Material Strains and Relevance to Seismic Design

By

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Abstract

With the seismic provisions in the Loadings Standard, NZS4203:1992, being replaced by NZS 1170.5:2004 (Structural Design Actions- Earthquake actions), a number of changes have been made to seismic design in New Zealand. The most significant of these is the requirement that the level of detailing used in potential plastic regions (nominally ductile, limited ductile or ductile) be determined on the basis of the predicted magnitudes of deformation they are required to sustain in the ultimate limit state. Previously the level of detailing was determined principally on the basis of the structural ductility factor. However, it is shown that the structural ductility factor does not give a reliable guide to the deformation sustained in an individual plastic region. This paper outlines the background to the change in the way that the level of detailing is determined and it discusses how predicted inelastic deformation demands can be found in the design of concrete structures.

1. Introduction

In recent years there has been an increased emphasis throughout the world on performance based design of structures for earthquake resistance. This includes "Direct displacement based design" [1], the "Capacity spectrum method" for assessing/designing structures [2] and other approaches [3]. In New Zealand the earthquake provisions in the Standard, NZS 1170.5:2004 [4], have followed this international trend and as a result it contains;

- More specific requirements for the serviceability limit state than was contained in the previous Loadings Standard, NZS 4203:1992 [5];
- The requirement that the level of detailing in primary potential plastic regions (generally plastic hinge zones) is based on the predicted deformation they are required to sustain in the ultimate limit state.

A basic step in capacity design for ductile structures is to identify a ductile failure mechanism, which will develop prior to other failure modes in the event of a major earthquake. The plastic regions, which are generally plastic hinges, associated with the chosen mechanism are referred to as primary plastic regions. Secondary plastic hinges may form in other locations as a result of actions not considered in an analysis, such as elongation of members due to plastic hinge formation and dynamic magnification of actions associated with a change of dynamic characteristics caused by the formation of primary plastic hinges. Generally the inelastic demand is considerably less in secondary plastic hinges than it is for primary plastic hinges.

The requirement to detail potential plastic regions on the basis of the level of deformation they are required to sustain in the ultimate limit-state is a major step forward in seismic design. It results in more efficient structures with a better defined level of seismic performance. This approach follows a trend given in several different methods of seismic design or assessment of structures in the last decade [1, 2 & 3]. However, this is the first time that such a requirement has been introduced into a national seismic design code of practice. As shown in this paper this step can be readily incorporated into current practice without involving appreciable extra effort in design.

The 2006 revision of the Structural Concrete Standard, NZS 3101:2006 [6], is the first material Standard to be written to comply with the seismic design approach in the Standard for Earthquake Actions, NZS 1170.5:2004 [4]. This paper gives the background to the design steps involved in determining the predicted deformation levels in potential plastic regions of reinforced concrete structures.

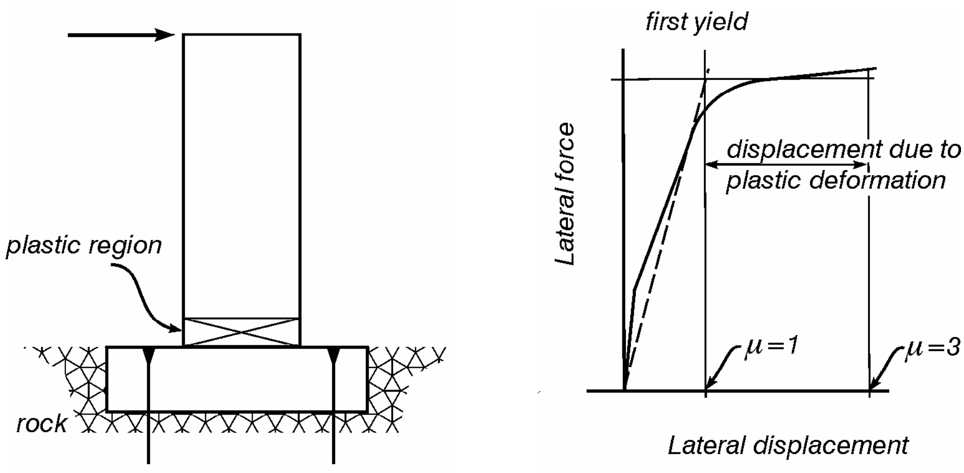
2. Structural ductility factor and material strains

Numerous tests of structural elements have shown that magnitude of inelastic deformation that can be sustained in a plastic region without loss of strength depends on many factors. The most important of these is the level of detailing in terms of confinement of concrete, restraint against buckling of longitudinal reinforcement and the proportion of shear resisted by shear reinforcement. Other significant factors influencing strength degradation include the form of plastic region, namely reversing or unidirectional, and the number and magnitudes of the inelastic load cycles that are applied.

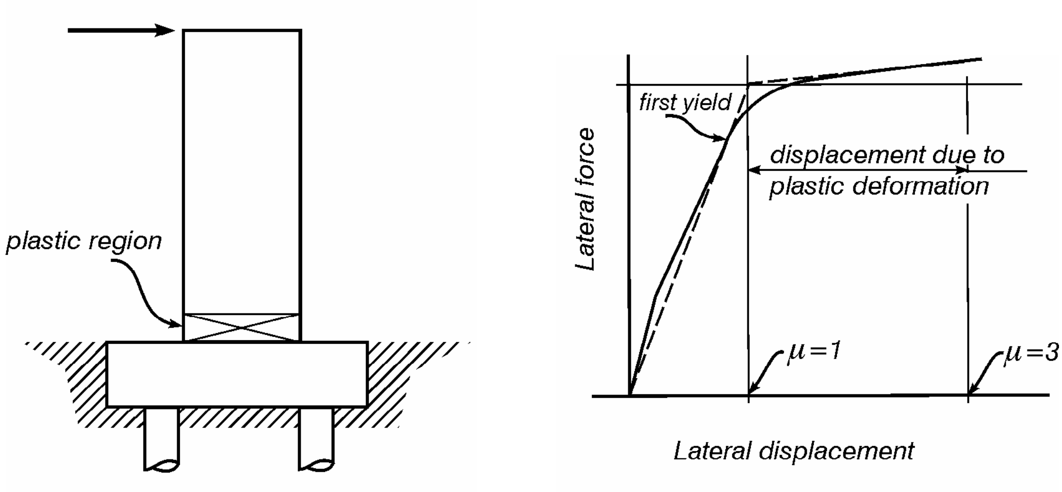
The structural ductility factor, μ, gives a measure of the ductility of a structure as a whole. This is recognised in the previous Loadings Standard, NZS 4203:1992 [5], and the new Standard for Earthquake Actions, NZS 1170.5:2004 [4]. The structural ductility factor is used to determine the seismic design forces and it influences the predicted deflected shape of the structure. In the previous Loadings Standards [NZS 4203:1992 and 1985], it was the principle factor used to define the level of detailing required in all the primary plastic regions throughout the structure. However, as illustrated in Figure 1, the structural ductility factor does not give a reliable measure of the inelastic deformation imposed on any specific potential plastic region.

To improve this situation the new Standard for Earthquake Actions [4] and the 2006 revision of the Structural Concrete Standard [6] require the inelastic deformation demand consistent with the ultimate limit state actions to be found and the magnitude of the resultant deformation used to determine the level of detailing.

Part (a) of Figure 1 shows a structural wall supported on a stiff foundation. The resultant load deflection relationship is shown on the right hand side of the figure. The ductility one displacement (μ=1.0) is relatively small as the deformation in the foundation and supporting structure is small. A similar structural arrangement is shown in part (b) of Figure 1. In this case an identical wall to the one shown in part (a) is supported on a flexible foundation. When lateral seismic design forces are applied the ductility one displacement is greater for this wall due to the flexible foundation. As can be seen from the figure when the two structures are taken to the same level of ductility much greater inelastic deformation is imposed on the plastic hinge in the wall with the flexible foundation than in the case of the wall with the stiff foundation. It is this deformation which is the main factor determining the level of detailing required to prevent failure. From this it can be seen that the structural ductility factor does not give a reliable guide to the inelastic deformation demand placed on plastic regions and hence the need for more rational approach in assessing the required detailing level.



(a) Structural wall with stiff foundation



(b) Structural wall with flexible foundation

Figure 1. Effect of foundation rigidity on displacement ductility

Inelastic deformation in a plastic region may take several different forms. In a plastic hinge it is predominately a curvature, while in the active links of a structural steel eccentrically braced frame or in a diagonally reinforced concrete coupling beam it takes the form of shear deformation. In other cases, where seismic resistance is provided by ties or struts, it may take the form of axial tensile or compressive strains. To cover all these cases the Earthquake Actions Standard [4] introduced the general term "material strain".

In the new Structural Concrete Standard [6] three levels of detailing are specified. For plastic hinges the material strain limits are given in the form of curvatures while for diagonally reinforced coupling beams the limits are given in the form of shear deformation. In this edition of the Structural Concrete Standard no limits have been specified for axial tensile or compressive strains. The Nominally Ductile Plastic Region (NDPR) requires no special detailing for seismic actions. Members designed with nominally ductile plastic regions have limited ductility, which is sufficient to enable the levels of moment redistribution defined in the Standard to be sustained. However, the level of ductility is generally inadequate for plastic regions that are required to sustain appreciable inelastic deformation in a major earthquake. Typically NDPR detailing will be found in regular nominally ductile structures (formerly referred to as elastically responding structures). However, it should be noted that nominally ductile structures **may** require ductile or limited ductile detailing to be used in some potential plastic regions. This situation arises where a collapse mechanism may develop in a major earthquake that would not be acceptable for a ductile or limited ductile structure (see clause 2.6.6.1 in the Structural Concrete Standard [6]). Limited ductile plastic region (LDPR) detailing is required in plastic regions that are predicted to sustain moderate levels of inelastic deformation in the ultimate limit state, while ductile plastic region (DPR) detailing is required where high inelastic deformation demands are predicted to occur in the ultimate limit state.

As indicated in the previous paragraph the required level of detailing in a plastic region is not tied to the structural ductility factor. It follows that a nominally ductile structure may contain a mixture of potential plastic regions detailed as nominally ductile and limited ductile plastic regions. Similarly a limited ductile structure may contain a mix of limited ductile and ductile plastic regions. With ductile structures it is recommended in the commentary to NZS 3101:2006 [6] that ductile detailing should be used for all the primary plastic regions as the reliability with which the inelastic demand can be predicted decreases with increasing structural ductility factor. However, for ductile structures it is necessary to check that the curvatures sustained in the plastic regions do not exceed the maximum permissible values. Where this is found to occur the structure should be redesigned changing members sizes, the structural form or the structural ductility factor as appropriate.

3. Calculation of inter-storey drift

The magnitude of the predicted inelastic rotation acting in primary plastic regions may be obtained by a number of different methods. With time history analyses, in which inelastic deformation characteristics of members are modelled and P-delta actions are included, the plastic hinge rotations may be obtained directly from the output of the analysis. The critical values are taken as the maximum obtained from the set of the ground motions (minimum number of three) used to analyse the structure.

Where elastic based analyses are used, such as the equivalent static or the modal response spectrum methods, the ultimate limit-state lateral displacement envelope is obtained by modifying the corresponding elastically predicted envelope, first to allow for inelastic deformation and second for P-delta actions, as specified in the Standard for Earthquake Actions, NZS 1170.5:2004 [4]. This envelope may then be used as described in the following paragraphs to obtain the predicted values of inter-storey drift. The required magnitude of rotation sustained in plastic hinges in beams may be assessed from the inter-storey drifts for the ultimate limit state in the storeys above and below the beam being considered, while the rotation in columns or walls can be found from the inter-storey drift containing the wall or column.

The difference in the maximum lateral displacements between two adjacent levels is found from the displacement envelope described above and it is multiplied by a factor to give the predicted inter-storey drift. This factor varies between the limits of 1.2 and 1.5 (see clause 7.3.1 of NZS 1170.5 [4]) depending on the height of the structure. There are two reasons why this factor is required to convert the difference in the peak lateral displacements between two levels to the inter-storey drift.

1. The peak lateral displacements, recorded by the displacement envelope, do not occur simultaneously at all levels. Consequently the difference in the maximum lateral displacement of adjacent levels must always be less than the maximum inter-storey drift.

2. Many analyses of frame and wall structures have shown that drift envelopes based on elastic methods of analysis (equivalent static and modal), modified for inelastic deformation and P-delta actions as specified in the Standard for Earthquake Actions [4], under-estimate the inter-storey drifts in the lower reaches of multi-storey buildings. This is illustrated in Figure 2, which shows typical lateral displacement envelopes obtained by modal and time history analyses. The displacement analyses have been normalised so that both have the same displacement at the top level. The discrepancy in inter-storey displacement in the lower levels of a building obtained from time history or elastic methods of analysis increases with the height of the building, the structural ductility factor and the over-strength of members in the higher reaches of the building [7, 8 & 9].

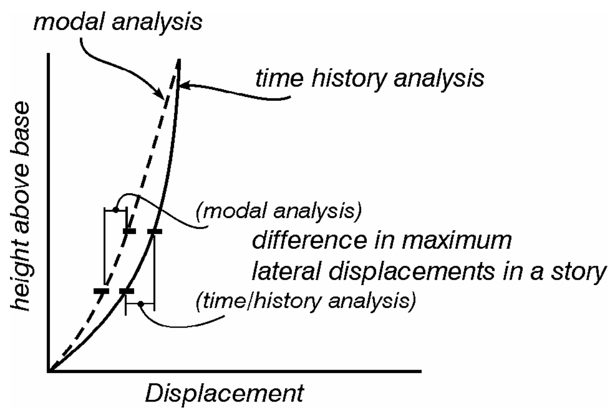


Figure 2: Lateral drift profiles for a multi-storey frame building obtained from modal analysis and time-history analysis

In the context of point 2 above the over-strength of members in the upper reaches of the building refers to the design strength being greater than that required to resist earthquake actions alone. In beams, for example, this can occur due to the critical design strength arising from gravity or wind load cases. Alternatively minimum longitudinal reinforcement requirements may be greater than the reinforcement required to resist the design actions.

Different ground motions, even when they have very similar response spectra, can result in very different values of inelastic deformation in a structure [8 & 10]. Consequently to find reliable trends for the factor, which is applied to the difference in displacement envelope values to predict the maximum inter-storey drift, a large number of analyses using different earthquake records are required. At present even though the structural ductility factor and the over-strength of members are known to influence results there is insufficient information to enable these factors to be quantified. Consequently clause 7.3.1.1 in the Earthquake Actions Standard [4] provides a simple factor to transform the difference between maximum lateral displacements at the top and bottom of a storey to the predicted inter-storey drift. The factor was found by assessing the results of comparative analyses for ductile structures [7, 8 & 9].

4. Calculation of primary plastic hinge rotations in the ultimate limit-state

The ultimate inter-storey drift in any storey in a frame or in a wall structure may be broken down into elastic and plastic components, as illustrated in Figure 3. The elastic component may be conservatively taken as equal to the value found from an equivalent static or first mode analysis of the structure. Alternatively a less conservative value may be found by scaling the elastic value found in either analysis by the ratio of the average design flexural strength of the primary potential plastic zones in the storey to the corresponding average value of the seismic design moments. The plastic inter-storey drift is obtained by subtracting the elastic component of inter-storey drift from the total inter-storey drift. The resulting plastic rotations in the plastic hinges may be calculated as illustrated in Figure 3 by considering the geometry. In a frame the rotation of columns, θ_c, at a level may be taken as the average gradient associated with the plastic deformation in the storey above and the storey below the level being considered, see Figure 3 (a), that is θ_c is given by;

$$\theta_c = 0.5\left[\frac{\delta_{i,p}}{h_i} + \frac{\delta_{i-1,p}}{h_{i-1}}\right] \qquad (1)$$

where $\delta_{i,\,p}$ is the inter-storey displacement due to plastic drift in the storey *i* and h_i is the height of storey *i*. The corresponding plastic rotation in a plastic hinge, θ_p, is given by;

$$\theta_p = \theta_c\left(\frac{L}{L'}\right) \qquad (2)$$

where L is the span of the beam between column centre-lines and L' is the distance between the centres of the plastic hinges in the beam. The expression for the plastic rotation in walls and columns can be readily derived from the geometry shown in Figure 3 (b) and (c).

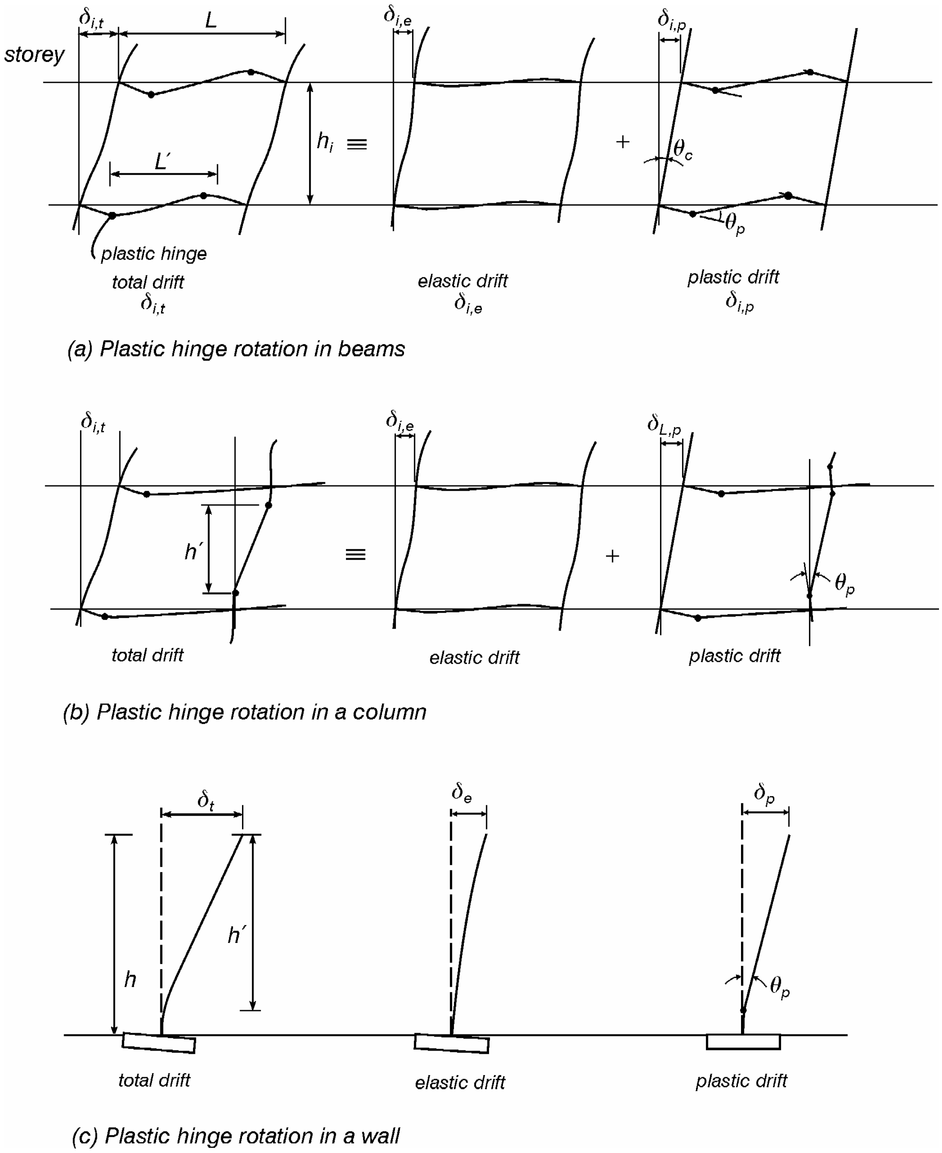


Figure 3. Calculation of plastic hinge rotations

5. Material strains

There was limited time available during the revision of the Structural Concrete Standard to develop criteria to define limiting material strains. Since the publication of the Standard this aspect has been reviewed and a number of recommendations have been made for a proposed amendment to the Standard. It is believed that the suggested changes will both simplify the design process and give material strain limits which are more soundly based than was previously the case. The values given in the remainder of this paper refer to the values in the Standard [6] but modified as proposed for the amendment to the Standard.

In the design of plastic regions it is important to recognise the different forms of plastic regions that may develop. In reversing plastic regions inelastic rotations or shear deformations in both directions (positive and negative) are induced in the same zone as the structure sways backwards and forwards. However, in unidirectional plastic regions the inelastic curvature accumulates in the same direction [11 & 12].

For nominally ductile plastic regions the limiting curvatures for unidirectional plastic regions are set at a strain at which either spalling may be expected to be initiated, or when a specified tensile strain limit is reached in flexural reinforcement. For reversing nominally ductile plastic regions the limiting value has been set at 60% of the corresponding values for unidirectional plastic regions. There was little information in the literature on which these values could be assessed. Consequently a research project is current underway to enable more realistic values to be obtained.

For ductile and limited ductile plastic regions curvature limits are set as a product of a curvature, which corresponds to the onset of appreciable inelastic deformation in the critical section of a plastic hinge, ϕ_y, and factors that vary with the type of member and the grade of reinforcement. The recommended limiting material strains for different types of members and the way in which they were derived are described in a companion paper [13].

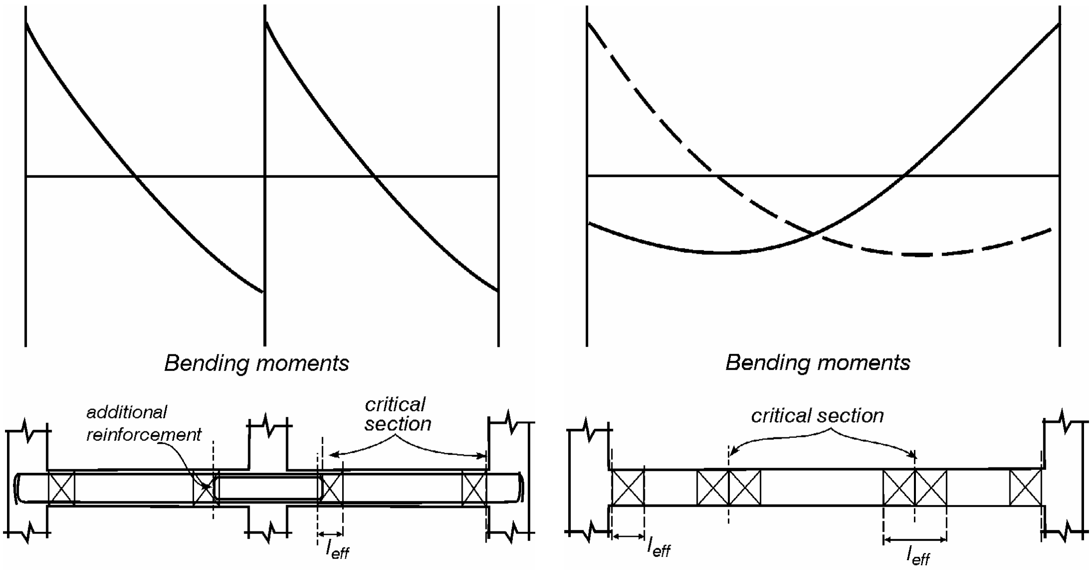
The predicted maximum curvature demand, or material strain, in a plastic region is obtained by dividing the plastic hinge rotation, θ_p, by the effective plastic hinge length, l_{eff}, and adding on the curvature, ϕ_y, associated with the first significant yield in the plastic region. Hence the maximum curvature in a plastic hinge is given by;

$$\phi_u = \frac{\theta_p}{l_{eff}} + \phi_y \qquad (3)$$

Figure 4 shows effective plastic hinge lengths for reversing and unidirectional plastic hinges in beams. For a reversing plastic hinge, or a unidirectional plastic hinge, where yielding can only occur on one side of the critical section, the effective plastic hinge length in beams or columns is taken as the smaller of;

- $h/2$,
- $0.2M/V$, but not less than $h/4$,

where M/V is the moment divided by shear at the critical section and h is the section dsepth. For walls the effective plastic hinge length is taken as the smaller of $L_w/2$ or $0.15M/V$, where L_w is the length of the wall. For unidirectional plastic hinges, which form away from the supporting member, see Figure 4 (b), yielding can develop on both sides of the critical section. In such cases the effective length of the plastic hinge may be taken conservatively as twice the corresponding value for reversing plastic hinges. In these cases the gradient of the moment diagram (equal to the shear force) is low and hence a little strain hardening causes the plastic length to extend over an appreciable distance. In practice where unidirectional plastic hinges form, it is the curvature of the plastic hinge, which is located against the face of the supporting member (generally a column), that limits the deformation that the member can sustain.



(a) Reversing plastic hinges (b) Unidirectional plastic hinges

Figure 4. Effective plastic hinge lengths for reversing and unidirectional plastic hinges

It should be noted that the term “effective plastic hinge length” is a value used to calculate a material strain. The length that defines the distance over which yielding of the reinforcement, or spalling of the concrete, may be expected to occur is described in the concrete standard [6] by the term “ductile detailing length”.

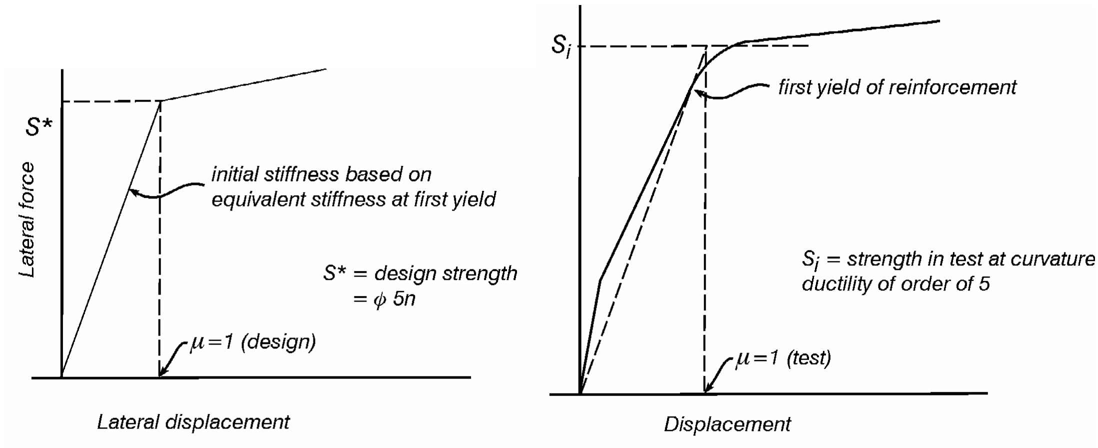
In design, the curvature in plastic hinge regions is only required to ensure that the material strains do not exceed values appropriate for the level of detailing that is used. Consequently, a number of conservative approximations may be made with more detailed calculations required only if the simplified methods indicate curvature limits are too high. Conservative short cuts, which may be made for determining required detailing levels in moment resisting frame structures include;

1. Checking curvatures only for beams in the storey sustaining the greatest inter-storey drift and using the level of detailing required for this beam for a wide group of beams;
2. Assuming the rotation of a column due to plastic deformation, θ_c, is equal to the drift in the storey with the maximum inter-storey drift;
3. Assuming the drift due to the elastic deformation of the frame is negligible.

The material strains (curvatures) calculated by the design approach outlined in this paper should be considered as an index to the conditions in a plastic hinge. As outlined in the companion paper [13] they are not true curvatures. In particular reinforcement strains assessed from these curvatures can be considerably greater than the actual values. In practice provided that seismic grade reinforcement (can sustain a strain at maximum stress equal to or greater than 10%) is used the plastic rotations should not be limited by the reinforcement.

6. Relationship of structural ductility factor obtained from tests to values used in design

Figure 5 (a) illustrates typical lateral force deflection relationships assumed in design for moment resisting frame or wall. In design the structural ductility factor (the structural ductility factor relates to the displacement ductility of the structure as a whole) is used to determine the level of lateral seismic design forces. The ductility one displacement of the structure is assumed to be equal to the lateral deflection sustained when these forces act. The initial elastic stiffness of the structure is ideally based on member properties corresponding to an equivalent stiffness when the reinforcement in the primary potential plastic regions reaches the yield point having first been subjected to several pre-yield load cycles [6]. The lateral forces sustained at a displacement ductility of 1 are represented by $S*$ in the figure. The design strengths are based on lower characteristic material strengths and strength reduction factors, which are equal to 0.85 for flexure and axial load and less for shear and other potential failure modes.



(a) Assumed in design (b) Obtained from tests

Figure 5. Lateral force-displacement relationships

Figure 5 (b) illustrates the lateral force versus displacement relationship, which could be expected, if a frame or wall, designed as outlined above, was subjected to a steadily increasing set of lateral forces. In this case the average area of reinforcement in the members is typically greater than the minimum area required in design due to the limited number of bar sizes that may be used in practice and minimum reinforcement contents that codes require to ensure that members behave in a ductile manner. In addition the reinforcement strengths are generally 10% greater than the values assumed in design. This is due to the difference in average reinforcement strength and lower characteristic strength used in design calculations. A consequence of this is the average strength of the members is at least 1.1/0.85 (1.29) times the design strength assuming the exact minimum areas of reinforcement required in the design are used. Allowing for an average reinforcement content of 10% in excess of the minimum amount of reinforcement increases the difference between the design and test strengths to the order of 1.4. In practice all plastic hinges do not form simultaneously and not all reinforcement in a wall reaches its yield point at the same time, hence there is some rounding of the load deflection relationship between the initial elastic response and the response of the structure when all the plastic hinges have formed and strain hardening develops. From this it can be seen a structural ductility factor of 6 in practice corresponds on average to displacement ductility of the order of 4 to 4.5. It is important that this difference is maintained to ensure that the designed structure can meet seismic performance required for the ultimate limit state with a high level of certainty. However, in interpreting experimental results it is important that the difference in design and experimental ductility values is understood.

7. Conclusions

1 The magnitude of deformation that a plastic region can sustain depends on the level of detailing. Previously the structural ductility factor was the principle factor used to define the required level of detailing.

2 It is shown that the structural ductility gives a poor indication of the required level of deformation in plastic regions. To improve the reliability of seismic performance of structures the Earthquake Actions Standard, NZS 1170.5:2004, introduced the requirement that the detailing of plastic regions be determined on the basis of calculated material strains they sustain when subjected to the deformations defined in the ultimate limit state.

3 The basis of calculating a plastic hinge rotation and hence material strain (curvatures in plastic hinges) is outlined together with a number of approximations which may be used to simplify design.

4 The relationship between structural ductility factors used in design and values obtained from tests is outlined. It is shown that due to the difference in design strength and the average strength of a member a design structural ductility factor is typically 1.35 times the value found from a test of the designed structure.

Acknowledgements

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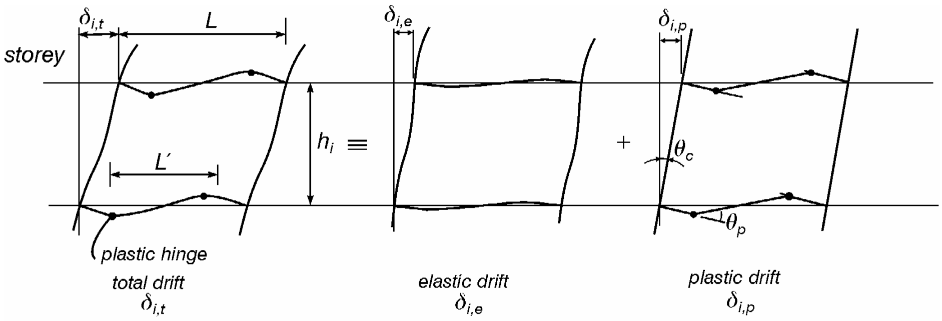
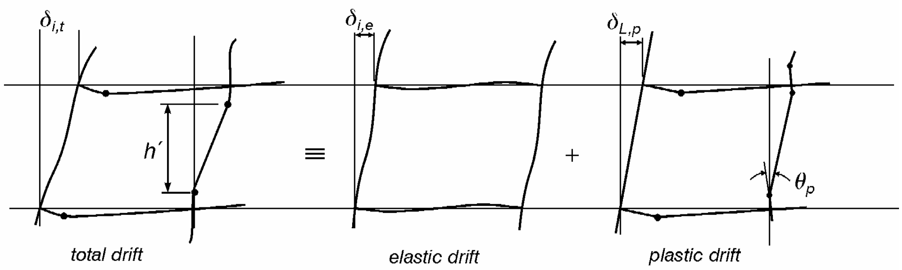
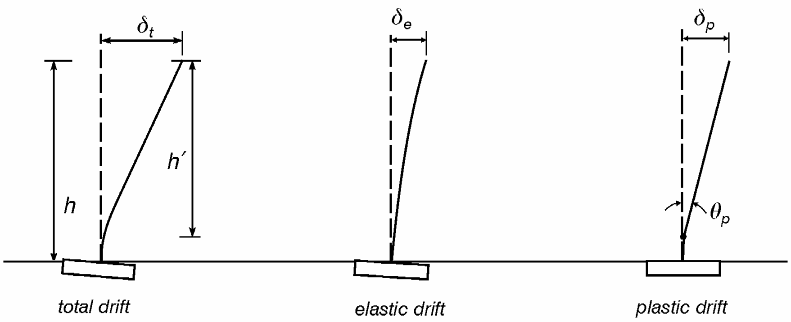


(a) Plastic hinge rotation in beams



(b) Plastic hinge rotation in a column



(c) Plastic hinge rotation in a wall