# Experimental estimation of the parameters of crack progression in concrete

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**Abstract.** This article presents the influence of the structure on the characteristics of the crack resistance of concrete. Due to ease of manufacture and testing of samples, the method of determining the results of the test $K_{IC}$ beams notched. Depending on the conditions of application of the load, the schemes were used to obtain the characteristics of the crack resistance of concrete in four-point bending, stretching, splitting. They allow to accurately determine stress coefficient $K_{IC}$.

## 1 Introduction

The first attempts to study the development of cracks in concrete and cement stone began in the works of foreign scientist A.Kaplan. Although Griffiths had already proposed the concept of the energy balance of fracture, it took several decades before the basic principles of fracture mechanics were developed for materials such as concrete. For the first time, this was done in the works of A. Kaplan, where the strength was associated with a random distribution of Griffith cracks in the concrete body. For the first time, concrete samples with a notch for bending were tested and the values of Gc were calculated [1].

## 2 Methods

Experimental studies of the crack resistance characteristics of concrete and its components are carried out on samples of various shapes and sizes, in which, depending on the scheme and method of loading, stable or unstable crack growth is observed, mainly of a detached nature. The characteristics obtained from tests can be divided into force ($K_I$, $K_{ZC}$), deformation ($\partial$_c), and energy ($G_I$, $G_{IC}$, Jc - integral, R - curves). All prototypes shall contain a notch or initial crack and be tested in standardized test devices. Parameters $G_I$, $G_{IC}$ and R -curves are determined from complete strain diagrams, therefore, in these cases, it is necessary to use rigid testing machines or special devices for redistributing forces. Types of samples and schemes of their loading are shown in Fig.1.

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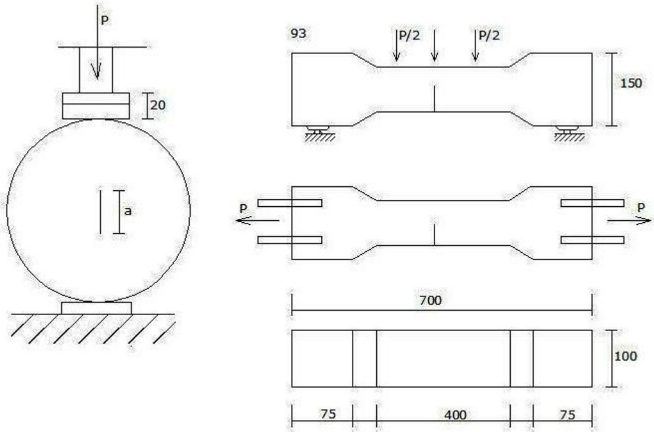



**Fig 1.** Scheme of testing prototypes (splitting, four-point bending, and tension of beam samples with various artificial notches).

Experimental testing of the possibility of applying methods of fracture mechanics to concretes has found wide coverage in a number of works by such researchers as Y.M. Bazhenov, G.I. Gorchakov, L.A. Alimov, A.P. Pak and L.L. Trapeznikov, E.A. Guzeev, V.I. Shevchenko, L.A. Seylanov [2-4] and others. In work [1] for the determination of KIC, the method of cylinder splitting was used, in work [3] - the so-called compact samples, in work [4] - the bending of a round sample with an annular groove, in work [5] - the scheme of a two-pillar beam, and in [6] - the schemes of eccentric compression and stretching of the plate with a central notch. It is worth noting that over the years of independence in our country, experimental studies of the parameters of the stress intensity factor (SIF) KIC, Gc of the destruction of concrete-like materials have not been carried out. Further research will begin a new, rapidly developing scientific direction - the mechanics of concrete destruction. Data from various investigators are shown in Table 1.

**Table 1.** Comparison of KIC and GIC test values from various investigators

| Research authors | W/C | Age, days | $K_{IC}$, MPa $/\sqrt{m^3}$ | $G_{IC,}$ kN/m |
|---|---|---|---|---|
| Kaplan | –<br>0.5–0.6<br>0.5<br>0.5 | 28<br>7–28<br>28<br>28 | 0.64–0.78 0.67–0.89 0.55–0.92 0.57–1.15 | 0.0173–0.0285<br>0.0109–0.0268<br>0.0140–0.0294<br>0.0067–0.0275 |
| Glucklich | – | – | – | 0.0193 |
| Lott and Kesler | 0.27–0.36<br>-<br>- | 7–28<br>-<br>- | 0.29–0.33 0.34–0.77 0.31–0.45 | 0.0035<br>0.0051<br>– |
| Moavenzada, Kuguel | –<br>– | 3–28<br>3–28 | 0.13–0.17 0.14–0.15 | 0.0035–0.005<br>0.0245–0.0043 |
| Welch and Heisman | 0.5<br>-<br>-<br>- | 3-28<br>-<br>-<br>- | 0.23-0.26<br>0.85<br>1.08<br>1.78-1.28 | -<br>0.0245<br>0.0385<br>0.0187-0.0363 |
| Okada and Koyanagi | -<br>-<br>- | 28<br>-<br>- | 0.22<br>0.29-0.39<br>0.33-0.46 | 0.0078<br>0.0098-0.0157<br>0.0118-0.0147 |

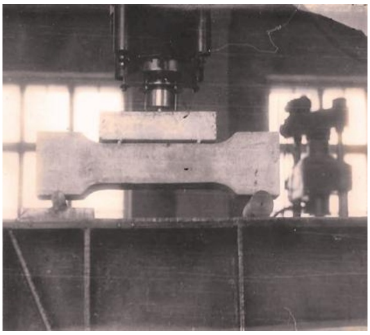
| | | | | |
|---|---|---|---|---|
| Brown and Pomeroy | 0.35-0.59<br>0.47<br>0.35-0.5<br>- | 14<br>14<br>14<br>- | 0.21-0.39<br>0.45-0.82<br>0.51-0.95<br>1.19-1.76 | -<br>-<br>-<br>- |
| Mindes, Nado and Hai | 0.4<br>0.5<br>0.5<br>05 | 120-180<br>120-180<br>78-99<br>78-99 | 0.32<br>033-0.48<br>0.36<br>0.76 | -<br>-<br>-<br>- |
| Mindes, Lawrence and Kesler | -<br>0.46 | -<br>- | -<br>- | 0.0088-0.0154<br>0.0172-0.0175 |
| Henry, Packet and Tankrose | - | - | 0.46-2.12 | - |
| Hillimeyer and Hilsdorf | 0.4<br>-<br>- | 7<br>-<br>- | 0.49-0.31<br>0.10-0.12<br>2.0-3.3 | -<br>-<br>- |
| Mindes, Lawrence and Kesler | 0.3<br>0.3-0.45 | 35-91<br>35-91 | 0.50-0.50<br>0.87-0.88 | -<br>- |
| Pak and Trapeznikov | 0.5<br>0.5 | 28<br>28-90 | 0.56-1.01<br>0.15-0.61 | -<br>- |
| Panasyuk, Berezhgitsky, Chubrikov | -<br>0.45 | -<br>28 | -<br>0.27-0.58 | 0.036 |
| Yentov, Yagust | – | 14 | 1.41 | – |
| Bazhenov, Gorchakov, Alimov, Voronin and others. | 0.20–0.35 | 28 | 4.97–3.06 | 0.0113–0.0058 |

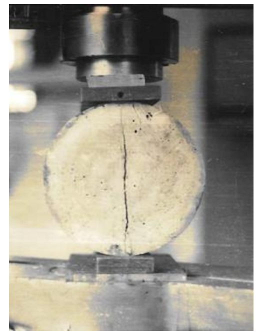
Comparison of the experimental values shows that for different test schemes with the same size of cuts, the KIC index is different. The value of fracture toughness for the contact zone is always lower, and for hard rock filler, it is an order of magnitude higher than for cement stone. Studies by various researchers show a different value of the surface energy for these materials, which is mainly associated with various microcracks, deviations of a macrocrack from a straight trajectory.

## 3 Results and Discussion

To experimentally determine the value of KIC and identify their dependence on various factors, six series of different samples of expanded claydite-concrete of various compositions and an additional comparative series of heavy concrete were tested. Portland cement and a pilot batch of alinite cement were used as a binder with an activity of 400. Water-cement (W/C) ratio of concretes of all series was W/C=0.5, compaction coefficient changed within Kc=0.97...0.98. As a fine aggregate, quartz sand with a maximum particle size of 5 mm was used in all series; coarse aggregate-expanded clay and crushed granite with a maximum fineness of 20mm.

The methodology of the study was to test the following specimens: a) the four-point bending (Fig. 2., a) beam size 100x100x700 mm with an artificial crack-notch length 50mm, which was performed during concreting samples by laying oiled metal plates 1 mm thick; b) on axial tension (Fig. 2., b) - specimens with dimensions of the working part of 100x100x400mm with an artificial crack, similar to the bending specimens; c) on cracking (Fig.2., c) - samples-cylinders with diameter 200 mm and height 200 mm in which the artificial crack with length a = 10, 20, 40, 60, 100 mm was created by laying of oil-lubricated foil with thickness 0.15 mm.

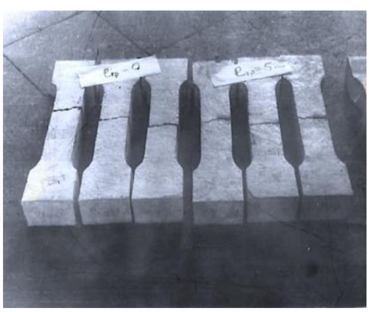




*a)* *b)* *c)*

**Fig.2.** General view of the test specimens to obtain the crack resistance characteristics of concrete in four-point bending (a), stretching (b), and cracking (c).

The age of all specimens at the time of testing was 28 days. Specimens were made in special metal molds and immediately after concreting were covered with polyethylene film. At one day of age, the samples were partially stripped to eliminate shrinkage cracks. After complete stripping, the specimens were covered with damp sackcloth with daily moistening. The day before testing, the specimens were transferred to the press room. A total of 168 specimens were tested in seven series. In addition to these specimens, each series was made on 6 specimens - cubes (100x100x100 mm) and 6 samples-prisms (100x100x400 mm). (Fig.3.).




*a)* *b)*

*c)*

**Fig.3.** General view of specimens after four-point bending (a), stretching (b), cracking (c) tests.

The main results of the tests are summarized in Table 2. Fracture stress $\sigma_{fr}$ for the specimens was determined by the formulas:

$$\sigma_{fr}^{bend}=M/W = 3PZ/H2 \text{ (bending)}; \quad \sigma_{\mathrm{fr}}^{str}=P/H \text{ (stretching)}; \quad \sigma_{fr}^{crc}=2P/\pi dH \text{ (cracking)}$$

where $P$ is the breaking load; $Z$ is the distance from the axis of support to the point of application of the force.

The values of the $K_{IC}$ and $l_{cr}$ for each type of test were determined by the formulas:

$$K_{IC}=\sigma_{fr}^{*}\sqrt{b}f(a,b);\ \ l_{cr}=bf-1\,[\sigma_{fr}^{*}/\,\sigma_{fr}] \qquad (1)$$

Here $\sigma_{fr}^{*}$– is the limit of a specimen with an artificial crack; ($a$–is the length of the notch; $b$– is the characteristic size of the specimen; $\sigma_{fr}$ –is the ultimate strength of the specimen without notch; $f-1$– is the inverse function of $f$.

**Table 2.** Lab test results

| Composition of concrete (C: S: Cs) by series | $\frac{\bar{R}}{R_b}$, MPa | $\frac{R_{bt}(R_n)}{R_{bt}^{*}}$ | Type of test, number of specimens, pcs | Notch size, $a$, mm | $K_{IC}$, $\text{MPa}\sqrt{m^3}$ | $L_{cr}$, mm | $\sigma_{str}^{*}$, MPa | $\sigma_{str}$, MPa |
|---|---|---|---|---|---|---|---|---|
| Series I 1:2:0.65 | 22.0/22.3 | 1.80(2.24)/1.35 | cracking, 18 | 0 | 0 | 0 | - | - |
| | | | | 10 | 16.22 | 1.05 | | |
| | | | | 20 | 21.55 | 1.76 | | |
| | | | | 40 | 27.54 | 2.27 | | |
| | | | | 60 | 32.64 | 3.62 | | |
| | | | | 100 | 38.81 | 4.78 | | |
| | | | stretching, 12 | 50 | 41.70 | 1.17 | 0.37 | 1.79 |
| | | | bending, 12 | 50 | 49.20 | - | 0.83 | 2.12 |
| Series II 1:2:0.65 | 19.7/22.7 | 2.03(2.98)/1.60 | cracking, 18 | 0 | 0 | 0 | | |
| | | | | 10 | 15.98 | 1.0 | | |
| | | | | 20 | 21.61 | 1.68 | | |
| | | | | 40 | 28.11 | 2.71 | | |
| | | | | 60 | 28.89 | 2.81 | | |
| | | | | 100 | 38.40 | 4.53 | | |
| | | | stretching, 12 | 50 | 46.11 | 1.32 | 0.44 | 2.03 |
| | | | bending, 12 | 50 | 49.19 | - | 0.83 | 2.98 |
| Series III 1:2:0.844 | - | (3.16)/1.48 | cracking, 12 | 60 | 36.23 | 3.52 | 1.04 | 1.48 |
| | | | bending, 12 | 50 | 55.17 | - | 0.93 | 3.03 |
| Series IV 1:2:1 | 26.7/24.3 | 3.05(2.65)/1.22 | cracking, 12 | 60 | 36.97 47.59 | 4.9 | 1.06 | 1.22 |
| | | | bending, 12 | 50 | 33.01 | - | 0.78 | 2.65 |
| Series V 1:2:0.63 | 25.5/25.2 | 1.84(1.97)/1.34 | cracking, 6 | 60 | 33.01 | 3.97 | 0.95 | 1.25 |
| | | | cracking, 6 | 60 | 33.05 | 3.54 | 0.95 | 1.34 |
| Series VI 1:2:1 | -/24.2 | -/1.25 | stretching, 6 | 50 | 43.05 | 1.22 | 0.39 | 1.85 |
| | | | bending, 6 | 50 | 43.02 | - | 0.72 | 2.01 |
| Series VI 1:2:3 | 21.2/18.3 | 1.94(-)/ 1.48 | cracking, 6 | 60 | 37.52 | 3.69 | 1.08 | 1.48 |
| | | | stretching, 6 | 50 | 46.45 | 1.22 | 041 | 1.94 |

For each type of test, the working formulas for calculating the KIC and the corresponding correction functions f(d/S) are used as follows:

$$K_{IC}^{ben}=\frac{6M}{bH^2}\sqrt{\alpha}\,[1.99\text{-}2.47(a/a)+12.97(a/d)^2-23.17(a/d)^3+24.8(a/d)^4]$$

$$K_{IC}^{str} = \frac{P}{bH}\sqrt{\alpha}[1.99\text{-}0.41(a/a) + 18.7(a/d)^2 - 38.48(a/d)^3 + 53.85(a/d)^4] \quad (2)$$

$$K_{IC}^{crack} = \frac{2P}{Hd}\sqrt{\frac{\alpha}{2\pi}}[1+\frac{3}{2}(a/d)^2 + \frac{3}{4}(a/d)^6 + \frac{3}{64}(a/d)^8]$$

The results of calculations for heavy concretes using these formulas are summarized in the graphs in Fig. 4, where one can see that the KIC value increases with increasing notch-crack length.

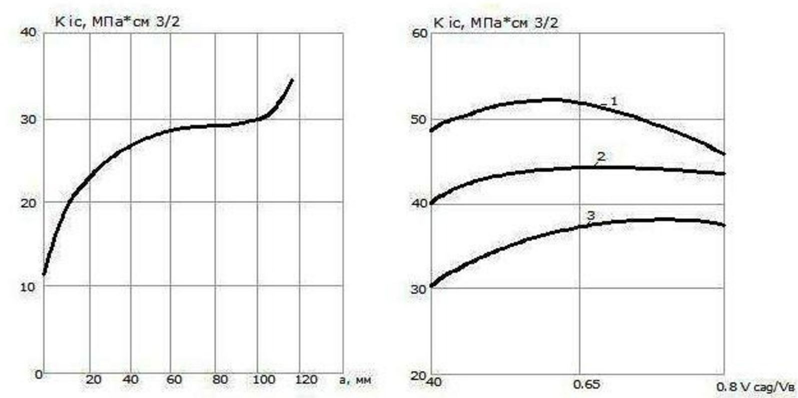



**Fig.4.** Dependence of KIC on notch length and expanded clay content: I- bending, 2- stretching, 3-cracking.

For different test schemes with the same size of notches the $K_{IC}$ values are different. However, within $a$ = 40...80 mm this change is insignificant, which indicates that the Griffiths-Irvin theory can be applied to the processes of lightweight concrete fracture. The influence of changes in the volumetric content of coarse aggregate on the value is also insignificant and can be neglected within the limits of the tested compositions.

The available studies on the influence of various factors on the value of $K_{IC}$ for cement stone, as a material subject to the laws of linear elastic fracture mechanics, can be summarized as follows. The value of $K_{IC}$ for cement stone is less than for mixtures and concrete. The works [5, 7, 8, 9, 10, 11] show that the fracture toughness of $K_{IC}$ mixtures increases with increasing sand content, as well as the roughness and angularity of its particles, which is also associated with a change in the nature of fracturing. Increasing the coarse aggregate content of the concrete also increases the $K_{IC}$ value. Fracture toughness increases strongly in the first 10-15 days of cement hardening, remaining almost constant thereafter. The increase of W/C in the range of 0.3...0.5 reduces the value of $K_{IC}$ by 20-25%. The growth of the loading rate by several orders of magnitude leads to an insignificant (3...5%) increase in the value of $K_{IC}$. Values of specific surface energy used in the calculation formulas can be estimated, according to the work [12]; $\gamma_m$ - varies depending on the test methods in the range from 3 ~15 J/m$^2$; $\gamma_{ad}$ varies from 17 J/m$^2$ for limestone to – 127 J /m$^2$ for sandstone; $\gamma_c$ is approximately (0.4...1.0)$\gamma_m$.

## 4 Conclusions

Generalization of the available experimental data on determining the parameters of crack resistance of concrete and its components allows us to draw the following conclusions:

1. These characteristics strongly depend on the structure and type of material. The scatter of $K_{IC}$ and $G_{IC}$ values for concrete is greater than for mixtures and cement stone [4,5]. In experiments with different loading schemes, materials, and sample sizes with equal values of W/C there was a clear tendency to decrease $K_{IC}$ and $G_{IC}$ values in the transition from concrete to mixtures matrix and from it to cement stone. As the volume of dense aggregate increases, these values increase, but for lightweight concretes such an increase leads to a decrease in $K_{IC}$ and $G_{IC}$. When changing the content of dense sand up to 50%, the $K_{IC}$ value almost does not change. Increasing the maximum aggregate size $d_{max}$ to 30 ... 40 mm leads to an increase in the $K_{IC}$. The influence of aggregate coarseness also manifests itself in the fact that with an initial crack smaller than $d_{max}$, the sensitivity of the sample to notch decreases sharply and the strength of such a sample may not be inferior to the sample without incision. The value of $K_{IC}$ during unstable crack growth is less than during its steady growth. As the ratio $d_{max}$ to the sample size decreases, the maximum values of $K_{IC}$ during stable and unstable crack growth converge.
2. Fracture is usually brittle (more brittle for cement stone than for concrete and mixtures). Cement stone is more sensitive to cuts than mixtures and concrete.
3. As well as strength, the $K_{IC}$, $\gamma$ and $G_{IC}$ decrease as the W/C increases. However, there is evidence that this is true only for cement stones and mixtures. The effect of environmental moisture, which causes an adsorptive lowering of concrete strength, affects the reduction of surface energy, which facilitates the formation of a pre-fracture zone due to the adsorption of water through cracks.
4. As expected, the KIC increases with age, and in contrast to the strength, its maximum values are reached within the first month of hardening.
5. Sample geometry and the size of the crack or notch have a clear influence on the fracture parameters. The sharpness and width of the notch affect the ease or difficulty with which the crack develops from its end. However, with slow crack growth, the type of notch had no significant influence, and the $K_{IC}$ parameters for kerf cuts and embedded slots were practically equal. As the crack length increases, the $K_{IC}$ increases. This dependence is more pronounced for concretes with finer aggregate. In many experiments, the graph of this dependence has a stable horizontal section, which indicates the constancy of $K_{IC}$ values for a certain region of crack length change and the possibility of applying Griffiths - Irwin theory in this area. The mentioned area of stability of $K_{IC}$ values at identical specimens and test schemes decreases with the transition from cement stone to mixture and from it to concrete.
6. The value of $K_{IC}$ increases with the age of concrete with the same regularity as the strength under short-term static loading. Under prolonged loading, the $K_{IC}$ limit values may be observed at a lower stress level but over a longer period of time (t - T). This phenomenon is associated with creep and prolonged strength of the material and indicates the dependence of fracture parameters on the rheological properties of concrete. The stress state around the aggregate grains is influenced by neighboring grains, it is less pronounced for concretes with lower relative aggregate content, since at a distance more than $K_{IC}$ from the inclusion they have little effect on the stress state of the matrix.

## References

1. Ashrabov A.A. New methods and models in the mechanics of reinforced concrete. Tashkent, Spectrum Media Group, 2014. Pp.76-96.
2. Bazhenov Yu.M. Concrete technology. 2nd ed. reslave. Moscow, 1987, 415 p.
3. BazhenovYu.M., Gorchakov G.I., Alimov L.A., Voronin V.V. Reception of concrete of the set properties. Moscow. 1978. 83p.
4. Pak A.P., Trapeznikov A., Yakovleva E.N. Investigation of the crack resistance characteristics of concrete under axial tension and bending of specimens with notches. Leningrad .: Izvestia VNIIG. Russia T. 1, Pp.20-27.
5. Pak A.P. Study of concrete strength under biaxial compression. Izv. VNIIG, Moscow 1968, No. 87, Pp.36-45.
6. Panasyuk V.V. Ultimate balance of brittle bodies with cracks. Kiev, 1968. p. 82
7. Zaitsev Yu.V. Destruction mechanics for builders. Destruction mechanics for builders. Moscow: VS. 1991. p. 288.
8. Zaitsev Yu.V. Modeling of deformations and concrete strength by the methods of fracture mechanics. Moscow, 1982. p.196.
9. Scherbakov E.N. Physical and phenomenological foundations for predicting the mechanical properties of concrete for the calculation of reinforced concrete structures. Diss. Doctor of Technical Science, Moscow., 1987. p.150.
10. Ashrabov A.A., Muratov O.A. Calculation of the service life of concrete and reinforced concrete structures of hydraulic structures on the basis of crack resistance. NEWSLETTER of TarSU named after M.Kh. Dulakhi. International scientific journal "Nature Management and Anthroposphere Problems" №4, 2014, Pp.22-30.
11. Peresypkin V.P. Calculation of core reinforced concrete elements. Moscow, 1988.
12. Cottrel A.A. Mechanical Properties of Materials. Wiley. New York., 1964. USA.
13. Kurasova G.P. *K* To the standardization of the strength of lightweight concrete under axial compression. Stroyizdat. Moscow. 1975. Pp.16-35.
14. Muratov O.A, Ashrabov A.A., Yangiev A.A. Service life of intake part catastrophic spillway. International Journal of Multidisciplinary Research And Publication (IJMRAP). Volume1, Issue 10, 2019. Pp.17-24.
15. Muratov O.A, Ashrabov A.A., Yangiev A.A. Life prediction for spillway facility side wall. XXI Int. Sc. Conf. on Advanced in Civil Engineering. 2019. Pp. 56-76.
16. Muratov O.A., Ashrabov.A.A. On the influence of structural defects on the stress – strain state and crack formation in concrete. Mezh. scientific J. TarSU named after M.Kh.Dulakhi "PiPA" ,. Issue number 4. 2014. Pp. 11-20.
17. Glanvill. J., Neville A., Sommerville. G. Prediction of Concrete Durability. E & FN, 1996, 208p.
18. Petersson P.E. Crack grows and development of fracture zone in plain concrete and similar. Lund. of Technology, Report TVBM-2006, 174p.
19. Schiessl P., Gehlen C. New approach of service life design for concrete structures. Proc. Int. Workshop on Durability of Reinforced Concrete under Comb, Mech., and Clim., Loads, Qingdao, 2005. Pp.3–14.
20. Schiessl P., Gehlen C. New approach of service life design for concrete structures. Proc. Int. Workshop on Durability of Reinforced Concrete under Comb, Mech., and Clim., Loads, Qingdao, China. 2005. Pp.3–14.