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NUMERICAL SIMULATION OF THE STRESS-STRAIN STATE OF A CONCRETE TUNNEL LINING UNDER SURFACE BLAST LOADING USING THE RHT MODEL

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ЧИСЕЛЬНЕ МОДЕЛЮВАННЯ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ БЕТОННОЇ ОБРОБКИ ТУНЕЛЮ ПРИ ПОВЕРХНЕВОМУ ВИБУХОВОМУ НАВАНТАЖЕННІ З ВИКОРИСТАННЯМ RHT-МОДЕЛІ

Purpose. Calculation of the stress-strain state of the concrete lining of a tunnel located at a depth of 10 m in a soil mass under surface blast loading, and substantiation of a methodology for the correct interpretation of the results taking into account dynamic effects.

Methodology. The calculation is performed using the ANSYS Explicit Dynamics software package with a fully coupled Eulerian–Lagrangian formulation. The behaviour of concrete C35/45 is described by the dynamic Riedel–Hiermaier–Thoma (RHT) model, and the behaviour of the soil is described by the Drucker–Prager Piecewise model. The influence of the strain rate on the strength of concrete is taken into account through the dynamic increase factor (DIF), defined as the ratio of the dynamic strength to the static strength.

Findings. The maximum principal stress σ_1 in the concrete lining amounts to the following values, respectively: the peak level at the monitoring point – 3.82 MPa at the moment t = 43.5 ms; the spatial maximum – 3.57 MPa at the moment t = 50 ms. The maximum soil displacement at the epicentre of the explosion is 4.21 m, and at the level of the tunnel – 0.2–0.5 m. The dynamic tensile strength of concrete is $f_{t,dyn} \approx 6.27$ MPa, and the safety margin is about 76%.

Originality. Relationships between the peak stress level in the concrete tunnel lining and the time required for the shock wave to pass through the soil mass have been established; a two-stage scheme for strength assessment has been substantiated – comparison of the calculated stresses with the static strength, and then with the dynamic strength taking into account the actual strain rate $\dot{\varepsilon} \approx 10^2$ s^{-1}. An RHT-model parameter f_t/f_c = 0.171 is proposed for a homogenised model of reinforced concrete C35/45 with reinforcement ratio ρ = 0.5% using class A500C steel.

Practical value. The obtained calculation data make it possible to assess the residual service life of tunnel linings under surface blast loading without the need to switch to computationally expensive discrete modelling of the reinforcement, as well as to formulate substantiated requirements for the thickness of the soil overburden and the concrete class for protective underground structures.

Keywords: *physical and mechanical properties, stability safety factor, RHT model, ANSYS Explicit Dynamics, surface explosion, tunnel lining, dynamic increase factor, reinforced concrete.*

Introduction. Protecting underground structures – in particular metro tunnels, hydraulic workings, utility tunnels, and mine openings – against short-duration blast loads is one of the pressing problems of modern geomechanics and structural mechanics. Such loads may originate from man-made accidents or technological blasting in the mining industry, as well as from deliberate explosive actions associated with military or terrorist threats. Under the conditions of the full-scale war on the territory of Ukraine, the problem of ensuring the stability of underground structures has acquired particular significance owing to the massive use of ballistic and cruise missiles, strike unmanned aerial vehicles, and other precision-guided weapons in the vicinity of critical infrastructure facilities. Under such circumstances, there is a need for rapid assessment of the load-bearing capacity and safety of underground structures subjected to surface explosions of varying intensity.

For the investigation of such processes, numerical methods based on the use of hydrocodes – in particular the Explicit Dynamics module of the ANSYS software package – have become widely adopted. These approaches make it possible to solve highly nonlinear transient problems of continuum mechanics, accounting for shock-wave propagation and the interaction of the "explosive – soil – concrete" system in a fully coupled formulation. Particular attention is paid to the selection of a concrete material model capable of correctly reproducing damage accumulation, the strain-rate dependence of strength, and structural failure under a complex stress-strain state.

Analysis of previous research. Among the concrete models implemented in modern commercial hydrocodes, one of the most widely used for dynamic and blast loading problems is the Riedel–Hiermaier–Thoma (RHT) model [1, 2]. Its application makes it possible to account for the complex nonlinear behaviour of concrete under high-rate loading, including damage accumulation, strength degradation, and strain-rate effects. The model is based on three limit-state surfaces: the failure surface Y_{fail} (which defines the maximum equivalent stress that the intact material can sustain and is specified as a function of pressure, the azimuthal angle in the deviatoric plane (the Lode angle), and the strain rate), the elastic limit surface Y_{el} (which bounds the region of elastic behaviour of the material and is defined by scaling the failure surface through the ratio of the elastic limit to the strength), and the residual strength surface Y_{res} (which describes the load-bearing capacity of the fully failed (cracked) material that retains compressive resistance through friction between fragments but loses resistance), together with a p–α equation of state, which accounts for the initial porosity of the material and its compaction under shock-wave action. The features of the RHT model implementation in LS-DYNA and the results of its verification are presented in [3, 4], whereas the determination of RHT model parameters for concretes of various classes is examined in detail in [5, 6].

One of the key factors governing the behaviour of concrete under blast loading is the dependence of its strength on the strain rate. To describe this effect, the dynamic increase factor (DIF) is used. Analytical relationships and generalisations of experimental data for compression and tension are presented in [7]. According to the results of [8], at strain rates on the order of 10^2–10^3 s^{-1} the DIF value for concrete in compression may exceed 2, while in tension it may reach 6, which substantially affects the assessment of structural stability under short-duration impulsive loading.

The response of underground structures to surface explosions has been considered in a number of studies. In particular, box tunnels were analysed in [9–13]; the influence of the type of soil medium was studied in [12]; the performance of tunnels with different cross-sectional shapes was compared in [13]; and segmental tunnel linings accounting for longitudinal joints were investigated in [17]. A general overview of the effects of blast loading on building structures is presented in [18]. The fundamentals of the mechanics of crater formation in soil and the regularities of shock-wave propagation are based on the classical works [19–21].

The nature of blast-wave propagation in an underground medium depends largely on the physical and mechanical properties of the soil or rock mass. In highly compressible soils, such as sandy loams and sandy clay loams, a significant portion of the blast energy is expended on internal friction between grains, the destruction of structural bonds, and the compaction of the pore space. As a result, a relatively wide blast crater with gentle slopes is formed, and the wave amplitude decreases rapidly with depth.

Clayey soils are characterised by greater stiffness and lower compressibility; therefore, blast craters have a smaller radius but a greater depth. Under such conditions, the soil medium absorbs a smaller portion of the energy, as a result of which a considerable impulse is transmitted directly to the tunnel lining.

In monolithic rock, structural deformations usually remain insignificant owing to the high stiffness of the mass. At the same time, such rocks transmit elastic waves practically without significant energy losses, which may cause spalling of the concrete or rock from the inner surface of the tunnel. Comparative characteristics of the behaviour of different types of geomaterials under dynamic loading are presented in Table 1.

Table 1

Behaviour of geomaterials under surface blast loading

Type of geomaterial	Longitudinal wave velocity c, m/s	Surface blast crater parameters	Energy dissipation	Main risk to the tunnel
Dry sandy clay loam	200–400	Wide crater with a large radius, gentle slopes	Very high	Uneven settlement of the soil above the tunnel
Water-bearing sandy loam	1000–1500	Medium radius, signs of liquefaction	Moderate	Crown deflection, soil washout through joints
Stiff plastic clay	500–800	Narrow, deep crater with steep slopes	Low	Bending moments in the walls, shearing of bolted connections
Water-saturated clay	1400–1600	Deep crater, slumping of walls	Minimal	Sudden brittle collapse of the crown
Weathered sandstone / granite	1500–2500	Localised slab-type failures	High	Localised rock fallouts from the crown
Monolithic granite	4500–5500	Surface flaking, fine crumbling	None	Spalling of the concrete

Despite the considerable body of research, the issues of interpreting simulation results obtained with the RHT model still remain insufficiently systematised. In particular, in most works the assessment of the stress state of structures is carried out by directly comparing peak stresses with the static strength of concrete, whereas the effect of dynamic strength enhancement is not fully taken into account. Insufficient attention has also been paid to practical approaches for the transition from a plain-concrete model to an equivalent homogenised reinforced-concrete model, which is particularly important for the analysis of typical tunnel linings in blast loading problems.

Statement of the unsolved part of the problem and the objective. The objective of this work is the numerical simulation of the response of the concrete lining of a tunnel located at a depth of 10 m to surface blast loading, using the finite element method and the RHT concrete model; the justification of the choice of model parameters for concrete of class C35/45; and the analysis of the influence of the blast impulse on the stress-strain state of the structure. Based on the simulation results, the work aims to formulate recommendations for the correct interpretation of stress fields, taking into account the strain-rate effect, the dynamic increase factor, and the transition from a plain-concrete model to an equivalent homogenised reinforced-concrete model.

Main part (Results). The numerical simulation was performed in the ANSYS Explicit Dynamics software package using a fully coupled Eulerian–Lagrangian formulation, which makes it possible to account for the interaction between the blast wave, the soil mass, and the concrete structure. The object of the study was a cylindrical concrete tunnel lining located in a homogeneous sandy mass at a depth of H = 10 m. The blast loading was specified as a surface point charge (surface burst). The duration of the computational process was $5 \cdot 10^{-2}$ s (50 ms) after the moment of blast initiation.

To describe the behaviour of the soil medium, the DP4 (Drucker–Prager Piecewise) model from the ANSYS Explicit material library was used. The model parameters were adopted on the basis of the results of experimental studies of sandy soils presented in [21]. The main physical and mechanical characteristics of the soil mass used in the calculation are given in Table 2.

Table 2

Parameters of the DP4 soil model (Drucker–Prager Piecewise)

№	Parameter	Value	Unit
1	Density	1750	kg/m³
2	Shear Modulus	$2{,}3 \cdot 10^{7}$	Pa (23 MPa)
3	Strength	Drucker–Prager Piecewise	—
4	Maximum Tensile Pressure	$-1{,}0 \cdot 10^{4}$	Pa (–0.01 MPa)
5	Equation of State	Compaction (linear)	—
6	Solid grain density	2641	kg/m³

The behaviour of the concrete lining was described using the RHT model [1–4], which is widely applied for the analysis of dynamic and blast loading on concrete structures. The basic set of model parameters for concrete of class C35/45 is given in Table 3. The adopted values of density 2314 kg/m³ and specific heat capacity 654 J/(kg·°C) correspond to typical characteristics of heavy structural concrete. The compressive strength f_c = 35 MPa is consistent with the requirements for concrete of class C35/45 according to DSTU B V.2.7-176:2008 [20] and is typical for concretes used in the construction of tunnel linings and protective underground structures.

Table 3

Main parameters of the RHT model for concrete C35/45

No.	Parameter	Value	Unit
1	Density	2314	kg/m³
2	Specific heat	654	J/(kg·°C)
3	Compressive strength, f_c	$3.5\cdot10^7$	Pa (35 MPa)
4	Tensile-to-compressive strength ratio, f_t/f_c	0.10	—
5	Compressive strain-rate exponent, α	0.032	—
6	Tensile strain-rate exponent, δ	0.036	—

The initial ratio f_t/f_c = 0,10 corresponds to a concrete tensile strength of approximately 3.5 MPa, which is a typical value for ordinary structural concrete [11, 12]. The parameters α = 0.032 and δ = 0.036 characterise the effect of strain rate on the strength gain of concrete in compression and tension, respectively. Their values are adopted in accordance with the recommendations of the model [11], which is widely used to describe the dynamic behaviour of concrete materials under high-rate loading. The dynamic strength is calculated using the generally accepted relationship

$$\frac{f_{t,dyn}}{f_t} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{\delta} \tag{1}$$

where $\dot{\varepsilon}_0$ – is the reference (static) strain rate; $\dot{\varepsilon}$ – is the current strain rate in the concrete [7, 11].

The object of the simulation was a reinforced-concrete arch-type tunnel lining located in the soil mass at a depth of 10 m from the ground surface to the crown of the arch. The computational scheme reproduced the combined effect caused by a missile impact, accompanied by both kinetic action and blast loading, with the formation of a blast crater and the propagation of a wave disturbance through the soil mass above the structure.

The assessment of the stress-strain state of the structure was carried out on the basis of the maximum principal stresses σ_1, which is one of the key criteria for analysing the behaviour of brittle materials, in particular concrete. Positive values of this parameter correspond to tensile stresses and make it possible to identify zones of potential crack formation.

The field of total soil displacements, shown in Fig. 1, reflects the nature of the deformations arising as a result of the surface explosion. The maximum displacements are observed at the epicentre of the explosion at the ground surface and reach 4.21 m, which corresponds to the formation of a classic blast crater. The zone of intense displacements (>1.5 m) is limited to a radius of approximately 3–4 m from the centre of the explosion.

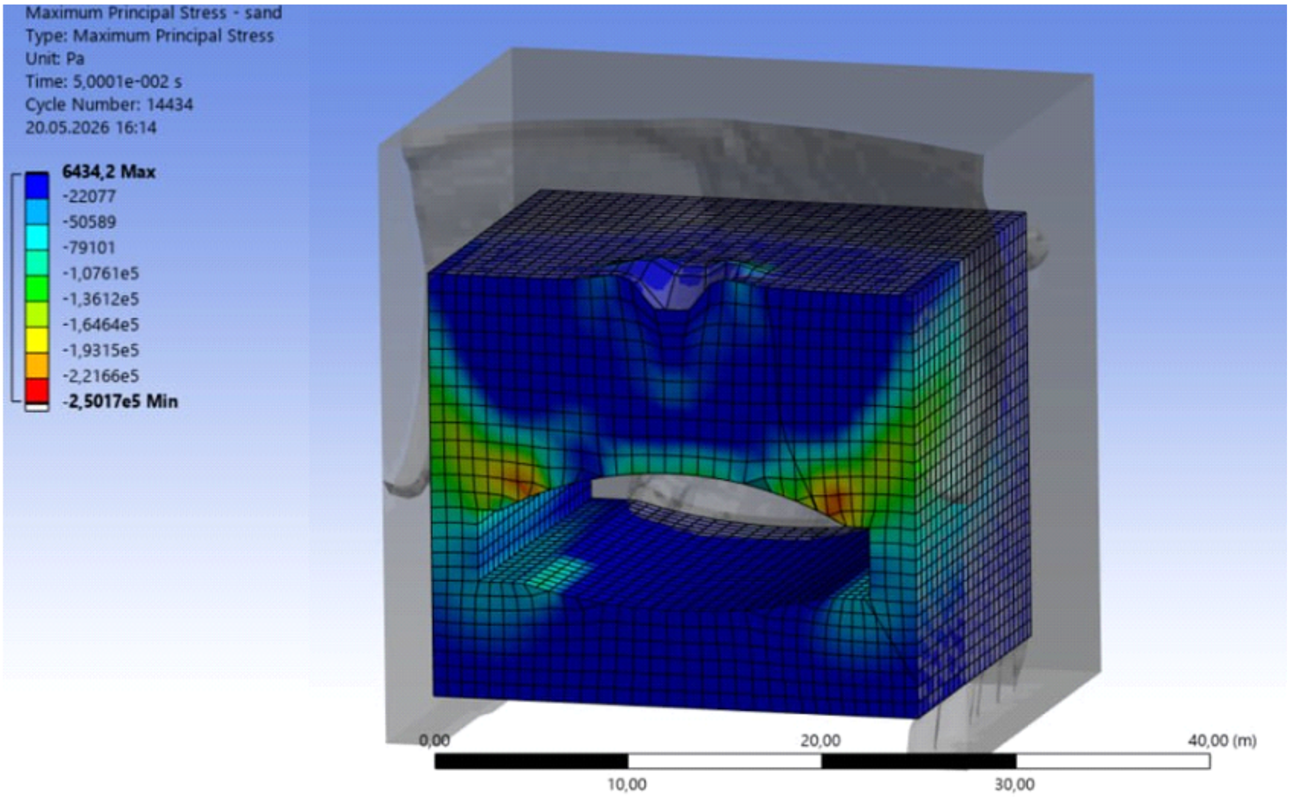

Fig. 1. Field of total displacements of the soil mass at the moment $t = 5\cdot10^{-2}$ s (Total Deformation module, ANSYS Explicit Dynamics); the maximum displacement of 4.21 m is localised at the epicentre of the explosion

The resulting crater has a radius of approximately 3–4 m and a depth of 2–3 m, with a ratio *r/h* ≈ 1.3–1.5, which is typical for the detonation of a charge of approximately 500 kg in soils with similar characteristics [16, 17]. At the depth of the tunnel (10 m), the displacements are substantially smaller – about 0.2–0.5 m, which corresponds predominantly to elastic deformations of the soil under the action of the compression wave.

Analysis of the displacement distribution shows that the zone of the most hazardous deformations is confined to the upper part of the mass and effectively does not extend to the level at which the underground structure is located. This indicates a substantial attenuation of the blast-wave energy as it propagates deeper into the soil and confirms the protective role of the soil layer above the tunnel.

Analysis of the time history of the extreme values of σ_1 in the structural elements of the tunnel showed that the development of the wave process has a clearly pronounced stage-by-stage character (Fig. 2). At the initial stage, in the time interval from 0 to approximately 25 ms, a relatively low level of stresses is recorded in the structure. This is explained by the fact that the shock-wave front requires a certain time to pass through the ten-metre layer of soil. In this case, the soil mass acts as a natural damping medium, which partially absorbs the energy of the explosion and reduces the intensity of the impulse transmitted to the tunnel lining.

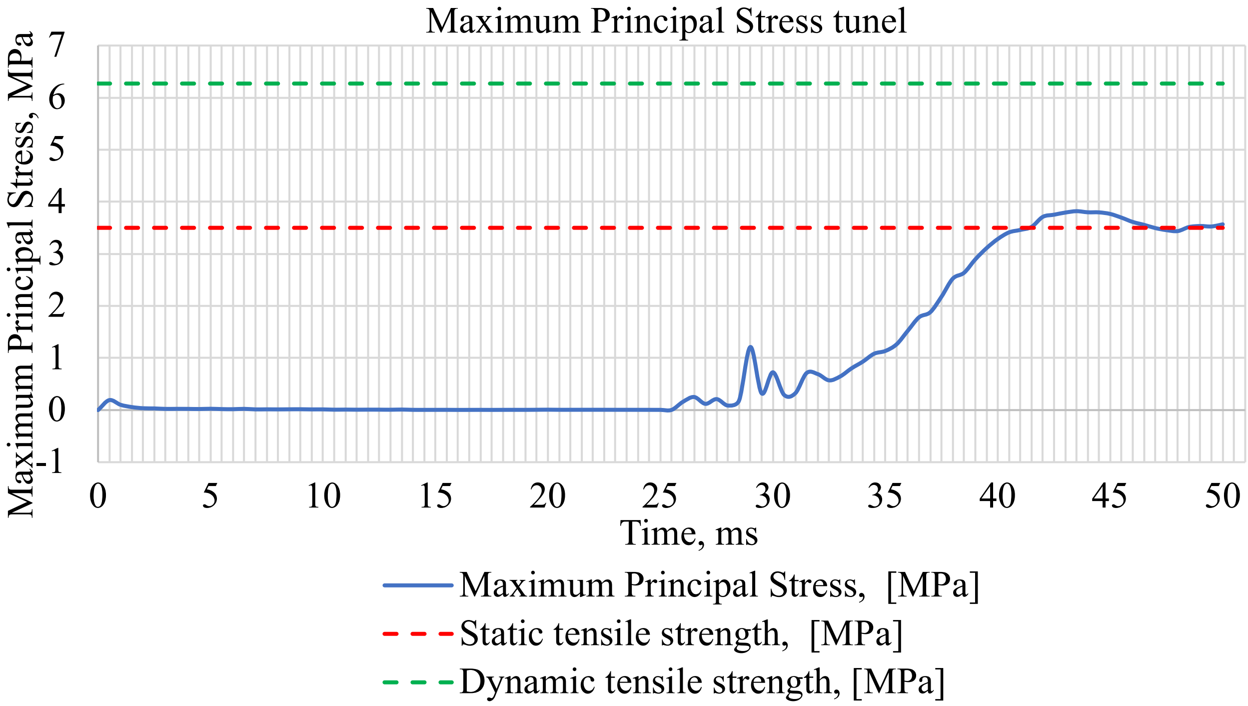

Рис. 2. Time dependence of the maximum principal stress σ_1 at the monitor point of concrete treatment; dashed lines are static and dynamic (taking into account DIF) tensile strength

The further development of the process is characterised by a sharp increase in stresses in the structure. In the time interval of approximately 25–43 ms, the active propagation of the compression wave and its transformation into a tensile wave following interaction with the reinforced-concrete lining take place. The maximum value of the principal stresses at the monitoring point, $\sigma_1 \approx 3.82$ MPa, was recorded at approximately $t \approx 43.5$ ms after the onset of loading. This moment corresponds to the most hazardous phase of the dynamic action.

After the peak load has passed, a gradual decrease in the stress level is observed, which indicates the onset of energy dissipation in the structure and the soil mass.

Spatial analysis of the stress distribution showed (Fig. 3) that the most hazardous zones are localised in the haunch regions of the arch and in the invert area of the tunnel. It is precisely here that the maximum tensile stresses are formed, $\sigma_1 \approx 3.57$ MPa, the magnitude of which exceeds the design tensile strength of structural concrete (2.9–3.5 MPa). In the upper part of the crown and on the outer contour of the arch, the stress level proved to be lower and predominantly corresponded to the elastic-plastic regime of the material behaviour. At the same time, in the crown (key) region of the tunnel, compressive stresses prevail ($\sigma_{1,\,min} \approx -0.26$ MPa), which confirms the effectiveness of the arch geometry, which ensures the redistribution of the vertical impact load into compressive forces along the contour of the lining.

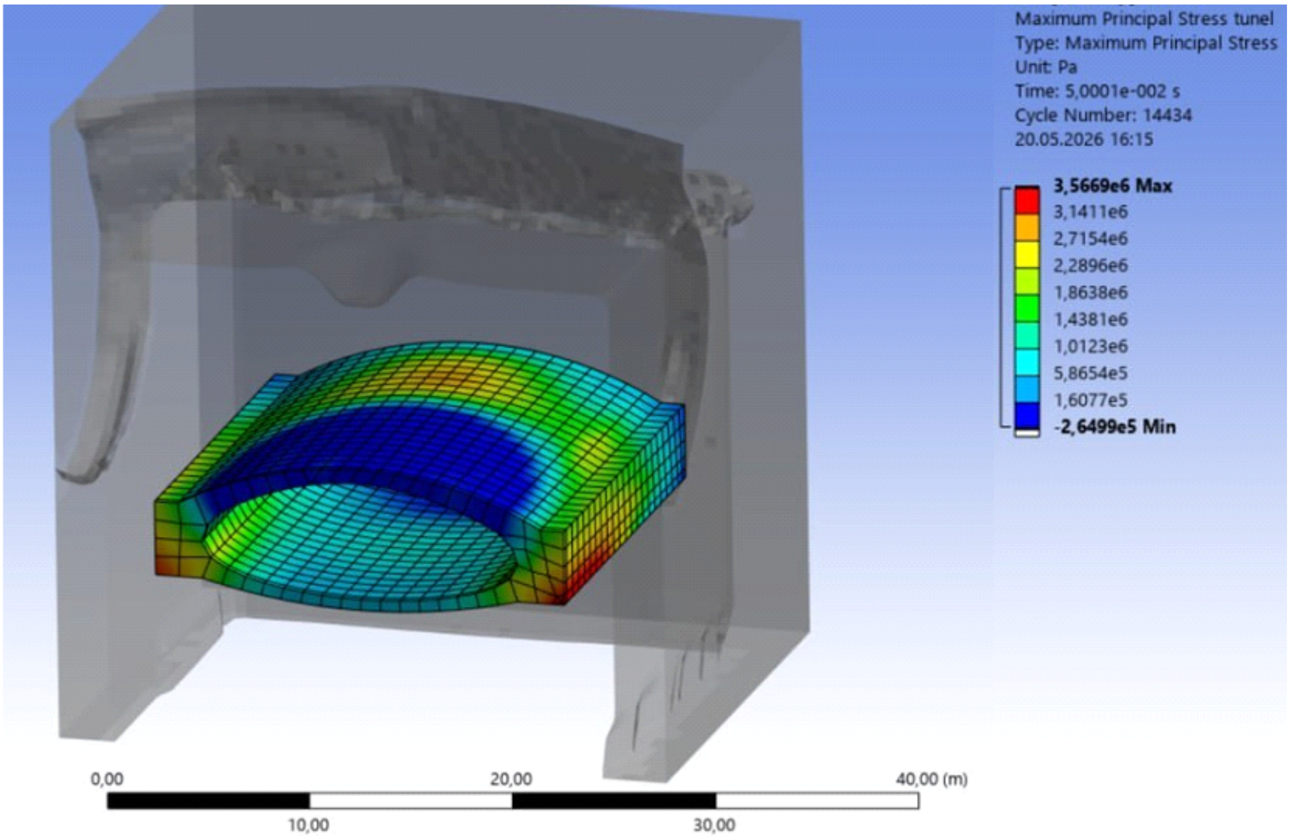

Fig. 3. Field of the maximum principal stress σ_1 in the concrete tunnel lining at the moment $t = 5 \cdot 10^{-2}$ s; scale from $\sigma_{1,min} = -264.5$ kPa (blue, compression) to $\sigma_{1,max} = +3.57$ MPa (red, tension)

The major part of the tunnel lining operates within the range of principal stresses $\sigma_1 = +1.0…+3.0$ MPa. The difference between the peak value at the monitoring point (3.82 MPa at $t = 43.5$ ms, Fig. 2) and the maximum value recorded in the spatial stress field at the end of the calculation (3.57 MPa at $t = 50$ ms, Fig. 3) is explained by the fact that the time history reflects the variation of stresses at a specific point of the model throughout the entire loading process, whereas the 3D field characterises the instantaneous stress distribution over the whole volume of the structure at a given moment in time.

A formal comparison of the obtained stresses with the static tensile strength of concrete shows that the maximum value of the principal stresses $\sigma_{1,max} = 3.57$ MPa slightly exceeds the adopted strength limit $f_t = 3.5$ MPa. From the standpoint of the classical static approach, this would indicate the onset of crack formation and damage accumulation in the material. Within the framework of the RHT model, such a situation corresponds to the transition of the concrete to the stage of local damage with possible formation of macrocracks in the most heavily loaded zones.

However, for blast loading it is necessary to take into account the influence of the strain rate on the mechanical characteristics of concrete. During short-duration dynamic action, the strain rate may reach 10^2–10^4 s^{-1}, whereas under static testing it is about 10^{-5} s^{-1} [7, 10]. The model used the following strain-rate parameters: for compression $\alpha = 0.032$ and for tension $\delta = 0.036$, which characterise the strength increase of the material under high-rate loading. Under such conditions, concrete exhibits a substantial increase in strength, which is taken into account through the dynamic increase factor (DIF), described by the equation:

$$DIF_t = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{\delta} \tag{2}$$

where $\dot{\varepsilon}_0 = 10^{-5}$ is the static strain rate, s^{-1}; $\dot{\varepsilon}$ is the current strain rate, s^{-1}; $\delta = 0.036$ is the strength enhancement coefficient under tension.

For the conditions of a surface explosion at a tunnel depth of 10 m, the strain rate in the concrete lining is estimated at approximately $\dot{\varepsilon} \approx 10^2$ s^{-1}. Based on this,

$$DIF_t = \left(\frac{10^2}{10^{-5}} \right)^{0.036} = \left(10^7\right)^{0.036} \approx 1.8$$

Thus, the dynamic tensile strength of concrete increases by approximately 1.8 times compared with the static value and amounts to $f_t^{dyn} = 1.8 \cdot 3.5 = 6.27$ MPa. Based on this, the calculated peak stress $\sigma_{1,max} = 3.57$ MPa remains substantially below the dynamic strength of the concrete (see fig. 2).

Thus, despite the formal exceedance of the static strength, under the conditions of real blast loading the concrete lining does not reach a critical state of failure. The material operates in a nonlinear regime with the development of local damage, yet it retains a sufficient safety margin (≈76%) and overall structural stability.

The obtained result is of fundamental importance for assessing the serviceability of the structure. Although, under a static approach, the recorded stress level could be regarded as critical, taking into account the dynamic strength enhancement it turns out to be approximately 43% below the actual dynamic strength of the concrete. A similar effect of the increase in concrete resistance under high-rate loading is reported in [7, 9] and is consistent with the provisions of [10].

At the same time, the limitations of the adopted numerical model should be taken into account. In the calculation, the tunnel lining was considered as plain concrete, without accounting for the influence of the reinforcement cage. Under real conditions, underground structures are usually made of shotcrete or precast reinforced-concrete elements, in which the reinforcement carries the major part of the tensile forces. Reinforced concrete is a composite material in which the concrete works effectively in compression, while the steel reinforcement provides resistance to tension. It is precisely the combined action of these components that determines the actual load-bearing capacity of the structure.

For the modelling of reinforced-concrete elements in dynamic problems, two main approaches are used. The first involves discrete modelling, in which the concrete and the reinforcement are described by separate finite elements. This method makes it possible to reproduce the interaction of the materials in detail; however, it requires substantial computational resources. The second approach is based on the use of a homogenised model, in which the concrete and the reinforcement are treated as a single equivalent material with averaged mechanical characteristics [13, 17].

For an equivalent model of reinforced concrete of class C35/45 with a reinforcement ratio $\rho = 0.5\%$, which is characteristic of tunnel linings and hydraulic structures, and with reinforcement of class A500C ($f_y = 500$ MPa in accordance with DSTU 3760:2019 [20]), the contribution of the reinforcement to the overall tensile strength can be estimated as $\rho \cdot f_y = 0.005 \cdot 500 = 2.5$ MPa. In this case, the effective tensile strength of the equivalent reinforced concrete amounts to: $f_t^{eff} = f_t + \rho \cdot f_y = 3.5 + 0.005 \cdot 500 = 6.0 МПа$, which substantially exceeds the corresponding value for plain concrete.

In Table 3, the parameter Tensile Strength f_t/f_c is taken equal to 0.1; to obtain equivalent reinforced concrete of class C35/45 with a reinforcement ratio $\rho = 0.5\%$:

$$\frac{f_t^{eff}}{f_c} = \frac{6.0}{35} = 0.171 .$$

By substituting this value into the corresponding parameter of the RHT model, an approximate equivalent reinforced-concrete model can be formed without the need to switch to discrete modelling of the reinforcement cage, which substantially simplifies the calculation and reduces the computational cost.

The obtained results are in good agreement with the data of works [8–11], in which it is noted that a soil overburden with a thickness exceeding 8–10 m above a tunnel ensures a substantial reduction of peak loads from surface explosions of moderate intensity. In addition, the calculated parameters of the blast crater – a radius of approximately 3–4 m and a depth of 2–3 m in sandy soil – correspond to the classification of surface explosions [18] and are consistent with the empirical data given in [16, 18].

In contrast to the studies [9, 10], in which the concrete lining was considered mainly in the form of structures with a rectangular cross-section, the present work investigates a tunnel lining of cylindrical shape, characteristic of structures constructed by the shield or drill-and-blast method. Such a geometry is more typical for real underground transport facilities and better corresponds to the conditions of current geotechnical practice in Ukraine.

The proposed approach to the assessment of the stress state is based on a two-stage analysis: first, the obtained stresses are compared with the static strength of the concrete, and then with the dynamic strength, determined taking into account the dynamic increase factor calculated for the actual strain rate in the model. Such a scheme makes it possible to avoid the common error associated with overestimating the level of concrete damage when results are compared directly only with the static tensile strength.

Conclusions. As a result of numerical simulation performed in the ANSYS Explicit Dynamics environment using the RHT concrete model, it was established that, under a surface explosion above a tunnel located at a depth of 10 m, the maximum principal stress in the concrete lining reaches +3.57 MPa. In doing so, the character of the stress state of the structure is determined predominantly by local tensile zones in the support (haunch) regions of the lining, whereas the upper part of the arch crown works mainly in compression.

Taking into account the strain-rate effect and the dynamic increase factor, the dynamic tensile strength of concrete of class C35/45 amounts to approximately 6.27 MPa. Thus, the calculated stresses remain substantially below the dynamic strength, and the safety margin of the structure is about 76%. This indicates the absence of critical failure of the concrete lining under the blast loading considered.

The analysis carried out has shown that the use of the static strength criterion alone for the interpretation of dynamic simulation results is incorrect, as it leads to a substantial overestimation of the predicted level of structural damage. For an adequate assessment of the performance of concrete under blast loading, the inclusion of the dynamic increase factor is mandatory.

It has also been established that, for the approximate modelling of equivalent reinforced concrete of class C35/45 with reinforcement (ρ = 0.5%) using class A500C steel, it is sufficient to adjust the f_t/f_c parameter in the RHT model from 0.10 to 0.171. This approach makes it possible to account for the effect of reinforcement without resorting to computationally expensive discrete modelling of the reinforcement cage.

The simulation results have confirmed the effectiveness of the 10-metre soil overburden in reducing the dynamic effect of the surface explosion. The main zone of failure and significant soil deformations is confined to the upper layers of the mass at a depth of approximately 2–3 m and does not reach the level at which the tunnel is located.

Further research should be directed at analysing the influence of the explosive charge mass, the physical and mechanical characteristics of the soil, and the reinforcement ratio of the structure, as well as at verifying the obtained results against full-scale experimental data and the results of other studies.

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АНОТАЦІЯ

Мета. Розрахунок напружено-деформованого стану бетонного оправлення тунелю, розташованого на глибині 10 м у ґрунтовому масиві, при дії поверхневого вибухового навантаження та обґрунтування методики коректної інтерпретації результатів з урахуванням динамічних ефектів.

Методика. Розрахунок виконано у програмному комплексі ANSYS Explicit Dynamics із застосуванням повністю зв'язаної ейлерово-лагранжевої постановки. Поведінка бетону С35/45 описується динамічною моделлю Ріделя–Хірмаєра–Томи (RHT), а поведінка ґрунту – моделлю Drucker–Prager Piecewise. Вплив швидкості деформації на міцність бетону враховано через коефіцієнт динамічного зміцнення (DIF), який визначається як відношення динамічної міцності до статичної.

Результати. Максимальне головне напруження σ_1 у бетонному оправленні становить відповідно: піковий рівень у точці моніторингу – 3,82 МПа в момент часу $t = 43{,}5$ мс; просторовий максимум – 3,57 МПа в момент часу $t = 50$ мс. Максимальне переміщення ґрунту в епіцентрі вибуху становить 4,21 м, а на рівні тунелю – 0,2–0,5 м. Динамічна міцність бетону на розтяг становить $f_{t,dyn} \approx 6{,}27$ МПа, а запас міцності – близько 76 %.

Наукова новизна. Встановлено залежності між піковим рівнем напружень у бетонному оправленні тунелю та часом проходження ударної хвилі крізь ґрунтовий масив; обґрунтовано двоетапну схему оцінки міцності – порівняння розрахункових напружень зі статичною міцністю, а потім із динамічною міцністю з урахуванням фактичної швидкості деформації $\dot{\varepsilon} \approx 10^2$ с$^{-1}$. Запропоновано параметр RHT-моделі $ft/fc = 0{,}171$ для гомогенізованої моделі залізобетону C35/45 з коефіцієнтом армування $\rho = 0{,}5$ % сталлю класу A500C.

Практична значимість. Отримані розрахункові дані дозволяють оцінювати залишковий ресурс оправлень тунелів при поверхневому вибуховому навантаженні без необхідності переходу до обчислювально витратного дискретного моделювання арматури, а також формулювати обґрунтовані вимоги до товщини ґрунтового захисного шару та класу бетону для захисних підземних споруд.

Ключові слова: *фізико-механічні властивості, коефіцієнт запасу стійкості, RHT-модель, ANSYS Explicit Dynamics, поверхневий вибух, оправлення тунелю, коефіцієнт динамічного зміцнення, залізобетон.*

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