Numerical Simulation of the Dynamics of a Reinforced Concrete Slab under an Air Shock Wave

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Abstract—The work considers the deformation and fracture of a reinforced concrete slab under the effect of an air shock wave. The research involves data from the “Blind Blast Test” experimental case. The slab is loaded by detonating an explosive in a shock tube. The numerical and experimental results are compared quantitatively and qualitatively. Quantitative comparison is made for the history of movement of key points of the reinforced concrete slab during deformation. Qualitative comparison is made for photographs of the destruction of a real reinforced concrete slab and distribution of the damage fields obtained by calculation. The numerical simulation is carried out using LS-DYNA code and the finite element method with an explicit time integration scheme. The CSCM (Continuous Surface Cap Model) model is used to model the concrete material. This model assumes that the material is isotropic and has a three-invariant yield surface. The strength characteristics of the material depend on the rate of loading, and its damage is considered separately for compressive and tensile loads, which allows taking into account the partial recovery of compressive strength. The mathematical description of the model is given as part of the paper. Steel reinforcement of the concrete slab is modeled explicitly with beam finite elements. Finite element meshes of the concrete volume and reinforcing elements are coupled by means of kinematic dependences, automatically created by the design code. The properties of the reinforcement material are specified within the classical theory of elastoplastic flow with the criterion of limiting states in the Huber–Mises form and taking into account viscoplastic effects. The influence of boundary conditions, practical mesh convergence, and the capability of the mathematical model to predict the location of zones of material failure, displacement, and deformation of the structure are studied.

Keywords: reinforced concrete slab, Blind Blast Test, cap surface model, fracture, shock wave, crack, numerical simulation, LS-DYNA, CSCM concrete model

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1. INTRODUCTION

One of the relevant problems in the design of protective reinforced concrete (RC) structures is to predict their dynamics and strength under various external influences, including the impact of massive deformable and nondeformable projectiles, earthquakes, and air shock waves. The aim of this work is to calculate the impact of an air shock wave on a reinforced concrete slab and compare the results with experimental data [1]. Calculation is carried out by the finite element method using LS-DYNA software. The CSCM (Continuous Surface Cap Model) model is used to describe the properties of concrete [2].

This model has passed a series of validation calculations [3] and is widely used for calculating the dynamics and strength of concrete structures [4–6].

The parameters of experiments carried out in the “Blind Blast Test” program [1, 7] are taken as the main initial data in the problem setup. The object of research is an RC slab with ASTM Grade 60#3 reinforcement (diameter 9.525 mm) [8]. Figure 1 shows the slab dimensions and reinforcement location. The slab is installed in the shock tube on a rigid steel support frame of high strength. From the front the slab rests on two square bars with dimensions of 76.2 × 76.2 mm, and from the back, on two bars of rectangular cross-section with dimensions 152.4 × 203.2 mm. Figure 2 shows the general view of the support
NUMERICAL SIMULATION OF THE DYNAMICS

1177

Fig. 1. Slab dimensions (in mm) and reinforcement arrangement in two projections.

Fig. 2. Slab support [1]: frame (a), support bars (b).

On such a test bench, two experiments are carried out for explosive charges of different power. Figure 4 shows the time dependences of the pressure acting on the plates; they are obtained by averaging the measurements of the sensors shown in Fig. 3. The experiments yield qualitative and quantitative results, namely, the deformed state of a reinforced concrete slab after exposure to an air shock wave and the history of movement of the central point of its back surface.

2. MATHEMATICAL MODEL

2.1. General Setup

Consider the deformation in time of an arbitrary fixed volume $\Omega_0$, bounded by a smooth closed surface $\Gamma = \Gamma_1 \cup \Gamma_2 \cup \Gamma_3$. The motion equation can be written as:

$$\rho \ddot{x}_i = \sigma_{i,j} + \rho f_i,$$

In the general case, three types of boundary conditions can be imposed on Eq. (1):

- For $\Gamma_1$, the force boundary conditions: $\sigma_{y}n_{y}\Big|_{\Gamma_1} = \tau_i (t)$.
—For $\Gamma_2$, the kinematic boundary conditions: $u_i(t)|_{\Gamma_2} = U_i(t)$.

—For $\Gamma_3$, the surface conditions: $(\sigma^+_{ij} - \sigma^-_{ij})n_j|_{\Gamma_3} = 0$.

The designation is the following: $\sigma_{ij}$ is the strain tensor; $f_i$ is the volume force; $\dddot{x}_j$ is acceleration; $n_j$ is the external normal to the boundary; $\sigma^+_{ij}$, $\sigma^-_{ij}$ are strain tensors of the bodies in contact; and $u_i$ are displacements.

The mass conservation law can be written as:

$$\rho V = \rho_0 V_0,$$

where $\rho_0$, $\rho$ are initial and current densities and $V, V_0$ are the initial and current volumes.

The energy equation has the form:

$$\dot{e} = \Theta s_{ij} \varepsilon_{ij} - p \Theta,$$

where $s_{ij}$ are components of the deviator of the strain tensor, $\varepsilon_{ij}$ is the strain rate tensor, $p$ is pressure, $\Theta$ is the specific volume, and $e$ is the internal energy of a unit volume.
Deformations and displacements obey geometric relationships:

\[ \varepsilon_y = \frac{1}{2} (u_{i,j} + u_{j,i}). \]

In order for this system of equations to become complete, it is also necessary to set physical relations connecting stresses, deformations, strain rates, temperature, and other quantities. This work uses the CSCM material model [2, 9] to describe the deformation of concrete, its physical relationships will be given further. The behavior of the reinforcement material is modeled within the framework of the classical flow theory based on the associated flow law. The model takes into account the criterion of limiting states in the Huber–Mises form [10] and velocity hardening.

To solve the presented system of equations, the finite element method is used; details of its implementation in the LS-DYNA code can be found in [9].

2.2. Model of Material (Concrete)

The CSCM material model is a surface cap model with a continuous (limiting) surface used to specify nonlinear properties of concrete [2]. The model assumes that concrete is isotropic and has a three-invariant yield surface, and takes into account material damage. The latter can be of two types: brittle (accumulates during stretching) and viscous (accumulates during compression). The model also includes velocity hardening of the material and the phenomenon of dilatation—an increase in its volume during plastic deformations [11].

Before moving on to the mathematical representation of the CSCM model, let us introduce the concepts of multiaxial tension and multiaxial compression. Experiments under these types of loading are common in laboratory studies, usually carried out on cylindrical specimens. At the first step, the samples are subjected to hydrostatic compression, that is, the axial stress \( \sigma_x \) in the sample is equal to the radial, \( \sigma_r \). Under multiaxial compression, the sample is then compressed in the axial direction until fracture, while the radial stress \( \sigma_r \) remains constant. Under multiaxial tension, the sample breaks in the axial direction, while \( \sigma_r \) remains constant as well.

Below are the main relationships of this model. A detailed discussion, as well as obtaining the parameters included in it, is described in [2], and the results of validation on a set of tests are discussed in [3].

So, according to the model, in the elastic region concrete is considered isotropic and obeying Hooke’s law, the limiting yield surface has the form:

\[ f(I_1, J_2, J_3, \chi) = J_2 - \omega^2 F_f^2 F_e, \]

where \( I_1 = \sigma_{ii} \) is the first invariant of the strain tensor, \( J_2 = s_{yy}s_{yy}/2 \) and \( J_3 = s_{yy}s_{jk}s_{kl}/3 \) are the second and third invariants of the deviator of the strain tensor, \( F_f \) is the meridian (side) surface, \( F_e \) is the elliptical surface (cap), \( \omega \) is the Rubin scaling function, and \( \chi \) is the hardening parameter of the elliptical surface. Figure 5 presents invariant surfaces and the limiting surface (Fig. 5d).

The stresses calculated from the elastic relations are referred to below as test elastic stresses \( \sigma_y^T \), the corresponding invariants are denoted as \( I_1^T, J_2^T, J_3^T \). When the inequality \( f(I_1^T, J_2^T, J_3^T, \chi_0) \leq 0 \) is true, the material behaves elastically. With \( f(I_1^T, J_2^T, J_3^T, \chi_0) > 0 \), the material exhibits elastoplastic properties, and in this case, the algorithm, in accordance with the associated flow law, returns the stresses to those values at which \( f(I_1^p, J_2^p, J_3^p, \chi_0) = 0 \).

Meridian surface \( F_f \) has the following equation:

\[ F_f(I_1) = \alpha - \lambda e^{-\beta I_1} + \gamma I_1, \]

where \( \alpha, \beta, \gamma, \lambda \) are material parameters obtained from triaxial compression tests of concrete cylinders. Figure 5a shows the meridian surface in the space of principal stresses.

The elliptical surface \( F_e \) describes the change in volume due to the collapse of concrete pores. Its equation is:

\[ F_e(I_1, \chi) = 1 - \frac{(I_1 - L(\chi))(I_1 - L(\chi)) + I_1 - L(\chi))}{2(L(\chi) - L(\chi))^2}, \]

where \( L(\chi) = \begin{cases} \chi, & \text{if } \chi > \chi_0, \\ \chi_0, & \text{if } \chi < \chi_0. \end{cases} \)
The equation for $F_c$ with $I_1 = L(\chi)$ is equal to one; with $I_1 > L(\chi)$ it corresponds to an ellipse. The cap and the side surface intersect at $I_1 = \chi$. In this case, $\chi_0$ is the value of $I_1$ at the intersection of the cap and the meridian surface until the hardening starts (before the cap is displaced). The cap intersects the hydrostatic axis at the point $I_1 = X(\chi)$. The position of that point in the space of principal stresses depends on the ellipticity parameter $R$:

$$X(\chi) = L(\chi) + RF_f(L(\chi)).$$

The movement of the cap simulates a plastic change in volume: when $X(\chi)$ and $\chi$ increase, the cap expands, and the volume contracts; when the cap shrinks, the volume expands, that is, dilation takes place.

The movement of the cap obeys the following law of hardening:

$$\varepsilon^p_{\psi} = W\left(1 - e^{-D_1(X-x_0)-D_2(X-x_1)^2}\right),$$

where $\varepsilon^p_{\psi}$ is the plastic volume deformation, $W$ is maximum plastic volumetric deformation, $D_1$ and $D_2$ are the material parameters, $X_0$ and $X$ are initial and current intersection points between the cap and the hydrostatic axis. Figures 5a–5c show the invariant surfaces. It has been experimentally established that concrete begins to collapse under conditions of triaxial tension and torsion at lower values of $J_3$ than under triaxial compression. This suggests that the limiting surface depends on the third invariant of the strain tensor deviator $J_3$. In the deviatoric plane, the three-invariant limiting surface takes the form of a triangle or a hexagon.

The scaling function $\omega$ introduces the dependence of any stress state on the limiting surface on the stress state under triaxial compression in the form $\omega F_f$. In this case $\omega$ depends on the angle $\hat{\beta}$. Angle $\hat{\beta}$ varies within $-(\pi/6) < \hat{\beta} < (\pi/6)$ and can be expressed in terms of $J_2$ and $J_3$ as:

$$\sin 3\hat{\beta} = \hat{J}_3 = 3\sqrt{3}J_3/(2J_2^{3/2}),$$

Fig. 5. Invariant surfaces in the space of principal stresses: meridian $J_2 - F_c = 0$ (a); elliptical $J_2 - F_f^2 = 0$ (b); combined $J_2 - F_f^2 F_c = 0$ (c); limiting surface $J_2 - \omega^2 F_f^2 F_c = 0$ (d).
where $\hat{J}_3$ is a normalized invariant whose values lie within $-1 < \hat{J}_3 \leq 1$. For triaxial compression $\hat{J}_3 = 1$, for torsion $\hat{J}_3 = 0$, and for triaxial tension $\hat{J}_3 = -1$.

The formulas for determining the scaling function $\omega$ are:

$$\omega = \frac{-b_1 + \sqrt{b_1^2 - 4b_2b_3}}{2b_2},$$

$$b_2 = (\cos \beta - a \sin \beta)^2 + b \sin^2 \beta, \quad b_1 = a(\cos \beta - a \sin \beta), \quad b_3 = -\frac{(3 + b - a^2)}{4} b = (2Q_1 + a)^2 - 3,$$

$$a = -\frac{a_1 + \sqrt{a_1^2 - 4a_2a_3}}{2a_2}, \quad a_2 = Q_1, \quad a_1 = \sqrt{3}Q_2 + 2Q_1(Q_2 - 1), \quad a_0 = 2Q_2^2(Q_2 - 1).$$

Here $Q_1$ and $Q_2$ are functions of $I_1$

$$Q_1 = \alpha_1 - \lambda_1 e^{-\beta_1 I_1} + \gamma_1 I_1,$$

$$Q_2 = \alpha_2 - \lambda_2 e^{-\beta_2 I_1} + \gamma_2 I_1,$$

where $\alpha_1$, $\lambda_1$, $\beta_1$, $\gamma_1$ and $\alpha_2$, $\lambda_2$, $\beta_2$, $\gamma_2$ are parameters of the material. When the limiting surface is cut by the deviatoric plane, the three-invariant limiting surface transforms from a triangle into an irregular hexagon and a circle with increasing pressure. For triaxial tension $\omega = Q_1 F_1$ and for torsion $\omega = Q_2 F_1$. Functions $Q_1$ and $Q_2$ control the shape of the surface only under compressive pressures; in the case of tension they take on the values $Q_1 = 0.5774$ and $Q_2 = 0.5$, which leads to a triangular section of the limiting surface by the deviatoric plane.

When stretched with a small amount of compression, concrete softens, which is modeled by introducing damage:

$$\sigma_{ij}^d = (1 - d) \sigma_{ij}^p. \quad (2)$$

The scalar damage parameter $d$ converts the viscoelastic strain tensor $\sigma_{ij}^p$ into the strain tensor $\sigma_{ij}^d$ taking into account the damage. The damage $d$ can increase from 0 to 1 and is subdivided into viscous and brittle. Damage begins to accumulate when the boundary of the deformable fixed volume reaches the limiting surface. At a compressive (positive) pressure $P$ and the value of $\tau_c = \sqrt{\sigma_{ij} \epsilon_{ij}}/2$, exceeding the limiting value $\tau_{oc}$, the material receives viscous damage:

$$d_c = \frac{d_{max}}{B} \left( \frac{1 + B}{1 + Be^{-d(\tau_c - \tau_{oc})}} - 1 \right).$$

Brittle damage occurs when the pressure $P$ is tensile (negative) and the value of $\tau_c = \sqrt{E\epsilon_{max}}$ exceeds the limiting value $\tau_{oc}$. Here $\epsilon_{max}$ is the maximum principal deformation and $E$ the Young’s modulus of undamaged concrete. The damage is determined by the formula:

$$d_i = 0.999 \left( \frac{1 + D}{1 + De^{-c(\tau_c - \tau_{oc})}} - 1 \right).$$

Quantities $A$, $B$, $C$, $D$ and $d_{max}$ are parameters of the material. The maximum current viscous or brittle damage is selected for substitution into Eq. (2): $d = \max(d_c, d_i)$.

The strength properties of concrete are significantly influenced by the strain rate. When it increases, the material hardens. Within the framework of the CSM material model, implemented in LS-DYNA code, rate effects are presented in viscoelastic form [8].

According to the algorithm, in order to obtain viscoelastic stresses (taking into account the rate hardening), at each time step interpolation between elastic test stresses $\sigma_{ij}^T$ and nonviscous plastic stresses $\sigma_{ij}^p$ is carried out (excluding rate hardening):

$$\sigma_{ij}^{ep} = (1 - \gamma) \sigma_{ij}^T + \gamma \sigma_{ij}^p, \quad \gamma = \frac{\Delta t / \eta}{1 + (\Delta t / \eta)}.$$
As you can see, interpolation depends on the effective yield factor $\eta$ and time step $\Delta t$. Coefficient $\eta$ is calculated using seven specified parameters of the material according to the formulas:

- for tensile pressure

$$\eta = \eta_s + \left( -\frac{I_1}{\sqrt{3J_2}} \right)^{PWRT} (\eta_t - \eta_s),$$

- for compressive pressure

$$\eta = \eta_s + \left( \frac{I_1}{\sqrt{3J_2}} \right)^{PWRC} (\eta_c - \eta_s),$$

where $\eta_s = SRATE \cdot \eta_s$, $\eta_t = \eta_{0t}/\varepsilon^{N_t}$, $\eta_c = \eta_{0c}/\varepsilon^{N_c}$, $\dot{\varepsilon}$ is the strain rate intensity, $SRATE$, $\eta_{0t}$, $\eta_{0c}$, $N_t$, $N_c$, $PWRT$, $PWRC$ are parameters of the material.

2.3. Model of Reinforcement Material

To describe the behavior of the reinforcement material, a nonlinear elastoplastic model with isotropic hardening is used, implemented in LS-DYNA by the *MAT_PIECEWISE_LINEAR_PLASTICITY card [9]. Viscous effects in the material are taken into account in this work by scaling the current flow stress. The coefficient of dynamic hardening proposed in [12] is applied; for the reinforcement material in question (reinforcement material complies with the ASTM Grade 60 standard [8]) it has the from: 

$$(\dot{\varepsilon} \times 10^4)^{0.028}.$$
Only the transverse beams of the support frame are involved in loading the slab (see Fig. 6d), the rest of the support elements are not taken into account. Moreover, the fixtures are assumed to be absolutely rigid, which allows the support beams to be considered as rigid shell bodies.

3.2. Loads and Boundary Conditions

The vertical posts of the support frame, on which the sensors are located (see Fig. 3), do not restrict the movement of the slab in the direction of the shock wave and are therefore excluded from the analysis model. Figure 7 presents kinematic boundary conditions imposed on all nodes of the support frame. The load on the RC slab is the pressure uniformly distributed over its surface, which changes over time (Fig. 8).

3.3. Parameters of Material Models

The reinforcement material complies with the ASTM Grade 60 standard [8]. To describe the behavior of the reinforcement material, we use an elastoplastic model with kinematic hardening and the possibility of directly setting the $\sigma-\varepsilon$ curve for plastic deformations. The model is described by the *MAT_PIECEWISE_LINEAR_PLASTICITY card.
The cross-sectional area of beam elements simulating reinforcement is considered constant. Thus, the calculation can directly use the experimental dependence $\sigma - \varepsilon$ for the plastic deformation zone [1]. The model assumes that upon reaching the modulus of rupture, the material becomes ideally plastic. The $\sigma - \varepsilon$ curve is shown in Fig. 9.

The CSCM model used for concrete has the parameters given in LS-DYNA *MAT_CSCM card [14] (parameters are formulated for measurements in mm–ms–g):

```
*MAT_CSCM
  $# MIDRHONPLOTINCREIRATEERODERECOVITRETRC
  159 2.4E - 09 1 0 1 0.90 10 0
  $# PRED
  0
  $# G K ALPHA THETA LAMBDA BETA NH CH
  1.451E + 04 9.169 0.3373 3.981 0.04177 0 0

  $# ALPHA1 THETA1 LAMBDA1 BETA1 ALPHA2 THETA2 LAMBDA2 BETA2
  0.8200 0 0.2407 0.009924 0.7600 0 0.2600 0.009924
  $# R X0 W D1 D2
  2.177 82.30 0.065 0.0006112.225E - 06
  $# B GFC D GFT GFS PWRC PWRT PMOD
  100 6.959 0.1 0.06959 0.06959 5 1 0
  $# ETA_0_C ETA_0_TN_C N_T OVERC OVERT SRATE REPOW
  0.0001090 0.78 6.601E - 05 0.48 23.47 23.47 1 1
```

The parameters of the CSCM model are taken from the documentation [2, 3], as well as from [15], which proposes an alternative method for calibrating the strength surface for a similar class of models. Thus, the settings for the concrete model are generated based on the data on the compressive strength of cylindrical samples $f_c = 34.5$ MPa and the conventional value of the characteristic size of the concrete filler $d_{max} = 8$ mm [1] and are used in all the calculation cases considered below.

4. CALCULATION RESULTS AND COMPARISON WITH EXPERIMENTAL DATA

4.1. Refining Parameters of RC Slab Restraint

During the experiment, there is a gap between the slab and the support bars (Fig. 10), the exact value of which is unknown [16]. However, this geometric parameter can directly affect the results, since different types of boundary conditions are implemented for different values of the gap.

If the gap turns out to be small, then under the influence of a shock wave of pressure in the plate, three plastic hinges are formed (in the center and near the points of attachment) (Fig. 11a). This does not correspond to the pattern of deformations and destruction of the slab observed in the experiment (Fig. 11c). Moreover, in the presence of three plastic hinges, the maximum deflection of the structure would be significantly less than in the presence of a single one, since the energy of the shock wave is spent on creating
three zones of destruction, and not one. Therefore, for the correct simulation of the considered process, it is necessary to ensure the formation of a single plastic hinge by setting the gap (see Fig. 11b).

To determine the probable size of the gap, a series of calculations was carried out. According to the results, the maximum deflection of the slab and its deformed state were estimated. The calculations were performed for a model with a characteristic finite element mesh size of 10 mm, which ensures 10 mesh elements in thickness. This level of discretization is assumed sufficient to describe the pattern of deformations and damages at this stage of the study. A more detailed study of practical grid convergence was carried out at the next stage of the work.

Figure 12 presents the numerical visualization of formation zones of plastic hinges, zones of maximum damage to concrete slab. It can be seen that for 6-, 8- and 10-mm gaps (Figs. 12a–12c), the three plastic hinges are clearly visible. For a 12-mm gap (Fig. 12d), there are almost no plastic hinges at the fixing points. And, finally, for gaps of 14 and 17 mm (Figs. 12e, 12f) only one plastic hinge remains in the system.
It can be concluded that the best qualitative agreement with the experimental results is the presence of only small surface defects from interaction with the support, provided by a gap of 14 mm.

Figure 13 shows the dependence of the maximum deflection on the size of the gap between the plate and the support. It can be seen that, starting from a gap of 14 mm, the maximum deflection of the structure does not change.

It should be noted that in the real experiment, the maximum deflection of a slab under the influence of an air shock wave was 112.3 mm. Thus, setting the right size of the gap and the corresponding boundary conditions of the problem can reduce the error in calculating the deflection of the structure from 19% for a gap of 6 mm to 4% for a gap of 14 mm.

Based on a series of numerically solved problems, it is shown that in the presence of a gap of 14 mm, the deformation of a reinforced concrete slab qualitatively corresponds to the real deformation pattern of the slab in the experiment, and a further increase in the gap does not change in the deflection of the slab. Based on the above, the size of the gap for further calculations is taken equal to 14 mm.

4.2. Comparison of the Calculated and Experimental Maximum Deflections

Figure 14 shows the time dependences of the maximum slab deflection for loads 1 and 2 (see Fig. 4), calculated on meshes with different characteristic size of the finite element.
The applied mathematical model represents well not only peak deflections, but also the nature of slab vibrations around a new equilibrium position. It can be seen from the figures that the error of the calculated maximum deflection for the considered loads, even on the coarsest mesh, does not exceed 15%. With an increase in the mesh dimension the agreement with the experimental results improves.

Generally, we can report a good agreement between the results both for the peak deflection and for the description of slowly damped vibrations of the system around a new equilibrium position.

### 4.3. Qualitative Pattern of Slab Damage

Consider a qualitative pattern of the state of reinforced concrete slabs after exposure to an air shock wave. Figures 15 and 16 show numerically obtained patterns of damage distribution in a slab with finite...
element dimensions of 20, 10, and 5 mm for the first load (see Fig. 4); photographs of real fracture zones are given for comparison.

It can be seen from the presented figures that the calculation results qualitatively correspond to the experimental data: with the considered sets, one plastic hinge is formed in the center of the reinforced concrete slab. Refinement of the finite element mesh leads to an increase in the detail of the damage field. Thus, the damage begins to visually correspond to the picture of real fractures in the material, which is best seen in Fig. 15c.

5. CONCLUSIONS

The paper presents a numerical study of the dynamics and strength of a reinforced concrete slab under the action of an air shock wave. It was revealed that the geometrical dimensions of the support frame restraining the slab have a significant effect on the boundary conditions, and, consequently, on the physics of the modeled process. Based on a multivariate study, we selected the geometry of the support frame that provides the best qualitative and quantitative agreement between the calculation and experimental results. Based on this, the boundary conditions corresponding to the case of the best agreement with experiment were taken for the model, and the practical convergence of the calculation results was studied. It is shown that the difference in the maximum slab deflection between the calculated and experimental values, even on a coarse mesh, does not exceed 15%. An increase in the dimension of the discrete model with a decrease in the size of the FE mesh element to 5 mm makes it possible to obtain a detailed field of structural damage, which is in good agreement with the picture of real fractures of the structure.

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