

## Lateral Restraint of Yielding Regions in Columns and Beams in Multi-Storey Buildings

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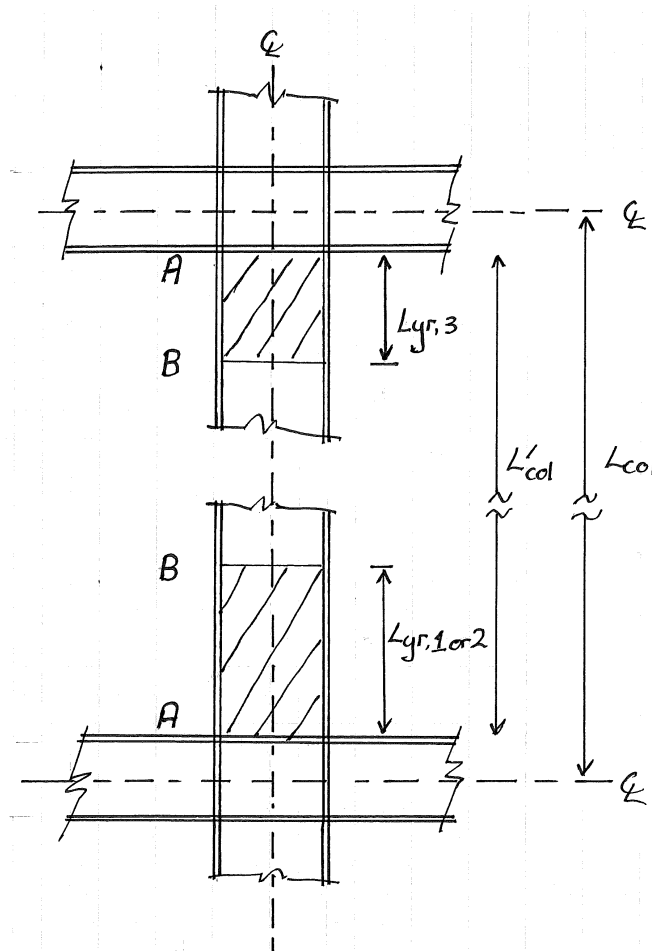
### Key Words

Steel columns and beams, earthquake, yielding regions, lateral restraint

### Introduction

This article provides general guidance on applying the lateral restraint provisions for yielding regions given in Clause 12.6.2 of NZS3404 (SNZ, 2007) to columns and beams of multi-storey buildings. It includes simplification of those provisions which can be made when designing these types of member.

The article is to be read in conjunction with Figure 1.



**Figure 1 Application of Lateral Restraint Provisions to the Yielding Regions of a Column**

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## Philosophy of NZS 3404

This is explained in detail in the commentary clause C12.6.2. The following is a summary of the key points as applied to columns and beams of multi-storey buildings:

1. Each category 1, 2 or 3 member is divided into segments. Each segment either contains one or more yielding regions or does not contain a yielding region.
2. A column typically carries load from the structural system at the top and bottom of the column, as shown in Figure 1. In that instance, the member comprises one segment which contains two yielding regions, shown as lengths AB.
3. Any segment containing a yielding region must have full lateral restraint, such that  $\phi M_{bx} = \phi M_{sx}$ . In making this calculation for a member subject to load combinations including earthquake loading, the member moment gradient used along the segment must not be taken as greater than triangular; for a column this means maximum moment at one end and no moment at the other end. This means the moment magnifier,  $\alpha_m \leq 1.75$  in all instances and  $\alpha_m = 1.75$  when no transverse load is applied to the column between the ends.
4. When the yielding region is loaded into the inelastic range, the Young's modulus (E) for the material within that region decreases, meaning that the yielding region requires more closely spaced restraints than do the elastic lengths of the member. This is achieved by considering the yielding region as a segment within a segment, with the ends of the yielding region taken as being restrained when applying Clause 12.6.2.3. For example in Figure 1, the ends B of the yielding regions are taken as points of restraint, where this is provided by the continuing length of the member that does not yield.
5. At least one full restraint of the type shown in Table 12.6.3 must be applied within the length of yielding region. In Figure 1 this is provided at ends A. The section restraint at points A is F to Clause 5.4.2.1, provided by a combination of the column continuity, incoming beam(s) and floor slab. A connection detail and description is not given directly for this case in HERA Report R4-92 (Clifton, 1997), however it can be seen that the cross section restraint is comparable to that of connection detail 29 and better than that of connection detail 28.
6. Other actual restraints must be added as required to satisfy Clause 12.6.2.3.1. For a typical column, those provisions will be satisfied with only the restraint at ends A, as the slenderness ratio of the yielding region,  $L_{yr}/r_y \leq k_{yr} \sqrt{(250/f_y)}$ , where  $k_{yr}$  is obtained from Table 12.6.3 and  $r_y$  = the minor principal y-axis radius of gyration for the member containing the yielding regions.
7. Where the member comprises more than one segment and some segments don't contain a yielding region, these segments must have sufficient moment capacity to allow the yielding regions to develop.

When applying these provisions, there is a complex method for determining the maximum design bending moment to be considered for restraint,  $M_{res,max}^*$ . For columns and beams in multi-storey buildings, this can be simplified, without violating the requirements of NZS 3404, and the guidance is given below.

## Application of Philosophy to Columns of Multi-Storey Buildings

1. Determine where the yielding regions may form. This will be at the ends of the columns, from the face of the incoming beams in the plane of the frame extending into the column span. Two yielding regions are shown in Figure 1, the top one showing the typical length of yielding region for a category 3 column and the bottom one showing the typical length of yielding region for a category 1 or 2 column.
2. For each yielding region, take  $M_{res,max}^*$  as the maximum moment from the analysis/design procedure. There is no need to apply Clause 12.6.1.3 as the values of  $L_{yr}$  calculated will be the same as if the full procedure had been applied.
3. Because the maximum design bending moment gradient that can be used in the column of a seismic frame is triangular, then  $L_{yr}$  from Equation 12.6.2 can be more simply expressed as:  
 $L_{yr} = L'_{col}(1 - C_1)$ , where  $L'_{col}$  = the clear length of the column and  $C_1$  = the coefficient from Clause 12.6.2.1. This means for a column with  $N^* / \phi N_s > 0.15$ ,  $L_{yr} = 0.25L'_{col}$ . Note that the minimum length requirement of Table 12.6.1 also applies.
4. The column member is equivalent to the column segment must have full lateral restraint.
5. Clause 12.6.2.3 is then applied to, hopefully, show that no further restraints are needed to the yielding regions as these will be difficult to apply. If it shows they are needed, then a fly-brace type arrangement would be used which would be difficult to provide especially to an isolated column.

### Example

Consider a category 3 column that is part of a moment resisting framed seismic-resisting system comprising a 610UB101 member of clear column length 3.5m. The maximum design bending moment is at one end of the column;  $M^* = 780 \text{ kNm}$  and the ratio of  $N^* / \phi N_s = 0.20$ .

$L_{ez}$  for clause 5.6.3.1 =  $k_t k_l k_r L = 3.5\text{m}$

$k_t = 1.0$  for segment length  $L_{col}$  having F section restraint at each end (this is typical)

$k_l = 1.0$  as the loads are applied at the segment end for end restraint arrangement FF

$k_r = 1.0$  as the yielding region will preclude any rotational restraint in plan of the critical flange through the yielding region

$\phi M_{bx} = \alpha_m \phi M_{bx, \alpha_m=1.0} = 1.75 * 585 = 1023 \leq \phi M_{sx} = 782 \text{ kNm}$ , hence section moment capacity governs and the segment has full lateral restraint as required.

Checking the yielding region adequacy:

$L_{yr}$  from Table 12.6.1 =  $1.0 d_{col} = 602 \text{ mm}$

$L_{yr}/r_y = 602/47.5 = 12.7$ ;  $r_y$  is from (ASI, 1999)

$k_{yr} \sqrt{(250/f_y)} = 60 \sqrt{(250/300)} = 55$

Check  $L_{yr}/r_y \leq k_{yr} \sqrt{(250/f_y)}$  ✓ OK; only 1 physical restraint is required to the yielding region; this is achieved by the F section restraint at the beams; points A in Figure 1.

Type of cross section requirement from Table 12.6.3 is met

⇒ all lateral restraint provisions for the yielding region are met

### Application of Philosophy to Beams of Multi-Storey Buildings

Seismic-resisting system beams are subjected to a combination of seismic and gravity actions. These beams typically support a floor slab and are also typically composite with the floor slab, except at the ends where the shear studs are terminated  $1.5d_{beam}$  from the column face to ensure the yielding region hinging is reversing in nature as required by Clause 12.10.2.2 of NZS 3404 and (Feeney and Clifton, 1995/2000). This termination of the shear studs is made to force yielding regions to occur at the ends of the beams; this detail is shown in Figure C6(a) of NZS 1170.5 (SNZ, 2004).

Along these beams the bending moment varies from negative (top of slab in tension) to positive (top of slab in compression). At the negative moment end of the beam, where the direction of rotation from seismic and gravity actions combine, the moment will be negative and a maximum at the column face. At the positive moment end of the beam, where the direction of member rotation caused by seismic induced loading opposes the direction of member rotation caused by gravity loading, the beam bending moment will typically be positive at the face of the column. In such applications, the key points in regard to the lateral restraint provisions are as follows:

1. The yielding regions will form at the ends of the beams.
2. For each yielding region, take  $M_{res, max}^*$  as the maximum moment from the analysis/design procedure and  $M_{res}^*$  as the moment from the design/analysis procedure. There is no need to apply Clause 12.6.1.3.
3. When applying Equation 12.6.2,  $L_{yr}$  = the length of member for which  $M_{res}^* > 0.85 M_{res, max}^*$ . Note that the minimum length requirement of Table 12.6.1 also applies.
4. For the negative moment end of the beam, the bottom flange is the critical flange. In this instance, the segment effective length from Clause 5.6.3.1,  $L_{ez}$ , should be taken from the face of the column to the point of contraflexure, when the beam is composite with the floor slab away from the column face, and the section restraint is taken as F at the column face and P at the point of contraflexure. The bending moment gradient over the segment can be taken as triangular, giving  $\alpha_m = 1.75$ .  
When the beam is not composite with the floor slab away from the column face, the segment is taken as from the face of the column to the first point of cross section restraint past the point of contraflexure into the positive moment region, the system restraint is F at both ends of the segment and the effective length is calculated using Clause 5.6.3.  
If additional restraints to the yielding region are required from Clause 12.6.2.3 these can be fly-braces from the slab/top of beam down to the bottom flange. It is unlikely that they will be needed.
5. For the positive moment end of the beam, the top flange is the critical flange and each point of attachment to the slab is a point of F or L section restraint. In this instance, each segment is the distance between points of cross section restraint, which is very short, and a check on lateral restraint is; (a) easily met and (b) unlikely to be explicitly required.

## References

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