Discussion of "Using Crushed Clay Brick as Coarse Aggregate in Concrete" by Fouad M. Khalaf

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The discussers wish to congratulate the author for his article on using crushed clay brick as coarse aggregate in concrete, which is an important contribution to concrete technology for sustainable development. Moving toward the sustainable development of recycled concrete aggregate (RCA) has the potential to be used in the same types of concrete as natural coarse aggregates.

The discussers, being stimulated by the author's work, would like to remark the following subject.

According to Fig. 2 in the paper, the author stated that for a given water/cement ratio, 28 days' compressive strength of concrete containing crushed brick as the coarse aggregate is correlated to brick unit compressive strength. Namely, the stronger the original brick, the stronger the compressive strength of the concrete made with aggregates produced by crushing these bricks, whereas aggregates in concrete exist in granular form. In other words, strength of aggregates in granular form should be expressed to assess aggregate effect on concrete strength for a certain water/cement ratio. Chang and Su (1999) presented a formula to directly calculate the compressive strength of a coarse aggregate particle. They found that the influence of aggregate strength on the compressive strength of high-strength concrete is significant. The drawbacks of this test are that there is a great variation in measured compressive strength [coefficient of variation varied from 20–42%, which can be considered unacceptable by Arıoğlu et al., (2006)] and the preparation of coarse aggregate specimens for compression test is laborious. Under these conditions, indirect means of assessing granular strength (aggregate crushing value, impact value, and 10% fines aggregate test) seem to be practical in concrete sector.

In our discussion, to define the coarse aggregate effect on 28 days' concrete compressive strength for a given water/cement ratio by weight, the aggregate factor concept was used (Arıoğlu, and Köylüoğlu 1996). The aggregate factor were estimated from the Bolomey 1926 and the Feret Formulas 1892 (Popovics 1998).

As all explanation related to estimation of the aggregate factor corresponding the Bolomey and Feret formula are given in Table 1 and 2, respectively. They will be not repeated here. The aggregate impact value (IV) as a measure of aggregate granular strength was accepted in this discussion. These values belonging to brick/aggregate type used were obtained from Table 2 in Khalaf and DeVenny (2005). Since two values [the aggregate factor (K_a) and aggregate impact value (IV)] represent quite same measures of compressive strength, correlation between K_a (see Table 1 and 2) and IV values may be meaningful. Fig. 1 and Fig. 2 display relationships between two values in question by means of regression analysis.

The regression equations for the relationships demonstrated in Figs. 1 and 2 are expressed as follows

$$K_a = 0.92 \,.\, e^{-0.01.\mathrm{IV}} \tag{1}$$

r=0,883

$$K_a = 9.02 \,.\, e^{-0.01.\mathrm{IV}} \tag{2}$$

r=0,883 where r=coefficient correlation.

In passing, it should be stressed here that indirect measures of aggregate strength are markedly influenced by the flakiness index of coarse aggregate (FI) according to Irvine and Montgomery (1999), and there is a reasonable correlation between the FI and the IV values. As the FI value increases the IF value also increases. In addition, 28 days' concrete compressive strength for M1 and M2 mixes (Tables 1 and 2) can be estimated from the following formulas

The Bolomey formula

Table 1. Estimation of Aggregate Factor Belonging the (Bolomey Formula) for Normal- and High-Strength Concrete (M1 and M2).

Aggregate type[a]	Concrete compressive strength at 28 days, N/mm^2		Aggregate factor, Ka[d]			Aggregate impact value, IV[e](%)
	α=0.55[b]	α=0.40[c]	α=0.55	α=0.40	Aritmetic mean	
1	37.6	53.8	0.681	0.640	0.660	31
2	40.5	59.2	0.733	0.704	0.718	25
3	42.9	63.6	0.776	0.756	0.766	19
4	46.7	66.7	0.845	0.793	0.818	19
5	45.7	66.8	0.827	0.794	0.810	9

[a]1=common solid-clay brick; 2=five-slot brick; 3=three-slot brick; 4=ten-slot brick; 5=granite.

[b]$\alpha = W_w/Wc$, water/cement ratio: mix number: M1 (normal-strength concrete).

[c]Mix number: M2 (high-strength concrete).

[d]Aggregate factor corresponding to the Bolomey formula. This value was calculated from $K_a = f/f_{ce} \cdot [(W_c/W_w) - 0{,}5]$ where f=concrete compressive strength at 28 days-100 mm cube; f_{ce}=28 days ordinary Portland cement compressive strength W_c; W_w=weight of cement and water used in mix (kg/m^3).

[e]These values "IV" were taken from Table-2 reported by Khalaf and DeVenny (2005).

Table 2. Estimation of Aggregate Factor (Feret Formula) for Normal- and High-Strength Concrete (M1 and M2)

Aggregate type	Concrete Compressive Strength at 28 days, N/mm^2		Aggregate factor, ka[a]			Aggregate impact value, IV (%)
	$\alpha=0.55$	$\alpha=0.40$	$\alpha=0.55$	$\alpha=0.40$	Aritmetic mean	
1	37.6	53.8	6.54	6.40	6.47	31
2	40.5	59.2	7.05	7.04	7.04	25
3	42.9	63.6	7.46	7.57	7.52	19
4	46.7	66.7	8.13	7.94	8.03	19
5	45.7	66.8	7.95	7.95	7.95	9

[a]Aggregate factor corresponding to the Feret formula. This value was calculated from $K_a=(1+3,1\alpha)^2/f_{ce}\cdot f$.

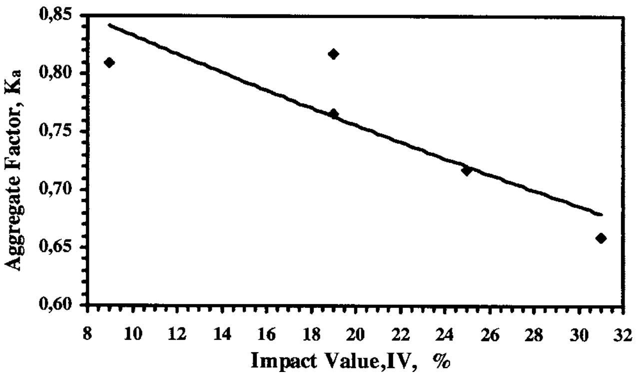

Fig. 1. Correlation between aggregate factor corresponding to the Bolomey formula and aggregate impact value

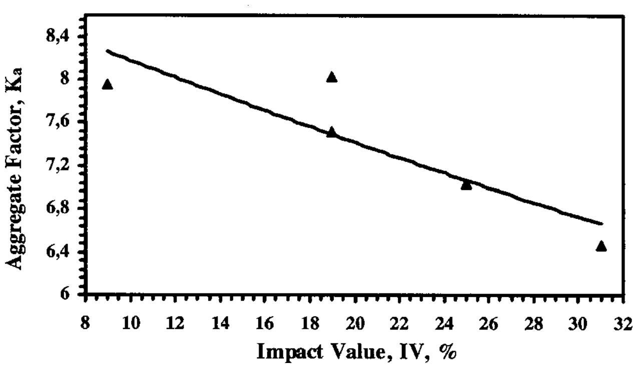

Fig. 2. Correlation between aggregate factor corresponding to the Feret formula aggregate impact value

$$f=0.92\,.\,e^{-0.01\mathrm{IV}}\,.\,f_{ce}\,.\left(\frac{W_c}{W_w}-0.5\right) \tag{3}$$

The Feret formula

$$f=\frac{9.02\,.\,e^{-0.01\mathrm{IV}}\,.\,f_{ce}}{(1+3.1\,.\,\alpha)^2} \tag{4}$$

All notations are given in Tables 1 and 2.

The figures suggest that the correlation between the aggregate factor corresponding to the Bolomey and the Feret formulas and impact value are considerably meaningful, so that the aggregate impact test can be regarded as a practical measure for aggregate strength in granular form. In brief, for the same water/cement ratio, the concrete containing coarse aggregate with a low impact value (indicated by a high aggregate compressive strength) produces a high compressive strength. The same finding was also reported in Alexander M. and Mindess S. (2005) from Goldman and Bentur (1993) and Irvine and Montgomery (1999).

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Issues Related to Aggregate Impact Value and Concrete Compressive Strength

The writer would like to thank the discussers for their remarks about the paper and their contribution to prove that the aggregate impact test is a simple, acceptable, and practical test to measure aggregate strength in its granular form.

The aggregate impact value (AIV) determines from the impact test gives a relative measure of the resistance of an aggregate to sudden shock or impact. In some aggregates these can be differ from its resistance to solely applied compressive strength load. However, the test gives a good indication to the strength of aggregate in its granular form.

The impact test is carried accordance to BSI (1990) and is applicable to aggregates passing the 14.0 mm test sieve and retained on a 10.0 mm test sieve. The test specimen is compacted in a standardized manner into an open steel cup. The specimen is then subjected to a number of standard impacts from a dropping weight. This action breaks the aggregate to a degree that is dependent on the impact resistance of the material. This degree is assessed by a sieving test on the impacted specimen and is taken as the AIV.

The discussers kindly proved that the AIV determined by testing can be correlated to an aggregate factors (K_{a1} and K_{a2}) corresponding to the Bolomey and Feret formulas. They then, based on my test results for the compressive strength of concrete and the AIVs, derived two interesting formulas [Eqs. (3) and (4)] of the discussion to determine the compressive strength of concrete.

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Discussion of "Tension Stiffening Model for Concrete Beams Reinforced with Steel and FRP Bars" by Rim Nayal and Hayder A. Rasheed

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Renewed interest in deflection analysis of concrete structures is due to growing application of advanced composite reinforcing materials. The paper by Nayal and Rasheed has reflected the differences in tension stiffening effect of concrete structures reinforced with fiber-reinforced polymer (FRP) and conventional steel bars. Tension stiffening due to its complexity and significance to numerical results is probably the most important factor in short-term deflection analysis.

The authors of the paper using experimental data of concrete beams from various reports have performed an inverse parametric analysis and defined the key parameters of the assumed tension stiffening relationship given in the original paper. It was shown that huge quantitative difference between the relationships obtained for the beams reinforced with steel and FRP bars can be reflected by parameter F_t. This parameter multiplied by the cracking strain, ε_{cr}, specifies the limit strain of the descending branch of the tension stiffening relationship. Based on results of the parametric analysis, F_t was assumed to be 10 and 100 for steel and FRP bars, respectively.

As noted in the original paper, Kaklauskas and Ghaboussi (2001) have proposed a method for deriving a tension stiffening relationship from experimental moment-curvature or other moment-deformation diagrams of reinforced concrete bending members. The stress-strain relationship is computed incrementally from the equilibrium equations for the extreme tensile concrete fiber. The computation is based on an idea of using the previously computed portions of the stress-strain relationships at

Table 1. Tension Stiffening Parameters

Reinforcement	Beam	ρ_s	R_t	F_t
FRP	F1	0.0123	0.37	97.0
	FRP1A-HF	0.0112	0.45	120.0
	Group 3	0.0116	0.40	53.4
	BC2HA	0.0120	0.61	133.3
	Series 1	0.0110	0.46	76.3
Steel	A	0.0066	0.51	15.2 (17.7)[a]
	Group 1	0.0138	0.35	9.1 (8.3)[a]
	B1	0.0037	0.50	22.0 (23.6)[a]

[a]Values of parameter F_t calculated by Eq. (1).

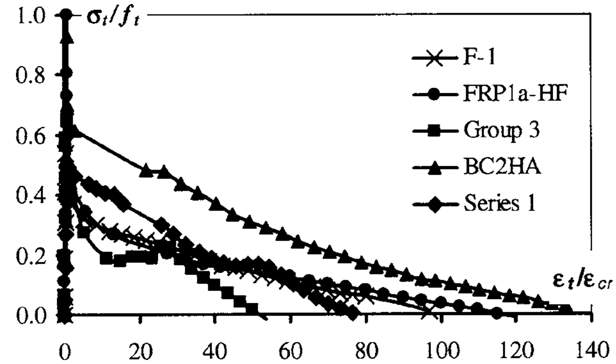

Fig. 1. Tension stiffening relationships for FRP reinforced beams

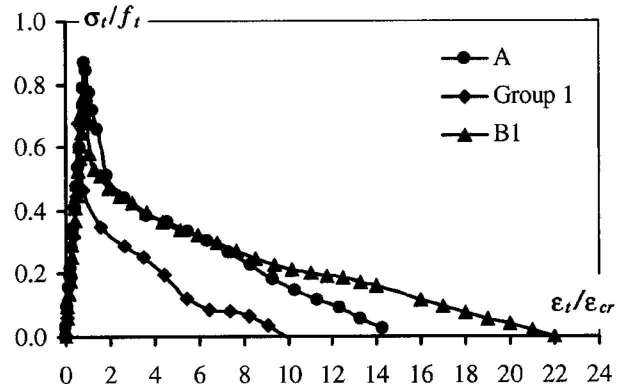

Fig. 2. Tension stiffening relationships for steel reinforced beams

each load increment, to compute the current increments of the stress-strain relationships (Kaklauskas 1998). Using this method, tension stiffening relationships were obtained for the selected beams taken from the original paper and listed in Table 1. Beams FRP-S, due to their small size, were excluded from the group of FRP-reinforced members. One beam with the highest reinforcement ratio (Group 1) and two beams with the lowest reinforcement ratio (A and B1) were taken from the group of steel-reinforced members. The tension stiffening relationships calculated for the beams reinforced with FRP and steel bars are shown in Figs. 1 and 2, respectively. The relationships are given in relative terms, i.e., the stresses were divided by tensile strength, f_t ($f_t = 0.23\sqrt[3]{f_{cube}^2}$, where f_{cube}=compressive strength of 150 mm cube) and the strains were divided by cracking strain, ε_{cr}.

For FRP-reinforced beams (Fig. 1), the authors in general have correctly captured the characteristics of the tension stiffening relationships, and particularly the values of parameters F_t and R_t (see Fig. 4 of the original paper) which are mostly responsible for the numerical results of deflection analysis. Values of these parameters are given in Table 1. Parameter F_t ranging from 53.4 to 133.3, in support to the original findings gave an average value around 100. Parameter R_t taken by the authors as 0.45 can be also considered an accurate assumption.

Present analysis has confirmed huge differences in parameter F_t for the beams reinforced with FRP and steel bars (see Figs. 1 and 2). For the latter, when reinforcement ratio ρ_s is over 1.0%, assumption of $F_t = 10$ is reasonable. However, for the beams with smaller reinforcement ratios, higher values of parameter F_t were obtained (see Table 1). These values were in good agreement with the results of earlier investigations (Kaklauskas and Ghaboussi 2001) resulted in the following relationship:

$$F_t = 32.8 - 27.6\rho_s + 7.12\rho_s^2 \quad (F_t = 6 \text{ if } \rho_s \geqslant 2\,\%) \tag{1}$$

The above relationship was derived for steel-reinforced members and obviously cannot be applied for other reinforcement with different modulus of elasticity. It should be noted that application of Eq. (1) contradicts the idea of using parameter $\lambda = E_s A_s / \# d_b$ as a generalized characteristic of tension stiffening. Values of parameter λ presented in Table 2 of the original paper do not seem to correlate with F_t given in Table 1 or calculated according to Eq. (1). This could be due to the fact that the derived tension stiffening relationships were not free of concrete shrinkage effects. On the basis of tests on tensile steel-reinforced members, Bischoff (2001) has shown that due to shrinkage parameter F_t decreases with increasing reinforcement ratio. Similar results were reported by Kaklauskas and Gribniak (2005) on a basis of numerical investigations of tensile and flexural reinforced concrete (RC) members. Thus, to relate parameters λ and F_t, shrinkage has to be eliminated. On the other hand, shrinkage effects should be negligible for the members reinforced with FRP bars, as the latter have modulis of elasticity comparable to concrete leading to reduced restraint action of reinforcement.

Further research is needed to develop a universal tension stiffening relationship applicable to different reinforcing materials.

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The authors would like to thank the discussers for their interest in the paper. The discussers are also commended for their accurate description of the contribution of the original paper. The discussers have derived R_t and F_t values for some of the experiments highlighted by the authors in the original paper using the authors' tension stiffening model. However, it is evident that these results, in the case of fiber-reinforced polymer (FRP) bars, are inconsistent. A simple examination of these numbers clearly reveals that they are not a function of the reinforcement ratio ρ_f, which shouldn't be the case. The five beams reinforced with FRP had similar reinforcement ratios. Nevertheless, the values of R_t and F_t clearly did not correlate with the reinforcement ratio or $E_f \rho_f$. There was no alternative correlation suggested to help provide the user with guidance. The main purpose of the original paper was to determine accurate parameters for the tension stiffening model for use in future analysis. The discussers' determination of different parameters corresponding to a very narrow range of the reinforcement ratio or ($E_f \rho_f$) defeats the purpose. If for each experiment with comparable reinforcement ratio, one needs to obtain a new F_t parameter, one wouldn't be able to offer a calibrated model for use by analysts. It is interesting to note that the average value of F_t from the calculations performed by the discussers was 96 which is around the 100 determined by the authors for FRP reinforcement—a point mentioned by the discussers. Similarly, the average value of R_t from the calculations performed by the discussers was 0.46 compared to 0.45 determined by the authors. It was also not clear to the authors why the discussers ignored including two of the beams reinforced with steel bars (i.e., No. 1 and A3). For the other two beams (A and Group1), the F_t parameter was accurately reported by the discussers with an average of 12.15 and compared with the 10 selected by the authors in the original paper. The third beam (B1) had a significantly low steel ratio. Accordingly, the authors determined the F_t value of 20 in the original paper, which was confirmed to be 22 by the discussers. The average value of R_t for the three beams is 0.45, which is the same implemented by the authors. It is important to note that the quadratic empirical relationship in Eq. (1), of the discussion, requires the reinforcement ratio to be $\rho_s * 100$ instead of ρ_s. More importantly, this equation does not extend to FRP reinforcement.

As for the parameter λ, it was used to explain the huge difference between the F_t values for steel and FRP, and correlate on average to the 10, 20, and 100 values selected for steel and FRP. This was successfully accomplished by having close λ parameter

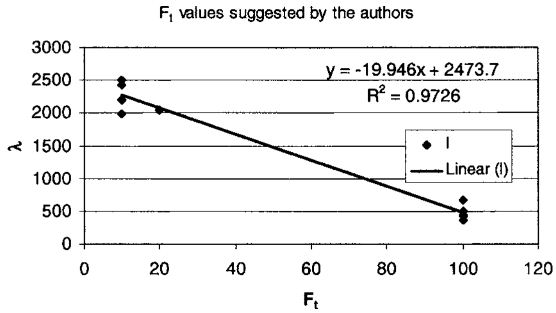

Fig. 1. F_t-λ correlation using the authors results

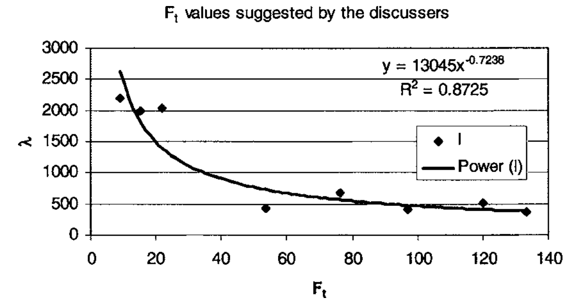

Fig. 2. F_t-λ correlation using the discussers results

values for all the beams reinforced with steel (average=2234 and standard deviation=227) compared to those reinforced with FRP (average=480 and standard deviation=119). In this closure, Fig. 1 shows the excellent correlation that F_t of 10, 20 and 100 has with the λ parameter. The linear fit presented in Fig. 1 has an excellent coefficient of correlation ($R^2=0.9726 \approx 1$). This linear fit provides the following equation that correlates F_t with λ for both steel and FRP reinforced beams:

$$F_t = 124 - 0.05\lambda \quad (\lambda \text{ } in \text{ kN/mm}) \tag{1}$$

The resulting equation indicates that high λ values correspond to smaller values of F_t (as in steel reinforcement) and vice versa (as for FRP reinforcement). Shrinkage is believed to affect tension stiffening as previously established. However, there is no direct relationship between shrinkage and the use of the λ parameter to explain the difference in F_t. λ is a parameter that combines the geometric variables as they relate to the progressive cracking process that reflects the tension stiffening behavior. Plotting the λ parameters versus the F_t values computed by the discusser, a power function is fitted showing less correlation ($R^2=0.8725$) (Fig. 2).

There was no mention in the discussion of the conversion factors used to switch f'_c to f_{cube} in order to determine f'_t of concrete in tension. Also, the moment-curvature relationship was not provided by the original paper for all beams. Accordingly, it was not clear from the discussion whether the moment-mid-span deflection was used in the Kaklauskas procedure to extract the model parameters R_t and F_t.