frame based on an elastic analysis is considered to be reached when the first plastic hinge forms. Obviously, for a plastic analysis to be performed, it has to be ensured that the joints where plastic hinges form have a sufficient rotation capacity.

Besides, a second order analysis permits to take account of the stabilising second order membrane forces developing in the directly affected beams when the vertical displacements become significant (after the formation of the plastic mechanism).

Using this method, it has been shown that very small modifications could make the structure robust. In conclusion, performing a full non-linear analysis requires quite sophisticated software and is much more difficult than a simple first order elastic analysis but it leads to a more economical design. However, it is important following this approach to ensure both sufficient resistance and ductility of the structural elements (the joints in particular).