FRACTURE PROPERTIES OF MODE II FOR HIGH STRENGTH CONCRETE

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Abstract – *The aims of the present paper are to investigate the effect of some variables on fracture properties (mode II) for high strength concrete (HSC) and to provide some experimental data that can be useful in engineering practice. The investigation used three levels of concrete grades (30, 50, 70 MPa), two maximum nominal size (10, 20 mm), two coarse aggregate types (dolomite, gravel). In each case, mechanical properties and fracture properties for mode II crack (K_{II}) were determined. A comparison test results were performed. It was found that: (a) The fracture properties (K_{II}) increase as the maximum size of coarse aggregate and compressive strength of concrete increase; (b) K_{II} is observed to be higher in gravel mixes than in dolomite mixes by 10% and (c) Test results showed that K_{II} of concrete generally increases as KI increases. It is also found that K_{II} is higher than K_I by 25 %.*

Key Words: High strength concrete; fracture properties; mode II, stress intensity factor, fracture toughness

1.INTRODUCTION

The fracture mechanics science is a new concept between mechanical and structural view of material study. Fracture mechanics is based on the assumption that all engineering materials contain cracks from which failure starts. The estimation of the remaining life of machine or structural components requires knowledge of stresses caused by the introduction of cracks in conjunction with a crack growth condition.

In recent years, the construction industry has shown significant interest in the use of high strength concrete (HSC). This is due to the improvements in structural performance. In the last two decades, its use has been extended to high rise buildings structures and has received enormous research attention. The research on the fracture behavior of mode II for HSC has been limited. Failures have occurred for many reasons, including uncertainties in the loading or environment, defects in the materials, inadequacies in design, and deficiencies in construction or maintenance.

Design against fracture has a technology of its own, and this is a very active area of current research. After a quarter century of intense research, it has now become clear that the use of fracture mechanics can yield safer and more efficient design against all kinds of brittle failures of concrete structures. K_{II} have been found to be a fracture mechanics parameter to describe the resisting properties of concrete to cracking. KII can be considered a material property characterizing the crack resistance in case of a shear stress acting parallel to the plane of the crack and perpendicular to the crack front. When the value of K_{II} increases, the resistance to cracking increases, and these lead to improving the fracture behaviors. A total of 108 cubes specimens have been submitted to tests. K_{II} was obtained by experimental method according to failure load and calculated by using finite element method. The parameters obtained in the experiments can provide a database for calibrating of numerical models and the development of design criteria for HSC structures.

1.1 The stress Intensity Factor

Stress analysis by the finite element method revealed that three-point bend specimen undergoes large tensile stresses which are responsible for the propagation of the crack in the tensile zone remote from the loading. This is confirmed by the laboratory tests in which the specimens failed by crack propagation in the opening mode of fracture resulting in the specimen broke into two halves.

The numerical analysis also enabled the computation of the stress intensity factor K by several methods. These methods may be placed into two categories, that is, the ’displacement ’ and ’energy ’ approaches. Whereas it was found that all methods can be used to obtain accurate estimates for K, from the point of view of the amount of post-processing operations required on the fundamental data obtained from the finite element analysis, it is preferable to use the energy approach.

Amongst the energy methods, there are two methods namely, strain energy release rate per unit crack extension area (G) and the energy integral (*J*) (1). Both of these methods utilize the results for the displacements and their derivatives at regions remote from the crack tips, and thus relatively larger errors associated with the results for points near the crack tip are avoided.

$$EJ = EG = K^2 (1 - v^2) \quad (1)$$

1.2 Fracture Toughness and Fracture Energy

The fracture toughness K_c is defined as the critical value of the stress intensity factor, which is computed from the product of K (per unit compressive load) and the load at failure. The equivalent critical values of J and G are related to K_c by

$$EJ_c = EG_c = K_c^2 (1 - \nu^2) \qquad (2)$$

Equation (2) is applicable to plane conditions and E and ν are the elasticity modulus and Poisson's′ ratio, respectively.

2. OBJECTIVE AND SCOPE

The main objective of this investigation was to study the effect of increasing compressive strength on stress intensity factor in mode II (K_{II}). K_{II} was calculated by using Finite element analysis using commercial program (4) Comparison between normal strength concrete and high strength concrete for K_{II} were being obtain. The investigation used three levels of concrete grades (30, 50, 70 MPa), two coarse aggregate sizes (10, 20 mm), two coarse aggregate types (dolomite, gravel). This study was getting the relation between concrete mechanical properties and K_{II}. Moreover, relation between K_{II} and stress intensity factor in mode I (K_I) were be obtained.

3. GEOMETRY OF THE TEST SPECIMEN

A practical testing arrangement employing beam specimens was examined. Specimens as shown in Fig. 1 were prepared by means 100 × 100 × 500 beam moulds to get K_I. A 50 mm deep notch was introduced in the center of each beam for each mix. The geometry of a three-point bend test on notched beams is shown in Fig. 1. Also, the specimen used to find the value of K_{II} was cubes 100×100×100 mm supported from the top and bottom as shows in Fig. 2 (3). To ensure a uniform distribution of load, three metal pieces were used around the specimen. One of them was put on the top and its dimensions were 50×100 mm and the others were at the bottom and their dimensions were 25×100 mm. The space between the two notches was 50 mm. The cube specimen was placed at the centerline of machine to ensure that the axis of the load was exactly passing through the centerline specimens. The finite element analysis was performed using the ANSYS package (4). The recommended element type for a two-dimensional fracture model is PLANE82, the eight-node quadrilateral solid as shown in Fig. 3 (4). The first row of elements around the crack tip should be singular, as shown in Fig 3. A plane strain 8-node quadrilateral element was used to form the mesh. The two dimension model using PLANE82 is created by automatic mesh generation Fig. (4, 5). Due to the symmetry of geometry and boundary conditions, only half of the specimen was considered. Twelve different mixes were considered for investigation.

Tests are used to obtain failure load for all mixes. The ANSYS input data was written in terms of variable-failure load for each mix to change the variables easily. The data is used to get critical stress intensity factor K_{1C}, K_{IIc} by displacement extrapolation methods (ANSYS, 2004).

4. EXPERMENTAL PROGRAMME

Detailed information about the materials used and their characteristics are given in this section.

Cement Portland cement has been found to be adequate for production of high strength concrete. Experience Physical and mechanical properties of cement were measured according to Egyptian standard specification (ESS 373-1963)

One of the difficulties associated with applying fracture mechanics theory to concrete materials is the effect of slow crack growth that may take place during testing (an elastic – plastic analysis may be required). It was shown (2), however, that the geometry of the test specimen used here results in unstable crack propagation and that very little crack growth occurs prior to its failure. The load-deflection response is linear up to the failure point, and linear elastic conditions are assumed to prevail.

For linear elastic situations it can be shown (2) that the value of *J* is identical to the strain energy release rate G, giving

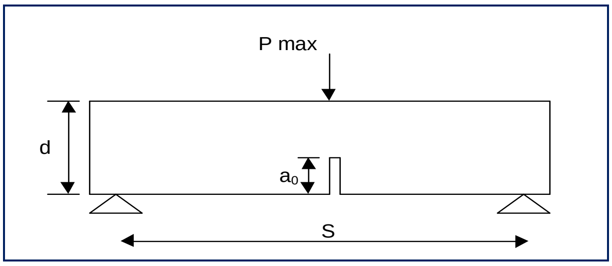

Fig -1: Sketch of specimens of three points bend beam

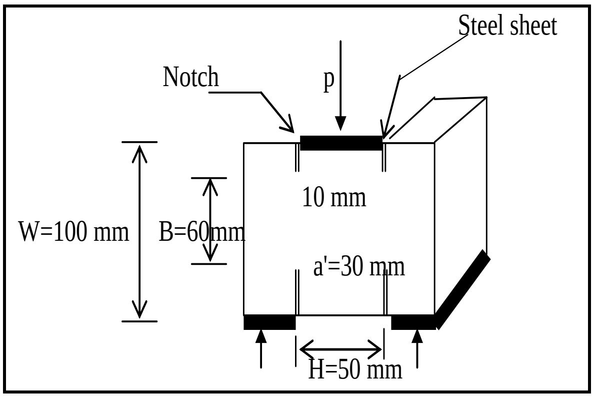

Fig -2: Sketch of specimens of mode II

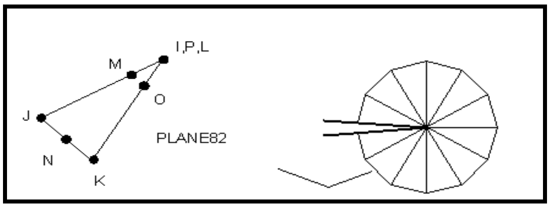

Fig -3: The element row around the crack tip

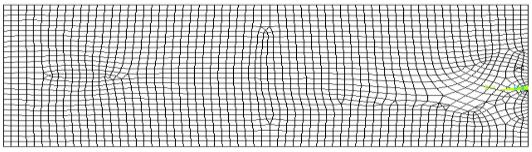

Fig -4: Finite element mesh for the model for KI

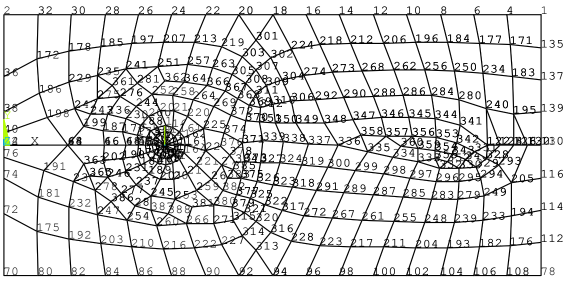

Fig -5: Finite element mesh for the model for K_{II}

Mineral admixtures. Silicafume (sf) was used in this programme and particular care was taken over curing since the pozzolanic reaction takes place over a longer time.

Aggregate Gravel and dolomite were used as coarse aggregate (CA). The coarse aggregate had a maximum size of 10 mm and 20 mm. Coarse aggregate batches employed were clean, free from impurity matter. The main properties of coarse aggregate were measured according to Egyptian standard specifications (ESS 1109-1971). The fine aggregate (FA) used in concrete was desert sand. It was clean and almost free from impurities, silt, lay and saltiness. Main properties of sand were measured according to the Egyptian standard specification (ESS 1109-1971). Table 1 shows the physical and mechanical properties of aggregates.

Chemical Admixtures (Superplasticisers). High range water reducer complies with ASTM C494 type F was used. The admixture is a brown liquid ready to use directly during concrete mixing.

4.1 Mix design

In order to establish a basic range of mixes a number of trial mixes were investigated. From the results of trial mixes, eight mixes were selected for the main programme. These mixes type of coarse aggregate, size of aggregate, silicafume content as variables. In addition, four reference mixes in the normal strength range with cement contents of 350 kg/m³ and water/cement ratio of 0.5 were used. Table 2 gives the composition of selected mixes.

4.2 Mixing, Casting and Curing

Mixing was carried out in a 150-litre revolving paddle pan mixer. The concrete was cast into steel moulds using a minimal amount of mould oil and compacted using a vibrating table. The fresh concrete had a slump greater than 100mm as measured with a standard 305 mm cone. The specimens were demoulded and of water curing commenced after 24 hours. Twelve mixes were selected for the main programme. These mixes involved a size and type of coarse aggregate ratio (CA), cement content and silicafume content. Table 1 gives the composition of selected mixes. The fracture specimens were notched by using electric saw. The details of all specimens are shown in Table 3.

Table -1: Main properties of coarse and fine aggregates.

Type	Specific gravity	Unit weight, kg/m³	Void, %	Fineness Modulus	Clay, silt , dust (by weight) %
Sand	2.60	1650	36.5	2.5	1
Dolomite 10 mm	2.68	1619	39.50	5.6	0.3
Dolomite 20 mm	2.66	1617	39.21	6.85	0.28
Gravel 10 mm	2.63	1690	36.3	5.83	0.35
Gravel 20 mm	2.60	1650	37.35	5.6	0.28

4.3 Test specimens and instrumentation

Consistency Test: Consistency of all mixes was measured by conventional slump test method. The test was carried out according to BS 1881 part 102: 1983. All mixes were designed to have slump rang from (100 ± 20) mm.

Compressive strength: Compression tests were carried out on 150 mm cubes according to British Standard BS 1881: Part 116:1983. (5)

Splitting tensile strength:-Tests were carried out on 150 diameter and 300 mm length cylinders according to British Standard BS 1881: Part 117:1983 (5).

Flexural strength:- Tests were carried out on 100×100×500 mm beams according to British Standard BS 1881: Part 118:1983 (5).

Fracture tests The investigation used two types of specimens, notched beams for Mode I and notched cubes for Mode II as shown in Fig. 6.

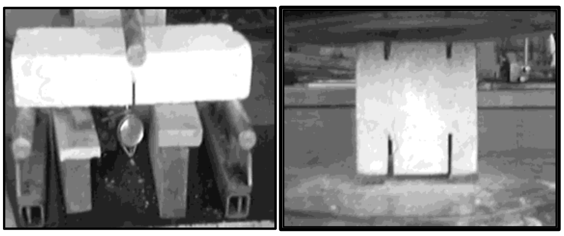

Fig -6: Test setup

Table -2: Composition of mixes used in main programme

Series	Mix No.	C kg	Coarse Aggregate			SP/ (C+ SF) %	SF/ (SF+C) %	W/ (SF+C)
			CA/ CA+FA	type	Size mm			
I NSC	M1	350	0.65	dolomite	10	0	0	0.5
	M2		0.60	dolomite	20	0	0	0.5
	M3		0.65	gravel	10	0	0	0.5
	M4		0.60	gravel	20	0	0	0.5
II	M5	450	0.65	dolomite	10	2	10	0.32
	M6		0.65	dolomite	20	2	10	0.32
	M7		0.65	gravel	10	2	10	0.32
	M8		0.65	gravel	20	2	10	0.32
III HSC	M9	500	0.62	dolomite	10	2.5	15	0.28
	M10		0.60	dolomite	20	2.5	15	0.28
	M11		0.62	gravel	10	2.5	15	0.28
	M12		0.60	gravel	20	2.5	15	0.28

Table -3: Test Specimens

Kind of test	Shape of specimens	Geometry of specimens	Numbers of specimens
Compressive Test	Cubes	150 mm	72
Splitting test	Cylinders	150×300 mm	36
Flexural Test	Beams	100×100×500 mm	36
Mode I crack	Notched beam	100×100×500 mm	36
Mode II crack	Notched Cubes	100 mm	36

5. EXPERIMENTAL RESULTS AND DISSCUCTION

According to the experimental and theoretical studies that were conducted in the present study, the results summarized as follows in Table 4.

No	Type	Size (mm)	f_{cu} (MPa)	E (MPa)	f_{felx} (MPa)	f_{sp} (MPa)	K_I MPa √ mm	K_{II} MPa √ mm
m1	Dolomite	10	31.25	27671	4.3917	2.248	27	35
m2		20	31.66	27852	4.4513	2.58	32	37
m3	Gravel	10	31.66	27852	4.6288	2.348	33	38
m4		20	31.77	27560	4.8288	2.6831	35	42
m5	Dolomite	10	49.50	34826	7.1975	4.371	38	50
m6		20	49.55	34738	7.245	4.6435	40	52
m7	Gravel	10	49.58	34597	7.525	4.341	42	53
m8		20	49.83	34632	7.638	4.6246	45	55
m9	Dolomite	10	68.56	40870	10.315	6.057	47	58
m10		20	68.83	41088	10.45	6.152	49	59
m11	Gravel	10	69.16	41148	10.523	6.0265	53	61
m12		20	69.89	41151	10.96	6.763	56	65

Table -4: Study Results.

Effect of Concrete Compressive Strength on K_{II}

From the results it was found that, the increase of compressive strength leads to a significant increase of K_{II}. It can be observed that the increase of f_{cu} from 30 to 50 and 70 MPa leads to a significant increase of K_{II} by 35 % and 60 % respectively compared with f_{cu} = 30 MPa as shown in Chart 1.

Effect of Coarse Aggregate Size on K_{II}

The value of K_{II} increases as the maximum nominal size of coarse aggregate increases. Concrete containing coarse aggregate with 20 mm size has higher K_{II} than concrete containing 10 mm size by 10% - 12 % as shown in Chart 1.

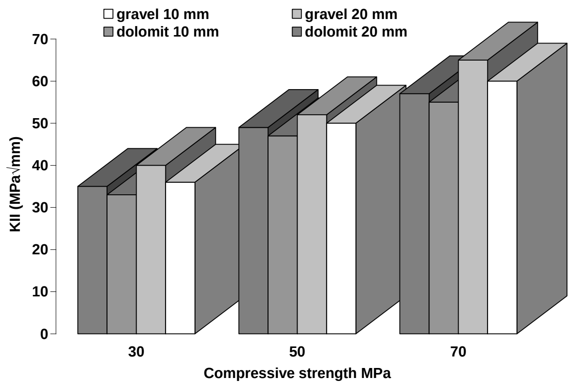

Chart -1 Relationship between K_{II} and compressive strength

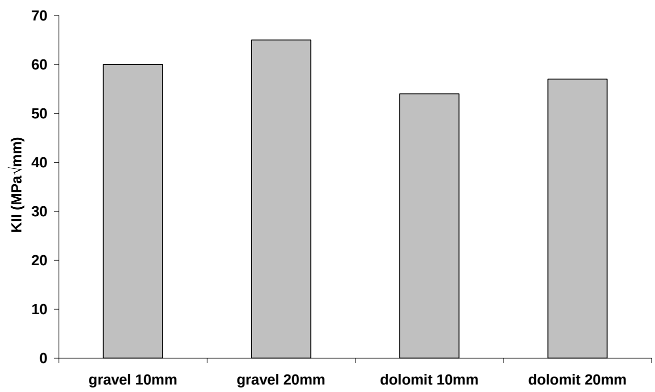

Chart -2: Relationship between KII and aggregate type.

Effect of Coarse Aggregate Type on K_{II}

Concrete containing gravel as coarse aggregate has K_{II} higher than concrete containing dolomite by 9 % in case of NSC and 11% in case of HSC. From the results, it is found that the type of aggregate has an influence on the fracture properties of high and normal strength concrete as shown in Chart 2.

The Relation between Tensile Strength and K_{II}

The test results indicate that the values of K_{II} increase as the tensile strength increases as shown in Table 4 and Chart 3. From test results, it is possible to get emperical equations for K_{II} in term of f_{sp} ,. Where f_{sp} in MPa and K_{II} in MPa√mm.

$$K_{II} = 9.875 \times f_{sp} + 19.769 \qquad 2 \text{ MPa} < f_{sp} < 7 \text{ Mpa} \qquad (3)$$

The Relation between Flexural Strength and K_{II}

The test results show that K_{II} increase as the flexural strength increases as shown in Table 4 and Chart 3. From test results, it was found that empirical equations for K_{II} as the following:-

$$K_{II} = 6.27 \times f_{flex} + 16.45 \qquad 4\ MPa < f_{flex} < 11\ MPa \qquad (4)$$

The Relation between Modulus of Elasticity and K_{II}

The values K_{II} increase when the modulus of elasticity increases as shown in Table 4. From test results it is possible to get empirical equations between K_{II} and E.

$$K_{II} = 0.0028 \times E - 33.45 \qquad 27\ GPa < E < 41\ GPa \qquad (5)$$

The Relationship between K_I and K_{II}

Test results showed that K_{II} of concrete generally increases as K_I increases. It is also found that K_{II} is higher than K_I by 25 % as shown in Chart 4. From test results it is possible to get empirical equation for K_I in term of K_{II}

$$K_{II} = 1.1763 \times K_I + 2.4842 \qquad (6)$$

Where K_{II} and K_I in MPa $\sqrt{mm}$

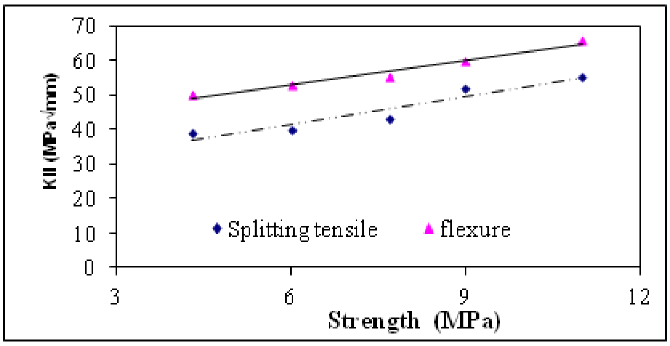

Chart -3 Relationships between K_{II}, tensile strength and flexural Strength

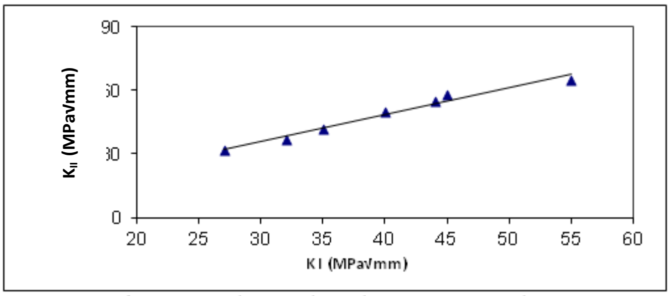

Chart -4 Relationships between K_I and K_{II}

6. CONCLUSION

An experimental study has been conducted to investigate fracture properties of K_{II} , of HSC and NSC with normal aggregate (Gravel – dolomite). The following conclusions may be drawn.

1-The increase of compressive strength leads to a significant increase of K_{II}. It can be observed that, the increase of compressive strength from 30 to 50 and 70 MPa leads to an increase in K_{II} by 40 and 85 % respectively from the K_{II} value at f_{cu} 30 MPa.

2-For all mixes, test results showed that the value of K_{II} of concrete generally increase as the maximum nominal size of coarse aggregate increases.

3-The result showed that the value of K_{II} for concrete with gravel aggregate was higher by 9% than dolomite.

4-Test results showed that K_{II} of concrete generally increases as K_I increases. It is also found that K_{II} is higher than K_I by 25 %.

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