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Evaluation of ductility requirements in current design guidelines for FRP strengthening

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Abstract

Advanced composites are widely used for the strengthening of existing concrete structures. Current design guidelines give basic requirements on how to model the enhancement of structural performance of concrete members using surface bonded FRP (fibre reinforced polymer) reinforcement. With respect to this, it is of interest to evaluate the ductility requirements which are explicitly or implicitly imposed by design guides. Based on an evaluation of four major design guidelines in Europe, Japan and North-America, and a small parametric study, the ductility aspect of the design of FRP strengthened concrete members is verified. It appears that the ductility of flexural members strengthened with FRP should be considered with care, as reduced deformability is obtained at ultimate, though generally a minimum deformability is implicitly obtained in a proper design. At the other hand, ductility enhancement by means of FRP confinement is explicitly considered in the design guidelines.

Keywords: Fibre reinforced polymer; Externally bonded reinforcement; Concrete; Design guidelines; Deformability; Ductility

1. Introduction

Each year, considerable investments in construction engineering are related to the maintenance, repair (retrofit) and strengthening (upgrading) of infrastructure. Among the different techniques available for repair and strengthening of existing concrete structures, systems based on advanced composites as externally bonded reinforcement are nowadays often applied worldwide, as they appear efficient and competitive [1–3]. The strengthening of the concrete members is achieved with surface bonded FRP (fibre reinforced polymer) systems, though alternative systems such as near surface mounted FRP and prestressed FRP are available as well. The application of FRP EBR (externally bonded FRP reinforcement) combines excellent material properties with ease and flexibility of application, which makes this technique attractive.

Whereas applications in Europe have taken place since the late 1980's, commercial use of externally bonded FRP reinforcement (FRP EBR) started mainly in Switzerland around 1993 and soon followed in other countries [1]. Based on extensive experimental and analytical investigations by various researchers and the appearance of different design guidelines during mainly the last 5 years, the use of FRP EBR is rapidly becoming a standard technique.

Existing design guidelines give basic requirements on how to model the enhancement of flexural and shear strength and confinement action on concrete. Further to these design models, it is of interest to understand which ductility requirements are considered with respect to FRP strengthening (see also next section). The following design guidelines in Europe, Japan and North-America are considered in this investigation on ductility of concrete members strengthened with surface bonded FRP:

– *fib* Bulletin 14 'Design and use of externally bonded fibre reinforced polymer reinforcement (FRP EBR) for reinforced concrete structures' [4]. This technical report is abbreviated in the following as fib14.

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Nomenclature

A_f	cross-sectional area of the external FRP reinforcement;
A_g	gross concrete cross-section;
A_r	cross-sectional area of the reinforcement;
A_s	cross-sectional area of the steel reinforcement;
b	width of the beam;
b_f	width of the FRP;
d	effective beam depth;
E_c	modulus of elasticity of the concrete;
E_f	modulus of elasticity of the FRP reinforcement;
E_s	modulus of elasticity of the steel reinforcement;
$E_f t$	unit stiffness of the FRP reinforcement;
f_{cd}	design value of the compressive cylinder strength of concrete;
f_{ck}	characteristic compressive cylinder strength of concrete;
f_{cc}	confined concrete strength;
f_{co}	unconfined concrete strength;
f_{ctm}	mean value of the concrete tensile strength;
f_l	maximum lateral confining pressure
f_y	yield strength of the steel reinforcement;
f_{yk}	characteristic value of the yield strength of the steel reinforcement;
G_f	interfacial fracture energy between the FRP and the concrete;
k_v	constant which depends on the beam and load configuration;
ℓ	span of the beam;
M_{Ad}	resisting moment corresponding to accidental loss of the FRP EBR;
M_n	nominal flexural strength;
M_0	acting moment during strengthening;
M_{RdS}	resisting design moment of the strengthened member;
M_{RdU}	resisting design moment of the unstrengthened member;
M_{serS}	service moment of the strengthened member under rare load combination;
M_{serU}	service moment of the unstrengthened member under rare load combination;
M_u	required moment strength referring to factored loads;
n	number of plies;
Q	point load;
R_d	resisting design load;
R_n	nominal strength;
S_d	acting design load;
S_{DL}	dead load effects;
S_{LL}	live load effects;
t	total thickness;
t_f	effective thickness of one ply (one layer);
x	depth of the compression zone;
χ_{dS}	curvature of the strengthened member at ultimate limit state;
χ_{yU}	curvature of the unstrengthened member when reaching the characteristic yield strain;
δ	displacement;
ε_c	concrete strain;
ε_{cc}	ultimate axial strain of the confined concrete;
ε_{co}	concrete strain at unconfined strength;
ε_{fe}	maximum effective FRP strain;
ε_{fu}	(design) rupture strain of the FRP;
ε_s	strain in the steel reinforcement;
ε_{sy}	yield strain of the steel reinforcement;
Φ_{rS}	global load safety factor between design load and rare load combination (strengthened);
Φ_{rU}	global load safety factor between design load and rare load combination (unstrengthened);
ϕ	strength reduction factor;
γ_b	member safety factor;
γ_i	structure safety factor;
η_a	ratio M_{serS}/M_{Ad};
η_0	ratio M_0/M_{serU};
η_S	strengthening ratio;
ρ_f	FRP reinforcement ratio;
ρ_r	reinforcement ratio;
ρ_s	steel reinforcement ratio;
μ_{Rd}	normalized resisting design moment;
μ_Δ	ductility index (in terms of displacement);
μ_χ	ductility index (in terms of curvature);
v	factor $\ell/(k_v d)$;
ξ	normalized depth of the compression zone;

– ACI 440.2R-02 'Guide for the design and construction of externally bonded FRP systems for strengthening of concrete structures' [5], abbreviated in the following as ACI440.2R.
– CSA S806-02 'Design and construction of building components with fibre-reinforced polymers' [6]. This code is abbreviated in the following as CSA-S806.
– JSCE Concrete Engineering Series 41 'Recommendations for upgrading of concrete structures with use of continuous fibre sheets' [7], abbreviated in the following as JSCE41.

It is noted that more design guidance documents are available (e.g. [8–10]), however the study has been restricted to *fib* Bulletin 14 and the mentioned documents from USA, Canada and Japan. The *fib* bulletin can be regarded as a design guideline following the Eurocode design format, and has been issued by *fib* (International

Federation of Structural Concrete) Task Group 9.3 'FRP Reinforcement for Concrete' of which the authors are secretary and convenor, respectively.

In the present paper, an evaluation is given of the ductility aspect of the design of FRP EBR strengthened concrete members. In addition, the results of a parametric study are reported with respect to this issue for flexural strength enhancement of simply supported concrete members.

2. Initial considerations

FRP materials exhibit a linear elastic stress–strain behaviour in tension, and do not have the typical yield plateau of steel. As the ultimate strain capacity of FRP in tension is about 5–10 times higher than the yield strain of steel rebars, ductile behaviour of flexural members strengthened with FRP EBR can be expected. This depends however on the obtained FRP strain at failure of the strengthened member. Particular for the use of externally bonded FRP reinforcement is that the FRP stresses are transferred to the concrete through bond mechanisms. The ability of the concrete to withstand the initiated bond shear stresses is however limited, so that bond failures may govern the ultimate limit state. As a result, the effective ultimate FRP strains are considerably lower than the ultimate strain capacity of the FRP when performing a tensile test. Even in case bond failures are postponed or avoided (e.g. by extra mechanical fixings), limited FRP strains at ultimate may be achieved. Indeed, in most design cases the amount of FRP needed for the strengthened member is resulting in an over-reinforced section, e.g. because sufficient flexural stiffness should be provided to fulfil the serviceability criteria. From these observations it becomes clear that ductility of flexural members strengthened with FRP EBR is less obvious than for unstrengthened steel reinforced concrete members, whereas the latter are characterized by extensive inelastic deformations near ultimate due to yielding of the steel rebars. This is illustrated in Fig. 1, showing the load–deflection behaviour of CFRP (carbon fibre reinforced polymer) strengthened beams having a width of 200 mm, a total depth of 450 mm and a span of 3.8 m, tested in 4-point bending [11]. The beams have the same steel reinforcement ratio and two levels of strengthening are shown.

In the case of confinement of concrete columns a different situation is obtained. For the FRP wrapped columns, the fibres run essentially perpendicular to the longitudinal axis of the member and provide lateral support to the expanding concrete. The resulting multi-axial stress state of the confined concrete is characterized by large deformability, resulting from the increased effective compressive strain of the concrete. This is illustrated in Fig. 2 for axially loaded columns with a diameter of 400 mm and a length of 2 m, wrapped with different types of FRP [11]. Given the increased strain capacity, confinement of concrete is of special interest for ductility enhancement and particularly applies to seismic rehabilitation in achieving sufficient lateral drift capabilities of the structure.

3. Approaches regarding ductility of flexural members

As pointed out, ductility of concrete members strengthened in bending with FRP EBR may be of concern. Ductility is typically addressed by a strain level criterion and corresponds rather to deformability, as no explicit indications with respect to energy dissipation are given in the considered guidelines. Ductility enhancement by means of confinement of concrete is discussed in Section 4.

For the design of flexural enhancement in the ultimate limit state, the considered guidelines relate to a number of failure modes and hence multiple design verifications.

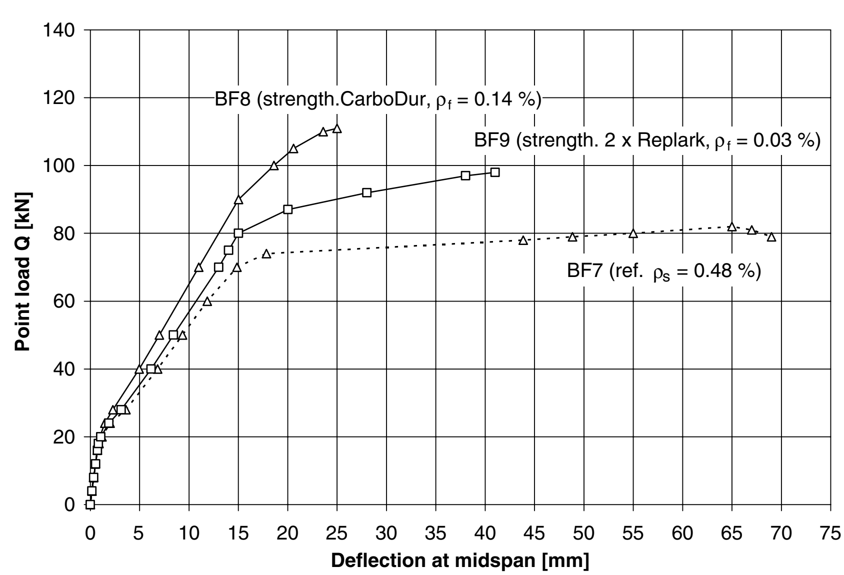

Fig. 1. Load–deflection behaviour of concrete beams strengthened in flexure.

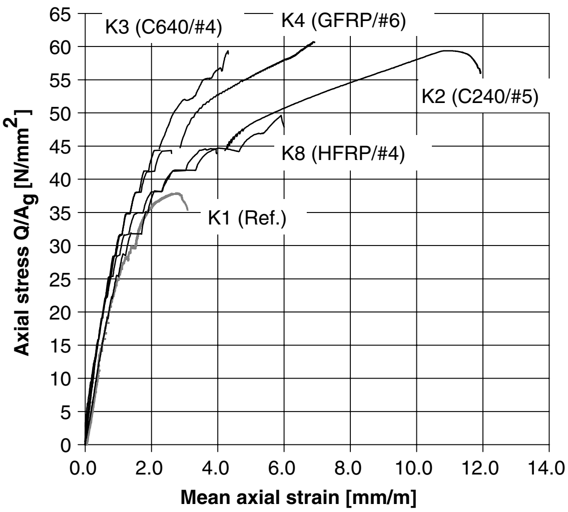

Fig. 2. Axial stress–strain behaviour of confined columns.

Depending on the governing failure mode, the resulting FRP strain at ultimate may strongly differ and hence so the deformability at ultimate. Given the calculation models, a certain FRP strain at ultimate will govern the design and will yield a degree of deformability. This aspect, which may be regarded as an implicit deformability criterion, is discussed hereafter and is further investigated by means of a parametric study reported in Section 5. Given the ductility concern, the considered guidelines, except for CSA-S806 and JSCE41, also specify a minimum deformability requirement.

3.1. Minimum deformability

A target deformability is given in fib14 and ACI440.2R. No explicit minimum deformability criterion was found by the authors in JSCE41 or CSA-S806, though in the latter it is required that 'strengthening of a member shall not result in the transformation of a ductile failure mode of the unstrengthened member to a brittle failure mode of the strengthened member'.

The fib14 refers to EC2, Section 2.5.3.4.2(5) [12] which gives (in the critical design section of continuous girders and stiff frames) the following limitation on the depth of the compression zone at ultimate:

$$\begin{aligned} &\xi \leqslant 0.45 \quad \text{for} \quad \leqslant C35/45 \\ &\xi \leqslant 0.35 \quad \text{for} \quad > C35/45 \end{aligned} \tag{1}$$

where $\xi = x/d$, with x the depth of the compression zone and d the effective beam depth. The criterion depends on the concrete class, whereas $C35/45$ stands for a concrete with characteristic compressive cylinder (diameter 150 mm × 300 mm) strength f_{ck} of 35 MPa or a characteristic compressive cube (side length 150 mm) strength of 45 MPa. For higher concrete classes a larger minimum deformability is required at ultimate, as a smaller depth of the compression zone will correspond with increased curvature (ultimate concrete strain divided by depth of compression zone) at ultimate. Assuming an ultimate concrete strain of 0.0035 [12], Eq. (1) corresponds to the following criterion for the strain in the internal steel reinforcement at ultimate [4]:

$$\begin{aligned} &\varepsilon_s \leqslant 0.0043 \quad \text{for} \quad \leqslant C35/45 \\ &\varepsilon_s \leqslant 0.0065 \quad \text{for} \quad > C35/45 \end{aligned} \tag{2}$$

Further to this criterion, fib14 specifies that the ductility requirement should no longer be fulfilled if there exists extra safety between the acting (S_d) and the resisting (R_d) design load in ultimate limit state

$$R_d \geqslant 1.2S_d \quad (\text{or } 0.83R_d \geqslant S_d) \tag{3}$$

ACI440.2R refers to ACI318-99 [13] Chapter 2 and Appendix B, specifying that the strain in the steel reinforcement at ultimate should be at least 0.005 to maintain a sufficient degree of ductility, whereas a section with low ductility should compensate with a higher reserve of strength

$$\phi = \begin{cases} 0.90 & \varepsilon_s \geqslant 0.005 \\ 0.70 + \frac{0.20(\varepsilon_s - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} & \varepsilon_{sy} < \varepsilon_s < 0.005 \\ 0.70 & \varepsilon_s \leqslant \varepsilon_{sy} \end{cases} \tag{4}$$

with $\phi \geqslant M_u/M_n$, M_u the required moment strength referring to factored loads and M_n the nominal flexural strength, ε_s the strain in the steel at the ultimate limit state and ε_{sy} the yield strain.

The approach by the two guidelines appears very similar, whereas at one hand a ductility criterion is given, yet at the other hand this ductility criterion can be relaxed or ignored if there exists a degree of extra safety between the flexural design strength versus the acting design load.

Though the target deformability implies yielding of the internal steel at ultimate limit states, the guidelines do not allow inelastic deformations of the reinforced concrete members strengthened with bonded FRP under service load level.

3.2. Ductility versus strength safety factor

From the deformability requirements of fib14 and ACI440.2R, and the lack of it in JSCE41 and CSA-S806, it appears that for flexural strengthening of concrete beams, ductility is not considered as a strong explicit criterion as far as there is a preset strength safety factor. In addition, some degree of deformability will implicitly be provided (see Sections 3.3 and 5) and ductility enhancement considerations with respect to confined concrete and for seismic strengthening are given by the design guidelines.

Regarding ductility versus strength safety fib14 states that if the design is governed by e.g. serviceability criteria, the amount of FRP provided to the structure may be considerably higher than what is needed for the ultimate limit state (ULS). In this case the ductility condition may be difficult to fulfil while a high safety factor between the acting and the resisting design load in ULS is obtained, and hence also a high safety factor between the service load and the resisting design load.

A schematic representation of the strength safety factor as a function of the ultimate strain in the internal steel reinforcement is given in Fig. 3, for fib14 and ACI440. The strength safety factor is considered as the ratio of the resisting flexural design strength of the FRP EBR strengthened concrete member over the acting design load. This corresponds to R_d/S_d for fib14 and $1/\phi$ for ACI440.2R, as given in Eqs. (3) and (4). In JSCE41, a member factor γ_b is mentioned which is generally set to 1.15 for flexural strengthening [7], [Section 6.4 (i)]. In addition, a structure factor γ_i between 1.0 and 1.2 should be considered, expressing the importance of the upgraded structure and the impact on society when it reaches the limit state [7], [Section 6.3(6)]. Note that a direct comparison of these strength safety factors can not be made, as the load and material safety factors to derive the factored or design values of flexural strength and acting load differ between the considered documents.

3.3. Failure modes and ductility

The resisting flexural design strength and the deformability at ultimate of a concrete member strengthened with FRP EBR will basically depend on the controlling mode of failure. An overview of the flexural failure modes to be verified according to the considered guidelines is given in Table 1. A schematic representation of the failure modes is given in Fig. 4.

Assuming that debonding is not occurring, the considered documents give the same failure modes to verify. Whereas two of these failure modes involve yielding of the steel, this implies an implicit deformability, taking $\varepsilon_s > \varepsilon_{sy}$ as ductility criterion. A third failure mode is governed by concrete crushing without steel yield and exhibits no inelastic deformations near ultimate. The latter failure mode is only obtained for strongly over-reinforced sections (see Section 5) and will generally not occur as this would correspond to uneconomically high amounts of external FRP reinforcement.

As illustrated before, the governing failure mode will often be determined by some kind of debonding failure. The approach of the guidelines with respect to debonding failure modes differs considerably (see also Table 1). Compared to the other guidelines, fib14 provides the most detailed way of dealing with different types of debonding failure modes, yet allowing for three different approaches. In the most simplified approach, besides anchorage verification a simplified FRP strain limitation approach is

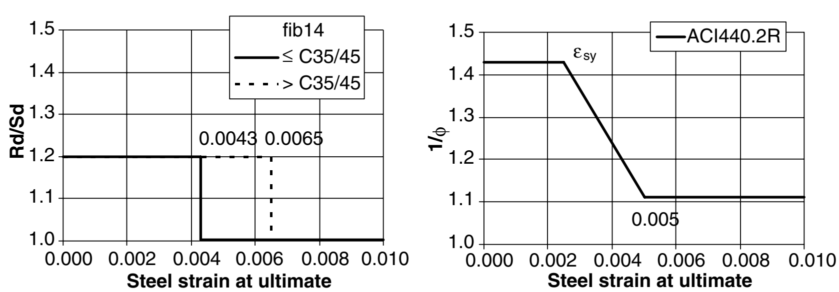

Fig. 3. Schematic overview of strength safety as a function of the strain at ultimate.

Table 1
Controlling flexural failure modes to be verified

Class	Failure mode[a]	Guideline
No debonding	(1) Crushing of concrete before yielding of steel	fib14, ACI440.2R, CSA-S806, JSCE41
	(2) Yielding of steel followed by FRP rupture	fib14, ACI440.2R, CSA-S806, JSCE41
	(3) Yielding of steel followed by concrete crushing	fib14, ACI440.2R, CSA-S806, JSCE41
Debonding	(4) FRP end shear failure/concrete rip-off	fib14
	(5) Peeling-off in an uncracked anchorage zone	fib14
	(6) Peeling-off caused at flexural cracks	fib14
	(7) Peeling-off caused at shear cracks	fib14
	(4) Shear/tension delamination of the concrete cover	ACI440.2R
	(5–7) Debonding of the FRP from the concrete substrate	ACI440.2R
	(4–5) Shear/tension failure of concrete substrate at FRP cutoff point (anchorage failure)	CSA-S806
	(7) Debonding of adhesive bond line due to vertical section translations from cracking (delamination)	CSA-S806
	(4–5) Anchorage failure of the FRP	JSCE41
	(6–7) Interfacial fracture of the FRP to concrete due to progress of flexural cracking and shear cracking	JSCE41

[a] The mode numbers refer to Fig. 4.

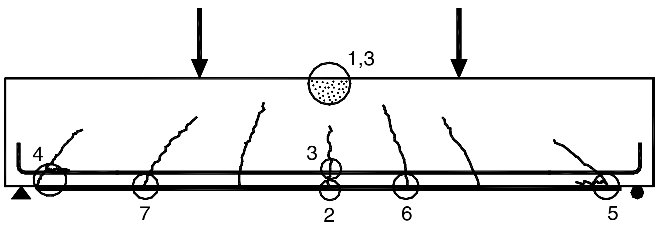

Fig. 4. Schematic representation of failure modes.

applied. However, it is stated that a global strain limit may not be suitable to represent the range of applications and more detailed calculation approaches are given. For the global FRP strain limit reference is made to [14], giving a value ranging from 0.0065 to 0.0085. Assuming a ratio of 1.1 between the level of the external and internal reinforcement, this yields $\varepsilon_s \geqslant 0.0059$–$0.0077$. These provisional numbers indicate that debonding tends to become critical only after yielding of the internal steel (provided that sufficient anchorage length is available to avoid anchorage failure of the external reinforcement).

A constant strain limit is also specified in CSA-S806, whereby the maximum tensile FRP strain shall not be greater than 0.007 (assuming no anchorage failure). In ACI440.2R, an FRP strain limit is specified as a function of the unit stiffness of the FRP, given as the modulus of elasticity E_f times the total thickness $t = nt_f$ (with n the number of plies)

$$\varepsilon_{f\,lim} = \begin{cases} \frac{1}{60}\left(1 - \frac{E_f t}{360\,000\ \mathrm{N/mm}}\right) \leqslant 0.9\varepsilon_{fu} & E_f t \leqslant 180\,000\ \mathrm{N/mm} \\ \frac{1}{60}\left(1 - \frac{90\,000\ \mathrm{N/mm}}{E_f t}\right) \leqslant 0.9\varepsilon_{fu} & E_f t > 180\,000\ \mathrm{N/mm} \end{cases} \tag{5}$$

with ε_{fu} the (design) rupture strain of the FRP. This FRP strain limit is illustrated in Fig. 5. Unless the unit stiffness $E_f t$ is beyond 550000 to 700 000 N/mm (high modulus of elasticity and a large number of plies), debonding will only occur after yielding of the steel. Considering for example CFRP with E_f equal to 165 GPa, the limit $E_f t < 550\,000$ N/mm corresponds to a thickness limitation $t < 3.33$ mm.

In JSCE41 it is stated that no initiation of peeling of the FRP occurs when the stress in the FRP at the location of flexural cracking caused by the maximum bending moment satisfies

$$\sigma_f \leqslant \sqrt{\frac{2G_f E_f}{nt_f}} \tag{6}$$

with G_f the interfacial fracture energy between the FRP and the concrete, which can be estimated as 0.5 N/mm [7]. Taking into account the linear elastic behaviour of the FRP, Eq. (6) can be written as an FRP strain limit

$$\varepsilon_f \leqslant \sqrt{\frac{2G_f}{nt_f E_f}} \tag{7}$$

This strain limit is also given in Fig. 5. It should be recognized that this limit corresponds to initiation of debonding and not bond failure. Higher FRP strains are allowed by JSCE41 before attaining debonding, provided that the difference in FRP tensile stress between two sequel flexural cracks fulfils Eq. (6) (a similar approach is used in fib14).

From the above observations it can be tentatively concluded that ductility, considering $\varepsilon_s > \varepsilon_{sy}$ as minimum criterion, will be implicitly guaranteed in the design unless very high amounts of FRP or numbers of plies are applied for flexural strengthening by means of FRP EBR, and assuming that proper reinforcement detailing is provided, e.g. with respect to FRP end anchorage.

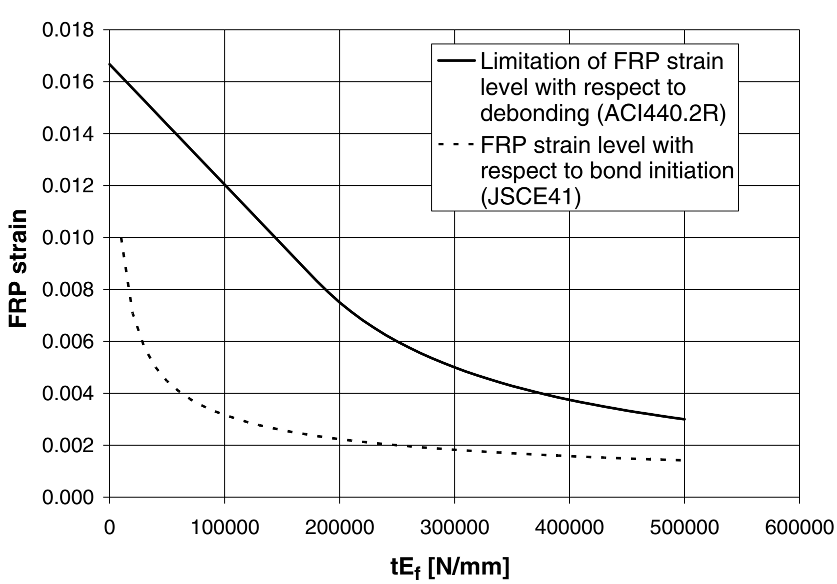

Fig. 5. FRP strain limitation.

3.4. Strengthening limitation

Another means of implicit guarantee of ductility (deformability) can be found in the so-called strengthening limitation, whereby a limit state is considered of the structure after strengthening, however with the FRP surface bonding no longer functional. If this limit is imposed, the structure will not collapse should the FRP EBR system fail. The load deformation behaviour of the concrete element relates in this case to that of a normal steel reinforced beam and will be characterized by yielding of the steel near ultimate.

This strengthening limitation is referred to in fib14 as accidental design situation. The verification is performed in ultimate limit state, considering the partial safety factors for the materials to be 1.0 and considering reduced partial safety coefficients and combination factors for the loads as provided in Eurocode 0 (EC0) [15]. If the accidental design situation is fulfilled, loss of the externally bonded reinforcement will not result in member collapse. Provided that the design takes into account all relevant situations which may involve accidental loss of the FRP strengthening (e.g. impact, vandalism, etc.), fib14 allows strengthening ratios higher than those imposed by the accidental design situation.

A strengthening limit is also recommended by ACI440.2R, stating that the existing strength of the structure should be sufficient to resist a level of load as described by

$$(\phi R_{\mathrm{n}})_{\mathrm{existing}} \geqslant (1.2 S_{\mathrm{DL}} + 0.85 S_{\mathrm{LL}})_{\mathrm{new}} \tag{8}$$

with ϕ the strength reduction factor, R_{n} the nominal strength, S_{DL} the dead load effects and S_{LL} the live load effects. In Eq. (8) reduced load factors 1.2 and 0.85 are specified, whereas for ultimate limit state design these factors equal 1.4 and 1.7, respectively [13].

No explicit strengthening limit is given in CSA-S806, however it is stated (CSA-S806, Section 11.2.3.1) that the engineering analysis shall consider the condition of the structure after strengthening, with the FRP surface bonding no longer functional.

Also in JSCE41 no explicit strengthening limit with respect to accidental loss of the FRP EBR is mentioned, though a performance category 'restorability' is incorporated in the design philosophy. The 'restorability' is such that the performance can be easily restored if damage is suffered during the service life. For example JSCE41, Section 6.4.7 gives collision safety requirements and Section 6.6.1 deals with restorability after earthquake.

4. Approaches regarding ductility in case of confinement

Confinement is generally applied to concrete columns or similar elements, either in terms of enhancing the behaviour under (axial) compression or in terms of improving the seismic behaviour. The increased performance follows from the higher compressive strength, the increased ultimate compressive strain and related ductility, the higher shear capacity, the increased buckling resistance of the internal steel in compression and the clamping effect on lap splices of longitudinal steel reinforcement.

4.1. Confined concrete ultimate axial strain

Both fib14 and ACI440.2R refer to the work by [16] to predict the stress–strain response of FRP wrapped concrete through an iteration approach. In JSCE41 the stress–strain response of FRP confined concrete is not discussed.

Furthermore, practical (approximate) equations for the ultimate strain of the confined concrete are given in fib14 and ACI440.2R. These formulae can be considered in a normalized way, assuming $\varepsilon_{\mathrm{co}} \approx 0.002$ the concrete strain at unconfined concrete strength f_{co}. Hereby, the normalised confined concrete strain $\varepsilon_{\mathrm{cc}}\varepsilon_{\mathrm{co}}$ basically depends on the normalised confining pressure $f_{\mathrm{l}}/f_{\mathrm{co}}$. In fib14 two equations are given making reference to [17] and [16] respectively. These equations can be written as

$$\varepsilon_{\mathrm{cc}}/\varepsilon_{\mathrm{co}} = 2 + \frac{2500(f_{\mathrm{l}}/f_{\mathrm{co}})\varepsilon_{\mathrm{fe}}}{f_{\mathrm{cc}}/f_{\mathrm{co}}}, \quad \text{with}$$

$$f_{\mathrm{cc}}/f_{\mathrm{co}} = 2.254\sqrt{1 + 7.94(f_{\mathrm{l}}/f_{\mathrm{co}})} - 2(f_{\mathrm{l}}/f_{\mathrm{co}}) - 1.254 \tag{9}$$

$$\varepsilon_{\mathrm{cc}}/\varepsilon_{\mathrm{co}} = 2 + 1.25\frac{E_{\mathrm{c}}}{f_{\mathrm{co}}}\varepsilon_{\mathrm{fe}}\sqrt{f_{\mathrm{l}}/f_{\mathrm{co}}} \tag{10}$$

with, $\varepsilon_{\mathrm{cc}}$ the ultimate axial strain of the FRP confined concrete, f_{l} the maximum lateral confining pressure by the FRP wrapping, f_{co} and f_{cc} the unconfined and confined concrete strength, $\varepsilon_{\mathrm{fe}}$ the maximum effective strain in the FRP wrapping reinforcement, and E_{c} the modulus of elasticity of concrete.

The equation in ACI440.2R gives the so-called maximum usable compressive strain and is independent of the strain capacity of the FRP. This equation is normalised as

$$\varepsilon_{\mathrm{cc}}/\varepsilon_{\mathrm{co}} = 855\frac{f_{\mathrm{co}}}{E_{\mathrm{c}}}(5(f_{\mathrm{cc}}/f_{\mathrm{co}}) - 4) \tag{11}$$

with $f_{\mathrm{cc}}/f_{\mathrm{co}}$ according to Eq. (9).

A comparison of Eqs. (9)–(11), with $f_{\mathrm{co}}/E_{\mathrm{c}}$ based on a concrete strength of 20 MPa (and E_{c} calculated following EC2 [12] for Eq. (10) and ACI318 [13] for Eq. (11)), is given in Fig. 6, for $\varepsilon_{\mathrm{fe}} = 0.005$ and 0.010 respectively. From this figure it appears that there is a considerable difference between the design models and that the prediction is strongly influenced (except for ACI440.2R) by the maximum effective strain that can be achieved in the FRP wrapping reinforcement.

4.2. Ductility with respect to seismic action

As also stated in ACI440.2R (Section 8.1), life safety is the primary performance objective of seismic designs with an allowance for some level of structural damage to provide energy dissipation. Generally, reference is made to a displacement demand (under load-carrying capacity) with respect to the yield displacement (ductility index $\mu_{\Delta} = \delta_{\mathrm{ultimate}}/\delta_{\mathrm{yield}}$).

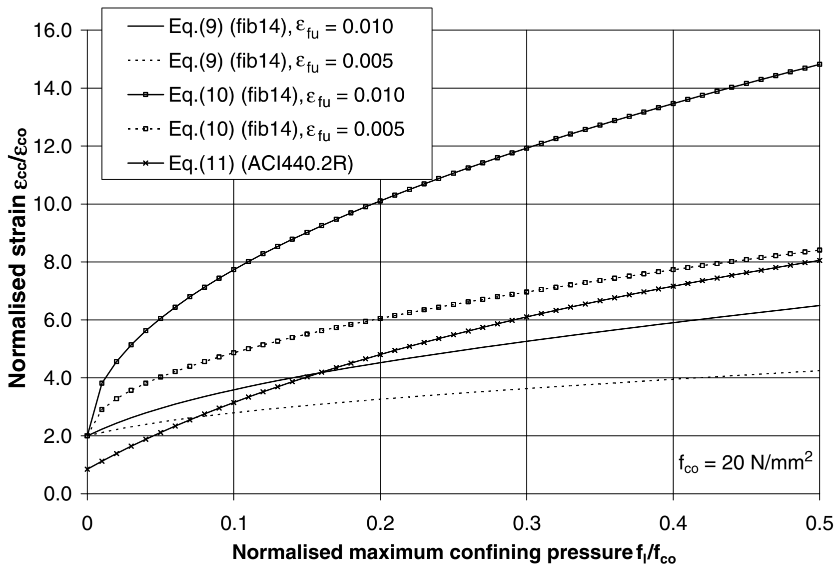

Fig. 6. Normalised ultimate strain of FRP confined concrete.

The approach by fib14 and ACI440.2R is basically the same, stating that the FRP EBR should be designed to provide a confining stress sufficient to develop concrete compression strains associated with the displacement demands, based on a load–displacement (or load–curvature) analysis and whereas the maximum required displacement (or curvature) may be obtained in terms of the ultimate concrete strain of the confined concrete (see previous section). As an alternative design and making reference to [18], fib14 gives an equation for the thickness of the wrapping reinforcement in case of a circular column, as a function of the ratio between target and available sectional displacement, the (steel-hoop) confined concrete strength and ultimate strain available before upgrading, the FRP properties and the column diameter. A similar equation is given in CSA-S806, whereby the thickness of the FRP wrapping is determined as a function of the design lateral drift ratio, the ratio of axial load over axial load resistance, the concrete strength, the FRP strength and the column diameter. An efficiency reduction factor is used for square or rectangular columns.

In JSCE41 a different approach is followed for the displacement ductility index of FRP upgraded members. Based on curve fitting of experimental results, the ductility index is given as a function of basically the shear capacity of the concrete and the internal steel, the maximum shear force when the member reaches the existing flexural load carrying capacity, and the amount and properties of the FRP.

5. Parametric study on ductility and strengthening limitation

5.1. Outline of the parametric study

A parametric study on using FRP EBR for strengthening of singly reinforced RC members subjected to bending has been executed in [11]. As an extension of this work, a small parametric study has been conducted with respect to the deformability at ultimate.

Based on a section analysis, the strength increase and the curvature at ultimate can be determined as a function of basically the amount of externally bonded FRP reinforcement. The latter is typically addressed by the FRP reinforcement ratio $\rho_f = A_f/bd$, with A_f the cross-sectional area of FRP, d the effective beam depth and b the width of the beam. Nevertheless, for a given amount of FRP the strengthening will also depend on the characteristics of the existing RC element (steel and concrete grade, steel reinforcement ratio ρ_s, rectangular or T-beam) and the type of FRP EBR, as demonstrated in [11].

In the study reported here, the strengthening ratio η_S and the ductility index μ_χ are investigated as a function of the FRP reinforcement ratio ρ_f, for a singly reinforced RC rectangular section strengthened with CFRP (carbon fibre reinforced polymer), following fib14 and assuming the material characteristics given in Table 2. The analysis is based on the following assumptions:

- The strengthening ratio $\eta_S = M_{RdS}/M_{RdU}$ follows from the resisting design moment of the strengthened versus the unstrengthened member. The calculation is performed in the ultimate limit state (ULS).
- The ductility index $\mu_\chi = \chi_{dS}/\chi_{yU}$ follows from the curvature at ultimate limit state of the strengthened member divided by the curvature of the unstrengthened member when reaching the characteristic yield strain $\varepsilon_{sy} = f_{yk}/E_s$ in the internal steel reinforcement. Herewith, f_{yk} is the characteristic value of the yield strength of the steel rebars and E_s is the modulus of elasticity.
- A first calculation assumes that bond failure is not occurring. The section analysis is based on strain compatibility, internal force equilibrium and moment equilibrium.

Table 2
Material characteristics used in the parametric study

Item	Property
Concrete grade C20	Characteristic cylinder strength = 20 MPa Modulus of elasticity = 28850 MPa Concrete failure strain = 0.0035
Steel grade S400	Characteristic yield strength = 400 MPa Modulus of elasticity = 200 GPa Steel rupture strain = 0.025 Steel reinforcement ratio = 0.5%
CFRP grade	Characteristic tensile strength = 2800 MPa Modulus of elasticity = 165 GPa

The ULS is either governed by yielding of the steel followed by fracture of the FRP reinforcement (YS/FR) or by yielding of the steel followed by concrete crushing (YS/CC). The calculation is stopped, for increasing ρ_f, when the value 0.001 is reached or if the internal steel is no longer yielding.

– A second calculation takes into account possible bond failure outside the anchorage zone, whereas it is assumed that proper reinforcement detailing is provided to avoid anchorage failure. The calculation is performed based on 3 approaches. In approach 1, a global FRP strain limit $\varepsilon_f \leqslant 0.0065$ to avoid bond failure is applied. In approach 2, the FRP strain limit given by ACI440.2R is applied, whereas the parameter tE_f in Eq. (5) can be rewritten as $\rho_f d(b/b_f)E_f$, with b_f the width of the FRP. In approach 3, bond failure due to peeling at vertical crack displacement or due to force transfer between FRP and concrete to avoid debonding at flexural cracks is considered according to fib14 sections 4.4.2.1 and A.3. These models, normalized with respect to $\mu_{Rd} = M_{Rd}/(bd^2 f_{cd})$, can be written as [11]

$$\mu_{Rd}^{debond1} = \left(0.38\ \mathrm{MPa} + 1.51\ \mathrm{MPa}\left(\rho_s + \frac{E_f}{E_s}\rho_f\right)\right)\frac{v}{f_{ck}},$$

$$\text{with } v = \frac{\ell}{k_v d} \tag{12}$$

$$\mu_{Rd}^{debond2} = 0.84 f_{ctm}\frac{v}{f_{ck}}\frac{b_f}{b} \tag{13}$$

where, f_{ctm} is the mean concrete tensile strength and ℓ/k_v equals the ratio of the maximum moment to the maximum shear force of the RC member (the latter to be taken within the zone where the FRP EBR is provided), with ℓ the span of the beam and k_v a constant which depends on the beam and load configuration.

– The initial strains in the RC member prior to strengthening are taken into account. This is done by introducing the ratio $\eta_0 = M_0/M_{serU} = \Phi_{rU}M_0/M_{RdU}$, where M_0 is the acting moment during strengthening (load safety factors equal to 1), M_{serU} is the service moment of the unstrengthened member under the rare load combination and $\Phi_{rU} = M_{RdU}/M_{serU}$ is the global load safety factor between the design load and the rare load combination. This global safety factor follows from the load safety factors for the dead and live load, as well as the ratio of dead versus live loads acting on the unstrengthened member. A value $\Phi_{rU} = 1.43$ can be assumed [11]. For the parametric study η_0 was set to 0.5.

– The ductility requirement of fib14, given by Eq. (2) is verified.

– The strengthening limitation by fib14 (see Section 3.4) is verified. This is done by introducing the ratio $\eta_a = M_{serS}/M_{Ad} = M_{RdS}/(\Phi_{rS}M_{Ad})$, where M_{serS} is the service moment of the strengthened beam under the rare load combination, M_{Ad} is the resisting moment corresponding to accidental loss of the FRP EBR (calculated in ULS, with material safety factors equal to 1) and $\Phi_{rS} = M_{RdS}/M_{serS}$ is the global load safety factor between the design load and the rare load combination. This global safety factor follows from the load safety factors for the dead and live load, as well as the ratio of dead versus live loads acting on the strengthened member. A value $\Phi_{rS} = 1.43$ can be assumed [11]. If $\eta_a \leqslant 1$ the accidental design situation is fulfilled.

5.2. Results for the case of no bond failure

The results of the parametric study, assuming the properties given in Table 2 and assuming that bond failure is not occurring, are given in Fig. 7 (solid lines). The ductility index of the unstrengthened member equals 6.58 and is not shown in Fig. 7. For ρ_f almost 0%, Fig. 7 shows a lower ductility index, as the calculation relates to the ultimate FRP strain which is lower than the steel strain at ultimate of the unstrengthened member.

Increasing strengthening ratio and decreasing ductility index are obtained with increasing amount of FRP. The governing failure mode is generally yielding of the steel followed by concrete crushing, while FRP fracture is only obtained for small FRP reinforcement ratios and hence low strengthening ratios. For the considered case, a strengthening ratio of 2 (doubling the design moment capacity) is achieved for $\rho_f \approx 0.2\%$, while the ductility index decreases at that point to about 30% with respect to that of the unstrengthened beam.

The ductility condition by fib14 is no longer fulfilled for the considered case when the ductility index drops below $\mu_\chi = 2.8$, which is reached for $\rho_f \geqslant 0.33\%$ (strengthening ratios above 2.2). Nevertheless, the steel will remain yielding at ULS, while concrete crushing without steel yielding is only obtained for extremely high amounts of FRP ($\rho_f \geqslant 1.7\%$). If the ductility condition is not fulfilled, fib14 states that the resisting design moment should be increased by 20%. This will generally involve a considerable amount of extra FRP reinforcement, as for such high FRP reinforcement ratios the FRP becomes less efficient in increasing the resisting design moment (Fig. 7).

The accidental design situation (strengthening limitation) is no longer fulfilled for a strengthening ratio above

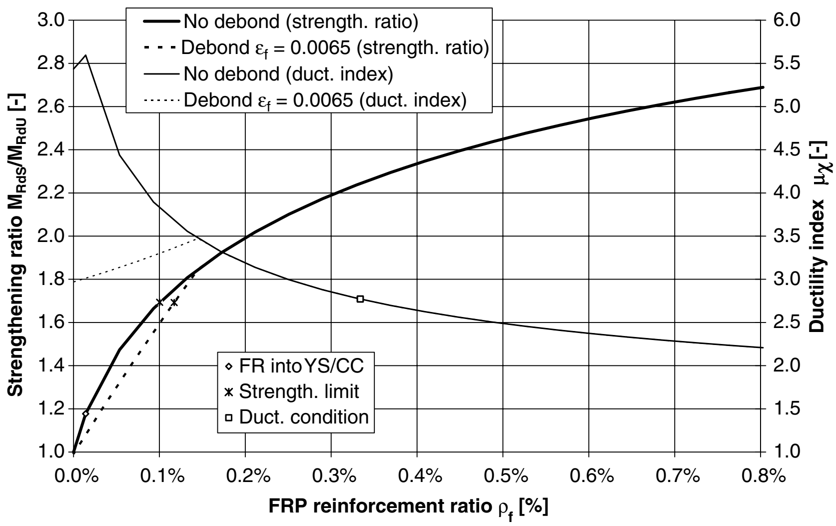

Fig. 7. Strengthening effect, debonding approach 1.

1.7 ($\rho_f \approx 0.1\%$), at which point the ductility index is about 40% higher compared to the ductility limit of fib14.

5.3. Results for the case of bond failure

The results of the parametric study, for the 3 debonding approaches are given in Figs. 7–9 (dotted lines). As can be noted from these figures, the strengthening ratio and ductility index are considerably influenced by bond failure. In the case of a constant strain limit (approach 1), bond failure is obtained at limited FRP reinforcement ratios (less than 0.15% in the considered case) and the decrease in ductility remains above the ductility condition. For high values of ρ_f the FRP strain at ultimate becomes lower than the strain limit and bond failure is no longer governing the design. This trend is however not confirmed by the more detailed approaches 2 and 3, where bond failure is especially critical for higher values of ρ_f. When reaching a certain value, no further strength enhancement is obtained with increasing the amount of FRP, while yet the ductility index further decreases.

From Figs. 8 and 9, it follows that the impact of debonding mechanisms increases especially when the bond width is reduced (high values of b/b_f) and for deep beams (high values of the effective depth d). In the most negative cases, a ductility index below 1 may be obtained, while in the case of no bond failure a ductility index below 1 will not occur unless considering unrealistically high amounts of FRP.

Considering large bond widths with respect to the beam width and assuming that the zone is not entered at which no further strength enhancement is obtained in the case of debonding, typically a minimum ductility curvature index of at least 1.5–2.0 is obtained (Figs. 8 and 9).

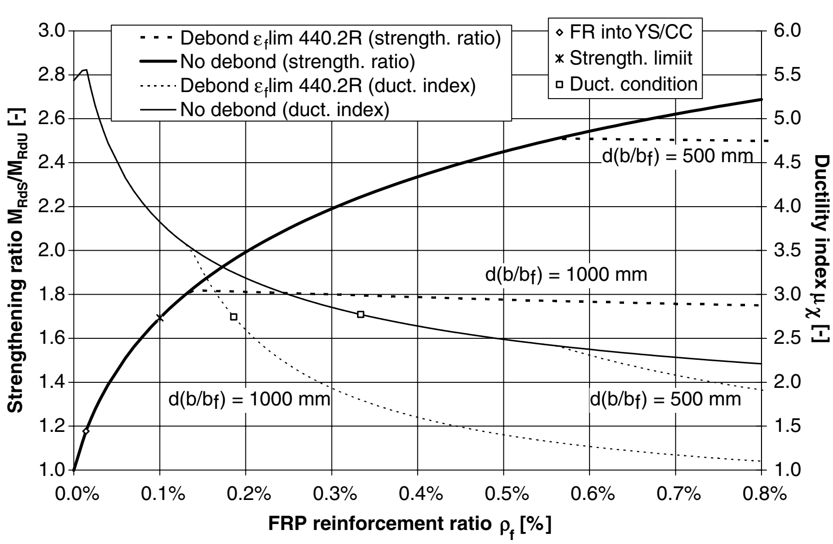

Fig. 8. Strengthening effect, debonding approach 2.

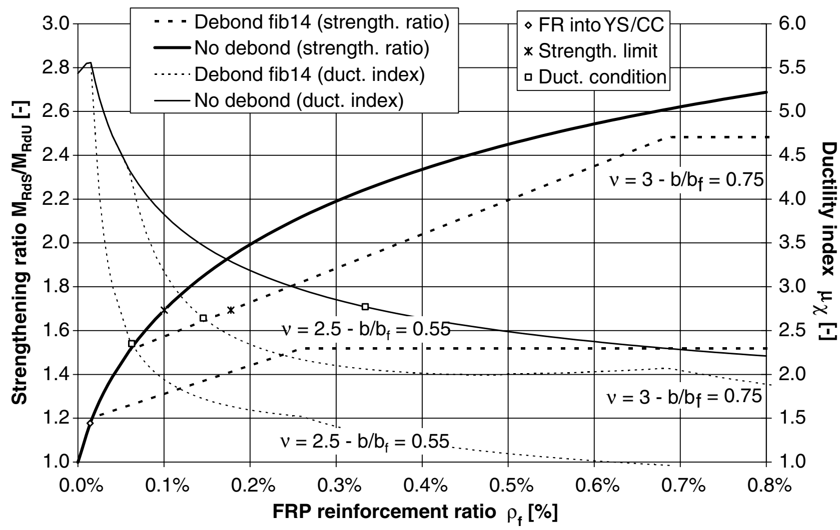

Fig. 9. Strengthening effect, debonding approach 3.

6. Conclusions

Based on an evaluation of ductility requirements in current design guidelines (fib14 [4], ACI440.2R [5], CSA-S806 [6] and JSCE41 [7]) and a small parametric study for flexural strengthening, the following conclusions can be drawn:

– The design guidelines do not consider energy dissipation requirements but address ductility typically by a strain level criterion.
– The ductility of flexural members strengthened with FRP, in terms of e.g. curvature at ultimate, may be considerably lower compared to unstrengthened members. With this respect fib14 and ACI440 give a target deformability, which also depends on the reserve of flexural strength (strength safety factor).
– The ductility of concrete members strengthened in bending with surface FRP will decrease with increasing amount of FRP. Considering how the governing failure modes influence the deformability at ultimate, a certain degree of ductility is implicitly guaranteed unless unrealistically high amounts of FRP are applied. Moreover, a strengthening limit approach is suggested in the considered codes, which also restricts the use of high amounts of FRP.
– Assuming that bond failure is not governing the design, or if so that efficiency is maximized by taking large bond widths, and assuming that in case of bond failure the zone is not entered at which no further strength enhancement is obtained, typically a minimum ductility curvature index of at least 1.5–2.0 is obtained (for the case considered in this parametric study).
– Ductility enhancement can be achieved by means of confinement of concrete through FRP wrapping. By providing a confining stress, increased concrete compression strains and hence increased deformability can be developed. Equations for the confined concrete ultimate axial strain given in fib14 and ACI440.2R yield large differences, by a factor up to at least 2, in predicting the confined concrete strain capacity.
– Ductility enhancement for seismic strengthening is addressed in the considered guidelines. The approach by fib14 and ACI440.2R is based on section analysis and confinement action, to associate increased concrete compression strains with displacement demands. Alternative equations (based on fitting of experimental data) are given in fib14, CSA-S806 and JSCE41.

From the conducted study it appears that the ductility of FRP strengthened flexural members should be considered with care, as reduced deformability is obtained at ultimate, though generally a minimum deformability (considering reinforcement strains larger than the yield strain as minimum criterium) is implicitly obtained in a proper design. At the other hand, ductility enhancement by means of FRP confinement is explicitly considered in the design guidelines.

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