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A new approach to the design of RC structures based on concrete mix characteristic length

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Abstract A new approach to the design of reinforced concrete (RC) structures is proposed. It does not rely on the traditional characteristic compressive strength of the concrete mix which is the basis of all current codes for the design of RC structures. Instead, the approach is based on the characteristic length of the concrete mix that has its origins in the concepts of fracture mechanics. Based on the research done in Cardiff University over the past 6 years on long and short beams and slender columns, it is shown that this new approach leads to a substantial reduction in the amount of reinforcing steel needed in RC structures made from high strength concrete mixes without jeopardising their ductility. This provides conclusive evidence that the current design code provisions for reinforcement based on the mix characteristic compressive strength grossly overestimate the requirements for high strength mixes leading to wastage of steel, reinforcement congestion and high cost of construction. The adoption of this new design approach, which is based on sound physical principles, should help promote the use of high performance, durable and sustainable concrete in the construction industry without increasing the cost of construction or compromising the safety of structures.

Keywords RC structures · Fracture mechanics · Mix characteristic length · Design

It had been planned to report this work for the first time at the 20th European Conference on Fracture (ECF20, Trondheim, July 2014) but that was not possible. However, a brief summary of some of the results that had been submitted to this Conference appears in *Procedia Materials Science*, Elsevier, vol 3, 2014, pp 369–377.

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1 Introduction

The current codes for the design of RC structures throughout the world are based exclusively on the characteristic compressive strength f_c' of the concrete mix and ignore completely its tensile capacity. This means that as the tensile strength and the brittleness of concrete increase with an increase in f_c', the minimum reinforcement required to attain the required structural ductility has to be increased without utilising the higher tensile strength. This leads to unnecessary wastage of reinforcement, higher cost of construction and to severe reinforcement congestion, especially near joints which in turn leads to a lack of adequate compaction and cover, i.e. to honeycombing.

There is indeed another, more fundamental, reason for not using f_c' as the basic design parameter. This is to do with the fact that the failure of any engineering material is a result of the breaking of bonds that can only take place when the bonds are stretched beyond a limit. What is often regarded as failure under compression is in fact a result of the coalescence of local tensile micro-fractures. The fundamental material property of concrete is not therefore its compressive strength but rather its tensile strength.

To avoid the total reliance of RC design on a dubious material property of concrete (we shall give another reason below), we propose a completely new design approach based not on f_c' but on the characteristic length l_{ch} of the concrete mix. We shall demonstrate in this paper how beneficial this new design approach is in reducing the amount of minimum reinforcement needed when high strength concrete is used in an RC structure without jeopardising the ductility of the structure. We shall give examples of long and short span beams and slender columns in support of this new design approach.

The mix characteristic length was first introduced more than three decades back by Hillerborg et al. (1976) and Bache (1986) based on the concepts of fracture mechanics. These concepts are particularly relevant to concrete because they take into consideration internal dissipation through non-fatal micro-cracking of a lot of energy that is imparted to a concrete structural element when it is under external mechanical and/or environmental loading (Hawkins 1985; Hawkins and Hjorteset 1992; Committee ACI 446 Report ACI 446–2R 1992). The characteristic length involves three independent fundamental properties of the mix; its elastic stiffness (E), tensile strength f_t', and specific fracture energy or toughness G_F

$$l_{ch} = (EG_F)/(f_t')^2 \tag{1}$$

It captures the intrinsic ductility of the mix; the larger the l_{ch}, the more ductile the mix. It is clear from the definition of l_{ch} that it decreases sharply as f_t' increases.

From the characteristic length of the mix used in an RC structure, it is also possible to predict the response of the latter under external loading, depending upon its characteristic size W. For this the structural ductility index

$$\beta = l_{ch}/W \tag{2}$$

is used (Karihaloo 1995); the higher this index the more ductile the structural response, and vice versa. It shows that RC structures made from the same mix and containing the same amount of reinforcement will exhibit lesser and lesser ductility, as the characteristic size of the structure increases.

If however the RC structural members are made from the same mix but contain different amounts of reinforcement as measured by the reinforcement ratio ρ (i.e. the ratio of the area of steel to the area of the member cross section), then Bosco and Carpinteri (1992) have derived the following modified structural ductility index using dimensional analysis

$$\beta^* = (\sqrt{\beta}) f_t'/(f_{yk}\rho) \tag{3}$$

where f_{yk} is the yield strength of the reinforcing steel. The larger the value of β^*, the more ductile is the structural response. It follows that RC structural members of the same characteristic size W and made from the same mix will exhibit lesser and lesser ductility as more and/or higher strength reinforcing steel is used in them. Note that β^* will reduce to β to within a constant multiplier if the reinforcement ratio ρ is the same.

The new design approach proposes to use the minimum reinforcement in RC structures corresponding to a fixed l_{ch} of a normal strength concrete mix irrespective of the f_t' or f_c' of the actual mix used in the structure. Thus, if the base l_{ch} is chosen to correspond to a mix with, say $f_c' = 40$ MPa or 50 MPa, but in the actual RC structure a mix with, say $f_c' = 100$ MPa is used whose l_{ch} would be much smaller than the base value [according to Eq. (1)], then it must be increased to coincide with the base value. In turn this means that the minimum reinforcement required for RC structures of the same characteristic size made of mixes with different f_t' (i.e. f_c') but with the same l_{ch} will be the same and that these structures will exhibit identical ductile response under external loading. As the stiffness E increases only marginally with an increase in f_c' (i.e. f_t') it is clear that the toughness G_F of the mix must be increased to compensate for the reduction in l_{ch} caused by the increase in f_t' [see Eq. (1)]. This is achieved by the addition of short steel fibres. The amount of fibre to be added will depend on f_t' of the mix and on the type and texture of the steel fibre used.

This paper will give an overview of the research done over the past 6 years in Cardiff University on different RC members (long and short beams, slender columns) made from mixes with $f_c' = 40$ or 50 and 100 MPa to test the validity of this new design approach (Wei 2007; Khan et al. 2009; Pei et al. 2010; Qureshi et al. 2011; Gougoulias et al. 2012; Qiu and Zhang 2013). The higher strength mix had to be supplemented by about 0.18–0.20 % by volume 30 mm

Table 1 Material Properties

Concrete mix designation	Static modulus of elasticity E (GPa)	True specific fracture energy G_F (N/m)	Tensile splitting strength f_t (MPa)	Characteristic length l_{ch} (mm)
C40–C50	30–40	90–125	3.5–4.0	225–305
C100–C110	45–52	60–80	6.5–7.5	55–85
C100–C110 + 40.18–0.20 % by vol steel fibre	45.5–52.5	450–500	7.8–8.4	335–375
C85 + 0.2 % SF	41.2	546	6.9	470
C85 + 0.5 % SF	41.6	1361	8.7	750
C85 + 0.75 % SF	41.8	2448	10.1	970

long and 0.55 mm diameter steel fibres with crimped ends for it to have nearly the same l_{ch} as the 40 or 50 MPa mix (approximately 300 mm). All members of a given type e.g. slender columns were reinforced with the minimum reinforcement required for 40 or 50 MPa concrete mix according to the European Norm EC2 (European Committee for Standardization 2004). They were tested and found to exhibit exactly the same failure mode, irrespective of the mix f_c'. The members made from 100 MPa concrete mix with 0.18–0.20 % by volume steel fibres carried more load, as expected, but failed in a more ductile manner than the corresponding members made from 40 or 50 MPa mix despite the fact that they contained the same minimum reinforcement as the latter. This confirmed our suspicions that the current design code provisions for the minimum reinforcement based on the mix compressive strength grossly overestimate the requirements for high strength mixes leading to wastage of steel, reinforcement congestion and high cost of construction.

It should be mentioned that the choice of the base mix is dictated by the most common grade of concrete used by the UK construction industry. It does not mean that the minimum reinforcement prescribed for 40 or 50 MPa concrete mix in the European Norm EC2 (European Committee for Standardization 2004) has any sound physical basis. Indeed, the minimum reinforcement prescribed even for these concrete grades is known to be somewhat excessive (Karihaloo 1995; Bosco and Carpinteri 1992).

The measured material properties and the characteristic length of mixes are given in Table 1. The cube compressive strength (f_c', designated $f_{ck,c}$ in EC2), the modulus of elasticity (E) and the tensile splitting strength (f_t', designated f_{ctm} in EC2) were measured by standard tests (British Standard BS 1881–121:1983 1983; British Standard BS EN 12390–3:2009 2009; British Standard BS EN 12390–6:2009 2009), whereas the size-independent G_F was measured using the boundary effect method (Abdalla and Karihaloo 2003; Ramachandra Murthy et al. 2013; Cifuentes and Karihaloo 2013). For later use, we have also included three mixes of grade C85 with three different volume fractions of steel fibre.

In connection with the measurement of G_F it should be mentioned that there is still no standard method available. In fact, it can be argued that the lack of a standard method has delayed, if not prevented, the application of fracture mechanics concepts to the design of concrete and RC structures. Whichever method is eventually agreed upon as standard will however have no bearing on the conclusions reached in this paper, because it will affect all grades of concrete, so that their relative characteristic lengths will remain essentially unaltered.

2 Flexural members

According to sections 9.2.1.1 (1) and 9.2.1.1 (3) of EC2, the minimum and maximum permitted longitudinal tension reinforcement of flexural members is determined by the following relationships:

$A_{s,min} = 0.26(f_{ctm}/f_{yk})b_t d$, but not less than

$0.0013 b_t d$ and also $A_{s,max} \leq 0.04 A_c$,

where f_{ctm} is determined with respect to characteristic compressive cylinder strength of concrete, f_{yk} is the characteristic yield strength of reinforcement, b_t is the mean width of tension zone, d is the beam depth, and A_c is the cross sectional area of concrete.

The minimum longitudinal reinforcement ratio is given in Table 2. In the concrete grade designation,

Table 2 Minimum reinforcement ratio according to EC2 (9.2.1.1(1)), using steel $f_{yk} = 500$ MPa

Concrete	C25/30	C30/35	C40/50	C50/60	C60/75	C80/95	C90/105	C100/115
f_{ctm}	2.6	2.9	3.5	4.1	4.4	4.8	5.0	5.2
$A_{s,min}/(b_t d)$ (%)	0.13	0.15	0.18	0.21	0.23	0.25	0.26	0.27

Table 3 Minimum shear reinforcement of beams, using steel $f_{yk} = 500$ MPa

Concrete	C25/30	C30/35	C40/50	C50/60	C60/75	C80/95	C90/105	C100/115
$\rho_{w,min\%}$	0.08	0.09	0.10	0.11	0.12	0.14	0.15	0.16

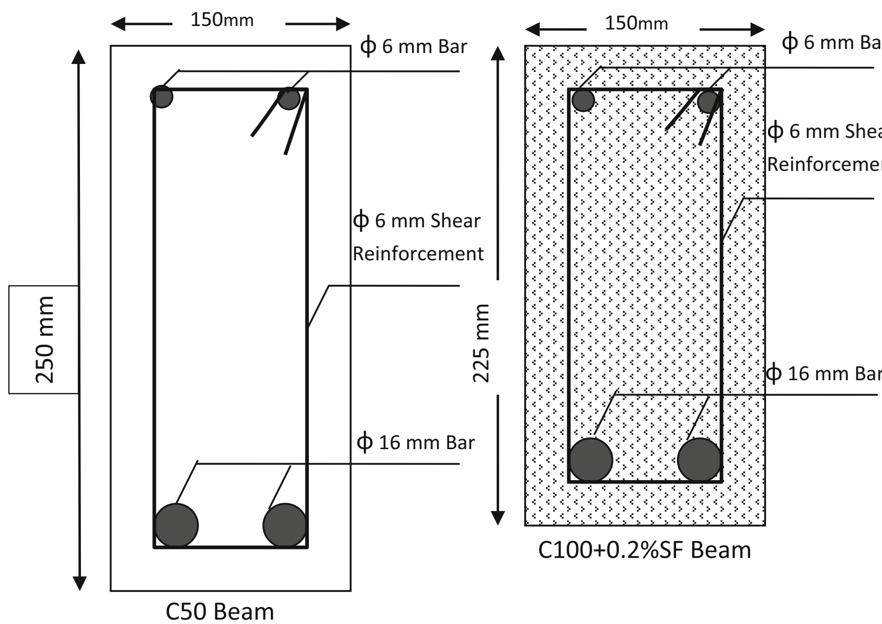


Fig. 1 Cross section view of C50 and C100 + 0.2 % SF concrete beams

e.g. C25/30, the first number corresponds to the characteristic cylinder strength f_{ck} and the second to the characteristic cube strength $f_{ck,c}$. This variability of the characteristic compressive strength of one and the same concrete mix with the size and shape of the test specimen is another reason for its dubious nature as a design parameter.

Throughout this present paper, the concrete class refers to the second number, e.g. C50 designates concrete with a cube characteristic strength of 50 MPa. We shall also use concrete grade C40 which is not listed in EC2, but by common consent it would read C32/40, with the required amounts of steel being obtained by interpolation. Likewise, concrete grade C100 reported below could be regarded as C85/100 according to EC2, grade C110 as C95/110, and grade C85 as C70/85.

The minimum shear reinforcement requirement in beams according to EC2 (9.2.2(5)) is given in Table 3.

It will be noted from Tables 2 and 3 that the minimum longitudinal and shear reinforcement ratios according to EC2 must be increased by nearly 50 and 60 %, respectively when the concrete characteristic cube compressive strength is increased from C50 to C110.

2.1 Long flexural members

At least four long beams (length 1700 mm) were cast from C50 and four from C100 + 0.2 % SF mixes which had similar l_{ch} (Table 1). The longitudinal and shear reinforcement in each beam irrespective of the con-

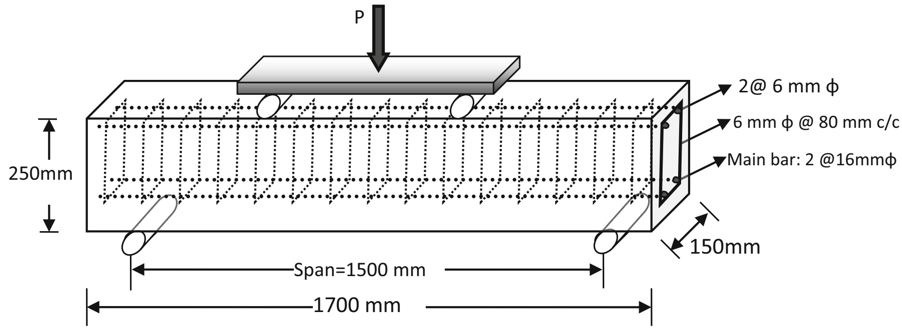


Fig. 2 C50 test beam without fibre (span-depth ratio = 6) for ultimate load 120 kN

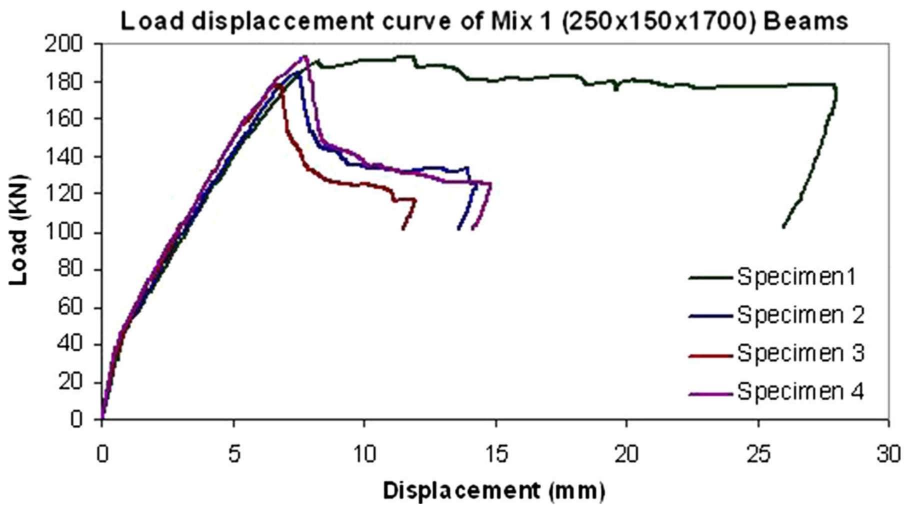


Fig. 3 Load mid-span displacement curves of C50 test beams (four specimens)

crete grade corresponded to C50 grade concrete, as shown in Fig. 1. The beams from both grades of concrete were designed to carry the same nominal ultimate load (120 kN), and were doubly reinforced. The depth of the higher grade concrete beam was reduced from 250 to 225 mm (see Fig. 1). This means that the reinforcement ratio is slightly higher in the C100 + 0.2 % SF beams.

The beams were tested in four-point bending, as shown in Fig. 2 over a span of 1500 mm and the mid-span deflection was recorded. The recorded load-deflection plots are shown in Figs. 3 and 4. The load carrying capacity of all beams was nearly the same, approximately 185 kN. EC2 envisages a partial factor of 0.7, so that the expected ultimate load should have been 171.4 kN.

All C50 beams (except one which showed a flexural response) failed predominantly in shear (Fig. 3), whereas all C100 + 0.2 % SF beams exhibited pure flexural response (Fig. 4), notwithstanding the fact that they contained the same longitudinal and shear reinforcement as the C50 beams. That the C100 + 0.2 % SF beams exhibited a more ductile response than the C50 beams is not at all surprising. The C100 + 0.2 % SF mix has a slightly longer l_{ch} than the C50 mix (Table 1) and the characteristic size of the beams (i.e. their depth) made of this mix is smaller than that of C50 beams, so that their structural ductility index (l_{ch}/W) is higher than that of the C50 beams according to Eq. (2). The ductility is only slightly reduced because of a higher reinforcement ratio ρ [see Eq. (3)] (see also Table 4 below).

2.2 Influence of larger reinforcement content on structural ductility

It is interesting to examine how the response of C100 + 0.2 % SF beams would change if they had a larger reinforcement ratio than the C50 beams, not necessarily as large as recommended by EC2 (Tables 2, 3), but larger than that in C50 beams nonetheless.

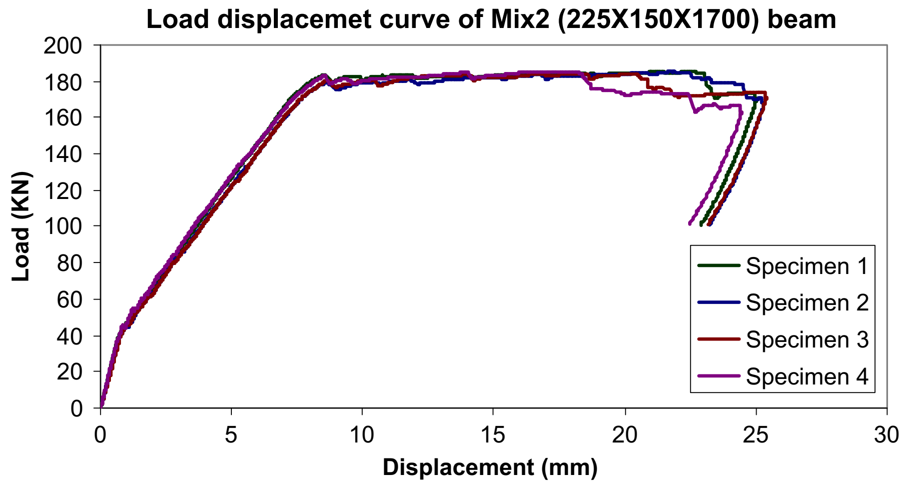


Fig. 4 Load mid-span displacement curves of C100 + 0.2 % SF test beams (four specimens)

Table 4 Comparison of the structural ductility index β^*

Concrete mix and beam type	Tensile splitting strength f_t' (MPa)	Yield strength of steel f_{yk} (MPa)	Characteristic length of mix l_{ch} (mm)	Beam depth W (mm)	Reinforce-ment ratio ρ	Beam Ductility Index β^*
C50	4.0	500	305	250	0.0109	0.83
C100 + 0.2 % SF	8.4	500	375	225	0.0119	1.82
C100 + 0.2 % SF	8.4	500	375	200	0.0134	1.72

This was achieved by using the same amount of reinforcement as in C50 beams but decreasing the depth of C100 + 0.2 % SF beams from 225 mm (Fig. 1) to 200 mm (Fig. 5). As a result the reinforcement ratio ρ increased from 0.0107 in C50 beams of 250 mm depth to 0.0134 in C100 + 0.2 % SF beams of 200 mm depth. The load carrying capacity of these beams (Fig. 5) is expected to be lower than the beams shown in Fig. 1 but still significantly higher than nominal ultimate load (120 kN). These beams were also tested in four-point bending over a span of 1500 mm and the mid-span deflection was recorded. The recorded load-deflection plot is shown in Fig. 6.

As expected the ultimate load of C100 + 0.2 % SF beams of 200 mm depth was only 160 kN, which is significantly higher than the nominal design load 120 kN, but less than EC2 recommendation of 171.4 kN. It should be mentioned that the EC2 recommendation is based on a distributed patch loading on the middle third of the beam, but we have used the more severe point loading. Moreover, as the intention of this limited investigation was only to examine how the response of C100 + 0.2 % SF beams would change if they had

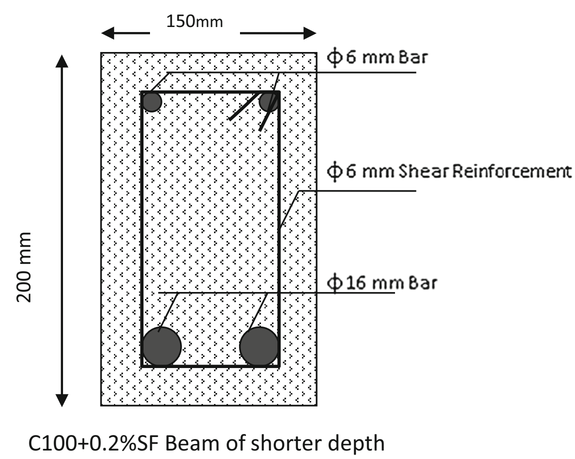


Fig. 5 Cross section of C100 + 0.2 % SF concrete beam with shorter depth (200 mm)

a larger reinforcement than the C50 beams, not necessarily as large as recommended by EC2 (Tables 2, 3), but larger than that in C50 beams nonetheless, then that aim has been achieved. The larger reinforcement ratio (0.0134) in the C100 + 0.2 % SF beams of 200 mm depth than in same grade mix beams of 225 mm depth

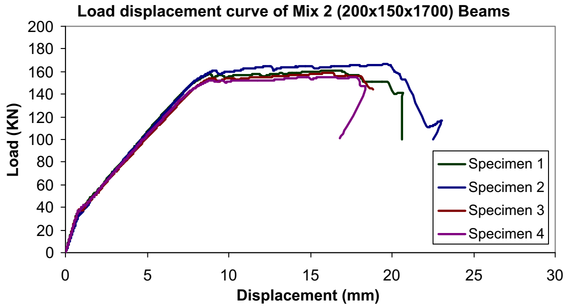


Fig. 6 Load displacement plots of shorter beams (1700 mm × 200 mm × 150 mm). Mix 2 refers to C100 + 0.2 % SF

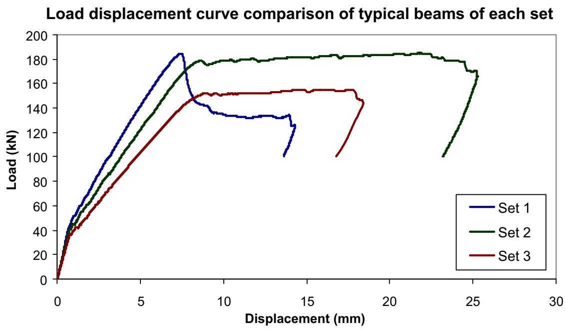


Fig. 7 Comparison of typical load displacement curves of three sets of beams. Set 1 refers to C50 beams, set 2 to C100 + 0.2 % SF beams of 225 mm depth, and set 3 to C100 + 0.2 % SF beams of 200 mm depth

(0.0119) does reduce their ductility (Fig. 7). This is easily explained by using the structural ductility index β^* [Eq. (3)]. Table 4 gives the calculated β^* for the three types of beam. The material properties of the concrete grades C50 and C100 + 0.2 % SF are taken from Table 1.

The structural ductility index of Table 4 is consistent with the experimental observations of Fig. 7—the response of C100 + 0.2 % SF beams of 225 mm depth is slightly more ductile than those of C100 + 0.2 % SF of 200 mm depth, but considerably more ductile than those of C50 beams.

It is seen also from Fig. 7 that the stiffness of the C100 + 0.2 % SF beams is lower than that of C50 beams between the end of elastic response and the attainment of the ultimate load. This is a result partly of the smaller depth of these beams and partly because of the manner in which damage evolves in them in the form of cracks. In C50 beams one observes few flexure cracks in the loaded middle third and a diagonal crack from the support to the nearest loading point. As the load is increased, the flexural cracks stop growing, whereas the diagonal crack keeps on becoming wider. Failure takes place rather suddenly when the open diagonal crack meets crushed concrete under the load (Fig. 8). In C100 + 0.2 % SF beams on the other hand the flexure cracks that form in the loaded middle third of the span are distributed evenly because of the presence of steel fibres. They grow towards the load points under increased loading but because of the closure pressure exerted by fibres their faces do not open much. Failure takes place when one of these flexural cracks meets crushed concrete under the load (Figs. 9, 10).

2.3 Influence of higher mix ductility on structural response

In order to highlight the role of mix ductility (i.e. its characteristic length) on the response of beams made from it, three beams from each of the three mixes C85 + 0.2 % SF, C85 + 0.5 % SF and C85 + 0.75 SF were cast. All beams were 1700 × 250 × 150 mm in dimensions and contained the same longitudinal and shear reinforcement as a C50 beam according to EC2 (see Fig. 1 for C50 beam reinforcement). They were tested in three-point bending (not four-point as previously) over a span of 1500 mm. The load mid-span deflection diagrams are shown in Figs. 11, 12 and 13. All beams failed in flexure with the mean ultimate loads of the beams from the three mixes being 155, 162, and 166 kN, respectively. The response was highly ductile with an extended yield plateau (Figs. 11, 12, 13). In fact, this resulted in one instance in the buckling of the top longitudinal reinforcing bar at failure (Fig. 14).

From a comparison of Figs. 11, 12 and 13 it can be concluded that an increase in the C85 mix characteristic length by an increase in the volume fraction of steel fibre has a marginal effect on the ultimate load carrying capacity and stiffness of the flexural members made from these mixes. The major influence is on the ductility of these members; they have an extended yield plateau, i.e. an extended range of sustained load carrying ability beyond the maximum load. Moreover, this extended yield plateau becomes smoother (i.e. has fewer local instabilities that cause fluctuations in the load displacement diagram) with an increase in the fibre content. This is a result of an increase in the density of micro-cracks with an increase in the fibre content (Fig. 15).

Fig. 8 Typical failure pattern of C50 beams

Fig. 9 Typical failure pattern of C100 + 0.2 % SF beams of 225 mm depth

2.4 Short beams

Several short beams ($150 \times 300 \times 1000$ mm) were designed from C40 and C100 + 0.2 % SF concrete mixes using only the minimum longitudinal (and shear) reinforcement according to EC2 (Tables 2, 3) for C40 concrete (Fig. 16). The beams were tested in four-point bending over a span of 850 mm (Fig. 17) and the mid-span deflection was recorded. The load-deflection plots are shown in Figs. 18 and 19. It is clear from a comparison of these figures that the C100 + 0.2 % SF beams exhibit a far more ductile response than the C40 beams despite the fact that they have the same span to depth ratio and contain the same longitudinal and shear reinforcement as the C40 beams. This proves that there is no need to increase the minimum longitudinal and shear reinforcement in the higher grade concrete short beams to ensure ductility, as it is recommended in EC2. That

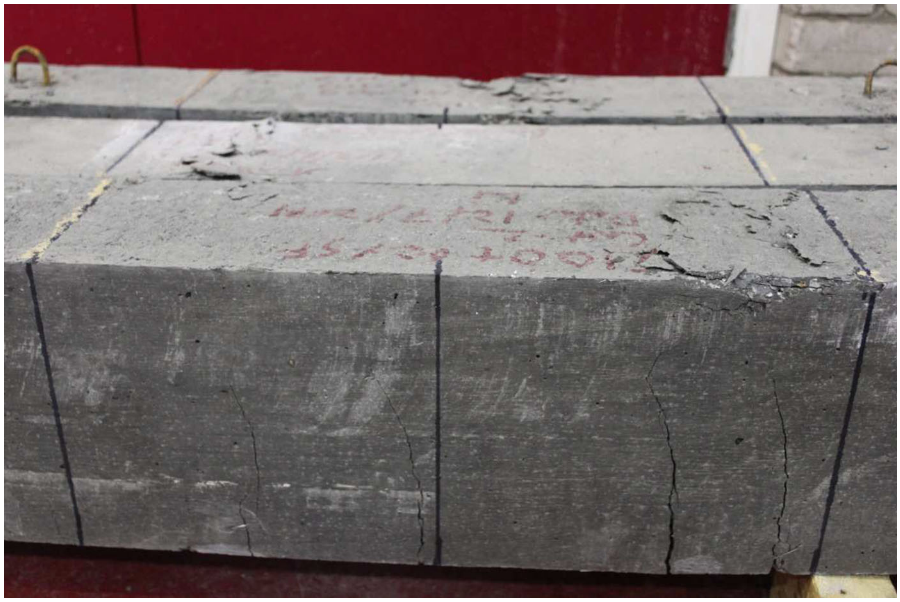


Fig. 10 Typical failure pattern of C100 + 0.2 % SF beams of 200 mm depth

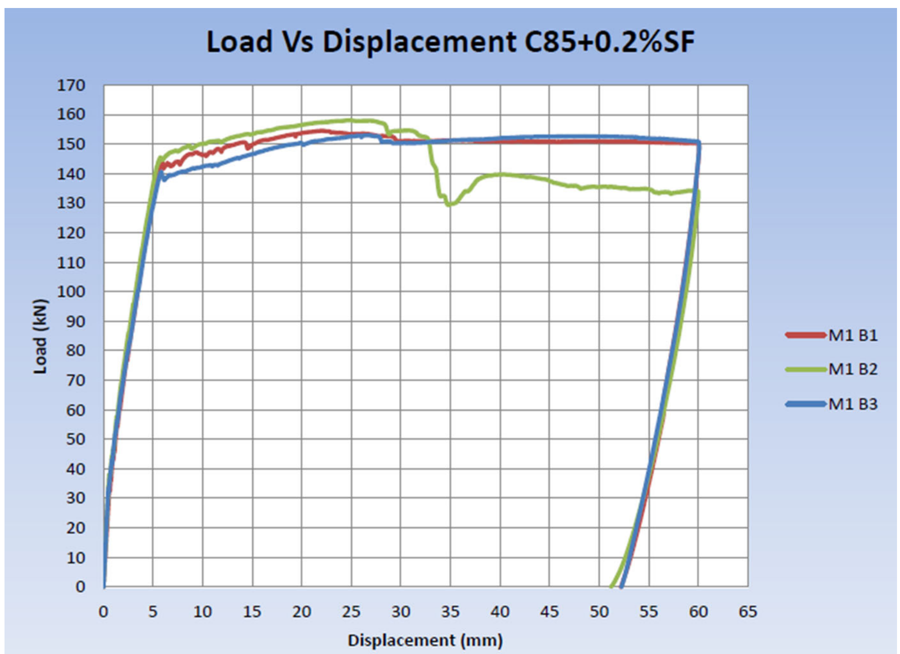


Fig. 11 Load displacement diagrams of C85 + 0.2 % SF beams

the C100 + 0.2 % SF beams exhibited a more ductile response than the C40 beams is again not surprising in view of the fact that the C100 + 0.2 % SF mix is more ductile than the C40 mix (i.e. has a longer l_{ch}; see Table 1). Moreover, the superior load carrying capacity of the C100 + 0.2 % SF beams is solely a reflection of the superior strength properties of this mix over the C40 mix.

3 Column longitudinal reinforcement

EC2 (section 9.5.2) provides all the necessary data and relationships to calculate the longitudinal reinforcement of columns. The following provisions cover only the columns in which the larger dimension h is up to four times greater than the smaller b. The minimum diameter of longitudinal reinforcement steel bars is rec-

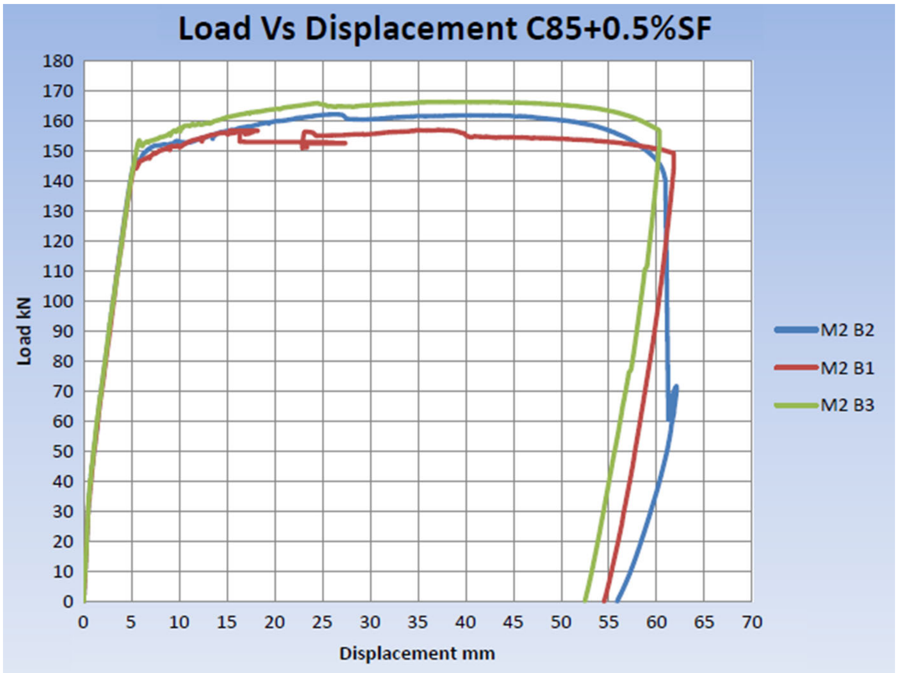


Fig. 12 Load displacement diagrams of C85 + 0.5 % SF beams

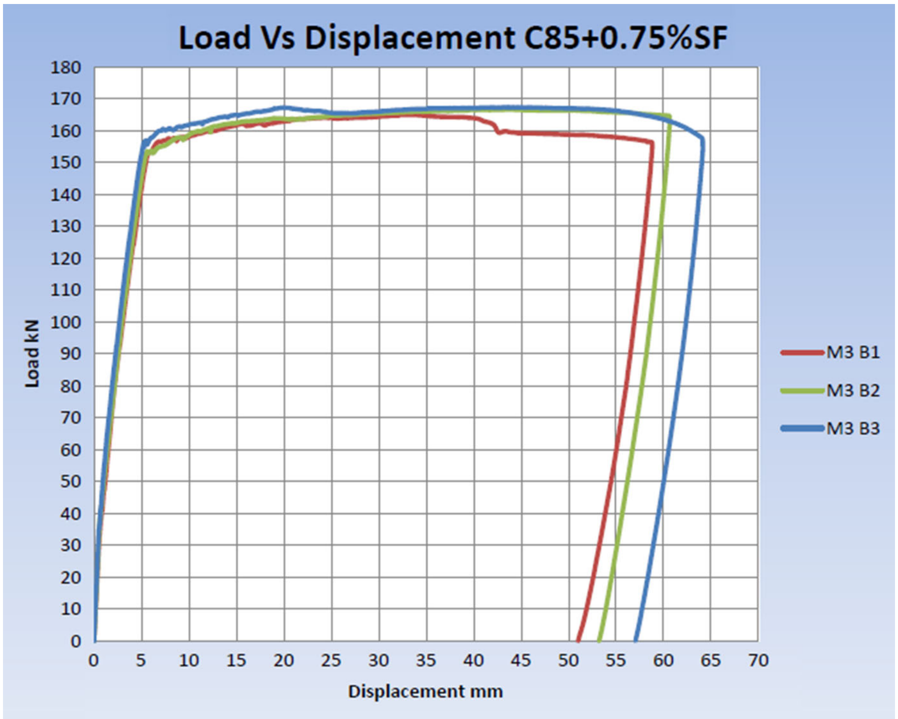


Fig. 13 Load displacement diagrams of C85 + 0.75 % SF beams

Fig. 14 C85 + 0.2 % SF beam B2 (Fig. 11) at failure when the mid-span deflection to span ratio had reached 60/1500. Note the buckle in the longitudinal compressive reinforcing bar which was accompanied by a drop in the load (Fig. 11). This was an exception observed only in one beam out of the nine tested

Fig. 15 A very high density of micro-cracks in a C85 + 0.75 % SF beam

ommended to be more than $\varphi_{min} = 8$ mm and the minimum amount of longitudinal reinforcement must be the greater of the following:

$A_{s,min} = (0.1N_{ed}/f_{yd})$ or $A_{s,min} = 0.002A_c$

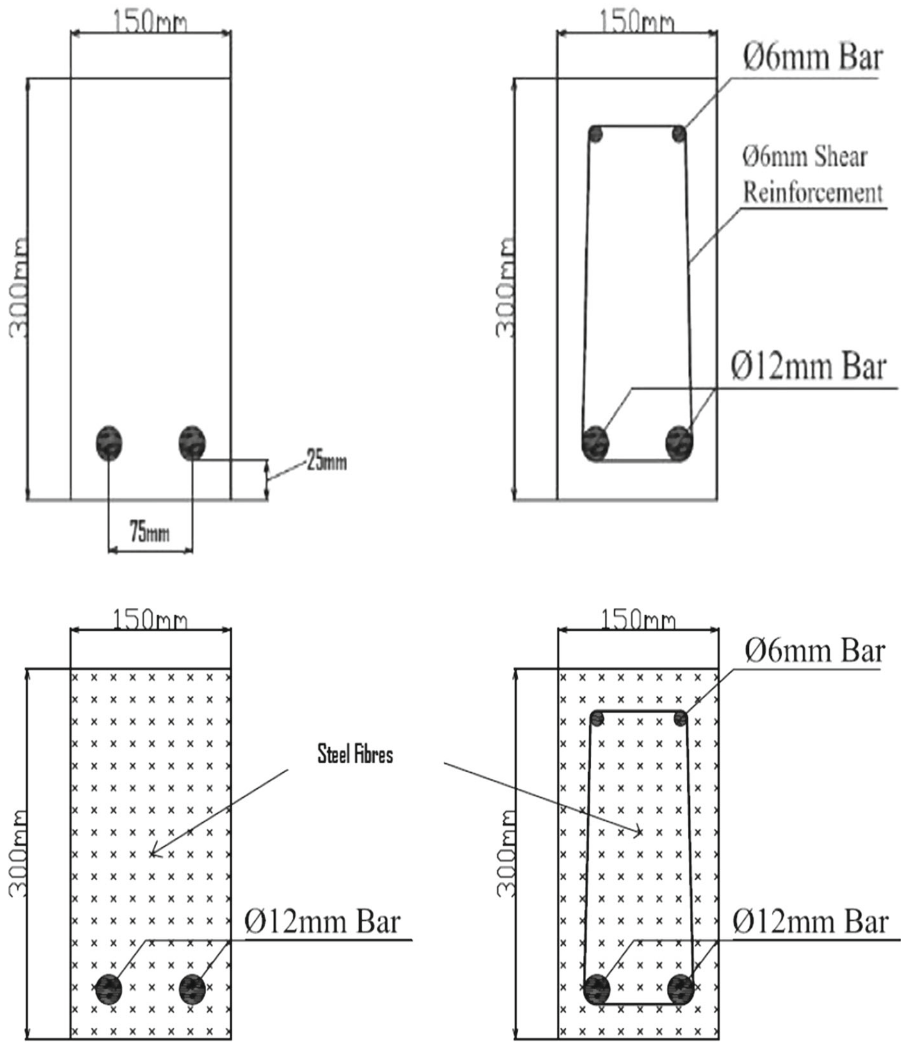
where f_{yd} is the design yield strength of the reinforcement, N_{ed} is the design axial compressive force, and A_c is the column cross sectional area.

The maximum permitted area of longitudinal reinforcement $A_{s,max}$ should not exceed $0.04A_c$ outside lap locations. The only exception is in the case where concrete integrity does not seem to be affected and the full strength is attained at ULS. Then, the above mentioned value can be increased to $0.08A_c$ at laps. At least four longitudinal steel bars are required in the case of circular cross section columns and one bar at each corner for the corresponding polygonal section.

3.1 Column transverse reinforcement

In conformity with section 9.5.3 of EC2, the diameter of transverse reinforcement (links, loops or helical spi-

Fig. 16 Cross sections of C40 and C100 + 0.2 % SF beams (150 × 300 × 1000 mm) with and without shear reinforcement



ral reinforcement) needs to be larger than one quarter of the diameter of longitudinal bars or 6 mm. In addition, the least diameter of the wires of welded mesh fabric for transverse reinforcement should be 5 mm. The code also highlights the need for adequate anchorage of the transverse reinforcement. Finally, the maximum spacing of transverse reinforcement should be less than $s_{cl,tmax}$, where $s_{cl,tmax}$is the smallest of the following three distances: 20 times the minimum diameter of longitudinal bars, the lesser dimension of the column, or 400 mm.

3.2 Slender columns

Twelve slender columns (Table 5), four from each of the three concrete mixes, C40, C110, C110 + 0.18 % SF were designed according to EC 2 using only the minimum longitudinal and transverse reinforcement for C40 grade concrete. The details of the reinforcement are shown in Figs. 20 and 21. The columns were tested in a universal testing machine. Each column was first placed in a safety cage before being located in the testing machine (Figs. 22, 23). The peak load and the corresponding lateral deflection were recorded, as well as the mode of failure (Tables 6, 7, 8). As expected, all C40 columns failed by buckling (Fig. 24), whereas all the C110 columns failed by compression crushing (Fig. 25). Two of the four C110 + 0.18 % SF failed by buckling (Fig. 26) and two by compression crushing (Fig. 27). The variability in the failure mode of C110 + 0.18 % SF columns is explained by the fact that the buckling and compression crushing loads are very

Fig. 17 Four point bending test set up of short beams over a span of 850 mm (span to depth ratio = 2.83)

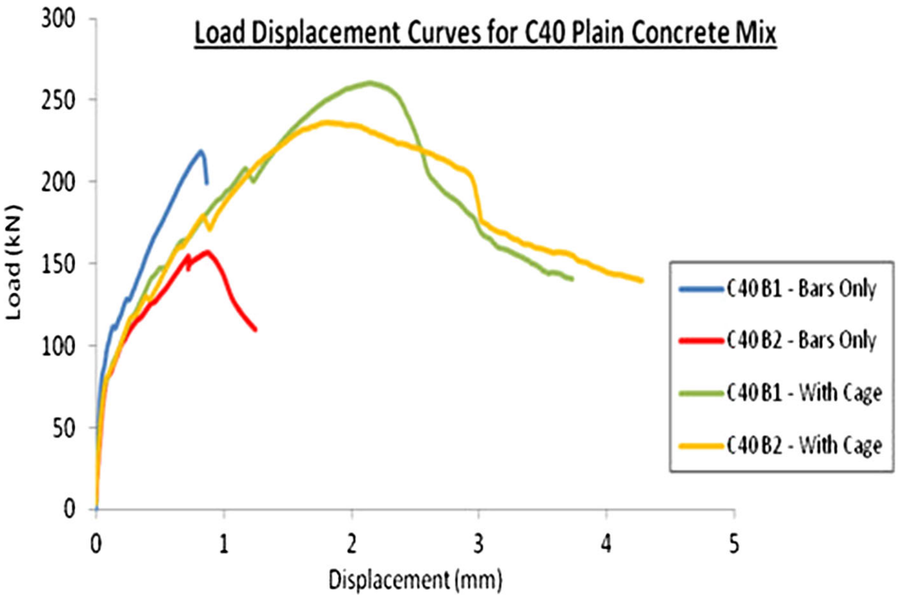


Fig. 18 Load-mid-span displacement diagrams of C40 beams with and without stirrups

nearly the same (cf. Tables 7, 8), because the increase in stiffness induced by the fibres is negligible (Table 1 for Young's modulus of C110 and C110 + 0.18 % SF mixes).

The most important observation here is that the slender columns made from C110 + 0.18 % SF exhibit the same behaviour as the C40 slender columns, despite containing the same longitudinal and transverse reinforcement as the C40 grade columns. There is thus no need to provide any additional confining reinforcement to prevent sudden and explosive brittle failure of the column. The confinement is provided locally by the steel fibres. It should be noted that the increase in the buckling load of slender columns from 458 kN for C40 grade to 1192 kN for C110 + 0.18 % SF is therefore solely thanks to the higher concrete grade. Thus, the superior performance of the high strength concrete has been fully utilised without any additional steel rein-

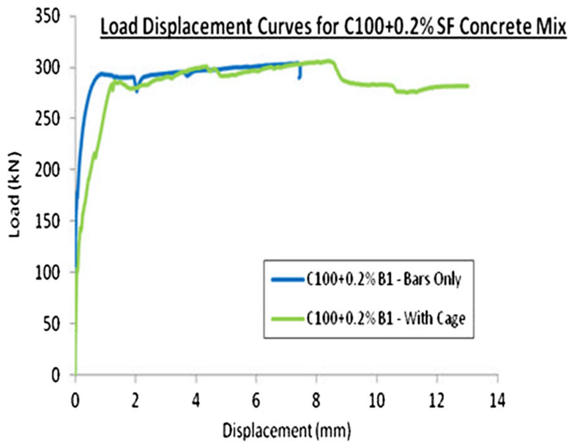


Fig. 19 Load-mid-span displacement diagrams of C100 + 0.2 % SF beams (span to depth ratio = 2.83) with and without stirrups

Table 5 Dimensions of slender test columns

Mix	Quantity	Dimensions (mm)	Reinforcement
C40	4	120 × 120 × 2000	EC2 Standard
C110	4	120 × 120 × 2000	As for C40
C110 + 0.18 % SF	4	120 × 120 × 2000	As for C40

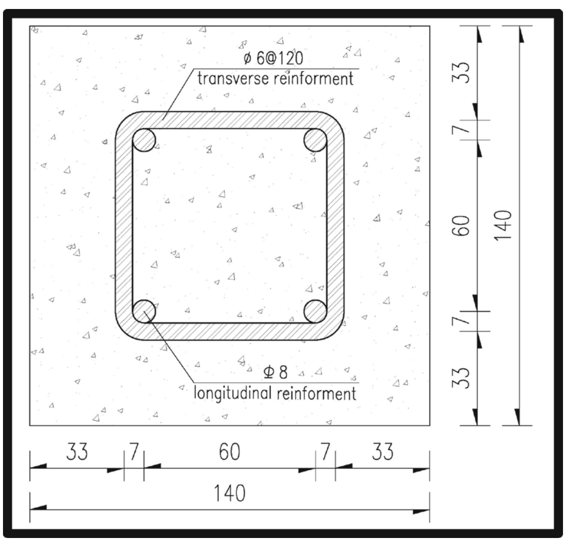


Fig. 20 Cross section of column and reinforcement arrangement. All dimensions in mm

forcement and without compromising the safety of the column. This proves that the current provisions of EC 2 code for high grade concrete slender columns are both over-conservative and unnecessary.

4 Conclusions

From the extensive series of tests on long and short beams and on slender columns, it has been conclusively demonstrated that:

(i) there is no need to increase the minimum amount of reinforcement in RC structures with an increase in the grade of concrete used in the construction, as required by EC2 and similar codes worldwide;

(ii) the amount of reinforcement needed in C100 or C110 concrete structures is the same as in C40 or C50 structures, provided that the characteristic length of the higher grade concrete mix is the same as the lower grade concrete. This is achieved by the inclusion of a small volume fraction of steel fibres in the higher grade mix;

(iii) the ductility of the structures does not deteriorate if the characteristic length of the concrete mixes is the same. There is thus no need to provide any additional confining reinforcement to prevent sudden and explosive brittle failure of the RC structure made from the higher grade concrete beyond that needed for the lower grade concrete. The confinement is provided locally by the steel fibres. In fact, if the amount of reinforcement in C100 or C110 grade structure is more than that in C50 grade structure but the mix characteristic lengths are the same, then the higher grade concrete structure exhibits reduced ductility;

(iv) the increase in load carrying capacity of the higher grade structures is therefore solely thanks to the higher concrete grade. Thus, the superior performance of the high strength concrete has been fully utilised without any additional steel reinforcement and without compromising the safety of the structure. The use of a larger amount of steel fibre in the higher grade concrete than that required for it to have the same characteristic length as the base lower grade concrete increases the stiffness and load carrying capacity only marginally, but does significantly extend the range of sustained load carrying ability beyond the maximum load;

(v) the current provisions of EC 2 code for high grade concrete are both over-conservative and unnecessary.

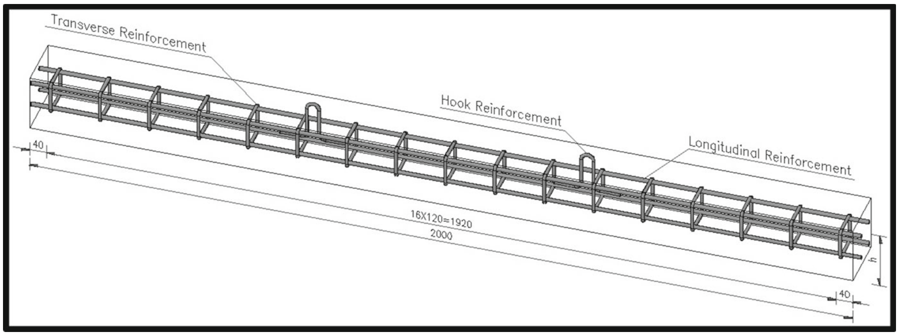


Fig. 21 Longitudinal (four 8 mm bars) and transverse (6 mm bars @ 120 spacing) reinforcement

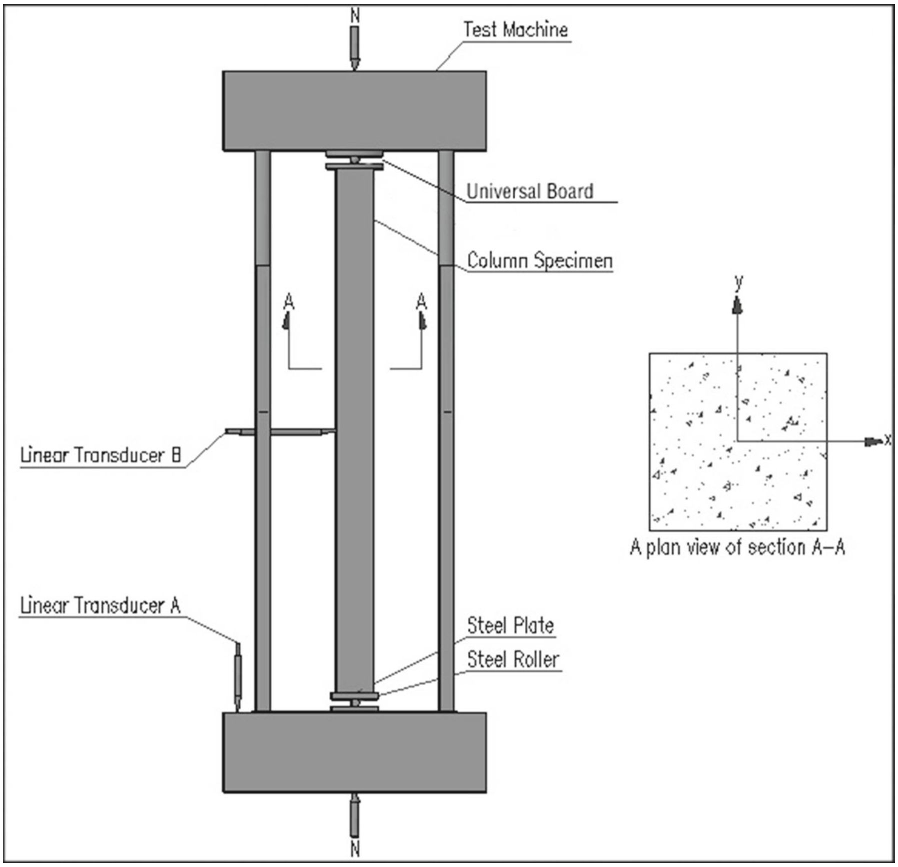


Fig. 22 Specimen loading and displacement measurement arrangements

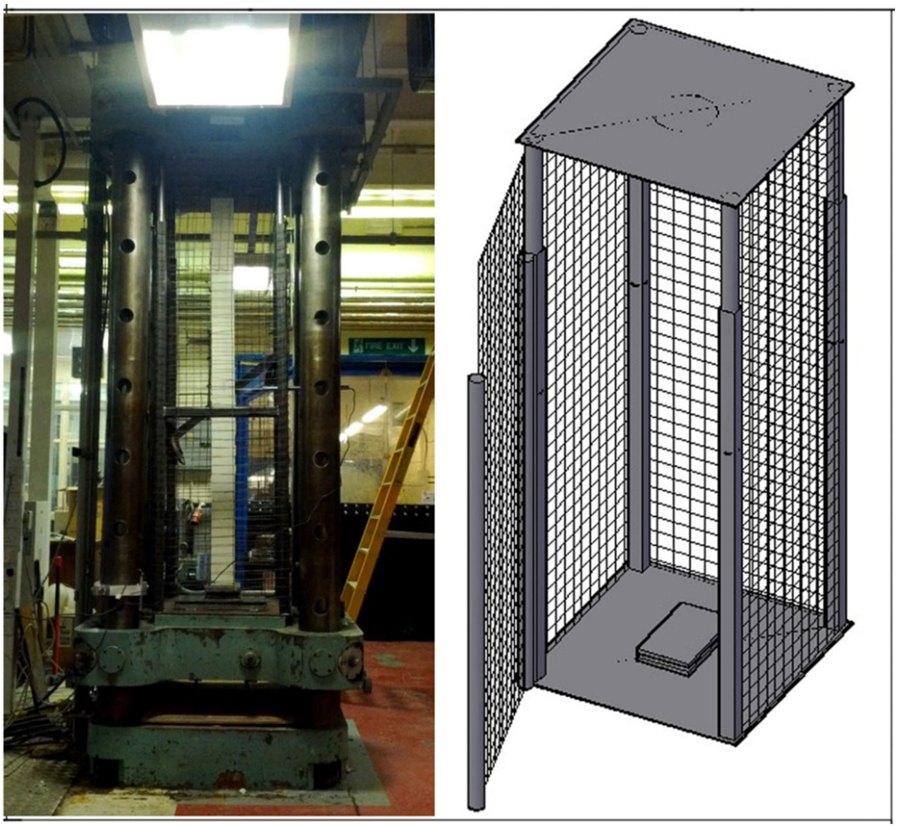


Fig. 23 Column in the testing machine, showing the protection cage

Table 6 Test results of C40 columns

Specimen number	Maximum lateral deflection (mm)	Peak load (kN)	Average peak load (kN)	Coefficient of variation (%)	Failure mode
1	6.30	479.31			Buckling
2	5.19	373.71	457.89	7.63	Buckling
3	6.29	417.28			Buckling
4	4.64	476.78			Buckling

Table 7 Test results of C110 columns

Specimen number	Maximum lateral deflection (mm)	Peak load (kN)	Average peak load (kN)	Coefficient of variation (%)	Failure mode
1	3.78	1129.41			Crushing
2	Transducer failure	881.37	1195.58	7.00	Crushing
3	2.92	1289.71			Crushing
4	1.79	1167.72			Crushing

Table 8 Test results of C110 + 0.18 % SF columns

Specimen number	Maximum lateral deflection (mm)	Peak load (kN)	Average peak load (kN)	Coefficient of variation (%)	Failure mode
1	9.31	1022.67			Buckling
2	6.23	1243.22	1192.54	13.85	Buckling
3	2.13	1104.95			Crushing
4	1.21	1399.31			Crushing

Fig. 24 Buckled C40 column 2

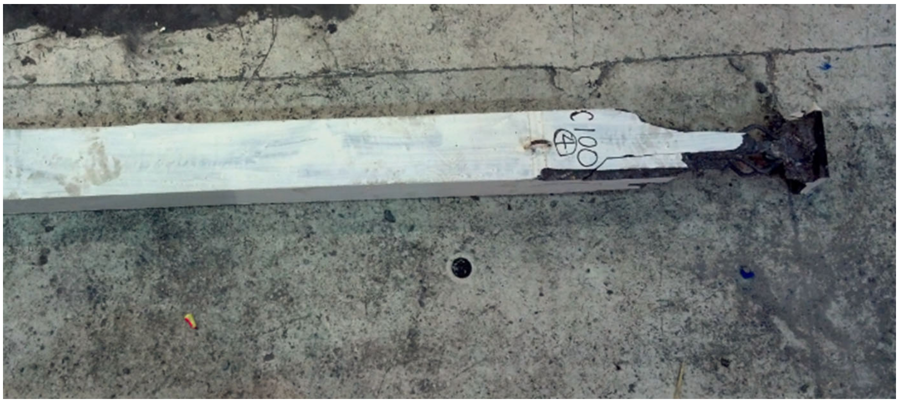


Fig. 25 Crushed C110 column 4

Fig. 26 Buckled C110 + 0.18 % SF column 2

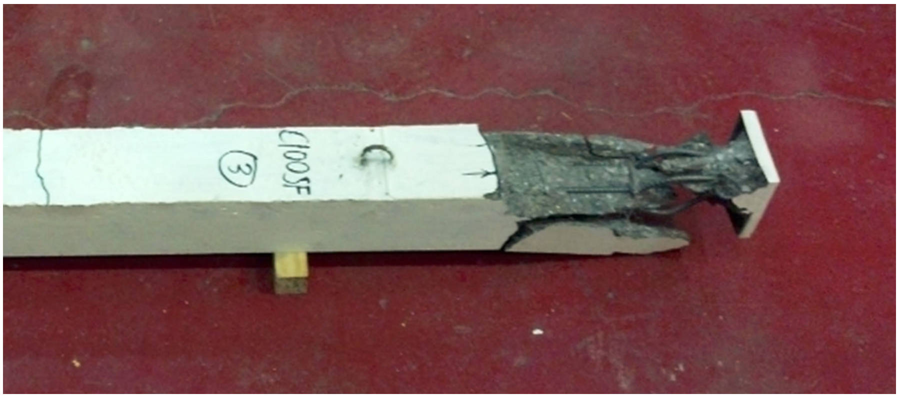


Fig. 27 Crushed C110 + 0.18 % SF column 3

5 Perspectives for the future

The adoption of this new design approach, which is based on sound physical principles, should help promote the use of high performance, durable and sustainable concrete in the construction industry without increasing the cost of construction or compromising the safety of structures. It is now up to the research community to develop it further and for the code committees to promote it in the concrete structural design community.

(i) The new design approach proposed here points the future direction that the concrete structural design codes should follow in order to make concrete construction truly durable and sustainable without incurring a cost penalty or compromising the safety.
(ii) The design codes should as a matter of priority base their recommendations on sound physical principles and fundamental material properties, rather than on the dubious concrete characteristic compressive strength. Whilst it is commendable that EC2 acknowledges that fracture mechanics can be used for analysing the structural integrity of cracked concrete structures, it is regrettable that it continues to recommend that the specific fracture energy G_F needed for this analysis may be inferred from the concrete characteristic compressive strength (an energy inferred from a stress!). Such non-scientific approach is fundamentally wrong, just as it is fundamentally wrong to infer the tensile strength and the elastic modulus from this dubious material property (a stress inferred from the fractional power of a stress!).
(iii) It is time that the many international committees that have been addressing the problem of the measurement of size-independent G_F for many years agree on a standard procedure. However, it should be emphasised that whichever method is eventually agreed upon as standard, the conclusions reached in this paper will remain in force, because the agreed method will affect all grades of concrete, so that their relative characteristic lengths will remain essentially unaltered.

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