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Bearing capacity evaluation of keyed joints in precast-cast-in-place reinforced concrete structures for effective rehabilitation

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Abstract. In order to effectively rehabilitate buildings and structures and ensure their serviceability, it is important to improve precision of the estimation of the keyed joints ultimate load in prefabricated and prefabricated monolithic structures made of reinforced concrete and their remaining service life. In this direction, the introduction of a general theoretical method for calculating bearing capacity is promising. Adapted to concrete and reinforced concrete apparatus of the plasticity theory, the variational method, and the possible velocities principle are used. The criterion for the plasticity theory application is the condition of simultaneous achievement of the ultimate stresses in the compressed and tensile areas of the failure surface. The kinematic schemes of failure of keyed joints are proposed, which are adopted on the basis of the failure nature study and reflect the behavior of the element in a limit state. Formulas are proposed for calculating the ultimate load of single keyed joints that fail by key shearing, seam shearing along an inclined plane, and splitting of an inclined compressed strip. They allow to fully consider the factors influencing the joints bearing capacity. Results obtained in experiments confirm the theoretical assessment reliability.

1. Introduction

In modern reinforced concrete structural systems, which are advisable to use for the rehabilitation of the destroyed housing fund of Ukraine, precast-cast-in-place structures of floors (roofs), walls (columns) are widely used, provided that the members are reliably connected. Design, construction and technological measures ensure the precast-cast-in-place concrete joint operation. The options for the arrangement of the contact joint can vary and depend on the type of surface of the precast member (smooth, rough, keyed) and the need to calculate it. When designing reinforced concrete structural systems, it is imperative to pay particular attention to the members joints, which ensure joint operation under the load. Keyed joints play an important role. They have an increased shear bearing capacity.

At present, a considerable value of experimental material has been accumulated on strength, the failure character and the various factors influence on the keyed joints bearing capacity [1-12]. Concrete compressive strength and bearing surface friction are critical factors influencing the shear capacity of joints. The pre-stressing (compressing), number, dimensions, and configuration also have an effect on the ultimate load of a keyed connection. However, a clear pattern of influence of these factors has not yet been identified. Numerous empirical

dependencies for determining the ultimate load are proposed. Extending the formulas obtained for specific experimental cases to other cases tends to reduce the accuracy of such calculations. Therefore, it is necessary to develop a simple and accurate theoretical relationships for the joints calculation. In our opinion, it is promising to develop a general methodology for bearing capacity (strength) calculation based on the theory of concrete plasticity under compressive hydrostatic pressure. Methods based on this idea and providing reliable results are already known [13-17]. The calculation taking into account plastic strains is based on the method of estimating the lower limit (static) or upper limit (kinematic), while the kinematic approach is less developed than the static one.

To calculate the strength of reinforced concrete and masonry members under shear, local compression and punching shear, a variational method of concrete plasticity theory was developed [18, 19]. In view of the limited concrete plastisity, the simultaneous existence of a limit state over the entire failure area can be accepted as a necessary condition for the formation of a plastic kinematic mechanism of bodies with a nonuniform stress-strain state. This assumption is regarded as a primary criterion for the feasibility of implementing the plasticiti theory in the calculation of the reinforced concrete structures bearing capacity, despite their apparently fragile character of their failure. There is a need for the create a methodology for evaluating the structures connections that ensure their joint operation.

2. Method of design

The aim of this article is to formulate recommendations for calculating the contact joints bearing capacity of reinforced concrete structures.

This study uses the variational method in the theory of concrete plasticity as a theoretical basis. According to the accepted sequence of solving problems by the method in the beginning a kinematic failure diagram of the member is created. Such a scheme for the calculation of a concrete rectangular key is shown in Fig. 1, a.

It includes two disks, which are divided between themselves by the failure surface ABC (or surface of velocities jumps). Disk I moves in relation to a disk II with speed V (V_x, V_y). On the part BC the limit normal σ_u and tangent τ_u stresses act, which are determined according to the condition of concrete strength. The part AB is principal with tensile stress $\sigma_u = f_{ct}$.

Unknown quantities of the considered problem are: the limit load q_u, the inclination angle α of the failure part AB and angle β of BC to a vertical and velocities ratio $k = V_x / V_y$.

In the second stage, for the proposed kinematic scheme, the jumps of normal and tangential velocities on the part AB and the part BC of failure surface and the areas of these parts through the unknown geometric parameters and velocities of the disks are recorded. A functional of the vitrual velocity principle is also created.

It includes the power of plastic strain at the failure surface.

$$I = \int_{S_l} m \left[2B \left(1 + 0.25 \left(\Delta V_t / \Delta V_n \right)^2 \right)^{0,5} - 1 \right] \Delta V_n dS - \int_{S_f} f_i^* V_i dS , \tag{1}$$

where $m = f_c - f_{ct}$, $B = \dfrac{1}{\sqrt{3}} \sqrt{1 + \chi / (1 - \chi)^2}$, $\chi = f_{ct} / f_c$, f_i^* are forces on the sections S_f of the

solid surface, S_l is failure surface, ΔV_n, ΔV_t are normal and tangent to S_l jumps of velocities components.

In order to ascertain the minimum value of the plastic strain power at the ultimate state, it is necessary to investigate the functional (2) for the equilibrium state.

$$I = m \left\{ 2B \left[\left(1 + \frac{1}{4} \left(\frac{V_x \sin\beta + V_y \cos\beta}{V_x \cos\beta - V_y \sin\beta} \right)^2 \right)^{0,5} - 1 \right] \left(V_x \cos\beta - V_y \sin\beta \right) h_k \frac{b_k \sqrt{1 + tg^2\beta}}{tg\beta + tg\alpha} tg\alpha + \right.$$

$$\left. + f_{ct} b_k h_k \frac{\sqrt{1 + tg^2\alpha}}{tg\alpha + tg\beta} tg\beta \left(V_x \cos\alpha + V_y \sin\alpha \right) - q_u l_k V_y \right\} = 0. \tag{2}$$
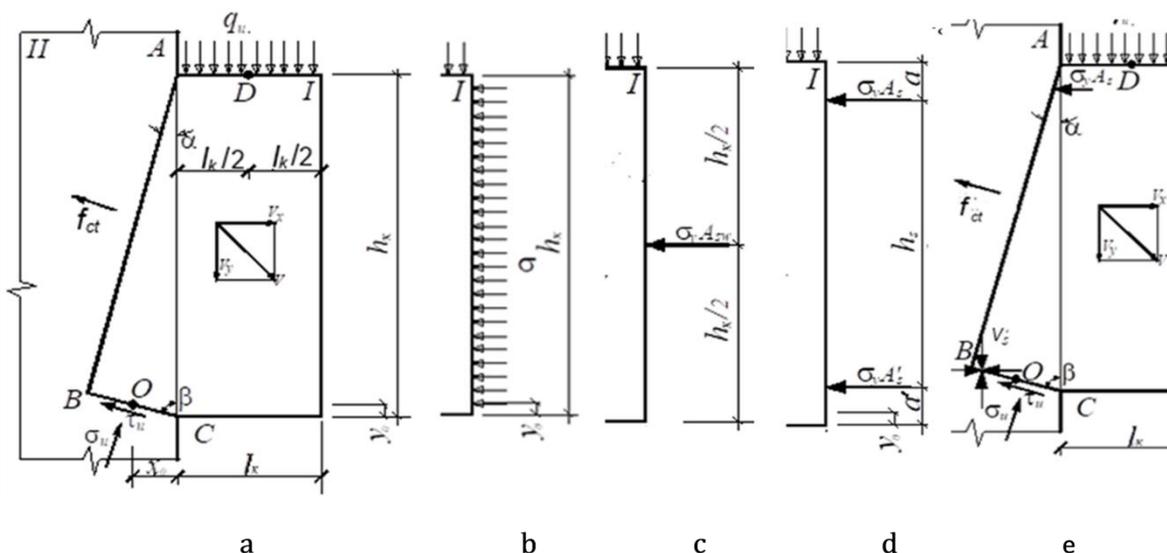

Figure 1. Kinematic failure mechanism of the rectangular key: a – is made of concrete;
b – compressed; c – reinforced with rebars in one level; d – reinforced with rebars in two levels;
e – reinforced with rebars in two levels when taking into account the dowel effect in the compressed
reinforcement.

The limit load depending on determining factors is

$$q_u = \left[2B \left(\sqrt{a^2 + 0.25c^2} - a \right) \frac{m}{1 + tg\beta/tg\alpha} + f_{ct} \frac{D}{1 + tg\alpha/tg\beta} \right] \frac{1}{\gamma}, \tag{3}$$

where $a = k - tg\beta$, $c = 1 + ktg\beta$, $D = k + tg\alpha$, $\gamma = l_k / h_k$, l_k is key depth, h_k is key height.

To partially account for the rotation in the kinematics of the key limit state, additional equations of the moments for disk I (Fig, 1, a) separated by the velocity discontinuity surface are used. The full consideration of the rotation in the kinematics of the disc motion I on the kinematic scheme in the functional makes the solution of the problem very complicated, since the velocities and their discontinuities on AB and BC become variable. Generally, three moment equations: $M_B = 0$; $M_O = 0$; $M_D = 0$ can be used regarding characteristic points B, O, D.

The normal and tangential stresses at part AB are determined as a function of failure surface geometry and disc velocities:

$$\frac{\sigma_u}{m} = 2 \left(0.5 \pm \frac{Ba}{\sqrt{a^2 + 0.25c^2}} \right), \tag{4}$$

$$\frac{\tau_u}{m} = \pm \frac{Bc}{2\sqrt{a^2 + 0.25c^2}}. \tag{5}$$

Unknown parameters are determined by finding the conditional minimum of the

function (3).

Newton's method can be used to do this.

It should be noted that finite element calculations [20] involve the use of physical equations. These equations have not yet been sufficiently investigated for different stress states. Instead, in the proposed method, the inaccuracy of the physical equations is partially compensated by the use of the true kinematic mechanism of element fracture, which has been confirmed experimentally [21].

Compression is considered to be an external load in a plane perpendicular to the shear (Fig. 2, b). In formula 3, we add a term:

$$\sigma \, k \, / \, \gamma, \tag{6}$$

where σ is compressive stress evenly distributed over the key's cross-sectional plane.

The operation of the reinforcement in the limit state is considered by applying a concentrated external load equal to the force in the reinforcement.

If the dowel effect is not considered in rebars of reinforced concrete key, regardless of its location in key height (in one or two levels – Fig. 1, c and Fig. 1, d), an additional component (7) appears in (3) (for single or double reinforcement).

$$+\frac{\sigma_y A_{sw} k}{b_k h_k}\frac{1}{\gamma} \text{ or } +\frac{\sigma_y (A_s + A_s')k}{b_k h_k}\frac{1}{\gamma}, \tag{7}$$

where σ_y is yield strength of the reinforcement, $A_{sw} = A_s + A_s'$ is cross-sectional area of the reinforcement.

If the dowel effect is considered in rebars located in a zone of shear with compression, a term (8) is added to (3) instead of (7).

$$\frac{\sigma_y A_s k}{b_k h_k}+\frac{\sigma_y A_s' k_e}{b_k h_k}\sqrt{1+\frac{k^2}{4k_e^2}}\frac{1}{\gamma}, \tag{8}$$

where for heavy-weight concrete $k_b = 0,338$, for ceramsite concrete $k_b = 0,284$.

A separate accounting of work of reinforcement located within the boundaries of the tearing off and shear zones is carried out in reinforced concrete members. The reinforcement located in the shear zone is considered on the bases of the developed design scheme of longitudinal reinforcement in a zone of an inclined crack with the use of a bar model on a deformed foundation and with longitudinal and shear forces on the bar's end.

The shape of the key's cross-section is taken into account by changing the area of the failure surface and the law of external load application. The circular shape of the key cross-section is accounted for by introducing the factor 2/3 to formula (3) and the coefficient π/2 to formulas (6) - (8).

Most often, trapezoidal keys are used in reinforced concrete joints, which are characterized by a simpler technology installation compared to rectangular ones. Such keys have a higher strength, which is confirmed by experimental studies. The increase in the bearing capacity of the joints is explained by the influence of the load horizontal component, which occurs when it deviates from the shear plane by an angle θ.

The linear relationship (in degrees) between angles θ and ψ is proposed.

$$\theta = \psi / 15. \tag{9}$$

According to the available experimental data, it is proposed to classify the type of failure of single keyed joins as shear in four cases: without failure of the key (undesirable); by vertical and inclined cross section of the keys; by the seam. Failure of the joint occurs at the minimum force

value depending on the size of the key and seam, the shape of the keyed profile, reinforcement and compressing.

Fig. 2 shows the failure kinematic mechanism of the seam of a rectangular profile keyed joint with one-level reinforcement.

Figure 2. Failure kinematic mechanism of the reinforced concrete seam of a rectangular profile keyed joint.

The velocity jumps in the parts of the failure surface CB and OB do not change in comparison to parts BC and AB on Fig. 1, a, only the areas of these parts change.

Therefore, the first component in formula (3) is multiplied by $(tg\alpha_1 - t_j / h_k)$ instead of $tg\alpha$, in the second component, $tg\beta$ is replaced by $(tg\beta_1 + t_j / h_k)$.

The limit load depending on determining factors is

$$q_u = m\left(2B\sqrt{a_1^2 + 0.25c_1^2} - a_1\right)\frac{tg\alpha_1 - t_j / h_k}{tg\alpha_1 + tg\beta_1} + f_{ct}D_1\frac{tg\beta_1 + t_j / h_k}{tg\alpha_1 + tg\beta_1}\ \frac{1}{\gamma}, \tag{10}$$

where $a_1 = k - tg\beta_1$, $c_1 = 1 + ktg\beta_1$, $D_1 = k + tg\alpha_1$, t_j is seam width.

If the dowel effect in the reinforcement is not considered, it is necessary to use an additional term (7).

With a two-level location of the reinforcement and taking into account the dowel effect in it, the form of additional member is

$$\frac{\sigma_y A_s'(k^2 + 4k_в^2)}{2k_в h_k b_k}. \tag{11}$$

In this case, all reinforcement will be compressed.

The shape of the key profile and cross section is taken into account in the same way as for failure by key.

Fulfillment of the condition $t_j / h_k\ \ 1.25$ ensures the realization of the proposed kinematic cheme.

A calculation model of an inclined compressed strip that can be formed within a seam of keyed joint is also considered and its bearing capacity is determined.

In the limit state, the inclined compressed strip is divided into rigid blocks (Fig. 3). There are two wedges under the load platforms and two blocks separated by a splitting plane connecting the wedge tops. The wedges may be asymmetrical at certain keyed joint dimensions.
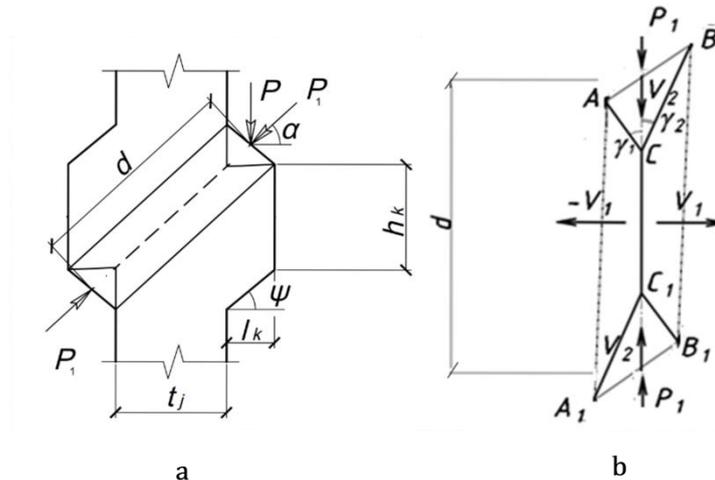

Figure 3. Kinematically possible failure scheme (b) of inclined compressed strip (a).

The wedges move toward each other under the action of an external load, causing the other two disks to move away in a direction perpendicular to the split plane. The two angles (γ_1, γ_2) at which the wedge shear surfaces oriented in relation to the split plane, the speed ratio of the rigid blocs $k = V_1 / V_2$, and the failure load are unknown.

The vertical load evenly distributed over the key's bearing surface is calculated as:

$$\frac{q_u}{m} = (2B\sqrt{a_2^2 + 0.25c_2^2} - a_2)\frac{1}{2tg\gamma_1} + (2B\sqrt{a_3^2 + 0.25c_3^2} - a_3)\frac{1}{2tg\gamma_2} +$$

$$+\frac{f_{ct}}{m}\left[\frac{d}{l_k} - \frac{1+tg\gamma_1 tg\psi}{tg\gamma_1}\right]\sin\alpha, \tag{12}$$

where $a_2 = k - tg\gamma_1$, $c_2 = 1 + ktg\gamma_1$, $a_3 = k - tg\gamma_2$, $c_3 = 1 + ktg\gamma_2$, α is the angle of the compressed strip in relation to the horizontal.

In the case of joint reinforcement in the middle of the key height, the component $\sigma_y A_{sw} k \cos\alpha / b_k h_k \gamma$ is added to formula (12).

3. Experimental researches

The experimental program included tests of keyed joints made of unreinforced and reinforced concrete to study the failure character data, value of ultimate load and factors that influence the strength. The research samples were divided into seven series. Each series was devoted to study one or more factors that influence the strength (Fig. 4): key size ratio l_k/h_k; key profile slope ψ; type and class of concrete; compressing value σ/f_c; reinforcement ρ_{sw} (number of rebar levels arrangement); seam width t_j; bonding new concrete to old; uneven work of the keys along connection (number of keys n_k).
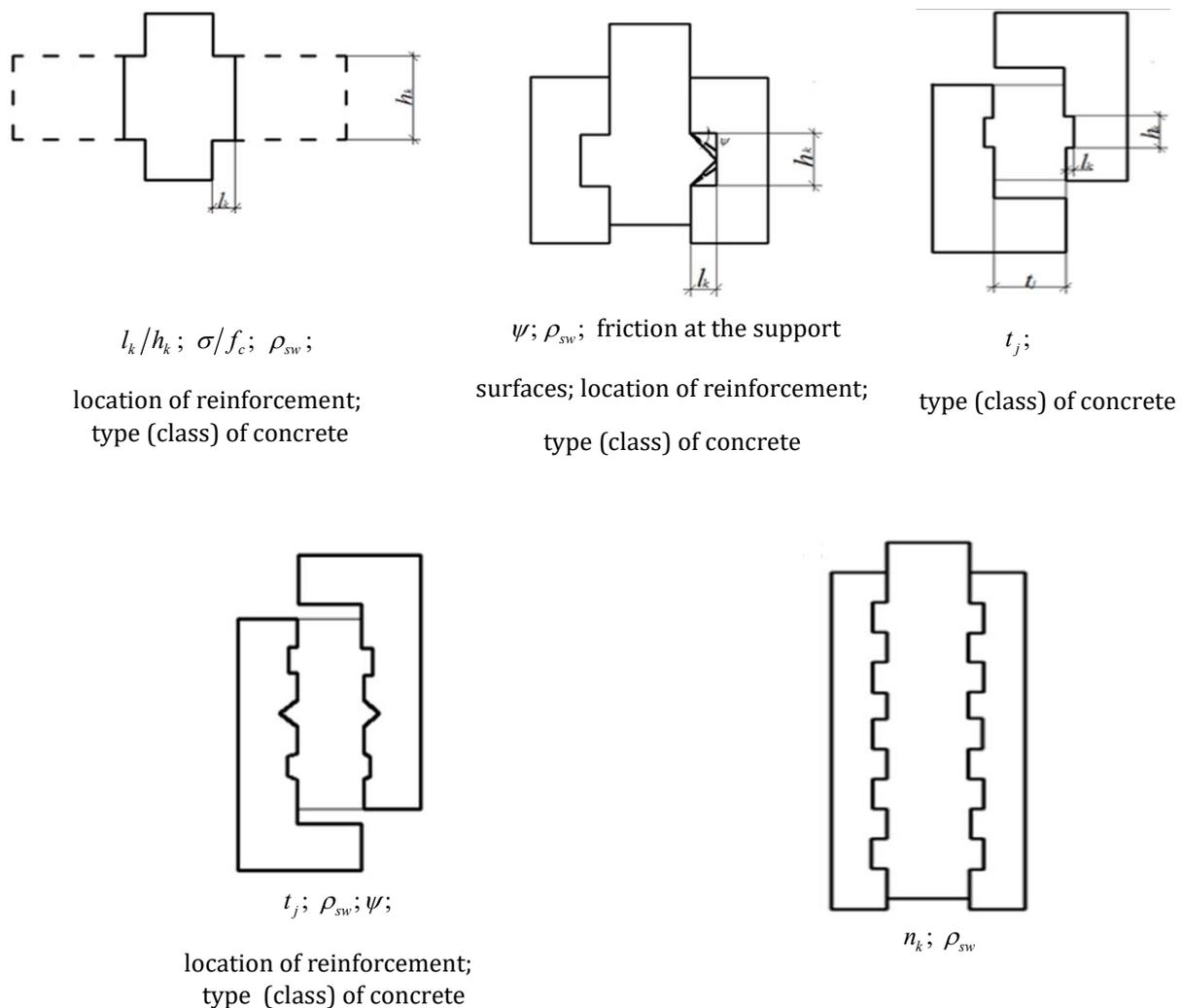
l_k/h_k ; σ/f_c ; ρ_{sw} ;

location of reinforcement;
type (class) of concrete

ψ ; ρ_{sw} ; friction at the support

surfaces; location of reinforcement;

type (class) of concrete

t_j ;

type (class) of concrete

t_j ; ρ_{sw} ; ψ ;

location of reinforcement;
type (class) of concrete

n_k ; ρ_{sw}

Figure 4. Types of samples.

Overall, during the research a total of 116 samples were made and tested. The variability factor ranged within wide limits: $l_k/h_k = 0.1-1$; $\sigma/f_{c,prizm} = 0-0.47$; $\psi = 0-45^0$; $\rho_{sw} = 0-3\%$; $t_j = 25-300$ mm; heavy-weight, ceramsite and fibre concrete was used; reinforcement location was one-level and two-level along the key height; key number in the joints was 1, 3, 5 and 8.

The received data lead to the following conclusions.

Value of ultimate load and the failure character of the concrete keys (local compression (shearing on inclined surface), shearing, tearing off) depends on ratio of key sizes. The key size ratio $l_k/h_k = 0.25$ ensures maximum strength. The bearing capacity of reinforced concrete and compressed keyed joints increases up to 2.5 times compared to joints made of concrete. In this case, destruction by shearing occurs up to $l_k/h_k = 1$ and improves concrete plasticity. Strength is increased by up to 10% and concrete compressive strain by up to 50% by separating the main reinforcement at two levels in the key height. A slight dowel effect can be seen in reinforcing bars located in the failure surface compressed zone. Joints with triangular key and size

ratio $l_k/h_k = 0.5$ are 10 % stronger than those with rectangular keys. When the load reaches 0.9 of the failure value, the intensity of deformation of concrete and reinforcement increases. A thin layer on the failure surface was found to contain plastic strains. At ρ_{sw} 1.5 % reinforcement stress reaches the yield point at failure. For single-keyed samples depending on the width seam the following failure character was noticed: key shearing along the vertical plane (key failure), seam shearing along the inclined plane (seam failure). A three keyed joint can fail under the shear along the keys, along the seam and have a combined failure: simultaneously along the keys and along the seam. As the seam width increases, the bearing capacity of join decreases.

In the multi-keyed joints, it was noticed that the key work was irregular along the length of the joint. For example, the bearing capacity of the five-keyed connection is only three times higher than that of the single key connection. The relationship between the number of keys and the ultimate load is therefore not linear. The utilisation of fibre-reinforced concrete in the construction of the joint has been demonstrated to enhance its resistance to cracking, augment its strength, and transform its failure mode from a fragile state to a plastic one.

The analysis of the convergence of the theoretical strength with the experimental strength was carried out by the value of their ratio. For 25 concrete joints it has shown $\bar{x} = 0.936$ with factor of a variation ν = 13.9 %; for 29 reinforced concrete joints $\bar{x} = 0.99$ and ν = 4,6 % were obtained; for 52 compressed concrete joints there are $\bar{x} = 1.026$ and $\nu = 17.96$ %

Conclusion

As an outcome of the examination of the stress-strain condition and of the manner of failure pertaining to key joints, a number of instances concerning the shear form of failure were identified. A methodology for assessing the bearing capacity of joints with increased shear resistance on a common theoretical basis, which is the variational method of the theory of concrete plasticity, has been developed. Using the methodology, the problems of bearing capacity of keyed joints at their failure by shear and diagonal splitting are solved. The obtained dependencies are refined compared to the normative ones on the basis of consideration of peculiarities of the specific joint stress-strain. This is the result of the application of an appropriate design scheme that is in line with the specifics of the task at hand and is introduced into the calculation: l_k/h_k ratio, key cross-sectional and profile shape, width of seam, compressing, reinforcement, compressive f_c and tensile f_{ct} strength of concrete. The various character of the reinforcement work is considered within the boundaries of zones of tearing off and shearing. The experimental results confirming the adopted members failure schemes and the influence of the factors determining strength are presented. The theoretical bearing capacity of keyed joints, determined on the basis of a proposed method, converges well enough with the tested values.

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