Additional concrete fracture energy tests performed by 6 laboratories according to a draft RILEM recommendation

Hillerborg, Arne

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DIVISION OF BUILDING MATERIALS
LUND INSTITUTE OF TECHNOLOGY

ADDITIONAL CONCRETE FRACTURE ENERGY TESTS PERFORMED BY 6 LABORATORIES ACCORDING TO A DRAFT RILEM RECOMMENDATION

REPORT TO RILEM TC50-FMC

ARNE HILLERBORG

REPORT TVBM-3017
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1. Introduction

This report comprises a continuation of tests performed at a number of laboratories in different countries /1/ for the evaluation of a draft RILEM recommendation regarding the determination of the fracture energy G_F /2/. The work has been initiated by RILEM TC 50-FMC (Fracture Mechanics of Concrete). The report is presented for discussion at a committee meeting in Bologna on the 1st of March 1984.

2. Laboratories taking part in the tests

The earlier report /1/ comprised tests performed at 9 different laboratories. This time 6 laboratories have taken part. 3 laboratories have taken part both times. Thus in total 12 different laboratories have taken part in the tests. These laboratories are situated in 8 different countries.

This time the following laboratories have taken part. For the further discussions the abbreviations within paretheses are used.

Universita Degli Studi Di Bologna, Istituto Di Scienza Delle Costruzioni (Bologna), Di Leo.

Ecole Polytechnique Fédérale de Lausanne (EPFL, Lausanne), F. H. Wittmann, I. Metzener-Gheorghita, 2 reports.

Facultad de Ingenieria, Departemento de Construcciones, La Plata, Argentina (La Plata), L.J. Lima, D. Violini, R. Zerboni.

Institut fur Massivbau und Baustofftechnologie, Universität Karlsruhe (MPA, Karlsruhe), W. Brameshuber, H.K. Hilsdorf.

Delft University of Technology, Department of Civil Engineering (Delft), H.A. Körmeling, H.W. Reinhardt.

Tohoku University (TU, Japan), H. Mihashi, N. Nomura.

The results reported by the different laboratories are summarised in table 1.

3. Some general remarks

In the previous report /1/ it was indicated that the modulus of elasticity E and the tensile strength f_t might be determined from the test results in the G_F-test. A formula was given for the determination of E from the initial slope of the load-deformation curve and a diagram was given for the determination of f_t from the maximum load, the fracture energy G_F and E.

The results of these determinations of E and f_t will be discussed below. However, a mistake was made in /1/ regarding the description of how f_t might be determined, and this mistake has to be corrected before commenting the test results.

For the determination of f_t the net bending stress att maximum load should first be calculated. The net bending stress is the formal bending stress in the net section above the notch, calculated with the ordinary beam formula. A formula was given for the calculation of this stress from the maximum load. Of course the weight of the beam also causes stresses in the net section, but this influence was forgotten. The correct formula should be

$$f_{net} = \frac{6(P_{max}+mg/2)\ell}{4b(d-a)^2}$$

where m is the mass of the beam between the supports, and g is the acceleration due to gravity. With a density of 2400 kg/m^3 and the proposed beam dimensions, the weight of the beam will add 0.45 MPa to the net bending stress. All values of f_t have been recalculated and corrected for the influence of the weight of the beam before beeing entered into table 1. The real size of the beam has been taken into account where this was given in the reports. Else the intended size has been used. The density has been asumed to be 2400 kg/m^3.

All laboratories have determined E, f_t, and the compressive strength in separate tests. Most laboratories have used the RILEM recommendations for this purpose, i e CPC 8 for E, CPC 6 (splitting test) for f_t, and CPC 4 for compression.

4. Bologna

The tests reported from Bologna are very extensive, comprising 7 different beam sizes, each with 6 different notch depths, and three tests of each type of specimen, thus a total of 126 beams. The largest beams had a span of 2.5 m and a depth of 2 m.

While measuring the load-deformation curves also time-deformation curves were recorded. In this way it was possible to make sure if a test was stable or unstable. It proved that only the smallest beams gave stable test, and also for these beams only where the notch depth was at least 0.2 times the beam depth. Thus the number of tests reported in table 1 is limited to only 36 of the 126 tests.

In the test report the author has calculated values of G_c and K_c according to linear elastic fracture mechanics. For some reason these two values do not correspond to each others according to the formula $K_c = \sqrt{EG_c}$. The value of G_c shows a great variation with the notch depth, whereas K_c is more constant for a given beam depth.

It is possible that the weight of the beam has been disregarded in the calculation of K_c.

The K_c-values given in the report are shown in Fig. 1 as functions of the beam depth, and a corresponding theoretical curve according to /3/ is also shown in the diagram as a comparison. By the comparison between the test results and the theoretical values it must be borne in mind that the theoretical curve has been calculated for a span/depth-ratio of 4, whereas this ratio for the deepest test beams was only 1.25.

In the report the initial compliance was given for all the tests. From this compliance it is possible to calculate the modulus of elasticity from the formula given in /1/, see below. In /1/ the value of the function g(a/d) in the formula was given only for $0.45<a/d<0.55$. For the analysis of beams with other notch depths the following values can be used for calculating the E-values

a/d	0.1	0.2	0.3	0.4	0.5
g(a/d)	0.035	0.14	0.32	0.63	1.20

By means of these values the E-values have been calculated for all the beams tested in Bologna. The result is shown in table 2. The applicability of the equation to beams with a small span/depth ratio is questionable. It should in the first place be applicable to the series denoted S, B, and C. This table will be further commented upon below when the possibility to determine E from the G_F-test is discussed.

Regarding the Bologna tests it should also be noted that the notches were cast and not sawn and that the specimens were stored in the open air without humidity control after a short wet curing period. Shrinkage stresses thus may have influenced the test results.

All the G_F-values taken together give an average of 160 N/m and a standard deviation of 27 N/m. There is a marked tendency that the G_F-value increases as the beam depth increases.

5. EPFL, Lausanne.

These tests have been reported in two reports, here denoted 1 and 2.

The tests in report 1 are all in agreement with the draft recommendation. One special thing about this report is that the tensile strength f_t has not been determined by means of a splitting test, but by means of a centrifuge. Probably this gives a better value of the uniaxial tensile strength than the splitting test.

The tests in report 2 are also in agreement with the draft recommendations, and they probably are performed in exactly the same way as in report 1. In this report it is however not explicitely said that the tensile strength has been determined by means of a centrifuge.

6. La Plata.

These tests have been made in full agreement with the draft recommendations. Only a preliminary report has been sent so far, but a more extended report is beeing prepared.

7. MPA, Karlsruhe.

These tests have been made in full agreement with the draft recommendations. Some interesting observations have been made beside the measurements necessary for the determination of G_F.

In all the tests the longitudinal deformation on a gage length of 150 mm has been recorded in the central part of the beam 10 mm and 50 mm above the bottom. The latter gage thus was was centered over the notch tip. These curves have not been analysed in detail, but it seems possible to draw at least a few conclusions.

At maximum load the lower gage showed a value between 190 and 300 microstrain with an average of about 250, whereas the upper gage showed values between 70 and 160 with an average of about 120. This indicates that the neutral axis at that stage was about 10-15 mm from the top of the beam, i e far above the center of the net section. This is in a good agreement with theoretical calculations.

The corresponding deformation on the length of the upper gage is as an average about 0.02 mm. Most of this is the deformation within the fracture zone at the notch tip, as the stresses far away from the fracture zone are small and hardly correspond to strains greater than about 20 microstrain. A deformation of 0.02 mm corresponds to about $0.5G_F/f_t$, which is in agreement with what might be expected from a theoretical analysis /3/.

At the moment when the beam failed (under the influence of its own weight, which corresponds to less than one tenth of the maximum load) the upper gage mostly showed values of 2000-4000 microstrains, corresponding to a deformation of 0.3-0.6 mm in the fracture zone at the notch tip. At that stage there are hardly any closing stresses across the fracture zone at that point. An analysis of the values also shows that the neutral axis at that stage practically coincides with the top of the beam.

Another interesting type of observations is that of the fracture zone propagation on the side of the beam with a special technique, utilising the capillary suction properties of the microcracks in that zone. The results from one of the tested beams are presented. At maximum load the visible fracture zone length with this technique was about 16 mm, corresponding to one third of the net depth. When the load had decreased to half the maximum value the visible fracture zone length was about 40 mm, and when it extended up to 1 mm below the extreme compression fibre the beam still carried 13 % of the maximum load. The authors pointed out that shrinkage stresses may play a role for the visibility of the microcracks on the surface, although the beam was kept wet, and that the fracture zone length in the interior of the beam may be smaller.

In the tests where the fracture zone was observed in the way described above, the region defined as the fracture zone had a width of 3-4 mm. This can be compared to the maximum aggregate size, which was 16-32 mm.

8. Delft.

The tests in Delft were performed on beams with a shorter span (450 mm) than the proposed standard beam. A special arrangement with parallel steel beams was used in order to get a sufficient stiffness of the testing mashine.

One of the concrete types tested was an epoxy concrete, where a certain amount of epoxy was mixed with the water before cement and aggregate was added.

The ordinary concrete tested had also been tested in stable tension tests on notched specimens for the determination of the complete stress-deformation curve. From these tests the values of G_F and f_t could be evaluated for a comparison. The average value of G_F from these tests was 113 N/m with a standard variation of 13 N/m. This average value happens to coincide with that from the beam test.

The average tensile strength from the tensile test was 3.30 MPa, standard deviation 0.51 MPa. This is somewhat higher than the value from the splitting test and much higher than the value calculated from the beam test.

It should be noted that the direct tension tests were performed on dry concrete, whereas the splitting tests and the beam tests were performed on wet concrete. The tensile strength of wet concrete can be expected to be markedly lower than that of dry concrete, maybe in the order 20-40 %.

9. TU Japan.

These tests were performed in full agreement with the draft recommendation.

10. Comments on the measured values of G_F.

All the test results (197 beams) are summarised in table 1. All results are within normally expected limits regarding average values as well as standard deviations.

The value of G_F can be expected to vary with many factors. Two of these factors, which may be taken from the table, are the maximum aggregate size and the water/cement ratio. In Fig. 2 all the average values from table 1 and the corresponding table in the previous report /1/ are shown as a function of the maximum aggregate size. Only beams with the standard size according to the draft recommendations have been included, in order to exclude scatter due to beam dimensions. A separation has been made between concrete with a low and a high water/cement ratio.

The diagram shows no major influence of the water/cement ratio. The maximum aggregate size has an influence, at least up to about 20 mm. There is however a great scatter, and evidently other factors also play an important part for the fracture energy. Such factors may be the strength of the aggregate, and the shape and surface characteristics of the aggregate.

The influence of the depth of the test specimen on the fracture energy was discussed in the previous report /1/. Some further information may be gained from the Bologna tests, where results from beams with depths 100 and 150 mm have been reported, see table 1. These results indicate an increase in G_F with a factor 1.2-1.35 (depending on how the comparison is made) when the beam depth is increased by a factor 1.5. This is in agreement with the findings of ISMES, Bergamo, but more than found in some other test series. This underlines the statement made in the previous report that the influence of the beam size on the measured values of G_F ought to be further investigated.

The intention of the G_F-test is to measure the energy absorbed within the fracture zone during the test. Energy is however also absorbed outside the fracture zone, and this additional energy becomes included in the measured value and causes this to be too high. The additional energy is mainly due to permanent strains in that region outside the fracture zone, where the stresses reach values not far below the tensile strength. How much energy that is absorbed within this region depends on the size of the region and on the shape of the ascending branch of the stress-strain curve for the material.

With a notched beam the size of the highly stressed region depends on the notch depth. With a deep notch this size is small. This is one reason why the notch depth is proposed to be half the beam depth in the draft recommmendation.

If the span and the depth of the beam are both increased with a factor 2, the fracture area increases with a factor 2 but the volume of the highly stressed region increases with a factor 4. Thus the relative influence of the energy absorbed in the highly stressed region can be approximately expected to increase with a factor 2. This is one reason why the proposed beam size is rather small.

The shape of the stress-strain curve in tension can probably vary much between different concretes, from a nearly straight line to a rather curved shape. Not much is known about this, but if this is the case, the influence of the ascending branch - and thus of the beam size - will be very different for different concretes.

It is possible to make a theoretical analysis of the influence of the shape of the ascending branch on the measured fracture energy. For this purpose it is necessary to use a finite element program where non-linerar stress-strain curves can be introduced. So far such an analysis has not been carried through. Before doing this the shape of the stress-strain curve in tension ought to be investigated for different concretes in order to have realistic curves to put into the analysis.

11. Comments on the modulus of elasticity.

In /1/ a proposal was given that the modulus of elasticity might be determined from the initial slope of the load-deformation diagram in the G_F-test. The following formula, based on the theory of elasticity, was given for that purpose

$$E = \left[1 + 3.15\left(\frac{d}{\ell}\right)^2 + 8\frac{d}{\ell}\, g\,(a/d)\right] \cdot \frac{1}{4b}\left(\frac{\ell}{d}\right)^3 \cdot \frac{dF}{d\delta}$$

$$g(a/d) = \frac{0.15}{\left(1-\frac{a}{d}\right)^3} \qquad \text{for } 0.45 < a/d < 0.55$$

Values of E calculated by means of this formula are shown in table 1 for all the G_F-tests where the relevant information was given and in table 2 for all the Bologna tests. (See above for a/d<0.45.) The corresponding values determined according to the RILEM recommendations are also given.

From the tables it is evident that this way of determining E does not give reliable values. The values determined on the standard beams according to the draft recommendations always seem to be too low. This is difficult to explain.

In most reports it is not clearly stated how the deflections have been measured. It can not be excluded that some laboratory has by mistake used the movement of the mashine head or measured the movement of the beam center with respect to the supporting table. Such things would tend to increase the measured value and lead to too low E-values. In /1/ it is pointed out that the equation may only be used if the recorded deflection is the true deflection of the center of the beam with respect to the supports.

In the Bologna report it is clearly shown that the deflection has been measured in the correct way for the application of the equation. From table 2 it can also be seen that the values calculated from the tests as a rule show a reasonable agreement with the E-values measured on cylinders, although they tend to be a little lower for slender beams and a little higher for beams with a so small l/d-value that the application of ordinary beam theory is questionable. There are only a few values which are much too low and the very lowest value happens to be that for the standard beam according to the draft recommendation.

One possible cause for the disagreement might be that there is something wrong with the coefficients of the equation. The equation contains three terms, each one corresponding to one cause for the deflection. The first term corresponds to the deflection due to the bending moments of a beam without a notch. This term can be checked from standard tables.

The second term corresponds to the deflection due to shear forces in a beam without a notch. The term is given for the simplified case where the Poissons ratio ν is neglected. It should be multiplied by $(1+\nu)$ if ν should be taken into account. The influence of this correction on the calculated value of E is insignificant, and it can not explain the disagreement.

The third term corresponds to the additional deformation due to the notch. The values have been taken from a Swedish table, but it has also been checked by means of two independent finite element calculations. These three values agree very well, so there can hardly be any major inaccuracy in these values. This is also confirmed by the results of table 2. The relative influence of the third term is greatest when the span/depth ratio is small. If the values of the third term were the reason for the disagreement, this would show most in beams D, E, and F.

Thus there can hardly be such inaccuracies in the equation, that this can explain the disagreement.

By the determination of E according to the RILEM recommendations the cylinder is loaded and unloaded 10 times before the modulus is measured. This procedure gives higher values than those which result from the first loading. Part of the disagreement can be explained from this fact.

It is also possible that there is a slight difference between the E-modulus in tension and in compression, which may explain some of the difference.

However it looks like the determination of E from the load-deflection curve gives too uncertain values to be recommended as a standard procedure. It must also be borne in mind that a reasonably correct value of E as a rule can be estimated when the composition of the concrete is known, and that this estimated value maybe is more correct than the one calculated from the load-deflection curve.

12. Comments on the tensile strength.

The tensile strength has by most laboratories been determined by means of splitting tests, but by EPFL in Lausanne by means of a centrifuge, at least in the first report.

Values of the tensile strength have also been determined from the maximum load in the bending test, using a diagram given in /1/. As has been pointed out above the weight of the beam has to be taken into account in this calculation, which by mistake was not pointed out in /1/. The values introduced in table 1 have been corrected for the weight of the beam. When the tensile strength has been evaluated from the bending test the E-value determined according to the RILEM recommendation has been used, not the E-value determined from the bending test, as the latter does not seem to be reliable. The calculation of the values of the tensile strength have been based on the average values of f_{net}, E and G_F for each series, not on the individual test values.

Table 1 shows the values from the splitting tests (respectively the centrifuge tests) and the values calculated from the bending tests. The ratios between these values have also been calculated.

The tests according to report 1 from EPFL, Lausanne, show a reasonable agreement between the two values of the tensile strength. This may be taken as an indication that the tensile strength determined from the bending test is acceptable. The tensile strength determined by means of a centrifuge is probably closer to the true uniaxial tensile strength than a value determined in a splitting test.

On the other hand the tests according to report 2 from EPFL, Lausanne, do not show the same good agreement. It is not clear whether these tests were also made with a centrifuge.

The tests from Bologna also show a reasonable agreement between the two values if we look on the average of all the values in table 1. However the scatter is great and there is a marked tendency that beams with deep notches give lower values than the average. The standard beams according to the draft recommendations gave a value which is about 20 % too low, compared to the value from the splitting test.

It must be remembered that the Bologna tests were made without moisture control, which may have influenced beams with different sizes and different notch depths in different ways.

In the tests at La Plata, Delft and TU-Japan the tensile strengths determined from the beam tests are much lower than the splitting strength, whereas the tendency is the opposite in the tests at MPA-Karlsruhe. These results are not very reassuring regarding the possibility to determine a reliable value of the tensile strength from the G_F-test.

The question may however be raised whether the splitting test gives reliable values of the uniaxial tensile strength. The splitting test as well as the beam test is an indirect method of determining the tensile strength, where the result depends on the validity of the assumptions on which the evaluation is based. It is not self-evident that the splitting test gives a better value than the beam test, especially when the value of f_t is intended for application in fracture analyses.

13. Comments on the characteristic length.

The characteristic length l_{ch} (EG_F/f_t^2) has been calculated for the different test series in table 1. As E-values have been used those determined according to the RILEM recommendations. Two different l_{ch}-values have been calculated, one for each metod of determining f_t.

The values of l_{ch} show a great scatter, and sometimes a great difference depending on the method for determining f_t, but the most noticeable observation is that there is an appreciable difference between different laboratories, possibly depending on differences in raw materials. The tests of Bologna and La Plata show l_{ch}-values of the order of 1 m, whereas e. g. those from EPFL-Lausanne are in the range 0.1-0.4 m. The latter values correspond to what has hitherto been looked upon as typical. Evidently some types of concrete may show very high l_{ch}-values. Such concretes are tougher and as a consequence linear elastic analysis is less applicable, whereas plastic analysis is more applicable.

Conclusions

1. The G_F-tests according to the draft recommendations have been performed by the laboratories without any major difficulties.

2. The standard deviations in the test results are within the limits which had been expected.

3. The influence of the beam size on the measured G_F-values ought to be better analysed, theoretically and by means of tests.

4. It does not seem suitable to determine the modulus of elsticity from the initial slope of the load-deformation diagram.

5. The agreement between the values of the tensile strength caclulated from the beam test and from the splitting test is not good, and it does not show systematic divergences. It is however not self-evident that the values from the splitting test are more true than those from the beam test.

References.

1. Hillerborg, A: Concrete fracture energy tests performed by 9 laboratories according to a draft RILEM recommendation. Lund Institute of Technology, Division of Building Materials, Report TVBM-3015, 1983.

2. Determination of the fracture energy of mortar and concrete by means of three-point bend tests on notched beams. Proposed RILEM recommendation, January 1982, revised June 1982. Lund Institute of Technology, Division of Building Materials.

3. Petersson, P-E: Crack growth and development of fracture zones in plain concrete and similar materials. Lund Institute of Technology, Division of Building Materials, Report TVBM-1006, 1981.

TABLE 1. SUMMARY OF TEST RESULTS.

Laboratory	w/C	Age days	D_{max} mm	l/d	a/d	Number	δ_0 mm	G_F N/m	E GPa	E(c) GPa	f_{net} MPa	$\frac{(d-a)f_{net}^2}{EG_F}$	$\frac{f_{net}}{f_t}$	$f_t(c)$ MPa	f_t MPa	$\frac{f_t}{f_t(c)}$	ℓ_{ch} from f_t mm	ℓ_{ch} from $f_{t(c)}$ mm	
Bologna	0.50	≈100	13	8	0.2	3		113	24.6	21.0	4.31	0.53	1.43	3.01	2.1	0.70	630	307	
					0.3	3		133		22.6	3.67	0.29	1.78	2.06		1.02	742	771	
					0.4	3		113		16.4	3.23	0.23	1.90	1.70		1.24	630	962	
					0.5	3		156		15.7	3.56	0.17	2.12	1.25		1.25	870	1360	
				3	0.2	3		176		18.9	3.55	0.35	1.67	2.13		0.99	982	954	0.15x0.15x0.45 m
					0.3	3		170		19.1	3.50	0.31	1.75	2.00		1.05	948	1046	
					0.4	3		174		19.5	3.69	0.29	1.78	2.07		1.01	971	1000	
					0.5	3		202		16.1	3.83	0.22	1.94	1.97		1.07	1127	1280	
				5.3	0.2	3		164		22.9	3.93	0.46	1.51	2.60		0.81	915	597	0.15x0.15x0.8 m
					0.3	3		170		22.6	3.57	0.32	1.74	2.05		1.02	948	995	
					0.4	3		178		25.4	3.16	0.21	1.97	1.60		1.31	993	1710	
					0.5	3		170		21.1	3.44	0.21	1.97	1.75		1.20	948	1366	
EPFL-Lausanne (report 1)	0.46	7	30	8	0.5	8	1.1 0.1	112 11	29.6		4.89	0.36	1.65	2.96	3.13	1.06	338	378	
	0.58	7	30			6	0.9 0.1	81 6	24.7		3.77	0.36	1.65	2.28	2.43	1.06	339	385	
	0.50	7	8			8	0.9 0.1	85 11	22.4		4.87	0.62	1.32	3.69	3.32	0.90	173	140	
	0.50	7	3			7	0.7 0.1	62 5	20.4		4.12	0.67	1.28	3.22	3.04	0.94	137	122	
	0.40	7	3			3	0.8 0.1	79 6	24.4		5.19	0.70	1.24	4.19	3.55	0.85	153	110	
	0.40	7	8			3	0.8 0.1	78 7	26.5	20.0	5.15	0.64	1.30	3.96	4.10	1.03	123	132	
EPFL-Lausanne (report 2)	0.58	34	30	8	0.5	6	1.1 0.1	105 15	29.1	21.2	4.60	0.35	1.67	2.75	3.81	1.39	210	404	
	0.48	33	30			8	1.1 0.1	120 12	32.7	22.6	6.00	0.46	1.51	3.97	4.27	1.08	215	249	
	0.40	30	30			9	1.2 0.2	146 40	35.1	26.3	6.17	0.37	1.65	3.74	4.41	1.18	263	366	
La Plata	0.48	30	9.5	8	0.5	7	1.2 0.1	99 9	33.9		4.03	0.25	1.86	2.17	2.96	1.37	383	713	
	0.58		9.5			7	1.5 0.2	109 14	-		3.57	0.20	1.97	1.81	2.34	1.29	637	1065	
	0.68		9.5			5	1.5 0.2	101 16	29.9		3.18	0.18	2.02	1.57	1.93	1.23	810	1225	
	0.48		20			13	2.0 0.3	188 30	42.2		5.12	0.16	2.10	2.44	3.79	1.55	555	1340	
	0.58		20			14	2.4 0.7	200 51	39.9		4.44	0.13	2.20	2.02	2.45	1.21	1330	1955	
	0.68		20			14	2.4 0.6	186 49	35.7		4.17	0.13	2.20	1.90	2.27	1.20	1290	1840	
MPA-Karlsruhe	0.40	28	16	8	0.5	10	1.5 0.2	152 29	36.7	23.8	6.34	0.37	1.65	3.84	3.37	0.88	493	378	
	0.58		32			10	1.9 0.5	168 36	30.4	22.9	5.51	0.30	1.77	3.11	2.34	0.75	934	528	
TH-Delft	0.50	28	8	4.5	0.5	5		113 8	(30)		3.80	0.21	1.97	1.93	2.9	1.50	403	910	l=0.45 m
	0.34	28	8			6		143 26	(40)		5.22	0.24	1.88	2.78	4.0	1.44	358	740	epoxy concr.
	0.35	3	1			3		94 16	(25)		3.86	0.32	1.74	2.22	4.19	1.89	134	477	mortar
	0.35	3	3			3		127 11	(25)		5.72	0.52	1.44	3.97	6.26	1.58	81	201	
	0.35	3	6			3		155 13	(25)		4.74	0.29	1.78	2.66	6.08	2.28	105	548	
TU-Japan	0.40	28	25	8	0.5	13	1.1 0.1	112 19	26.3		4.76	0.38	1.63	2.92	4.03	1.38	181	345	

Note: E(c) and $f_t(c)$ are calculated from the beam test.
Values within parantheses are only estimated.

TABLE 2. E(c) FOR ALL BOLOGNA TESTS.

Test series	l m	d m	l/d	a/d	Compli-ance mm/MN	E(c) GPa
S	0.8	0.1	8	0	54.9	24.5
				0.1	58.2	23.8
				0.2	72.3	21.0
				0.3	77.7	22.6
				0.4	130.8	16.4
				0.5	183.8	15.7
A	0.45	0.15	3	0	3.2	19.0
				0.1	3.5	18.6
				0.2	4.1	18.9
				0.3	5.2	19.1
				0.4	7.0	19.5
				0.5	12.7	16.1
B	0.8	0.15	5.33	0	13.0	21.6
				0.1	13.6	21.6
				0.2	14.6	22.9
				0.3	17.8	22.6
				0.4	20.5	25.4
				0.5	34.9	21.1
C	2.5	0.5	5	0	7.5	23.5
				0.1	7.9	23.4
				0.2	7.7	27.4
				0.3	9.5	26.9
				0.4	13.2	25.3
				0.5	17.6	27.0
D	2.5	1.0	2.5	0	0.80	29.4
				0.1	0.98	25.8
				0.2	1.34	22.8
				0.3	1.49	26.5
				0.4	1.88	29.3
				0.5	3.03	27.6
E	2.5	1.5	1.67	0	0.36	27.5
				0.1	0.37	28.8
				0.2	0.58	22.4
				0.3	0.66	25.7
				0.4	0.80	29.9
				0.5	1.16	31.5
F	2.5	2.0	1.25	0	0.22	26.8
				0.1	0.31	20.4
				0.2	0.35	21.8
				0.3	0.37	26.7
				0.4	0.49	28.1
				0.5	0.66	31.7

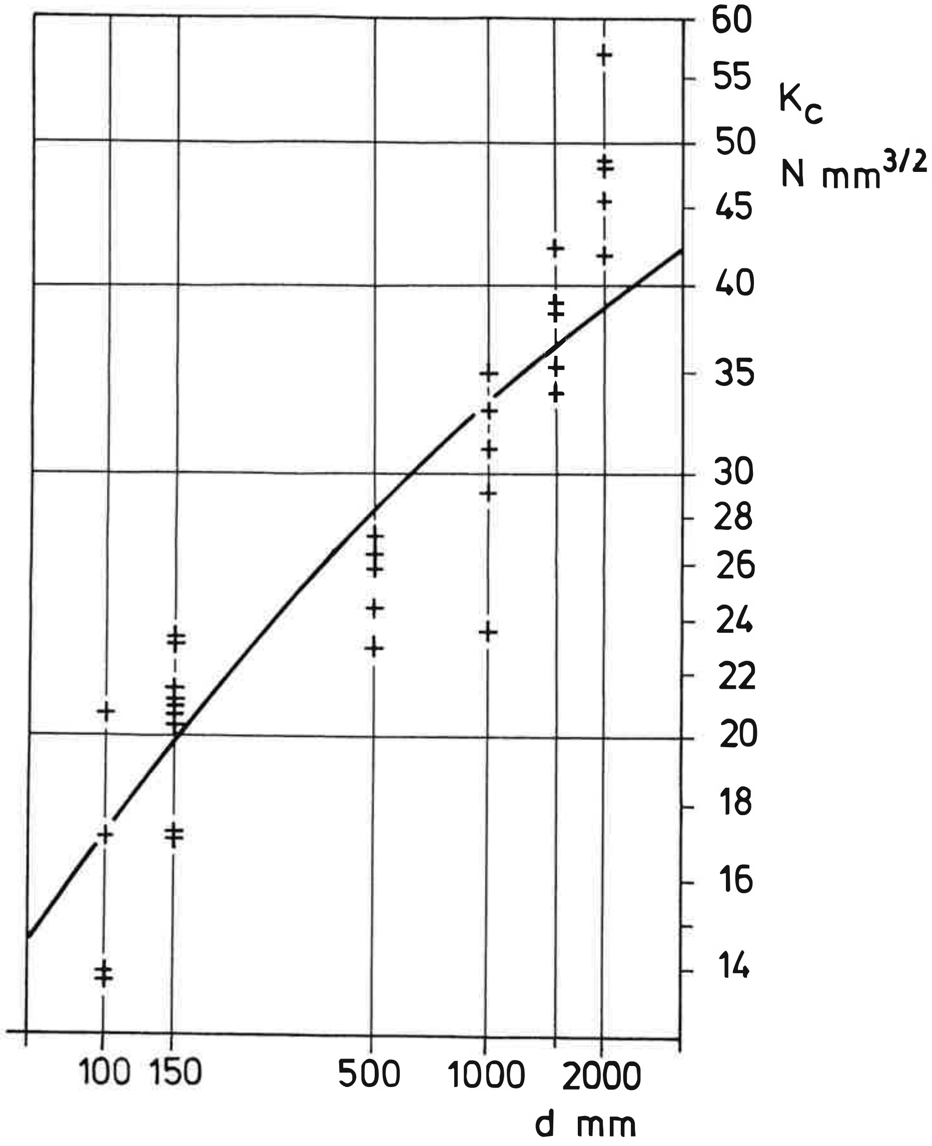

Fig. 1 K_c according to the Bologna tests as a function of the beam depth. The curve shows a theoretical relation for E = 24.6 GPa, G_F = 160 N/m, f_t = 2.1 MPa according to /3/.

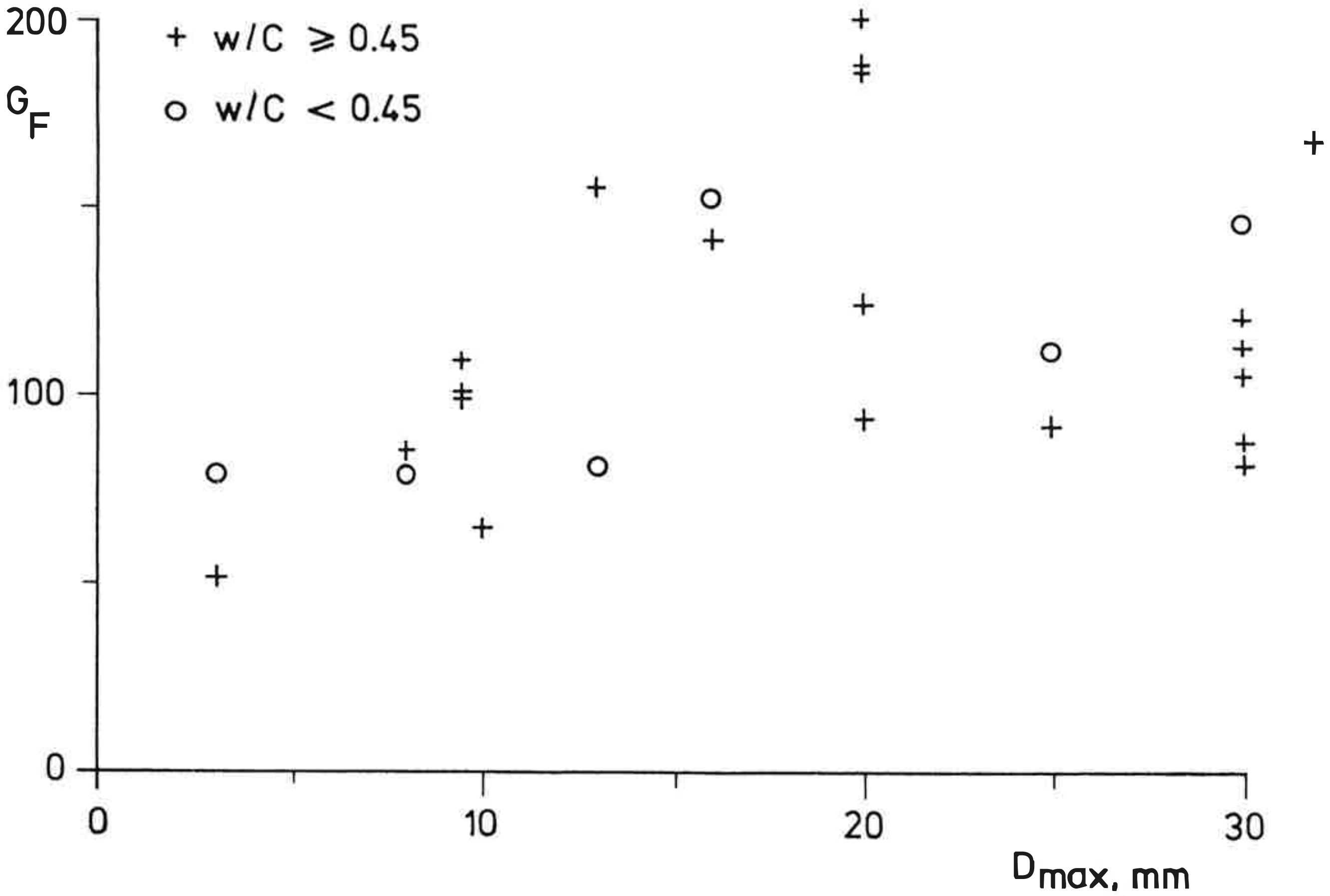

Fig. 2 Variation of G_F with maximum aggregate size and water/cement-ratio for standard beams according to the draft recommendations.