19th International Conference
ENGINEERING MECHANICS 2013
Svratka, Czech Republic, May 13 – 16, 2013

ASSESSMENT OF BUILDING STRUCTURE LOADED BY TERRORIST EXTERNAL EXPLOSION

D. Makovička*, D. Makovička**

Abstract: *The paper follows from the theory of explosion and interaction of an impact wave formed by the explosion and a structure. As a rule, a number of simplifying assumptions must be applied as regards the characteristics of the explosion and of the threatened structure to analyze the structure. An example of dynamic analysis of a new reinforced concrete structure, loaded with a terrorist charge explosion of 90 kg TNT located in a car in the distance of 18.3 m from the threatened structure, was used to apply the principles of simplified engineering analysis of an explosion-loaded structure. 3D computational model of the building structure was developed and dynamic response of the building was calculated. The way of structure failure was analyzed based on time courses of calculated internal forces and displacements of individual structure elements. The criteria of structural elements failure due to explosion load effects were determined as a part of the dynamic structure response assessment.*

Keywords: ***explosion, blast wave, building, dynamic response, assessment.***

1. Introduction

Evaluation of safety and reliability of building structure, particularly based on experience gained worldwide and today also based on Eurocodes, requires that some structures be designed for extraordinary loads caused by external influences. Explosion load (Makovička & Janovský, 2008) is also one of such influences, caused by an explosion of condensed explosives in flats, industrial structures and in the outside environment. A blast wave develops due to all types of explosions. Its intensity and course in time are given by chemical properties of the explosive (flammable) substance or by the physical state of the substance and its reactions with the surrounding environment. This essential manifestation of any explosion also depends on physical conditions during the explosion (in particular, the temperature, humidity, flowing, wind conditions, etc.).

Thus the blast wave starts propagate from the point of explosion approximately in spherical wave fronts, and upon hitting the surface of a building structure (walls, ceilings, floors, equipment, etc.) or terrain, the wave front is reflected and modified. The action of pressure in the propagated wave, together with the pressure wave reflected from the surface of a structure or terrain, determines the magnitude of the structure load and its course in time. Particularly, in enclosed areas such as rooms and industrial halls where multiple reflections may occur, precisely the size of the enclosed area and also that of exhaust openings through which any overpressure may escape from the place of explosion and thus modify the load characteristics are dominant factors for the structure load magnitude.

In the process of evaluating the building structure response to the effects of an explosion, specific conditions of the given locality and of the building structure should be considered, based on which the structure response to explosion load can be estimated, either more accurately by a calculation or approximately based on empiric formulas and criteria (Makovička & Makovička, 2011). In particular, this applies to the type and location of the pressure wave source compared to the structure under evaluation, characteristics of the pressure wave at the source, and especially the course of explosion pressure in time.

* Doc. Ing. Daniel Makovička, DrSc.: Klokner Institute, Czech Technical University in Prague, Šolínova 7; 166 08, Prague 6; CZ, e-mail: daniel.makovicka@klok.cvut.cz

** Ing. Daniel Makovička, Jr.: Static and Dynamic Consulting, Šultysova 170; 284 01 Kutná Hora; CZ, e-mail: d.makovicka@makovicka.cz

Properties of the structure as a unit or of its parts and its materials are decisive for the magnitude and nature of the response of any explosion-loaded structure. These include particularly mechanical characteristics of the material (especially its strength, way of failure, stress-strain diagram, behaviour beyond the elasticity limit, etc.), and distribution of masses and structure rigidity with corresponding frequency tuning of the structure, characteristics of surfaces loaded by the impact blast wave, structure geometry compared to explosion wave characteristics, any previous failures of the structure, including changes in the structure material properties in the course of time for existing structures, etc. (Design .., 1997).

In our specific case, the explosion load effect is applied to the analysis of a control, single-storey building (porter's lodge) within the threatened premises. It is a reinforced concrete monolithic structure that should be resistant, based on the user's requirements, against terrorist charge explosion of defined parameters.

2. Design principles

As a rule, failure of a limited part of the structure may be admitted in the structure design process providing that no crucial elements are included in such a part on which the stability of the entire structure depends.

When calculating building or technological structures, two procedures can be applied in principle. Either maximum possible simplifications are used in the structure analysis in terms of explosion effects (TM 5-1300, 1990), both as regards the load itself and the analyzed structure, or the structure is analyzed in a way so that this analysis describes with the highest accuracy possible the actual state of the structure and its explosion load (Makovička et al., 2009). In such a case, effects of non-linearities can be considered, both material and geometric, and probable courses of the load can be introduced, e.g. based on measurements. The simplified calculation, or its methodology at least, is often normalized worldwide so that it can be used by the broad public of technical specialists. Basic structure calculation principles were described in the publication (Koloušek, 1967). Appropriate limit states and their corresponding loads were used to evaluate the effects of an explosion.

Requirement to exclude an accident:

- The structure must tolerate design-based explosion load without collapsing, as a whole or in part, so that it maintains its structural integrity and residual bearing capacity after the explosion.
- The design-based explosion load, corresponding to the simplified course of load in time, is normally given by intensity of maximum overpressure and underpressure values of the impact wave and by the duration of both phases, and/or dynamic pressure and its duration. The load parameters should be considered based on the probability of explosion occurrence in the given locality, based on the structure, operation, etc.

Requirement of limited damage:

- The structure should resist any (higher) explosion load of higher occurrence probability than the design-based explosion load, with no damage and without any associated restrictions of operation, such that their price would be disproportionately high compared to the price of the construction.
- The resulting reliability against collapse and against limited damage is normally determined by national authorities for various types of buildings and engineering constructions according to the consequences of damage, or they are determined based on risk analyses for the appropriate operation, structure, etc.

3. Explosion load

The explosion load is very often substituted as follows to achieve simplification (Henrych, 1979 and Koloušek 1967):

a) Triangle-shaped development of the load in time with the maximum intensity corresponding to the sum of the pressures of the impacting and reflected wave and the duration of the action, usually corresponding only to the duration of the action of the overpressure phase of the shock wave;

b) The shock wave can be considered as having a flat front, meaning that the rise time to maximum intensity is neglected, and additionally that the load starts to act on the entire structure at one moment; the phase shift of the start of the action of the load at individual structure points is thus neglected;

c) It is usually assumed that the load acts on the building structure (walls, ceiling, windows, etc.) in a continuous and uniform manner (any local effect of the focused load is neglected);

d) The response of the structure is usually considered on the basis of the superimposition of two triangular loads, which correspond to the overpressure phase and subsequently the underpressure phase of the shock wave.

The authors used empirical formulas (Makovička & Makovička, 2011, 2012 and Makovička & Janovský, 2008) applicable to an explosive charge in an open area to calculate the dynamic load; the formulas were derived from tests using small explosive charges; then the overpressure value p_+ at the front of the aerial shock wave and its duration τ_+ are as follows:

$$p_+ = \frac{1.07}{\overline{R}^3} - 0{,}1 \quad \text{[MPa]} \quad \text{for } \overline{R} \le 1 \text{ m/kg}^{1/3} \tag{1a}$$

$$p_+ = \frac{0.0932}{\overline{R}} + \frac{0.383}{\overline{R}^2} + \frac{1.275}{\overline{R}^3} \quad \text{[MPa]} \quad \text{for } 1 < \overline{R} \le 15 \text{ m/kg}^{1/3} \tag{1b}$$

$$p_- = \frac{0{,}035}{\overline{R}} \quad \text{[MPa]} \tag{2}$$

$$\tau_+ = 1.6 \cdot 10^{-3} \cdot \sqrt[6]{C_\mathrm{w}} \cdot \sqrt{R} \quad \text{[s]} \tag{3}$$

$$\tau_- = 1.6 \cdot 10^{-2} \cdot \sqrt[3]{C_\mathrm{w}} \quad \text{[s]} \tag{4}$$

For reduced distance

$$\overline{R} = \frac{R}{\sqrt[3]{C_\mathrm{w}}} \quad .... \text{ [m/kg}^{1/3}\text{]} \tag{5}$$

where $\overline{R}$ is the reduced distance from the epicentre of the explosion, R is the distance from the explosion epicentre [m], and C_w is the equivalent mass of the explosive charge [kg TNT].

The wave motion from the explosion focus propagates in spherical wave fronts. In the event of a surface explosion (at the contact point with the ground), the explosion energy value is about double, given that when there is complete reflection from the ground surface the shock wave propagates in semi-spherical wave fronts. For a surface explosion, this effect can as a rule be taken into account by substituting twice the magnitude of the actually used mass of charge C for the equivalent mass of charge C_w in formula (5). For an above-ground explosion at a height of more than 20 m above ground, the mass of the charge C is substituted directly (without any increase in its value) for the equivalent mass of the charge. For a charge placed between the ground level (zero height) and 20 m above the ground, linear interpolation can be used to determine the equivalent mass of the charge; in this case, the equivalent mass of the charge substituted to the formulas above will range between

$$C_w = (1 \text{ to } 2)\ C \tag{6}$$

When there is a normal (perpendicular) impact of the explosion wave against a solid barrier, a reflected wave is formed with the reflection overpressure $p_{\mathrm{ref}+}$, which loads the building structure from the front side. The overpressure value in the reflected wave corresponds to approximately twice the overpressure for low overpressure values p_+ approximately up to 5 MPa (up to eight times the value for high overpressures of several tens of MPa) in the impact wave for the given distance R. The duration of the action of the overpressure t_D is about the same as the duration of shock wave τ_+

$$p_{\mathrm{ref}+} \approx 2\,p_{+} \tag{7}$$

$$t_{\mathrm{D}} \approx \tau \tag{8}$$

4. Response calculation taking into account the ductility of the structure

The structure response is generally calculated and assessed in accordance with design standards for the given type of structure material. In our case, Eurocodes are used. The dynamic response to the effects of the load due to an explosion must be superimposed on the effects due to static loads. These are usual procedures, but it should be noted that when the structure is loaded due to an explosion, inelastic deformations occur at a number of sections, causing damage to the structure by crack formation. In this case, the stability of the structure with the cracks should be assessed in order to prevent any collapse of the structure due to the formation of plastic joints and cracks.

When a structure is loaded by an explosion, the formation of cracks not leading to a collapse is as a rule permitted. Thus ductility factor q may be used to reduce the magnitude of the explosion load. This is a highly efficient way of taking inelastic manifestations of the dynamic load into account.

$$q = x_{\mathrm{m}} / x_{el} \tag{9}$$

where x_m is the maximum elastic plastic displacement of the structure, and x_{el} is the elastic part of the displacement.

The applicable ductility factor is usually q < 3 for reinforced concrete structures. On the basis of a more detailed analysis of the structure, higher ductility factor values may be used, for example, on the basis of seismic standard (ČSN EN 1998-1, 2006).

The strength characteristics of the structure material may also be increased in the calculation of the structure response. An estimate of this increase (material strengthening factor k_1) is shown in Tab. 1, in dependence on the duration of the explosion load effect t_{D}.

Tab. 1: Estimate of factor k_1 in dependence on load duration t_D

$\mathbf{t_D}$ **[s]**	≥ 1.0	10^{-1}	10^{-2}	10^{-3}
$\mathbf{k_1}$	1.0	1.05	1.10	1.20

5. Evaluation of the structure response

The magnitudes of the internal forces in the structure are considered as a part of the evaluation of the limit bearing capacity conditions, based on load combinations when they are reduced using ductility factor q (Makovička & Makovička 2011, 2012 and ČSN EN 1998-1, 2006). The resulting internal forces are then evaluated on the basis of design standards for the appropriate structure material type, or as a variant, also according to its increased strength using factor k_1. However, this procedure entails two important uncertainties in the case of bent structures, i.e. a suitable choice of the ductility factor, on the one hand, and the material strengthening factor, on the other. During very rapid reshaping of the structure, which is typical for explosion loads, both factors may achieve numeric values of the order of tens, and not only of units, as mentioned above. Thus they may lead to considerable overdesigning of the structure.

Evaluations of structures loaded by an explosion based on dynamic displacement and rotation round the central line of plate, wall or beam systems during the action of a dynamic load of this type have been of very topical interest in recent times, as regards the process of evaluating the effects of an explosion on a structure.

In earlier publications (Makovička 1998, 1999 and Makovička & Makovička, 2010, 2011, 2012), the authors applied this procedure to various types of materials and structure systems, and on the basis of an experimental comparison they determined the failure angle $\psi_{\max}$, i.e. the angle where damage is caused to the structure by breaking.

The dynamic rotation round the central line of an appropriate structure element is therefore the criterion used to evaluate the response occurring at the following angle

$$\psi = \text{arctg}\,(x_m / (0.5\, h_{span})) \tag{10}$$

where x_m is the maximum achieved dynamic displacement caused by the explosion load and h_{span} is the span of the plate ceiling structure or the height of the wall structure within one storey, or the span of any beam, the height of a column, etc.

Tab. 2: Limit failure angle ψ_{max} [°] upon breaking of the material (Makovička 1998, 1999)

Type	Structure material	ψ_{max} [°]
1	Concrete C16/20 to C40/50	6.5
2	Masonry, full bricks 10, mortar 4 or mortar 10	5.0
3	Masonry, cement bricks, mortar 4	4.5
4	Masonry, cellular concrete or perforated precise blocks, mortar 4	4.0
5	Steel S235	10.5
6	Wood, hard and soft	12
7	Window glass, thickness 3 mm	6

Tab. 3: Angle ψ [°] of the expected damage to bent structural elements (Makovička, 1999 and McCann & Smith, 2007 and Design ... 1997)

Structure	Expected damage to elements		
	Mean	High	Hazardous
Reinforced concrete structures, plates and beams with one-sided reinforcement	2	5	10
Reinforced concrete structures, plates and beams with two-sided reinforcement and with web reinforcement	4	6	10
Prestressed concrete, beams and plates	1	1.5	2
Masonry, common, non-reinforced	1.5	4	8
Masonry, reinforced	2	8	15
Steel bars	3	10	20

The approximate failure angle value on reaching the rupture limit value is shown in Tab. 2. More conservative limit values of angle ψ were derived according to (McCann & Smith, 2007 and Design ... 1997), which correspond to the chosen structure rupture risk. These values have been adapted and are shown in Tab. 3.

The mean occurrence of damage corresponds to the damage to reinforced concrete or masonry elements, e.g. spalling, or the occurrence of tiny cracks in the structure elements, which pose no threat to their stability and can be repaired, e.g. by grouting.

However, hazardous occurrence of damage approaches emergency level damage, and its failure angle is found at the lower limit, below the maximum failure angle value ψ_{max}, see Tab. 2.

6. Description of the threatened structure and its computational model

The reinforced concrete wall structure of the building was made of concrete C25/30, wall in thickness 200 mm, ceiling and floor slab in thickness 250 mm, and it was sufficiently reinforced using classic reinforcement in both directions (crosswise) along both surfaces.

Window and door openings of such a building are usually fitted with special windows and doors resistant against explosion given that regular window glasses do not transfer the effects.

The subsoil of the building is of gravel-sand nature and was modelled using the Winkler-Pasternak two-parametric subsoil model. The computational model of the building is illustrated in Fig. 1. The dimensions and distribution of individual structure parts were designed while respecting the structure geometry and its dimensions, in order to obtain the most precise model of the building's mass and rigidity. Besides its dead load, the equivalent (permanent component) of the variable load were included in the structure mass.

During an explosion, the specific course of load also depends on vortex flow around the structure surface, atmospheric pressure, temperature conditions and other factors that are usually neglected in the simplified analysis. In our case, only simplified flow around the building was considered. Explosion load parameters were determined based on average values; the formulas used to calculate the load are empirical and operate with mean (probable) values of the coefficients. Thus the structure calculations for the impact wave effects are burdened significantly by these inaccuracies of input quantities of the whole phenomenon, as well (Makovička & Makovička, 2010).

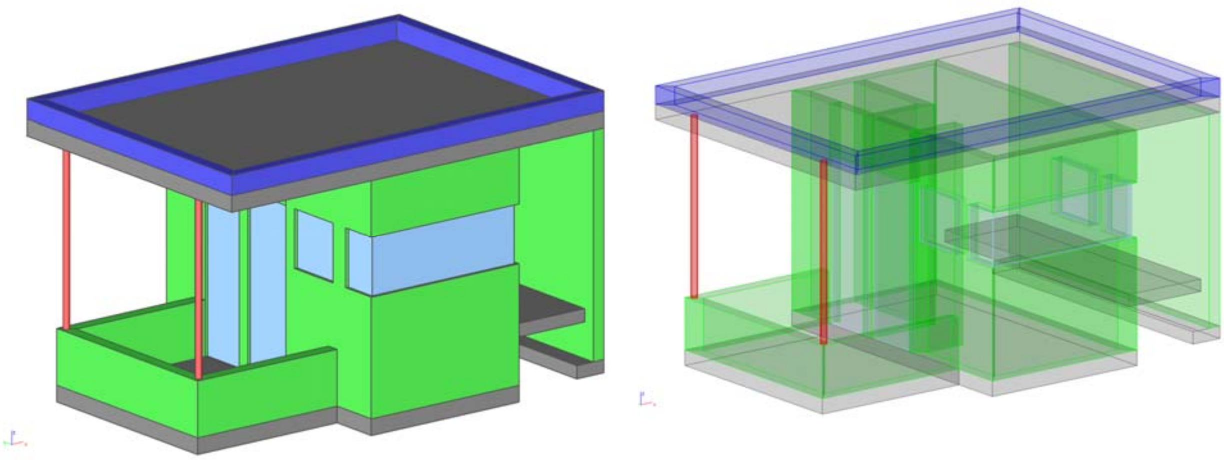

Fig. 1: Calculation model (solid and transparent) of the whole structure, north-west view

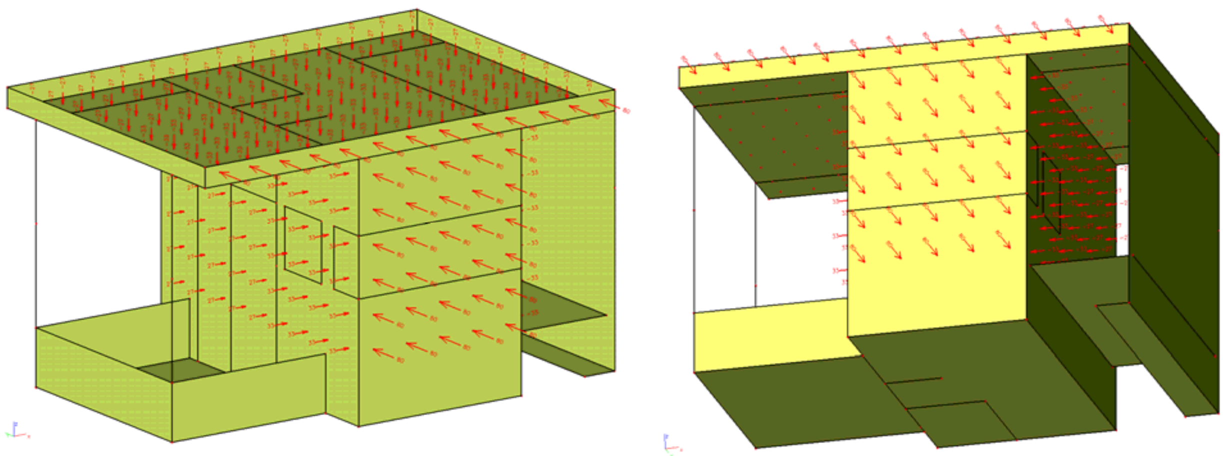

Fig. 2: Dynamic load intensities in selected points of above-ground parts of structure

The load exerted on circumferential wall surfaces and on ceiling structures was considered as uniform, graduated in three zones in terms of intensity as well as the initial moment of action of the reflective overpressure – dynamic load, as a function of the impact wave velocity of propagation. The distribution of points where dynamic load was applied, and its maximum values are illustrated in Fig. 2.

Zone 1, the whole west frontal wall:

– characteristics of incident shock wave $p_+ = 160$ kPa, $t_+ = 14$ ms,

– start of dynamic load on structure face after explosion ... $t = 26$ ms ($t^* = 0$ ms),

– angle of incidence ... 90 to 79 degs, reflection factor approximately 2,

– structure load ... 2 × 160 = 320 kPa.

Zone 2, perpendicular to the blast wave propagation, frontal parts of the north and south walls, frontal part of the roof:

– characteristics of incident shock wave $p_+ = 132$ kPa, $t_+ = 14$ ms,

– start of dynamic load on structure face after explosion ... $t = 30$ ms ($t^* = +4$ ms),

– structure load ... 133 kPa.

Zone 3, perpendicular to the blast wave propagation, rear parts of the north and south walls, rear part of the roof:

– characteristics of incident shock wave $p_+ = 108$ kPa, $t_+ = 15$ ms,

– start of dynamic load on structure face after explosion ... $t = 35$ ms ($t^* = +9$ ms),

– structure load ... 110 kPa.

7. Natural vibration

60 lowest natural modes of vibration were considered in the computation. Their natural frequencies and modes are described in Tab. 4.

Tab. 4: The natural frequencies and modes

n	$f_{(n)}$ [Hz]	The description of the natural mode
1	5.09	the whole structure - rotation on the subsoil around the axis x
2	5.78	the whole structure - rotation on the subsoil around the axis y
3	8.23	the whole structure - rotation on the subsoil around the vertical axis z
4	9.84	the whole structure - translation in z direction and rotation around the axis y
5	10.77	rotation on the subsoil around the vertical axis z and around the axis x
6	10.99	translation in y direction and rotation around the axis y
7	22.09	bending of the roof and south wall
8	24.91	bending of the roof, south wall and north columns
9th to 60th mode: 30.22 Hz to 157.00 Hz ... higher vibration modes		

8. Forced vibration

The decomposition of dynamic load history to the natural modes of vibration is used for the forced vibration analysis by means of Scia Engineer program.

The damping of the structure of the building has been set as a logarithmic decrement 0.314, that corresponds to about a damping ratio of 5 %. For higher natural frequencies the damping is usually higher, but the computer program does not allow setting a different damping for higher frequencies.

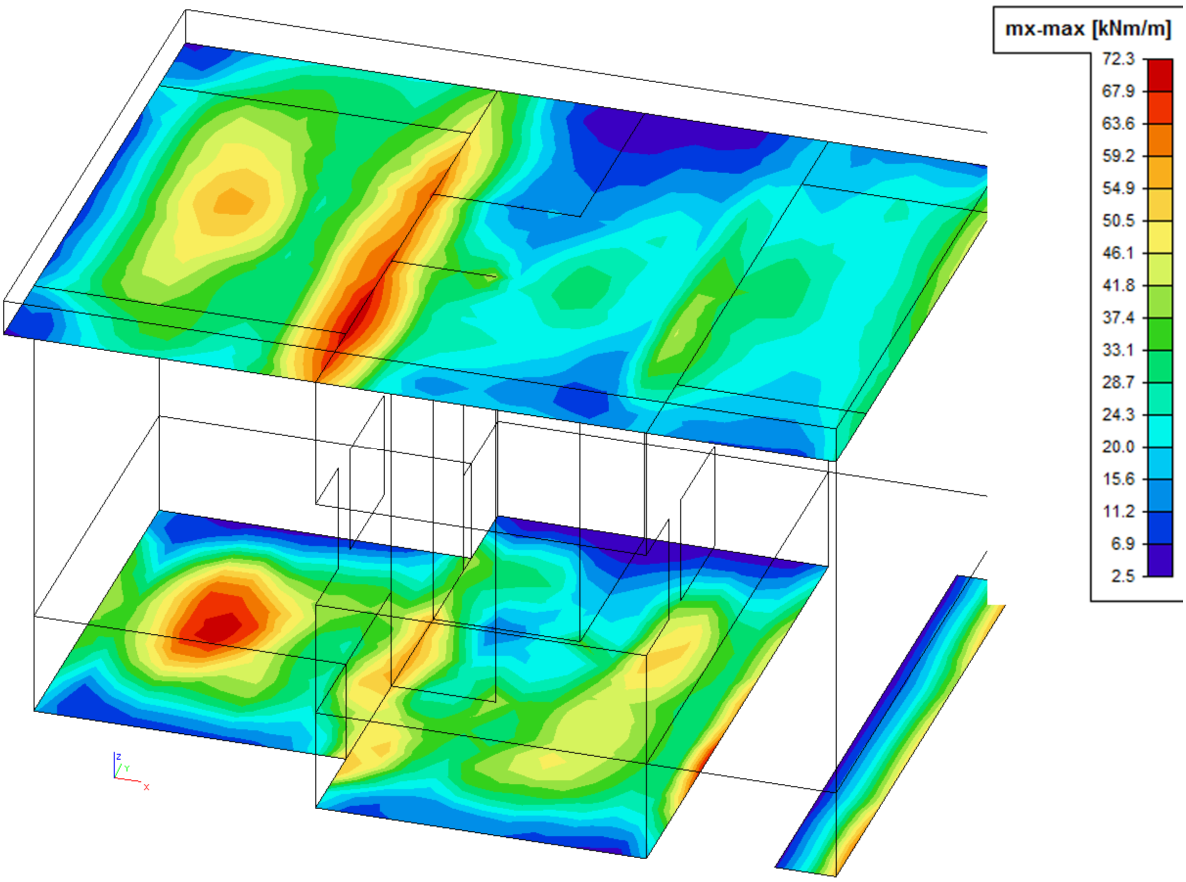

Fig. 3: Isolines of maximum values of dimension moments m_x

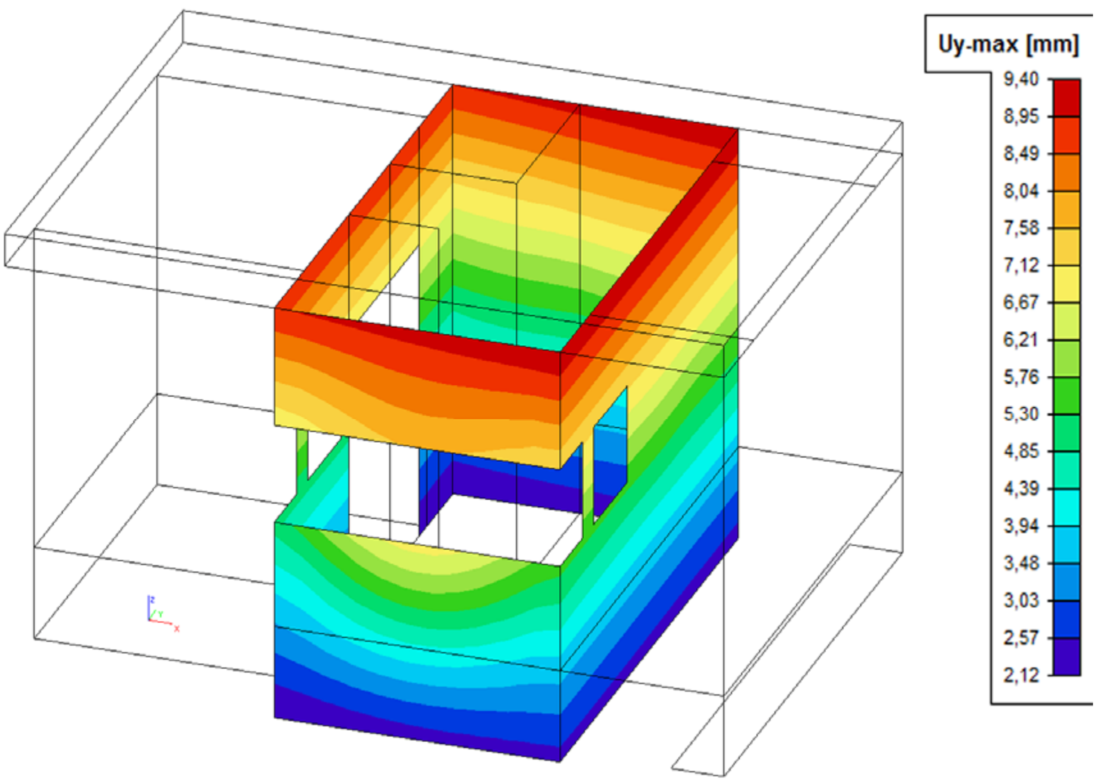

Fig. 4: Maximum dynamic displacement u_y in transversal direction y in the structure walls

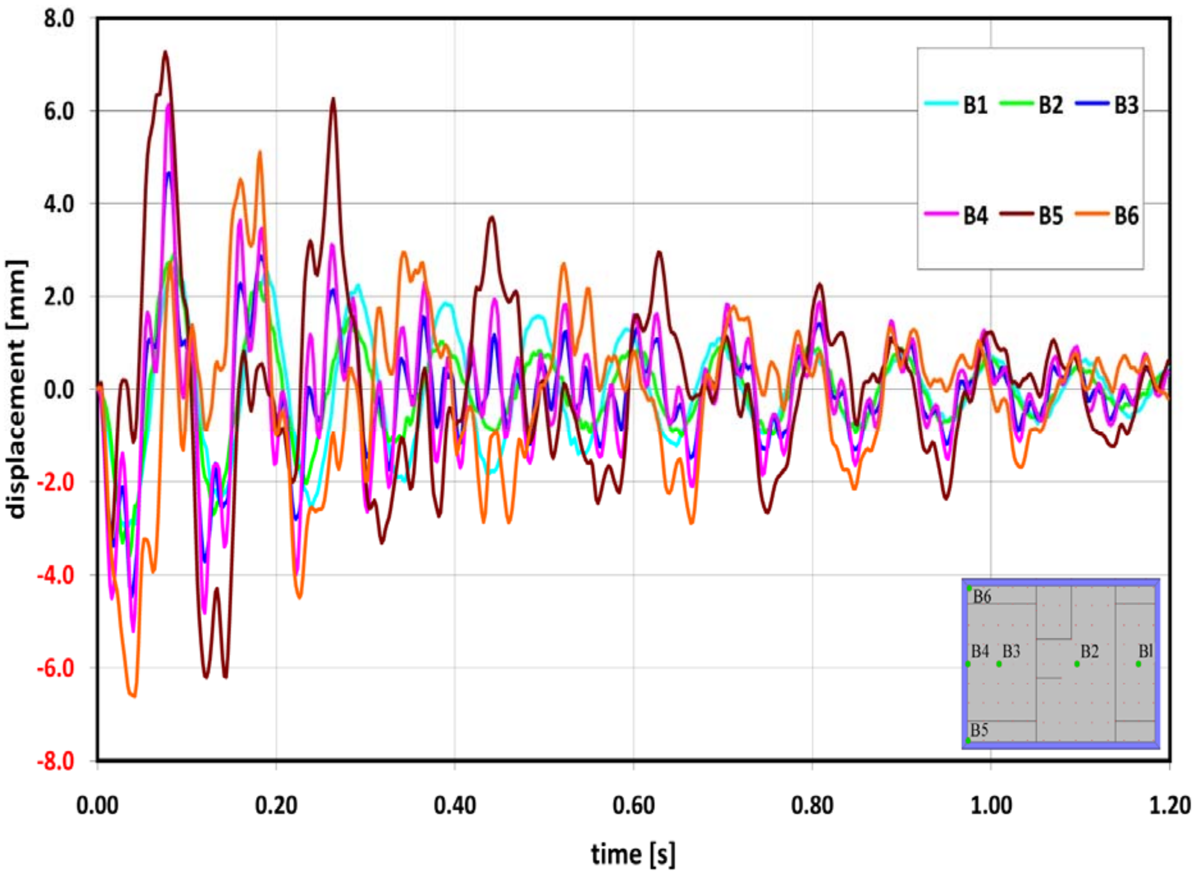

Fig. 5: Time history of vibration in selected points of floor plate

The calculation of forced vibration has been made with 300 time steps of 0.001 s and alternatively with 600 time steps of 0.002 s. The dynamic response is calculated respectively for each time step. The dynamic analysis was made for linear elastic behavior of the structure material. The dissipative properties of the structure can be respected during the results evaluation. The dynamic load or the calculated internal forces can be reduced by the means of ductility factor conservatively for floor plates, walls and beams under floor plates by value 2.5 and for columns by value 1.5.

As an example of the characteristics of internal forces, isolines of maximum values of dimension moments m_x in the ceiling and foundation slabs of the building are shown in Fig. 3. Fig. 4 presents maximum values of rotations in walls, and Fig. 5 indicates time histories in displacements at selected points B1 to B6 in the ceiling slab.

The calculated rotations (angles ψ) of the central line of structural elements is used for structure assessment. Fig. 6a illustrates partial rotation of central parts of the structure walls around the local axis z, which is perpendicular to the central plane of the concerned wall. The Fig. 6b shows the rotation of the same part of the structure walls around the local axis x, which is in horizontal direction in the central plane of the concerned wall

The maximal angle in the west wall is 0.24 degrees round vertical axis z and 0.17 degrees round horizontal axis x. In the north and south walls the maximal angle is 0.11 degrees round vertical axis z. Maximal rotation of the roof plate is 0.18 degrees round horizontal axis x.

From all the rotations it is clear that the concrete values are smaller than the limit value 4 degrees. The concrete structure is therefore safe enough and responds to the structure hazard smaller then the moderate hazard.

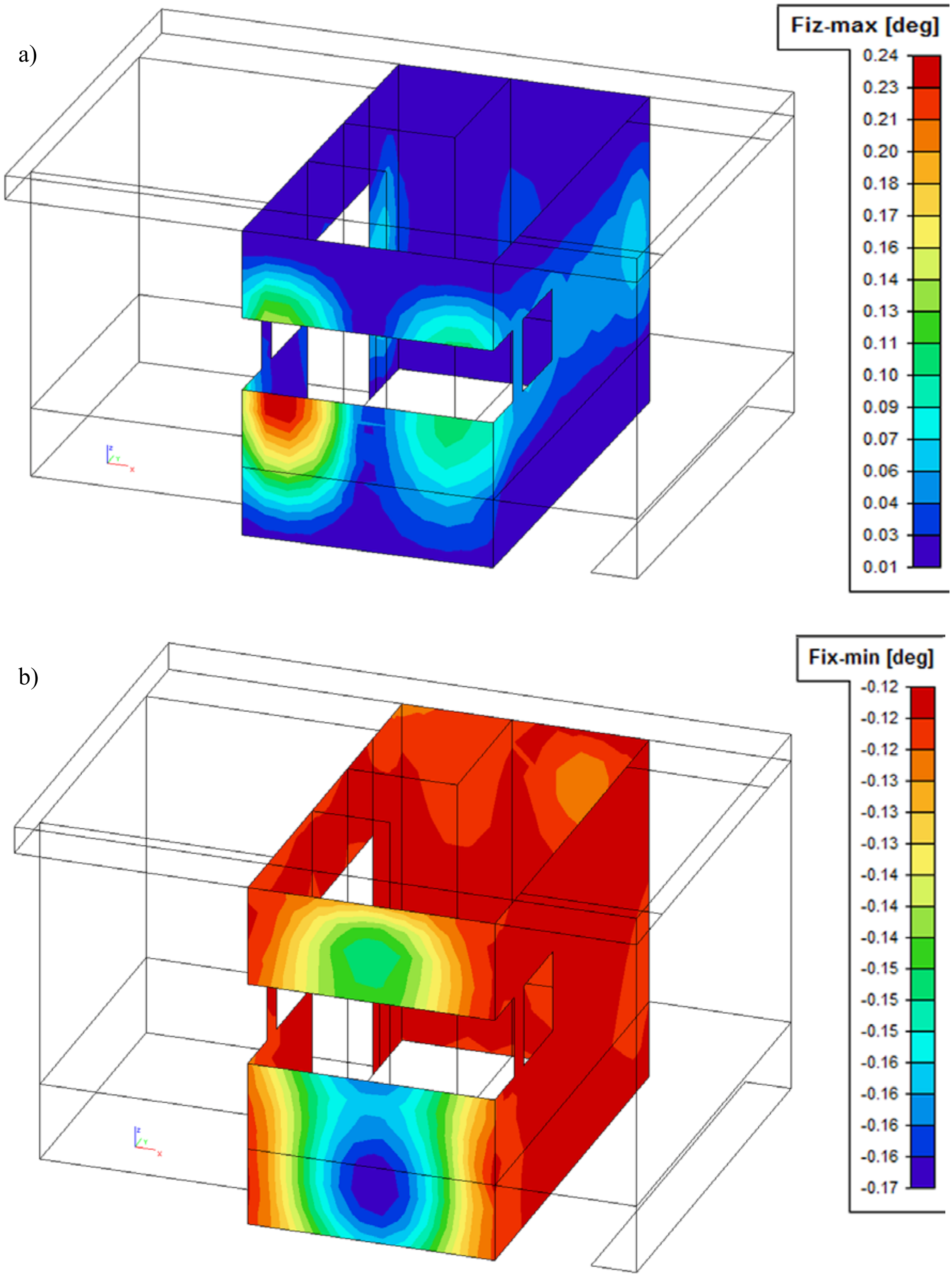

Fig. 6: Isolines of rotations; a)rotation round local axis z, b) rotation round local axis x

9. Conclusion

The example of a specific building was used to address the problem of explosion and threatened safety of the building upon an explosion of a rather large terrorist charge located in a car and detonated near the building, on circumferential roads.

Considering uncertainties associated with the determining of all parameters of the explosion load, methodology derived by the authors based on an analysis of experimentally determined explosion load parameters (Makovička & Makovička, 2011) used for an engineering estimate of the probable load. This methodology can be used to determine such parameters with sufficient accuracy and to assess the building structure in respect of the parameters.

The structure response was assessed based on the results of a 3D dynamic computation according to the magnitude of internal forces and displacements, and partial rotation of the central line of beam or slab cross-sections of the structure. Currently, structure assessment methodology based on partial rotation of the cross-sections has been in the process of development; it corresponds to the most recent research trends. The authors used limit values determined experimentally upon explosion load of brick-layered, reinforced concrete and window glass boards based on comparing their own (Makovička, 1998, 1999 and Makovička & Janovský, 2008) and other published results (Henrych, 1979 and MC Cann & Smith, 2007).

Acknowledgement

This research was supported as a part of the research projects in GAČR P105/11/1580 Transient response of structure under short-term dynamic or impact load due to seismic effects and explosions, for which the authors would like to thank the Agency.

References

ČSN EN 1998-1 (2006) *Eurocode 8, Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*, ČNI Praha.

Design of blast resistant building in petrochemical facilities (1997), American Society of Civil Engineers: Reston, VA, 1997.

Henrych, J. (1979) *The dynamics of explosion and its use*, Academia Prague.

Kloušek,V. a kol. (1967) *Stavebné konštrukcie namáhané dynamickými účinkami*. SVTL Bratislava.

Makovička, D. (1998) Shock wave load of window glass plate structure and hypothesis of its failure. In: *Structures Under Shock and Impact '98. Computational Mechanics Publications*, WIT Press Southampton, pp. 43-52.

Makovička, D. (1999) Failure of masonry under impact load generated by an explosion. *Acta Polytechnica*, Vol. 39, No. 1, pp. 63-91.

Makovička, D. & Janovský, B. (2008) *Příručka protivýbuchové ochrany staveb*, Česká technika – nakladatelství ČVUT v Praze.

Makovička, D., Makovička, D., Janovský, B. & Adamík, V. (2009) Ohrožení konstrukce budovy při výbuchu nálože ve vnitřním prostoru, *Stavební obzor*, č. 9, roč. 18, s. 257-265.

Makovička, D., Makovička, D. (2010) Simplified evaluation of a building impacted by a terrorist explosion, In: Jones, N., Brebbia, C.A. (Eds.): *Structures Under Shock and Impact XI*, WIT PressSouthampton, pp. 93-104,

Makovička, D. & Makovička, D. (2011) Analýza budovy, zatížené venkovním výbuchem, Spektrum, roč. 11, č. 2, s.17-20.

Makovička, D. & Makovička, D. (2012) Blast resistant design and limits of the response of a structure to an external explosion, In: Schleyer, G. & Brebbia, C.A (Eds.): *Structures Under Shock and Impact XII*, WIT Press Southampton, pp. 229-239.

McCann, D.M. & Smith, S.J. (2007) Blast resistant design of reinforced concrete structures, *STRUCTURE magazine*, pp. 22-26.

TM 5-1300 (1990) *Structures to resist the effect of accidental explosions*, U. S. Army Corps of Engineers: Washington, D.C., (Navy NAVFAC P-397 or Air Force AFR 88-22).