Saimaa University of Applied Sciences
The Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

Polina Kozlova

THE PHENOMENON OF PROGRESSIVE COLLAPSE ACCORDING TO RUSSIAN NORMS

Bachelor’s Thesis 2013

ABSTRACT

Polina Kozlova
The Phenomenon of Progressive Collapse According to Russian Norms, 93 pages, 2 appendices
Saimaa University of Applied Sciences, Lappeenranta
Technology, Double Degree Programme in Civil and Construction Engineering
Bachelor's Thesis 2013
Instructors: Mr Petri Himmi (Saimaa University of Applied Sciences), Mr Pekka Narinen (Finnmap Consulting Oy), Mr Johan Rosqvist (Finnmap Consulting Oy)

Finnmap Consulting Oy acted as the client. The research had several targets. The first one was to introduce the concept of progressive collapse and describe the causes and consequences of this type of failure. The second target was to highlight the main measures which can help to ensure the stability of a building against progressive collapse, to emphasize how it can be solved in Russia according to not numerous standards. And the third, the most important target lies in the issue of practice – to study possible methods of calculation, to identify the features of this process and to show the example of calculation.

The first part of the thesis gives the definition of the main concepts – progressive collapse and survivability, describes the types of progressive collapse and its causes. Also this part includes information about status of issue and gives examples of challenges, which can be met during studying the proposed topic.

The second part describes in detail the methods to prevent progressive collapse, including requirements for design, rules for choice of construction system and materials and recommendations for calculation. Measures for existing buildings are also included in this part.

The third part of the thesis contains the method of calculation and gives the consequence of calculation step by step.

And the last part includes the description of several attempts of calculation with comments and conclusions which were made during calculations.

As a result the clear concept of progressive collapse was received. The possible methods of prevention of this failure were described. And the consequence of calculation with examples was presented.

During study the emphasis was made on Russian norms and standards such as STO, MDS and Recommendations.

Keywords: progressive collapse, methods of prevention, consequence of calculation

CONTENTS

APPENDICES

SYMBOLS

Latin upper case letters

A_c	cross sectional area of concrete
A_p	area of a prestressing tendons
A_s	cross sectional area of reinforcement
E_c	module elasticity for concrete
E_s	module elasticity for reinforcement
F (U)	effort in a structural elements
M_c	plastic moment capacity of the beam at the support
N_c	permissible plastic longitudinal force in the beam
N_{ed}	design value of the applied axial force
R_s (f_{yd})	design yield strength of reinforcement
$R_{s,n}$ (f_{yc})	normative values of resistance to tension
R_p	design yield strength of tendons
S (W)	bearing capacity of structures

Latin lower case letters

b	width of beam
$d_{s1}(d)$	effective depth of a cross-section
d_{s2}	thickness of concrete cover
f_{cd}	design value of concrete compressive strength
g	dead load
h	height of beam
k_f	dynamic load factor
l	span
m_{px}	plastic moment capacity of the slab in x-direction
m_{py}	plastic moment capacity of the slab in y-direction
n_{px}	plastic capacity of the axial force of the slab
q	live load
t	thickness of slab
w_c	deflection
y	relative height of compression area

Greek lower case letters

γ_{b3}	coefficient of working conditions
γ_c	coefficient of reliability for concrete
γ_n	safety factor
γ_s	safety factor for reinforcement
ρ_c	density of concrete
φ	coefficient of combination
χ	height of compression area

TERMS AND DEFINITIONS

“Key” elements are elements which are located in areas of possible emergency actions and determine the load-bearing capacity and reliability of frame. These elements play a dominant role in ensuring the overall sustainability and geometric immutability. The output of these elements entails building collapse of the entire structure. For instance, in hanging and arched structures “key” elements are arches, supporting and a contoured elements; in beam and frame structures these are columns and cross-bars. (MDS 20-2.2008, p. 12).

Reliability of building object is its ability to perform the required functions during the projected service life. (STO 36554501 – 014 – 2008, p. 4).

Projected service life is a period of construction project (from the start of object operation or resumes its operation after repair or reconstruction) for its intended purpose established in the norms or in the design assignment. (STO 36554501 – 014 – 2008 p. 5).

Service life is duration of normal operation of building object to a state in which its further exploitation is not allowed. (STO 36554501 – 014 – 2008, p. 5).

Normal operation is operation of building object in accordance with conditions, laid down in standards or design task, including proper maintenance, repair or reconstruction. (STO 36554501 – 014 – 2008, p. 4).

Primary structural system of the building is the system adopted for normal operation of the building. (STO – 008 – 02495342 – 2009, p. 2).

Secondary structural system of the building is the primary structural system, as modified by removing one vertical bearing structural element (columns, pilasters, wall area) within a single floor. (STO – 008 – 02495342 – 2009, p. 2).

Special loads – loads and impacts (explosion, collision with vehicles, equipment breakdown, fire, earthquake, and the failure of the bearing element of the construction), creating an emergency situation with possible catastrophic consequences. (STO 36554501 – 014 – 2008, p. 16).

1 INTRODUCTION

Large-span structures have a higher level of responsibility and their failure can cause severe social and economic implications. And although the progressive collapse is not so frequent event, examples of consequences of such type of failure show that it has to be considered in design. Nowadays there is no generally accepted scientifically based approach of designing buildings which allow retaining structural integrity in emergency impacts (by abnormal loads).

For this reason, Finnmap Consulting Oy, the client of the thesis, is interested in this topic. This company is one of the leading companies of Europe that provides structural and civil engineering design and consulting engineering services. So for such a global company it is necessary to develop a clear sequence of measures to provide stability of the large-span building against progressive collapse, make the most appropriate method of calculation for structures. Especially it is important because the company is actively working in Russia, where development of solutions for this matter is on the initial stage.

The main problem lies in determination of collapse: it is difficult to define the probability of occurrence and the value of expected threat – in most cases emergency impacts cannot be quantified, the extent of possible initial damage and detailed knowledge of structure behavior during collapse are unknown.

So the main goals of this work are:

1. To introduce the phenomenon of progressive collapse: give description of the process, reasons and consequences;

2. To give an overview of ways to prevent progressive collapse for new and existing buildings;

3. To develop sequence of calculation and check the structures of the given building according this calculation

This thesis shows the issue of progressive collapse and possible measures and methods of calculation which can help to avoid this type of failure according to Russian standards.

2 CONCEPT OF PROGRESSIVE COLLAPSE

2.1 Status of issue

When considering problems there are some difficulties concerned with the condition and level of development of the issue and its solutions.

The topic which was chosen for research has some important features. This is a relative novelty – issue of progressive collapse and survivability of building structures is at the initial stage of formation and development, especially in Russia, that causes uncertainty and confusion of interpretations, techniques and solutions of problems. It is necessary to find a comprehensive approach to the topic namely studying plenty of sources together with practical examples. It allows to clearly understand the causes, effects and prevention of collapse.

This feature can also cause such a problem as shortage of references – learning the matter shows that in Russia there are not enough suitable documents which regulate in expanded form the issue of survivability of large-span buildings. The first normative document, touching on issue of safety and survivability of such type of buildings appeared relatively recently – in 2008 – STO 36554501 – 014 – 2008. “Reliability of the constructions and the foundations. General rules”.

In connection with this, it can be noted that acting and the mostly outdated normative and technical documentation (SNiP, MGSN, recommendations and instructions) does not give direct answers and guarantees of absolutely providing accident-free operation. It only makes demands on some parameters, which do not guarantee the required level of safety.

The emergence of topic “progressive collapse” is connected with number of tragic events which are united by one common feature – the disparity between the reason of disaster and the value of final damage.

The first event was failure of “Ronan Point” side facade in England (1968), caused by explosion of gas on the 18th floor (Figure 1). As a result the external panel was destroyed and became a reason for following failure of overlying panels – debris from upper floors cascading onto lower floors and walls caused collapse of entire corner of the building. Furthermore given system did not envisage possibility of load redistribution which caused a chain of failures. (Bruce et al., 2007, p. 91).

Commission, which investigated the causes of this tragedy, used for description term “progressive collapse” and gave some recommendations and methods how to protect panel buildings from such types of destruction.

Figure 1. Failure of “Ronan Point”

The finding was made that the most important factor contributing to structural vulnerability is a lack of continuity within the system and lack of ductility in structural materials, members, and connections. (Bruce et al., 2007, p. 17).

After this tragedy England adopted some form of regulatory standards to address prevention of progressive collapse – “Standards to prevent progressive collapse in large-panel constructions” were published in November 1968. This document first told about such terms as alternative way of load, compartmentalization, continuity and accidental (abnormal) loads. Later this norm became obligatory for design of buildings in England.

The common recommendations which were given in new norms are the following:

1. Buildings should be designed to resist disproportionate failure by tying together building elements, adding redundant members, and providing sufficient strength to resist abnormal loads. These requirements are considered to produce more robust structures that are strong, ductile, and capable of redistributing loads to provide “general structural integrity”. (Bruce et al., 2007, p. 2).

2. The main principle that helps prevent “progressive collapse” is the increase of fixity of constructive system of the building by improving the joints between structural elements, that is, the strength of anchors in precast elements should be appropriated to insure the load bearing capacity of connections. (Bruce et al., 2007, p. 2).

During the period of the 70s – the middle of 90s researching of progressive collapse was going slowly. But situation has changed after new serious collapses. (The following examples were found on http://www.sedigest.in/article/overview-progressive-collapse).

The first one occurred in North Virginia – failure of large hotel complex “Skyline Plaza” during construction in 1973, the reason of which became the punching shear of column at the 23rd floor (which not gained sufficient stripping strength) initiated by premature removal of formwork. The collapse involved the full height of the tower, and the falling debris also caused horizontal progressive collapse of an entire parking garage under construction adjacent to the tower (Figure 2).

Figure 2. Skyline Plaza, 1973 – premature formwork removal

The next accident happened in Lebanon (Marine Barracks), where the car bomb was detonated by a suicide bomber driving a delivery van packed on April 18, 1983. The blast collapsed the entire central facade of the horseshoe-shaped building, leaving the wreckage of balconies and offices in heaped tiers of rubble, and spewing masonry, metal and glass fragments in a wide swath (Figure 3).

Figure 3. Beirut barracks bombing

The next example was L’ambiance plaza, a 16-story apartment tower in Bridgeport, Connecticut totally collapsed on April 1987, during construction. The flat plate floors were designed to be constructed by the "lift slab system." The design used unbonded plastic-sheathed post-tensioning tendons in each direction. The major design or construction deficiencies which led to total collapse were; improper drape of post-tensioning tendons adjacent to elevator openings, overstressed concrete slab sections adjacent to two temporary floor slots for cast-in-place shear walls, overstressed and excessively flexible steel lifting angles during slab lifting, and unreliable and inadequate temporary slab-column connections to assure frame stability.

Figure 4. Failure of L’Ambiance Plaza under construction

And finally, prime example was the terrorist attack of 11 September 2001 in USA – towers of World Trade Center, designed for hit of plane and the possibility of fire (but not in case of their simultaneous action), were destroyed in 40-50 minutes as a result of focused and combined effects. After these disasters the careful consideration of progressive collapse phenomenon was resumed.

Among normative documents of Russia (USSR) term “progressive collapse” first appeared in “Manual for design of residential buildings”. Necessity of calculation on element failure was introduced to norm in 1988. Between 1999 and 2006 methods of calculation on “progressive collapse” were published in the number of recommendations (large-panel buildings – 1999, frame buildings – 2002, with load-bearing brick walls – 2002, residential monolithic building – 2005, high-rise buildings – 2006).

Since 2001 MGSN 3.01-01 “Residential buildings” and MGSN 4.19-05 “Multi-functional buildings and complexes” require to ensure resistance of buildings to progressive collapse. Also the problem of “progressive collapse” is considered in SNiP 20-01-2003 “Reliability of structures and buildings”. But the attention was attracted by normative documents which regulate this problem for large-span buildings – MDS 20-2.2008 “Temporary safety recommendations for insurance safety of large-span structures to avalanche (progressive) collapse during emergency exposures” and STO 36554501-014-2008 “Reliability of the constructions and the foundations. General rules”.

The most useful and full source, which was used in this work, is STO – 008 – 02495342 – 2009 “Prevention of progressive collapse of reinforced concrete monolithic structures of buildings”. This document gives definition of progressive collapse, recommendations about designing of elements that allow avoiding this type of destruction. There is also an example of calculation for a structural frame of reinforced concrete at preventing progressive collapse.

In addition to the problem of literature shortage there are many other real challenges which are raised in reasoning of some scientists. Most of them are connected directly with ensuring safety for buildings.

The first problem is the current system status of complex safety with an allowance for new global risks and threats. Nowadays in Russia the operational system of control and safety ensuring of buildings and structures during their life cycle significantly lags behind the high rates of construction. That means that the old system does not fully meet the requirements of safety in condition of growth of man-caused processes and manifestations of terrorist threat – the old system cannot solve new problems and tasks. It is the result, firstly, of difficulties of transitional stage of the Russian economy and, secondly, of erroneous concept of town planning of the Soviet period. (Korol et al., 2009, p. 23).

It is also good to know that in modern period of high technologies, market economy, and infatuation of “quick” money the matter of quality and reliability of site investigation, design, construction and operation of building objects does not consider properly while scientific and technical base lags behind in the formation. In other words the required level of safety in many areas has not been achieved: in estimation of natural and man-caused risks, fast deterioration of building structures in real urban condition; in control of reliability and quality of building materials and structures, technologies, construction and assembly work. Expert, inspection and supervisory activities in construction do not have modern theoretical, methodological and instrumental providing. Furthermore, practice shows that often there is replacing labor-intensive calculated, expert and supervisory databases of protocols, local findings and opinions. (Korol et al., 2009, p. 24).

One more problem is connected with unreasoned infatuation of information technologies, the aim of which is reducing the cost of calculated, technical and design accompaniment of engineers. In Russia substitution of concepts takes place – information technologies try to replace qualified engineering activities. Even in our investigation it is not allowed to use only specialized programs without monitoring the process because no program can consider all factors. (Korol et al., 2009, p. 26).

All these problems, which were met during researching, should be considered to understand clearly the complexity of the phenomenon.

2.2 Definition of progressive collapse

Progressive collapse means consecutive destruction of load-bearing structures of the building (structure), due to the initial local damage of the individual bearing structural components and leading to the collapse of the building or its significant part (two or more spans and two or more floors). (STO – 008 – 02495342 – 2009, p. 2). Another clear definition, which describes simplistically the process of destruction, is given in American standard ASCE 7-02: the spread of local damage, from an initiating event, in the form of a chain-reaction from element to element resulting, eventually, in the collapse of an entire building or disproportionately large part of it.

The underlying characteristic of progressive collapse is that the final damage is disproportionately greater than the original cause.

Because progressive collapse is a dynamic event, the failure front divides the structure into zones – without effects of the progression of failure and the failed parts of the structure. A failure front may propagate horizontally or vertically, or in both directions. Because column stiffness is much greater than beam bending stiffness, vertical propagation of the failure front will be faster than horizontal. Due to this difference in time scale, it is more logical to use different analytical models for different stages of the failure.

2.3 Causes of progressive collapse

Progressive collapse can be caused by abnormal loading which initiates the local damage in case if a structure does not have adequate continuity, ductility, and redundancy.

The initial local damage of structural elements of the building is possible under emergency situations (gas explosions, terrorist attacks, aircraft, fires, seismic impacts and failures of footings, assaults transport, defects of design, construction or reconstruction, etc.) which are not provided by the terms of the normal operation of the building. Accidents and damages of bearing structures, caused by design, manufacture or installation errors, inadequate quality of materials, and improper use of buildings can also be reasons of collapse.

Potential abnormal load hazards, which could trigger progressive collapse, have a low probability of occurrence and are either not considered in structural design for economic reasons or addressed indirectly through passive protective measures rather than by structural calculations. Characteristically, the loads usually act over a relatively short period of time in comparison with ordinary design loads. The loads generally are time-varying, but may be static or dynamic, depending on the frequency content of the load and the dynamic response characteristics of the resisting structural system. (Bruce et al, 2007, p. 10).

A majority of structural failures in ordinary buildings occur as a result of errors in planning, design and construction. Such errors result from human imperfections and are difficult to quantify. This source of "abnormal" load is better dealt by the engineer recognizing that things can go wrong, through a consideration of hazard scenarios, and through improvements in quality assurance and control.

2.4 Types of progressive collapse

Cases of progressive collapse can be divided into 6 types depending on the reasons for the progressivity, respective mechanism and characteristic features: pancake-, zipper-, domino-, section-, instability- and mixed-type collapse.

2.4.1 Pancake-type collapse

This type of collapse is a sequential failure, features of which are separation of structural components, release of potential energy and the occurrence of impact forces. The potential energy of falling components can far exceed the strain energy stored. It depends on the size of the falling components (Figure 5). The collapse of WTC towers of New York in September 2001 is an example of this type of collapse – the loss of structural member was limited to the few stories but it progressively extended throughout the height of tower. The potential energy of the upper part of the collapsed members converted into kinetic energy which turned in to impact force that was far than the resisting capacity of the lower floors and ultimately resulted in to total collapse of the tower. (Starrosek, 2009, p. 12).

The mechanism of this type of a collapse is the following:

- initial failure of vertical load-bearing elements;

- separation of structural elements, their falling in vertical direction;

- energy conversion: gravitational potential energy into kinetic energy;

- impact of falling elements on the surviving part of structure;

- failure of other vertical load-bearing elements due to the axial compression forces from the impact loading;

- failure progression in the vertical direction.

(Starrosek, 2009, p.13)

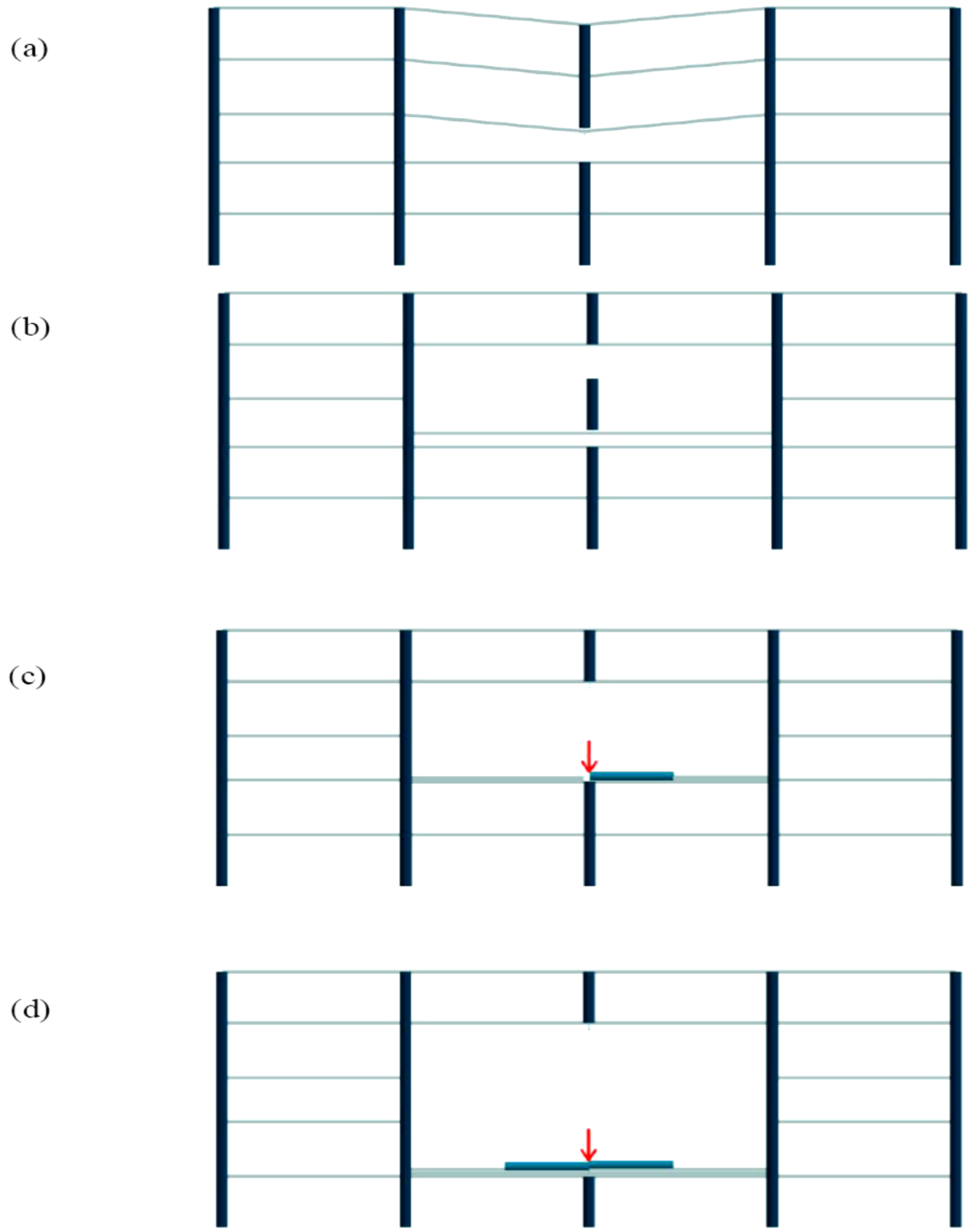

Figure 5. The steps of pancake-type progressive collapse: a) the initial failure of a column; b) transformation of the structures potential energy to kinetic energy; c) overloading of the structure below the initial failure; d) the progression of the failure. (Johan R., 2010, p. 17)

2.4.2 Zipper-type collapse

This type of collapse is initiated by rupture of one cable and propagating by overloading that leads to rupture of adjacent cables (Figure 6). Characteristic features for this type are the redistribution of loads into alternative load paths, the impulsive loading due to sudden element failure, and the concentration of static and dynamic forces in the adjacent element. (Starrosek, 2009, p. 14).

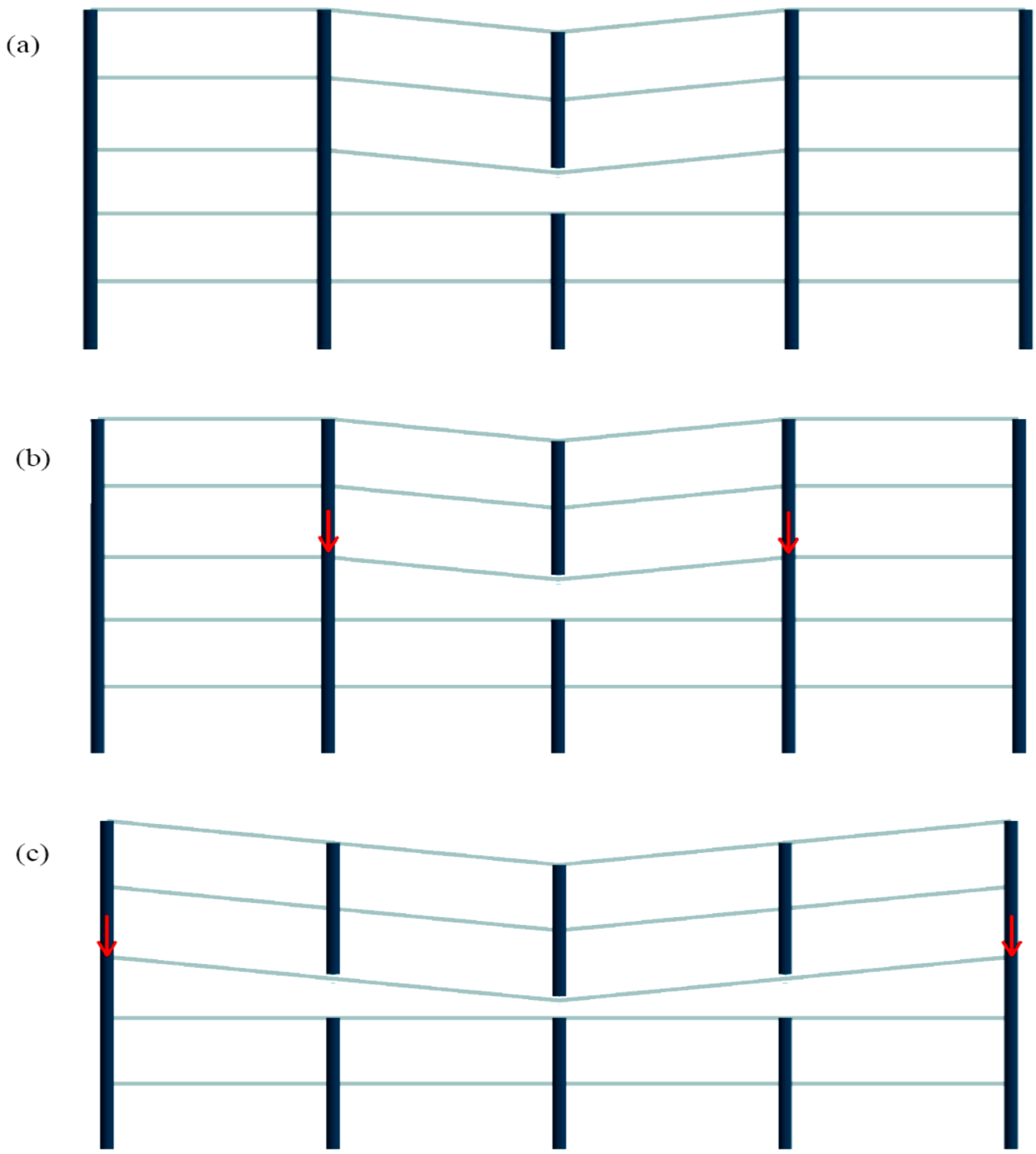

Figure 6. The steps of zipper-type progressive collapse: a) the initial failure of a column; b) increasing loading of the closest columns; c) overloading the columns resulting in a progressive collapse. (Johan R., 2010, p. 19)

This type of collapse is initiated by rupture of one cable and propagating by overloading that leads to rupture of adjacent cables. Characteristic features for this type are the redistribution of loads into alternative load paths, the impulsive loading due to sudden element failure, and the concentration of static and dynamic forces in the adjacent element. (Starrosek, 2009, p. 14).

An example of this type of a collapse can be suspension Tacoma Narrows Bridge, where strong wind caused the destruction of the central span of the bridge (Figures 7, 8). There has been a mistake in the design – the bridge was properly designed for the action of static loads, including wind, but the aerodynamic effect of the load was not taken into account. Rupture of hangers of the central span led slack side spans and the slope of the pylons. After the first hangers of the bridge snapped due to wind induced vibrations of the bridge girder, the entire girder peeled off and fell. Impact force does not typically occur in this type of a collapse, which is the case in pancake-type collapse.

The sequence of a collapse:

- initial failure of one or several load-bearing elements;

- redistribution load carried by this elements into the surviving structure;

- impulsive dynamic loading due to the suddenness of the initial failure and redistribution of loads;

- dynamic response of the surviving structure to impulsive dynamic loading;

- concentration of forces in load-bearing elements that are similar in type and function to and adjacent to initially failing elements due to the combine static and dynamic structural response to that failure;

- overloading and failure of those elements;

- failure progression in a direction transverse to the principal forces in the failing elements.

(Starrosek, 2009, p.15)

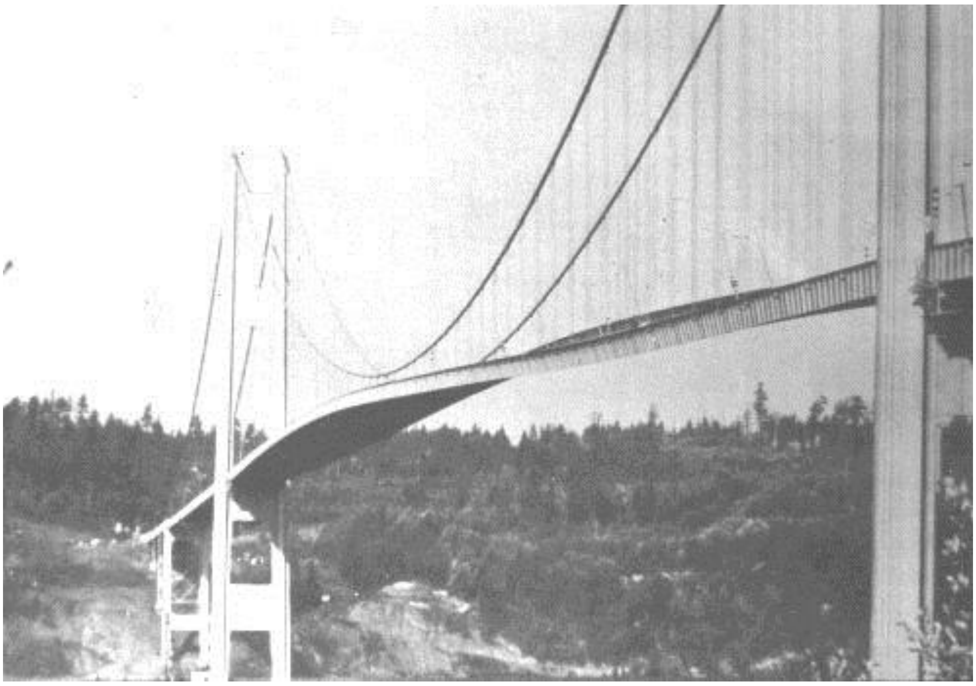

Figure 7. Tacoma Narrows Bridge - Galloping Gertie. Swaying roadbed in windy weather

Figure 8. Tacoma Narrows Bridge after collapse

2.4.3 Domino-type collapse

A row of dominoes collapses in a chain reaction when one element falls at the push of finger (Figure 9). Features of this collapse are overturning of the individual element and the next elements due to the propagating the horizontal pushing force as the principal forces and the propagating action are not parallel (the principal forces in the failing elements are vertical). (Starrosek, 2009, p. 16).

The mechanism is the following:

- initial overturning of one element;

- fall of that element in an angular direction around a bottom edge;

- energy conversion: gravitational potential energy into kinetic energy;

- lateral impact of the upper edge of the overturning element on the side face of an adjacent similar element;

- the horizontal pushing force transmitted by that impact is of both static and dynamic origin;

- overturning of the adjacent elements due to the horizontal pushing force from the impacting elements

- failure progression in the overturning direction.

(Starrosek, 2009, p. 16)

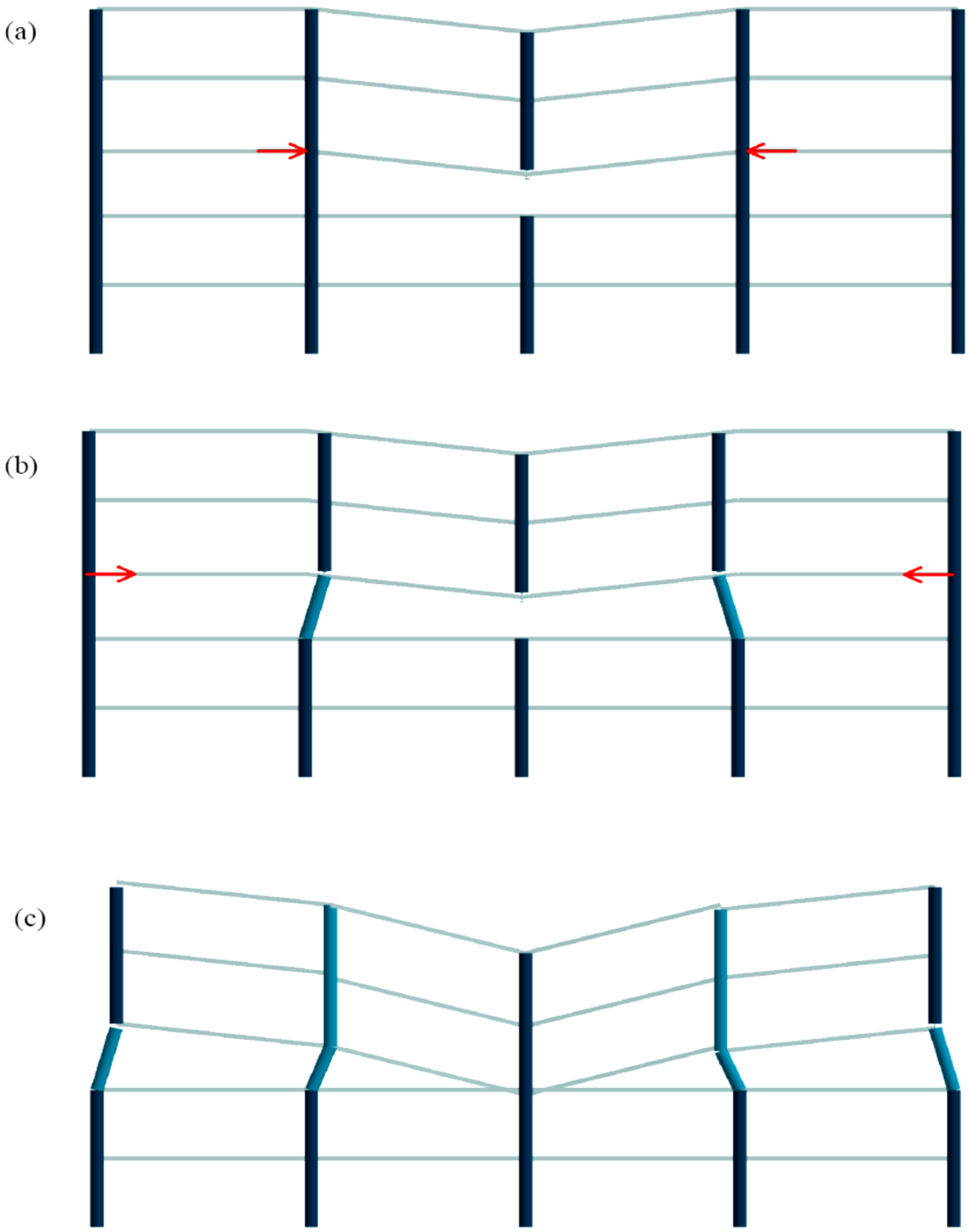

Figure 9. The steps of domino-type progressive collapse: a) initial failure and loading of the column next to it; b) overturning of the columns; c) the progression of the collapse due to the overturning (Johan R., 2010, p. 21)

2.4.4 Section-type collapse

When a member under bending moment or axial tension is cut, the internal forces transmitted by that part are redistributed into the remaining cross section. The corresponding increase in stress at some locations can cause the rupture of further cross sectional parts, and, in the same manner, a failure progression throughout the entire cross section. This type of failure can be termed as “fast fracture” instead of progressive failure. (Starrosek, 2009, p. 19)

The features and mechanism of this type of collapse are similar to zipper-type collapse. But instead of such a term as “element” or “remaining structure” it is more reasonable to use “part of cross-section” and “remaining cross-section”.

2.4.5 Instability-type collapse

Instability of structure is characterized by small imperfection which leads to large deformations or collapse. For example, the failure of a bracing element due to some small triggering event can make a system unstable and result in collapse (Figure 10). Another example is failure of a plate stiffener leading to local instability and failure of the affected plate, and possibly to global collapse. Here propagating destabilization occurs when the failure of destabilized elements leads to the failure of stabilizing elements. (Starrosek, 2009, p. 20).

The mechanism is the following:

- initial failure of bracing or stiffening elements that have been stabilizing load-bearing elements in compression;

- instability of the elements in compression;

- sudden stability failure of these destabilized compressed elements due to small influences;

- immediate collapse of failure progression.

(Starrosek, 2009, p. 21)

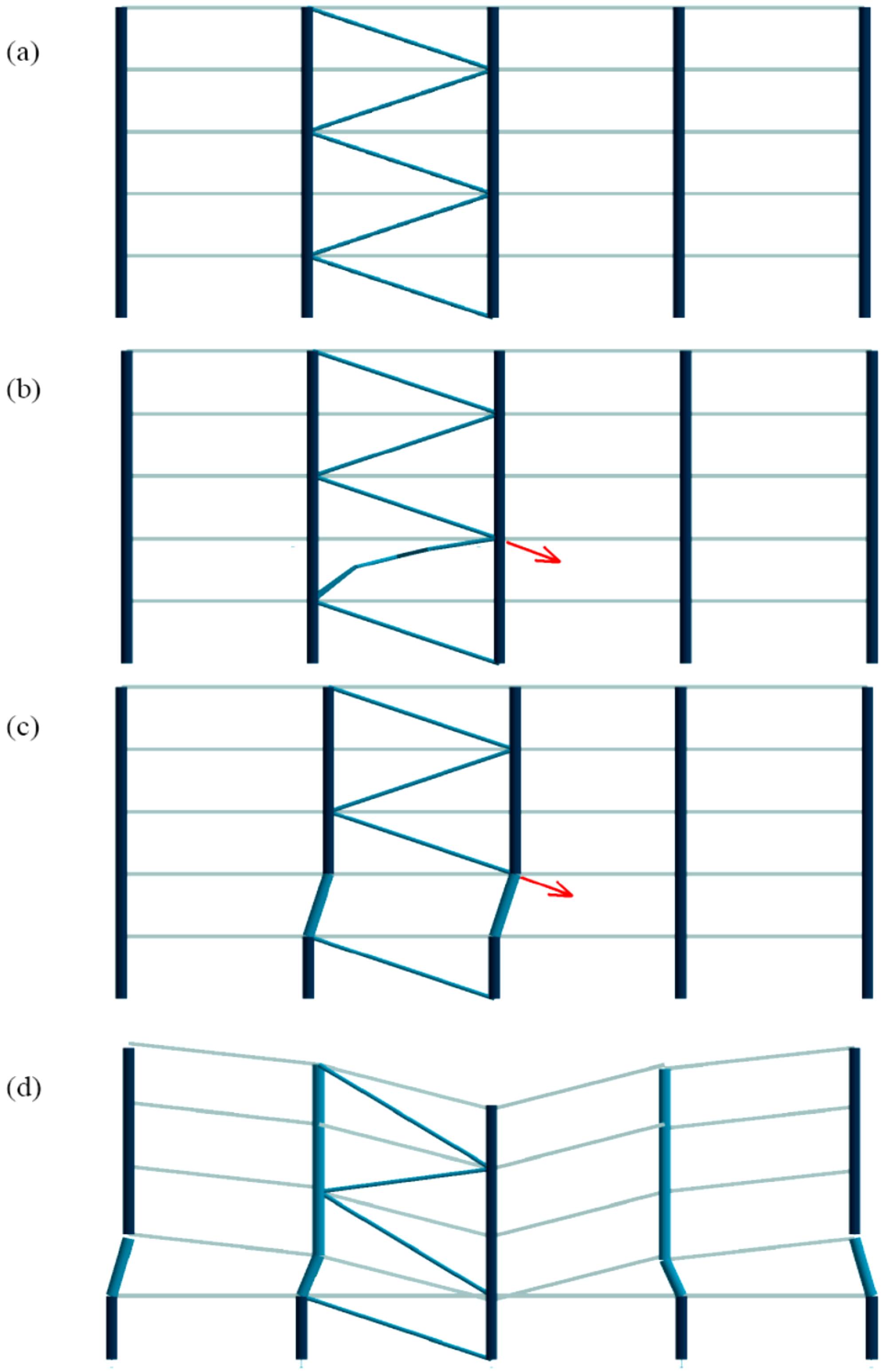

Figure 10. The steps of instability-type progressive collapse: a) the original structure with a bracing truss; b) initial failure in the truss; c) loss stability due to the lost part of the truss; d) the collapse due to the instability. (Johan R., 2010, p. 23)

2.4.6 Mixed-type collapse

This type of collapse can be assigned to the structure where one or more possible failure reasons fall in to different category of progressive collapse. The feature of this type is the occurrence of horizontal forces that lead to the overturning of other elements. (Starrosek, 2009, p. 23)

For example, the partial collapse of the 9 storey Murrah Federal Building (Oklahoma City), which was the target of terrorist attack in April, 1995, seems to have involved features of both a pancake-type and domino-type scenario (Figures 11, 12). The blast destroyed one of the perimeter concrete columns and caused brittle failure of 2 others. Suspended transfer girder resting on the exterior columns failed due to loss of support allowing collapse of upper floors. The horizontal forces, induced by an initial failure, that lead to overturning of other elements. This horizontal tensile force could have been induced by falling components and transmitted to other elements through continuous reinforcing bars.

Figure 11. Murrah Federal Building after bomb attack

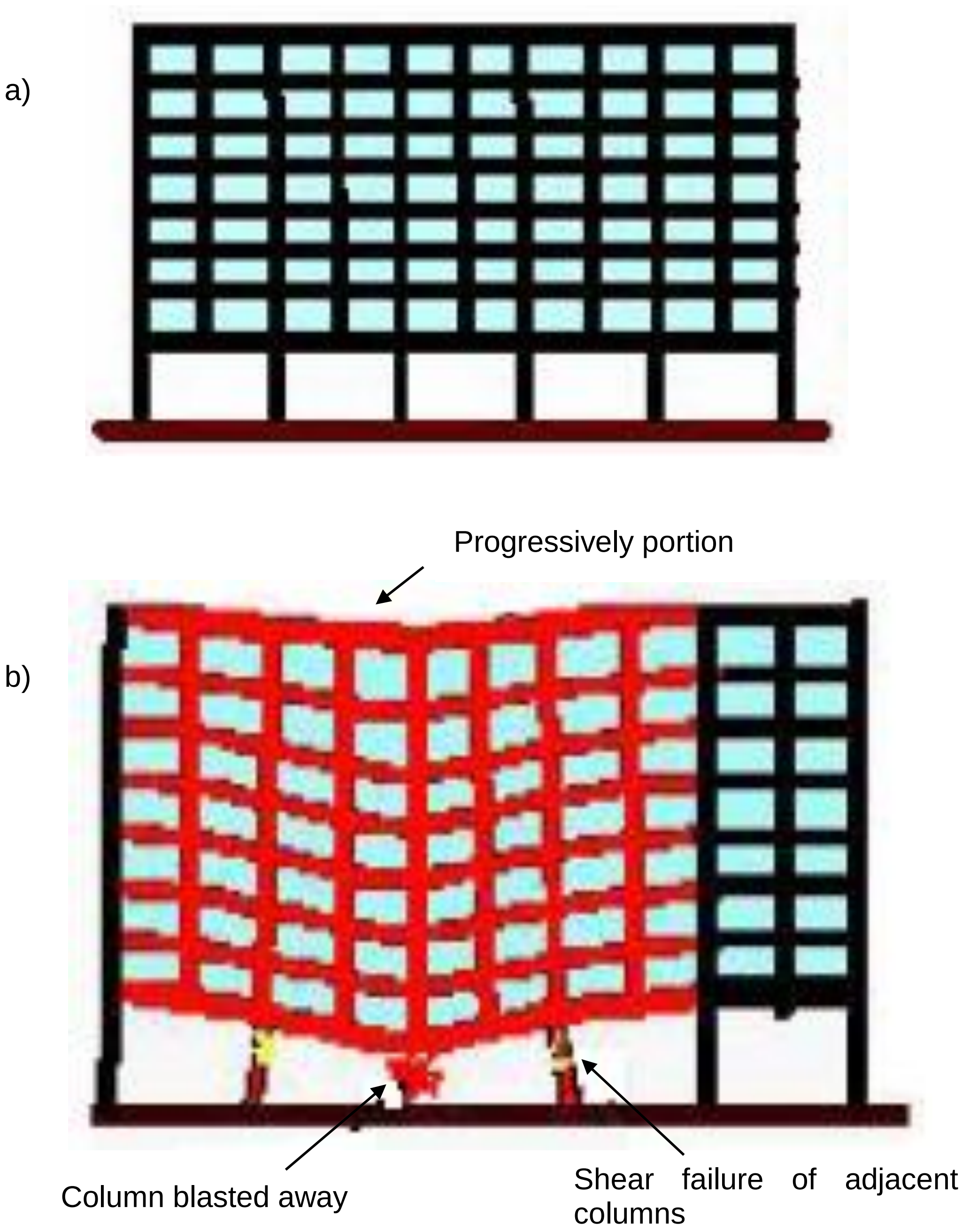

Figure 12. The structure of Murrah Federal Building before (a) and after (b) the car bomb attack. (Department of Civil and Environmental Engineering, University of California, 2004)

2.5 Issue of survivability

Sometimes isolated failures of individual elements do not cause the accident, it is connected with spare capacities of the system. The property of a system to maintain its load-bearing capacity in case of failure of one or several elements is called survivability, or viability. (Perelmyter, 2007, p. 13).

In construction this term is applied to earthquake engineering. It was suggested the idea of selecting “key” load-bearing elements whose reliability ensures the work of buildings without full collapse in emergency actions. The object of the research is not the original system but damaged construction with modified properties.

To check on the viability of the building one should distinguish between random and planned in advance (sabotage, terrorist attacks) damage.

For accidental (random) damages the principle of single failure is taken into account. It means that, even in a complex system, only one structural element can fail, after that the exploitation of the building must be stopped. If in this situation the construction is capable of redistribution of stresses to adjacent elements, then it means that the building has the potential survivability. (Kudishin, 2009, p. 30). Then problem boils down to the right choice of “key” (the most strenuous) elements.

For a planned damage multi-element failures are possible. In this case it is necessary to predict damage script and perform checking for survivability. The probability of such types of damage can be reduced by the safety measures – technical and organizational.

Nowadays there are attempts to explain earthquake resistance as a solution of issue of survivability. But it is not correct because in the issue of earthquake resistance efforts are known and design is going by the rules like for acting of wind, snow and other impacts. In the issue of survivability the reasons of damages, in which place and in which way destruction will happen, are unknown.

2.6 Application area

STO – 008 – 02495342 – 2009 defines which concrete constructions of residential, public and industrial buildings shall be calculated and protected from the progressive collapse in case of emergency. These include objects destruction which can lead to great social, environmental and economic costs:

1. Residential buildings with height more than 10 floors;

2. Public buildings with congestion of 200 people and over at the same time within the block limited by movement joints:

- teaching and educational;

- of health and social services;

- of servicing (trade, catering trade, consumer and utility services, telecommunications, transport, health and public services);

- of cultural and leisure activities and religious ceremonies;

- for administrative and other purposes (local government offices, libraries, research, design and construction organizations, financial institutions, judicial and legal institutions and prosecutors, publishing organizations);

- for temporary stay (hotels, motels, dormitories, etc.).

3. Industrial and secondary buildings with congestion of 200 people and over at the same time within the block limited by movement joints.

3 METHODS TO PREVENT PROGRESSIVE COLLAPSE

3.1 Direct method

Direct methods require more sophisticated analyses compared with the usual load analyses used in routine design. In this method attention is directed on corresponding designing of "key" structural elements. It is needed to increase the strength of these elements. It helps to ensure the resistance of failure under abnormal loads or design structure so that it can bridge across the local failure zone. (Bruce, et al., 2007, p. 43).

3.1.1 Alternate load path method

It is necessary to fulfill the requirement so that the construction does not lose bearing capacity in case of removal part of elements in emergency impacts.

The constructive system is designed to cover the loss of one or several elements, providing the alternative path loads through the redistribution of efforts. In other words, a building must bridge across a removed element, and transfer gravity loads to undamaged portions of the structure while maintaining stability.

The work of the whole construction by the alternative load path method can be analyzed by removing one or several elements in design model and checking the possibility of progressive collapse at the same time – it can be perimeter columns or load-bearing walls, interior load bearing elements (Figure 13). The load carried by the lost element must find an alternate load path to the building supports without precipitating structural collapse. Large deformations are permitted before the onset of failure of an element. This method reduces the risk of progressive collapse by ensuring structural redundancy. The method does not require characterization of the threat causing loss of the element. But on practice difficulties arise with justification for removal of a particular element, selection of significant elements among a large number of possible local damage and determination of allowable quantitative criteria of damage.

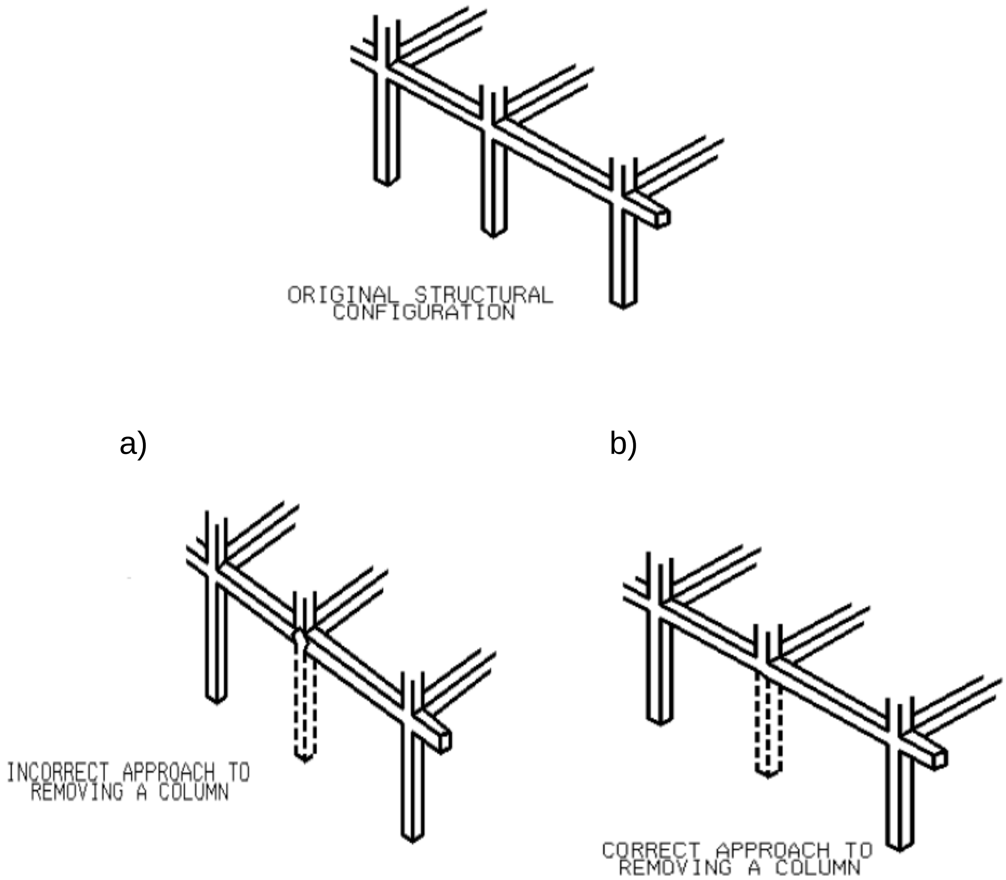

Figure 13. Removal of column from Alternate Path Model: a) incorrect removing a column; b) correct removing a column (UFC 4-023-03, 2010)

3.1.2 Specific local resistance method

This method requires that the building or its parts provide sufficient strength to resist a specific load or threat. (UFC 4-023-03, 2010, p. 13).

Critical vertical load-bearing building components are locally hardened and detailed to withstand the given emergency impacts (such as blast pressures) or to develop the full resistance of each key element without failing the connections or supporting members. In this case strength, structural integrity and rigidity of constructive “key” elements are provided by reinforcement. This option assumes obligatory regulation of intensity of emergency loads.

There is such an item as Enhanced Local Resistance in which the shear and flexural capacity of the perimeter columns and walls are increased to provide additional protection by reducing the probability and extent of initial damage. (Jesse, 2010).

3.2 Indirect method

The aim of this method is finding the approaches to provide a minimum level of connectivity between structural components and increase the overall robustness of the structure. This is accomplished by general structural integrity and continuity, layout of load-bearing elements, and working-out of connections (requirements for minimum joint resistance). In other words, in this method it is needed to provide minimum levels of strength, continuity and ductility. In this method catenary action of structure can be regarded.

This method consists of two options:

2.1. The 1st option lies in excessive increase in the extent of static indeterminacy for system. It can be reached by the inclusion of additional ties in frame construction or the involving in the work of secondary elements that provides a spatial work of construction.

2.2. It is elimination or reduction of the influence of emergency impacts by preventive and organizational measures.

3.3 Basic measures to ensure the safety of large-span structures from the progressive collapse

All of the above activities are examined in more detail.

The reliability of building constructions should be provided at the stage of development of general conception of buildings, during designing, manufacturing of structural elements, their construction and operation.

Buildings must be designed, built and operated so that the detriment of the accident impacts does not reach the size, much larger than the impact of the initial local damage.

Aspects which should be considered and regulated in designing to avoid destruction are described in detail in the main sources.

The main attention is focused on the constructive system which should ensure the bearing capacity of buildings even after initial local damages. STO – 008 – 02495342 – 2009 makes the following demand: “Constructive system of the building should not be subjected to progressive collapse in the event of local failure of individual components in emergency situations, not provided the terms of the normal operation of the building. That means that with special load combination a local destruction of individual elements of the structural system of the building is allowed, but the damage should not lead to the destruction of other structural elements are altered (secondary) structural system”.

To prevent progressive collapse in a primary structural system the following points should be provided in designing:

1. Reasonable design and planning solution of the building with taking into account probability of an accident;

2. An increase static indeterminacy of the system;

3. Using of constructive solutions for load-bearing structural elements and their connections that provide plastic (inelastic) deformations;

4. Necessary strength of load-bearing constructive elements;

5. Determine concrete class and reinforcement of structural elements required for normal operation.

All of the above arrangements should be considered for normal operation of buildings. But besides them it is required to make calculations for other conditions, namely:

1. After accident several structural elements break down resulting in design model will be changed. So it is needed to carry out static calculations of changed constructive systems according to new design models with using special load combinations;

2. New reserves of safety should be set for secondary structural system. If it is not enough there is possible the following arrangements – to increase the dimension of elements or change design and planning solution of building.

To avoid progressive collapse in secondary structural system, it should be designed to provide the following condition: value of effort in structural elements, determined with certain loads according to standard must be less then value of effort in this elements identified with the limiting values of materials characteristics. (STO – 008 – 02495342 – 2009, p.3).

3.3.1 Design and planning solution

Rational design and planning solution of buildings in terms of preventing progressive collapse is a constructive system that in the event of failure of a single (any) of the vertical bearing components provides transformation constructions over the outgoing element in a "hung" system which able to transmit loads on the surviving vertical construction. (STO – 008 – 02495342 – 2009, p.3).

For creating the above constructive system the following measures according STO – 008 – 02495342 – 2009, p.3 are provided:

1. Monolithic connection of floor structures with concrete vertical elements (columns, pilasters, exterior and interior walls, ventilation shafts, etc.);

2. Reinforced concrete monolithic belt along the perimeter of the floor slab, combined with structures of floors and performed the functions of a jumpers above the windows;

3. Reinforced concrete monolithic parapets, combined with the structures of floors;

4. Reinforced concrete walls in the upper floors of the building or reinforced concrete beams in floors, which combine columns (pilasters) between themselves and with other vertical reinforced concrete elements (walls, fences stairwells, ventilation shafts, etc.);

5. Jumpers in concrete walls are made not on the whole height of the story – there are plots of solid walls above the jumpers.

6. Enhancing of the overall robustness in frame structures and ensuring of integrity are possible by the proper plan layout of walls and columns. Column spacing should be limited. Large column spacing decreases the likelihood that the structure will be able to redistribute load in the event of column failure. In bearing-wall structures, interior longitudinal walls should be arranged to support and reduce the span of long sections of cross wall, thus enhancing the stability of individual walls and of the structures as a whole. In the case of local failure, this will also decrease the length of wall likely to be affected.

Exceeding the applied step of columns can create the conditions for the loss of stability in case of the progressive collapse. Here preventing the collapse will require a significant increase in consumption of concrete and reinforcement and complexity of the design of reinforcement and connections.

It is recommended to use spatial structures as a long-span roof – solid and core shells, domes, hanging guyed and tent roof. When applying traditional structures – trusses, frames and arches – it is needed to raise the degree of static indeterminacy by including in the system additional ties that provide spatial work of large-span roof.

Every activity during design should be aimed at producing robust structures capable of limiting the spread of damage due to an initiating event.

3.3.2 Preferred system

A preferred system to resist progressive collapse is cast-in-place reinforced concrete with beams in two directions. For this system beams should have continuous top and bottom reinforcement with tension lap splices. Stirrups should develop the shear capacity of the beams and be closely placed along the entire span. Two-way floor slabs are preferred over one-way slabs, as they provide greater redundancy. Top and bottom reinforcement in concrete slabs should extend into beams and columns to improve capacity to withstand load reversals. (Bruce et. al., 2007, p. 56).

In concrete flat slab or flat plate systems, it should be included features to enhance their punching shear resistance, such as using column capitals, drop panels, or special shear reinforcement. Continuous bottom reinforcement should be provided through columns in two directions to retain the slab in the event that punching shear failure occurs. (Bruce, et al., 2007, p. 57).

3.3.3 Tying systems

In the tie force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. “Key” elements of structure must be tied together so that after the loss of major structural members of system the loads should be redistributed throughout the surviving structures through load paths. This ability of a structure is based in large part on the interconnectivity between adjacent members called “tying a building together” by using a system of ties in three directions along the principal lines of structural framing (figure 14).

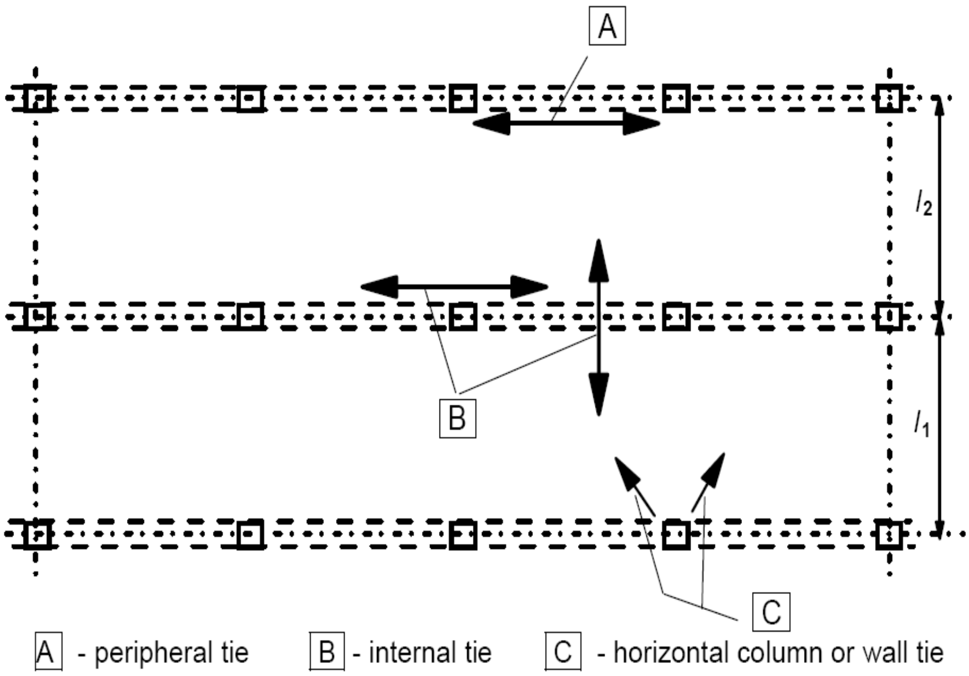

Figure 14. Tying system according to Eurocode EN 1992-1-1

Where a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system. The ties consist of peripheral, internal, vertical (particularly in panel buildings) and horizontal ties to columns and walls (Figure 15).

The design tie strengths are considered separately from the forces that are typically carried by each structural element due to live, dead, wind and other loads. In other words, the design tie strength of a slab, beam, column, rebar, or connection with no other loads acting must be greater than or equal to the required tie strength. In addition, the tie members, its splices and its connections only resist the calculated tensile forces. (UFC 4-023-03, 2010, p. 24).

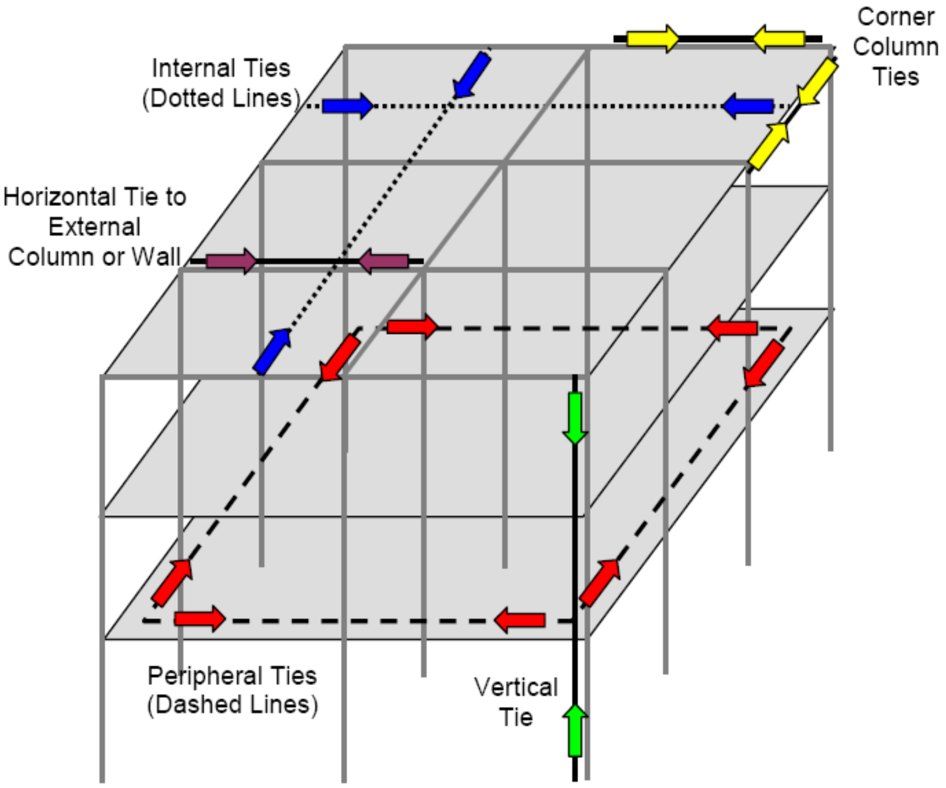

Figure 15. Tie forces in a frame structure for Accidental Actions (Bruce et al., 2007).

Internal ties at each floor and roof level should be situated in two perpendicular directions approximately at right angles. The ties should be continuous throughout their length and should be anchored to the peripheral ties at each end to transfer the load. The ties may be spread or grouped in different structural elements. The ties should not be spaced greater than 150 % of the spacing of the columns, frames or walls supporting any two adjacent floor spans in the direction of the tie. In walls, the ties should be relatively close to the top or bottom of the floor slabs. These ties should be capable of resisting a prescribed tensile force in each direction. (Bruce et al., 2007, p. 40-41).

Continuous peripheral tie should be provided at each floor and roof level within 1,2m from the edge of the building to be capable of resisting a prescribed tensile force. Each external column and the load-bearing external wall should be anchored or tied horizontally into the structure at each floor and roof level with a tie capable of developing a prescribed force, which calculated as a percentage of the total design ultimate vertical load carried by the column or wall at that level. (Bruce et al., 2007, p. 41).

Corner columns should be tied into the structure at each floor and roof level in each of two perpendicular directions. These ties have to develop the prescribed force, which is considered as a percentage of the total design ultimate vertical load carried by the column or wall at that level.

Vertical ties should be provided in columns and walls to limit damage of collapse of a floor in case of accidental loss of the column or wall below. These ties should form part of a bridging system to span over the damaged area. Each column and wall carrying vertical load should be tied continuously from the lowest to the highest level. The tie should take the largest factored vertical load, received by the column or wall from any one story. Where a column or wall is supported at its lowest level by an element other than a foundation (beam or flat slab) accidental loss of this element should be considered in the design and a suitable alternative load path should be provided. Setting ties, continuity and anchorage of these ties should be ensured. (EN 1992-1-1, 2004, p. 171).

Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure. Ties may be provided wholly within the insitu concrete topping or at connections of precast members. Where ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered. Ties should not normally be lapped in narrow joints between precast units. Mechanical anchorage should be used in these cases. (EN 1992-1-1, 2004, p. 171).

3.3.4 Catenary action

The theory of catenary action assumes that vertical load is carried by the plastic moment and the axial force of a horizontal member after the deflection of the member has become significant. Due to the large deflection the member will stretch which leads to plastic stretching and bending (Figure 16).The axial force, which decreases the plastic moment capacity of the member, has to be taken into account. Catenary action and plastic moment capacity underlie the calculation.

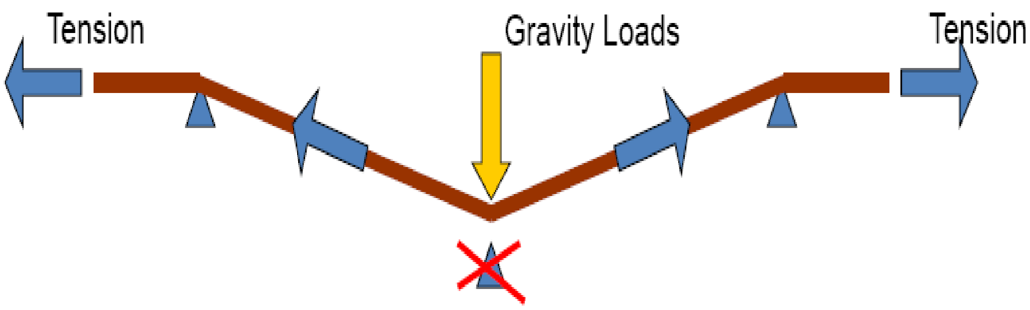

Figure 16. Catenary action (Jesse, 2010)

The concept of hardening a building using catenary action of steel cables consists of placing cables inside the floor slabs for new construction or adding the cables under the slab for existing structures as a measure of retrofit.

Figure 17 shows the application of this concept in a building. When a single column is removed and the floor starts to collapse, the catenary action of the cable prevents the collapse and transfers the load of the floor to neighboring columns and rest of the structures. Since cables are used in every floor, the loads of all floors above the removed column will be transferred to the adjacent columns. As a result, although the floors might have relatively large deformations (40-60 centimeters), the full progressive collapse is prevented.

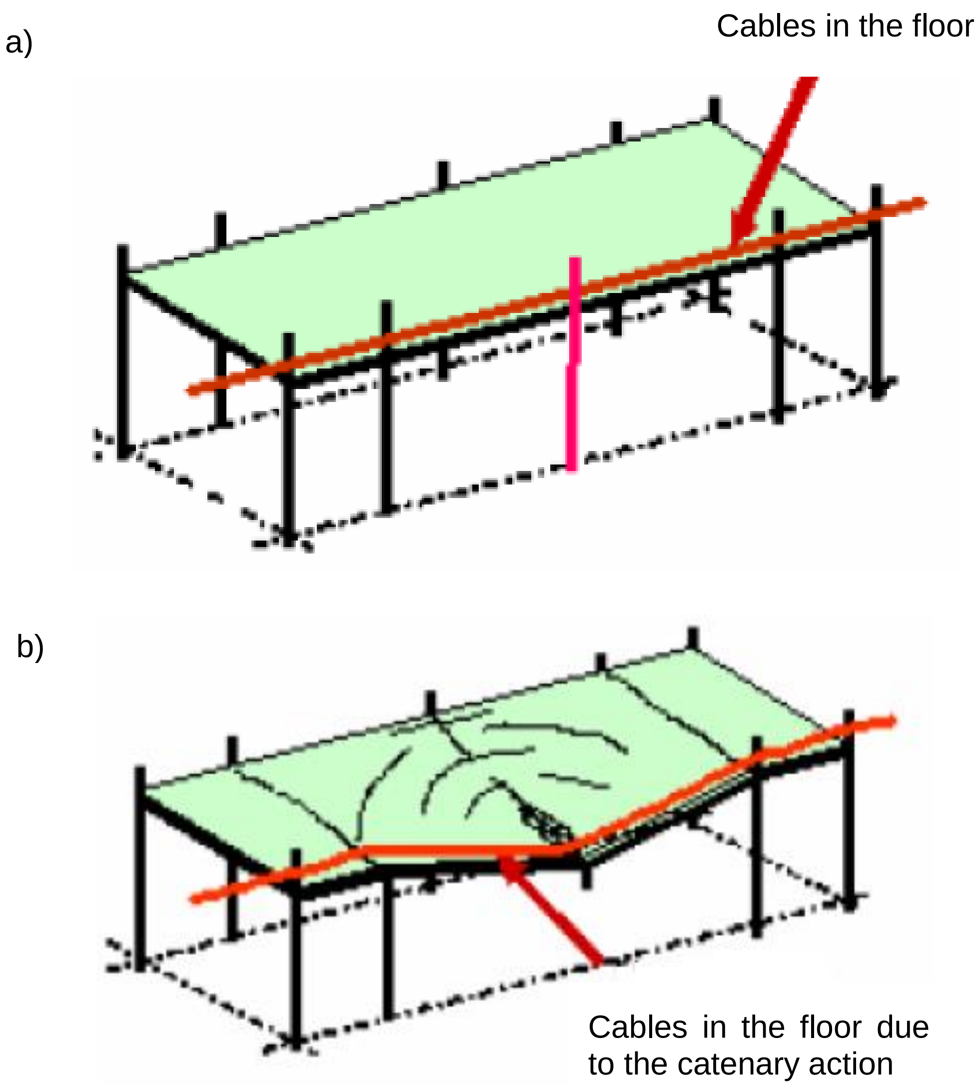

Figure 17. A typical floor with cable before removal of column (a) and after removal of the middle column (b). (Department of Civil and Environmental Engineering, University of California, 2004)

3.3.5 Loads

When calculating the large-span structures, it should be assigned the necessary reserves of bearing capacity of the key elements that ensure the overall sustainability of buildings. For "key" elements additional coefficients of working conditions are used. Their values depend on design life, span of structures and responsibilities of "key" elements.

Estimation of secondary structural systems to prevent the progressive collapse should be made on a special combination of loads, including normative values of dead and long-acting live loads, with the coefficient of combinations Ψ = 1,0. All loads are considered like a static with coefficient of reliability for the load $\gamma_n = 1{,}0$. (STO – 008 – 02495342 – 2009, p.3).

In this case dead load consist of dead weight of load-bearing reinforced concrete structures, weight of building parts, lateral pressure from soil weight and the weight of road coating. Long-acting live loads include reduced load of people and equipment, 35% of full normative loads of vehicles and 50 % of full normative snow loads. (STO – 008 – 02495342 – 2009, p.3).

For each of the estimated situation it is necessary to take into account all possible adverse settlement combinations of loads. For this purpose it should be considered all possible real options of simultaneous action of different loads and with a view to the implementation of various schemes of application of live loads. (STO 36554501 – 014 – 2008, p.17).

3.3.6 Characteristics of concrete and reinforcement

Reinforced concrete has a number of advantages by the resistance to progressive collapse. For instance, its significant mass helps to improve response to explosions because the mass is mobilized only after the pressure wave has diminished significantly reducing deformation. Members can be proportioned and reinforced to make structure ductile and continuity between the members. Physical size of reinforced concrete columns make them less susceptible to buckling in the event of the loss of a floor system. (Bruce et al., 2007, p. 56).

STO – 008 – 02495342 – 2009 gives certain instructions about applied characteristics:

1. The calculated values of resistance of concrete to axial compression, equal to their normative values, multiplied by coefficient of working conditions $\gamma_{b3} = 0{,}9$;

2. The calculated values of resistance of concrete to axial tension equal to their normative values, divided by a coefficient of reliability for concrete $\gamma_c = 1{,}15$;

3. The calculated values of resistance of longitudinal reinforcement to tension, equal to their normative values;

4. The calculated values of resistance of longitudinal reinforcement to compression, equal to the normative values of resistance to tension, except for reinforcement of class A500, for which Rs = 469 MPa (4700 kg/cm2), and reinforcement class B500, for which Rs = 430 MPa (4400 kgf/cm2);

5. The calculated values of resistance of transverse reinforcement on tension, equal to normative values, multiplied by the coefficient of working conditions $\gamma_{s1} = 0{,}8$;

6. The normative values of resistance of concrete and reinforcement, as well as the values of the module of elasticity for reinforcement E_s and the initial module of elasticity for concrete E_b in accordance to SP 52-101-2003.

If the building contains materials posing a high risk (flammable liquids, gases, explosive materials) the effects of fire must be considered by a separate analysis. To protect against a reduction in strength and stiffness that may result in progressive collapse from extended exposure to fire, the structural members and should have a fire resistance appropriate to their designed function.

3.3.7 Calculations

Constitutive models lie at the heart of any numerical solution of a structural problem. In the calculations on the local emergency impact of the individual elements should be used spatial design model. As such a model takes into account elements, which are not load-bearing in normal operation but they are involved in the redistribution of loads in case of occurrence of local impacts.

The design model should consider the capability of changing in nature of work for the whole system or its parts: consistent exclusion of constructive elements, change in the sign of efforts, redistribution of loads and change of strength and stiffness characteristics of the material. The greatest difficulty when a structure is being evaluated to determine its risk to progressive collapse is establishing the analysis approach and selecting performance criteria. In other words, it is needed to determine scenarios of collapse and find appropriate principles of calculations.

3.3.7.1 Options of calculation

There are three possible options for calculating (MDS 20-2.2008, p. 20):

1. Linear static method: based on small deformations of system and elastic work of the material;

2. Nonlinear static method: taken into account the physical and geometric non-linearity, loading history – from the zero state to the exhaustion of the bearing capacity;

3. Non-linear dynamic: taken into account the physical and geometric nonlinearity. Dynamic analysis is performed instantaneously by removing one of the elements of the loaded structure and analyzing the operation of the system until the damping of oscillations.

3.3.7.2 Preconditions for structural analysis

Preconditions for calculations are the following:

- almost immediate destruction of the elements of an arbitrary point;
- node connections of structural elements are the equal strength like the basic elements;
- survivability is ensured when the primary element failures do not lead to the destruction of other structures, on which loads are redistributed.

3.3.7.3 Recommendations for calculations

Recommendations about structural analysis can also be found in STO – 008 – 02495342 – 2009. The most important of them are:

1. Calculation of secondary constructive system is performed separately for each local failure. It is allowed to make calculation for the most dangerous cases of failure. These include system with destruction of vertical load-bearing elements in turn, which have the greatest load area, situate near the edge of floor or in the corner. Then the obtained results are spread for the rest construction.

2. For calculating should be applied primary structural system, which was estimated for normal conditions of operation, and transfer this system into secondary structural system by alternate exclusion of vertical load-bearing constructive elements. Vertical structures apply as rigid fixed at the top level of foundation.

3. Static calculation of secondary structural system should be performed as for elastic system, using such program as SCAD, Lira, STARK – ES.

4. The result of a structural analysis for primary and secondary structural systems is efforts in constructive elements, based on which the class of concrete and reinforcement for elements and connections are assigned, reserve the stability is chosen. If value of these characteristics is not enough it is needed to increase dimensions of sections or change the constructive solution of the building.

Strength and stability in the case of local emergency actions should be provided for at least the time required for evacuation. Moving structures and crack opening in emergency situations are not limited.

3.3.8 Design requirements

“Key” elements should be designed with taking into account the perception of the possibility of emergency impacts in addition to the standard design loads. Column or wall instability may result from the loss of lateral support due to the failure of a supporting floor system. In this case, exterior columns or walls should be capable of spanning two or more stories without buckling. The key elements should be detailed to develop the ultimate capacity of the materials in shear, flexure, and axial load. This detailing entails the use of continuous bottom reinforcement over supports, confinement at joints, adequate ties.

If no specific threats are identified for a structure, key elements may still be designed to be more robust and resistant to abnormal loading. In this case, key elements should be designed so that they can develop their full resistance against unexpected load without failing the connections or supporting members, thus, it is possible to avoid redistributing of loads.

Load bearing reinforced concrete construction can provide a considerable level of protection if adequate reinforcement is provided to achieve ductile behavior. Reinforced concrete main load bearing elements should be designed with a larger number of clamps, spiral reinforcement or external reinforcement sheet.

Longitudinal reinforcement should be continuous. The sectional area for this reinforcement in floor slabs and beams is applied not less than 0,2% of sectional area of elements. Longitudinal reinforcement of vertical load-bearing constructive elements must take the effort of tension not less than 10 kN per square meter of the load area. (STO – 008 – 02495342 – 2009, p.5, 6).

Special attention should be paid to designing of elements and their connections, especially when performing reinforcement. Joints of the elements should be located outside the zone of maximum effort. In place of overlapping of constructive elements must be provided reliability of anchorage, which ensure continuity in reinforcement and help to achieve ductility. This is very important property of structure in emergency events because in this case members and their connections may have to maintain their strength through large deformations and load redistributions associated with the loss of key structural elements. The length of reinforcement and overlap of reinforcing bars should be increased by 20% in relation to the required. (STO – 008 – 02495342 – 2009, p.5).

Some advices for designing of structural elements are given in Bruce, et al, 2007, p. 57-58:

1. Beam design

- do not splice reinforcement near joint regions or middle spans;

- provide closely spaced confining steel to improve ductility and increase shear and torsion strength;

- larger members may provide more torsion resistance.

2. Column design

- ensure plastic hinging in beams by designing columns for larger moment than beam can deliver;

- detail columns to have confinement;

- splice column reinforcement at third-points, not at ends or middle span;

- continue confining ties through joint region;

- for loss of corner column, consider possible large moments and axial loads in adjacent columns.

3. Slab design

- lightweight concrete floor slabs will reduce load but the blast resistance performance can be enhanced by using normal weight concrete;

more reinforcing steel can help tieback adjacent beam to share load in event of column loss under beam;

provide some amount of continuous top and bottom reinforcement in both directions. Do not splice at middle span or at ends;

- slabs cast monolithically with beams and girders will provide more continuity and allow more load redistribution.

3.3.9 Preventive and organizational measures

Together with design measures, preventive measures also can be applied which help to exclude, prevent or decrease the degree of hazard from emergency impacts.

The main organizational measure is deceleration of the collapse by using space-planning, design, engineering, organizational arrangements to provide sufficient time and safe ways of evacuation from the building after the start of the local structural damage. The important thing is monitoring of the state of load-bearing structures and providing proper maintenance of the building. For example it is not allowed to store explosive materials in the construction, special rooms are provided to store them with the monitoring of correct operation.

3.4 Measures to prevent progressive collapse in existing buildings

Structural elements and connections in an existing structure can be strengthened. The purpose is increasing of the load capacity and ductility of certain critical structural elements or connections so that they can survive the effects of abnormal loads. The difficulty of achieving these requirements depends on the threat and the specific detailing of the existing structural elements.

If specific threats to a building are known, it is possible to upgrade elements against the expected hazards, for example – gas explosion (when the locations of gas lines and sources are known). Given the size, configuration, and volume of explosive gases, one can reasonably determine the energy release and the potential influence on surrounding structural components.

But usually specific threats are not well known in advance so forcing strengthening schemes to be performed from a general perspective. Then it is needed to identify and strengthen vulnerable elements and connections considering their role on the integrity of the structure but without specifying hazards. In this approach, the vulnerability of the structure as a whole with the ductility and strength of individual components is considered, disregarding the nature, location, and time of abnormal loading events.

Existing structures present certain limitations to the implementation of upgrading schemes: existing geometry, space limitations, and aesthetics. Often it is difficult to develop details: critical elements might be unreachable or it is impractical to install the needed upgrades due to space constraints. In this case the important role has the detailing of connections. But in practice it is difficult to determine the actual state of the existing building due to uncertainties or inaccessibility to documentation and deterioration of structures over time.

3.4.1 Enhance Redundancy

If a decision is made to modify the building, the solution will require the introduction of redundancy to the structure. This can be achieved by providing additional rotational and tensile capacity in joints or connections or by creating new alternate load paths, or using both of these technologies.

When it is difficult technically or economically to provide the required localized resistance, then there is the alternative way to strengthen structural elements and systems, and increase their ductility and capacity.

3.4.2 Local strengthening

For precast concrete structures increasing of tensile capacity in the connections is required. Usually the connections of these structures have been designed for gravity loads only. Positive connections for tension could be introduced to tie together the components of structures. (Bruce et al., 2007, p. 67).

3.4.3 Addition of alternative load paths

Alternate load paths can be performed by introducing two-way behavior in structures. Such modifications force structural systems to engage the resistance of more components when one or more critical elements have been damaged. This ability to spread out the load over more existing elements reduces the demand on each element. (Bruce et al., 2007, p. 67).

3.4.4 Two-way action

Two-way action in the framing system is the first solution to increase redundancy. Existing structural framing systems that can span two ways have greater robustness than structures that are constructed to span just one way. For instance, an interior column designed to support one-way framing system typically has only two adjacent columns to redistribute loads. In a two-way frame, eight nearby columns could be available to help share the load of interior column.

In general, it may be difficult to add two-way-action features to existing buildings. But sometimes addition of elements such as new beams can be enough.

3.4.5 Secondary trusses

This method is the second solution to provide of redundancy and lies in addition of diagonal elements at upper levels, to turn two or multiple-story column and beam systems into trusses. In this case, trusses would be engaged if a lower level column were to be removed, with the columns above the initial damage becoming tension members.

For this method attention should be paid to the connection between the new diagonal members and the existing structure with adequate strength to carry the new loads, and the ability of columns to act as tension members. The advantage of secondary truss systems is that they often can be designed to resist forces with relatively little deformation, as compared with other alternatives.

3.4.6 Strong floors

This method allows avoiding introduction of new elements. So it is the most successful method for ensuring redundancy. Few floors, which often distribute loads throughout the building, should be identified and developed. Individual floors are strengthened to support the load of several adjacent floors so that the areas where repairs are needed will be limited.

An advantage to the strong floor approach is that the floors with added robustness can be distributed throughout the height of the building.

3.4.7 Catenary action

The concept of providing of the catenary action involves engagement of tensile forces in members. In this case, elements will take on load as tension members herewith the adjacent structure needs to be able to resist the high horizontal loads and the flexural members must work while deforming to relatively small angles to the horizontal line.

Connections are complicated. Collapse actions cause very high forces in the cables, and these forces need to be anchored appropriately at cable ends and transferred effectively at each column. In addition, there is need for sufficient strength and stiffness in the framing system to be able to resist the horizontal component of the cable anchorage forces.

3.5 Conclusion

Progressive collapse is a complex phenomenon, distinguishing feature of which is that the final state of damage is disproportionately greater than the failure which initiated the failure.

There are two methods to prevent the progressive collapse – direct and indirect.

The direct method is carried out in several ways. Firstly, it can be alternate path way, which involves selection of such structural system that can cover the loss of element by redistribution of loads. Secondly, specific load resistance can be used – strengthening the most vulnerable elements and their connections.

The indirect method is aimed at increasing total hardness by a reasonable location of bearing elements and careful design of connections. It can be achieved by the catenary action (design of floor with cables) that allows increasing plastic moment capacity and by adding the extra ties to provide the spatial work of the system.

Protection of the existing building is more difficult task. It is caused by the fact that in practice it is problematic to change elements, strengthen them or change the location of the bearing members and the connections may be difficult to reach. So the attempts of local strengthening or adding the new elements are made.

In general providing the work of building without collapse requires thorough learning with consideration of many cases.

4 METHOD OF CALCULATION

Calculations were made with the student Anastasia Vasilieva.

Reinforced concrete buildings should be protected against progressive collapse in the event of local failure of load-bearing structures as a result of accidental emergencies. Survivability of buildings must be provided to verify by the calculations and constructive measures which contribute to development of plastic deformation in load-bearing structures under ultimate loads.

4.1 Condition

Checking of stability for resistance to progressive collapse is based on comparing the efforts of individual structural elements derived from static analysis with critical efforts that can be perceived by these elements. Stability against progressive collapse of the building is provided, if for any elements the following condition is observed (1):

$$F \leq S, \tag{1}$$

F – effort in a structural elements,

S – calculated bearing capacity.

In constructions, where strength requirements are not met, the solution can be increase of cross-sectional area of reinforcement and cross-section of elements.

4.2 Calculation of the structures against progressive collapse

Some prerequisites and assumptions for calculation were described earlier in 3.3.5 and 3.3.7. Here general information necessary for the calculation will be presented.

1. Calculation for the strength and survivability of the building should be made using special combination of loads, including the impact of the local forces on the structure, dead and long-acting live loads at the most dangerous case of local failure. In frame buildings this role can be performed by notionally removing columns, located on one (any) floor on the area of local failure. Local destruction may be located anywhere in a building. Herewith, spatial design model is used. Removal of one or more elements of the constructive scheme changes the character of the work of the elements, which are adjacent to the place of damage or hovering over it. This is necessary to consider in the appointment of the stiffness characteristics of the elements and their ties (joints).

2. Dead load is applied like dead weight of load-bearing reinforced concrete structures, the weight of building parts. Long-acting live loads include reduced load of people and equipment, 35% of full normative loads of vehicles and 50 % of full normative snow loads. Normative values of dead and long-acting live loads are considered according to current normative documents (or by the special task) with the coefficient of combinations $\varphi = 1{,}0$ and safety factor $\gamma_n = 1{,}0$.

3. When determining the limit forces in the elements (their carrying capacity) the following aspects should be considered:

a) The part of the long-acting live loads which is taken from the structural scheme for the design scheme without local destructions.

b) The part of the short acting live loads is taken like the difference of forces received from calculation of the structural scheme with the removal one of the bearing elements.

4. The calculated strength and deformation characteristics of the materials are taken as their normative values according to the current normative documents.

5. The values of the deformation and the width of the cracks in structures are not regulated.

6. In case when the plastic work of the structural scheme is provided in limit state the, it is recommended to make it by kinematic theory method of limit equilibrium, which gives the most economical solution. In this case the calculation of the building at each selected scheme is performed by the following procedure:

6.1 The most probable ways of progressive collapse of the structure, which lost their support is determined (to make the ways of destruction and determine all breakable connections, including the formed plastic hinges and find possible generalized displacements (ω_i) in the direction of efforts in these joints);

6.2 For each of the selected mechanisms of progressive collapse the limit efforts are determined. These efforts can be taken by all sections of plastic destroyed elements and connections (S_i) and also the plastic hinges. When the results of the external forces (G_i), which acted to the individual links of the structure, are found, the non-destructive individual elements or its parts move them in the direction of their actions (u_i);

6.3 The works of the internal forces (W) and external loads (U) to the possible displacements of the mechanism are determined:

$$W = \sum S_i, w_i;\ U = \sum G_i, u_i \tag{2}$$

6.4 After that the equilibrium condition have to be checked:

$$W \geq U. \tag{3}$$

6.5 In estimating the possibility of simultaneous structural collapse of all floors equilibrium condition (3) is replaced by (4):

$$W_f \geq U_f, \tag{4}$$

Where W_f and U_f are the work of internal and external forces for the movement of one floor structure element, the floors are separated by the bottom surface of slab, which belongs to the floor, located above the floor.

6.6 For each selected local destruction it is necessary to consider all of the following mechanisms of progressive collapse:

a) The first type of progressive collapse is characterized by simultaneous translational displacement down of all vertical structures (or parts thereof), located on a place of local destruction.

b) The second type of progressive collapse is characterized by simultaneous rotation of each structural part of the building, located over local destruction area, around its center of rotation. This kind of movement needs the destruction of the connections between failure and non failure constructions, and the connection destruction by the shear of vertical elements with overlapping.

c) The third type of progressive collapse is not only the collapse of the overlap, which is located directly above the beaten out vertical structure and initially supported on it.

d) The fourth type includes movement of structures of only one floor directly above knocked out vertical element. In this case starts the separation of the vertical structures from the overlap, which is located above them. If any design scheme condition (3) or (4) is not satisfied, it is necessary by the strengthening of structural elements or other actions to achieve its fulfillment.

6.7 In some cases it is more comfortable to consider the work of the slab above the removed column (wall) with large deflections as the elements of the hanging system, or with the membrane effect.

6.8 In bearing columns, which will not be situated above the local destruction its influence leads to increase stresses. It is necessary to make the calculation of the strength of these elements.

6.9 Every overlap of the building should be calculated for the perception of the weight of the overlying floor area of overlap (permanent and long-term dynamic load factor k_f = 1,5).

4.3 Sequence of calculation

In this part the sequence of calculation and explanation of applied values and factors which were used for the given building are shown.

1. To ensure work of construction in an emergency situation, it is needed to perform condition of stability against progressive collapse in case of removing a column (formula 1). It is allowed to delete any load-bearing vertical element and check the load-bearing capacity of the rest structure.

2. The most dangerous load-bearing vertical element is selected (for example the most loaded column);

3. Conventional removal of chosen element (Figure 18). When any load-bearing element is deleted, loads are increased for the other elements. Because of enhanced loads, plastic hinge in the middle of the span appears, that can lead to significant deformation. But construction can withstand these increased loads because part of these loads comes to membrane structures.

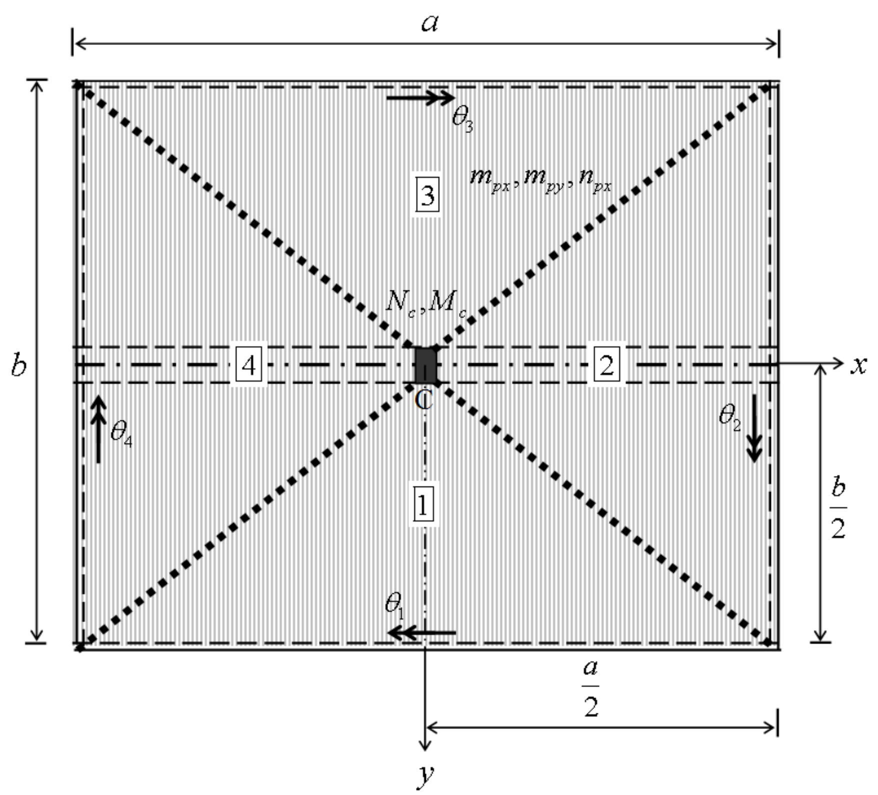

Figure 18. The shape of the yield lines when a column is notionally removed. (Aalto, 2010)

4. Finding the external load for the beam and slab structure with load factors (for live and dead loads and coefficient on responsibility equal to 1,0).

4.1 the average characteristic own weight for the slab and beam:

$$g = t_{slab} \cdot \rho_c + b_{beam} \cdot \frac{h_{beam}}{l_{beam}} \cdot \rho_c; \tag{5}$$

g – own weight of slab and beam structure (dead load), $\left[\frac{kN}{m^2}\right]$;

t_{slab} – thickness of slab, $[m]$;

ρ_c – density of concrete, $\left[\frac{kN}{m^3}\right]$;

b_{beam} – width of beam, $[m]$;

h_{beam} – height of beam, $[m]$;

l_{beam} – span, $[m]$.

4.2 final external loads which consists of calculated dead load and given live load:

$$F = g + q; \tag{6}$$

F – final external loads (effort in a structural elements), $\left[\frac{kN}{m^2}\right]$;

g – dead load, $\left[\frac{kN}{m^2}\right]$;

q - live load, $\left[\frac{kN}{m^2}\right]$.

5. Finding internal forces (bearing capacity of structures) – S, $\left[\frac{kN}{m^2}\right]$.

In this part material and geometrical characteristics for slabs and beams were applied according to existing Russian standards and norms. While safety factors γ, coefficients of combination ψ and materials factors equal to 1,0.

1. Calculation of plastic moment capacity for a beam

1.1. The following initial data is required for calculation:

N_{ed} – design value of the applied axial force (with reserve 3% due to post-tension tendons), $[kN]$;

$R_s = f_{yd}$ – design yield strength of reinforcement, $[MPa]$; can be found using the following formula:

$$R_s = f_{yd} = \frac{R_{s,n}}{\gamma_s}, \tag{7}$$

$R_{s,n} = f_{yc}$ – normative values of resistance to tension, $[MPa]$;

$R_{s,n} = 500\ \mathrm{MPa}$ is applied for reinforcement A500;

γ_s – safety factor for reinforcement:

$\gamma_s = 1{,}15$ – for reinforcement A500 in case of ordinary operation, $f_{yd} = 435\ \mathrm{MPa}$;

$\gamma_s = 1{,}0$ – in case of calculation against progressive collapse, $f_{yd} = 500\ \mathrm{MPa}$.

Geometrical characteristics of elements' section:

b – width of a beam, $[\mathrm{mm}]$;

h – heignt of a beam, $[\mathrm{mm}]$.

Characteristics of applied reinforcement:

A_{s1} – cross sectional area of upper longitudinal reinforcement, $[\mathrm{mm}^2]$;

A_{s2} – cross sectional area of lower longitudinal reinforcement, $[\mathrm{mm}^2]$;

$d_{s1} = d$ – effective depth of a cross-section, $[\mathrm{mm}]$;

d_{s2} – thickness of concrete cover, $[\mathrm{mm}]$;

f_{cd} – design value of concrete compressive strength, $[\mathrm{MPa}]$.

1.2. Finding height of compression area according to given initial data:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{f_{cd} \cdot b}, [\mathrm{mm}] \tag{8}$$

1.3. Finally, plastic moment capacity of the beam at the support can be found:

$$M_c = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (d - d_{s2}), [\mathrm{kNm}] \tag{9}$$

2. Calculation of plastic moment capacity for a slab

This type of calculation is almost the same with calculation of plastic moment capacity for the beam. But in this case, it is necessary to calculate this value in both directions.

2.1. Moment capacity in x-direction

Initial data is applied analogous the calculation of the beam:

N_{ed}, $[\mathrm{kN}]$;

$R_{s,n} = 500\ \mathrm{MPa}$; $\gamma_s = 1{,}0$;

$R_s = f_{yd} = 500\ \mathrm{MPa}$, using formula (7).

Geometrical characteristics of elements' section:

b – design strip of a slab, $[\mathrm{mm}]$. Calculation of moment capacity is made for design strip of the slab equals to 1000 mm;

h – thickness of a slab, $[\mathrm{mm}]$.

Characteristics of applied reinforcement:

A_s – cross sectional area of ongitudinal reinforcement in x-direction, $[\mathrm{mm}^2]$;

d_{s1} – effective depth of a cross-section, $[\mathrm{mm}]$;

f_{cd} , $[\mathrm{MPa}]$.

Height of compression area is found according to formula (10):

$$\chi = \frac{f_{yd} \cdot A_s}{0{,}8 \cdot b \cdot f_{cd}};\ [\mathrm{mm}] \tag{10}$$

$$y = 0{,}8 \cdot x, [\mathrm{mm}]. \tag{11}$$

Formula (11) is used to find relative height of compression area (when the diagram of stress was simplified – instead of real diagram was applied rectangular).

Plastic moment capacity of the slab in x-direction can be found by formula (12):

$$m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y), [\mathrm{kNm}] \tag{12}$$

In addition, in this part the plastic capacity of the axial force of the slab should be found (13):

$$n_{px} = f_{yd} \cdot A_{s1}, [\mathrm{kN}] \tag{13}$$

2.2. Moment capacity in y-direction

The initial data is accepted the same as in the previous case:

$N_{ed} -$ longitudinal force from stressing, $[\mathrm{kN}]$;

$R_{s,n} = f_{yc} = 500\ \mathrm{MPa}$;

$\gamma_s = 1{,}0$; $R_{s,} = f_{yd} = 500\ \mathrm{MPa}$.

Geometrical characteristics of elements' section:

b – design strip of a slab, $[mm]$. Calculation of moment capacity is made for design strip of the slab equals to 1000 mm;

h – thickness of a slab, $[mm]$.

Characteristics of applied reinforcement:

A_s – cross sectional area of ongitudinal reinforcement in y-direction, $[mm^2]$;

d_{s1} – effective depth of a cross-section, $[mm]$;

f_{cd}, $[MPa]$.

Height of compression area can be calculated by the following formula:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}}, [mm]; \quad (14)$$

$$\chi \leq \xi_{c2} \cdot d_2; \quad (15)$$

If this condition (15) is not performed, then the height of compression area should be recalculated with the following caracteristics:

$$\chi = \xi \cdot d_1, [mm]; \quad (16)$$

$$\xi = 2 \cdot \lambda_1; \quad (17)$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}); \quad (18)$$

$$\alpha_n = \frac{N_{ed}}{f_{cd} \cdot b_w \cdot d_{s1}}; \quad (19)$$

$$\alpha_{s1} = \frac{f_{yd} \cdot \rho_1}{f_{cd}}; \quad (20)$$

$$\rho_1 = \frac{A_s}{A_c}; \quad (21)$$

$y = 0{,}8 \cdot \chi, [mm]$ (11).

After that plastic moment capacity of the slab in y-direction can be found by formula (22):

$$m_{py} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y); \quad (22)$$

According to received values internal forces in structures can be calculated:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py}, \left[\frac{kN}{m^2}\right]. \quad (23)$$

If condition (1) will be performed, then the structure can resist to progressive collapse in case of removal of one load-bearing element. Then calculation can be finished.

If condition (1) is not performed then the case of inclusion in the work intermediate columns should be considered. Here the following sequence of calculation is accepted:

1. in calculation for beams in addition to finding value of ordinary moment capacity (when upper reinforcement is compressioned, formula 8), it is needed to calculate moment capacity for case when compressed reinforcement is lower.

Here height of cmpression area is found by formula (25):

$$\chi = \frac{f_{yd} \cdot A_{s2} + N_{ed}}{f_{cd} \cdot b}, [\mathrm{mm}] \tag{24}$$

All components have the same values as for the first case but here the cross sectional area of lower reenforcement was used – A_{s2}, $[\mathrm{mm}^2]$.

After that the moment capacity in other direction with using new height of compression area (25):

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s1} \cdot (d - d_{s2}), [\mathrm{kNm}] \tag{25}$$

2. sequence of calculation for slab remains the same.

1. formula to find internal force becomes another kind:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+}{b} + \frac{2 \cdot M_c^-}{b}\right) + \frac{12}{b^2} \cdot m_{py}, \left[\frac{\mathrm{kN}}{\mathrm{m}^2}\right], \tag{26}$$

If condition (1) is still not performed then the following options are possible:

1. Change of cross sectional of reinforcement in the slab or in the beam

For this case the amount of reinforcement is recalculated. But the sequence of calculation can be applied without changes.

2. change cross-sections of slabs and beams (thickness and heght).

The calculation is made on the initial scheme but with changed dimension of elements.

3. consideretion of post-tensioned reinforcement

Initial method of calculation with some chages is accepted.

3.1 Calculation of a beam

All characteristics should be found according initial method.

Permissible plastic longitudinal force in the beam additionally has to be found (28):

$$\mathrm{N_c = (A_{s1} + A_{s2}) \cdot R_s + A_p \cdot R_p, [MN]} \tag{27}$$

A_{s1}, A_{s2} – cross sectional area of upper and lower reinforcement, $[\mathrm{mm^2}]$;

$\mathrm{R_s}$ – design yield strength of reinforcement, $[\mathrm{MPa}]$

$\mathrm{A_p}$ – area of a prestressing tendons;

$\mathrm{R_p}$ – design yield strength of tendons, $[\mathrm{MPa}]$.

3.2 Calculation of a slab is applied without changes.

3.3 Internal force in this case can be found in the following way (26)

$$\mathrm{S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py} + \frac{12 \cdot w_c}{a^2} \cdot \left(\frac{n_{px}}{2} + \frac{N_c}{b}\right), \left[\frac{kN}{m^2}\right]} \tag{28}$$

w_c – deflection, $[m]$.

For this case the table which shows permissible value of external load as the function of deformation in place of removed column.

5 CALCULATION

5.1 General information about the building

The given building is a shopping center with the height of about 35 m which is situated in Saint-Petersburg. The building consists of 6 floors, where 2 of them are underground and the rest 4 floors are above the ground level. Underground floors serve as two-level parking. The shops, cinema and restaurants occupy over ground floors.

In the building the following levels were accepted:

6 floor: +31.000;

5 floor: +23.400;

4 floor: +18.000;

3 floor: +12.000;

2 floor: +7.200;

1 floor: 0.000;

-1 floor: -3.600;

-2 floor: -7.200.

Where the absolute level 0.000 corresponds to the relative level +7.500.

The total area of the center is about 140720 m^2 and the total building volume is 993780 m^3. The applied service life of the building is 50 years and the level of responsibility is normal.

5.2 Structures

Piles with diameter 520/670 mm combined by base plate were used for foundations. The thickness of the base plate is 800 mm.

Columns are accepted with spacing 16,8 m and 8,4 m. Columns have rectangular cross-section with the side of 600-900 mm and a circular cross-section diameter of 600-1000 mm. The material used for columns is concrete B40 and reinforcement A500C with diameter 28 mm and 32 mm.

Floor plate structures are monolithic multi-span reinforced concrete beams, the cross-section height is 600 mm and the width is 1200 mm, combined with plate 200 mm. The beams are stacked in the direction of span 16,8 m in steps of 8,4 m. Beams and slabs are made of post-tension concrete B45 with tension cables.

Walls of staircases and elevators shafts are made of reinforced concrete, the thickness is 250 mm.

Landings are made of reinforced concrete and stairs are made of precast concrete.

The internal walls inside of the building made of the steel sections and plasterboard, brick or lightweight concrete.

The roof structure consists of steel trusses, bolt-on and covered with corrugated steel sheet. The maximum spans are 32,4 m and 24,3 m.

The facade of the building is a complex space-spatial system. Wall of aerated concrete blocks with mineral wool insulation is installed on concrete floors. Metal supports for the light facade elements (cement composite panels) are fixed to the supporting reinforced concrete columns and beams of the building.

The over part of the building is divided by expansion joints into 10 blocks. Their dimensions are 67x60 (2), 55x40 (3), 63x60, 73x40, 63x45, 53x63 (2).

5.3 Process of calculation

The stability of structure’s internal forces was checked. These efforts appear in case of removal load-bearing vertical element. In result we will receive external loads, permissible load-bearing capacity of structures, deformation and dependence between bearing capacity and deflection when removing a column.

Calculations were made for block 3 at level +0.000. The removed column is situated at intersection of axis 11/C. This column is in-situ concrete element (B40) with square section 600x600 mm. (Figure 19).

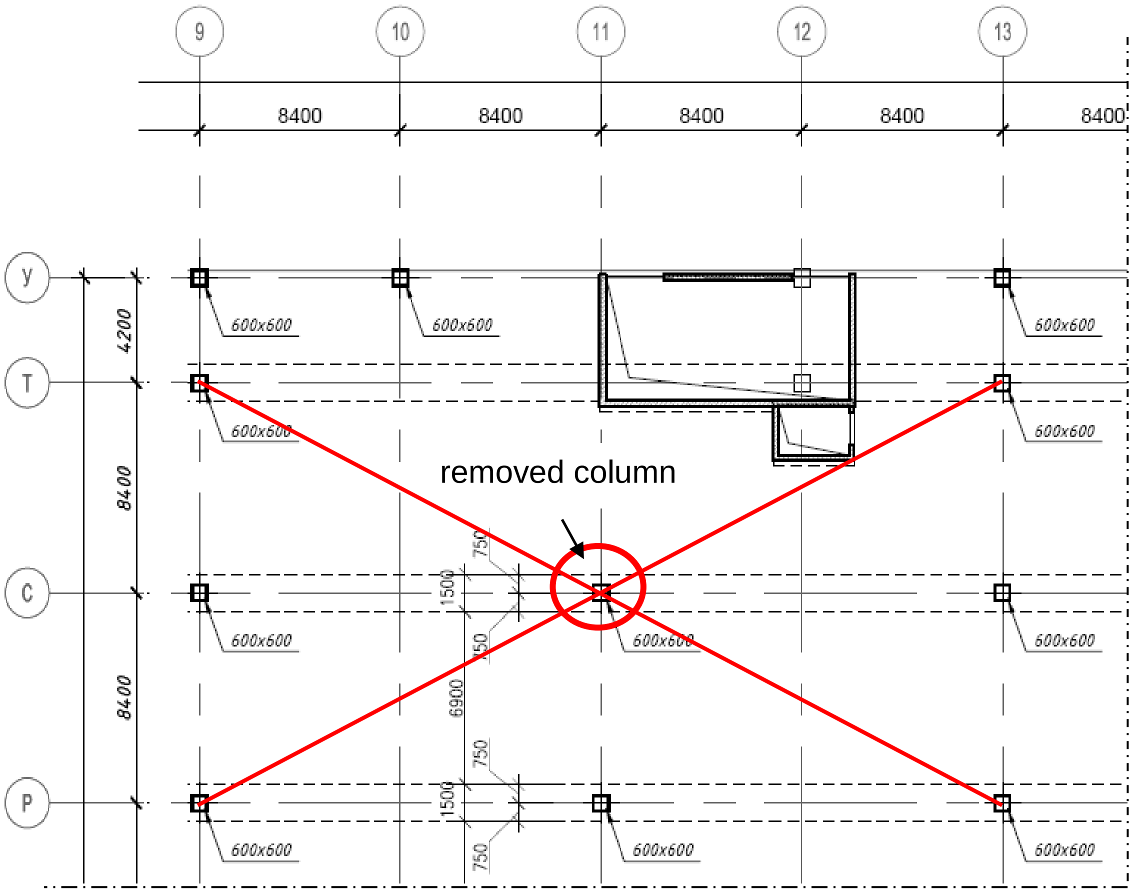

Figure 19. Scheme of notionally removed column

5.3.1 Attempt 1

1. External loads

Dead load (self-weights of slab and beam) according to formula (5):

$$g = t_{slab} \cdot \rho_c + b_{beam} \cdot \frac{h_{beam}}{l_{beam}} \cdot \rho_c;$$

$$g = 0{,}2 \cdot 25 + 0{,}6 \cdot \frac{1{,}2}{8{,}4} \cdot 25 = 7{,}14\ \frac{kN}{m^2}.$$

Live load:

$$q = 4 \frac{kN}{m^2}.$$

Total load with load factor equals to 1,0 (6):

$$F = 7{,}14 + 4 = 11{,}14 \frac{kN}{m^2}.$$

2. Permissible value of bearing capacity of structure

To ensure the work of the building in case of load-bearing element's failure under abnormal load, it is needed to perform the condition (1).

Several attempts were made to ensure this condition. For each attempt some characteristics were changed (for example amount of reinforcement or dimensions of cross-section).

2.1. Calculation of the beam

The material of the beam is concrete (B45) with reinforcement A500 and tendons – Y1860. The beam has the following dimensions: height – 600 mm, width – 1200 mm and length – 16800 mm (figure 20).

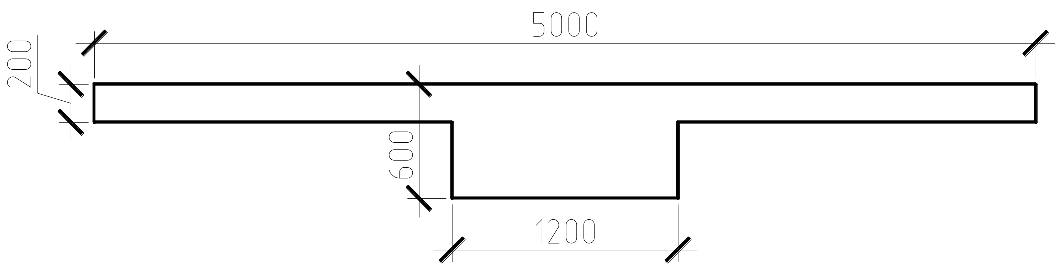

Figure 20. Section of the beam and slab

Initial data was found from calculation for ordinary conditions of operation. In such type of calculation safety factors γ, coefficients of combination ψ, materials factors are used according to existing Russian standards and norms. While for calculation against progressive collapse it is necessary to recalculate elements of structures by using pointed coefficients equal to 1,0.

Then the following results were received:

$N_{sd} = 1794$ kN;

N_{ed} was taken with reserve 3% due to post-tension tendons:

$N_{ed} = 1848 \text{ kN};$

$R_{s,n} = f_{yc} = 500 \text{ MPa};$

$\gamma_s = 1{,}0,\ f_{yd} = 500 \text{ MPa};$ (7);

$b = 1200 \text{ mm};$

$h = 600 \text{ mm}.$

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (figure 21):

$A_{s1} = 6432 \text{ mm}^2;$

$d_{s1} = 550 \text{ mm};$

$f_{cd} = 25 \text{ MPa}.$

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (figure 21):

$A_{s2} = 2412 + 1473 = 3885 \text{ mm}^2;$

$d_{s2} = 50 \text{ mm};$

$d = 550 \text{ mm}.$

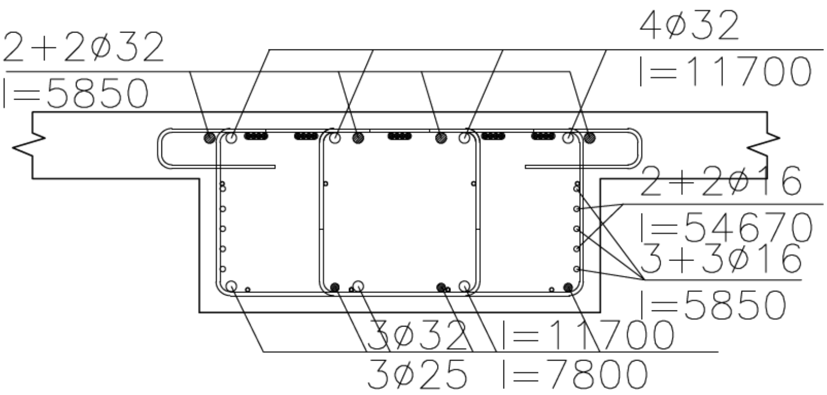

Figure 21. Applied reinforcement in the beam

Height of compression area (8):

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 6432 + 1848000}{25 \cdot 1200} = 169 \text{ mm}.$$

Plastic moment capacity of the beam (9):

$\mathrm{M_c} = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$

$M_c = 25 \cdot 1200 \cdot 169 \cdot (550 - 0{,}5 \cdot 169) \cdot 10^{-6} + 500 \cdot 3885 \cdot (550 - 50) \cdot 10^{-6};$

$M_c = (2360 + 971) = 3331\ \mathrm{kNm}.$

2. Calculation of the slab

The material of the slab is concrete (B45) with reinforcement A500 and tendons – Y1860. The slab was designed from element to element with thickness 200 mm (figure 20). Conditions about initial data for calculations against progressive collapse for slabs are the same with conditions for beams.

2.1 Plastic moment capacity in x-direction

$\mathrm{N_{sd}} = 300\ \mathrm{kN};$

$\mathrm{N_{ed}}$ was taken with reserve 3% due to post-tension tendons:

$\mathrm{N_{ed}} = 310\ \mathrm{kN};$

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$

$\gamma_\mathrm{s} = 1{,}0;\ \mathrm{f_{yd}} = 500\ \mathrm{MPa}.$ (7)

Longitudinal reinforcement for a slab in x-direction was applied as Ø8 s 200 (figure 22).

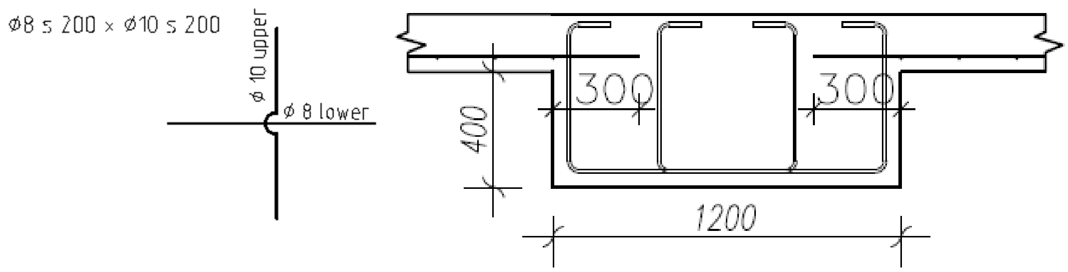

Figure 22. Applied reinforcement in the slab

Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$\mathrm{A_s} = \mathrm{A_{s\emptyset 8}} \cdot \mathrm{n_{bars}} = 252\ \mathrm{mm^2};$

$d_{s1} = 150\ \text{mm};$

$f_{cd} = 25\ \text{MPa};$

$b = 1000\ \text{mm};$

$h = 200\ \text{mm}.$

Height of compression area (10):

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3\ \text{mm};$$

$y = 0{,}8 \cdot \chi = 0{,}8 \cdot 6{,}3 = 5{,}04\ \text{mm}$ (11).

Plastic moment capacity of the slab in x-direction (12):

$$m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);$$

$$m_{px} = \left(25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)\right) \cdot 10^{-6} = 18{,}6\ \text{kN}.$$

Plastic capacity of the axial force of the slab in x-direction (13):

$$n_{px} = f_{yd} \cdot A_{s1} = 500 \cdot 252 \cdot 10^{-3} = 126\ \text{kN}.$$

2.2 Plastic moment capacity in y-direction

$N_{sd} = 300\ \text{kN};$

N_{ed} was taken with reserve 3% due to post-tension tendons:

$N_{ed} = 310\ \text{kN}$ (longitudinal force from stressing);

$R_{s,n} = f_{yc} = 500\ \text{MPa};$

$\gamma_s = 1{,}0;\ f_{yd} = 500\ \text{MPa}$ (7).

Longitudinal reinforcement for a slab in y-direction was applied as Ø10 s 200 (figure 22). Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{s\emptyset 8} \cdot n_{bars} = 393\ \text{mm}^2;$

$d_{s1} = 150\ \text{mm};$

$f_{cd} = 25\ \text{MPa};$

$b = 1000\ \text{mm};$

$\mathrm{h} = 200\ \mathrm{mm}.$

Height of compression area (14):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{0{,}8 \cdot \mathrm{b} \cdot \mathrm{f_{cd}}};$$

$$\chi = \frac{500 \cdot 393 + 310000}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3\ \mathrm{mm};$$

$$\chi \leq \xi_{c2} \cdot \mathrm{d_2};$$

$$\chi = \xi \cdot \mathrm{d_1} = 0{,}1534 \cdot 150 = 23\ \mathrm{mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}077 = 0{,}1534;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}0827 + 0{,}04) = 0{,}077;$$

$$\alpha_n = \frac{\mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b_w} \cdot \mathrm{d_{s1}}} = \frac{310000}{25 \cdot 1000 \cdot 150} = 0{,}0827;$$

$$\alpha_{s1} = \frac{\mathrm{f_{yd}} \cdot \rho_1}{\mathrm{f_{cd}}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{\mathrm{A_s}}{\mathrm{A_c}} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 23 = 18{,}4\ \mathrm{mm}.$ (11)

Plastic moment capacity of slab in y-direction (22):

$$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{py}} = \big(25 \cdot 1000 \cdot 18{,}4 \cdot (150 - 0{,}5 \cdot 18{,}4)\big) \cdot 10^{-6} = 64{,}8\ \mathrm{kNm}.$$

Then it is possible to calculate capacity of structures (23):

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(18{,}6 + \frac{3331}{16{,}8}\right) + \frac{12}{(16{,}8)^2} \cdot 64{,}8 = 5{,}1\ \frac{kN}{m^2};$$

$$S = 5{,}1\ \frac{kN}{m^2} < F = 11{,}14\ \frac{kN}{m^2}.$$

For the first attempt necessary condition is not performed. So other options were offered.

5.3.2 Attempt 2

In this case the span is changed namely the additional four supports are included in the work.

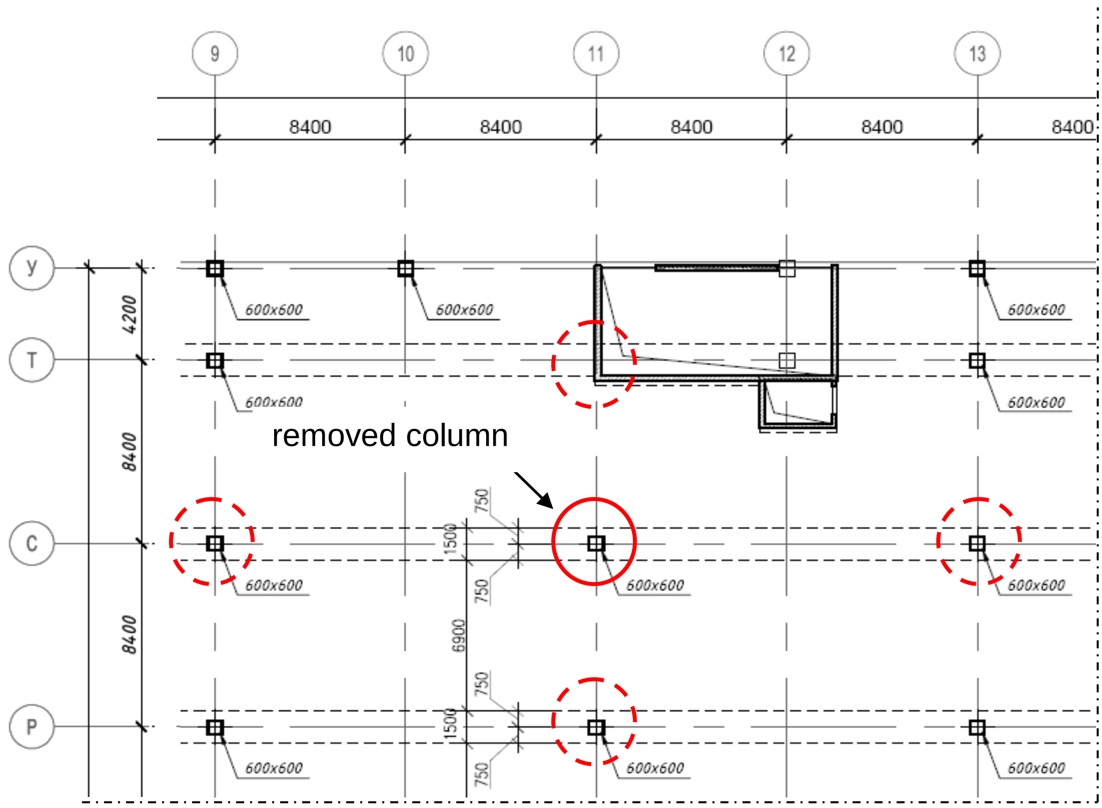

Figure 23. Model of removed column with changed span

1. Calculation of the beam

$N_{sd} = 1794$ kN;

$N_{ed} = 1848$ kN;

$R_{s,n} = f_{yc} = 500$ MPa;

$\gamma_s = 1{,}0$; $f_{yd} = 500$ MPa (7);

$b = 1200$ mm;

$h = 600$ mm.

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (figure 21):

$A_{s1} = 6432$ mm^2;

$d_{s1} = 550$ mm;

$f_{cd} = 25$ MPa.

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (figure 21):

$\mathrm{A_{s2}} = 2412 + 1473 = 3885\ \mathrm{mm^2}$;

$\mathrm{d_{s2}} = 50\ \mathrm{mm}$;

$\mathrm{f_{yd}} = 400\ \mathrm{MPa}$;

$\mathrm{d} = 550\ \mathrm{mm}$.

Height of compression area (when the upper reinforcement is compressioned) (8):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 6432 + 1848000}{25 \cdot 1200} = 169\ \mathrm{mm}.$$

Plastic moment capacity of the beam (9):

$$M_c^+ = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^+ = 25 \cdot 1200 \cdot 169 \cdot (550 - 0{,}5 \cdot 169) \cdot 10^{-6} + 500 \cdot 3885 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c^+ = (2360 + 971) = 3331\ \mathrm{kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned) (24):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s2}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 3885 + 1848000}{25 \cdot 1200} = 126{,}35\ \mathrm{mm}.$$

Plastic moment capacity of the beam (25):

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s1} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^- = 25 \cdot 1200 \cdot 126{,}35 \cdot (550 - 0{,}5 \cdot 126{,}35) \cdot 10^{-6} + 500 \cdot 6432 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c^- = (1845 + 1608) = 3453\ \mathrm{kNm}.$$

2. Calculation of the slab

2.1 Plastic moment capacity in x-direction

$\mathrm{N_{sd}} = 300\ \mathrm{kN};$

$\mathrm{N_{ed}} = 310\ \mathrm{kN};$

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$

$\gamma_\mathrm{s} = 1{,}0;\ \mathrm{f_{yd}} = 500\ \mathrm{MPa}$ (7).

Longitudinal reinforcement for a slab in x-direction was applied as Ø8 s 200 (figure 22). Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$\mathrm{A_s} = \mathrm{A_{s\emptyset 8}} \cdot \mathrm{n_{bars}} = 252\ \mathrm{mm}^2;$

$\mathrm{d_{s1}} = 150\ \mathrm{mm};$

$\mathrm{f_{cd}} = 25\ \mathrm{MPa};$

$\mathrm{b} = 1000\ \mathrm{mm};$

$\mathrm{h} = 200\ \mathrm{mm}.$

Height of compression area (10):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}}}{0{,}8 \cdot \mathrm{b} \cdot \mathrm{f_{cd}}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3\ \mathrm{mm};$$

$\mathrm{y} = 0{,}8 \cdot \chi = 0{,}8 \cdot 6{,}3 = 5{,}04\ \mathrm{mm}.$ (11)

Plastic moment capacity of the slab in x-direction (12):

$$\mathrm{m_{px}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{px}} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6} = 18{,}6\ \mathrm{kNm}.$$

Plastic capacity of the axial force of the slab in x-direction (13):

$$\mathrm{n_{px}} = \mathrm{f_{yd}} \cdot \mathrm{A_{s1}} = 500 \cdot 252 \cdot 10^{-3} = 126\ \mathrm{kN}.$$

2.2 Plastic moment capacity of the slab in y-direction

$\mathrm{N_{sd}} = 300\ \mathrm{kN};$

$\mathrm{N_{ed}} = 310\ \mathrm{kN}$ (longitudinal force from stressing);

$R_{s,n} = f_{yc} = 500 \text{ MPa};$

$\gamma_s = 1{,}0;\ f_{yd} = 500 \text{ MPa}.$ (7)

Longitudinal reinforcement for a slab in y-direction was applied as Ø10 s 200 (figure 22). Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{sØ8} \cdot n_{bars} = 393 \text{ mm}^2;$

$d_{s1} = 150 \text{ mm};$

$f_{cd} = 25 \text{ MPa};$

$b = 1000 \text{ mm};$

$h = 200 \text{ mm};$

Height of compression area (14):

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 393 + 310000}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3 \text{ mm};$$

$$\chi \leq \xi_{c2} \cdot d_2;$$

$$\chi = \xi \cdot d_1 = 0{,}1534 \cdot 150 = 23 \text{ mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}077 = 0{,}1534;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}0827 + 0{,}04) = 0{,}077;$$

$$\alpha_n = \frac{N_{ed}}{f_{cd} \cdot b_w \cdot d_{s1}} = \frac{310000}{25 \cdot 1000 \cdot 150} = 0{,}0827;$$

$$\alpha_{s1} = \frac{f_{yd} \cdot \rho_1}{f_{cd}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{A_s}{A_c} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$y = 0{,}8 \cdot x = 0{,}8 \cdot 23 = 18{,}4 \text{ mm}.$ (11)

Plastic moment capacity of the slab in y-direction (22):

$$m_{py} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);$$

$$m_{py} = \left(25 \cdot 1000 \cdot 18{,}4 \cdot (150 - 0{,}5 \cdot 18{,}4)\right) \cdot 10^{-6} = 64{,}8 \text{ kNm}.$$

Capacity of structure in this case is calculated using (26):

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+}{b} + \frac{2 \cdot M_c^-}{b} \right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(18{,}6 + \frac{3331}{16{,}8} + \frac{2 \cdot 3453}{16{,}8} \right) + \frac{12}{(16{,}8)^2} \cdot 64{,}8 = 9{,}4 \ \frac{kN}{m^2};$$

$$S = 9{,}4 \ \frac{kN}{m^2} < F = 11{,}14 \ \frac{kN}{m^2}.$$

Consideration of the supports did not help – condition is still not fulfilled.

5.3.3 Attempt 3

For this case the work of four supports and changed cross-section of reinforcement in the slab were taken into account.

1. Calculation of the beam

$\mathrm{N_{sd}} = 1794 \ \mathrm{kN};$

$\mathrm{N_{ed}} = 1848 \ \mathrm{kN};$

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500 \ \mathrm{MPa};$

$\gamma_\mathrm{s} = 1{,}0; \ \mathrm{f_{yd}} = 500 \ \mathrm{MPa};$ (7)

$\mathrm{b} = 1200 \ \mathrm{mm};$

$\mathrm{h} = 600 \ \mathrm{mm}.$

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (figure 21):

$\mathrm{A_{s1}} = 6432 \ \mathrm{mm^2};$

$\mathrm{d_{s1}} = 550 \ \mathrm{mm};$

$\mathrm{f_{cd}} = 25 \ \mathrm{MPa}.$

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (figure 21):

$\mathrm{A_{s2}} = 2412 + 1473 = 3885 \ \mathrm{mm^2};$

$\mathrm{d_{s2}} = 50 \ \mathrm{mm};$

$\mathrm{d} = 550 \ \mathrm{mm}.$

Height of compression area (when the upper reinforcement is compressioned) (8):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 6432 + 1848000}{25 \cdot 1200} = 169 \text{ mm}.$$

Moment capacity of the beam:

$$M_c^+ = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^+ = 25 \cdot 1200 \cdot 169 \cdot (550 - 0{,}5 \cdot 169) \cdot 10^{-6} + 500 \cdot 3885 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c^+ = (2360 + 971) = 3331 \text{ kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned) (24):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s2}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 3885 + 1848000}{25 \cdot 1200} = 126{,}35 \text{ mm}.$$

Moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s1} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^- = 25 \cdot 1200 \cdot 126{,}35 \cdot (550 - 0{,}5 \cdot 126{,}35) \cdot 10^{-6} + 500 \cdot 6432 \cdot (550 - 50) \cdot 10^{-6};$$

$$(\mathrm{Ne})_{\mathrm{Rd}} = M_c^- = (1845 + 1608) = 3453 \text{ kNm}.$$

2. Calculation of the slab

2.1 Plastic moment capacity in x-direction

$\mathrm{N_{sd}} = 300$ kN;

$\mathrm{N_{ed}} = 310$ kN;

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500$ MPa;

$\gamma_s = 1{,}0; \mathrm{f_{yd}} = 500$ MPa. (7)

Longitudinal reinforcement for a slab in x-direction was applied as Ø12 s 200.

Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{s\emptyset 8} \cdot n_{bars} = 565\ mm^2;$

$d_{s1} = 150\ mm;$

$f_{cd} = 25\ MPa;$

$b = 1000\ mm;$

$h = 200\ mm.$

Height of compression area (10):

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 565}{0{,}8 \cdot 1000 \cdot 25} = 14{,}125\ mm;$$

$y = 0{,}8 \cdot x = 0{,}8 \cdot 14{,}125 = 11{,}3\ mm.$ (11)

Plastic moment capacity of the slab in x-direction (12):

$$m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);$$

$$m_{px} = \left(25 \cdot 1000 \cdot 11{,}3 \cdot (150 - 0{,}5 \cdot 11{,}3)\right) \cdot 10^{-6} = 40{,}78\ kNm.$$

The plastic capacity of the axial force of the slab (13):

$$n_{px} = f_{yd} \cdot A_{s1} = 500 \cdot 565 \cdot 10^{-3} = 282{,}5\ kN.$$

2.2 Plastic moment capacity of the slab in y-direction

$N_{sd} = 300\ kN;$

$N_{ed} = 310\ kN$ (longitudinal force from stressing);

$R_{s,n} = f_{yc} = 500\ MPa;$

$\gamma_s = 1{,}0;\ f_{yd} = 500\ MPa;$ (7)

Longitudinal reinforcement for a slab in y-direction was applied as Ø16 s 200. Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{s\emptyset 8} \cdot n_{bars} = 1005\ mm^2;$

$\mathrm{d_{s1}} = 150\ \mathrm{mm};$

$\mathrm{f_{cd}} = 25\ \mathrm{MPa};$

$\mathrm{b} = 1000\ \mathrm{mm};$

$\mathrm{h} = 200\ \mathrm{mm}.$

Height of compression area (8):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{0{,}8 \cdot \mathrm{b} \cdot \mathrm{f_{cd}}};$$

$$\chi = \frac{500 \cdot 1005 + 310000}{0{,}8 \cdot 1000 \cdot 25} = 40{,}6\ \mathrm{mm};$$

$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 40{,}6 = 32{,}5\ \mathrm{mm}.$ (11)

Plastic moment capacity of the slab in y-direction:

$$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{py}} = (25 \cdot 1000 \cdot 32{,}5 \cdot (150 - 0{,}5 \cdot 32{,}5)) \cdot 10^{-6} = 108{,}7\ \mathrm{kNm}.$$

Capacity of structure (26):

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+ + 2 \cdot M_c^-}{b}\right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(40{,}78 + \frac{3331 + 2 \cdot 3453}{16{,}8}\right) + \frac{12}{(16{,}8)^2} \cdot 108{,}7 = 11{,}51\ \frac{kN}{m^2};$$

$$S = 11{,}51\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

This attempt gave good result but it is required to increase the cross sectional area of reinforcement in slab almost in 1,5 times that can lead to excessive reinforcement.

5.3.4 Attempt 4

Another attempt of the calculation was made with changed cross-sections of slabs and beams (thickness and heght) and considered the work of four supports.

1. Calculation of the beam

The height of the beam was changed – instead of 600 mm, 800 mm was applied (figure 20).

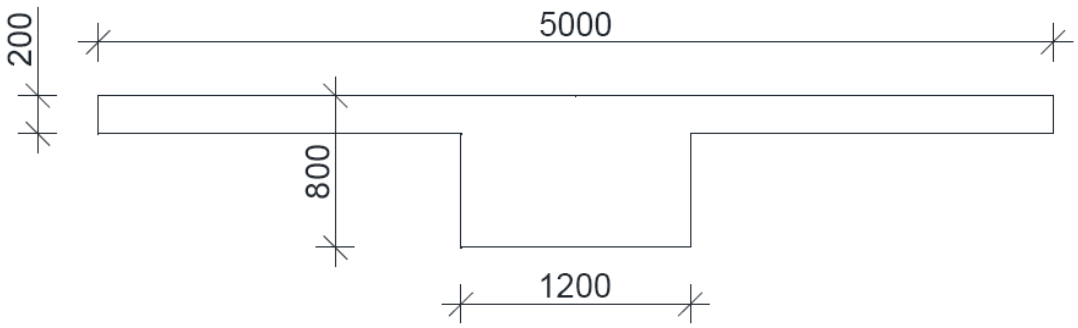

Figure 24. Changed section of beam

$N_{sd} = 1794 \text{ kN};$

$N_{ed} = 1848 \text{ kN};$

$R_{s,n} = f_{yc} = 500 \text{ MPa};$

$\gamma_s = 1{,}0; f_{yd} = 500 \text{ MPa};$ (7)

$b = 1200 \text{ mm};$

$h = 800 \text{ mm}.$

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (figure 21):

$A_{s1} = 6432 \text{ mm}^2;$

$d_{s1} = 750 \text{ mm};$

$f_{cd} = 25 \text{ MPa};$

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (figure 21):

$A_{s2} = 2412 + 1473 = 3885 \text{ mm}^2;$

$d_{s2} = 50 \text{ mm};$

$f_{ycd} = 400 \text{ MPa};$

$d = 750 \text{ mm}.$

Height of compression area (when the upper reinforcement is compressioned) (8):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 6432 + 184800}{25 \cdot 1200} = 169\ \mathrm{mm};$$

Plastic moment capacity of the beam:

$$M_c^+ = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^+ = 25 \cdot 1200 \cdot 169 \cdot (750 - 0{,}5 \cdot 169) \cdot 10^{-6} + 500 \cdot 3885 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^+ = (3374{,}085 + 1359{,}75) = 4734\mathrm{kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned) (24):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s2}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 3885 + 1848000}{25 \cdot 1200} = 126{,}35\ \mathrm{mm};$$

Plastic moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s1} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^- = 25 \cdot 1200 \cdot 126{,}35 \cdot (750 - 0{,}5 \cdot 126{,}35) \cdot 10^{-6} + 500 \cdot 6432 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^- = (2603 + 2251) = 4855\ \mathrm{kNm}.$$

2. Calculation of a slab

2.1 Plastic moment capacity in x-direction

$\mathrm{N_{sd}} = 300\ \mathrm{kN};$

$\mathrm{N_{ed}} = 310\ \mathrm{kN};$

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$

$\gamma_\mathrm{s} = 1{,}0;\ \mathrm{f_{yd}} = 500\ \mathrm{MPa}$ (7).

Longitudinal reinforcement for a slab in x-direction was applied as Ø8 s 200 (figure 22).

Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$\mathrm{A_s = A_{s\emptyset 8} \cdot n_{bars} = 252\ mm^2;}$

$\mathrm{d_{s1} = 150\ mm;}$

$\mathrm{f_{cd} = 25\ MPa;}$

$\mathrm{b = 1000\ mm;}$

$\mathrm{h = 200\ mm.}$

Height of compression area (10):

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3\ \mathrm{mm};$$

$\mathrm{y = 0{,}8 \cdot x = 0{,}8 \cdot 6{,}3 = 5{,}04\ mm.}$ (11)

Plastic moment capacity of the slab in x-direction (12):

$\mathrm{m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);}$

$\mathrm{m_{px} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6} = 18{,}6\ kNm.}$

Plastic capacity of the axial force of the slab in x-direction (13):

$\mathrm{n_{px} = f_{yd} \cdot A_{s1} = 500 \cdot 252 \cdot 10^{-3} = 126\ kN.}$

2.2 Plastic moment capacity in y-direction

$\mathrm{N_{sd} = 300\ kN;}$

$\mathrm{N_{ed} = 310\ kN\ (longitudinal\ force\ from\ stressing);}$

$\mathrm{R_{s,n} = f_{yc} = 500\ MPa;}$

$\mathrm{\gamma_s = 1{,}0;\ f_{yd} = 500\ MPa.}$ (7)

Longitudinal reinforcement for a slab in y-direction was applied as Ø10 s 200 (figure 22). Calculation was made for strip of width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$\mathrm{A_s = A_{s\emptyset 8} \cdot n_{bars} = 393\ mm^2;}$

$\mathrm{d_{s1}} = 150\ \mathrm{mm};$

$\mathrm{f_{cd}} = 25\ \mathrm{MPa};$

$\mathrm{b} = 1000\ \mathrm{mm};$

$\mathrm{h} = 200\ \mathrm{mm};$

Height of compression area (8):

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 393 + 310000}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3\ \mathrm{mm};$$

$$\chi \leq \xi_{c2} \cdot \mathrm{d_2};$$

$$\chi = \xi \cdot \mathrm{d_1} = 0{,}1534 \cdot 150 = 23\ \mathrm{mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}077 = 0{,}1534;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}0827 + 0{,}04) = 0{,}077;$$

$$\alpha_n = \frac{\mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b_w} \cdot \mathrm{d_{s1}}} = \frac{310000}{25 \cdot 1000 \cdot 150} = 0{,}0827;$$

$$\alpha_{s1} = \frac{\mathrm{f_{yd}} \cdot \rho_1}{\mathrm{f_{cd}}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{\mathrm{A_s}}{\mathrm{A_c}} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 23 = 18{,}4\ \mathrm{mm}.$ (11)

Plastic moment capacity of the slab in y-direction (22):

$$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{py}} = \big(25 \cdot 1000 \cdot 18{,}4 \cdot (150 - 0{,}5 \cdot 18{,}4)\big) \cdot 10^{-6} = 64{,}8\ \mathrm{kNm}.$$

Capacity of the structure (26):

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+}{b} + \frac{2 \cdot M_c^-}{b}\right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(25{,}0 + \frac{4734 + 2 \cdot 4855}{16{,}8}\right) + \frac{12}{(16{,}8)^2} \cdot 89{,}6 = 12{,}06;$$

$$S = 12{,}06\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

This attempt was successful. So the first finding can be made: the best solution to provide necessary condition is increasing of elements' cross-section or cross sectional area of reinforcement.

To provide more reserve, it is possible to combine both solutions – increase the dimension of elements and cross sectional area of reinforcement.

5.3.5 Attempt 5

The important thing which should be considered in the calculation is using the post-tensioned cables that take a significant part of the load. It is known that post-tensioned reinforcement at ultimate limit state may show greater strength compared to ordinary reinforcement. In this case it is needed to find out the dependence of the deflection on the applied load – find allowed deflection which can be accepted to provide condition.

For this attempt of calculation the initial cross-section of elements and amount of reinforcement were applied. Cases without and with consideration of additional columns were calculated.

1. Calculation of the beam

The material of the beam is concrete (B45) with reinforcement A500 and tendons – Y1860. The beam has the following dimensions: height – 600 mm, width – 1200 mm and length – 16800 mm (figure 20).

$\mathrm{N_{sd}} = 1794\ \mathrm{kN}$;

$\mathrm{N_{ed}} = 1848\ \mathrm{kN}$;

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa}$;

$\gamma_\mathrm{s} = 1{,}0,\ \mathrm{f_{yd}} = 500\ \mathrm{MPa}$; (7)

$\mathrm{b} = 1200\ \mathrm{mm}$;

$\mathrm{h} = 600\ \mathrm{mm}$.

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (figure 21):

$\mathrm{A_{s1}} = 6432\ \mathrm{mm^2}$;

$\mathrm{d_{s1}} = 550\ \mathrm{mm}$;

$f_{cd} = 25$ MPa.

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (figure 21):

$A_{s2} = 2412 + 1473 = 3885\ \mathrm{mm}^2$;

$d_{s2} = 50$ mm;

$f_{ycd} = 400$ MPa;

$d = 550$ mm.

Height of compression area (8):

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 6432 + 1848000}{25 \cdot 1200} = 169\ \mathrm{mm}.$$

Plastic moment capacity of the beam (9):

$$M_c = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (d - d_{s2});$$

$$M_c = 25 \cdot 1200 \cdot 169 \cdot (550 - 0{,}5 \cdot 169) \cdot 10^{-6} + 500 \cdot 3885 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c = (2360 + 971) = 3331\ \mathrm{kNm}.$$

In this case it is needed to consider the plastic capacity for the axial force for the beam (27):

$$N_c = (A_{s1} + A_{s2}) \cdot R_s + A_p \cdot R_p;$$

$$N_c = (6432 + 3885) \cdot 500 + 5 \cdot 4 \cdot 150 \cdot 1860 = 10{,}7\ MN.$$

2. Calculation of the slab

The material of the slab is concrete (B45) with reinforcement A500 and tendons – Y1860. The slab was designed from element to element with thickness 200 mm (figure 20).

2.1 Plastic moment capacity in x-direction

$N_{sd} = 300$ kN;

$N_{ed} = 310$ kN;

$R_{s,n} = f_{yc} = 500$ MPa;

$\gamma_s = 1{,}0; f_{yd} = 500$ MPa. (7)

Longitudinal reinforcement for a slab in x-direction was applied as Ø8 s 200 (figure 22). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{s\emptyset 8} \cdot n_{bars} = 252\ mm^2;$

$d_{s1} = 150\ mm;$

$f_{cd} = 25\ MPa;$

$b = 1000\ mm;$

$h = 200\ mm.$

Height of compression area (10):

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3\ mm;$$

$y = 0{,}8 \cdot x = 0{,}8 \cdot 6{,}3 = 5{,}04\ mm.$ (11)

Plastic moment capacity of the slab in x-direction (12):

$$m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);$$

$$m_{px} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6} = 18{,}6\ kNm.$$

Plastic capacity of the axial force of the slab in x-direction (13):

$$n_{px} = f_{yd} \cdot A_{s1} = 500 \cdot 252 \cdot 10^{-3} = 126\ kN.$$

2.2 Plastic moment capacity in y-direction

$N_{sd} = 300\ kN;$

$N_{ed} = 310\ kN;$

$R_{s,n} = f_{yc} = 500\ MPa;$

$\gamma_s = 1{,}0;\ f_{yd} = 500\ MPa.$ (7)

Longitudinal reinforcement for a slab in y-direction was applied as Ø10 s 200 (figure 22). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$\mathrm{A_s} = \mathrm{A_{s\emptyset8}} \cdot \mathrm{n_{bars}} = 393\ \mathrm{mm^2}$;

$\mathrm{d_{s1}} = 150\ \mathrm{mm}$;

$\mathrm{f_{cd}} = 25\ \mathrm{MPa}$;

$\mathrm{b} = 1000\ \mathrm{mm}$;

$\mathrm{h} = 200\ \mathrm{mm}$.

Height of compression area (8):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{0{,}8 \cdot \mathrm{b} \cdot \mathrm{f_{cd}}};$$

$$\chi = \frac{500 \cdot 393 + 310}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3\ \mathrm{mm};$$

$$\chi \leq \xi_{c2} \cdot \mathrm{d_2};$$

$$\chi = \xi \cdot \mathrm{d_1} = 0{,}1534 \cdot 150 = 23\ \mathrm{mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}077 = 0{,}1534;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_\mathrm{n} + \alpha_{\mathrm{s1}}) = 0{,}625 \cdot (0{,}0827 + 0{,}04) = 0{,}077;$$

$$\alpha_\mathrm{n} = \frac{\mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b_w} \cdot \mathrm{d_{s1}}} = \frac{310000}{25 \cdot 1000 \cdot 150} = 0{,}0827;$$

$$\alpha_{\mathrm{s1}} = \frac{\mathrm{f_{yd}} \cdot \rho_1}{\mathrm{f_{cd}}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{\mathrm{A_s}}{\mathrm{A_c}} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$$\mathrm{y} = 0{,}8 \cdot \chi = 0{,}8 \cdot 23 = 18{,}4\ \mathrm{mm}. \ (11)$$

Plastic moment capacity of the slab in y-direction (22):

$$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y})$$

$$\mathrm{m_{py}} = \big(25 \cdot 1000 \cdot 18{,}4 \cdot (150 - 0{,}5 \cdot 18{,}4)\big) \cdot 10^{-6} = 64{,}8\ \mathrm{kNm}.$$

Then it is possible to calculate structure’s capacity and the allowed deflection:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py} + \frac{12 \cdot w_c}{a^2} \cdot \left(\frac{n_{px}}{2} + \frac{N_c}{b}\right).$$

The following table (1) shows the dependence between applied forces and deflection for a case when additional four supports are not considered.

Table 1. Between applied forces and deflection (without consideration of supports)

S(x), m	F, kH/m^2
0	5,06
0,1	5,804
0,5	8,780
0,9	11,756
1,3	14,732

As we can see from the table to ensure the condition of resistance against progressive collapse, we need to apply the allowed deflection (the most optimal) equal to 900 – 1000 mm ($\approx \frac{1}{9} l$). Then the following result was received:

$$S = 11{,}756\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

The second table (2) – for case with consideration of middle columns (span is decreased).

Table 2. Between applied forces and deflection (with consideration of supports)

S(x), m	F, kH/m^2
0	9,43
0,1	10,174
0,2	10,918
0,3	11,662
0,5	13,149

This table shows that the most optimal deflection can be accepted equal to about 450 mm ($\approx \frac{1}{28} l$). Then the following result was received:

$$S = 11{,}662\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

Made calculations show that to ensure the given condition, the following options are possible:

1. increasing of cross-section for such elements as beams and slabs;

2. increasing of reinforcement cross-section;

3. obligatory account of plastic capacity for the beam (in case of using post-tensioned structures.

5.4 Conclusions

The main difference between ordinary calculation and calculation against progressive collapse lies in used coefficients. The basic coefficients for such calculation – coefficient of combinations φ, safety factor γ_n and material factors γ_s should be applied equal to one.

Different combinations of removing bearing vertical element with different combinations of abnormal loads should be considered and checked to provide stability of structure (equilibrium between effort in a structural elements and its bearing capacity). Calculation should be made for the primary as well as for secondary structural systems.

Removal of any load-bearing vertical element increases the span of slabs and beams and loads for the other elements that leads to appearance of plastic hinge in the middle of the span and big deflection but damage is not accured. It can be reached if the construction of beams and slabs is designed as membrane structure which will withstand the part of enhanced loads. Membrane structure will take vertical (with consideration of resistance to moment) and tension loads.

So when calculating reinforced concrete structures against progressive collapse it would be appropriate to choose the dimensions of the cross-sectional area for elements, area of working reinforcement and mechanical properties of materials to ensure the maximum disclosure of the plastic hinge.

So during calcucation several attempts with with various options of the size of cross section of elements and area of reinforcement were made to find optimal ratio between indicated characteristics and value of deflection which allow to perform the condition (1) – stability structure under abnormal loads.

6 CONCLUSIONS

Progressive collapse, caused by abnormal loads in case of emergency situation, can lead to large amount of human victims, to harsh economic and social consequences. It must be understood that the initial cause can be very small compared to the final destruction. So it is more logical to make attempts to prevent such event as progressive collapse. For this aim this thesis was developed – the concept of progressive collapse was described, the measures to prevent and avoid progressive collapse were offered. On the basis of the foregoing the following conclusions can be made.

In the process of writing the main problem was revealed – lack of the appropriate regulatory framework. The existing documents are recommendations – do not give clear guidelines for the calculation and criteria for checking. Furthermore, these standards are used for Moscow.

During collecting the information, consultation with experts from Russia was held. As a result it was managed to figure out that for Saint-Petersburg the regulatory framework is lagging behind. The reports on surveys of occurred collapses are used as the sources of information. Probably for this reason, the load-bearing structures are not calculated on abnormal loads at all – it is unknown how to do it correctly.

Looking at ways to prevent progressive collapse, it was concluded that it is a complicated task which requires comprehensive approach. In general to prevent progressive collapse we need to ensure adequate continuity, ductility and redundancy of elements and structure in whole to make the system robust to withstand abnormal loads. There are a lot of ways to provide such conditions which can be combined into 2 groups.

For the vast majority of structures, the design requirements may be prescribed using the indirect method which consists of the ensuring of minimum level of connectivity between structural elements, right selection of layout and detailing of connections. This method also involves providing of static indeterminacy by incorporation of additional ties to ensure the spatial work of system. These help to get more robust structure with greater capacity to take abnormal loads.

For special structures, direct analytical methods can be used to determine the required design details. These direct methods may be used to design key elements (their strengthening) to resist a specified threat or to develop its full resistance against loads without failing of the connections or supporting members. This method ensures strength of elements, structural integrity and rigidity by proper calculations, design, using of reinforcement and providing alternate ways for redistribution of loads.

In general to prevent progressive collapse, emphasis is placed, firstly, on the rational choice of constructive system, capable to withstand, and if it is necessary, to redistribute the load and, the secondly, on careful design of bearing elements and their connections.

The carried out calculations led to certain results. First of all, the main requirement for reinforced concrete structures is complete exhaustion bearing capacity without collapse. A significant plastic deformation of the reinforcement and large displacement and cracks can be marked. The ultimate state is standardized by limit deflection and opening angles in the plastic hinge, which arises because of increased loads and large spans. Herewith structures of slabs and beams should be designed as membrane structure to take increased loads.

Calculation of structures against progressive collapse is a difficult and long process, where plenty of combinations of load bearing elements' failure have to be considered. In doing so the main condition for stability of system (balance of external and internal forces) must be performed. In case of nonperformance of this condition, some measures can be taken, for example the cross sectional area of elements and reinforcement can be increased. Calculation is complicated by the fact that it is impossible to use the program to calculate, since they do not take into account all the uncertainties – random nature of an emergence of abnormal load and an unknown pathway of destruction.

The research on the proposed topic showed that the specific behavior of progressive collapse makes it a unique phenomenon, which requires more detailed and thorough study. Design for mitigating the risk of progressive collapse requires a different way of thinking than traditional building structural design – there is no universal approach for evaluating progressive collapse because there are many potential means by which a local collapse may propagate from its initial extent to its final state. Despite the fact that design of structures to resist collapse is more time-consuming than usual design, it should be done properly to avoid disasters.

FIGURES

TABLES

REFERENCES

An overview of progressive collapse. Technical article.

http://www.sedigest.in/article/overview-progressive-collapse

(Accessed on 30 April 2013)

Anonym, 2004. Progressive collapse prevention in new and existing buildings. Department of Civil and Environmental Engineering, University of California, USA.

ASCE 7-02. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, Reston, VA, 2002

Bruce R. Ellingwood, Robert Smilowitz, Donald O. Dusenberry, Dat Duthinh, H.S. Lew, Nicholas J. Carino, 2007. Best Practices for Reducing the Potential for Progressive Collapse in Buildings. National Institute of Standards and Technology. USA

Eurocode 1992: Design of concrete structures. Part 1-1: General rules and rules for buildings. 2004

Jesse K. 2010. Progressive collapse and blast protection for structures. Progressive collapse concept. Side plate.

Johan R., 2010. Limiting the extent of localised failure according to SFS-EN 1991-1-7. Master’s thesis. Aalto University, Espoo.

Korol E.A., Zavalishin S.I., Hlistunov M.S. 2009. Status of normative ensure the safety responsible construction projects in Moscow in conditions of extreme dynamic loads. Scientific and technical journal Vestnik MGSU. Special Issue №2/2009. The implementation of the priority directions in development of science, technology and engineering in the construction of the Russian Federation, pp. 23-28

Kudishin U.I. 2009. Conceptual problems of survivability of building structures. Scientific and technical journal Vestnik MGSU. Special Issue №2/2009. The implementation of the priority directions in development of science, technology and engineering in the construction of the Russian Federation, pp. 28-36

MDS 20-2.2008 “Temporary recommendations providing safety of large-span structures of the avalanche (progressive) collapse in case of emergency impacts”. 2008. Moscow

Perelmyter A.V. 2007. Selected problems of reliability and safety of building structures. Moscow

STO – 008 – 02495342 – 2009 “Prevention of progressive collapse of reinforced concrete monolithic structures of buildings”. 2009. Moscow

STO 36554501 – 014 – 2008 “Reliability of the constructions and the foundations. General rules”. 2008. Moscow

Telichenko V.I., Hlistunov M.S., Zavalishin S.I. 2009. Global risks and new security threats of responsible construction projects of metropolis. Scientific and technical journal Vestnik MGSU. Special Issue №2/2009. The implementation of the priority directions in development of science, technology and engineering in the construction of the Russian Federation, pp. 4-10

UFC 4-023-03: Design of buildings to resist progressive collapse. 2010

Example of calculation for level +7.200

Calculations were made for block 1 at level +7.200. The removed column is situated at intersection of axis 3/E. This column is in-situ concrete element (B40) with square section 600x600 mm.

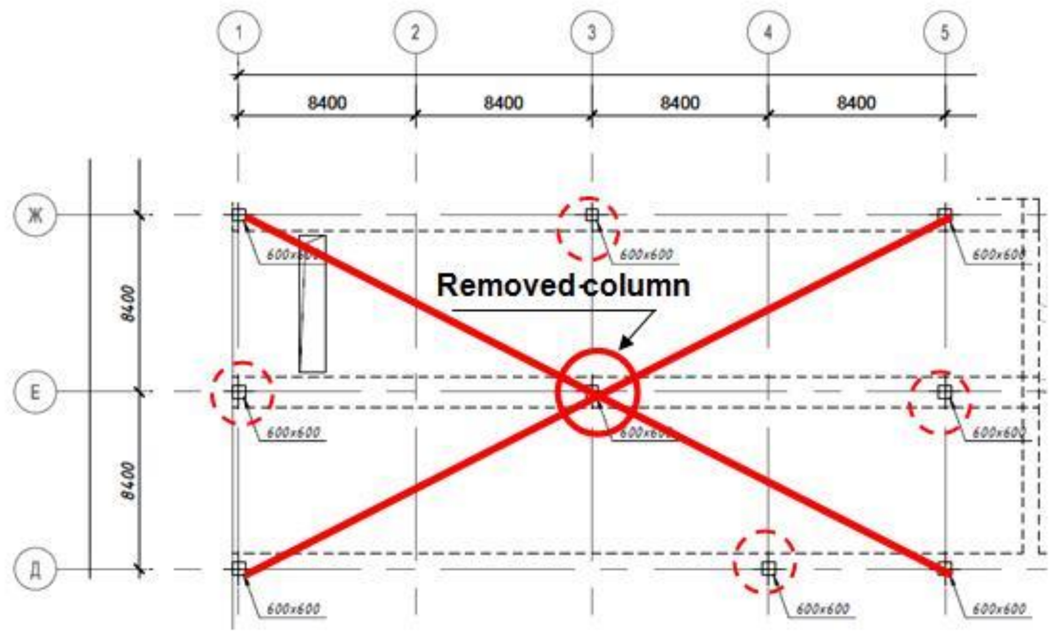

Figure 1. Model of removed column with work of additional supports

1 External loads

Dead load (self-weights of slab and beam):

$$g = t_{slab} \cdot \rho_c + b_{beam} \cdot \frac{h_{beam}}{l_{beam}} \cdot \rho_c;$$

$$g = 0{,}2 \cdot 25 + 0{,}6 \cdot \frac{1{,}2}{8{,}4} \cdot 25 = 7{,}14\ \frac{kN}{m^2}.$$

Live load:

$$q = 4\ \frac{kN}{m^2}.$$

Total load with load factor equals to 1,0 (6):

$$F = 7{,}14 + 4 = 11{,}14\ \frac{kN}{m^2}.$$

2. Permissible value of bearing capacity of the structure

The calculation was made with changed cross-sections of beam (hieght) and considered the work of four columns. The amount and cross section area of reinforcement according the project.

2.1 Calculation of the beam

The material of the beam is concrete (B45) with reinforcement A500 and tendons – Y1860. The beam has the following dimension: height – 600 mm, width – 1200 mm and length – 16800 mm, but in this attempt was used another section: height – 800 mm, width – 1200 mm and length – 16800 mm (Figure 2).

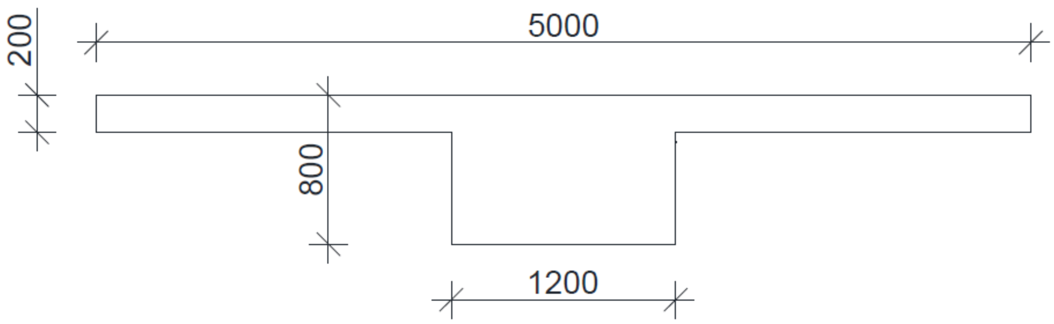

Figure 2. Changed section of the beam and slab

Then the following results were received:

$N_{sd} = 1937$ kN;

$N_{ed} = 1995$ kN;

$R_{s,n} = f_{yc} = 500$ MPa;

$\gamma_s = 1{,}0$, $f_{yd} = 500$ MPa;

$b = 1200$ mm;

$h = 800$ mm.

Upper longitudinal reinforcement in a beam was applied as 8Ø32 (Figure 3):

$A_{s1} = 6432\ \mathrm{mm}^2;$

$d_{s1} = 750\ \mathrm{mm};$

$f_{cd} = 25\ \mathrm{MPa}.$

Lower longitudinal reinforcement in a beam was applied as 3Ø32 and 3Ø25 (Figure 3):

$A_{s2} = 2412 + 1473 = 3885\ \mathrm{mm}^2;$

$d_{s2} = 50\ \mathrm{mm};$

$d = 750\mathrm{mm}.$

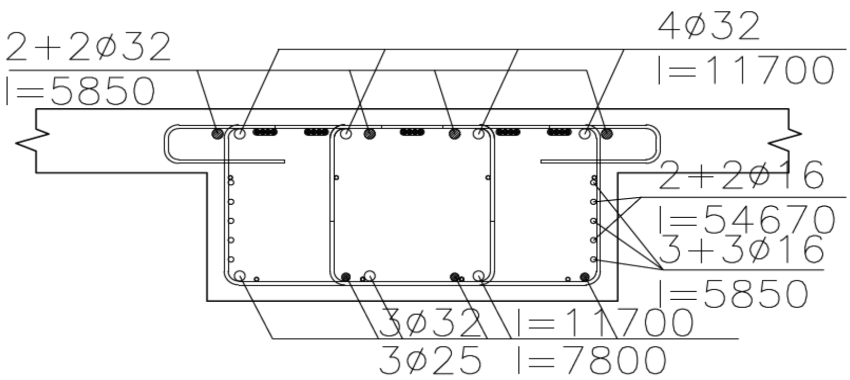

Figure 3. Applied reinforcement in the beam

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 6432 + 187900}{25 \cdot 1200} = 173{,}7\ \mathrm{mm}.$$

Plastic moment capacity of the beam:

$$M_c = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s2} \cdot (d - d_{s2});$$

$$M_c^+ = 25 \cdot 1200 \cdot 173{,}7 \cdot (750 - 0{,}5 \cdot 173{,}7) \cdot 10^{-6} + 500 \cdot 3885 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^+ = (3455{,}7 + 1359{,}7) = 4815{,}45\ \mathrm{kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned):

$$\chi = \frac{f_{yd} \cdot A_{s2} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 3885 + 1995000}{25 \cdot 1200} = 131{,}25\ \text{mm}.$$

Plastic moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s1} \cdot (d - d_{s2});$$

$$M_c^- = 25 \cdot 1200 \cdot 131{,}3 \cdot (750 - 0{,}5 \cdot 131{,}3) \cdot 10^{-6} + 500 \cdot 6432 \cdot (800 - 50) \cdot 10^{-6};$$

$$M_c^- = (2694{,}7 + 2251{,}2)\text{kNm} = 4945{,}9\ \text{kNm}.$$

2.2 Calculation of the slab

Conditions about initial data for calculations against progressive collapse for slabs are the same and the thickness of slab is 200 mm.

2.2.1 Plastic moment capacity in x-direction:

$N_{sd} = 290$ kN;

$N_{ed} = 299$ kN;

$\gamma_s = 1{,}0$; $f_{yd} = 500$ MPa.

Longitudinal reinforcement in x-direction for slab was applied as Ø8 s 200 (Figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{sØ8} \cdot n_{bars} = 252\ \text{mm}^2$;

$d_{s1} = 200$ mm;

$f_{cd} = 25$ MPa;

$b = 1000$ mm;

$h = 200$ mm.

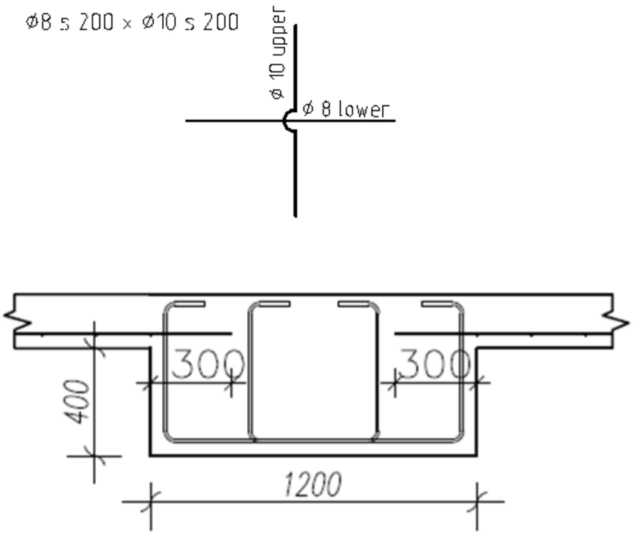

Figure 4. Applied reinforcement in the slab

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0,8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0,8 \cdot 1000 \cdot 25} = 6,3 \text{ mm};$$

$$y = 0,8 \cdot x = 0,8 \cdot 6,3 = 5,04 \text{ mm}.$$

Plastic moment capacity of the slab in x-direction:

$$m_{px} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0,5 \cdot y);$$

$$m_{px} = (25 \cdot 1000 \cdot 5,04 \cdot (150 - 0,5 \cdot 5,04)) \cdot 10^{-6};$$

$$m_{px} = 18,6 \text{ kNm}.$$

Plastic capacity of the axial force of the slab in x-direction:

$$n_{px} = f_{yd} \cdot A_{s1};$$

$$n_{px} = 500 \cdot 252 \cdot 10^{-3} = 126 \text{ kN}.$$

2.2.2 Plastic moment capacity of the slab in y-direction

$$N_{sd} = 290 \text{ kN};$$

$N_{ed} = 299$ kN (longitudinal force from stressing);

$R_{s,n} = f_{yc} = 500$ MPa;

$\gamma_s = 1{,}0$; $f_{yd} = 500$ MPa.

Longitudinal reinforcement in y-direction for slab was applied as Ø10 s 200 (Figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$$A_s = A_{s\varnothing 8} \cdot n_{bars} = 393 \text{ mm}^2;$$

$$d_{s1} = 200 \text{ mm};$$

$$f_{cd} = 25 \text{ MPa};$$

$$b = 1000 \text{ mm};$$

$$h = 200 \text{ mm}.$$

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 393 + 299000}{0{,}8 \cdot 1000 \cdot 25} = 24{,}8 \text{ mm};$$

$$\chi \leq \xi_{c2} \cdot d_2;$$

$$\chi = \xi \cdot d_1 = 0{,}1148 \cdot 200 = 22{,}4\text{mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}057 = 0{,}1148;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}060 + 0{,}032) = 0{,}057;$$

$$\alpha_n = \frac{N_{ed}}{f_{cd} \cdot b_w \cdot d_{s1}} = \frac{299000}{25 \cdot 1000 \cdot 200} = 0{,}060;$$

$$\alpha_{s1} = \frac{f_{yd} \cdot \rho_1}{f_{cd}} = \frac{500 \cdot 0{,}0016}{25} = 0{,}032;$$

$$\rho_1 = \frac{A_s}{A_c} = \frac{393}{250 \cdot 1000} = 0{,}0016;$$

$$y = 0{,}8 \cdot x = 0{,}8 \cdot 22{,}9 = 17{,}9\text{mm}.$$

Plastic moment capacity of the slab in y-direction:

$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$

$\mathrm{m_{py}} = (25 \cdot 1000 \cdot 17{,}9 \cdot (150 - 0{,}5 \cdot 17 \quad 10^{-6};$

$\mathrm{m_{py}} = 63{,}1\ \mathrm{kNm}.$

Capacity of the structure in this case is calculated:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+}{b} + \frac{2 \cdot M_c^-}{b}\right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(18{,}6 + \frac{4815{,}45 + 2 \cdot 4945{,}9}{16{,}8}\right) + \frac{12}{(16{,}8)^2} \cdot 63{,}1 = 12{,}16\ \frac{kN}{m^2};$$

$$S = 12{,}16\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

Attempt 2

The 2nd attempt was made with initial dimension of the beam to show the allowed value of the external load as a function of the deformation in the removed column.

1 External load

$$\mathrm{F} = 11{,}14\ \frac{\mathrm{kN}}{\mathrm{m}^2}.$$

2. Permissible value of bearing capacity of structure

2.1. Calculation of the beam

First it is need to be calculated all moments in the beam. So is used the in-situ concrete beam (B45) with reinforcement A500 and tentonds Y1860. The beam has the following dimensions: height – 600 mm, width – 1200 mm and length – 16800 mm.

$\mathrm{N_{sd}} = 1937\ \mathrm{kN};$

$\mathrm{N_{ed}} = 1995\ \mathrm{kN};$

$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$

$\gamma_\mathrm{s} = 1{,}0,\ \mathrm{f_{yd}} = 500\ \mathrm{MPa};$

$\mathrm{b} = 1200\ \mathrm{mm};$

$\mathrm{h} = 600\ \mathrm{mm}.$

Upper longitudinal reinforcement in beam was applied as 8Ø32 (Figure 3):

$\mathrm{A_{s1}} = 6432\ \mathrm{mm}^2;$

$\mathrm{d_{s1}} = 550\ \mathrm{mm};$

$\mathrm{f_{cd}} = 25\ \mathrm{MPa}.$

Lower longitudinal reinforcement in beam was applied as 3Ø32 and 3Ø25 (Figure 3):

$\mathrm{A_{s2}} = 2412 + 1473 = 3885\ \mathrm{mm}^2;$

$\mathrm{d_{s2}} = 50\ \mathrm{mm};$

$\mathrm{d} = 550\ \mathrm{mm}.$

Height of compression area:

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s1}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 6432 + 1995000}{25 \cdot 1200} = 173{,}7\ \mathrm{mm}.$$

Plastic moment capacity of the beam:

$$M_c^+ = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^+ = 25 \cdot 1200 \cdot 173{,}7 \cdot (550 - 0{,}5 \cdot 173{,}7) \cdot 10^{-6} + 500 \cdot 3885 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c^+ = (2413{,}5 + 971{,}25) = 3384{,}7\ \mathrm{kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned):

$$\chi = \frac{\mathrm{f_{yd}} \cdot \mathrm{A_{s2}} + \mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b}};$$

$$\chi = \frac{500 \cdot 3885 + 1995000}{25 \cdot 1200} = 131{,}25\ \mathrm{mm}.$$

Plastic moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s1} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^- = 25 \cdot 1200 \cdot 131{,}25 \cdot (550 - 0{,}5 \cdot 131{,}25) \cdot 10^{-6} + 500 \cdot 6432 \cdot (550 - 50) \cdot 10^{-6};$$

$$M_c^- = (1907{,}2 + 1608) = 3515{,}2 \text{ kNm}.$$

Also it is needed to consider the plastic capacity for the axial force for the beam:

$$N_c = (A_{s1} + A_{s2}) \cdot \mathrm{R_s} + \mathrm{A_p} \cdot \mathrm{R_p};$$

$$N_c = (6432 + 3885) \cdot 500 + 5 \cdot 4 \cdot 150 \cdot 1860;$$

$$N_c = (6432 + 3885) \cdot 500 + 3000 \cdot 1860;$$

$$N_c = 10{,}7\ MN;$$

2.2 Calculation of the slab

The material of the slab is concrete (B45) with reinforcement A500 and tendons – Y1860. The slab was designed from element to element with thickness 200 mm (Figure 2). Conditions about initial data for calculations against progressive collapse for slabs are the same with conditions for beams.

2.2.1 Moment capacity in x-direction

$$\mathrm{N_{sd}} = 290\mathrm{kN};$$

$$\mathrm{N_{ed}} = 299 \text{ kN};$$

$$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500 \text{ MPa};$$

$$\gamma_\mathrm{s} = 1{,}0; \mathrm{f_{yd}} = 500 \text{ MPa}.$$

Longitudinal reinforcement in x-direction for slab was applied as Ø8 s 200 (Figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$$\mathrm{A_s} = \mathrm{A_{sØ8}} \cdot \mathrm{n_{bars}} = 252 \text{ mm}^2;$$

$$\mathrm{d_{s1}} = 150 \text{ mm};$$

$$\mathrm{f_{cd}} = 25 \text{ MPa};$$

$$\mathrm{b} = 1000 \text{ mm};$$

$$\mathrm{h} = 200 \text{ mm}.$$

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3\ \mathrm{mm};$$

$$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 6{,}3 = 5{,}04\ \mathrm{mm}.$$

Plastic moment capacity of the slab in x-direction:

$$\mathrm{m_{px}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{px}} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6};$$

$$\mathrm{m_{px}} = 18{,}6\ \mathrm{kNm}.$$

Plastic capacity of the axial force of the slab in x-direction:

$$\mathrm{n_{px}} = \mathrm{f_{yd}} \cdot \mathrm{A_{s1}};$$

$$\mathrm{n_{px}} = 500 \cdot 252 \cdot 10^{-3} = 126\ \mathrm{kN}.$$

2.2.2 Moment capacity in y-direction

$$\mathrm{N_{sd}} = 290\ \mathrm{kN};$$

$$\mathrm{N_{ed}} = 299\ \mathrm{kN};$$

$$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$$

$$\gamma_s = 1{,}0;\ \mathrm{f_{yd}} = 500\ \mathrm{MPa}.$$

Longitudinal reinforcement in y-direction in slab was applied as Ø10 s 200 (Figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$$\mathrm{A_s} = \mathrm{A_{sØ8}} \cdot \mathrm{n_{bars}} = 393\ \mathrm{mm^2};$$

$$\mathrm{d_{s1}} = 150\ \mathrm{mm};$$

$$\mathrm{b} = 1000\ \mathrm{mm};$$

$$\mathrm{h} = 200\ \mathrm{mm}.$$

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 393 + 299}{0{,}8 \cdot 1000 \cdot 25} = 24{,}7 \text{ mm};$$

$$\chi \leq \xi_{c2} \cdot \mathrm{d}_2;$$

$$\chi = \xi \cdot \mathrm{d}_1 = 0{,}15 \cdot 150 = 22{,}45 \text{ mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}075 = 0{,}15;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_\mathrm{n} + \alpha_{\mathrm{s}1}) = 0{,}625 \cdot (0{,}08 + 0{,}04) = 0{,}075;$$

$$\alpha_\mathrm{n} = \frac{\mathrm{N_{ed}}}{\mathrm{f_{cd}} \cdot \mathrm{b_w} \cdot \mathrm{d_{s1}}} = \frac{299000}{25 \cdot 1000 \cdot 150} = 0{,}08;$$

$$\alpha_{\mathrm{s}1} = \frac{\mathrm{f_{yd}} \cdot \rho_1}{\mathrm{f_{cd}}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{\mathrm{A_s}}{\mathrm{A_c}} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 22{,}45 = 17{,}96 \text{ mm}.$$

Plastic moment capacity of slab in y-direction:

$$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{py}} = (25 \cdot 1000 \cdot 17{,}96 \cdot (150 - 0{,}5 \cdot 17{,}96)) \cdot 10^{-6};$$

$$\mathrm{m_{py}} = 63{,}3 \text{ kNm}.$$

Then it is possible to calculate internal forces:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py} + \frac{12 \cdot w_c}{a^2} \cdot (\frac{n_{px}}{2} + \frac{N_c}{b});$$

The following table (1) shows dependence between applied forces and deflection for case when additional four columns are not considered.

Table 1. Dependence between applied forces and deflection without work of additional four supports

S(x), m	**F, kH/m^2**
0	5,03
0,1	5,774
0,5	8,750
0,85	11,354
1,3	14,702

The second table (2) – for case with consideretion of middle columns (span is decreased).

Table 2. Dependence between applied forces and deflection with work of additional four supports

S(x), m	**F, kH/m^2**
0	9,48
0,1	10,223
0,2	10,966
0,3	11,710
0,5	13,198

Example of calculation for level +12.600

Calculations were made for block 1 at level +12.600. Removed column is situated at intersection of axis 11/C. This column is in-situ concrete element (B40) with square section 600x600 mm (Figure 1).

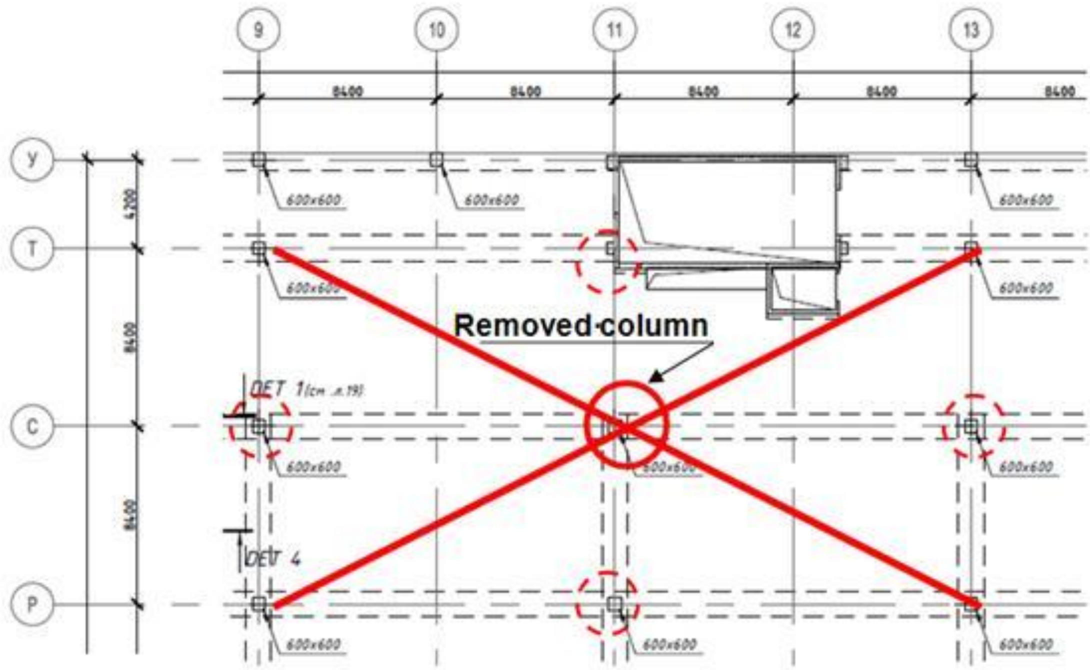

Figure 1. Model of removed column with work of additional supports

1. External loads

Dead load (self-weights of slab and beam):

$$g = t_{slab} \cdot \rho_c + b_{beam} \cdot \frac{h_{beam}}{l_{beam}} \cdot \rho_c;$$

$$g = 0{,}2 \cdot 25 + 0{,}6 \cdot \frac{1{,}2}{8{,}4} \cdot 25 = 7{,}14\ \frac{kN}{m^2}.$$

Live load:

$$q = 4\ \frac{kN}{m^2}.$$

Total load with load factor equals to 1,0 (6):

$F = 7{,}14 + 4 = 11{,}14\ \frac{kN}{m^2}.$

2. Permissible value of bearing capacity of structure

The calculation was made with changed cross-sections of slabs and beams (thickness and heght), reinforcement in beam and considered the work of four columns. The amount and cross section area of reinforcement in slab according the project.

2.1 Calculation of the beam

The material of the beam is concrete (B45) with reinforcement A500 and tendons – Y1860. The beam has the following dimension: height – 800 mm, width – 1200 mm and length – 16800 mm, (Figure 2).

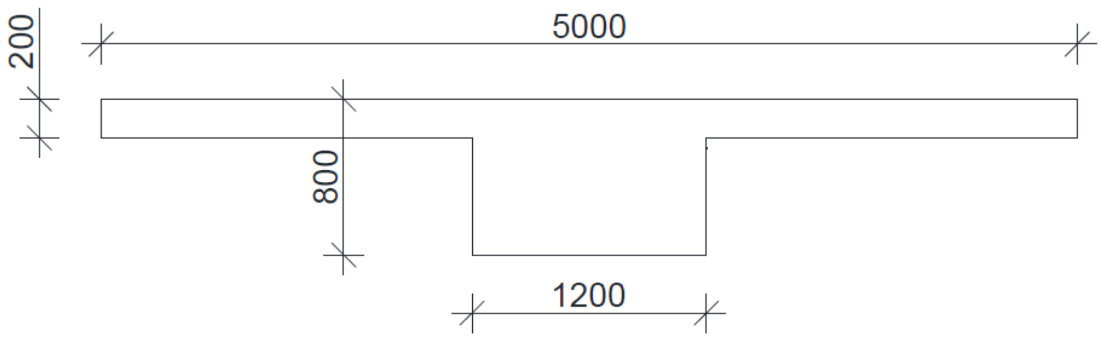

Figure 2. Section of the beam and slab

Then the following results were received:

$N_{sd} = 5464\ kN;$

$N_{ed} = 5623\ kN;$

$R_{s,n} = f_{yc} = 500\ MPa;$

$\gamma_s = 1{,}0$ – in case of calculation against progressive collapse, $f_{yd} = 500\ MPa;$

$b = 1200\ mm;$

$h = 850\ mm.$

Upper longitudinal reinforcement in beam was applied as 2Ø25 and 4Ø32 (Figure 3):

$d_{s1} = 800\ \mathrm{mm}$;

$f_{cd} = 25\ \mathrm{MPa}$.

Lower longitudinal reinforcement in a beam was applied as 4Ø16 (Figure 3):

$A_{s2} = 804\ \mathrm{mm}^2$;

$d_{s2} = 50\ \mathrm{mm}$;

$d = 800\mathrm{mm}$.

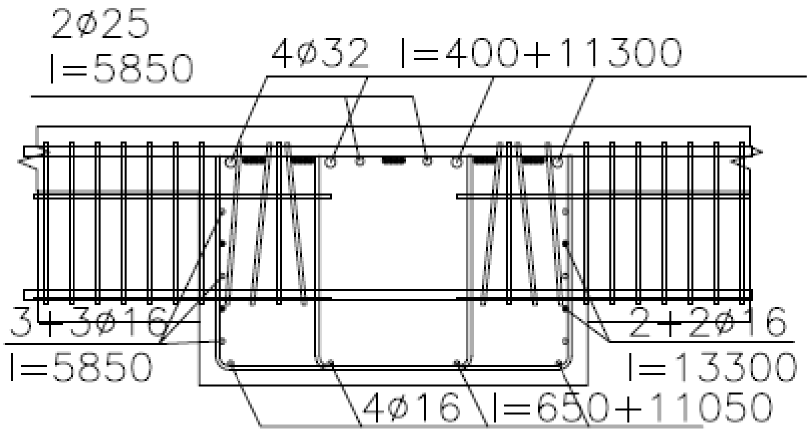

Figure 3. Applied reinforcement in beam according the project

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 5002 + 5623000}{25 \cdot 1200} = 270{,}8\ \mathrm{mm}.$$

Plastic moment capacity of the beam:

$$M_c = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (d - d_{s2});$$

$$M_c^+ = 25 \cdot 1200 \cdot 270{,}8 \cdot (750 - 0{,}5 \cdot 270{,}8) \cdot 10^{-6} + 500 \cdot 804 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^+ = (4993 + 281{,}4) = 5274{,}4\ \mathrm{kNm}.$$

Height of compression area (when the lower reinforcement is become compressioned):

$$\chi = \frac{f_{yd} \cdot A_{s2} + N_{ed}}{f_{cd} \cdot b};$$

$$\chi = \frac{500 \cdot 804 + 5623000}{25 \cdot 1200} = 200{,}8\text{mm}.$$

Plastic moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (d - 0{,}5 \cdot \chi) + f_{ycd} \cdot A_{s1} \cdot (d - d_{s2});$$

$$M_c^- = 25 \cdot 1200 \cdot 200{,}8 \cdot (750 - 0{,}5 \cdot 200{,}8) \cdot 10^{-6} + 500 \cdot 5002 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^- = (3913{,}2 + 1750{,}7) = 5663{,}9 \text{ kNm}.$$

2.2 Calculation of the slab

Conditions about initial data for calculations against progressive collapse for slabs are the same. In this case the thickness of slab was changed – instead of 200 mm, 250 mm was taken.

2.2.1 Plastic moment capacity in x-direction:

$N_{sd} = 300 \text{ kN};$

$N_{ed} = 310 \text{ kN};$

$R_{s,n} = f_{yc} = 500 \text{ MPa};$

$\gamma_s = 1{,}0; f_{yd} = 500 \text{ MPa}.$

Longitudinal reinforcement in x-direction for slab was applied as Ø8 s 200 (Figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{sØ8} \cdot n_{bars} = 252 \text{ mm}^2;$

$d_{s1} = 150 \text{ mm};$

$f_{cd} = 25 \text{ MPa};$

$b = 1000 \text{ mm};$

$h = 200 \text{ mm}.$

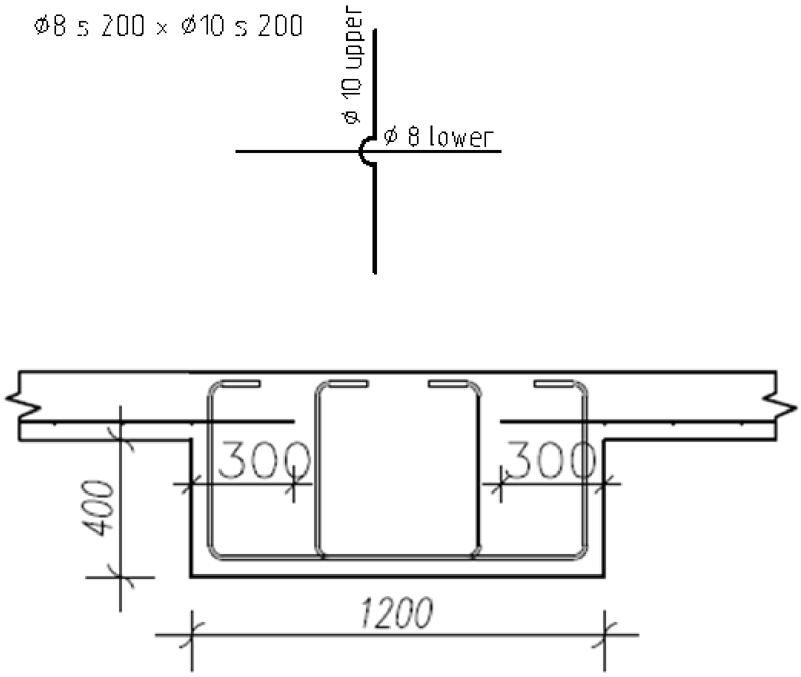

Figure 4. The Applied reinforcement in a slab

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3 \text{ mm};$$

$$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 6{,}3 = 5{,}04 \text{ mm}.$$

Plastic moment capacity of the slab in x-direction:

$$\mathrm{m_{px}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{px}} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6};$$

$$\mathrm{m_{px}} = 18{,}6 \text{ kNm}.$$

Plastic capacity of the axial force of the slab in x-direction:

$$\mathrm{n_{px}} = \mathrm{f_{yd}} \cdot \mathrm{A_{s1}};$$

$$\mathrm{n_{px}} = 500 \cdot 252 \cdot 10^{-3} = 126 \text{ kN}.$$

2.2.2 Plastic moment capacity in y-direction

$N_{sd} = 300\ \text{kN};$

$N_{ed} = 310\ \text{kN (longitudinal force from stressing)};$

$R_{s,n} = f_{yc} = 500\ \text{MPa};$

$\gamma_s = 1{,}0;\ f_{yd} = 500\ \text{MPa}.$

Longitudinal reinforcement in y-direction in slab was applied as Ø10 s 200 (figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$A_s = A_{sØ8} \cdot n_{bars} = 393\ \text{mm}^2;$

$d_{s1} = 150\ \text{mm};$

$f_{cd} = 25\ \text{MPa};$

$b = 1000\ \text{mm};$

$h = 200\ \text{mm}.$

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1} + N_{ed}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 393 + 310}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3\ \text{mm};$$

$$\chi \le \xi_{c2} \cdot d_2;$$

$$\chi = \xi \cdot d_1 = 0{,}1534 \cdot 150 = 23\ \text{mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}077 = 0{,}1534;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}0827 + 0{,}04) = 0{,}077;$$

$$\alpha_n = \frac{N_{ed}}{f_{cd} \cdot b_w \cdot d_{s1}} = \frac{310000}{25 \cdot 1000 \cdot 150} = 0{,}0827;$$

$$\alpha_{s1} = \frac{f_{yd} \cdot \rho_1}{f_{cd}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{A_s}{A_c} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 23 = 18{,}4\ \mathrm{mm}.$

Plastic moment capacity of the slab in y-direction:

$\mathrm{m_{py}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$

$\mathrm{m_{py}} = (25 \cdot 1000 \cdot 18{,}4 \cdot (150 - 0{,}5 \cdot 18{,}4)) \cdot 10^{-6};$

$\mathrm{m_{py}} = 64{,}8\ \mathrm{kNm}.$

Then it is possible to calculate structure's capacity:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c^+}{b} + \frac{2 \cdot M_c^-}{b} \right) + \frac{12}{b^2} \cdot m_{py};$$

$$S = \frac{12}{(33{,}6)^2} \cdot \left(18{,}6 + \frac{5274{,}4 + 2 \cdot 5663{,}9}{16{,}8} \right) + \frac{12}{(16{,}8)^2} \cdot 64{,}8 = 13{,}43\ \frac{kN}{m^2};$$

$$S = 13{,}43\ \frac{kN}{m^2} > F = 11{,}14\ \frac{kN}{m^2}.$$

Attempt 2

The 2nd attempt was made with initial dimension of beam to show the allowed value of the external load as a function of the deformation in the removed column.

1 External load

$$\mathrm{F} = 11{,}14\ \frac{\mathrm{kN}}{\mathrm{m^2}}.$$

2. Permissible value of bearing capacity of structure

2.1. Calculation of the beam

First it is need to be calculated all moments in beam. So is used the in-situ concrete beam (B45) with reinforcement A500 and tentonds Y1860. The beam has the following dimensions: height – 800 mm, width – 1200 mm and length – 16800 mm.

$\mathrm{N_{sd}} = 5464\ \mathrm{kN};$

$\mathrm{N_{ed}} = 5623\ \mathrm{kN};$

$\mathrm{\gamma_s = 1{,}0,\ f_{yd} = 500\ MPa;}$

$\mathrm{b = 1200\ mm;}$

$\mathrm{h = 800\ mm.}$

Upper longitudinal reinforcement was applied as 2Ø25 and 4Ø32 (Figure 3):

$\mathrm{A_{s1} = 5002\ mm^2;}$

$\mathrm{d_{s1} = 550\ mm;}$

$\mathrm{f_{cd} = 25\ MPa.}$

Lower longitudinal reinforcement was applied as 4Ø16 (Figure 3):

$\mathrm{A_{s2} = 2412 + 1473 = 3885\ mm^2;}$

$\mathrm{d_{s2} = 50\ mm;}$

$\mathrm{d = 550\ mm.}$

Height of compression area:

$$\chi = \frac{\mathrm{f_{yd} \cdot A_{s1} + N_{ed}}}{\mathrm{f_{cd} \cdot b}};$$

$$\chi = \frac{500 \cdot 5002 + 5623}{25 \cdot 1200} = 270{,}8\ \mathrm{mm.}$$

Plastict moment capacity of the beam:

$$M_c^+ = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s2} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^+ = 25 \cdot 1200 \cdot 270{,}8 \cdot (750 - 0{,}5 \cdot 270{,}8) \cdot 10^{-6} + 500 \cdot 804 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^+ = (4993 + 281{,}4) = 5274{,}4\ \mathrm{kNm.}$$

Height of compression area (when the lower reinforcement is become compressioned):

$$\chi = \frac{\mathrm{f_{yd} \cdot A_{s2} + N_{ed}}}{\mathrm{f_{cd} \cdot b}};$$

$$\chi = \frac{500 \cdot 804 + 5623}{25 \cdot 1200} = 200{,}8\ \mathrm{mm.}$$

Plastic moment capacity of the beam:

$$M_c^- = f_{cd} \cdot b \cdot \chi \cdot (\mathrm{d} - 0{,}5 \cdot \chi) + f_{yd} \cdot A_{s1} \cdot (\mathrm{d} - \mathrm{d_{s2}});$$

$$M_c^- = 25 \cdot 1200 \cdot 200{,}8 \cdot (750 - 0{,}5 \cdot 200{,}8) \cdot 10^{-6} + 500 \cdot 5002 \cdot (750 - 50) \cdot 10^{-6};$$

$$M_c^- = (3913{,}2 + 1750{,}7) = 5663{,}9\ \mathrm{kNm}.$$

Also it is needed to consider the plastic capacity for the axial force for the beam:

$$N_c = (A_{s1} + A_{s2}) \cdot \mathrm{R_s} + \mathrm{A_p} \cdot \mathrm{R_p};$$

$$N_c = (5002 + 804) \cdot 500 + 5 \cdot 4 \cdot 150 \cdot 1860;$$

$$N_c = 8{,}5MN;$$

2.2 Calculation of the slab

The material of the slab is concrete (B45) with reinforcement A500 and tendons – Y1860. The slab was designed from element to element with thickness 200 mm (Figure 2). Conditions about initial data for calculations against progressive collapse for slabs are the same with conditions for beams.

2.2.1 Plastic moment capacity in x-direction

$$\mathrm{N_{sd}} = 300\mathrm{kN};$$

$$\mathrm{N_{ed}} = 310\ \mathrm{kN};$$

$$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500\ \mathrm{MPa};$$

$$\gamma_\mathrm{s} = 1{,}0; \mathrm{f_{yd}} = 500\ \mathrm{MPa}.$$

Longitudinal reinforcement in x-direction for slab was applied as Ø8 s 200 (figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$$\mathrm{A_s} = \mathrm{A_{sØ8}} \cdot \mathrm{n_{bars}} = 252\ \mathrm{mm^2};$$

$$\mathrm{d_{s1}} = 150\ \mathrm{mm};$$

$$\mathrm{f_{cd}} = 25\ \mathrm{MPa};$$

$$\mathrm{b} = 1000\ \mathrm{mm};$$

$$\mathrm{h} = 200\ \mathrm{mm}.$$

Height of compression area:

$$\chi = \frac{f_{yd} \cdot A_{s1}}{0{,}8 \cdot b \cdot f_{cd}};$$

$$\chi = \frac{500 \cdot 252}{0{,}8 \cdot 1000 \cdot 25} = 6{,}3 \text{ mm};$$

$$\mathrm{y} = 0{,}8 \cdot \mathrm{x} = 0{,}8 \cdot 6{,}3 = 5{,}04 \text{ mm}.$$

Plastic moment capasity of the slab in x-direction:

$$\mathrm{m_{px}} = \mathrm{f_{cd}} \cdot \mathrm{b} \cdot \mathrm{y} \cdot (\mathrm{d_{s1}} - 0{,}5 \cdot \mathrm{y});$$

$$\mathrm{m_{px}} = (25 \cdot 1000 \cdot 5{,}04 \cdot (150 - 0{,}5 \cdot 5{,}04)) \cdot 10^{-6};$$

$$\mathrm{m_{px}} = 18{,}6 \text{ kNm}.$$

Plastic tension capacity in x-direction:

$$\mathrm{n_{px}} = \mathrm{f_{yd}} \cdot \mathrm{A_{s1}};$$

$$\mathrm{n_{px}} = 500 \cdot 252 \cdot 10^{-3} = 126 \text{ kN}.$$

2.2.2 Plastic moment capasity of the slab in y-direction

$$\mathrm{N_{sd}} = 300 \text{ kN};$$

$$\mathrm{N_{ed}} = 310 \text{ kN};$$

$$\mathrm{R_{s,n}} = \mathrm{f_{yc}} = 500 \text{ MPa};$$

$$\gamma_\mathrm{s} = 1{,}0;\ \mathrm{f_{yd}} = 500 \text{ MPa}.$$

Longitudinal reinforcement in y-direction for slab was applied as Ø10 s 200 (figure 4). Calculation was made for strip with width 1000 mm:

$$n_{bars} = \frac{1000}{200} = 5;$$

$$\mathrm{A_s} = \mathrm{A_{s\emptyset 8}} \cdot \mathrm{n_{bars}} = 393 \text{ mm}^2;$$

$$\mathrm{d_{s1}} = 150 \text{ mm};$$

$$\mathrm{f_{cd}} = 25 \text{ MPa};$$

$$\mathrm{b} = 1000 \text{ mm};$$

$$\mathrm{h} = 200 \text{ mm}.$$

Height of compression area:

$$\chi = \frac{500 \cdot 393 + 310}{0{,}8 \cdot 1000 \cdot 25} = 25{,}3 \text{ mm};$$

$$\chi \leq \xi_{c2} \cdot d_2;$$

$$\chi = \xi \cdot d_1 = 0{,}15 \cdot 150 = 23 \text{ mm};$$

$$\xi = 2 \cdot \lambda_1 = 2 \cdot 0{,}075 = 0{,}15;$$

$$\lambda_1 = 0{,}625 \cdot (\alpha_n + \alpha_{s1}) = 0{,}625 \cdot (0{,}08 + 0{,}04) = 0{,}075;$$

$$\alpha_n = \frac{N_{ed}}{f_{cd} \cdot b_w \cdot d_{s1}} = \frac{299000}{25 \cdot 1000 \cdot 150} = 0{,}08;$$

$$\alpha_{s1} = \frac{f_{yd} \cdot \rho_1}{f_{cd}} = \frac{500 \cdot 0{,}002}{25} = 0{,}04;$$

$$\rho_1 = \frac{A_s}{A_c} = \frac{393}{200 \cdot 1000} = 0{,}002;$$

$$y = 0{,}8 \cdot x = 0{,}8 \cdot 22{,}45 = 18{,}4 \text{ mm}.$$

Plastic moment capasity of the slab in y-direction:

$$m_{py} = f_{cd} \cdot b \cdot y \cdot (d_{s1} - 0{,}5 \cdot y);$$

$$m_{py} = (25 \cdot 1000 \cdot 17{,}96 \cdot (150 - 0{,}5 \cdot 17{,}96)) \cdot 10^{-6};$$

$$m_{py} = 64{,}8 \text{ kNm}.$$

Then it is possible to calculate internal forces:

$$S = \frac{12}{a^2} \cdot \left(m_{px} + \frac{M_c}{b}\right) + \frac{12}{b^2} \cdot m_{py} + \frac{12 \cdot w_c}{a^2} \cdot \left(\frac{n_{px}}{2} + \frac{N_c}{b}\right);$$

The following table (1) shows dependence between applied forces and deflection for case when additional four columns are not considered.

Table 1. Dependence between applied forces and deflection without work of 4 columns

S(x), m	**F, kH/m^2**
0	6,29
0,1	6,895
0,5	9,314
0,85	11,430
1,3	14,152

And table (2) is showed the dependence between applied forces and deflection for case when additional four columns are considered.

Table 2. Dependence between applied forces and deflection with work of 4 columns

S(x), m	**F, kH/m^2**
0	13,46
0,01	13,517
0,05	13,759
0,1	14,062
0,15	14,364