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2019

Tan, T. H., Nguyen, N. B., Li, B., Chew, K. C., & Thung, S. K. (2019). Design considerations and guidelines in the use of Grade 600 steel reinforcements for reinforced concrete construction. IOP Conference Series: Materials Science and Engineering, 603(5), 052037-. doi:10.1088/1757-899x/603/5/052037

<https://hdl.handle.net/10356/142594>

<https://doi.org/10.1088/1757-899X/603/5/052037>

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# Design Considerations and Guidelines in The Use of Grade 600 Steel Reinforcements for Reinforced Concrete Construction

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**Abstract.** There is growing interest within the construction industry in using Grade 600 reinforcing steel for certain applications. There are areas where Grade 600 steel reinforcement can help include improvement in construction efficiencies and reduction in manpower. However, the implementation of these Grade 600 reinforcements in designs and construction still pose some challenges, as a number of unresolved design issues still remain. Notably current EC 2 design provisions do not provide explicit guidelines on the use of these high strength steel reinforcements and may not have addressed some distinctive performance characteristics. This paper lists potential issues with the use of Grade 600 reinforcement, and also provides a comprehensive assessment of available literature on the impact of Grade 600 steel reinforcement on design practices.

## 1. Introduction

Over recent years, design codes have gradually increased their limits on the maximum allowable yield stress of longitudinal reinforcing bars because of advances in metallurgy leading to production of tougher reinforcing bars. Steel reinforcement with yield strength of 600 MPa, which have a linear pre-yield behaviour, a well-defined yield plateau and relatively high ductility, is commercially available. Its use is expected to be more wide-spread in the near future. There are many potential benefits from the use of high-strength steel reinforcement in reinforced concrete construction in Singapore including cost reductions in the form of reduced material and manpower requirements as well as shorted construction time. Hence, the need for more guidance for the use of this increasingly popular material and this has culminated in a new design guide.

The current SS EN 1992-1-1 (EC2) [1] code provisions for reinforced concrete (RC) structures limit the nominal yield stress ( $f_y$ ) of longitudinal steel reinforcement to 600 MPa. However, EC2 provides the principles and application rules on design but does not prescribe guidance or procedures to assist the designers. Unlike the common grade of steel of 500 MPa and below, there are also insufficient verifications of the design using the higher grade steel. The lack of information regarding the behaviour



of concrete members reinforced with high-strength steel reinforcement hinder design engineers from using the full strength of the material.

In an attempt to address some of these design challenges, this paper identifies the current knowledge on issues related to design using high strength Grade 600 steel reinforcements, and presents the guidance on how issues such as flexural, shear, moment redistributions and compression would be affected by the use of high yield strength steel reinforcement. Research on performance and design issues associated with the use of high yield strength steel reinforcements, design and construction issues reported in other countries are also reviewed to support the preparation of the needed design considerations and guidance to EC2 [1].

## 2. Summary of key issues

When used in reinforcement, Grade 600 steel has the potential to impact design provisions and performance in reinforced concrete construction.

### Section under flexure

The strength of reinforced concrete members under flexural, axial, or combined flexural and axial loading is a key consideration as provisions for these considerations would directly establish the size of the members. Understanding potential changes in the strength and behaviour of reinforced concrete members is necessary to ascertain that Grade 600 reinforcement is safe and economical to use in practice. A number of research programs have been carried out to evaluate the performance of concrete members reinforced with high yield strength steel reinforcements:

Yotakhong [2] demonstrated through the experimental testing of large-scale beams with different reinforcement ratios of high-strength steel (850 MPa) that all test beams exhibited a ductile behaviour with significantly strained steel when the crushing strain of the concrete was reached. Giduquio *et al* [3] evaluates the performance of reinforced concrete (RC) flexural members reinforced with two different types of high strength steels—Grade 100 (690 MPa) and SD685 (690 MPa) under monotonic loading. Flexural responses of RC beam specimens using conventional Grade 60 (414 MPa), SD685, and Grade 100 steels as flexural reinforcement were experimentally studied. Five pairs of RC beam specimens were tested under a monotonically increasing gravity-type loading. Flexural strength of the specimens reinforced with high-strength flexural reinforcement can be satisfactorily predicted using bilinear steel stress-strain relationships and equivalent concrete stress block per ACI 318-14 [4], provided that steel yield stress is selected appropriately. Seliem *et al* [5] also tested three full-scale bridge decks with a span-depth ratio of 12.5. The first and second decks were constructed with the same reinforcement ratio using high yield strength rebars (689 MPa) and Grade 60 (469 MPa) rebars, respectively. The third deck was reinforced with high yield strength rebars using 33% less reinforcement in an attempt to use its high strength. The test results indicated that the decks reinforced with 33% less high-strength steel than those reinforced with Grade 60 steel developed the same ultimate load carrying capacity and deflection at service load.

More recently, Shahrooz *et al* [6] conducted analytical and experimental research program to examine the behavior and design of flexural members with high-strength steel reinforcements. They tested six RC beams with various amounts of high-strength Grade 689 reinforcing steel indicated satisfactory performance in terms of beam capacity and ductility. The flexural resistance of such members with high strength steel reinforcements could safely and conveniently be computed using the well-established strain compatibility analysis method.

Based on the above reviews, flexural strength design using Grade 600 steel reinforcement is no different from that for normal yield strength steel reinforcement. The flexural resistance of the sections could be accurately computed using the well-established strain compatibility analysis method where the steel stress-strain relationship is idealized as being elastic-perfectly plastic and the ultimate concrete compressive strain equals to 0.0035 for normal strength concrete and reduced to as low as 0.0026 for high strength concrete.

### Moment redistribution

Moment redistribution often provides for reserve capacity in RC beams in the event of overload. At present, no test data are available to judge if moment redistribution is applicable to members with Grade 600 reinforcements. The neutral axis depth is considered the best parameter for quantifying the moment redistribution. EC2 states the permissible amount of redistribution depends on the tensile strain of the longitudinal reinforcement at the extreme layer, with the maximum amount being 30%, but it restricts the limitation of neutral axis depth to the effective depth  $x_w/d$  to a small value, intend to increase the maximum strain of tension reinforcement to be considered sufficient for rotational capacity.

### Shear behaviour

Shear design according to EC2 [1] accommodates steel reinforcement strength up to 600 MPa. However, the available safety margin in using steel bars of Grade higher than 500 MPa have not been adequately investigated [7]. The use of Grade 600 as shear reinforcement may not lead to conservative design if the actual yield strength of the steel bars is directly substituted into the current shear equations in the EC2. The use of Grade 600 rebars as flexural reinforcement may also affect the concrete contribution to shear strength. In cases where Grade 600 rebar is used to reduce the amount of longitudinal reinforcement, higher strains are likely to occur over the cross-section for the same amount of loading. Higher longitudinal strain will result in larger crack widths, which can reduce the amount of shear transferred across the cracks. In addition, a reduction in the amount of longitudinal reinforcement reduces the depth of the compression zone which is considered a primary region contributing to shear transfer.

The reinforced concrete beams without shear reinforcement is brittle, In EC2 [1], the shear capacity of a reinforced concrete member is a function of several factors – namely concrete compressive strength ( $f_{ck}$ ), ratio of tension reinforcement ( $\rho$ ), shear span to depth ratio ( $a/d$ ), size effect, depth factor ( $k$ ) and width of the section ( $b_w$ ). The performance of RC members subjected to shear and reinforced with high yield strength steel reinforcements is not well established. One concern is whether the high stress levels induced in the reinforcement may cause more or wider cracks in the concrete which can affect the shear capacity. Several studies have been conducted to investigate the use of high-strength reinforcement as shear reinforcement [8-15] to verify the applicability of the shear design for high strength steel reinforcement. In addition, a National Cooperative Highway Research Program (NCHRP) research project was conducted to evaluate the use of high-strength reinforcement [16] and tests were included specifically evaluating the use of high-strength reinforcement as shear reinforcement. In the NCHRP study, only small differences in crack widths between Grade 400 and Grade 690 stirrups were observed, indicating that crack control may be possible at higher stress levels, test results showed that design shear reinforcement with yield strength up to 690 MPa using AASHTO specifications [17] is still applicable and safe. It should be pointed out the design provisions in the AASHTO code [17], account for the effect of neutral axis depth on shear strength, provided adequate estimates of normalized shear strength for the range of concrete compressive and reinforcement yield strengths studied.

Sumpter et al [11], found that the use the strength of high yield strength steel stirrups cannot be utilized beyond 552 MPa because the failure was controlled by crushing of the concrete in the strut. Pairing high-strength concrete with high yield strength steel reinforcement could provide a better use for high yield strength steel reinforcement. Munikrishna et al [14] also tested 18 large-scale beams reinforced with high-strength longitudinal and transverse reinforcements (690 MPa). Failure was typically due to crushing of the concrete strut for beams with and without stirrups. For beams with high-strength stirrups (690 MPa), the measured strains in the stirrups were equal to or greater than the strain of 0.0035 corresponding to Grade 100 (690 MPa) prior to crushing of the concrete strut [14]. This suggests that a design yield strength  $f_{yk}$  of 550 MPa for Grade 100 (690 MPa) stirrups as shear reinforcement is appropriate if appearance and serviceability due to shear cracking is not a critical design consideration. Otherwise,  $f_{yk}$  should be limited to 410 MPa.

Similar research has been conducted by Lee et al [12], based on 18 RC beams tests, Lee et al [12,13] found that a yield strength of 550 MPa can be a threshold value for yielding of shear reinforcement. Yielding of shear reinforcement is closely associated with the compressive strength of concrete. For

specimens with 667 MPa of yield strength of shear reinforcement and different levels of concrete compressive strength, yielding had been achieved in cases where concrete compressive strengths are greater than 42 MPa. Meanwhile, yielding was not obtained in the case of 34 MPa concrete compressive strength, all specimens with 550 MPa of yield strength of shear reinforcement showed shear tension failure.

There are no research results available on high strength steel reinforcement designed for torsion. For lack of research data, the design yield strength should also be also limited to 410 MPa for Grade 100 (690 MPa) shear reinforcement designed for torsion.

Based on available experimental results [8-15] on the use of high strength steel bars as shear reinforcement, the experimental shear capacities ( $V_{exp}$ ) of 34 beam specimens are compared against the design shear resistances ( $V_{Rd}$ ) according to EC2 by Felicia and Teng [15]. If the actual yield strengths of shear reinforcement from 0.2% offset (actual  $f_{ywk} > 600$  MPa) are applied directly to the equations in EC2 to obtain the shear resistances, there are 53% of the specimens with  $V_{exp}/V_{Rd}$  less than 1.0. Even with the steel strength limit of 600 MPa, there are 35% of the specimens with  $V_{exp}/V_{Rd}$  less than 1.0. Based on the analysis of experimental data, a limit of  $f_{ywk}$  to 500 MPa in the design equation of EC2 was suggested for steel bars of Grade higher than 500 MPa [15].

#### Punching shear behaviour of slabs

Limited test data is available that examines the punching shear strength of two-way slabs system containing high yield strength steel as flexural reinforcement with any shear reinforcement [18]. The use of high-strength steel reinforcement increased the punching shear strength of slabs, and concentrating the top mat of flexural reinforcement showed beneficial effects on post-cracking stiffness, strain distribution, and crack control. This increase of punching shear resistance is due to the fact that the higher strength rebars did not yield prior to punching failure.

EC2 [1] specified a different design equation for punching shear resistance, which usually exists in two-way slabs. Yang *et al* [18] found the predictions using the EC2 [1] provision were un-conservative for two of the specimens tested (S1-B and MB2). The effective design strength of the punching shear reinforcement  $f_{ywd,ef}$  in EC2 [1] is based on the mean of the effective depth of the members and has to be lower than the yield strength of the shear reinforcement bars  $f_{ywk}$ . Therefore, it is recommended to ensure that the  $f_{ywd,ef}$  is lower than 500 MPa, or even 400 MPa is more preferable [15].

#### Issues on the use of grade 600 reinforcement in RC columns

In EC2 [1], the maximum strain of concrete in compression is 0.0035 for grade up to C50/60 and this reduces with higher strength concrete to 0.0026 for C90/105 (Table 3.1 of SS EN 1992), therefore when a normal strength rebar is used as the longitudinal reinforcement, the strain of the steel bar will be able to reach compression yield before the concrete reaches its maximum strain. But when Grade 600 rebar with higher yield strain is used, it may not yield even when the concrete reaches the maximum strain at the extreme fibre. The situation is even more limiting under pure or predominantly axial compression where the concrete strain at peak stress is 0.002 for C50/60 concrete, increasing to 0.0026 for C90/105 concrete. Hence, there is a concern that the Grade 600 steel bars may not reach yield potential before the concrete reaches the maximum stress under pure compression unless very high strength concrete is used.

#### Time dependent deformations in concrete columns

In reinforced concrete columns, creep and shrinkage lead to gradual load transfer from concrete to reinforcement. Assuming that cross sections remain flat caused by small strains due to creep and shrinkage under load, the stresses decrease in the concrete and increase in the reinforcing bars over time. A concrete element when kept under sustained load presents progressive strain over time, associated to the creep. In reinforced concrete columns, such deformations cause the stress increase in the steel bars of the reinforcement and may induce the material to undergo the yielding phenomenon [19].

Rossi and Maou [20] conducted an experimental study of creep behaviour of concrete at variable stress levels (30, 50 and 70% of concrete compressive strength). They reported that a strain value of 0.005 is obtained without the failure of the concrete specimen after more than one year of loading at 70%, and that the strain is about 2.5 times the strain at the peak of a classical compression test).

Madureira *et al* [21] conducted a numerical study on creep strains on reinforced concrete columns. They presented a figure to show the curves of the creep coefficient evolution with time. Percentage change of total axial strain (with long-term effects of concrete) compared to conventional axial strain is about 200% in 400 days.

Ranaivomanana *et al* [22] conducted an experimental study of creep behaviour of concrete at variable stress levels. They concluded that in terms of stress levels, non-linearity was found for compressive creep to arise somewhere between 30% and 50% of the high-strength concrete.

More recently, Zheng *et al* [23] also tested 12 groups of 100 x 100 x 400 mm reinforced concrete specimens under variable compressive stress to investigate effects of parameters on the creep of RC columns and the sectional stress redistribution. They found that the ultimate creep value of axial compression RC columns decreases with the increase of longitudinal compression steel reinforcement ratio. SAH [24] reported on their study on the time-dependent concrete deformations and their impact, including the comparison between the methods from EC2 [1] and ACI 318 [4]. The additional strain due to time effects were calculated for two cases: a simulation of a test-setup for a column subjected to concentric load until failure and a simulation of a load history for a multi-story building determining the additional strain in each level. For the first case of a concentric load until failure, the additional strain gained by the long term effects was approximately 0.1% after a year. For the second case, the strain development after 2 years for a 40 storey building including a construction time of one year was 0.05%. Falkner *et al* [25] demonstrated that by taking creep and shrinkage of the concrete into consideration, the yield strength (up to including 670 MPa) of high strength steel reinforcement steel can be fully exploited. TTK[26] in the study on the applicability of high strength rebar to concrete structures also proposed that the effect creep and shrinkage in increasing the compressive strain can be considered in column design. For example, if the permanent load is 30% of the factored load, the additional strain of 0.12% from the time effect can be attained. This enables the use of high strength steel of 600 MPa yield strength.

In EC2 [1], for cross-sections that are subjected to bending and compression at the same time, the compressive strain in the concrete shall be limited to  $\epsilon_{cu2}$  or  $\epsilon_{cu3}$ . However, cross-sections or part thereof are subjected to approximately concentric loading, the compressive strain should be limited  $\epsilon_{cu2}$  or  $\epsilon_{cu3}$ .

When normal strength steel is used as the main reinforcement for column, the yield strain of the rebar is equal or lower than the strain of concrete under pure compression and both materials can reach the full potential strength. But, when the high strength steel with a higher yield strain is used, it may not yield even when the concrete reaches the maximum stress. Hence, it is necessary to check the yield strain of rebar against the strain of concrete.

In EC2 [1],  $\epsilon_{cu2}$  is 0.0035 for normal strength concrete (NSC) and reducing to 0.0026 for high strength concrete (HSC). As shown in Figure 1 and this compression strain of concrete for NSC is equal or higher than the design yield strain of steel of strength  $f_y = 700$  MPa or lower and hence the steel has the potential to reach the yield strength under bending and compression load. Similarly, this is also true for  $f_y = 600$  MPa for even HSC. As for pure compression load, if the strain of concrete  $\epsilon_{c2}$  is equal or higher than the design yield strain of reinforcing bars for the case of steel with  $f_y = 460$  MPa, the pure axial strength equation without influence of creep and drying shrinkage is

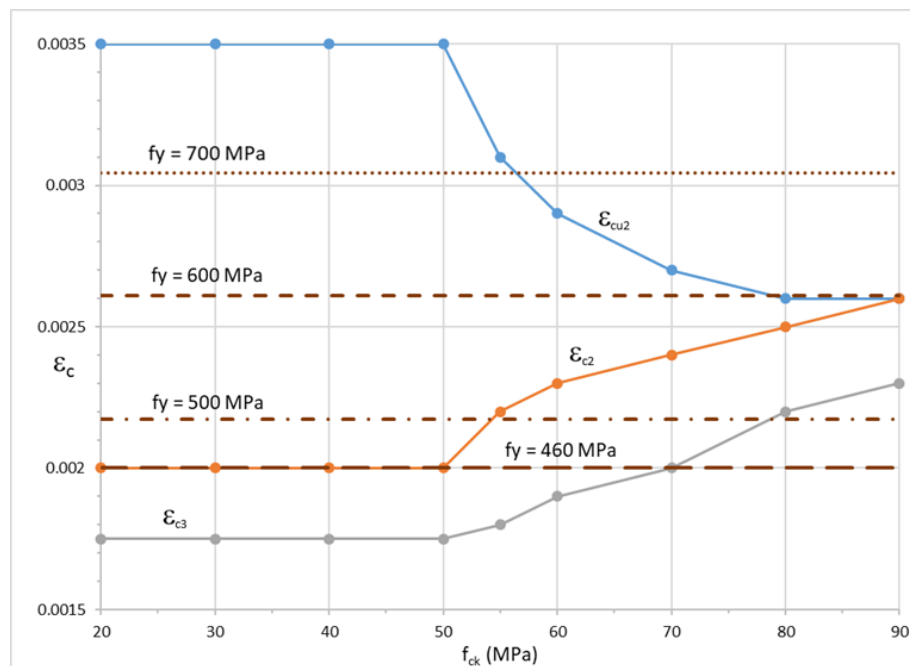
$$P_{uD} = 0.567 f_{ck} (A_g - A_{st}) + 0.87 f_{yk} A_{st} \quad \text{Equation 1}$$

However, if the strain of concrete  $\epsilon_{c2}$  is lower than the design yield strain of reinforcing bars,  $f_{ykd}$ , for reinforcement equal or higher than 600 MPa yield strength, then it is not possible to realise all the potential strength of the steel reinforcement. This is also true for 500 MPa yield strength steel if concrete

strength  $f_{ck}$  is less than 55 MPa. Therefore, when the influence of creep and drying shrinkage is neglected, the pure axial strength equation should be modified as following:

$$P_{UD} = 0.567 f_{ck} (A_g - A_{st}) + E_s \epsilon_{c2} A_{st} \quad \text{for } \epsilon_{c2} < \epsilon_{ykd} \quad \text{Equation 2}$$

In case of rebar with 500 MPa yield strength, when the compressive strength of concrete is less than 55 MPa, the normalised axial strength is below unity. This is because even though the value of  $\epsilon_{c2}$  increases with concrete strength, for concrete strength lower than 55 MPa, the value of  $\epsilon_{c2}$  is less than the design yield strain of 500 MPa ( $\epsilon_{c2} = 0.00217$ ). Similarly, rebar with >600 MPa yield strength, the normalised axial strength for all concrete strength is below unity.



**Figure 1.** Concrete and rebar strains

The above results are based on the analysis without considering the creep and the drying shrinkage in the reinforced concrete columns under the pure axial force. The effect of creep and shrinkage in the concrete will cause redistribution of forces between the rebar and concrete; the stress and strain in the rebar will be increased as a result. TTK[26] proposed that a simple formula as shown in (Equation 3) can be used to represent the hypotheses:

$$\epsilon_{c2,long} = (1 + \phi_{mcs}) \epsilon_{c,sust} + (\epsilon_{c2} - \epsilon_{c,sust}) = \phi_{mcs} \epsilon_{c,sust} + \epsilon_{c2} \quad \text{Equation 3}$$

$\epsilon_{c2,long}$  is the concrete strain in considering creep and drying shrinkage;  $\phi_{mcs}$  is the final increase coefficient in considering creep, drying shrinkage and rebar ratio;  $\epsilon_{c,sust}$  is the initial strain when the sustained load is applied.

If the fixed load was assumed to be 0.3 times of the factored load,  $\epsilon_{c,sust}$  could become 0.0006 or 0.3 times of the  $\epsilon_{c2}$  which is 0.002 for C50 and below concrete.

The coefficient  $\phi_{mcs}$  was taken as 2.0 and the additional strain would be 0.0012. The total strain of the concrete then became 0.0032 from the above equation. This value exceeds the design yield strain of 0.0026 of rebar with the yield strength of 600 MPa. Therefore, if the effects of creep and drying shrinkage are considered, the calculation formula of the pure axial strength in the Eurocode design could be applicable up to the Grade 600 rebar for all strength grades of concrete.

Similarly, SAH [24] reported that the value of the additional strain can be assumed to be around 0.0005 and together with  $\varepsilon_{c2}$  of 0.0022 for C55 concrete gives a higher strain than design yield strain of 0.0026 for Grade 600 rebar.

#### Columns subjected to combined bending and compression

Under combined bending and compression, the maximum concrete compressive strain is higher than the strain at yield for Grade 600 steel reinforcement. Hence, there is no limit placed on the steel reinforcement to reach yield potential. The design of columns will not be different from those for the lower strength steel.

#### Concrete Strength

Existing literature includes tests combining high-strength concrete with high strength reinforcement [10,29]. It is considered advantageous to use high-strength concrete in members that will use high-strength reinforcement. High concrete strength will reduce the required development and splice lengths of reinforcement, improve deformation capacity of flexural members, increase the shear strength of members, improve the strength of columns with high axial loads or combined axial load and flexure, increase the shear strength of joints in special moment frames, and reduce deflections ([30]

Use of high-strength concrete improves the deformation capacity of flexural members. Considering two beams close to their ultimate deformation capacity, the beam with higher strength concrete will have a shallower neutral axis depth, slightly increased peak moment strength, more curvature, higher bar tensile strain, and larger hinge rotation. This in turn will lead to greater deflection and wider crack width.

Comparative results Lee *et al* [13] reveal that diagonal crack width can be influenced by the amount of shear reinforcement and compressive concrete strength. In short, the width is inversely proportional to amount of shear reinforcement divided by compressive concrete strength. Yielding of shear reinforcement is closely associated with the compressive strength of concrete.

Similarly, high-strength concrete improves the strength of columns with combined axial load and flexure because a smaller stress block can support the same compressive force of normal strength concrete section. For the same axial load, a smaller stress block will increase the distance between the tension and compression couple, resulting in higher flexural strength.

#### Serviceability limit

With the use of Grade 600 reinforcement in design based on EC2 [1], for the same ultimate capacity, the amount of required flexural reinforcement can be decreased, resulting in lower reinforcement ratios or reduced member depth. Thus, the strain in the reinforcement at the service condition is higher than that in comparable members designed with Grade 460 reinforcement. Under service condition, steel stress is usually taken as 67% of  $f_y$ . Mast [27] shows that for Grade 600, the deflection at service load is 1.4 times that for members with Grade 460 reinforcement. So, slabs and beams using 600 rebars may result in lighter reinforced members. If, the same amount but higher grade reinforcement is used for the same member size, the ultimate capacity is increased but the deflection remains the same for the same steel service stress. When Grade 600 steel is used as longitudinal reinforcement, the designer should make direct calculation of deflections for the reinforced members.

#### Crack control

It is known that strain in the tension steel is one of the primary variables for controlling concrete crack width. However, slabs and beams using Grade 600 rebars may result in lightly reinforced members with increased steel service stress. As stated in the EC2 Commentary [1] for a stress level of 200 MPa there is a probability of 95% that a maximum crack width smaller than 0.3 mm occurs, this also imply that the formulas for calculation of crack width shall aim at controlling the stress in tensile steel around 200 MPa if the allowable width of crack is 0.3mm, therefore there is no gain in performance under service condition when using high strength steel instead of normal strength steel. In general, any



reduction in steel area would lead to higher steel stress under the same service condition, resulting in wider cracks than section with more steel area. The designer should make direct calculations to check the crack width under service condition.

### Development length

Adequate bond strength between concrete and reinforcing bars is necessary for the design of reinforced concrete structures. Therefore, developing the proper length of concrete-embedded rebar is crucial for obtaining its full tensile capacity. If the distance is less than the defined development length, the bar will pull out of the concrete. According to EC2 [1], the development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement. The development length is a function of steel bar yield stress, concrete compressive strength, and bar diameter. High strength concrete will reduce the required development and splice lengths of reinforcement.

There have been several investigations on bond and development length of high yield strength rebars of different sizes and with different concrete strengths. An experimental study was performed by Choi et al [30] to evaluate the applicability of ACI 318 [4] to the tension splice of 600 MPa reinforcing bars. Twelve simply supported beams (SB1 –SB12) with reinforcing bar splices were tested under monotonic loading. The results showed that the reinforcing bar splices gave a satisfactory performance for all D13 (diameter = 13 mm) bar splices. For D22 and D32 bars, the use of either transverse reinforcement or thick concrete cover was recommended to develop the specified yield strength of 600 MPa. The accuracy and safety of current design codes (ACI 318 [4], ACI 408R [28] and EC2 [1]) were evaluated by Choi et al [30] on the basis of available test results. The developed-calculated bar stress ratios  $f_{s,test}/f_{s,cal}$  for specimens SB1 through SB12 tested are compared. The average value of  $f_{s,test}/f_{s,cal}$  was 1.16 (COV = 0.21, minimum = 0.93), 1.18 (COV = 0.15, min = 0.97) and 1.25 (COV = 0.33, min = 0.86) for ACI 318 (Class B splice), ACI 408R [28] and EC2 [1], respectively. Both the average and the coefficient of variation (COV) were the greatest in EC2. In the case of high-strength concrete (68 MPa- SB10), EC2 [1] appears to be unsafe. In the case of high lateral confinement (SB4 and SB6), it appears to be excessively conservative. Lap-splice behavior is not widely understood due to lack of published data. Grade 600 rebars require long anchorage and splice lengths. Based on EC2 formulas the anchorage and splice lengths of Grade 600 would be 20% longer than the Grade 500 rebars, which may be uneconomical or impractical, thus the designer may consider using mechanical splices and headed bars.

### **3. Conclusions**

This paper synthesizes the current information on the use of high strength Grade 600 reinforcing steel so that engineers and researchers can work towards developing guidelines for their use, the following main findings can be summarised as follows

- Flexural strength design using Grade 600 steel reinforcement is no different from that for normal strength steel reinforcement. The flexural resistance of the section could be accurately computed using the well-established strain compatibility analysis method.
- To avoid abrupt shear failure due to concrete crushing before the yielding of shear reinforcement and to control the diagonal crack width, it is recommended to have a limitation on the yield strength of shear reinforcement of RC beams. Based on available experimental study, the  $f_{yw}$  is to be limited to 500 MPa for Grade 600 as shear reinforcement.
- Slabs and beams using Grade 600 rebars may result in lightly reinforced members with increased steel service stress. As there is no gain in performance under service condition when using high strength steel instead of normal strength steel, the designer should make calculations to check for crack width and deflection limits in serviceability limit states.
- Grade 600 steel reinforcements require long anchorage and splice lengths thus the designer may consider using mechanical splices and headed bars.
- Comparing Singapore National Annex to EC2 and EC2 recommendations on moment redistribution, section designed according to EC2 recommendation will have more ductile or more rotational capacity. With shallower depth of neutral axis, it will ensure reinforcement will yield before concrete reaches

ultimate strain 0.0035. For steel stronger than Grade 500, it is recommended that the parameters in Clause 5.5(4) on moment redistribution as suggested by the EC2 to be used instead of those given in the NA to SS EN 1992-1-1.

- In the design of columns using the EC2 material data, the potential strength of the steel reinforcement for yield strength of up to 460 MPa can be fully realised in compression (pure compression or with bending). However, for reinforcement of equal or higher than 600 MPa yield strength, it is not possible to realise all the potential strength of the steel reinforcement under pure compression. This is also true for 500 MPa yield strength steel if concrete strength  $f_{ck}$  is less than 55 MPa. The effect of creep and shrinkage in the concrete will cause redistribution of forces between the rebar and concrete and as a consequence, the stress and strain in the rebar will be increased to reach potential of the high strength steel reinforcement.
- At present, an insufficient amount of testing has been performed to warrant the performance and behaviour of RC member using high strength steel reinforcements into current design guidelines and practices.

### Acknowledgment(s)

The work forming the basis for this paper was conducted pursuant to a contract with the Singapore Building and Construction Authority (BCA). Contract No. / Issue No: BCA 000ECO17000237.

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