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Robustness of steel structures: consideration of couplings in a 3D structure

Thèse présentée par

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Sergii Kulik

Summary:

"Robustness of steel structures: consideration of couplings in a 3D structure"

According to recent collapse events, robustness becomes necessary to be included into design process. Definitely, point of reference for term of "Robustness" became collapse of Ronan Point in 1968. Further terrorist attacks, other accidental actions and exceptional events were proving importance of the robustness. But which recommendations should be included into design for certain type of structure? This is the question, which still doesn't have an answer. For coming closer to this answer in University of Liege were performed several investigations. By research was defined behavior of 2D frame, which submitted to the loss of supporting column. The research was verified and was developed analytical method, which allows prediction of structural response during loss of the column. First part of current thesis describes importance of necessity of the investigation, shows different examples, makes general introduction.

Purpose of second part is to determine main terms and show which standards and regulations are already developed according to this direction of study. Also this part presents detailed description of recent researches, performed at University of Liege.

Third part describes objectives of current thesis.

Fourth part includes investigation for certain structure. Research is done by different approaches for various conditions of the structure. The aim of quantifying the importance of 3D couplings on the global response of the structure is achieved. Certain conclusions are provided.

Last part of thesis contains general conclusions and perspectives.

Résumé:

"La robustesse des structures métalliques: examen des accouplements dans une structure 3D"

Selon les récents événements de l'effondrement, la robustesse devient nécessaire d'être inclus dans le processus de conception. Certainement, un point de référence pour le terme de “Robustesse” qui est devenu l'effondrement de Ronan Point en 1968. D'autres attaques terroristes, d'autres actions accidentelles et des événements exceptionnels ont prouvé l'importance de la robustesse. Mais quelles recommandations devraient être inclus dans la conception de certains type de structure? Question, qui n'a pas encore une réponse. Pour se rapprocher de cette réponse à l'Université de Liège ont été réalisées plusieurs investigations. Par la recherche a été trouvé le comportement des plans de portiques, soumis à la perte d'une colonne de support. La recherche a été vérifié et a été développé par une méthode analytique, qui permet la prédiction de la réponse structurelle lors de la perte de la colonne.

La première partie de la thèse décrit l'importance de la nécessité de l'investigation, qui présente différents exemples, qui généralise l'introduction.

La raison de la seconde partie consiste à déterminer les principaux termes et de montrer quelles normes et règlements sont déjà développés selon cette direction d'études. Aussi cette partie présente la description détaillée des recherches récentes, effectuées à l'Université de Liège.

La troisième partie décrit les objectifs de la thèse en cours.

La quatrième partie contient l'investigation de la structure particulière. La recherche a été réalisée par des approches différentes pour des diverses conditions de la structure. Le but de quantifier l'importance des accouplements 3D sur la réponse globale de la structure est obtenu. Certaines conclusions sont fournis.

La dernière partie de la thèse contient des conclusions générales et des perspectives.

Notations

Latin letters:

A	: area of cross section
A_{gc}	: area of one section between beams (girder cell)
DAP	: directly affected part
E	: modulus of elasticity, Young's modulus
F_H	: horizontal force acting on the spring K_H
f_y	: yielding stress of material
g	: permanent load, dead load
I	: moment of inertia
I_y	: moment of inertia of major profile axis
I_z	: moment of inertia of minor profile axis
IAP	: indirectly affected part
$I_{y.HE400B}$	: moment of inertia of major profile axis for HE400B profile
$I_{z.HE400B}$	: moment of inertia of minor profile axis for HE400B profile
K_H	: stiffness of the horizontal spring simulating the lateral restraint of the indirectly affected part
K_N	: axial stiffness of a plastic hinge submitted to bending and axial force
$k_{column,y}$	: stiffness of column of major axis
$k_{column,z}$	: stiffness of column of minor axis
$k_{y,i}$	: stiffness of the spring, which models stiffness of the indirectly affected part with major axis profiles orientation
$k_{z,i}$	: stiffness of the spring, which models stiffness of the indirectly affected part with minor axis profiles orientation
L	: length of the plastic hinge (plasticized zones)
L_0	: initial length of the beam
M	: bending moment at the extremities of the beams of the directly affected part

N	: axial force in the beams of the directly affected part
N_{AB}	: compression force in the column
$N_{AB,normal}$	: compression force in the column before it disappears
P	: vertical force, simulating loss of column, numerical method
p	: total distributed loading
q	: live load
Q	: vertical force, simulating loss of column, analytical method
Q_x	: vertical force, simulating loss of column in X-axis 2D frame
Q_y	: vertical force, simulating loss of column in Y-axis 2D frame
r	: ratio between different axis stiffness
u	: vertical displacement at the top of the lost column
W_{el}	: elastic section modulus
W_{pl}	: plastic section modulus

Greek letters:

γ_G	: safety coefficeint for permanent loads
γ_Q	: safety coefficeint for variable loads
δ_h	: horizontal elongation of the spring K_H
δ_N	: axial elongation a plastic hinge submitted to bending and axial force
Δ_L	: elastic elongation of the beams of the directly affected part
Δ_x	: vertical displacement corresponded to force Q_x
Δ_y	: vertical displacement corresponded to force Q_y
ε	: axial elongation
θ	: rotation at the extremities of the beams of the directly affected part
σ	: axial stress

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Part I

General introduction

I General introduction

Life of engineer nowadays becoming more and more various. I mean that with evolution in general, developing of all branches of science and technics, we founding more and more of questions. I don't mean that life of an engineer few hundred years ago was easier, it was limited due to evolution in certain way; metaphorically telling, in the past we had closer limits, but ways, how to achieve them were difficult, nowadays we have easier approaches to achieving of targets, due to developing, but, we have greater amount of them. According to developing in general, new and new branches and fields of engineering appearing; we construct higher, greater, stiffer buildings and structures, and due to this, we open new questions, which need to have an answer, saying another, more we know – more we don't know. Saying specifically about robustness, few decades ago, nobody could even predict the development of such specific branch of engineering, so appeared a new horizon, opened new questions, and I, as one of many, who dedicate themselves to investigation of robustness, I'm looking for answer to questions, which were established to be answered.

I.1 Question of robustness

Questions of robustness arise, when different situations appeared. If building or structure has sudden attack of terrorists, explosion of gas supply systems or suddenly removed structural element due to any unexpected scenario, what will happen in this case, what an Engineer going to answer? If world practice wouldn't know such famous examples of progressive collapse as those, which will be described below and lots of others, who knows, maybe term of "Robustness" wouldn't get so important value.

I.1.1. Importance and examples

The importance of Robustness can be explained even by one factor: if building, which not enough robust, will collapse, it will lead to material losses and, unfortunately, to victims.

As examples, I will show most famous cases in history for last few decades. Lots of examples of partial or total collapse of buildings happened due to gas supplying system explosion. Precisely saying, explosion of gas suddenly removed one structural element like a wall or beam, or column, in some cases, were removed bigger construction items, moreover, this action leaded to progressive collapse.

So, first example, I would like to put it in chronological order, was in Newham, in East London. On the 16th of May 1968, Ronan Point, 23-storey tower block had fatal partial collapse because of explosion of natural gas (Figure 1) [12].

Figure 1 Collapse of Ronan Point [12]

As written in [12], explosion happened when dweller of corner apartment at the 18th floor of the building went into her kitchen, and turn on electricity of the stove on purpose to make cup of morning tea. Spark at the stove produced explosion, which took away load bearing walls. Those walls were supporting four apartments situated above of explosion place. Later it was defined by the explosion survey: the weakest point was at joint of floor slabs to the vertical bearing walls. Flank walls dropped down because of gravity, and floor situated above became unsupported. Due to this fact, south-east part of building was destroyed totally, happened partial progressive collapse of building. Three out of four flats, situated above of explosion place were still free, because building was recently constructed. During the progressive collapse were killed 4 people and 17 were injured, totally inside of building during explosion were 260 dwellers. One young lady was standing on the narrow ledge, when part of the living room flew away. Amazingly, but the owner of the unfortunate flat survived. The partial collapse of Ronan Point produced sufficient influence to changes in the regulations and codes for buildings. After collapse in the 1970, first of these came with the 5th Amendment to the Building Regulations. The Government put temporary measures to ensure integrity and safety of the buildings in case of explosion. By this document, all of buildings, which have more than 5 floors and constructed after November 1968, should satisfy requirement to be capable to resist force of 5 lbs. per square inch, psi, produced by explosion (35 kPa). Buildings, which were already constructed, were allowed to satisfy requirement to resist force of only 2.5 psi, but in case if gas supply system would be replaced by electrical cooking and heating equipment. Gas supply was removed from Ronan Point and other eight blocks.

Second example of failure was in Hartford, due to excessive weather actions. As was mentioned in [12] the Hartford Civic Center Coliseum collapsed early in the morning on January 18th, 1978 [12]. This facility was often hosting different hockey games and concerts; just one hour before the collapse this facility was occupied with around 5 thousand of viewers. Winter of 1978 gave larges snow storm in 5 years, moreover, big snowfalls are unusual for this place in Connecticut, USA. Overloaded by snow, space roof frame deflected for 25 m in the center of arena. Result – is progressive collapse (Figure 2). As well, during the design and constructing of this project were made lots of mistakes, but, even with overloading and wrong performing of design, this structure was in use for 5 years, until big snowfall. So, as we can see, there are few different factors, as mistakes of design and constructing, overloading, which collected to scenario, leaded to progressive collapse. But, about scenarios we will talk precisely later.

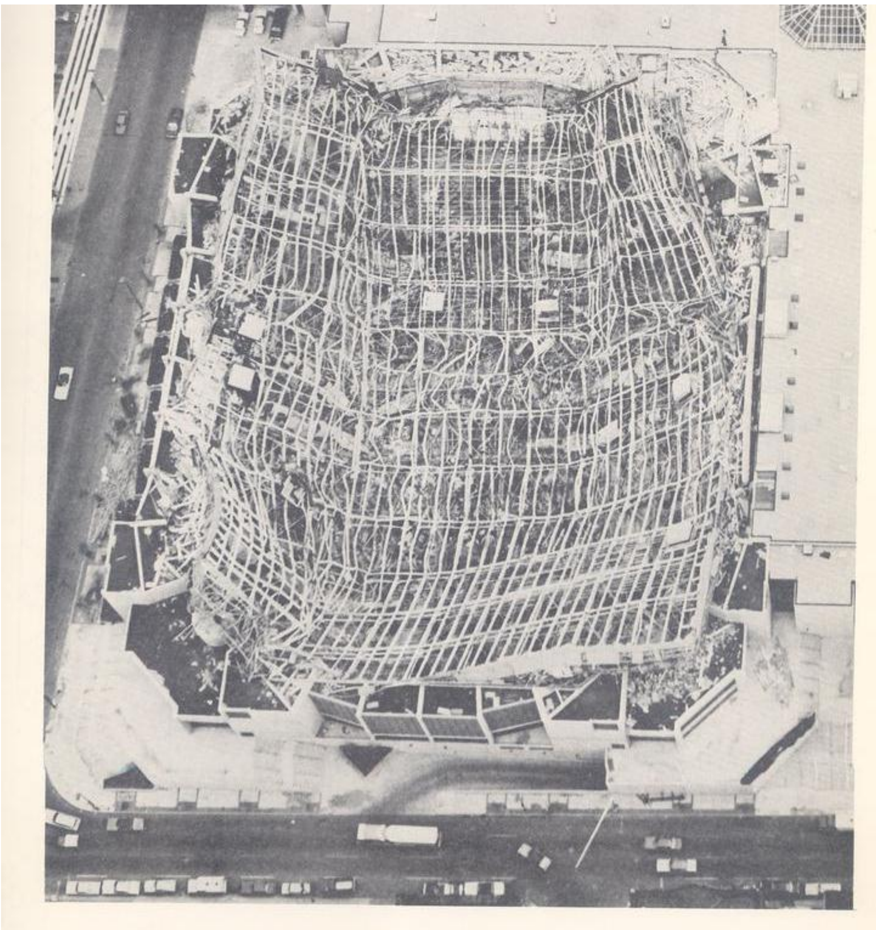

Figure 2 The Hartford Civic Center Coliseum collapse [12]

The situations, when safety factors used as traditional and methods used as nontraditional, never should be ignored. But, it should be understandable, that large errors in design couldn't be compensated just by increasing of safety factor. Sometimes, using of different software as calculation tool is like a black box, and more, computer is just an analytic tool, which allows you to simplify difficult computations. Person, who inputs to computer any data for design, should be experienced and know all information, which is given. Generally saying about this case, it was notable confusion with a number of design and construction issues, which contributed to the progressive collapse and possibly was avoided. After the collapse of roofing system for certain structure in Hartford, constructing of similar structures wasn't stopped. Different applications of truss structures were used in many different places in the world. Even it is known case, when similar structure collapsed, it happened in Kemper Arena in Kansas City.

The third incident happened also according to weather, but with different structure and different scenario. It happened with The Schoharie Creek Bridge over the Schoharie Creek near Fort Hunter in New York State on April 5th, 1987 [12].It collapsed due to bridge scour at the foundations after a record rainfall (Figure 3). The collapse killed ten people.

Figure 3 Schoharie Creek Bridge collapse [12]

As it is written in [12], supporting system of bridge were providing by pier frames and abutments from each side. Piers were constructed from two slightly tapered columns with tying beams. Fixing of columns was done with lightly reinforced plinth in shallow place. The footing spread had to be protected with a layer of dry riprap. Conditions of weather which were not estimated produced collapse. During snowmelt was rainfall, which together made effect of 50 years flood. Firstly, it collapsed only one pier, which led to collapse of span between collapsed and stable pier. Then, within 1.5 hours collapsed another pier, because of which also collapsed other span, and finally, last pier and span were shifted, but with 2 hours later. Here is example of quite “long term” progressive collapse, when collapse took few hours. After collapse and calming of flood, was performed an investigation according to this case. It was suggested, that second pier collapsed due to wreckage of first pier. Felt down span between those piers partially blocked water flow of the river. This produced redirecting of flow and, obviously, velocity was increased for part of flow, which streamlined another pier. For place, where flow velocity was increased, it was later defined, that soil under foundation of pier was erodible. High velocity of flow started to penetrate into bearing layers of soil. And here was made imperfection in performing of foundation works. The left part of footing wasn't fulfilled with riprap stone. Here we can observe that overlay of factors such as floods (extreme weather condition), not correctly performed parts of structure (riprap) and bad soil conditions into certain scenario led to collapse. The progressive collapse of Schoharie Creek Bridge provided motivation for providing development and improvement for design procedures of bridges and careful approach for performing of construction process.

The fourth example, what I wanted to describe happened in the Seocho-gu district of Seoul, South Korea. The Sampoong Department Store collapse was a structural failure that happened on June 29th, 1995 (Figure 3) [12]. This collapse is the largest peacetime disaster in South Korean history. As was written in [12] Beginning of construction of the Sampoong Department store was in 1987. Construction was performed by The Sampoong Group (Trade Company in South Korea). Firstly it is necessary to emphasize, that previously construction site was used as landfill. Then, project was designed as residential building with 4 floors. Then, purpose of building was changed to store, but it was done already during construction process.

Figure 4 The Sampoong Department Store collapse [12]

Change of the main purpose of the building led to necessity of cutting few bearing columns, for installation of escalators. Completing of the building was in the end of 1989. After the day of opening, Sampoong Department Store was visited by 40000 of people every day during next 5 years. It is difficult to imagine even significance of difference between designed load and real load. Connection between wings of store was provided by atrium. Then, it was taken decision to build-up extra floor. Construction of skating rink was a purpose of the fifth floor. After, increasing of load from the 5th floor was provided by change of decision to build at the new floor 8 restaurants instead of skating rink. Features of Korean restaurants require heated floor, because guests are sitting on the floor. Heating system provide extra loading. And as last point, ventilation unit was installed on the roof of building. Load from this unit was 4 times more, than designed. 3 years after exploitation of building in this regime, some why was taken decision to move ventilation unit. Moving was done across slim roof, there cracks started to appear. Distribution of cracks soon achieved visible size at the ceiling of 5th floor. After amount of cracks increased unbelievably, but customers were not evacuated from building. It just was closed last floor and turned off ventilation system. Managers of the store didn't want to close the store due to financial losses. At day of collapse amount of customers was unusually high. Nevertheless, chiefs and manager left the building because of safety purposes. Few hours before collapse, at the top floors were heard several loud sound, slabs were widening further. Distribution of cracks achiever width of 10 cm, it wasn't already possible to stop progressively coming collapse. Owner still didn't want to evacuate people. Only when cracking started to produce loud sounds, workers of center put alarm signal on, but it was already too late. Main columns of building weakened, and south wing packed like card house. During time of 20 seconds all the columns were destroyed. More than 1500 customers and workers were trapped, 502 killed. Financial losses resulted by 160 million euros. Here we see factors as overloading, not corresponded maintenance, fatigue, which combined to some scenario led to total collapse.

The fifth incident will demonstrate different factor of collapse. This exceptional event happened on 19th April 1995. Reinforced Alfred Murrah Federal Building collapsed in Oklahoma City because of the bomb detonation.

From [12] is known that as bomb was used truck, which arrived to the building from southern façade. That cysteine track was rented and fulfilled with 3000 kg of ammonium nitrate, nitro methane ad diesel fuel. Certain solution of liquids makes strong explosive material. Detonation of the track destroyed one third of the building and caused severe damage to several other buildings located nearby. Explosive wave sheared 3 floors of building from columns. Connection of third floor to main transfer beam was destroyed. By this fact was caused collapse of all vertical element (columns), and 4th and 5th floors fell down. Progressive collapse due to terrorist attack in Oklahoma became first example of building bombing on territory of the USA. During explosion and progressive collapse of this building were killed 168 people and around 700 were injured.

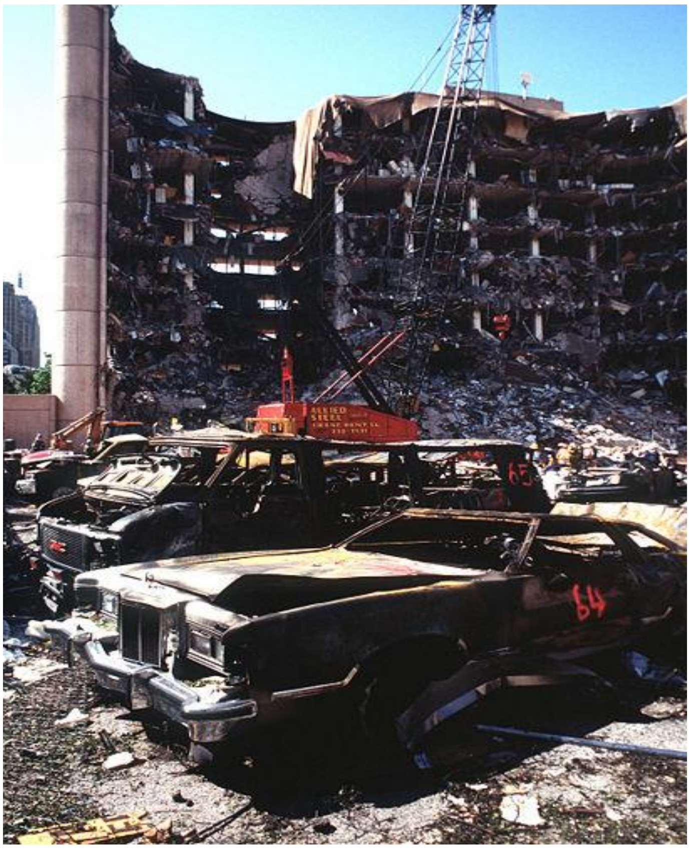

Figure 5 Alfred Murrah Federal Building collapse [12]

As conclusion to this part I would define, that importance of questions of robustness, which diametrically opposite to meaning of progressive collapse, shouldn't be underestimated. I collected such different examples of incidents to show the importance and magnitude of covered situations. Different factors, which can be combined into different scenarios, should be taken into account during design procedure. But as it is impossible to predict everything, even when possible to consider majority of cases and situations, we have to follow certain approach: "Building or structure could be damaged, but shouldn't be collapsed".

Part II

State of the art

II. State of the art

In this part I would like to describe or, saying another, summarize knowledge, which known nowadays, corresponded to term of “Robustness”. Actually, to make overall vision on all databases would be quite difficult, then, in this part is considered only partial state of the art, but which will highlight basics and recent research, done in University of Liege, and make link between them.

II.1. Standards and design methods

On present for regulations of design procedures already developed regulations and design methods in many countries. This is based on experience, which those countries of the world had, what leaded to necessity to regulate and develop design methods.

II.1.1. Standards

For the part of standards it is known and approved to use such famous to all regulations as Eurocode with National Annexes (those, which are corresponded to European Countries) and different Codes of other countries.

II.1.1.1 Eurocode

In the Eurocode 1 — Actions on structures —Part 1-7: General actions — Accidental actions [8] is written the important definition of robustness, which gives us certain limits and understanding about research or design field.

Robustness – 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.

According to accidental design situations, which can appear, it is given table (Figure 6), where described strategies based on different approaches. Normally it is two strategies of accidental design situations:

- First-one based on identified accidental action

- Second-one based in limiting the extent of localized failure

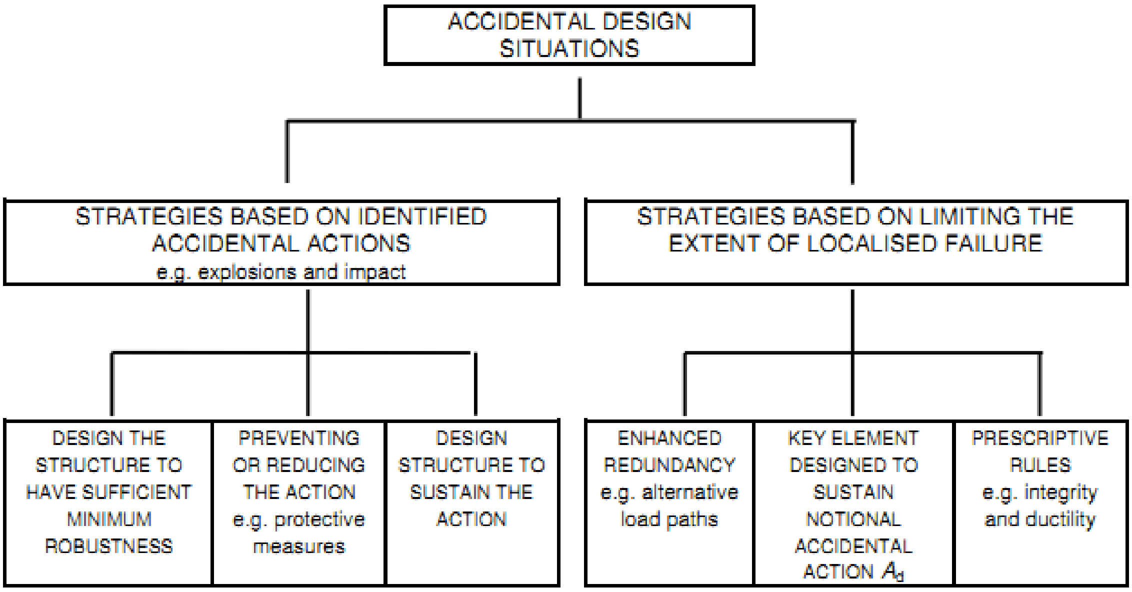

Figure 6 Accidental design situations. EN 1991-1-7

Also this document provides row of notes, which makes certain explanation for an exceptional design situations and regulate them:

- The strategies and rules to be taken into account are those agreed for the individual project with the client and the relevant authority.
- Accidental actions can be identified or unidentified actions.
- Strategies based on unidentified accidental actions cover a wide range of possible events and are related to strategies based on limiting the extent of localized failure. The adoption of strategies for limiting the extent of localized failure may provide adequate robustness against those accidental actions.
- Notional values for identified accidental actions (e.g. in the case of internal explosions and impact) are proposed. These values may be altered in the National Annex or for an individual project and agreed for the design by the client and the relevant authority.
- For some structures (e.g. construction works where there is no risk to human life, and where economic, social or environmental consequences are negligible) subjected to accidental actions, the complete collapse of the

structure caused by an extreme event may be acceptable. The circumstances when such a collapse is acceptable may be agreed for the individual project with the client and the relevant authority.

II.1.1.2 US codes

For the United States were developed certain amount of regulative documents [9], which corresponded to robustness. Main idea for document of The US General Services Administration "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" [6] is to reduce potential of progressive collapse.

In the document "Minimum design loads for buildings and other structures" [7], which developed by American Society of Civil Engineers is defined, that progressive collapse of building should be avoided, when structural member could be damaged.

II.1.1.3 Other regulations

In Russian Federation in 2009 by Central Scientific Institute of Industrial Buildings was developed document "Prevention of progressive collapse of reinforced concrete monolithic structures" [10], where main objectives according to progressive collapse is that structural system of the building should not be subject to progressive collapse in the case of local failure of certain structural elements in accidental situations not covered by the terms of the normal operation of the building. This means that at a particular combination of loads it is allowed local destruction of the individual elements of the structural system of the building, but this destruction should not lead to the destruction of other structural elements. Preventing of progressive collapse of the building should be provided. As a hypothetical local fracture should be considered destruction within one (each) alternately one floor of the building (each) of the column (pylon) or limited portion of the walls. Displacements, crack openings, deformations are not limited.

II.1.2.Methods

According to development of regulations it was also developed some design methods. In general, methods could be divided for two families, indirect and direct methods. Those, which are described below are corresponded with Eurocode.

II.1.2.1 Indirect methods

The tying method is in group of indirect methods because it isn't based on some specific scenario and doesn't require structural analysis. This method concludes just design requirements. By this method each assembly must resume a certain pulling force. The recommended minimum value for the tying force is equal to 75 kN. This means that the various elements of a structure must be sufficiently bonded to each other; various elements can be steel members, steel rebars in concrete slabs, steel mesh and hollow ribs in composite floors when they are directly connected to steel beams though studs, beam-column joints as well should provide transferring of tying force. Presence of ties provides increasing of structural continuity and redundancy. As imperfection of method it is possible to consider that ductility is not taken into account. As well, tension resistance could be unreliable compare to developed membrane forces in beams after losing of column in directly affected part. Estimation of reliable tensile forces can be achieved through alternative load path method, which will be described later.

II.1.2.2 Direct methods

Direct methods means that there is defined scenario. Normally scenario is defined in a conversation of designer with investor or owner of building; this discussion is intended to identify all possible risks, what owner wants to protect from. It is almost impossible to design building which will withstand to all terrorist attacks or all scenarios. For established scenario there are possibilities: "bridging method", "key element method" and "alternative load path method".

In case of “bridging method”, the considered specific scenario is the static loss of a column, and event of column’s losing shouldn’t be specified. The recommendation is that structure must have capability to bridge lost element and place of collapse should be localized and limited. The procedure of design concludes notional removing each untied element and checking if affected zone doesn’t distribute further than until adjacent stories and limited by 15% of area of certain storey or $70m^2$. “Bridging method” similar to “alternative load path method”, but difference is that it is simplified until consideration only first order elastic analysis of the structure.

The “key element method” is done when it’s not possible to bridge over the place, where element is missing (column), and in case of this method with difference of “bridging method”, accidental event has to be specified. The purpose of this method is to design structure in that way, that if certain element of structure will be subjected to accidental event, it will resist to this action. For example, this certain “key element” has to be design in way to oppose to accidental impact of the track, car or gas explosion. According to research of Ronan Point, it was established to put additional load of 34 kN/m^2 to a member which should be designed as protected. In EN 1991-1-7 defined amount and placement of concentrated force, this simulates accident of track or car.

For the “Alternative load path method” [13] the scenario is defined as well and there column disappears (can be achieved due to impact, fire, explosion, etc.). The main goal of this method is to understand, if structure with a lost element redistributes stresses to the rest of the elements, but not make strengthening of structure. Difference with “bridging method” is taking into account non-linearity of geometry and materials. Non-linear analysis is providing more precise achievements of real behavior of the structure. Creating and development of plastic hinges are considered with elasto-plastic analysis. For performing of plastic analysis joints has to have enough of rotational capacity. Second order analysis allows taking into account stabilizing second order membrane forces, which are developing in directly affected part after creating of plastic mechanisms. For this method, for performing of non-linear analysis is required to have powerful finite element software.

II.2. Recent development at University of Liege

For last few decades world saw a lot of terrorist attacks and killing natural catastrophes. This highlighted the necessity of buildings to resist of exceptional loads. For the Eurocode structural integrity of civil engineering structures should be ensured, but often there are no any practical guidelines, which explain how to reach this point.

In University of Liege (further called as ULg) work a team of professors and engineers for questions of robustness [11]. For current time there is already made research for certain questions according to robustness. Main works provided by Professor Jean-Pierre Jaspart [5], Professor Jean-François Demonceau [1], Professor Nguyen Nam Hai Luu [11] and PhD student Clara Huvelle [2], [14].

II.2.1. Introduction

As was written before, term of robustness means, that structure should be defined as capable to maintain global structural integrity when one of its supporting elements is damaged. Saying other words, structure should be designed in that way, that if one element will be damaged, it shouldn't lead to progressive collapse. One of possibilities how to reach this point is to activate alternative load paths, during missing of one of the structural elements. In ULg for research of robustness is considered specific scenario of "loss of the column" under an exceptional event. This scenario is considered for steel and composite plane frames and investigated already for several years. Approaches for investigation are numerical, analytical and experimental.

II.2.2. General philosophy (global approach)

General philosophy, what research team in ULg follows, is to understand how redistribution of the loads goes for structure, which has been damaged. When it is known how stresses redistribute, it is understandable if other elements are capable to resist for an additional loading, which is coming from redistribution.

A PhD thesis of Demonceau [1] was finalized on topics for development of an analytical method, which allows predicting behaviour of 2D frame, which losses a column. At present, method is completed.

Frame, where column is lost, will have two different parts, according to behaviour. First one is directly affected part (DAP), where beams and columns situated exactly above of the lost column. Second one is indirectly affected part (IAP) is rest of the structure (Figure 7).

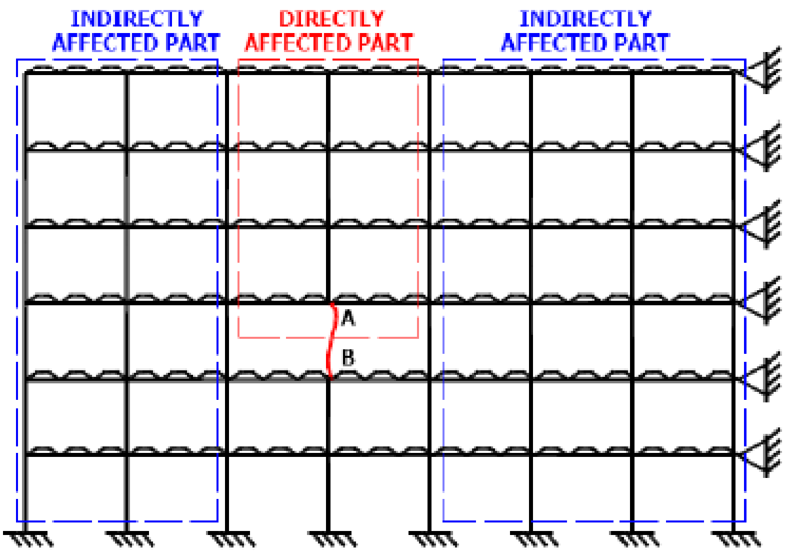

Figure 7 Directly and indirectly affected parts [2]

For the frame which will lose a column, on Figure 7 is column AB, we can observe behaviour of this element. Behaviour is described by diagram (Figure 8) which shows evolution of compression force N_{AB} in certain element AB versus vertical displacement u at level of top of column.

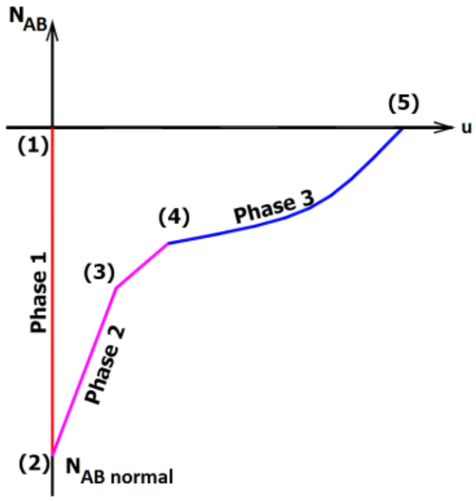

Figure 8 Interrelation between compression force N_{AB} and vertical displacement u [1]

During the phase 1 (from point (1) to point (2) on Figure 8), the column is loaded by loads from upper storeys (Figure 7).

The Phase 2 (from point (2) to point (4) in Figure 8) begins when the column starts to disappears. During the phase 2 (Figure 9), a plastic mechanism develops in the directly affected part. Each change of slope in the curve of Figure 8 corresponds to the development of a new hinge in the directly affected part, until reaching a complete plastic mechanism (point (4) on Figure 8).

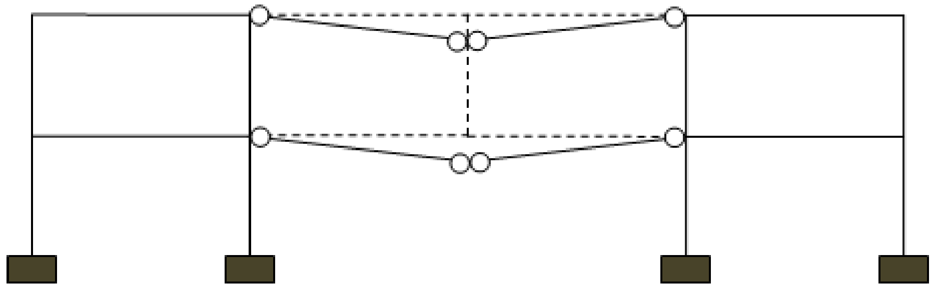

Figure 9 Phase 2 [2]

Phase 3 (from point (4) to point (5) on Figure 8) starts when this plastic mechanism is formed: the vertical displacement at the top of the column continues to highly increase since there is no more first order rigidity in the structure. Due to these large displacements, catenary action can develop in the beams of the directly affected part, giving new second-order stiffness to the structure. The role of the indirectly affected part during the phase 3 (Figure 10) is to provide a lateral anchorage to these catenary actions: stiffer the indirectly affected part, more catenary action will develop in the directly affected part.

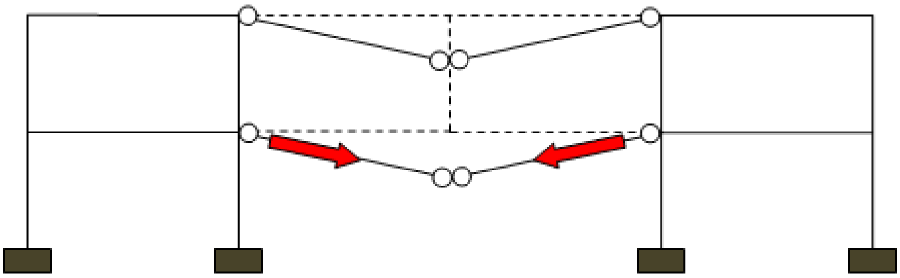

Figure 10 Phase 3 [2]

The loss of the column is simulated by applying a vertical force P going downwards (Figure 11): when P is equal to 0, the column is still in place, and when $P=N_{ABnormal}$, the column is assumed to be completely removed.

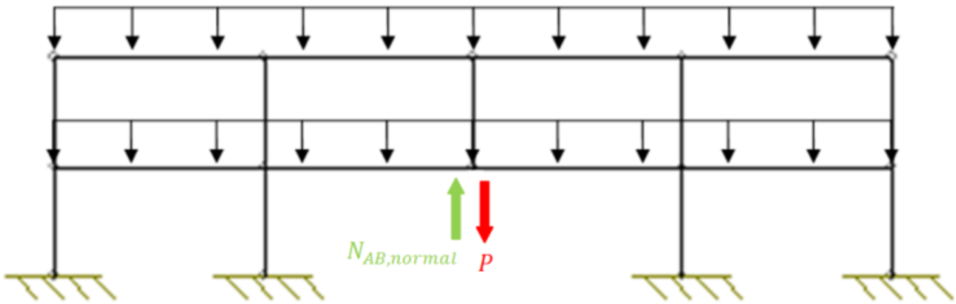

Figure 11 Simulating of loss of the column [2]

The final aim of the analytical method developed in Liege is to determine analytically curve P-u reflecting the behaviour of the structure during phase 2 and 3, to know if the structure is able to reach point (5), i.e. when $P=N_{ABnormal}$. Indeed, this point is reached only if there is enough resistance and ductility in the damaged structure to sustain these large displacements and additional forces coming from the activation of alternative load paths.

II.2.3. Analytical model for 2D frame

Analytical model for 2D frame was developed and verified in PhD thesis of Jean-François Demonceau [1] (further called as Demonceau's model). In PhD thesis of Demonceau was developed an analytical method which allows predicting behavior of curve P-u during the phase 3 (Figure 8) for 2D frame structure, statically losing one column. Method is focused on the phase 3, where dominant is second-order effect. Demonceau's model is based on study of substructure (Figure 11), which contains only lower beams of directly affected part (DAP), where the highest tension forces appears The surrounding structure is simulated by a horizontal spring, K_H (Figure 12). This K_H has a constant value in the model, because the indirectly affected is assumed to remain elastic during phase 3.

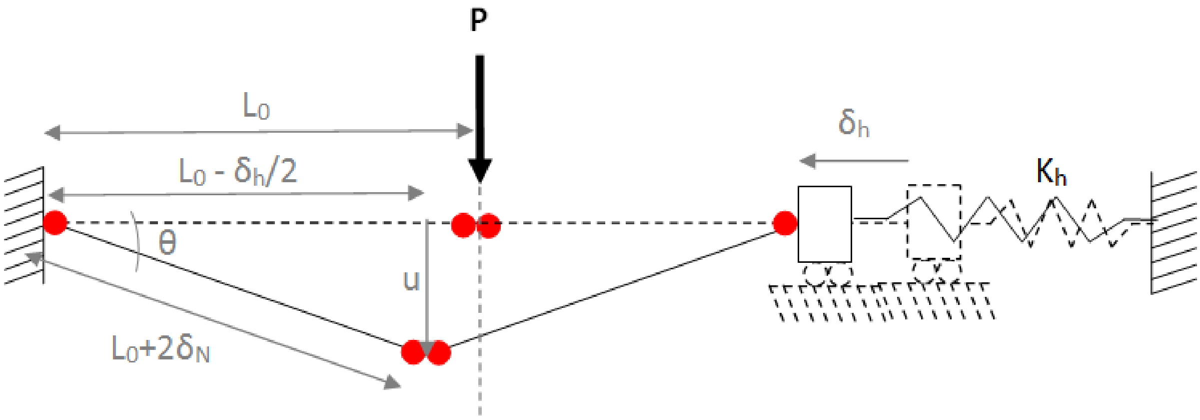

Figure 12 Substructure [14]

The input data for this method are:

- L_0 : initial length of the beam

- MN resistance interaction for both hogging and sagging moment at the plastic hinge level.

- K_H: stiffness of the horizontal spring

- K_N: axial stiffness of a plastic hinge submitted to both bending and axial forces.

During the phase 2, the hinge is only submitted to bending (A-B on Figure 13 left scheme), during phase 3, this plastic hinge is submitted to both M and N (B-C on

Figure 13 central scheme). At the very end of phase 3, this plastic hinge could even only be submitted to N (point C on Figure 13 left scheme). The law between N and δ_N is assumed to be linear and totally defined by parameter K_N (Figure 13 right scheme). This assumption has been validated through numerical and experimental tests.

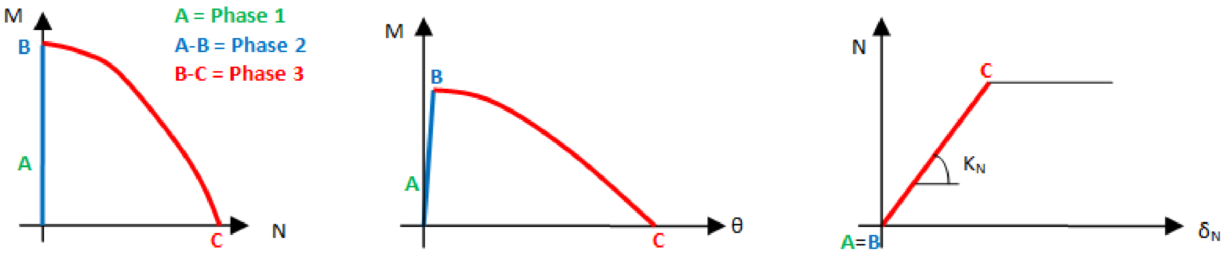

Figure 13 Schematic diagrams [14]

The unknowns and equations obtained from the study of the substructure in Figure 13 are given in Table 1. As the number of equation is equal to the number of unknowns, this system can be solved for different values of u.

Unknowns	Equations	
u	*u - input data*	
θ	$\sin(\theta)=\frac{u}{L_0+2\cdot\delta_N}$	Kinematic theory
δ_h	$\cos(\theta)=\frac{L_0-\frac{\delta_H}{2}}{L_0+2\cdot\delta_N}$	
δ_N	$\delta_h=\frac{F_H}{K_H}$	Static theory
P	$\delta_N=\frac{N}{K_N}$	
N	$M=f(N)$	
M	$-0.25\cdot P(L_0-0.5\cdot\delta_H)+0.5\cdot F_H\cdot u+2M=0$	
F_h	$N=F_H\cdot\cos(\theta)+0.5\cdot P\cdot\sin(\theta)$	

Table 1 Unknowns and equations [14]

In PhD thesis of Demonceau [1], it is demonstrated that this substructure model is able to reflect accurately the response of a frame further to a column loss if the parameters K_N and K_H are appropriately estimated.

As the substructure defined by Demonceau takes into account only one storey of the frame that suffers the column loss, the parameter K_H should reflect the behaviour of all the structure around, i.e. on the one hand, the stories of the directly affected part above the lost column, and on the other hand, the indirectly affected part located beside. However, no analytical procedure was proposed in [1] for this parameter. Also, the parameter K_N was numerically or experimentally estimated for the validation of Demonceau’s model in [1]. Therefore, to have a complete analytical procedure, it is necessary to be able to develop analytical models to predict the values of the parameters K_N and K_H.

After Clara Huvelle joined to the research team, analytical model was improved and automatized. Firstly, she contributed master thesis to the topic: “Contribution to the study of robustness of buildings structures: consideration of the progressive plasticization of the part of the structure “not directly affected” by the exceptional event” [2].

Then, she worked for the new approach [14] to analyse behaviour of 2D frame when column is lost was performed. For definition of an analytical model for the prediction of K_N, it is required to define a length for the plastic hinge. For new approach the cross section was fictively divided into 6 parts: 2 parts represent the flanges and 4 parts the web (Figure 14). Finally, the extremities of the beams of the directly affected part can be considered as 6 springs in parallel submitted to M and N, assuming that the section at the extremities of these springs remains straight, using the Bernoulli hypothesis.

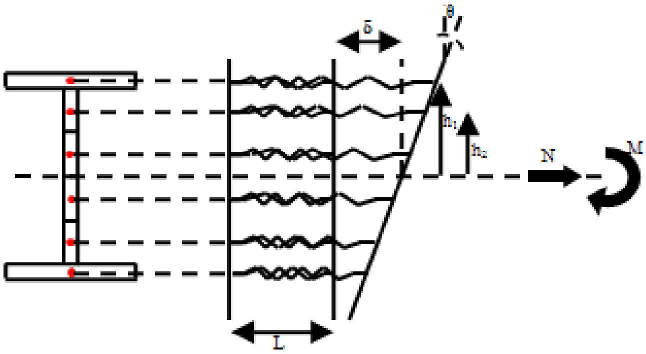

Figure 14 Fictive division of cross section [14]

The input data's of this new substructure model are:

- The geometrical characteristics of the cross section (A, I, W_{el}, W_{pl}, dimensions), also used for the definition of the spring properties simulating the behaviour of the hinges

- L_0, E, f_y, K_H

Unknowns	Equations	
u	u - *input data*	
θ	$\sin(\theta) = \dfrac{u}{L_0 - 2\cdot L + \Delta_L}$	Kinematic theory
δ	$\cos(\theta) = \dfrac{L_0 - 2\cdot L - \delta_H - 2\cdot\delta}{L_0 - 2\cdot L + \Delta_L}$	
δ_H	$\delta_H = \dfrac{F_H}{K_H}$	Static theory
Δ_L	$\Delta_L = \dfrac{F_H \cdot (L_0 - 2L)}{EA}$	
M	$M = \sum F_i \cdot h_i$	
F_H	$F_H = \sum F_i$	
F_i $i = 1\ldots6$	$F_i = f(\delta_i)$	
δ_i $i = 1\ldots6$	$\delta_i = \delta + h_i \cdot \theta$	Kinematic theory
P	$-P(L_0 - \delta_H) + F_H \cdot u + 2M = 0$	

Table 2 Unknowns and equations for improved method [14]

There is no need any more to define a law between M and N, neither between N and δ_N, because they are included in the definition of the stiffness's and resistances of the springs simulating the hinges at the extremities of the beam.

The method developed in ULg allows predicting the response of a frame submitted to a column loss. This method is fully analytical, and there is no input data extracted from experimental simulations. Analytical results are in good agreement with numerical and experimental results, for simple substructure and as well as for complete frames. The developed method takes into account the following phenomena:

- The global interaction between the different parts of the structure;

- The local phenomena happening in the yielded zones, submitted to both M and N.

Program for computation and automation of this approach have been written in program environment of Matlab. This program was used for current thesis research for part of analytical analysis.

II.2.4. Previous development for 3D structures of "Lemaire"

For 2009-2010 study year at ULg studied Florence Lemaire. She investigated one of topics of robustness: "Study of the behavior of 3D steel and composite structures further to loss of a column" [3]. In her work she considered scenario of the loss of column in a 3-dimensional structure. The investigation were aimed to improving of the knowledge about the redistribution of forces into the structure and the development of alternative load paths, taking into account 3D effects, which could influence the structural response. Firstly, she investigated structures, which are made only from steel beams and columns. This case was considered for two possible solutions, according to connection of secondary beams, one as fully pinned, second as fully rigid. For each of situations was extracted substructure from whole 3D structure, to the purpose of checking, if it possible to simulate behavior of real structure with certain level of accuracy. Then, behavior of isolated slab, submitted to concentrated load was investigated.

Investigated structure (Figure 15) constituted from steel elements:

- 6 spans, 7 meters each;
- 4 bays, 5 meters each;
- 5 stories, 3.5m each.
- Internal columns HE 320 B
- External columns HE 360B
- Beams IPE 450
- Profiles are made from S235

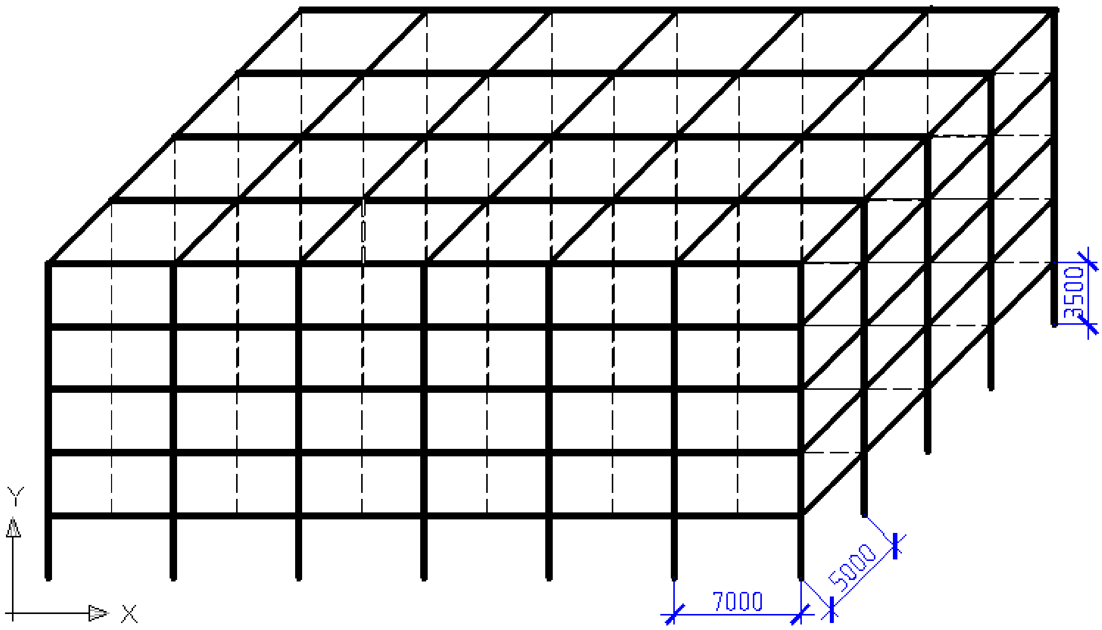

Figure 15 General 3D view of "Lemaire" structure

Then, it was simulated loss of the column. It was done for a central column, on purpose to save symmetric behavior of building (Figure 16).

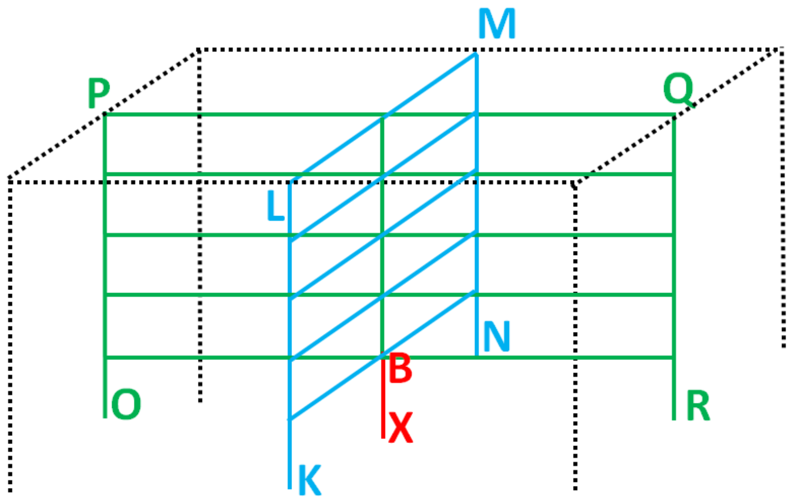

Figure 16 Loss column simulation [3]

Simulation was done by two ways: by numerical and analytical approach. I won't go to details to describe whole process of analytical approach of simulation, but briefly, analysis of behavior of building, when column is lost was performed with analytical method, with consideration of two perpendicular frames of DAP (Figure 17).

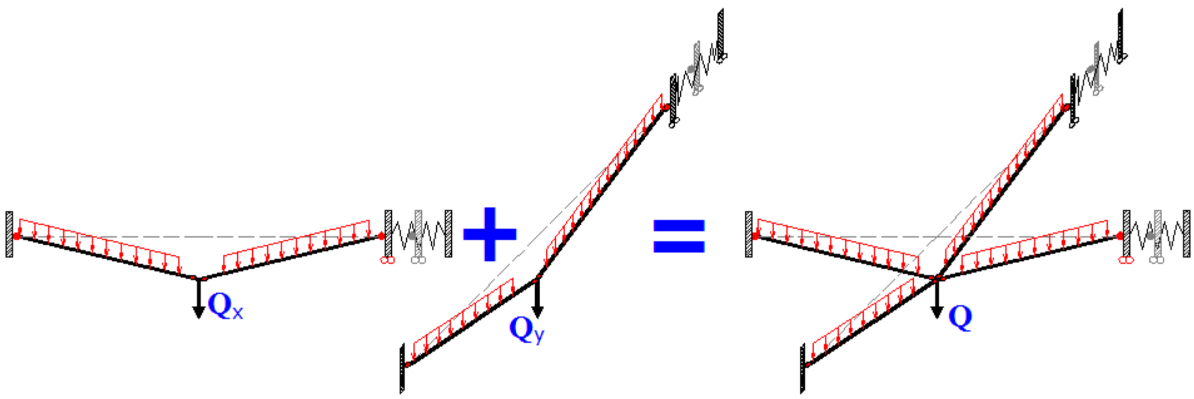

Figure 17 Approach to analytical method

An observed phenomenon was described. The most interesting part of Lemaire investigation for my thesis is comparison of numerical and analytical approaches to simulate loss of column in a building. Comparative results of two methods were presented (Figure 18). On the diagram is presented results of behavior of building depending on vertical force Q [kN] and vertical displacement Δ_B [mm]. How it is understandable from graph, it is a difference in behavior of complete structure, obtained from numerical approach (red line on Figure 18) and sub-structure, obtained analytically (green line on Figure 18).

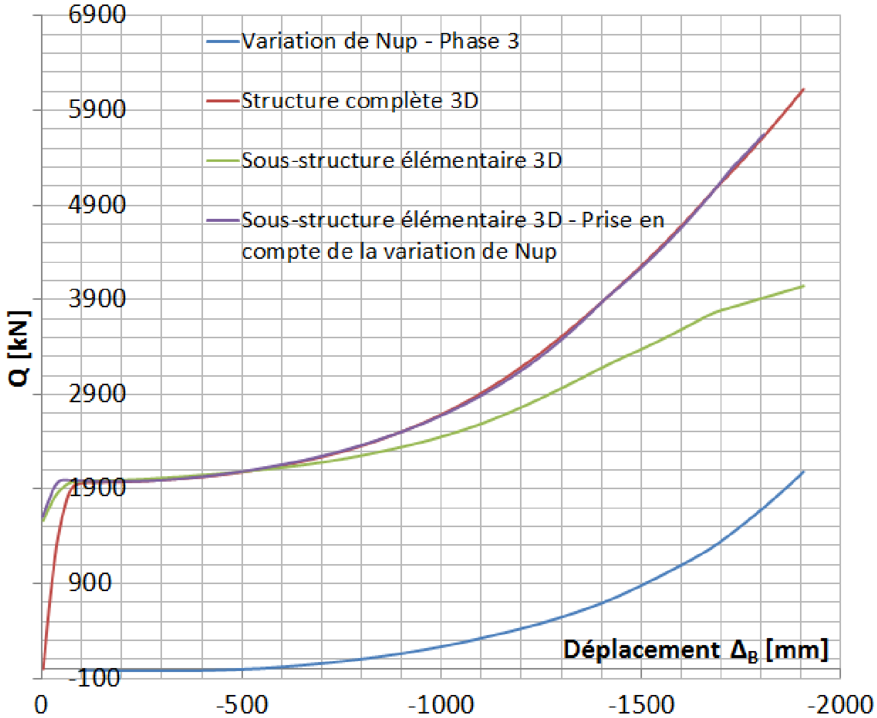

Figure 18 Results of Lemaire's investigations [3]

Difference between investigations by numerical and analytical approaches is caused by absence in analytical model taking into account couplings between stories. As analytical model describes behavior only of one storey, and numerical model describes behavior whole structure with floors above of lost column, this is causing difference. Global behavior is becoming different with beginning of the phase 3. Understanding, that difference is caused by presence of couplings between storeys and 3D couplings (in real behavior), pushes to investigate analytical model for taking into account analysis of several storeys, as firstly, and 3D coupling effects as secondly. Further investigations and research of Clara Huvelle [14] are concentrated on considering of coupling between storeys (described in II.2.3), then, comparable analysis between numerical and analytical approaches can be estimated for certain structure again. Current work investigation is based on analytical model, which is considering coupling effect between storeys, but not considering 3D coupling effect.

Part III

Objectives of the thesis

III. Objectives of the thesis

Determination of objectives of current thesis becoming understandable, if follow developing path of robustness investigations in ULg. It the thesis of Demonceau [1] was developed the analytical model, which describes behaviour of 2D sub-structure. Model was verified. When substructure was investigated in purpose to obtain result of behaviour for several storeys, difference between analytical and numerical approaches appeared. This difference is shown (Figure 18) and described in thesis of Lemaire [3]. By prediction of investigators difference was caused because of couplings in between of storeys. Next step of research was done by Huvelle, when analytical model was improved and couplings between floors were taken into account in analytical model.

The studies conducted in Liège have been based until now on simplifying hypothesis. In particular, most of the studies have been conducted on 2D-frames. For this plane case, coupling phenomena between stories have been identified and have been modelled in simple analytical methods, but coupling phenomena of 3D structure haven't been observed yet.

These 3D phenomena will be investigated within the present work by comparing numerical results and analytical predictions obtained with Demonceau and Huvelle model. Indeed, for the scenario "loss of a column", this lost column is part of 2 perpendicular frames which could interact because they are linked to each other through the remaining and undamaged structure. Part IV of current document will be concentrated on exploration of appearing fact of other couplings. By prediction, could be told, that during scenario "loss of column", stiffness of directly affected part can be affected by lateral stiffness of beams, which are surrounding DAP. This could make effect of 3D couplings.

III.1. Thesis structure

III.1.1. Main objectives

The aim of this master thesis is to quantify the importance of 3D couplings on the global response of the structure. During scenario "loss of column" in 3D structure, behaviour of structure and redistribution of forces may be affected of lateral stiffness of beams, which are surrounding directly affected part. Importance of those couplings is not known yet. Then, these couplings, if they will be observed, it would to be necessary to model and integrate in the analytical method already available at the University of Liège.

III.1.2. Structure of the thesis

In total, thesis consists of six parts. Further, I will describe each part and main objectives of each will be highlighted.

The first part "General introduction" and the second part "State of the art" of current thesis were described earlier.

The third part of thesis describes aim of the thesis and its structure, and it is under your reading right now.

Part number four calls "Investigation of 3D structures". This is the main part of the thesis. There described different approaches for investigation of reference structure. Numerical and analytical approach was performed for this investigation, link between them was made. Approach how to use developed in ULg analytical method for analysis of 2-dimensional structures for 3D building is explained. Also made comparison between approaches and obtained certain conclusions. For analysis of effect of indirectly affected part on behavior of structure and better understanding of investigated and observed phenomenon, was performed modification of initial structure and made analysis of it by both of known approaches. Certain conclusions are given. To make sure, that observed effect is correlated with earlier investigations in same field of study, was repeated whole analysis procedure with previously done analysis of “Lemaire” structure. Observed differences between of investigations were shown and made appropriate conclusions. For whole performed analysis is given general conclusion.

Part five collects overall conclusions and proposes certain visions and perspectives according to referent field and subject.

Part six concludes references.

Part IV

Investigation of 3D structures

IV. Investigation of 3D structures

Deeper investigation of analytical method and numerical studies come up to the moment, when it should be switched to 3-dimensional stage. Investigating of 3D structures allows obtaining results, which are closer to reality. Behavior of building in general, could be much more different, then behavior of the part of certain building, moreover, one-axial part of building. That's, actually, a reason, why we investigating further to 3D. Here, in this part will be described whole process of 3-dimensional investigation with different methods and approaches.

IV.1. Study of a reference structure

IV.1.1. Description of the reference structure

The structure, which will be subjected to the analysis, is presented on Figure 19. Firstly, structure will be academic on purpose to highlight all phenomena. It is chosen to be symmetrical and simple. Secondly, structure has only 2 stories. The directly affected part will have only 2 beams and 2 columns (if consider 2D frame). 3D effect is activated. Composition of building is managed in that way, that it has only one section of the indirectly affected part around the directly affected part. Thirdly, building is designed, as office. Fourthly, material of sections is steel.

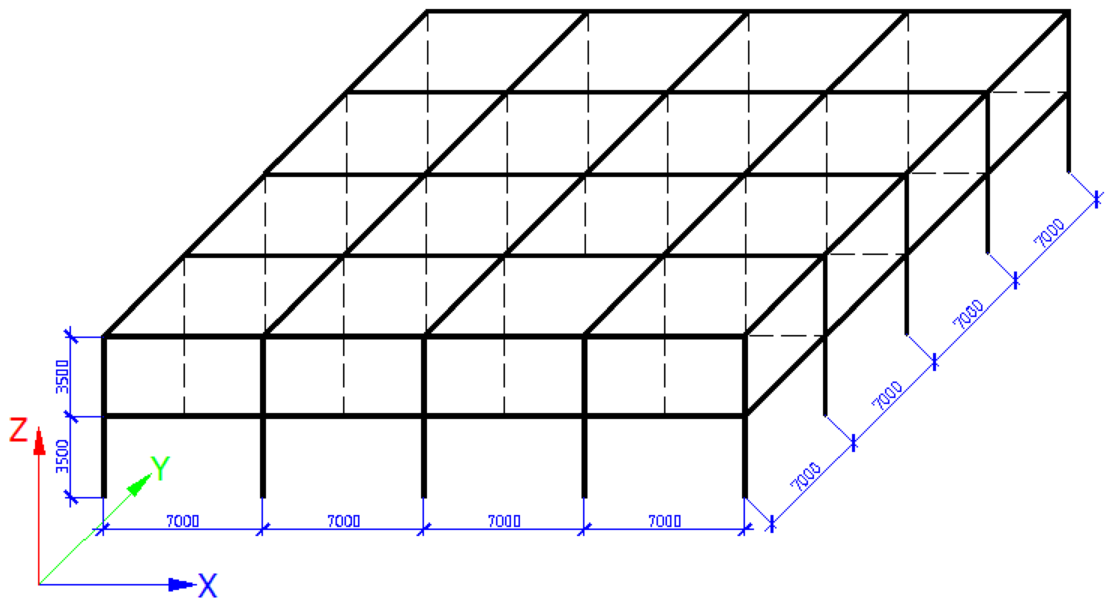

Figure 19 Schematically drawn structure

Main parameters of building:

– Span: 4x7m

– Bay: 4x7m

– Storeys: 2x3.5m

– Column section: HE 400B*

– Beam section: IPE 550*

– Beam-column connection: rigid and full strength**

- Column base: rigid

– Material: S355

– Permanent load: 3 kN/m^2***

– Live load: 2 kN/m^2 (as for office building by EN 1991-1-1:2002. Table 6.2) ***

* Definition of cross-sections for certain structure and certain loads firstly was defined by Robot Structural Analysis, and by design, obviously, for those sizes of structure, sections were defined as smaller (HE220B for columns and IPE300 for beams), but with analysis by the numerical method with FINELG-software (described in IV.1.2), chosen sections of beams were buckling before achieving the Phase 3. It was taken decision to increase cross-sections on the purpose to observe behavior of the structure during the Phase 3 avoiding lateral-torsional buckling of beams in compressed sections of upper storey.

** The choice of rigid and full strength beam-column connection is made on purpose to observe effect of 3D couplings by excluding of rotational capacity in beam-column connection. In case of pinned connection beams would obtain rotational capacity and transferring of bending moment would be excluded. As well, modeling of rigid connection is easier in certain FE software.

*** Presence or absence of uniformly distributed loads on structural elements are not affecting on behavior of structure during the Phase 3. When plastic mechanism is already formed, significant effect on behavior of the structure causes only by force, which simulates loss of the column, this fact was shown on Figure III.88 [1] and proved in thesis of Demonceau [1].

Permanent load defined approximately, if to take in consideration that slab will be done from reinforced concrete with thickness of 12 cm and volume weight of 25kN/m^3. Distributed load from weight of concrete then will be:

$$g = \rho \cdot \delta = 25\frac{kN}{m^3} \cdot 0.12m = 3\frac{kN}{m^2}$$

Firstly, structure (Figure 20) was designed in Robot Structural Analysis (RSA) for obtaining of compressive forces in column, especially, column, which will loss.

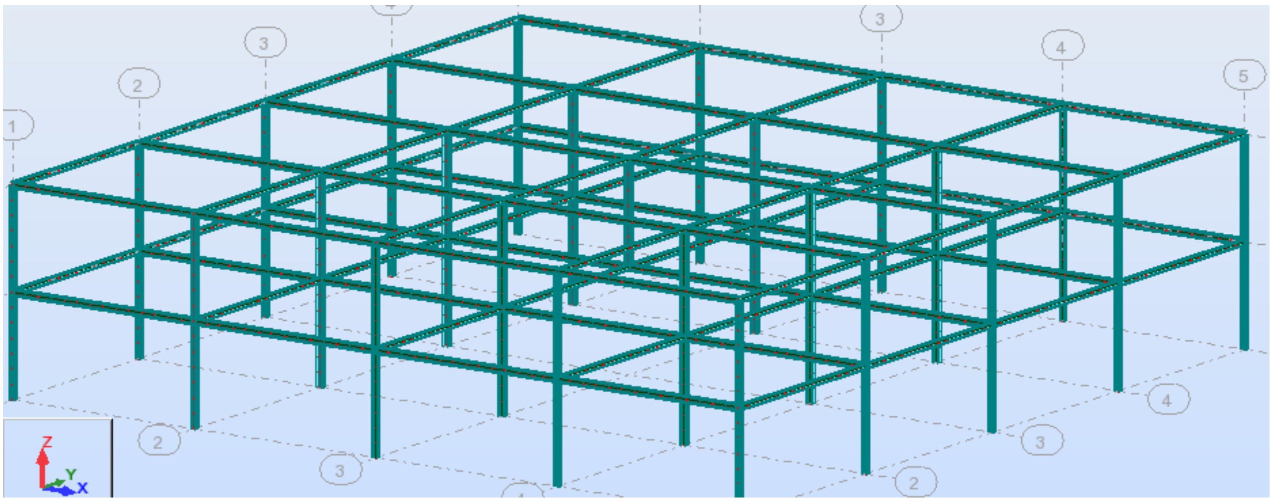

Figure 20 Design in Robot Structural Analysis

As was mentioned before, the building or structure, which subjected to lose structural element will have the directly affected part and the indirectly affected part. Corresponding to certain model DAP and IAP are presented on the Figure 21.

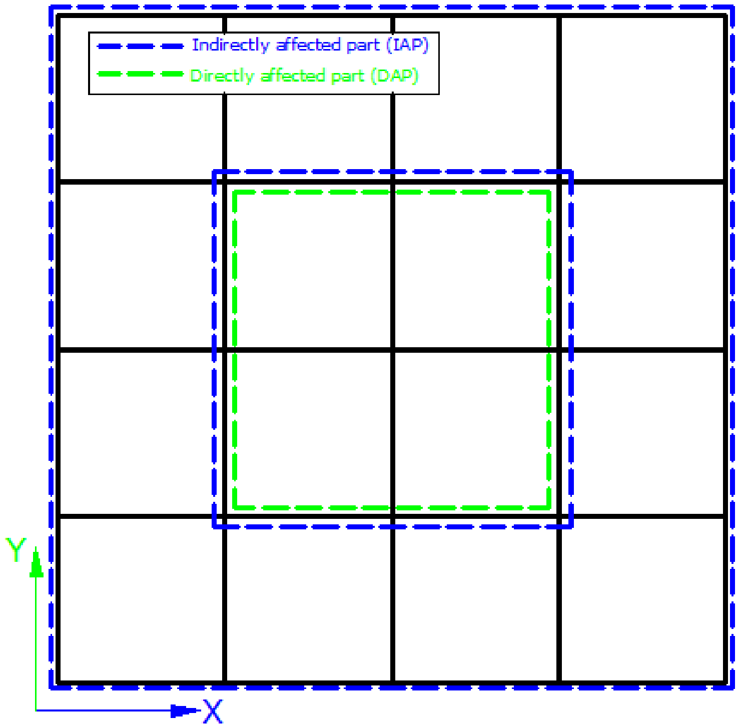

Figure 21 Parts of structure

On purpose to obtain precise results, it was taken decision to make discretization of the model. With help of the Robot Structural Analysis discretization of model was done half-automatically. Each structural element has been divided into seven parts. Each part of the element contained 2 nodes, beginning and end node, respectively. Finally, discretized model, with given parameters, contained 1408 nodes and 1463 elements.

IV.1.2. Numerical simulation

Numerical simulation was performed in software “FINELG”, which based on Finite Element Method and developed in University of Liege. I’d say about this program, that it is quite complicated for inputting of data, but on the other hand, it allows simulating of different situations and obtaining various results.

So, process of inputting of parameters to program starts with writing of calculation type, for this model in was chosen to be non-linear. Type of finite elements used for this model is “classical beam element for engineers, for three-dimensional frames”. In torsion is taken hyperbolic shape function by theory of Vlasov. Type of this element has special number “86” (Figure 23), which corresponds to number in FINELG [4].

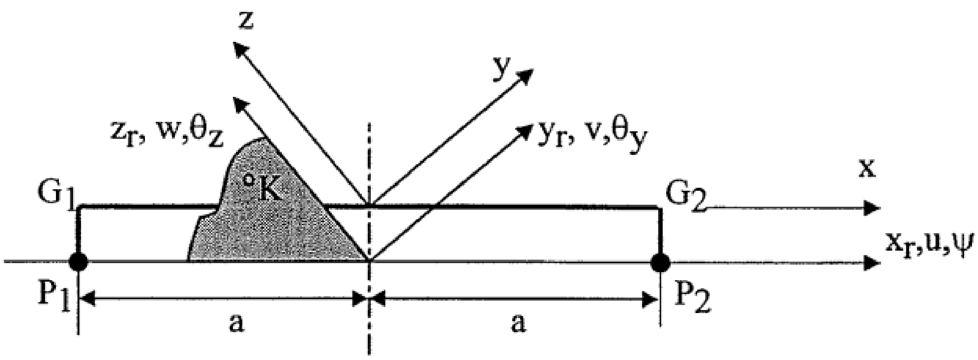

Figure 22 Type of finite element [4]

Element represented by three nodes P_1, P_2 and K. Nodes P_1, P_2 corresponded to beginning and end of the element, K corresponded to orientation of the element. Each node has seven degrees of freedom: $u, v, w, \psi, \theta_y, \theta_z, \theta_c$, where:

u, v, w - boundary conditions of linear displacements;

$\theta_y, \theta_z, \theta_c$ - boundary conditions of rotational displacements;

ψ - boundary condition of warping; warping is allowed.

Geometrical properties are chosen from catalog of cross-sections, which is built-in to program. Mechanical parameters of sections were chosen differently for different parts of building. For the indirectly affected part mechanical properties were chosen perfectly elastic, it was done on purpose to observe precisely behavior of the directly affected part, for which in its order was chosen material with elastic perfectly plastic behavior (Figure 23).

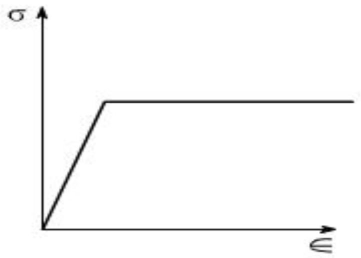

Figure 23 Elastic perfectly plastic material

After describing coordinates of nodes of the system, then, connecting nodes to elements and applying mechanical and geometrical properties to the elements, certain structure becoming to have possibility to be observed in module “FinGL”, and results are on Figure 24.

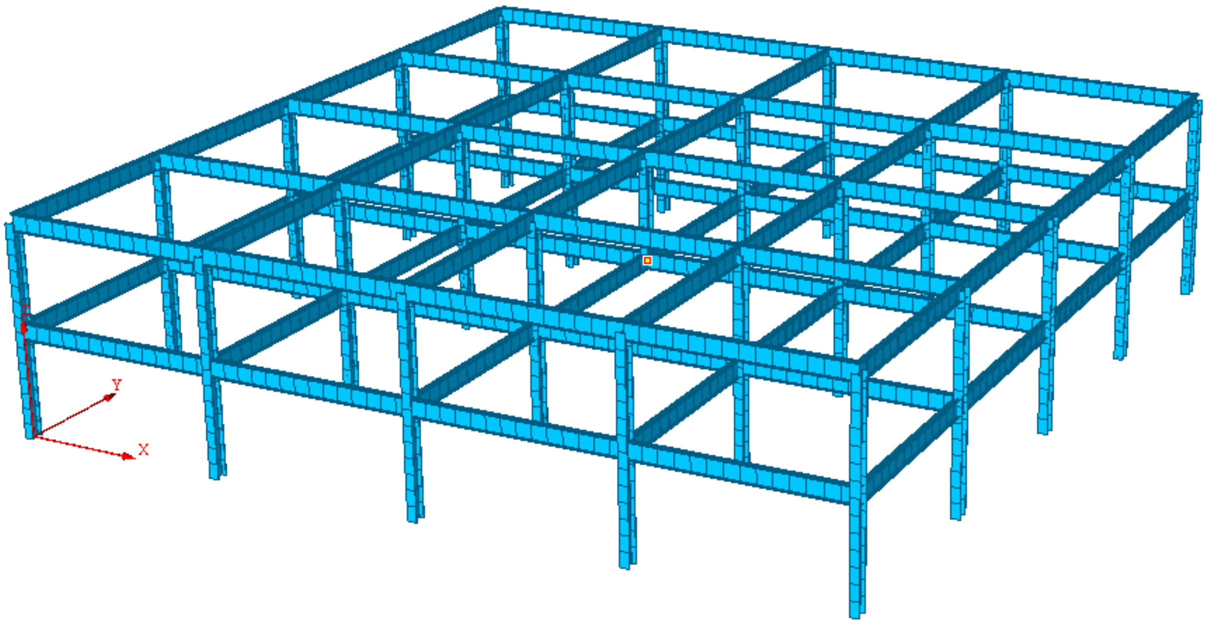

Figure 24 Building, modeled in FINELG

As it is visible from the Figure 24 how model is discretized. The point, which is highlighted with red color in the center of the structure, is the point, where structure has loss of the column. Also it is shown pointer of main axes. Accordingly to the axis further it will be called “strong axis frame”, which is correlated with “Y” axis, and “weak axis frame”, correspondingly to “X” axis. It is due to different geometrical properties of the column’s cross-section.

Loading of the structure is divided for two load cases:

First load case (Figure 25): applied live and dead loads. And main, as structure initially was modeled without central column, the force, which will simulate presence of the column, will be applied up, to the node, which is highlighted on Figure 24, for this certain model, this node has its own number "37", further output results will be taken for this node. On the Figure 25 is drawn central frame of the building with applied loads for first load case.

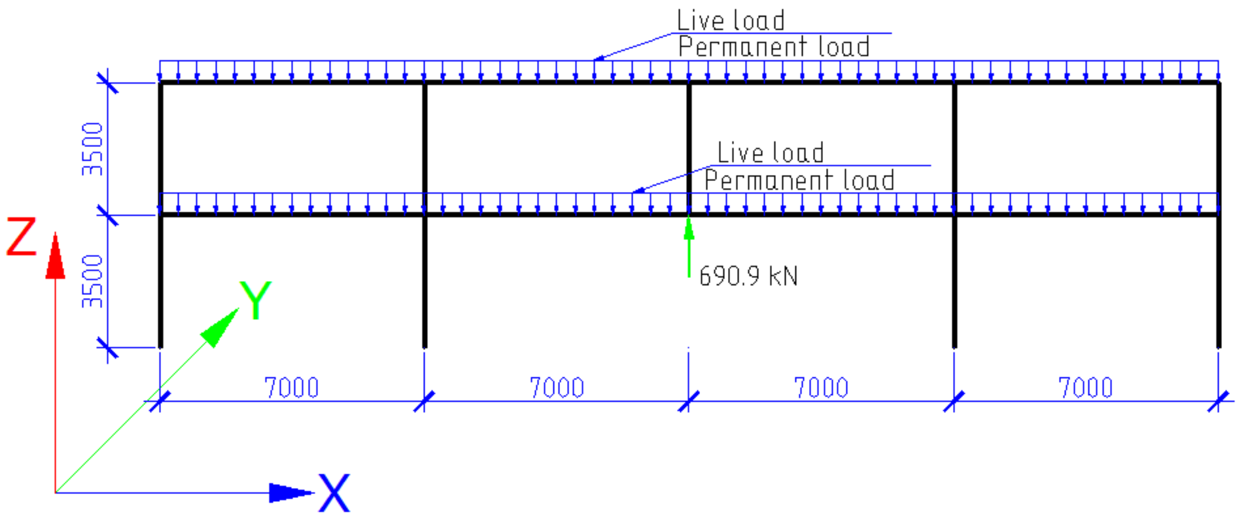

Figure 25 Central frame with applied loads

Explanation about applied loads:

$\gamma_G = 1.35$ - safety coefficeint for permanent loads; EN 1990:2002 Table A1.2(A)

$\gamma_Q = 1.5$ - safety coefficeint for variable loads; EN 1990:2002 Table A1.2(A)

$g = 3\frac{kN}{m^2}$ - permanent load; explained in part IV.1.1

$q = 2\frac{kN}{m^2}$ live load; EN 1991-1-1:2002 Table 6.2

$p = \gamma_G \cdot g + \gamma_Q \cdot q = 1.35 \cdot 3 + 1.5 \cdot 2 = 7.05\frac{kN}{m^2}$ -total distributed loading

$A_{gc} = 7 \cdot 7 = 49m^2$ area of one section between beams (girder cell)

$Q = 2 \cdot P \cdot 4 \cdot \frac{1}{4} \cdot A = 2 \cdot 7.05 \cdot 4 \cdot \frac{1}{4} \cdot 49 = 690.9kN$ - total concentrated force for two storeys.

On the Figure 26 is modelled first load case for the central frame, but in Robot Structural Analysis and with presence of the central column. Axial forces on diagram are presented in kN.

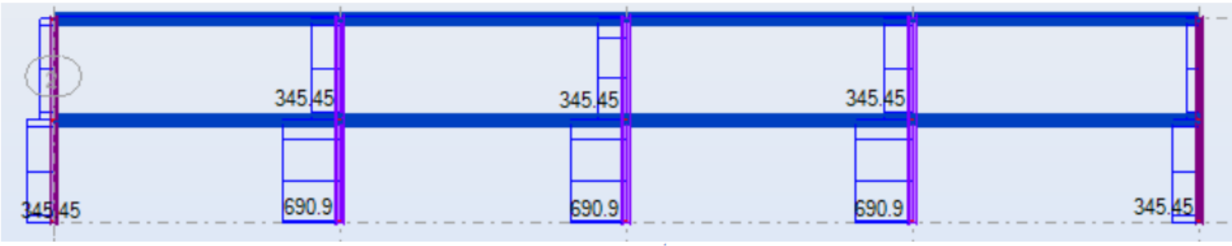

Figure 26 Axial forces for 1st load case from RSA

<u>Second load case</u> (Figure 27): simulating of loss of the column. The second load case is based on the first load case, from where taken results of first load sequence. For this load case will be applied force, which simulates loss of the column, means, force will be applied with same magnitude, but to opposite direction, down.

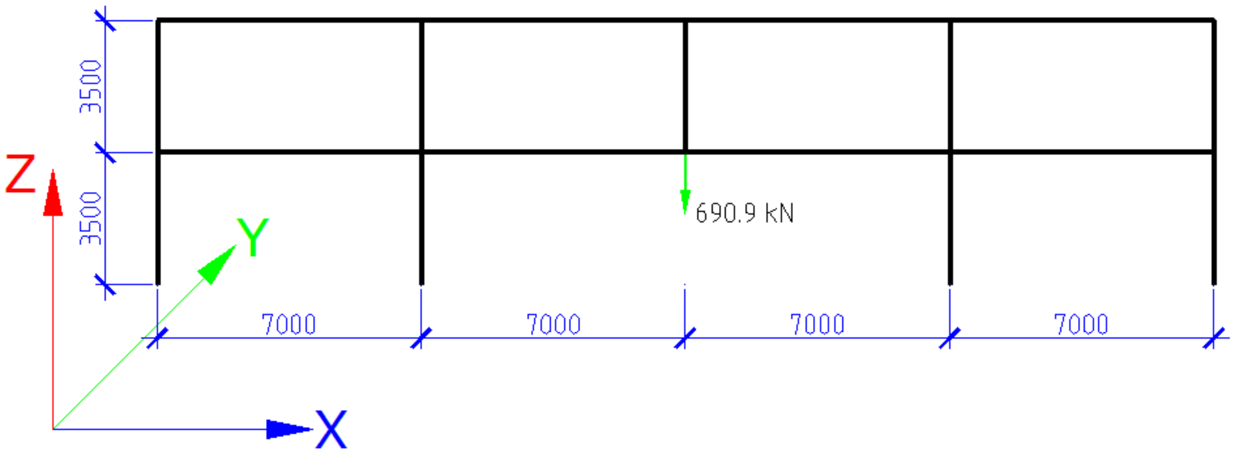

Figure 27 Second load case

Results from FINELG are getting with the help of modulus "SELFIN" and "DEPEXCEL". Application of load with consistent increasing of incremental parameters allows getting results for different steps of loading. Also FINELG gives possibility to extract animation of loading to video file.

Representing of graphical results is collected into proper way, to demonstrate different behavior of the referent structure during the loading. Few figures will demonstrate different stage and different response of the structure. With the red color is highlighted place and size of created plastic hinges. As it was mentioned before, the DAP is working until the plastic limit, the IAP mentioned as perfectly elastic. On Figure 28 is initial stage of loading, when incremental parameter of loading factor is close to 1 (load is applying by incremental steps of applied load, incremental parameter close to 1 means, that multiplication factor of load is 1 and load in this case is becoming near to applied value of load, defined in certain load case).

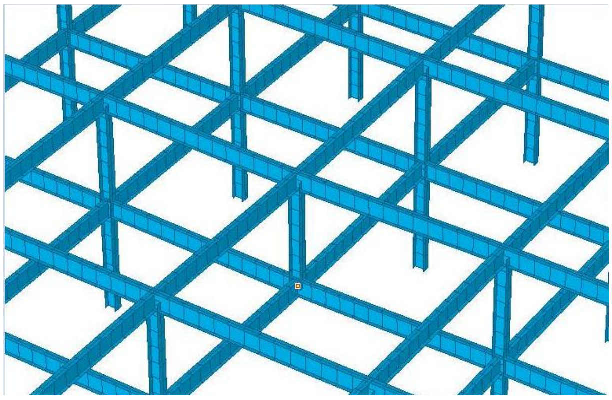

Figure 28 Initial stage

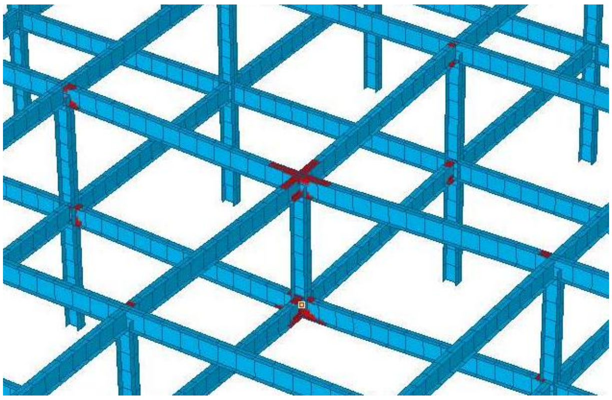

Figure 29 Creating of hinges

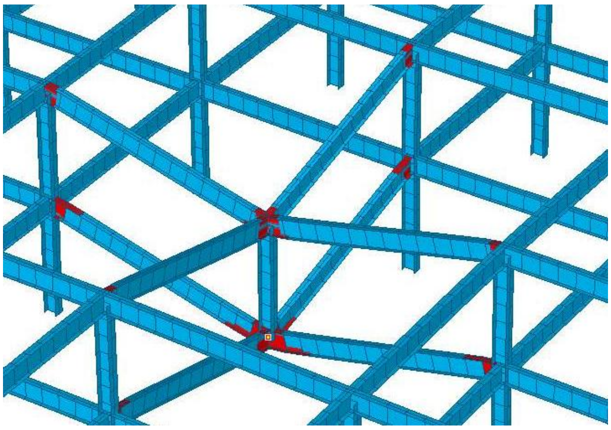

Figure 30 Developing of hinges and deformations

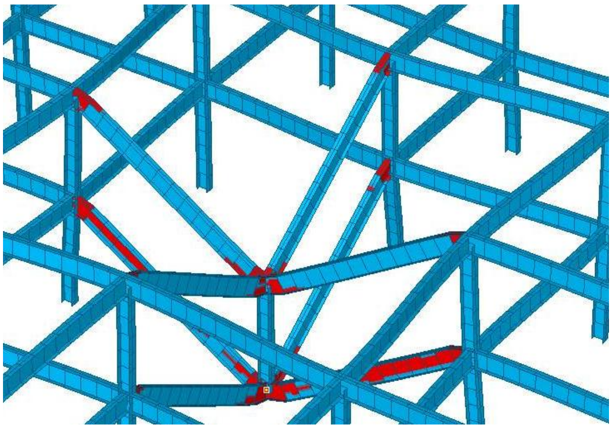

Figure 31 Destruction stage

On Figure 29 it is becoming visibility of the initiation of plastic hinges in places, near to connections. Figure 30 represents developing of plastic hinges.

Figure 31 demonstrates developed plastic hinges along whole beams and shows ruin mechanism.

That is how numerical simulation looks with graphical output. As output data it is possible to extract diagram, which will show dependence P-u (Force-displacement). On this diagram is possible to follow whole process of increasing loading parameter according to correlated displacement of given node. For certain analysis our interest is on node “37”, is a point, which is highlighted in the center of DAP (Figures 28-31), exactly at the place, where column is lost. As clearly visible curve describes work of element in elastic zone, then plateau on curve says about creating of plastic hinges, further curve P-u (Figure 32) truly describes behavior of the structure during the Phase 3.

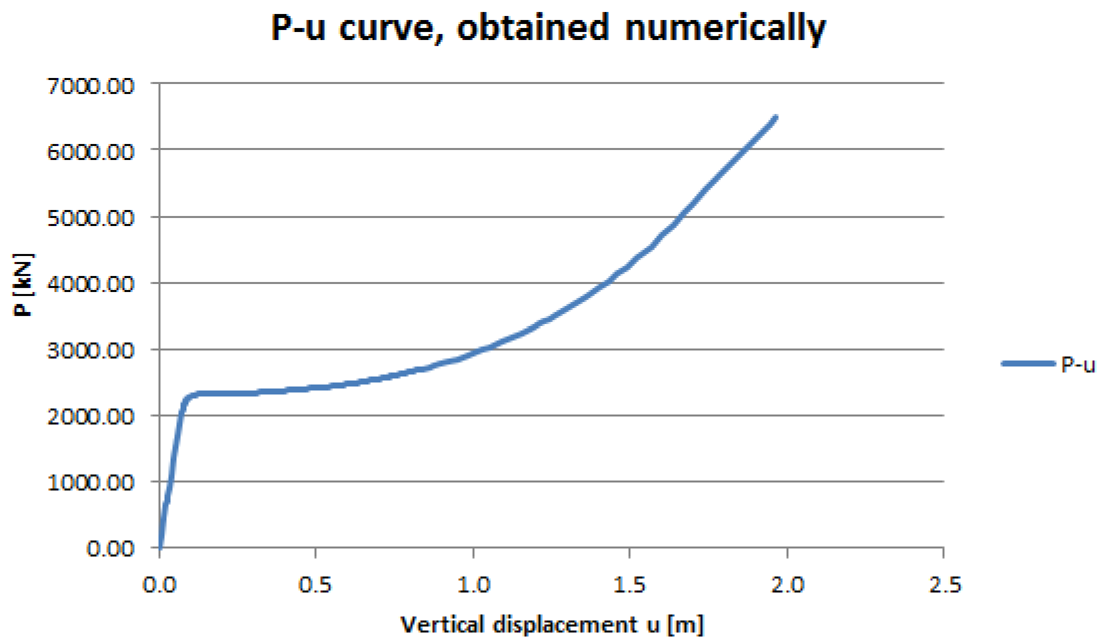

Figure 32 Output curve from FINELG

IV.1.3. Application of the analytical method

Mechanism of work of the application method, which was developed in ULg by Demonceau and Huvelle, was precisely described in the II.2.3 of the current dissertation.

Process of inputting of the data to analytical method, which was automatized and realized with help of Matlab, is quite simple. As was mentioned, method developed for analysis of the 2-dimensional frame. So, for obtaining results for 3D structure, firstly, model should be divided for 2D frames. As loss of the column is going only in one point, it means, that this point is crossed with 2 frames, one with X-direction, second with Y-direction. The idea is: to obtain results from analytical method for two 2D frames, then summarize results in purpose to obtain picture of the 3-dimensional behavior of the structure during the loss of the column. The difference between frames is caused by the different geometrical properties of the cross-section of the columns. Frames, which have cross-section of columns turned along the axis of the frame with it's strong axis called "strong frame axis" (Figure 33); frames, where cross-section of columns situated with weak position of axis along the frame's axis called "weak frame axis" (Figure 34).

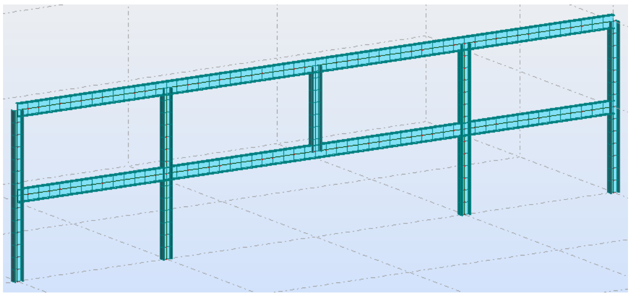

Figure 33 Strong frame axis

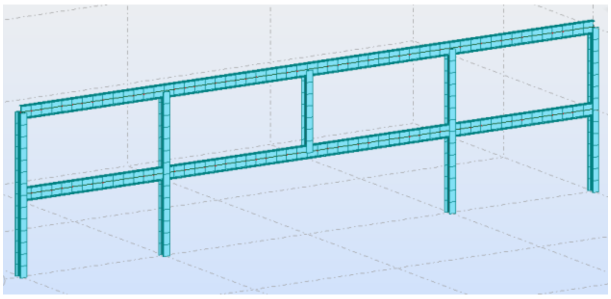

Figure 34 Weak frame axis

Data, which is required to input for analytical method program:

- Vertical maximum displacement we want to reach [m]

- Yield limit f_y [MPa]

- Modulus of elasticity [MPa]

- Geometrical parameters of elements: length and cross-section

- Number of stories above the lost column (DAP)

- Number of stories under the lost column (DAP)

- Presence of absence of bracing in left/right part of structure (IAP)

- Amount of columns in left/right part of the structure (IAP)

As results, it is extractable spreadsheet with all of calculated parameters, but interest of certain investigation is data about displacements and applied forces.

For description of the results, will be drawn curve "Q- Δ**"**, which presents behavior of the structure during the loss of the column. Parameter "Q" representing vertical force, simulating loss of column and dimensioned in [kN], parameter "Δ" represents vertical displacement and mentioned in [m], springs with parameters $k_{y,i}$ and $k_{z,i}$ are representing stiffness of the IAP according to their position (frame and storey) . But, as was written before, that total 3D response of structure, which takes into consideration couplings between storeys, will consist of two 2D frame responses (Figure 35).

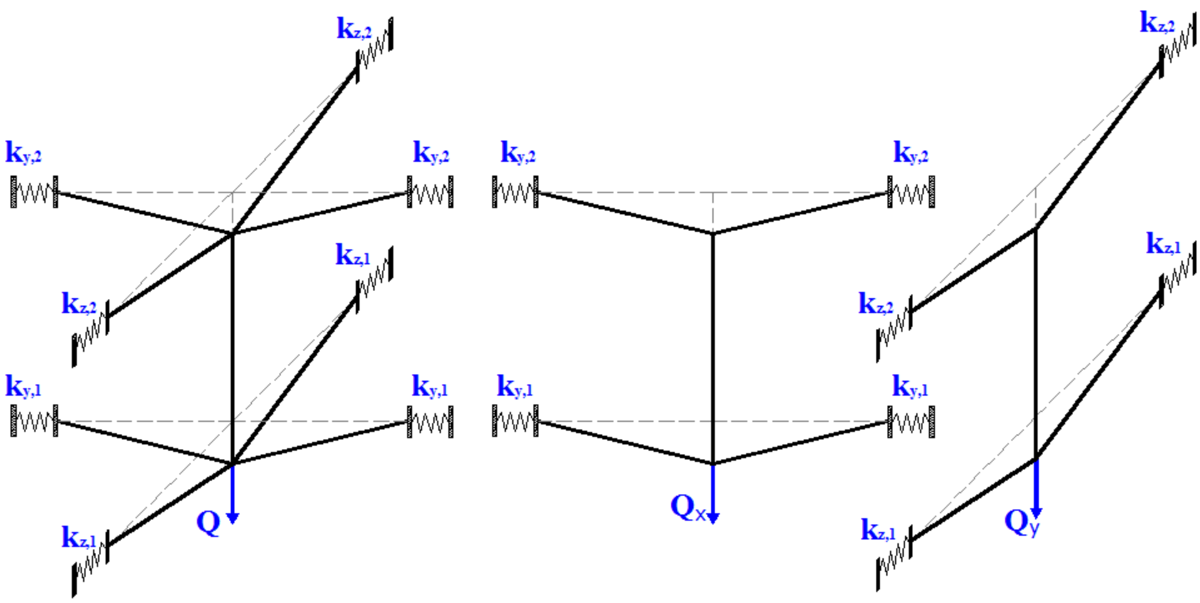

Figure 35 Idea of the response obtaining

Results of calculation by analytical method are presented on the Figure 36. Difference between "strong frame axis" – green curve and "weak frame axis" – red curve is visible on the graph of Figure 36. Force Q_x and vertical displacement Δ_x is corresponded to "strong frame axis", Q_y and Δ_y – to "weak frame axis" respectively. Difference is caused by the stiffness of the indirectly affected part.

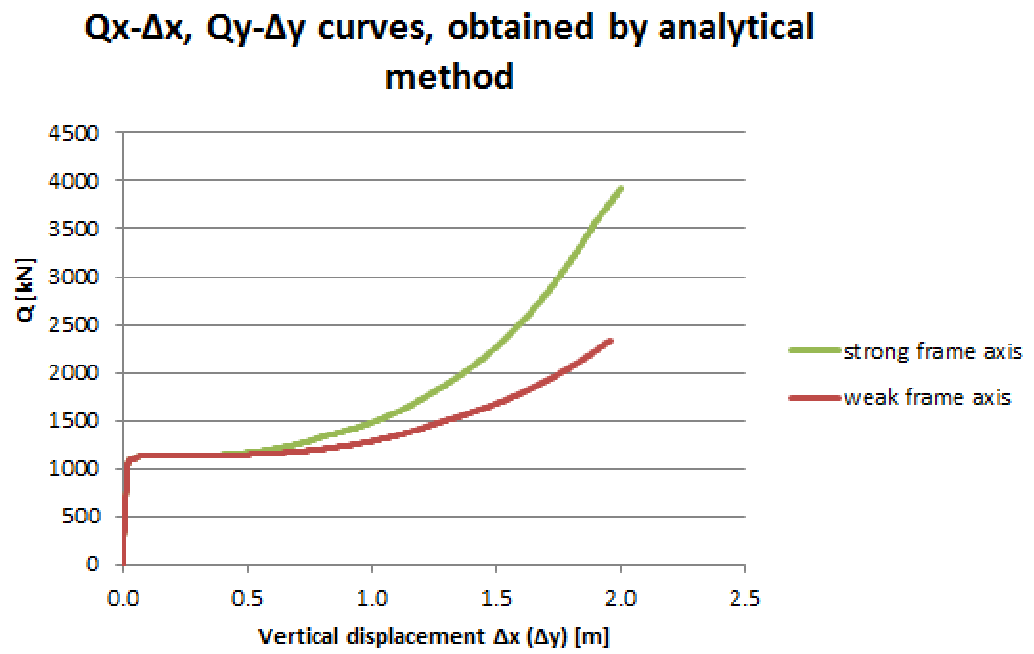

Figure 36 Results by analytical method

As columns oriented differently according to each frame and on Figure 36 is observable significant difference between their behaviors, it would be indicative to explain difference. This difference is caused only by column's moment of inertia, which is different because of profile's orientation for perpendicular frames.

Profile of column is HE400B; Then, stiffness ratio:

$$r = \frac{k_{column,y}}{k_{column,z}} = \frac{I_{y.HE400B}}{I_{z.HE400B}} = 5.6$$

this ratio describes difference between $k_{y,i}$ and $k_{z,i}$

$k_{column,y}$, $k_{column,z}$ - stiffness of column's major and minor axes, respectively

$I_{y.HE400B}$, $I_{z.HE400B}$ - moments of inertia of major and minor axes, respectively

Significant difference caused by profile's orientation causes difference in achieved results.

IV.1.4. Comparison between numerical and analytical results

Now is the time to show, which the difference between numerical and analytical approaches is, and to explain how both approaches intersect. Presenting of results (Figure 37), which will describe behavior of the structure during the loss of the column, realized by different methods will be very visual. Obtaining of numerical curve described in IV.1.2. For obtaining of analytical curve, need to make summarization of two curves, "strong frame axis" and "weak frame axis", which were presented on Figure 36. Process of summarization consists from adding values of vertical forces Q_x and Q_y for each certain value of vertical displacement (incremental values of Δ_x and Δ_y are equal). Resulting curve Q-Δ, which depends on total value of vertical force and displacement, is presented on Figure 37 as analytical curve.

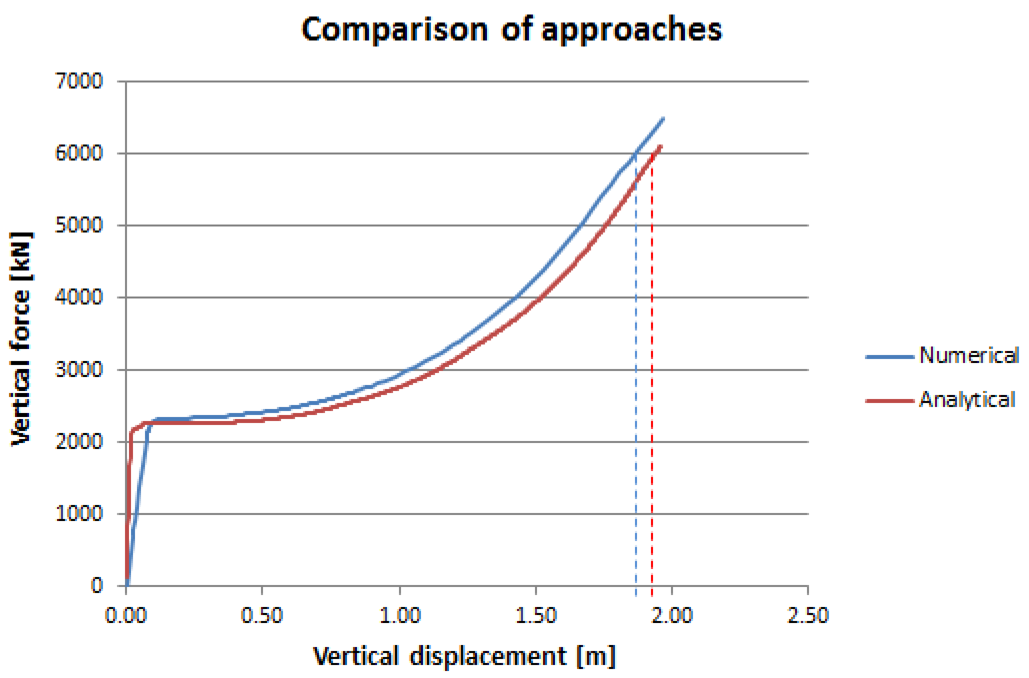

Figure 37 Results

Obviously, results are pretty close, but approaches for reaching of them are quite different. Difference between time, spent for reaching results by numerical and analytical methods is incomparably huge. Sources used for this procedure also much different.

As conclusion to this comparison it should be told, that comparison of two curves is indicative. If to compare displacements at the certain value of vertical force, for example of 6000kN, (on Figure 37 marked with dash-lines), by numerical and analytical approach, it is clearly observable, that behavior, described by analytical curve has bigger displacements. Bigger displacements produces bigger ductility, it means, that estimation by analytical method is on safer side.

For understanding of different approaches, if they are clearly close to each other in different conditions, investigation will move further and deeper.

IV.2. Parametrical studies on the reference structure

The investigation is moving further in purpose to verify obtained earlier results. The path of verification was chosen by modifying of different structural parameters.

IV.2.1. Modification of the horizontal supports

Choice of supports modification is based on purpose to see the effect of stiffness variation of the indirectly affected part to behavior during column loss scenario. Added supports will fix the indirectly affected part of the structure in different nodes, for understanding how it's affecting to the directly affected part. Supports were chosen as limiting one degree of freedom (fixing linear displacement). According to amount, situating and direction of supports were obtained 8 more different models. For those new models was performed full cycle of analysis, as was done for initial model. Means, for each of modified models was performed numerical simulation, analytical approach and made comparison.

Brief description of each modification will explain how different situations were considered.

Modification number one has obvious name "1st". This model along with previous "original" contains added supports. Supports are fixing frames, which are surrounding DAP, in longitudinal direction in both storeys (Figure 38). Results of numerical and analytical approach for "1st" modified model are presented on Figure 39.

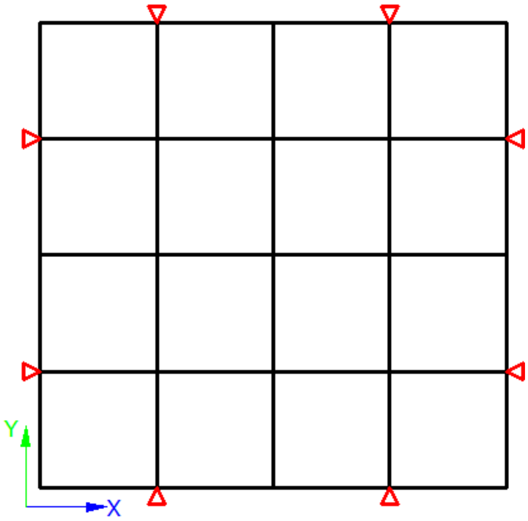

Figure 38 1st modified model

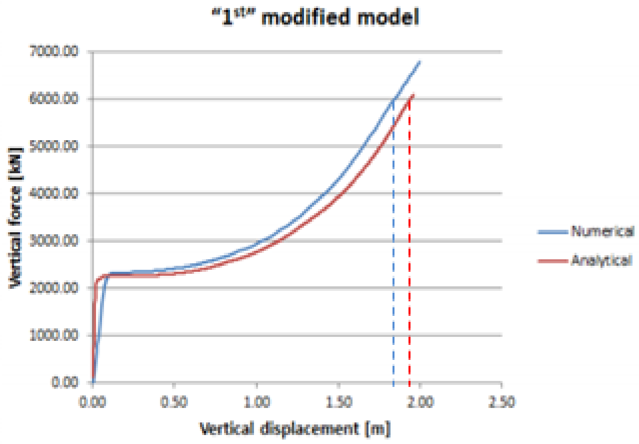

Figure 39 Results of 1st modified model

Results for "1st" modified structures realized by both approaches are close to each other, their behavior is similar to behavior of "original" (initial, not modified structure) and analytical method is on safe side by the same reason.

Second modification (Figure 40) of the model has name "2nd" and supports for this model are fixing in both levels "strong frame axis", which are surrounding DAP. Results for second modification are resented on Figure 41.

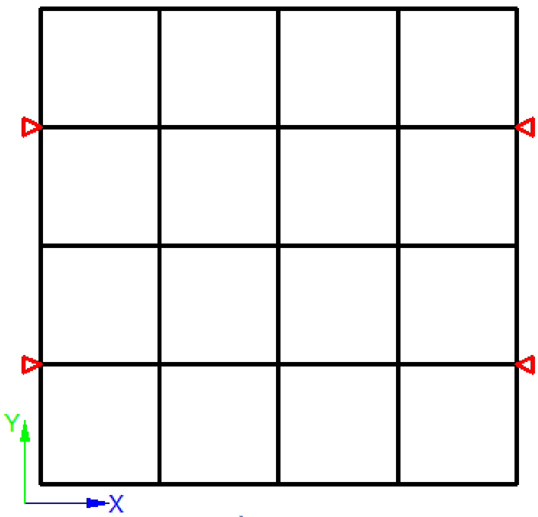

Figure 40 2nd modified model

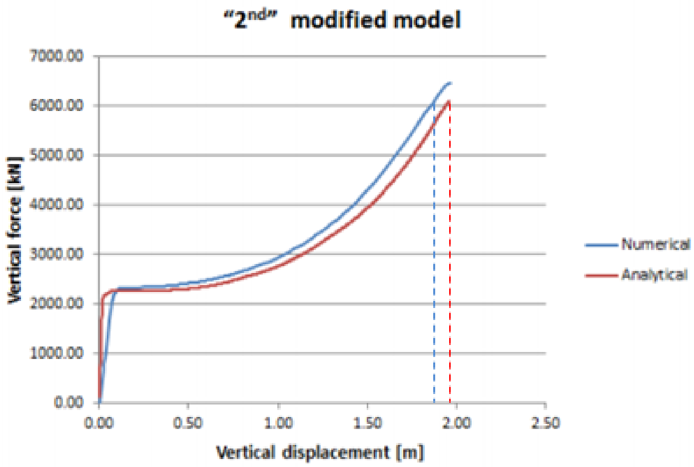

Figure 41 Results of 2nd modified model

The results for "2nd" modification are similar as well as 1st, and analytical method is also on the safe side.

Third modification (Figure 42) of the model has name "3rd" and supports for this model are fixing frames in both levels, which are crossing the DAP. In this case stiffness of the IAP will increase. Effect of stiffness increasing of IPA for third modification is presented on Figure 43.

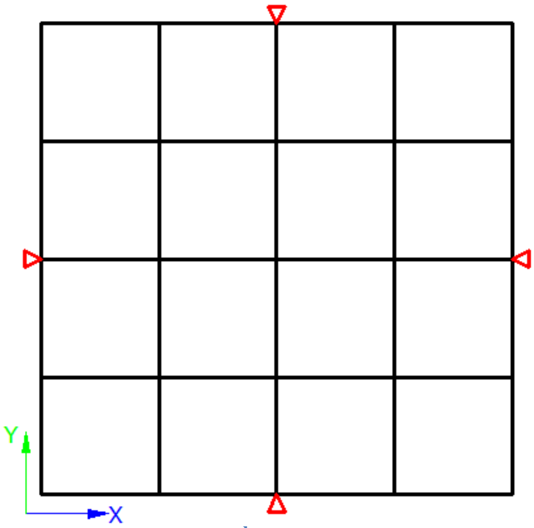

Figure 42 3rd modified model

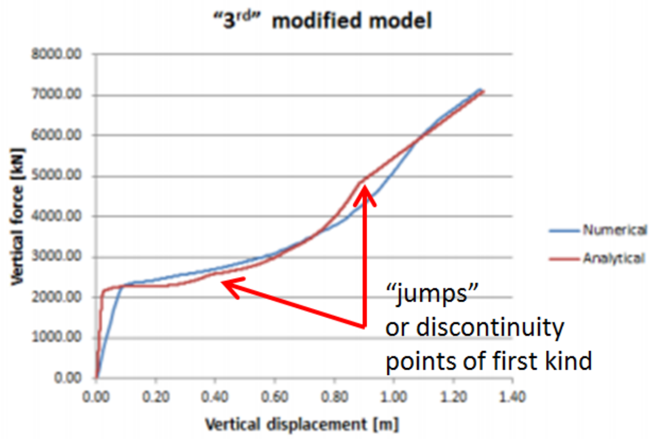

Figure 43 Results 3rd modified model

The results for "3rd" modification are quite similar, but have some insignificant differences. In the end of Phase 3 analytical method is safer, then numerical because of bigger displacements. But in behavior of structure with certain conditions of support are present some features. From diagram on Figure 43, if to follow curve of analytical approach, during the Phase 3 there are observed 2 point of discontinuity of the first kind (simply put "jumps" on curve). Both of "jumps" are corresponded with yielding effect of beams in "weak frame axis" (left point) and in "strong frame axis" (right point), respectively.

Next modification (Figure 44) of the model has name "3rd bis", because it has not so big difference with previous one. Supports for this model are fixing frames in both levels, which are crossing DAP, but exactly at border of DAP and IAP. In this case, supports are excluding effect of the indirectly affected part. Results of third modification are resented on Figure 45.

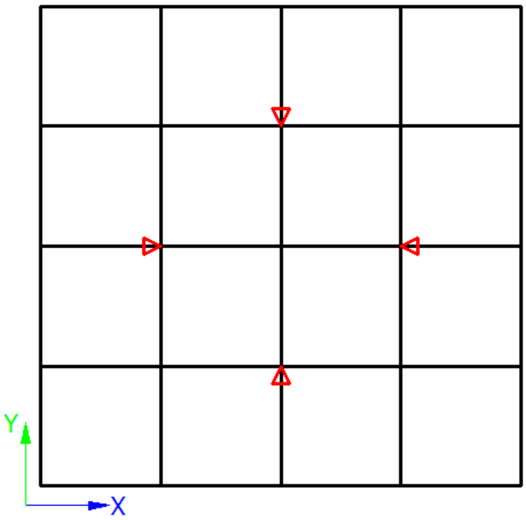

Figure 44 3rd bis modified model

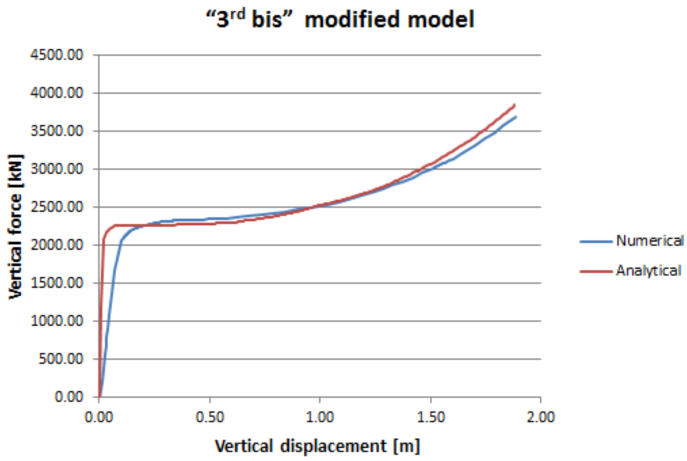

Figure 45 Results of 3rd bis modified model

Similarity of results for "3rd bis" structure is obvious, but behavior is different, because of work of only directly affected part, here numerical curve is on safe side.

Next ensuing modification is under name "4th". Here supports are fixing "strong frame axis" in both floors (Figure 46). Fixed frame crosses center of DAP. Results are presented on Figure 47.

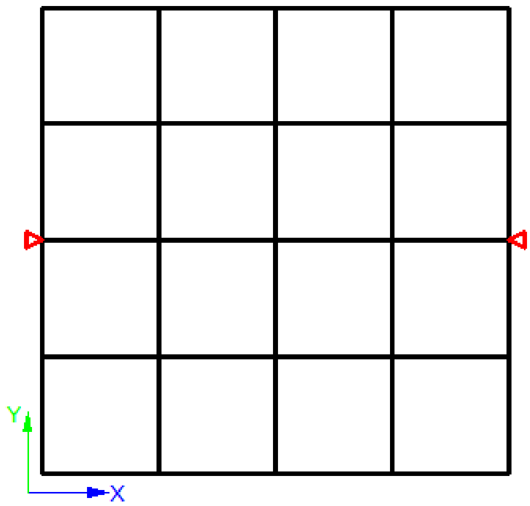

Figure 46 4th modified model

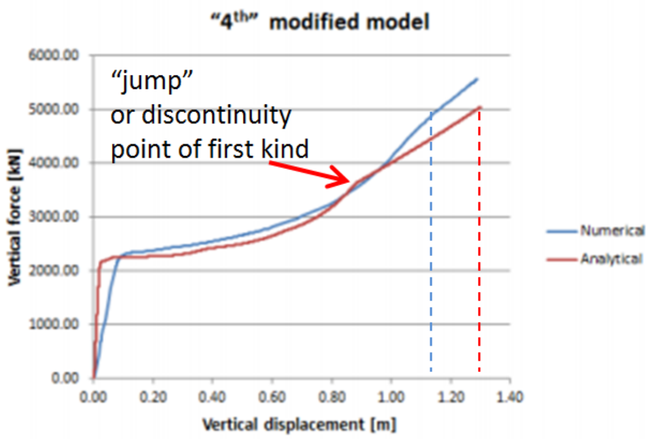

Figure 47 Results of 4th modified model

The results for "4th" modification are similar, and analytical method is also on the safe side. Observable point of discontinuity of first kind on analytical curve says about yielding in the beams of the fixed frame.

Sequent modification (Figure 48) of the model has name “4th bis”, because it has not so big difference with previous one. Supports for this model are fixing in both levels “strong frame axis”, which is crossing DAP; fixation made exactly at border of DAP and IAP. Results of this modification are resented on Figure 49. For certain conditions, structure behaves with effect of the indirectly affected part on the DAP for weak frame axis, but only with the DAP in strong frame axis.

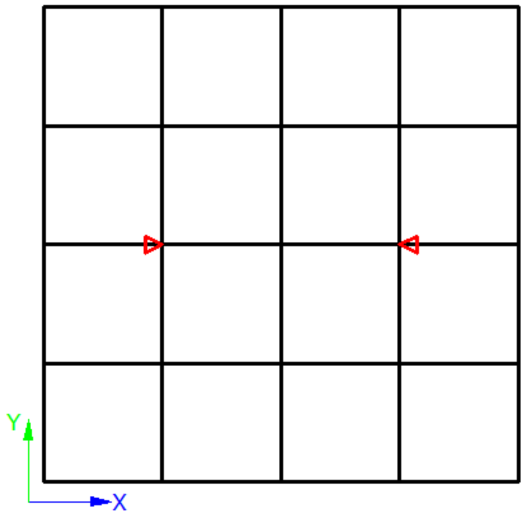

Figure 48 4th bis modified model

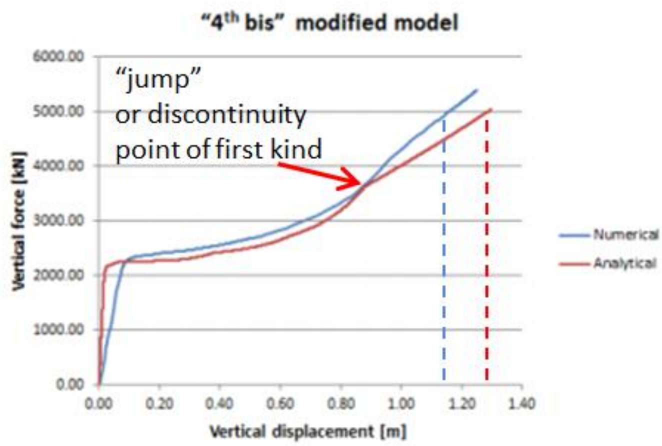

Figure 49 Results of 4th bis modified model

Results for both approaches are similar, and behavior of “4th bis” also similar with the “4th”. Yielding in beams of fixed frame is observable by the “jump” point on diagram of Figure 49. Anyway, if to follow same procedure of comparison of both approaches by certain value of vertical force and corresponded to this force displacements, analytical approach will show bigger displacements comparably to numerical, means, safer side.

Following modification is under name “5th”. Here supports are fixing “weak frame axis” at level of both floors (Figure 50). Fixed frame crosses center of DAP. Results are presented on Figure 51. Certain modification is opposite to the 4th one, because in this case weak frame axis will increase stiffness and strong frame axis will be as initially designed.

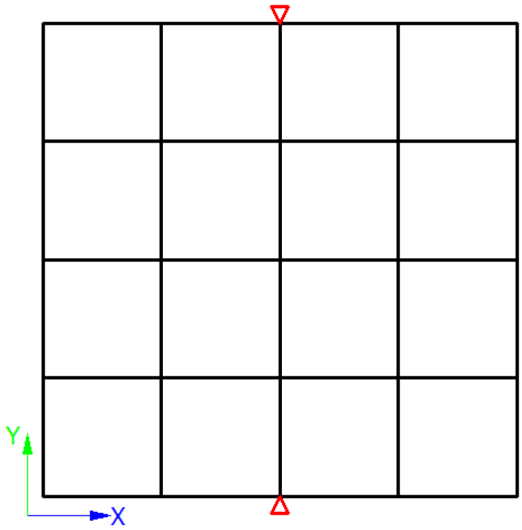

Figure 50 5th modified model

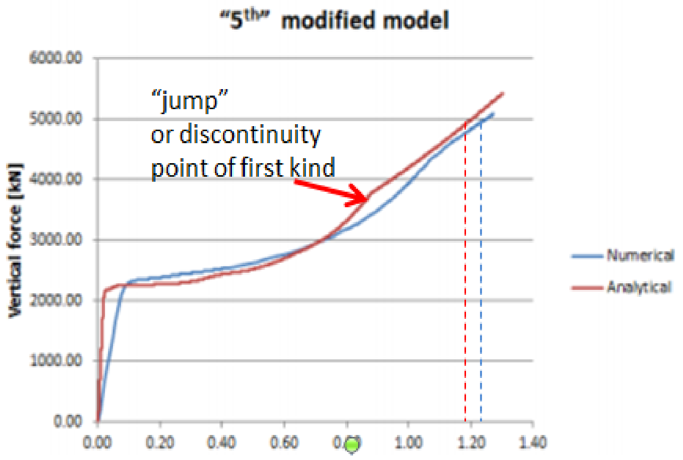

Figure 51Results of 5th modified model

According to results, behavior of "5th" modified model obtained by numerical and analytical approach is similar and close. Yielding of beams of fixed frame is observed by diagram of results on Figure 51.

The last modification (Figure 52) of the model has name "5th bis", because it has not so big difference with previous one. Supports for this model are fixing in both levels "weak frame axis", which is crossing DAP; fixation made exactly at border of DAP and IAP. Results of this modification are resented on Figure 53.

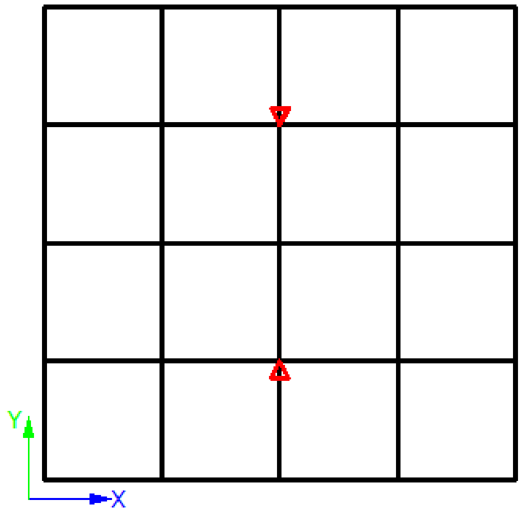

Figure 52 5th bis modified model

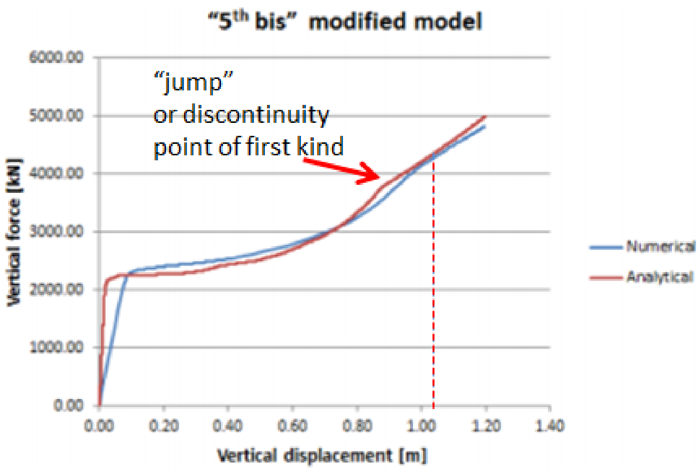

Figure 53 Results of 5th bis modified model

Results for both approaches are similar, and behavior of "5th bis" is close to "5th", this caused by slight change of structure.

IV.3. Investigation of "Lemaire" structure

As results of numerical and analytical methods are similar, research team came to conclusion of necessity make an additional investigation of structure, which was investigated in thesis of Lemaire [3]. Significant difference between approaches of Lemaire's investigation caused by absence in analytical method couplings between storeys. On the Figure 54 is presented behavior of "Lemaire" structure during loss of the column scenario with numerical (blue curve), current analytical method (red curve), and comparative curve, obtained earlier by Lemaire [3] with not improved analytical method (green curve "analytical_Lemaire"). Displacements at same value of vertical force, obtained by different approaches are shown with dash-lines. Difference between numerical and analytical approach obtained by Lemaire caused by couplings between storeys; difference between numerical and improved analytical methods obtained in current thesis caused by 3D couplings. Results on Figure 54 also show that analytical method was significantly improved with taking into account couplings between storeys.

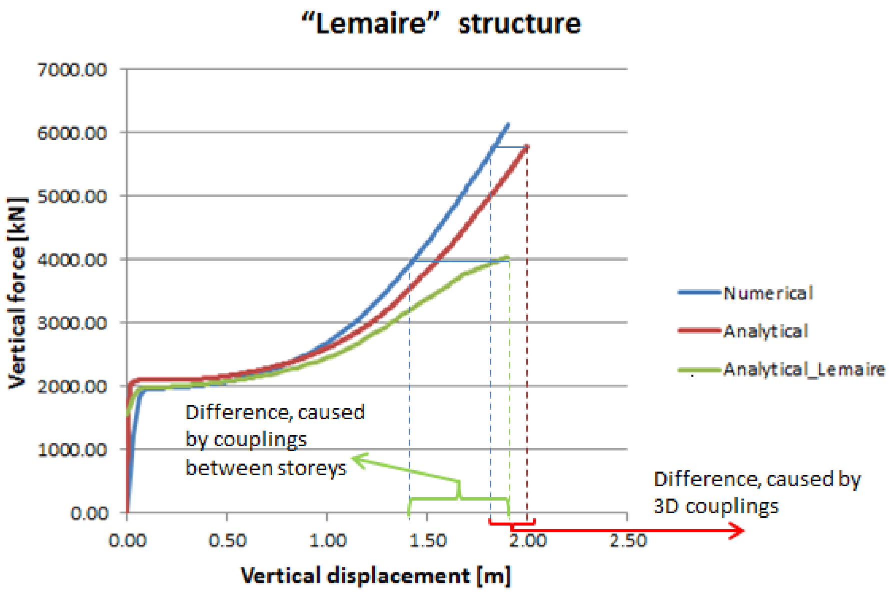

Figure 54 Results of "Lemaire" structure

Difference between of analytical (red curve) and two other curves at plateau is caused by the fact, that presence in the IAP different columns (HE320B external, HE360B - internal) was described as average column element (average area and moment of inertia).

IV.4. Conclusion

According to this part it suggests itself only one, but very important conclusion. All of investigated models, including modified models of current thesis, and as well, model of “Lemaire” structure, demonstrated similar close behavior of the structure during the scenario of loss of the column, provided by different approaches. As far as results of numerical approach, which takes into account behavior of whole 3D structure, are similar to results of analytical method, where considered independent behavior of 2D frames, it means, that the importance of 3D couplings on the global response of the structure is insignificant.

As well it is important to say about behavior of investigated structures analyzed by analytical method comparably to numerical approach. Displacements obtained by both of methods at the same value of vertical force, are bigger with analytical approach, which means, that design procedure will be safer, because of limiting by ductility.

Comparison between modified structures and initial one is presented on Figure55.

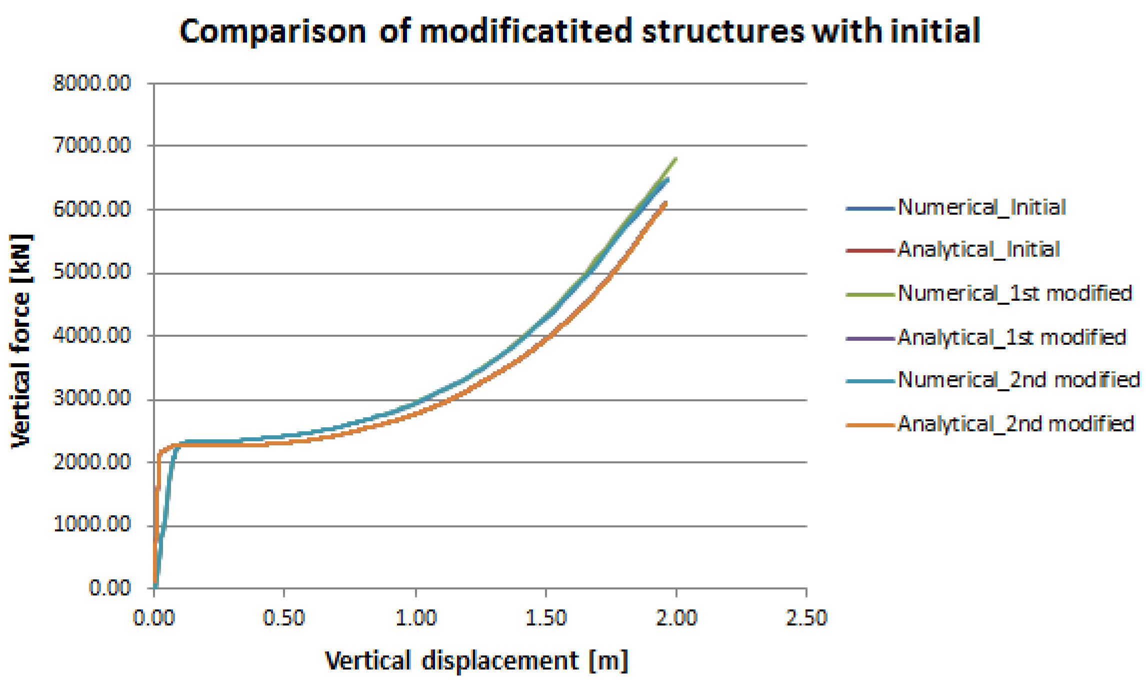

Figure 55 Comparison of investigated structures

From diagram of Figure 55 is traceable, that effect of support frames modifications in the indirectly affected part, which are not crossing directly affected part, is insignificant. On diagram of Figure 55 are presented 6 curves, which are presenting behavior of 3 structures (initial one, 1st and 2nd modified) by numerical and analytical approaches, and curves are overlapping, but not mixing, numerical are in one overlapping group, analytical in other.

Indicative more differences in behavior of structure will be comparison of modifications, where added to frames supports are on the line, which crosses directly affected part (3rd, 4th, 5th modified), with initial structure. Presence of supports changing stiffness of the indirectly affected part, which in it's order affecting to behavior of the directly affected part. Differences in behavior are presented of Figure 56. "Jumps" or discontinuity points of first kind are caused by yielding in beams. Plateaus on curves describe behavior of structure during creating of plastic hinged. The difference in between of plateaus is caused by various stiffness of the indirectly affected part.

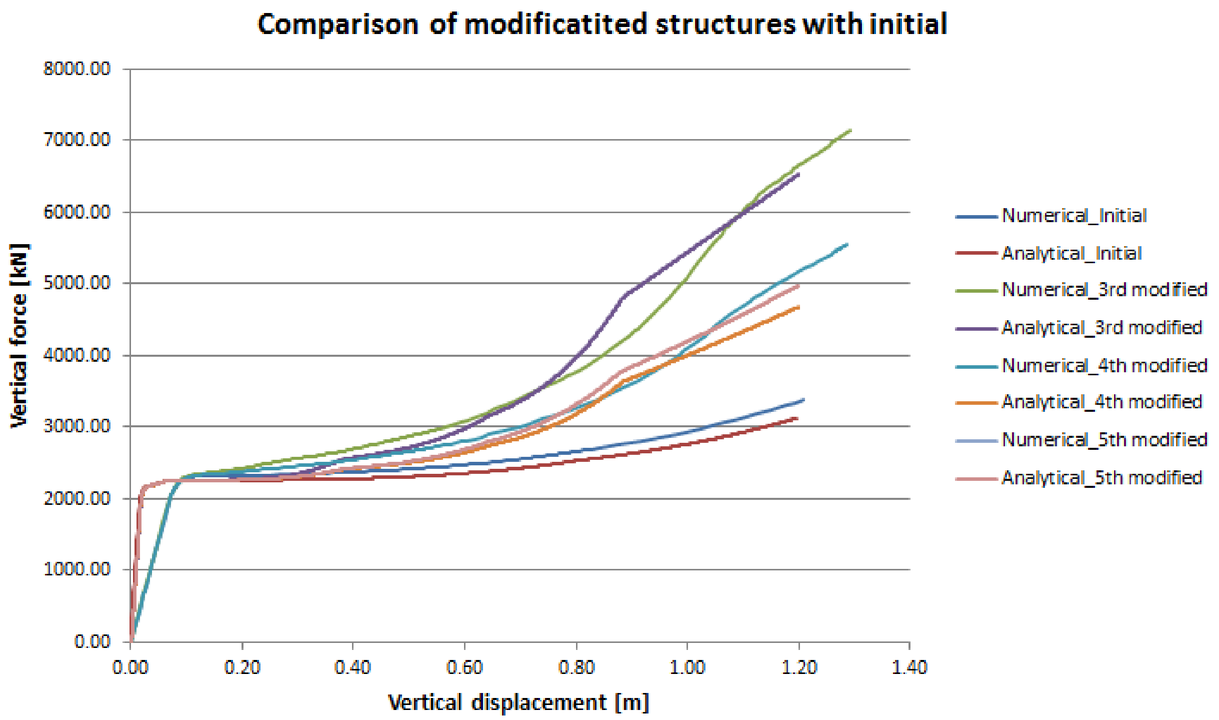

Figure 56 Comparison of investigated structures

Part V

General conclusions and perspectives

V. General conclusions and perspectives

V.1. General conclusions

Current thesis provides fully descriptive explanation term of “Robustness” and shows the most important and demonstrative cases of exceptional actions, happened in world practice with different buildings and structures. Examples of cases collected in way to show variety of possible cases.

Provided clear description of exist standards and regulations for different regions: Europe, United States and countries of post-Soviet countries.

Made overall global and detailed local overview of experience development in study field of robustness.

In main part of thesis made detailed comparative analysis of two methods: numerical and analytical. Analysis done for certain row of designed models with different conditions. As well, in work provided research of structure of previous investigation. Main conclusion, according to performed investigation – is insignificance of 3D couplings to global response of the structure. This conclusion simplifies life of engineer in question of robustness design.

V.2. Perspectives

For next step of investigation is necessary to change mechanical properties of the indirectly affected part to material with plastic behavior.

Interaction in beam-column joint has to be modified from rigid, by introducing to model numerical springs.

Analytical model can be improved for consideration of cases, when lost column can be not only in middle of span, but in edge also. Improving of model to have possibility take into account different spans of beams surrounding lost column place is necessary. As well, model should have possibility to describe frame with vertical element of different stiffness.

Based on conclusions, written above, it is necessary to develop finalized independent software, which will allow using analytical approach for design questions of robustness, considering steel structures.

Global perspectives of robustness development becoming more visible. For certain limit of known types of structures with global vision of robustness it is possible to describe direction of development (Figure 57). Certain table was modified and supplemented from initial one was provided in [5].

Type of structure	Steel structures	Composite structures	Concrete structures	Timber structures	Masonry structures	Glass structures
Design recommendations	TBD	TBD	TBD	TBD	TBD	TBD
Dynamic effects	I	TBD	TBD	TBD	TBD	TBD
3D behavior	D	I	TBD	TBD	TBD	TBD
2D behavior	D	D	I	TBD	TBD	TBD
Global approach	D	D	D	D	D	TBD
Notations	D	- developed	I	- initiated	TBD	- to be developed

Figure 57 Vision of robustness development

So, horizons are visible, need to move further to reach them.

Part VI

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