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The Effect of Reinforcement Strength on the Overstrength Factor for Reinforced Concrete Beams

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ABSTRACT: The design of earthquake resistant structures in New Zealand is based around the philosophy known as capacity design. In order for this philosophy to be successfully applied, it is essential that the flexural overstrength factor is appropriately defined. Overstrength factors for reinforced concrete structures are defined in the New Zealand Concrete Structures Standard, NZS 3101:2006, which currently prescribes the flexural overstrength factor for beams as 1.25 if the beam contains Grade 300E longitudinal reinforcement and as 1.35 if the beam contains Grade 500E longitudinal reinforcement. However, review of existing literature and consideration of structural behaviour does not support the use of different overstrength factors for different types of reinforcement. Analysis of a database of approximately one hundred beam-column joint tests indicates that the same overstrength factor should be used for reinforced concrete beams irrespective of whether they contain Grade 300E or Grade 500E longitudinal reinforcement.

1 INTRODUCTION

For approximately thirty years, New Zealand seismic design has been based around the philosophy known as capacity design. Anecdotally, capacity design was developed through discussions between New Zealand academics and practicing structural engineers during the 1960s before first being presented in the literature by Hollings (1969). Park and Paulay are generally the names most closely associated with the capacity design philosophy due to their seminal textbook (Park & Paulay 1975), which provided the first comprehensive treatment of the capacity design philosophy.

Application of capacity design for practical applications is complicated by a number of factors such as dynamic shear amplification and strain hardening of yielding members. This second factor is accounted for by application of overstrength factors, which are intended to allow calculation of the maximum ("overstrength") moment that could develop in the yielding elements of the structure. Correct definition of the overstrength factor is critical to successful application of capacity design because it is this factor that is used to determine the design actions for non-ductile parts of a structure and hence to minimise the chance of brittle failure.

For reinforced concrete members, overstrength factors can be defined as either material overstrength factors or flexural overstrength factors, with the method used to calculate the overstrength moment differing depending on the type of overstrength factor used. Using material overstrength factors, the nominal material properties are increased according to the overstrength factors and the overstrength moment capacity is then calculated by conducting some form of section analysis using the resulting increased material properties. Using a flexural overstrength factor, the overstrength moment capacity is calculated as the product of the nominal moment capacity and the overstrength factor. The value of the flexural overstrength factor is approximately equal to the value of the material overstrength factor of the reinforcement for rectangular beams without flanges, which are the focus of this paper.

However, for most members (such as flanged beams, walls, and columns) these two values can differ significantly.

2 BEAM OVERSTRENGTH FACTORS IN NEW ZEALAND

As implied in the previous section, the overstrength factor for beams without flanges is primarily determined by characteristics of the longitudinal reinforcement; other factors such as concrete strength, section shape, reinforcement ratio, and confinement do not have a significant effect on the overstrength factor (Andriono & Park 1986). Two aspects of the reinforcement have a significant effect on the overstrength factor:

- The mechanical properties of reinforcement are important because the overstrength moment occurs when the section curvature greatly exceeds the yield curvature. Large reinforcement strains occur as a result of these curvatures and therefore significant strain hardening of the reinforcement increases the moment resisted by the member.
- The variability of reinforcement properties is important because the overstrength moment is generally calculated as an upper characteristic (95th percentile) value whereas the nominal strength is calculated as a lower characteristic (5th percentile) value. Thus the overstrength factor is higher if the variability of reinforcement properties is large, resulting in a substantial difference between the upper and lower characteristic strengths.

Typically it will be found that the overstrength moment is 25% to 50% greater than the nominal flexural strength, meaning that the flexural overstrength factor ranges from 1.25 to 1.5. As outlined in the next section, overstrength factors in New Zealand have generally been at the lower end of this range due to the relatively tight control imposed on reinforcement properties in this country.

2.1 *History of New Zealand overstrength factors*

Different overstrength factors are specified in New Zealand depending on the reinforcement type used in the beam. The history of overstrength factors is thus closely linked to the historical changes in reinforcement grades used in New Zealand. These grades have always been denominated based on the yield strength of the reinforcement (hence Grade 300E reinforcement has a yield strength of 300 MPa), although as noted below the manner of defining the yield strength has changed.

When capacity design and thus overstrength factors were first included in the 1982 New Zealand Concrete Structures Standard (NZS 3101 1982), reinforcing steel grades were denominated by minimum yield strengths and consisted of mild steel Grade 275 reinforcement, and high strength Grade 380 reinforcement. The overstrength factor for Grade 275 reinforcement was set at 1.25, while the overstrength factor for Grade 380 reinforcement was set at 1.4 in recognition of its greater potential for strain hardening.

By the time that the Concrete Structures Standard was replaced in 1995 (NZS 3101 1995), reinforcing steel grades were denominated by lower characteristic yield strengths. Mild steel reinforcement was described as Grade 300E reinforcement, but was essentially the same material as the previous Grade 275 reinforcement. In contrast, Grade 380 reinforcement had by 1995 been replaced by more ductile Grade 430 reinforcement, which had superior structural characteristics. Due to the improved (i.e. reduced) strain hardening behaviour of Grade 430 reinforcement, the overstrength factor for all reinforcement was set at 1.25 in NZS 3101:1995.

In the early years of this century, Grade 500E reinforcement was introduced to New Zealand as a replacement for Grade 430 reinforcement. Shortly after, the overstrength factor for Grade 500E reinforcement was set at 1.4 by the third amendment to NZS 3101:1995. This value was initially carried over when the most recent edition of the Concrete Structures Standard (NZS 3101 2006) was introduced. Significant changes were made to the specification of overstrength factors when amendment 2 was applied to NZS 3101:2006 in mid 2008. This amendment reduced the overstrength factor for Grade 500E reinforcement to 1.35, based on work by Allington et al. (2006), and also introduced a requirement that overstrength actions for all members be calculated based on material overstrength factors. This second change was in contrast with previous standards that had permitted flexural overstrength factors to be used for beams. The change to material overstrength factors was made in recognition of the significant difference for flanged beams between the effective cross section at nominal strength and the effective cross section at overstrength.

2.2 *Background to current beam overstrength factors*

Two significant studies on the flexural overstrength factor have been conducted in New Zealand, and can be considered to provide the justification for the current beam overstrength factors.

Andriono and Park (1986) conducted statistical analyses of the properties of then current New Zealand reinforcing steel (Grades 275 and 380) and then used Monte Carlo techniques to generate stress-strain curves for reinforcing steel. Using the generated stress-strain curves, 500 moment-curvature analyses were then conducted on each of a total of 192 different beam sections (giving 96,000 analyses in total). Mean and upper characteristic overstrength factors were calculated for each beam section at curvature ductility levels of $\mu_\varphi = 10$ and $\mu_\varphi = 20$. It was found that the calculated overstrength factor was insensitive to factors such as reinforcement ratio, section size and shape, and concrete strength. Hence a single overstrength factor could be recommended for each curvature ductility level and reinforcement type. Overstrength values suggested by Andriono and Park can be seen in Figure 1. The overstrength factor for curvature ductility of 15 was a value assumed by Andriono and Park and recommended for use when the curvature ductility was not explicitly determined. It is assumed here, and implicitly by the current concrete design standard (NZS 3101 2006), that the work of Andriono and Park continues to be relevant for Grade 300E reinforcement.

Overstrength factors for Grade 500E reinforcement have been considered in a series of studies commissioned by Pacific Steel and undertaken by Holmes Solutions (Allington et al. 2006; Bull & Allington 2003). These studies followed a similar methodology to Andriono and Park (1986) but used approximately 1600 stress-strain curves obtained from testing Grade 500E reinforcement rather than generating stress-strain curves based on statistical assumptions. After conducting 154,800 moment-curvature analyses on different beam sections Allington et al. concluded, as had Andriono and Park, that variables other than reinforcement stress-strain response and curvature ductility level had little effect on the overstrength factor. They published overstrength ratios for curvature ductility levels of 5, 10, 15, 20, and 25 (shown in Figure 1) and recommended a value of 1.35 for beams reinforced with Grade 500E reinforcement. This conclusion was stated to be specifically applicable to reinforcement produced by Pacific Steel, but has since been adopted in the second amendment to NZS 3101:2006 for any Grade 500E reinforcement that complies with the New Zealand Standard for reinforcing steel (AS/NZS 4671 2001).

An additional point not published in the original research has been added to Figure 1 for each reinforcement type, representative of the overstrength factor when the curvature ductility is one. These extra values were estimated as the ratio of the upper characteristic (95th percentile) yield stress to the nominal yield stress, with upper characteristic values taken as those reported by Andriono and Park for Grade 300E reinforcement and by Allington et al. for Grade 500E reinforcement. The assumption inherent in this estimation is that beam moment is proportional to reinforcement stress. While not strictly correct, it can readily be shown that for typical beam dimensions this assumption is sufficiently accurate, particularly if the tension reinforcement has yielded because the centroid of compression remains approximately static after yielding has occurred.

Considering Figure 1 it is not evident why the New Zealand Concrete Structures Standard specifies a lower beam flexural overstrength factor for Grade 300E reinforcement than for Grade 500E reinforcement. While Figure 1 shows that the overstrength factor required for Grade 500E reinforcement is greater than for Grade 300E reinforcement when curvature ductility levels are approximately ten, for larger curvature ductility levels the required overstrength factor is larger for Grade 300E reinforcement. It should also be noted that if all other factors (i.e. displacement and member dimensions) are equal, larger curvature ductility values would generally occur in a member containing Grade 300E reinforcement than in a member containing Grade 500E reinforcement because of the proportionately of yield curvature with reinforcement yield stress (NZS 3101 2006; Priestley

1998). Consideration of this statement in conjunction with Figure 1 suggests that, if anything, it would be more likely for a lower overstrength factor to be appropriate for Grade 500E reinforcement than for Grade 300E reinforcement.

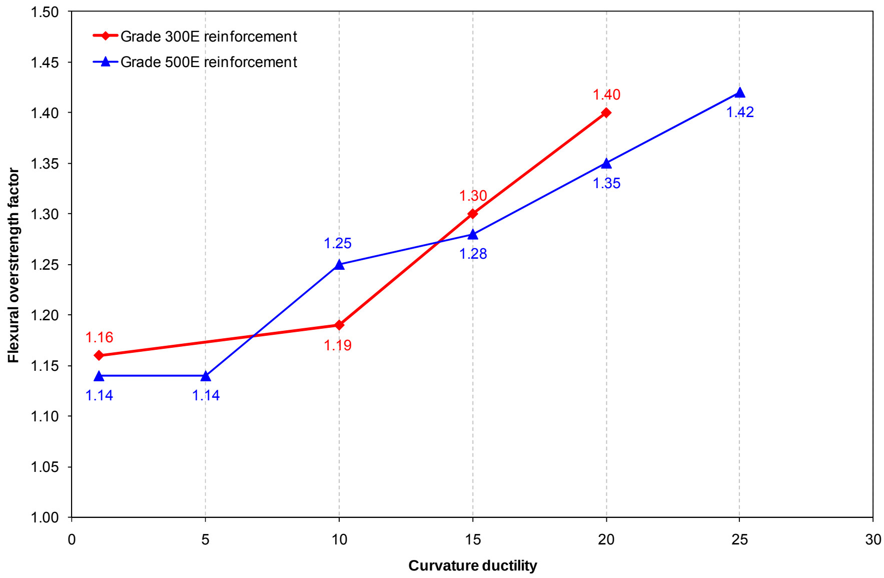

Figure 1. Beam flexural overstrength factors determined by previous researchers

3 REASSESMENT OF BEAM OVERSTRENGTH FACTORS

In order to investigate the possibility that it is inappropriate to have different overstrength factors depending on whether a beam is reinforced with Grade 300E steel or Grade 500E steel, an investigation was conducted that assessed the overstrength moments developed during testing of approximately 100 beam-column joints. The following sections discuss the composition of the database of beam-column joint tests, the methodology used to assess overstrength factors from the tests, and the appropriate values for overstrength factors for use in design.

The methods used in this research mean that the overstrength factors determined are flexural overstrength factors. However, due to the research considering only rectangular beams (i.e. without flanges) the overstrength factors are also equal to the material overstrength factor for the reinforcement used due to the equivalence discussed in section 1, and hence are compatible with NZS 3101:2006 amendment 2.

3.1 *Database composition*

The database of experimental results assembled for assessment purposes consisted of a large number of beam-column joint tests conducted in New Zealand and internationally over the last four decades, resulting in the database and the method of assessment being similar to, but larger than, those used in previous studies conducted by Lin (1999) and by Fenwick & Megget (2003). A few basic criteria were initially used to judge the suitability of a beam-column joint test for inclusion in the database, some of these criteria being based on the use of the database for other research (Brooke 2011). The primary factors considered were that the joint:

- Was a reinforced concrete interior beam-column joint
- Was subjected to a cyclic loading history that included multiple inelastic cycles
- Was not subjected to bidirectional loading
- Performed in a manner consistent with the weak beam – strong column design ideal.

In total 161 beam-columns joint tests reported in approximately 35 reports, theses, and papers were considered for inclusion in the database. In many cases complete series of tests or experimental programmes were considered but ultimately disregarded. Reasons for disregarding complete experimental programmes included use of an insufficiently demanding load history, insufficient data being provided in the literature regarding detailing or performance, shear failure obviously occurring in all beam-column joints tested, or the beam-column joints tested being unrepresentative of current New Zealand design practice. Research programmes considered but ultimately totally excluded included those conducted by Allington (2003), Birss (1978), Fenwick & Irvine (1977), Fenwick (1981), Hakuto (1995), Lee et al. (2007), Leon (1989; 1990), Liu (2002), Meinheit & Jirsa (1977), Park & Keong (1979), Stevenson (1980), Thompson (1975), The University of Texas at Austin (Guimaraes et al. 1989; Kurose et al. 1988), and Wong et al. (1985). The exclusion of these programmes resulted in 93 beam-column joint tests finally being included in the assembled database, comprising some or all of the joints tested by Amso (2005), Beckingsale (1980), Brooke (2011), Central Laboratories (Lawrance et al. 1991; Lawrance & Stevenson 1993; Stevenson & Beattie 1988, 1989), Cheung (1991), Dai (Park & Dai 1988), Durrani & Wight (1985), Englekirk et al. (Englekirk 1998a, b, 2003; Pourzanjani & Englekirk 2000), Joh et al. (1991a, b; Kurose 1987), Lin (1999), Milburn (1982), Oka & Shiohara (1992), Priestley (1975), Restrepo-Posada (1993), Soleimani (1978), Teraoka et al. (1997; 2005), the University of Tokyo (Kitayama et al. 1991; Kitayama et al. 1992; Kurose 1987; Lee et al. 1992), Xin (1992), and Young (1998). Table 1 shows the range of some key parameters covered by the beam-column joints included in the database, and full details of each joint are summarised in Brooke (2011). For all of the parameters shown in Table 1 it is evident that the database was broad enough to cover the whole range of beam-column joints likely to be constructed in New Zealand.

Table 1. Range of structural parameters covered by database

Parameter	Range
Beam reinforcement yield stress	265 – 858 MPa
Beam reinforcement diameter	9.5 – 35 mm
Ratio of top to bottom reinforcement	0.4 – 2.5
Concrete compressive strength	20.8 – 138 MPa
Beam depth	300 – 900 mm
Column depth	300 – 1000 mm
Column axial load ratio, $\frac{N}{A_g f_c'}$	0 – 0.43

3.2 *Evaluation of overstrength factor from test data*

Although previous studies conducted in New Zealand with the purpose of evaluating overstrength factors have used moment-curvature analyses to determine the maximum moment resisted by sections at different curvature ductility levels, it is equally valid to use experimental results from testing of suitable members instead. However, in order to conduct such an assessment it is necessary to first consider further the relationship between the nominal moment and the overstrength moment for a beam. The basic relationship linking these two quantities defines the overstrength factor:

$$M_o = \alpha_o M_n \tag{1}$$

When interpreting equation 1 for a single test, M_o is the maximum moment that occurred during testing, M_n is the nominal moment capacity of the member (i.e. calculated using nominal material properties and most commonly an assumption of a rectangular stress block), and α_o is the flexural overstrength factor derived from the previous two quantities, but obviously not immediately suitable for design purposes as it represents only a single data point. For design purposes the intent is to choose a value of the overstrength factor such that there is only a small chance that the maximum moment developed in a member during an earthquake will exceed the calculated overstrength moment M_o. In New Zealand the accepted probability of exceedence is typically taken as 5%, i.e. M_o and α_o are upper characteristic values. By considering a large enough number of test results a design value of α_o can be determined to meet this requirement with a suitable level of certainty.

In order to use experimental results collected from disparate test programmes conducted over several decades it is necessary to consider independently the components of the overstrength that arise due to variation between the nominal yield strength and the actual yield strength and that arise due to strain hardening. Fortunately independent consideration can be accomplished by assuming (with acceptable accuracy) that the moment in a beam is proportional to reinforcement stress, and by defining two "partial" overstrength factors, the first, α_{mat}, to account for the difference between the actual yield moment and the nominal moment capacity, and the second, α_{har}, to account for the difference between the maximum moment after strain hardening and the actual yield moment. That is:

$$M_i = \alpha_{mat} M_n \tag{2}$$

where M_i is the yield moment capacity predicted using actual material properties, and:

$$M_o = \alpha_{har} M_i \tag{3}$$

Introducing a constant k that accounts for the proportionality of moment and reinforcement stress, equation 1 becomes:

$$kf_{\max} = \alpha_o k f_{y,nom} \tag{4}$$

where f_{max} is the peak reinforcement stress that occurs in conjunction with M_o and $f_{y,nom}$ is the nominal reinforcement yield stress. Similarly:

$$kf_{y,act} = k\alpha_{mat} f_{y,nom} \tag{5}$$

$$kf_{\max} = k\alpha_{har} f_{y,act} \tag{6}$$

where $f_{y,act}$ is the actual value of the yield stress for the reinforcement used in a test specimen. By substitution of equations 5 and 6 into equation 4 it can be shown that:

$$\alpha_o = \alpha_{mat}\alpha_{har} \tag{7}$$

noting that the values of α_{mat} and α_{har} can be determined either from reinforcement stresses or from beam moments due to the proportionality of moments and stresses in beams.

An issue with the approach described above is that it becomes difficult to accurately determine an upper characteristic value of α_o. Determination of values for α_{mat} and α_{har} with a chosen percentile value (for example 95th) presents no problems. However α_o, being the product of these two values, is distributed according to a product distribution, which cannot be determined unless the degree of correlation between the distributions of α_{mat} and α_{har} is known. It can however be stated that if α_{mat} and α_{har} are both chosen to have the same percentile value, the percentile value of α_o will be higher unless the two original distributions are perfectly correlated, in which case the percentile value of α_o would be equal to the percentile value of α_{mat} and α_{har}. Thus a conservative estimate of the overstrength factor can be made using the process outlined above.

3.3 *Discussion of beam overstrength factors based on the database of beam-column joint tests*

Considering the procedure developed in the preceding section for determining overstrength factors in relation to the beam-column joint test database described previously, the database of experimental results can only be used to assess the part of the overstrength factor resulting from strain hardening (α_{har}). It would be incorrect to base any conclusion regarding the part of the overstrength resulting from variability of reinforcement yield stress (α_{mat}) on the database, because the variation of reinforcement properties between tests included in the database is not representative of the variation that occurs in reinforcement manufactured to meet New Zealand standards (AS/NZS 4671 2001).

It is appropriate to determine the partial overstrength factor for strain hardening, α_{har}, based on the maximum moment that occurred during testing of the beam-column joints comprising the database, because the beam-column joints were in all cases tested until failure (due to strength degradation), or until the interstorey drift to which the joint was subjected exceeded 5%. Thus the deformation of the beams and the resulting reinforcement strain hardening can be considered consistent with the maximum demands that would be imposed on beams in real structures during a "maximum credible" earthquake.

Figure 2 shows the ratio of maximum moment (M_o) to predicted yield moment (M_i, calculated using reported material properties and assuming a rectangular stress block) plotted against the beam longitudinal reinforcement yield stress for a subset of the beam-column joint database described in section 3.1. The ratio M_o/M_i plotted is equal to the partial overstrength factor α_{har} discussed previously. The subset of beam-column joints for which data is plotted was selected by excluding beam-column joints for which the beam reinforcement yield stress exceeded 600 MPa, which is the maximum value for the upper characteristic yield stress of Grade 500E reinforcement permitted by the New Zealand Standard for reinforcing steel (AS/NZS 4671 2001).

The data plotted in Figure 2 has been divided into three groups based on the strain hardening potential of the reinforcement (i.e. the ratio of reinforcement ultimate stress to yield stress, f_u/f_y). The three groups respectively consist of beam-column joints for which f_u/f_y was less than 1.6 (51 units), was greater than 1.6 (4 units), or for which f_u was unknown (15 units). To avoid overestimation of the value of α_{har} only joints for which f_u/f_y was known to be less than 1.6 were considered during the analysis described here (referred to for the remainder of this section as "the dataset"). The limit value of 1.6 was selected following consideration of the requirements of the New Zealand Standard for Reinforcing Steel (AS/NZS 4671 2001), which requires that the mean value of f_u/f_y be less than or equal to 1.5 for Grade 300E reinforcement and less than 1.4 for Grade 500E reinforcement. Setting a limitation of f_u/f_y removed from consideration a few beam-column joints that used reinforcement with excessive strain hardening potential. The ratio of f_u/f_y did not exceed 1.4 for joints that used high strength (f_y > 450 MPa) reinforcement, and hence no data was disregarded for these joints.

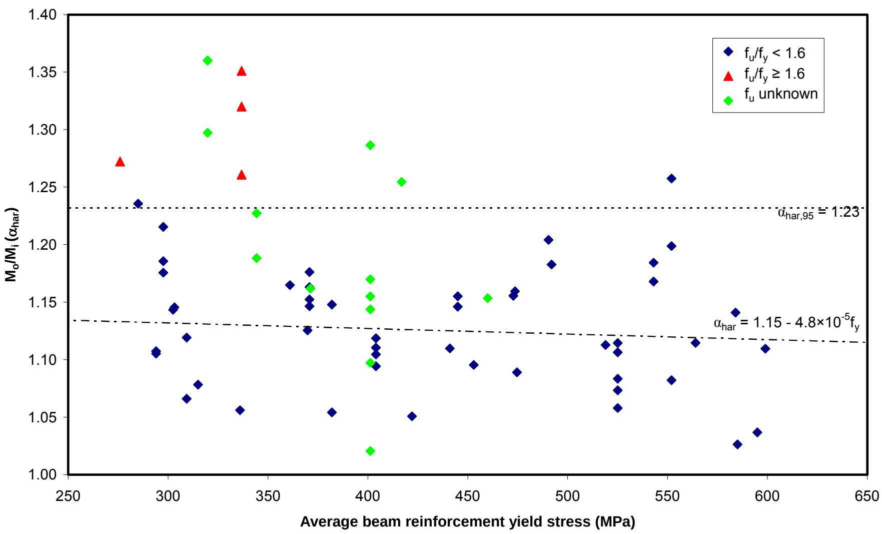

Figure 2. Values of α_{har} calculated from beam-column joint tests

Regression analysis of the group of data points for which f_u/f_y was less than 1.6 resulted in the regression line shown in Figure 2 with the equation $\alpha_{har} = 1.15 - 4.8\times10^{-5} f_y$. Although this regression line indicated a slight reduction of α_{har} as yield strength increases, statistical analysis showed that the slope of the regression line was not significant, i.e. there was a high probability that it occurred due to random scatter rather than an underlying relationship. The possibility of differing strain hardening behaviour for beams containing high or low strength reinforcement was further investigated by selecting two subsets from the dataset, the first of which contained beams that had reinforcement with $f_y \leq 350$ MPa and second of which contained beams that had $f_y \geq 490$ MPa. Analysis of these subsets using F- and T-tests indicated that there was no significant difference between the variances or means of the groups. It was therefore considered appropriate to determine a representative design value of α_{har} based on the mean and standard deviation of the complete set of data points for which f_u/f_y was less than 1.6. The mean value of α_{har} was calculated as 1.126 with the 95% confidence range from 1.108 to 1.143. The standard deviation was determined to be 0.054. Thus the upper characteristic value of α_{har} was calculated as 1.23 using Student's t-distribution, which was considered to be more appropriate than the normal distribution due to the relatively small sample size. The calculated upper characteristic value of α_{har} is plotted in Figure 2.

As was stated previously and shown in Figure 1, previous studies have shown that the ratio of upper characteristic yield strength to nominal yield strength is 1.16 for Grade 300E reinforcement and 1.14 for Grade 500E reinforcement. Based on these values, use of an average value of $\alpha_{mat} = 1.15$ for both grades of reinforcing steel can be justified. Combining this value with $\alpha_{har} = 1.23$ results in a value of $\alpha_o = 1.23 \times 1.15 = 1.42$ being calculated using equation 7. This value is appropriate for either reinforcement type.

The value of $\alpha_o = 1.42$ calculated above corresponds to the upper limit of values determined by previous studies (see Figure 1). However, as noted earlier the value calculated is probably more conservative than the 95th percentile rank normally used in New Zealand for the overstrength factor. The nature of the investigation conducted here prevents calculation of the correlation between α_{har} and α_{mat}, which would allow precise determination of the percentile rank of the calculated overstrength factor. However, the more important conclusion drawn from the assessment just presented is that use of a higher overstrength factor for Grade 500E reinforcement than for Grade 300E reinforcement is incorrect, as was postulated in section 2.2. However, no evidence was found supporting the possibility of a lower overstrength factor being appropriate for Grade 500E reinforcement due to the reduced curvature ductility demands that would be expected when Grade 500E reinforcement is used.

The findings of this research suggest that the New Zealand Concrete Structures Standard (NZS 3101 2006) should be amended to specify the same overstrength factor for both Grade 300E and Grade 500E reinforcement. Based directly on this research, an overstrength factor of $\alpha_o = 1.4$ could be adopted for both reinforcement types while maintaining a high degree of confidence that the adopted value was at least a 95th percentile value. Alternately, given the uncertainty arising from the unknown correlation between material variability and strain hardening, an overstrength factor of $\alpha_o = 1.35$ could appropriately be adopted for both types of reinforcement on the basis that recent investigations showed it to be an appropriate value for Grade 500E reinforcement (Allington et al. 2006), while the research presented here indicates that the same overstrength factor should be used for both Grade 300E and Grade 500E reinforcement.

4 CONCLUSIONS

This paper has examined the flexural overstrength factors used in New Zealand for reinforced concrete beams. Current practice requires that a higher overstrength factor be used for beams reinforced with Grade 500E reinforcement than for beams reinforced with Grade 300E reinforcement. The reason for specifying a lower overstrength factor for Grade 300E reinforcement was not apparent based on review of the literature on which the overstrength factors are based. An investigation was therefore conducted in which overstrength factors were assessed based on results from testing of approximately 100 beam-column joints. This investigation concluded that the same overstrength factor should be used irrespective of whether a beam is reinforced with Grade 300E or Grade 500E reinforcement, and that the value of the overstrength factor for both reinforcement types should be between 1.35 and 1.4.

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