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PROBLEMS RELEVANT TO POOR DUCTILITY PROPERTIES OF EUROPEAN REINFORCING STEEL

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SUMMARY

A significant percentage of European reinforcing steel, especially such with a bar diameter of less than 16 mm, exhibits poor ductility properties. The strain-hardening ratio R_m/R_p , i.e. the ratio of tensile strength R_m to yield strength R_p , and the elongation at maximum tensile force A_{gt} , are often very small. This leads to reinforced concrete structures which exhibit an insufficient ductility, i.e. an insufficient plastic deformation capacity, and hence an insufficient safety against structural failure.

A comprehensive research project on the ductility behaviour of slender RC structural walls, comprising static-cyclic tests, dynamic shaking table tests and pseudo-dynamic tests was carried out at the Swiss Federal Institute of Technology (ETH) Zurich. The tests and relevant theoretical investigations have shown that by poor ductility properties of reinforcing steel the length of the plastic region and the distribution of plastic deformations are adversely influenced. Furthermore, a tensile fracture of reinforcing bars often occurs, this even when only a small part of the reinforcing steel of a cross-section exhibits unfavourable ductility properties. Moreover, the buckling strain of compressed reinforcing bars is considerably reduced. Therefore, in this paper, minimum requirements on the reinforcing steel are formulated and relevant design recommendations are given. The minimum requirements and the design recommendations apply not only to cases of earthquake actions but also to cases of actions due to gravity loads, and they are also important to achieve a benign structural behaviour in cases of recurring extraordinary and not predictable actions such as overloading, impact, explosions and imposed deformations. Hence, as reinforcement of RC structures, principally and irrespective of the kind of action, only steel fulfilling the ductility requirements $R_m/R_p \geq 1.15$ and $A_{gt} \geq 6$ % should be used. Exceptions should only be allowed for domestic elements where, with high reliability, the reinforcement will never yield. Furthermore, the rules for design and detailing of RC structures should be made more dependent on the ductility properties of the reinforcing steel used.

INTRODUCTION

In Europe in recent years the technological development of reinforcing steel has been disastrous. On one hand, the development was characterised by the effort to obtain a higher and higher yield strength. On the other hand, under the governing pressure of costs – also caused by the import of cheap steel from Eastern Europe after the failure of the Iron Curtain 1990 – more and more procedures leading to poor ductility properties were and are still being used. This negative development means that today many RC structures have to be built which, seen from the viewpoint of structural safety, exhibit an insufficient ductility, i.e. an insufficient plastic deformation capacity. This observation is true both, for structures with predominant gravity loads (unidirectional loading) and structures with predominant earthquake actions (cyclic loading). The unfortunate development took place under the very eyes of the relevant code committees. Unfortunately, the committees did not recognise the problems in good time, or in any case they did not intervene. Therefore, the statement is justified that in the field of reinforcing steel and hence of RC structures there is a real problem with ductility.

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In the following the correlation between important mechanical characteristics, above all the ductility properties, of reinforcing steel and the ductility properties of RC structural elements is treated. For this purpose both results of experimental research and theoretical investigations are used. Finally, minimum requirements for reinforcing steel in RC structures are postulated and important conclusions are drawn.

DUCTILITY PROPERTIES OF REINFORCING STEEL AND RELEVANT PHENOMENA IN RC STRUCTURAL ELEMENTS

In view to the plastic deformation capacity of RC structures in particular the following characteristics of the reinforcing steel are important:

- Strain-hardening ratio R_m/R_p, i.e. ratio of tensile strength R_m to yield strength R_p (according to [ISO, 1998])
- Elongation at maximum tensile force A_{gt}

Figure 1 shows typical stress-strain diagrams of reinforcing steel from Europe (left) and from other regions such as New Zealand, USA and Japan (right). European reinforcing steel with bar diameters d < 16 mm is mostly hot- or cold rolled in coils and is straightened again when manufactured for the site. Due to this process the steel no longer exhibits a yield plateau and the yield strength, defined by the stress at 0.2 % permanent strain, becomes significantly higher as in the case of steel with the same or similar chemical composition but different and more traditional procedures of fabrication and manufacturing.

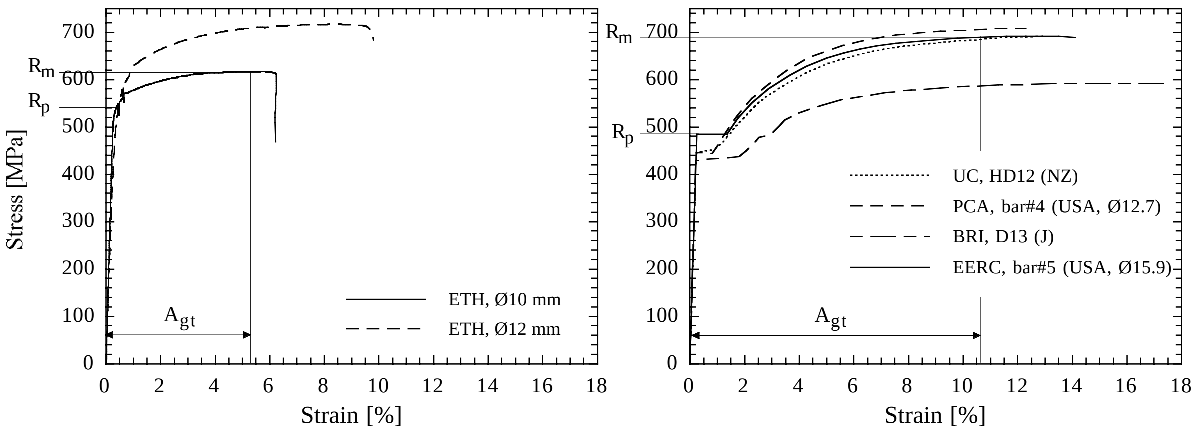


Figure 1: Typical stress-strain diagrams of reinforcing steel from Europe (left) and other regions (right)

Relevant European reinforcing steel (E) exhibits the following significant differences compared to reinforcing steel from other regions (0):

- Smaller strain-hardening ratio R_m/R_p
- Smaller elongation at maximum tensile force A_{gt}
- Higher yield strength R_p

Typical values are:

- R_m/R_p = 1.02 – 1.15 (E) and 1.15 – 1.30 (0)
- A_{gt} = 2 – 6 % (E) and 6 – 10 % (0)
- R_p = 500 – 600 MPa (E) and 420 – 500 MPa (0)

As a consequence of poor ductility properties of reinforcing steel the ductility of RC structural elements is affected by the following phenomena:

a) smaller length of the plastic region and unfavourable distribution of plastic deformations
b) earlier tensile fracture of reinforcing bars
c) earlier buckling of reinforcing bars

All three phenomena individually or in combination lead to a significant reduction of the plastic deformation capacity. For the engineer it mostly appears as a reduction of the curvature or the rotational ductility. The phenomena exhibit the following correlation to the individual steel properties:

- The phenomenon a) is primarily caused by the smaller strain-hardening ratio R_m/R_p.
- The phenomenon b) follows from both the smaller strain-hardening ratio R_m/R_p and the smaller elongation at maximum tensile force A_{gt}.
- The phenomenon c) follows mainly from the smaller strain-hardening ratio R_m/R_p (small tangent modulus).

Between the individual phenomena there are significant interactions:

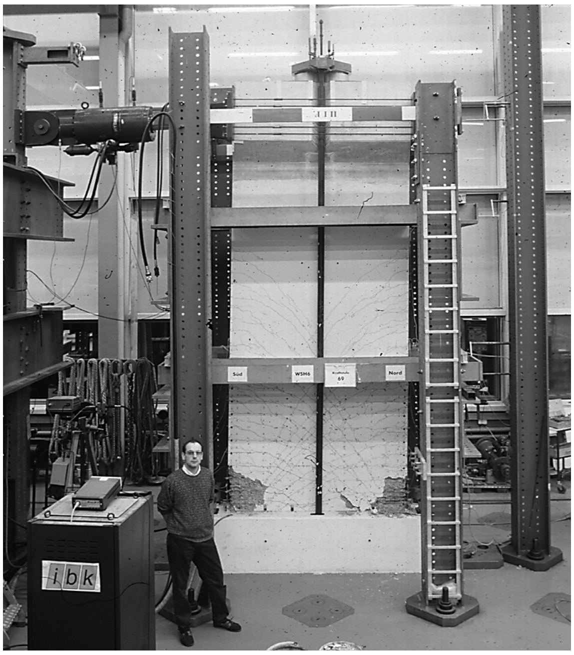
- The phenomenon a) contributes to the phenomenon b)
- The phenomenon b) can be accelerated by the phenomenon c)

The three phenomena are discussed in Sections 4 to 6. The discussion will mainly be based on the results of the research work described in the following Section 3.

WALL TESTS AT THE ETH ZURICH

At the Institute of Structural Engineering (IBK) of the Swiss Federal Institute of Technology (ETH) in the years 1996 to 1999 the following experimental investigations have been carried out:

- Static-cyclic tests on six 6-storey RC walls in scale 1:2 [Dazio et al., 1999]
- Dynamic tests on six 3-storey RC walls in scale 1:3 by means of the ETH earthquake simulator [Lestuzzi et al., 1999]
- Pseudo-dynamic tests on three 3-storey RC walls in scale 1:3 [Thiele et al., 1999]



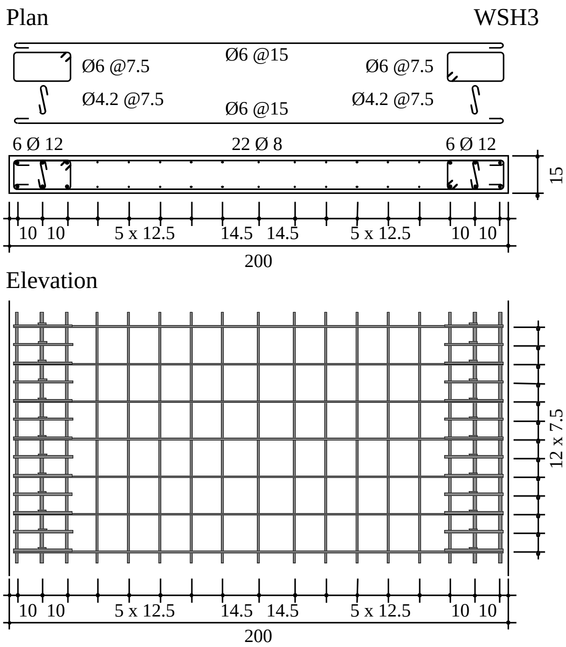


Figure 2: Test set-up of the static-cyclic tests on RC structural walls (left) and typical reinforcement of the plastic wall region (right)

Important test parameters were – among others – the ductility properties of the reinforcing steel used. Details can be taken from the cited test reports and from the web site of the IBK (http://www.ibk.baum.ethz.ch).

Figure 2 left shows the overall test set-up of the static-cyclic tests. The footing of the test wall is fully fixed by pretensioned bars to the very stiff strong floor. The test wall corresponds to the lower part of a slender structural wall of a 6-storey reference building. The test wall is about 5.5 m high and has a rectangular cross-section of size 2 m x 0.15 m. Figure 2 right shows a typical reinforcement of the plastic region at the base of the wall. The axial force due to gravity loads is simulated by two external prestressing tendons. The horizontal cyclic earthquake force at top of the test wall is induced by a 1000 kN hydraulic actuator braced to a stiff reaction frame. In general, the walls were designed and detailed according to the rules of the capacity design method for a displacement ductility μ_Δ of 3 to 5.

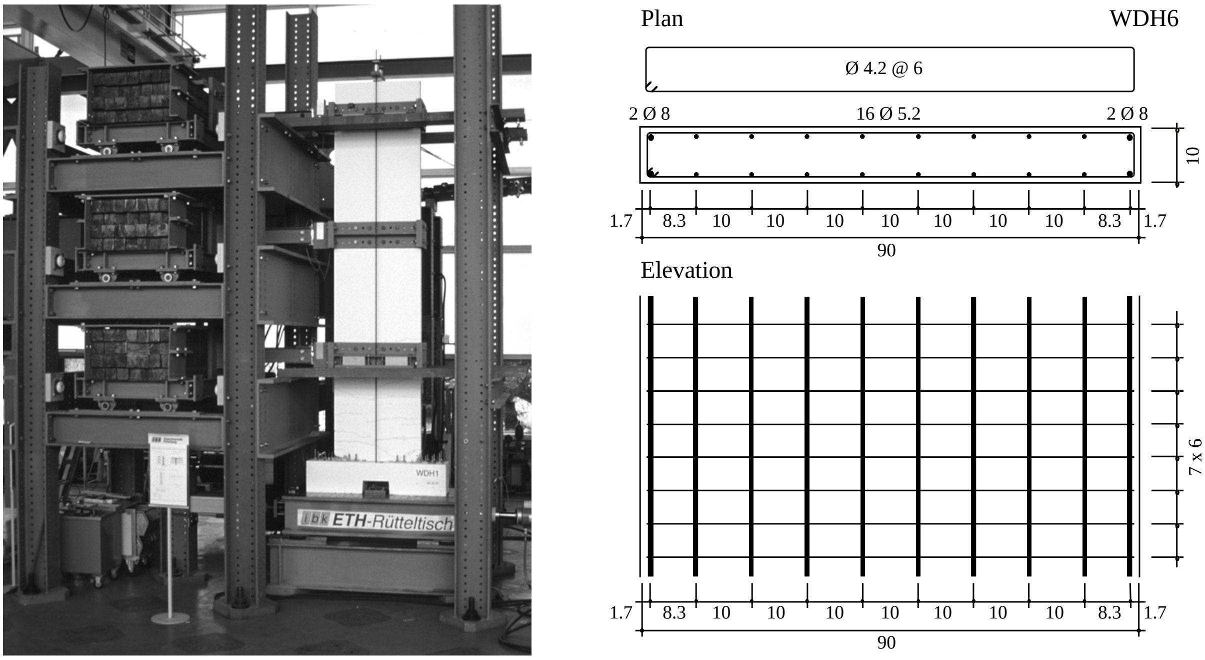


Figure 3: Test set-up of the dynamic tests on RC structural walls on the ETH earthquake simulator (left) and typical reinforcement of the plastic wall region (right)

Figure 3 left shows the overall test set-up of the dynamic tests. The footing of the wall is fully fixed by pretensioned bars to the horizontally and one-dimensionally moving table of the ETH earthquake simulator. The slide-bearings of the table are capable to transfer large vertical tension and compression forces allowing the action of big bending moments of the test specimen to the supporting structure. The test wall corresponds to a slender structural wall of a 3-storey reference building. The test wall is about 4.5 m high and has a rectangular cross-section of size 1.0 (0.9) m x 0.1 m. Figure 3 right shows a typical reinforcement of the plastic region at the base of the wall. The axial force due to gravity load is simulated by two external prestressing bars. The wall is connected by horizontal pinned struts with 3 storey masses of 12 tons each. The masses are placed on carriages rolling on rails on an adjacent structure. Therefore, the chosen test set-up is capable to reproduce a realistic ratio between horizontal inertia forces and vertical gravity loads. The tributary area of the latter is much smaller, due to the presence of columns carrying only gravity loads. For both the design of the reference building and the planning of the test set-up the frame action of the very flexible flat slabs and small columns is neglected and the RC structural walls are designed for the total earthquake forces. In general the walls were designed and detailed according to the rules of the capacity design method for a displacement ductility μ_Δ~3.

LENGTH OF THE PLASTIC REGION AND DISTRIBUTION OF THE PLASTIC DEFORMATIONS

Figure 4 shows typical results of the dynamic tests. Schematically drawn are the plastic region and the footing of walls WDH1 and WDH4. In order to consider pull-out effects of the vertical reinforcement the wall region was fictitiously lengthened by 60 mm into the footing. In the figure the maximum curvatures are shown. The curvatures were determined by measuring the vertical deformations at the left and right wall edges and averaging them over the relevant gauge length. On the left hand side the curvature in the case of tension at the left edge and on the right hand side the curvature in the case of tension at the right edge are presented. As an important reference value also the ratio of the actual curvature to the yield curvature is shown. For simplicity and in general this ratio is called "curvature ductility" even if no failure limit state (deformation capacity) is meant. Furthermore, in Figure 4 the length of the effective plastic region is indicated. This region can be defined as the zone where during the test at both the left and the right wall edges the yield curvature is exceeded. In wall WDH1 the vertical reinforcing bars in both the "end regions" of the cross section (having a length of ~ 200 mm on both sides) and the "web regions" (between the end regions) exhibit a strain-hardening ratio R_m/R_p of only 1.10. The elongations at maximum tensile force A_{gt} were 4.9 % and 3.9 %, respectively.

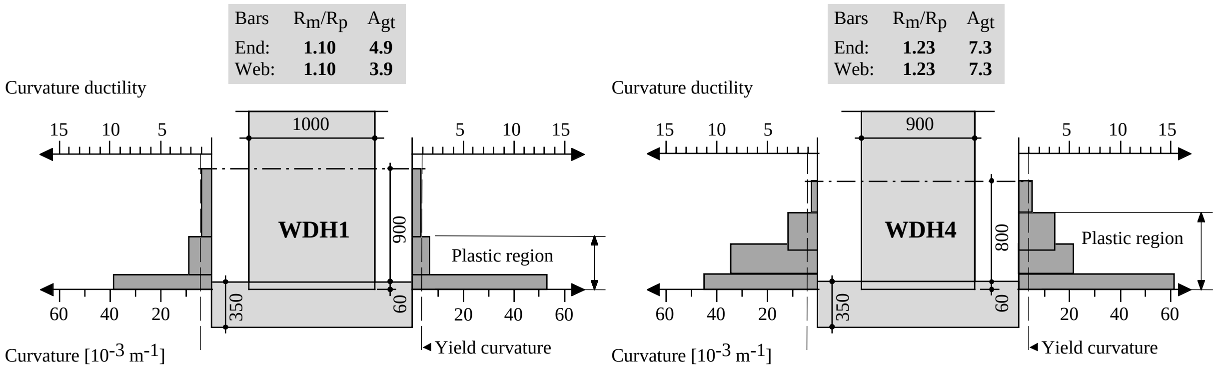


Figure 4: Maximum curvatures for poor (left) and better (right) ductility properties of the reinforcement

Figure 4 left shows the maximum curvatures of wall WDH1 before failure by fracture of vertical reinforcing bars. As a consequence of the poor ductility properties of the reinforcing bars the length of the plastic region was relatively small, amounting to only 0.36 l_w (wall length l_w = 1.0 m). The curvature ductility in the medium gauge region was only 2.2 (tension at left) and 1.7 (tension at right), respectively. The plastic deformations show a very unfavourable distribution, being concentrated in the lower gauge region, i.e. on the crack directly over the footing. The rotational angle (integration of curvatures) of the effective plastic region amounted to $7.2 \cdot 10^{-3}$ radians for tension at left and to $8.3 \cdot 10^{-3}$ radians for tension at right.

In wall WDH4 in the whole cross-section the same vertical reinforcing bars were used. They had a strain-hardening ratio R_m/R_p of 1.23 and an elongation at maximum tensile force A_{gt} of 7.3 %. Figure 4 right shows the maximum curvatures of a test (no fracture of reinforcement). In accordance with the significantly better reinforcement ductility properties compared to wall WDH1 the length of the plastic region was 0.61 l_w (wall length l_w = 0.9 m), i.e. 70 % larger. Also the distribution of the plastic deformations was considerably better than in WDH1. The rotational angle of the effective plastic region amounted to $16.6 \cdot 10^{-3}$ radians for tension at left and to $16.1 \cdot 10^{-3}$ radians for tension at right. Hence the angle was about two times greater than in the case of wall WDH1, this without leading to a fracture of reinforcement.

Similar conclusions can be drawn from the results of the static-cyclic tests. Even if only a part of the vertical reinforcement (e.g. that in the web region) has poorer ductility properties than the remaining reinforcement (e.g. that in the end regions), a smaller length of the effective plastic region and a more unfavourable distribution of the plastic deformations is produced. Accordingly, the rotational angle and hence the plastic deformation capacity is much smaller.

TENSILE FRACTURE OF REINFORCING BARS

Figure 5 shows the lower right wall region above the footing of the static-cyclic tested wall WSH1 after failure. Inset in the photo the reinforcement is drawn. The vertical reinforcement bars in the web region (totally 24 bars) had a diameter d = 6 mm and a horizontal spacing s = 125 mm and the very poor ductility properties strain-hardening ratio R_m/R_p = 1.03 and elongation at maximum tensile force A_{gt} = 1.8 %, and in the end regions (6 bars each) d = 10 mm, s = 75 mm, R_m/R_p = 1.13 and A_{gt} = 4.5 %.

Figure 6 left shows the hysteresis loops of wall WSH1. The horizontal displacement and the corresponding actuator force of the cyclic action with increasing plastification are displayed. As an important reference value also the ratio of the actual displacement to the yield displacement is shown. For simplicity and in general this ratio is called “displacement ductility” μ_Δ even if no failure limit state (deformation capacity) is meant.

The failure of wall WSH1 was initiated, after passing through two cycles of μ_Δ = 2 and only one cycle of μ_Δ = 3, by a tensile fracture of vertical reinforcing bars in the web region (x in Figure 5 and in Figure 6 left). This reduced the flexural strength in the cross-section with the fractured bars. Subsequently this cross-section was weaker than all other cross-sections above and below this one. As a consequence in the next cycle the plastic tensile deformations were concentrated in the weakened cross-section, where also the remaining vertical bars in the web region as well as those in the end regions fractured. Therefore, it can be stated: The ductility behaviour of an inflected RC structural element reinforced with bars exhibiting different ductility properties is mainly governed by the bars with the poorer ductility properties.

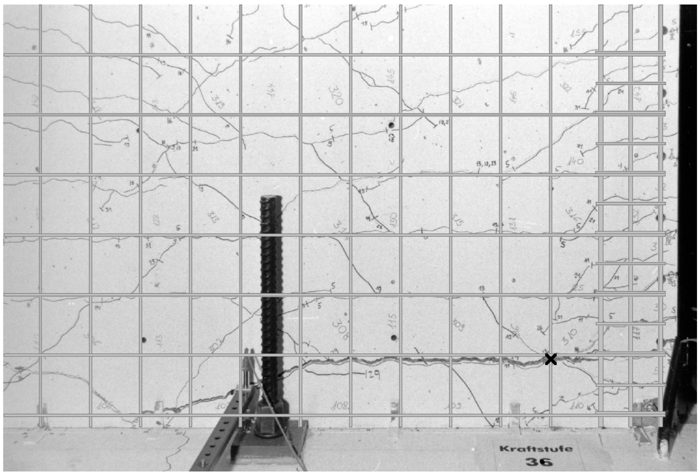


Figure 5: Failure by tensile fracture of reinforcing bars in the web region (x) followed by tensile fracture of bars in the end region (wall WSH1)

Compared to wall WSH1 wall WSH3 exhibited a much better ductility behaviour. The vertical reinforcement bars in the web region (totally 22 bars) had d = 8 mm, s = 125 mm, R_m/R_p = 1.23, A_{gt} = 6.5 %, and in the end regions (6 bars each) d = 12 mm, s = 100 mm, R_m/R_p = 1.21, A_{gt} = 6.8 %. Due to the better ductility properties of the reinforcing steel two full cycles of each displacement ductility up to μ_Δ = 6 could be performed. The hysteresis loops of Figure 6 right are very stable and exhibit only a relatively small pinching. A comparison of the dissipated energy (area within the loops of the walls WSH3 and WSH1) results in a factor of 7.

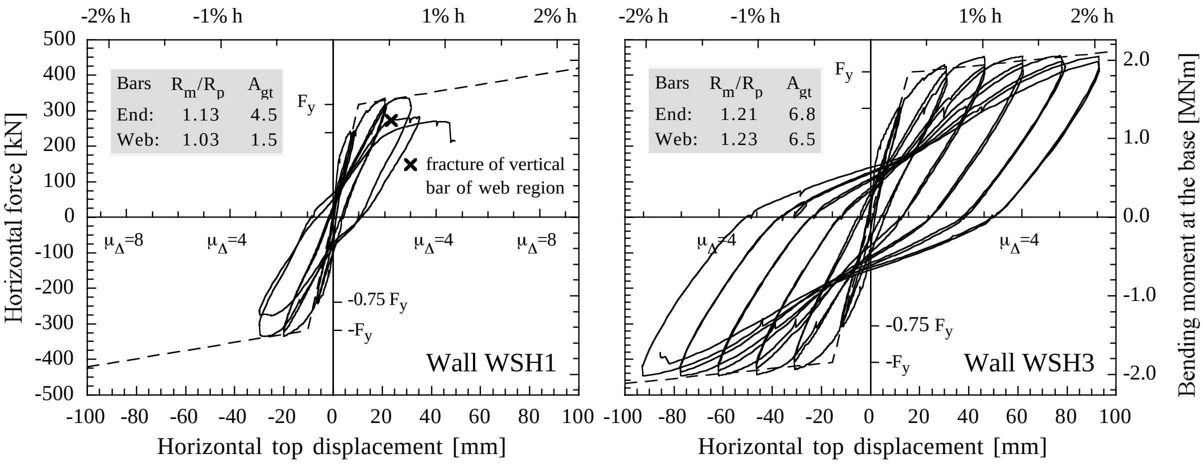


Figure 6:Hysteresis loops for poor (left) and better (right) ductility properties of the reinforcement

BUCKLING OF REINFORCING BARS

Figure 7 left shows buckled bars in the end region of the static-cyclic tested wall WSH2. Previously the concrete cover of the reinforcement had spalled off. In the tensile half-cycle after buckling the two corner bars fractured, which can be seen in Figure 7 right. The bars fractured exactly at the point where the preceding compression force caused the largest curvature. All 6 bars of the end regions were stabilised by horizontal ties, like in Figure 2 right, with a vertical spacing of s=75 mm ("stabilising reinforcement"). Hence in this case the ratio s/d, in the following called as "tie spacing ratio", amounted 75 mm / 10 mm = 7.5. Eurocode 8 [1997] recommends a value of 9 for Ductility Class "Medium" ($\mu_\Delta \sim 3$) and a value of 5 for Ductility Class "High" ($\mu_\Delta \sim 5$). Both the buckling stress and strain of round house type reinforcing steel bars (Figure 1 left) mainly depends from:

- strain-hardening R_m/R_p
- tie spacing ratio s/d.

This should be also the case for sharp knee type reinforcing steel bars, but neither experimental nor theoretical investigations supporting this statement were performed at the ETH.

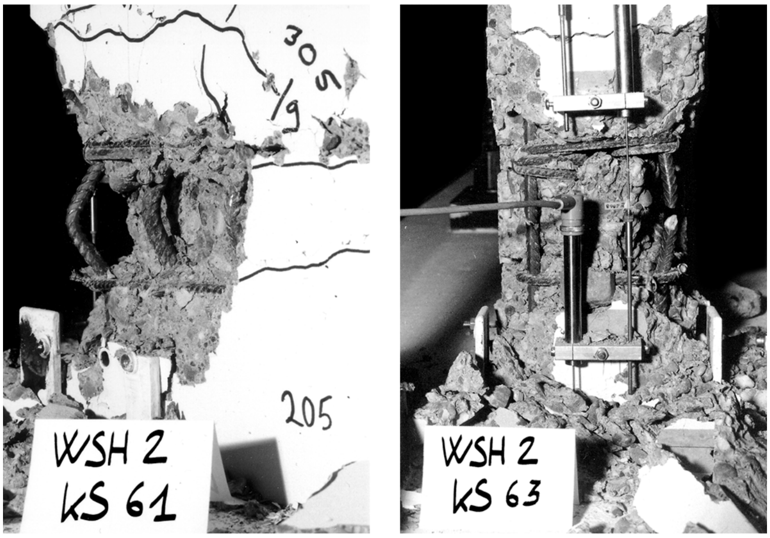


Figure 7: Reinforcing bars which buckled (left) and fractured in the next tension half-cycle (right)

If centric buckling is considered and it is assumed that the bar is only stabilised by the ties (concrete action neglected) and fully restrained at a distance s (no rotational effect), a theoretical buckling stress and the relevant buckling strain can be determined according to the classic buckling theory, by the help of the definitions given in Figure 8 left:

$$\sigma_k = \frac{\pi^2 T_k}{\lambda^2}$$

l_k:	Buckling length, $l_k = s/2$		
i:	Radius of inertia, $i = d/4$		
λ:	Slenderness, $\lambda = l_k / i = 2\ (s/d)$		
T_k:	Buckling modulus according to:	Engesser-Shanley $T_k = T$	
		Engesser-Karman $T < T_k < E$	

Figure 8 right shows the calculated buckling strain curves as a function of the spacing ratio s/d. The calculations were performed for unidirectional loading and for a material law according to the upper stress-strain relationship shown in Figure 1 left, which was approximated by a Ramberg-Osgood function [Bachmann et al., 1998]. The upper curve (upper bound) of Figure 8 right was calculated with the help of the modulus according to Engesser-Karman and the lower curve (lower bound) with the tangent modulus according to Engesser-Shanley [Petersen, 1980]. The curves show that it is a question of buckling of squat bars which exhibit a compression strain greater than the yield strain (~ 0.28 %). Therefore, a relatively small inclination of the stress-strain relationship in the strain-hardening region has unfavourable effects.

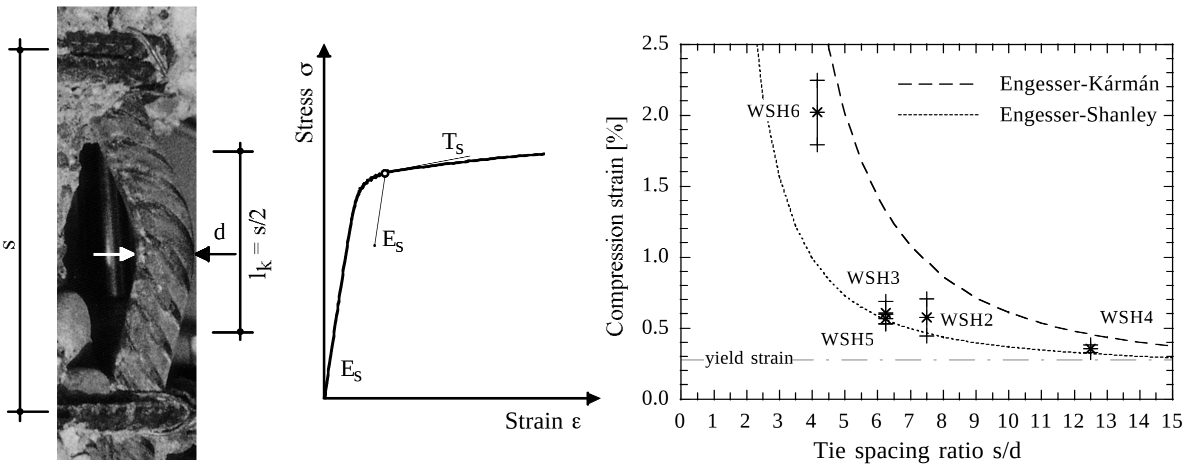


Figure 8: Definitions related to buckling of reinforcing bars (left) and buckling strain as a function of the tie spacing ratio (right)

In Figure 8 right the strain (mean value and standard deviation) measured at the 2x2 corner bars of the static-cyclic test walls WSH2 to WSH6, before obvious buckling occurred, is also shown. The strain was measured over a gauge length of 150 mm with help of bolts glued to the reinforcing bar. All test values lie between the two theoretical buckling curves. And it can be seen, for example, that in wall WSH6 by the small tie spacing ratio s/d = 4.2 a considerably larger buckling strain than in the other walls exhibiting higher values s/d could be reached. Thus the unfavourable influence of the small strain-hardening ratio R_m/R_p of the reinforcing steel used, has to be compensated by a smaller tie spacing ratio. That was also indicated by [Priestley et al., 1996].

DUCTILITY OF STRUCTURES WITH GRAVITY LOADS AND EXTRAORDINARY ACTIONS

The poor ductility properties of European reinforcing steel are not only a problem of structures under seismic actions. Such properties also reduce the safety of the great number of structural elements designed for gravity loads only or when they are stressed by extraordinary and unforeseen actions such as overloading impact, explosions and imposed deformations. Whereas seismic actions cause a cyclic loading, the other mentioned actions cause a unidirectional loading. In such cases the external and internal forces and the deformations always have the same direction, and the relevant values are always increasing.

Of interest is the comparison of local stresses and deformations of structural elements for unidirectional and cyclic actions demanding the same structural ductility (same curvature or rotational or displacement ductility). In the case of cyclic loading the deterioration of the bond between reinforcement bars and concrete in the vicinity of a cracked section, due to many relative displacements to and fro, is higher than in the case of unidirectional loading. Furthermore, in many cases smaller cycles precede the ones with maximum deformation. These effects reduce the maximum value of the plastic steel strain in the cracked section and hence the danger of an early tensile fracture of the reinforcement bars. If there is the same ductility demand of structural elements for unidirectional as for cyclic loading, the ductility properties of the reinforcing steel should principally be better and at least the same as for cyclic loading.

A frequent case where a ductile behaviour for gravity load actions is assumed is the design of RC continuous beams for dead and live load. Thereby, for the ultimate limit state design often principles of the theory of plasticity are used by redistribution of moments calculated by the theory of elasticity. For the design of the flexural reinforcement, for example, 20 to 30 % of the elastic moments at the supports are redistributed to the spans, or vice versa. However, such a redistribution is physically only possible if the plastifying regions are sufficiently ductile. Comparative investigations [Bachmann & Wenk, 1998] have shown that the rotational ductility demand of plastic regions in continuous beams for moment redistributions of 20 to 30 % is of the same order as the rotational ductility demand of slender RC walls under seismic actions exhibiting a displacement ductility of $\mu_\Delta \sim 3$ (medium ductility). Therefore, considering the above mentioned different bond behaviour, reinforcing steel in continuous beams should exhibit better but at least the same ductility properties as those in structural walls.

Further, more and more it happens that RC structures are stressed by extraordinary and unforeseen actions. Such actions may be for example considerable overloading, impact of vehicles, explosions, or deformations imposed by settlement or scouring of footings. In all these cases and many more a prerequisite of a benign behaviour of the structure avoiding failure is a sufficient ductility. Therefore, for most RC structures reinforcing steel fulfilling certain minimum conditions for ductility properties is of great importance and “important for survival”.

MINIMUM DUCTILITY REQUIREMENTS OF REINFORCING STEEL AND RELEVANT DESIGN RECOMMENDATIONS

Based on the explanations and statements above, the following minimum requirements for reinforcing steel and relevant design recommendations can be postulated:

1) For the seismic design of RC structures for medium ductility (displacement ductility $\mu_\Delta \sim 3$) the reinforcing steel should fulfil the following ductility requirements:

   - Strain-hardening ratio $R_m/R_p \geq 1.15$
   - Elongation at maximum tensile force $A_{gt} \geq 6$ %

2) For the design of RC structures for gravity loads and to achieve a benign behaviour also in the case of frequently occurring extraordinary and unforeseen actions such as overloading, impact, explosions, imposed deformations, etc., the reinforcing steel should fulfil at least the ductility requirements as described in 1).

3) For the seismic design of RC structures or in the case of a design for other actions for high ductility (displacement ductility $\mu_\Delta \sim 5$) the reinforcing steel should fulfil the following ductility requirements:

- Strain-hardening ratio $R_m/R_p \geq 1.25$
- Elongation at maximum tensile force $A_{gt} \geq 6$ %

4) The maximum spacing of ties to avoid buckling of longitudinal reinforcing bars under compression ("Stabilising reinforcement") has to be made dependent from the ductility properties of the reinforcing steel used. For $R_m/R_p \sim 1.15$ and $A_{gt} \sim 6$ %, a tie spacing ratio of ~ 4 to 5 seems to be appropriate

CONCLUSIONS

From the contents of this contribution the following conclusions can be drawn:

a) As a reinforcement of RC structures, basically and irrespective of the kind of action, only steel should be used which fulfils the following minimum conditions:

- Strain-hardening ratio $R_m/R_p \geq 1.15$
- Elongation at maximum tensile force $A_{gt} \geq 6$ %

Exceptions should only be allowed for domestic elements where, with high reliability, the reinforcement will never yield. In the case of a design for higher ductility the reinforcing steel should fulfil higher ductility conditions.

b) The rules for the design and detailing of RC structures should be made more dependable on the ductility properties strain-hardening ratio R_m/R_p and elongation at tensile force A_{gt} of the reinforcing steel used. This is true especially for Eurocode 8 and the pertinent clauses to avoid buckling of reinforcing bars under compressive forces.

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