

This paper is the result of deliberations of the Society's discussion group on
SEISMIC DESIGN OF DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES

SECTION G

COLUMNS - EVALUATION OF ACTIONS

T. Paulay*

G1.0 SCOPE

This section presents a procedure for the evaluation of the magnitudes of the axial loads, bending moments and shear forces to be used in the determination of the required strength properties at the critical sections of columns of regular framed buildings. The procedure is intended to satisfy the requirements of NZS 4203:1976. It uses the results of the equivalent static force analysis of that code of practice**.

G2.0 CAPACITY REDUCTION FACTORS

The appropriate capacity reduction factors for various load types are incorporated in the following sections so that, whenever earthquake loading governs the design, column sectional properties should be based on ideal strength.

G3.0 BEAM OVERSTRENGTH FACTORS

The beam overstrength factor, ψ_o , is the ratio of the sum of the flexural overstrengths developed by the beams, as detailed, and the sum of the flexural strengths required by the code specified lateral loading, both sets of values being related to the centre line of a column. The evaluation of the flexural overstrength needs to take into account all reinforcement that may participate at a potential plastic hinge, located at or near a column face, as a result of lateral inelastic displacements of a frame, separately in each direction, with a developed steel strength of $1.25f_y$. The beam overstrength factor needs to be determined at each floor and at each column at the centreline of the beam-column joints for both directions of the lateral displacements. The overstrength factor, ψ_o , appropriate at the ground floor or foundation level, where column hinging is to be expected should not be less than 1.4. Irrespective of the beam strength the value of ψ_o appropriate at roof level, where column hinging is acceptable, need not be more than 1.1.

G4.0 DYNAMIC MAGNIFICATION FACTOR

G4.1 The dynamic magnification factor, ω , allows for the departure of the column bending moment pattern in any one storey from that derived with an elastic analysis and, where applicable, for the increased moment demand on a column section due to concurrent lateral loading in both principal directions of the structure. The magnitude of ω is

(a) For columns of one-way frames -

* Professor of Civil Engineering, University of Canterbury.

** Subsequent reference to code loading imply the equivalent static forces according to 3.4 of NZS 4203:1976.

$$\omega = 0.6T_1 + 0.85 \quad (G-1)$$

but not less than 1.2 nor more than 1.8.

(b) For columns of two-way frames -

$$\omega = 0.5T_1 + 1.10 \quad (G-2)$$

but not less than 1.5 nor more than 1.9, where T_1 is the period of the building in its first mode of vibration in seconds, as defined by 3.4.4. of NZS 4203:1976.

G4.2 At the roof and at ground floor or foundation level, where column hinging is expected, the value of ω may be reduced to:

- (a) 1.0 for columns of one-way frames.
(b) 1.1 for columns of two-way frames.

G4.3 In the lower storeys in which, according to the analysis for the code loading, the magnitude of the column moment at the top of a storey is less than one half of that at the bottom of that storey, the value of ω at the first floor may be taken as 1.2 for one-way frames and 1.5 for two-way frames, and then linearly increased to the value obtained from Eq. (G-1) or Eq. (G-2) as appropriate at the floor below which the ratio of the same moments is more than one half.

G4.4 At the floor immediately below the roof, the value of ω may be taken as 1.2 for one-way frames and 1.5 for two-way frames.

G5.0 DESIGN AXIAL FORCES

The axial forces induced at any level by earthquake loading only, and used together with the moments determined according to G.6 and the appropriately factored gravity forces, to determine the column section strength, should not be less than

$$P_{eq} = \lambda \Sigma V_{oe} \quad (G-3)$$

where ΣV_{oe} is the sum of the earthquake induced oe beam shear forces at all floors above the level considered, developed at all sides of the column and taking into account the beam overstrengths and the appropriate sense of the forces. The value of λ is to be determined as follows:

(a) When $P_e < 0.4A_g f'_c$

$$\lambda = 1 - (0.35\omega - 0.2) \frac{n}{20} \quad (G-4)$$

(b) When $P_e > 0.4A_g f'_c$

$$\lambda = \frac{1.67(0.35\omega - 0.5) \left(1 - \frac{P_g}{f'_c A_g}\right) + 0.3 - \frac{20}{n}}{1.67(0.35\omega - 0.5) \frac{\Sigma V_{oe}}{f'_c A_g} - \frac{20}{n}} \quad (G-5)$$

(c) In no case need the value of λ be taken as more than

$$\lambda = 1 - 0.015n \geq 0.7 \quad (G-6)$$

where n is the number of floors above the column section considered, and in the above equations its value is not to be taken more than 20. The dynamic magnification factor, ω , shall be as appropriate to the floor considered, but in no case should a value less than 1.4 be used in Eq. (G-4) and Eq. (G-5). P_g is the appropriately factored gravity load on the column.

G6.0 DESIGN MOMENTS

G6.1 The design moment M_{col} to be used, together with the appropriate axial load, for the determination of the properties of the critical column section, separately in each of the two principal directions of the structure, should not be less than

$$M_{col} = \psi_o \omega M_{code} - 0.3h_b V_{col} \quad (G-7)$$

where V_{col} is the column design shear force as given in G.7 and h_b is the overall depth of the beam.

G6.2 When the total design axial compression P_e on a column does not exceed $0.1f_c A_g$ the design column moment may be reduced as follows:

$$M_{col, reduced} = \left[\left(20 \frac{P_e}{f_c A_g} + 1 \right) (\omega - 1) + 3 \right] \frac{M_{col}}{3\omega}$$

where P_e is to be taken as negative if causing tension and provided that:

(a) The value of $\frac{P_e}{f_c A_g}$ is not to be taken less than -0.15 nor less than $-0.5\rho_t \frac{f_y}{f_c}$.

(b) The moment reduction for any one column does not exceed 70%.

(c) The moment reduction used for columns of a bent is not more than 10% of the sum of the design moments for all columns of that bent, taken at the same level.

G7.0 DESIGN SHEAR FORCES

G7.1 The design shear force, V_{col} , across a column should be evaluated from the column end moments as follows:

(a) In the upper storey columns the lesser of

$$V_{col} = \frac{1.8}{l_c} \psi_o \omega M_{code, max} \quad (G-9)$$

or

$$V_{col} = \frac{1.2}{l_c} (\psi_o \omega M_{code, max} + \psi_o M_{code, min}) \quad (G-10)$$

should be used, where $M_{code, max}$ and $M_{code, min}$ are the larger and the smaller column end moments in the storey at the level of the beam centrelines, determined from the code required lateral loading, and l_c is the floor height.

(b) In first storey columns the lesser of

$$V_{col} = \frac{1.8}{l_c} M_{o, col} \quad (G-11)$$

or

$$V_{col} = \frac{1.2}{l_c} (M_{o, col} + \psi_o M_{code, min}) \quad (G-12)$$

should be used, where $M_{o, col}$ is the flexural overstrength of the column, as built, at its restrained base with the design axial load acting.

G7.2 In no case shall the design shear force be less than:

(a) In columns of one-way frames

$$V_{col, min} = 1.2\psi_o \frac{M_{col, reduced}}{M_{col}} V_{code} \quad (G-13)$$

(b) In columns of two-way frames

$$V_{col, min} = 1.8\psi_o \frac{M_{col, reduced}}{M_{col}} V_{code} \quad (G-14)$$

COMMENTARY

CG1.0 SCOPE

To be consistent with the intent of capacity design philosophy, it is necessary to modify the actions that are derived from an elastic analysis of a rigid jointed frame which is subjected to the equivalent lateral static forces specified by NZS 4203. The procedure intends to provide a high degree of protection against both premature yielding in columns, which could possibly occur before any beam has yielded, and the possibility of the development of storey failure mechanisms during the most severe seismic motions in regular building frames. Moreover, the procedure is likely to lead to reduced ductility demand at column hinges that may inevitably form at one or the other ends of a column. It does not apply to frames in which column hinges are intended to form the primary energy dissipating mechanism, or in which column displacements are controlled by shear walls or when columns act simply as props.

The procedure should be restricted to frames in which the relationship between the relative stiffness, k , of the columns and that of the beams framing into them is such that

$$\frac{\Sigma k_{upper beams} + \Sigma k_{lower beams}}{2k_{column}} > 0.2$$

where $k = \frac{I}{l}$.

CG2.0 CAPACITY REDUCTION FACTORS

The usual column capacity reduction factors, applicable when the dependable capacities for flexure, shear or axial load are to be determined, should be taken as unity when earthquake loading governs the design. The procedure requires the ideal strengths of the column sections to be determined. The procedure appropriately magnifies the actions that are derived from the code loading so that a direct comparison

between ideal column strength capacities and minimum strength demands, resulting from the code load, can be made. When, under adverse combined load conditions, the ideal moment of resistance of a column section is exceeded, yielding of the section and possibly the formation of a plastic hinge is to be expected.

CG3.0 BEAM OVERSTRENGTH FACTORS

One of the basic requirements of the design of ductile frames is that the possibility of column hinging during inelastic displacements of a frame be minimised. To this end the maximum real load input from the beams into the columns needs to be determined. This is achieved with the use of the beam overstrength factor, ψ_0 , at each column centreline at each floor and in each direction. Its value must be based on the properties of the critical beam sections as built. Plastic hinges in a beam may occur at the faces of the columns or somewhere else within the clear span of the beam. The beam flexural overstrength input with reference to the centreline of an interior column must be determined from the appropriate beam moment pattern, consistent with the direction of the lateral load, with two plastic hinges developing the flexural overstrengths of the relevant sections in each of the two adjacent beams. At an exterior column only one beam is considered. The reference to member centrelines is necessary to avoid additional calculations when checking joint equilibrium.

The overstrength factor enables the total moment input from beams into the columns to be considered, and therefore gravity load moments for columns need not be considered in the design.

At the ground floor or at foundation level, where the full column flexural capacity could be developed, no beams may frame into the column. Consequently a ψ_0 factor applicable to that locality would appear to be irrelevant. It is considered that the potential lateral load strength of 1st storey columns at their base should be comparable with the strength of the remainder of the frame. Consequently the value of ψ_0 , applicable at a column base, should not be less than 1.4.

In the evaluation of the flexural overstrength of a potential beam hinge the beam capacity reduction factor must be taken as unity and the strength of the reinforcement be based on $1.25 \times 275 = 344$ MPa. The ideal value of ψ_0 is therefore $1.25/0.9 = 1.39$. However, when gravity load dominates the required beam flexural strength, this value will be exceeded. When beam moment redistribution is applied the value of ψ_0 may be locally less than 1.39.

At roof level generally gravity load will govern the design of beams. At such a column-beam joint it is not necessary to increase the column capacity to match or exceed the beam flexural capacity because column hinging at this level is acceptable. The value of $\psi_0 = 1.1$ is chosen to compensate for the capacity reduction factor of $\phi = 0.9$, that would normally be used in this situation. The adequacy of such columns for gravity loads only should be checked.

CG4.0 DYNAMIC MAGNIFICATION FACTOR

CG4.1 The equivalent static load specified by the code is considered to lead to satisfactory distribution of potential beam strengths throughout the frame. To give columns a higher degree of protection against premature yielding approximate allowance must be made for the fact that the moment pattern along any column in a storey may vary considerably from that obtained with code static load distribution due to dynamic effects, for example the higher mode dynamic responses of the structure. This is demonstrated for example by the movement of the column point of contraflexure away from its fixed location, determined in the static load elastic analysis. The higher mode responses become more prominent as the fundamental period of vibration of the structure increases and Eqs. (G-1) and (G-2) recognise this.

The lower limit of $\omega = 1.2$ has been set to minimise the possibility of a storey mechanism forming in columns that are part of a one-way frame, in which beams frame into a column in one plane only. The columns of two-way frames, in which the lateral load in one direction is resisted entirely by shear walls, may be designed similarly.

The value of ω is larger for columns that may be subjected to biaxial bending when beams, that frame into the column in two directions at right angles, could simultaneously develop plastic hinges. The minimum value of ω is 1.5 for such columns. The likely concurrency of very large orthogonal moments at any one column section is, however, considered to diminish with lengthening of the fundamental period of the building. A comparison of Eqs. (G-1) and Eq. (G-2) is made in Fig. G.1.

A column section designed for unidirectional attack, separately in each of the two principal directions, in accordance with Eq. (G-2), is considered to possess adequate flexural strength to resist various possible combinations of biaxial flexural demands, satisfying the requirements of NZS 4203:1976.

CG4.2 Higher mode responses are not affecting the required strength of the bottom storey columns where base restraint exists. At this section columns hinging is to be expected and a column is to be detailed accordingly. To ensure that the flexural capacity of column sections in two-way frames is adequate to sustain at any angle an attack of code load intensity, the value of ω needs to be 1.1. Similar considerations apply to column sections at roof level.

CG4.3 In many cases in the lower storeys of frames, when columns are stiff relative to the beams, the column moment pattern obtained from the analysis for code loading may be such that no point of contraflexure appears in several storeys. This indicates increased cantilever action of the column which, at these levels, is not likely to be affected significantly by the higher modes of dynamic response. Moreover, the critical column moments in such cases are usually larger than the total beam moment input at the floor in question. Hence it is required that the minimum value of ω (i.e. 1.2 or 1.5)

be taken at the first floor and that ω be then linearly increased to its full value, corresponding with the period T_1 of the structure, at the floor below which the point of contraflexure, as indicated by analysis, is located in the middle third of the column height. The interpretation of these requirements is shown in Fig. G.2 which illustrates the moment pattern for the lower four storeys of such a column in a two-way frame.

The column reinforcement provided at ground floor level should not be less than that required at the first floor.

CG4.4 At the top storey of multistorey frames the development of storey mechanisms is considered to be acceptable. Consequently there is no need for a high degree of protection against yielding when column moments are magnified. Correspondingly the minimum values of the dynamic magnification factor at the floor immediately below roof level have been stipulated. The interpretation of these requirements for a two-way frame is illustrated in Fig. G.4.

CG5.0 DESIGN AXIAL FORCES

The derivation of the earthquake induced axial forces is based on the assumption that, with increasing number of storeys, the relative number of beam hinges at which the flexural overstrength may be developed is reduced. In Eq. (G-6) 1.5% reduction per floor in the maximum feasible earthquake induced column axial load has been allowed, up to a maximum of 30% in a frame of 20 storeys or higher. The maximum induced axial forces will not coincide with the maximum moments that are magnified due to higher mode responses. Consequently Eq. (G-4) allows for additional axial load reductions provided that the value of ω is larger than 1.4. The maximum reduction, when $\omega = 1.9$, is an additional 0.83% per floor. The ratio of the design axial earthquake force on the column and the sum of the maximum earthquake induced beam shear forces at overstrengths, λ , is shown in Fig. G.5.

The value of P_e is to be combined with gravity induced axial forces as follows: $D + 1.3L_R + E$ when causing compression and $0.9D - E$ when causing tension.

It is possible that, while a frame responds primarily in its first mode, the axial force then induced in a column is larger than that predicted by Eq. (G-4). However, such axial force is not likely to coexist with greatly magnified column moments. From a moment-axial load interaction relationship for a column, it is evident that, when the tension steel yields at the critical column section, any reduction of the applied moment will allow the simultaneous resistance of greatly increased axial compression. This may be assumed to be the case when $P_e < 0.4A_g f'_c$. In a compression dominated column section, however, when $P_e > 0.4A_g f'_c$, there is less reserve strength available to accommodate increased axial compression, while the moment demand on the section is reduced. To protect such compression dominated columns against possible overload by axial compression, Eq. (G-5) is to be used. Accordingly the value of λ changes from that given in Eq. (G-4) to that

given by Eq. (G-6) as the total design axial compression load increases from $0.4f'_c A_g$ to $1.0f'_c A_g$. Eq. (G-6) will give a conservative estimate of the axial load for all cases.

It should be noted that in summing the shear forces at the column faces all beams in both directions need to be considered. In general this procedure will not significantly affect the axial load on interior columns. However, for outer columns and corner columns in particular, significant increase in axial load will result and need to be considered as a consequence of a skew earthquake attack. When the dynamic magnifications in the two principal directions of the structure are different, the larger of the ω values, relevant to the level under consideration, may be taken when using Eq. (G-4) or Eq. (G-5) to evaluate the axial load due to concurrent actions.

CG6.0 DESIGN MOMENTS

CG6.1 The design column moments at the intersection of the reference axes of beams and columns are obtained by multiplying at each end of a column the corresponding moments, obtained for the code loading, by the product of ψ_0 and ω appropriate to that floor. The moment magnification applies to the end moments only and not to the moment pattern as the end design moments so obtained are not expected to occur simultaneously. The critical column section is assumed to be at the top or the soffit of the beams and accordingly the centreline moment $\psi_0 \omega M_{code}$ may be reduced. In this only 50% of the moment gradient, used for the determination of the column shear, should be considered. Hence the centreline column moment, as shown in Fig. G.3, could be reduced by $0.6 \times 0.5 h_b V_{col}$, where h_b is the depth of the beam and V_{col} is the design shear as determined in G.7.

CG6.2 When a column is subjected to small axial compression or to net tension, column yielding is more acceptable. For such cases the design moment may be reduced according to Eq. (G-8). The larger the axial tension load and the value of the dynamic magnification factor ω , given by Eq. (G-1) or Eq. (G-2), the larger the moment reduction allowed. The values of Eq. (G-8) are shown for the entire range in Fig. G-6. The requirements of G6.2 (a) and (b) are intended to ensure that the reduction of the magnified moments is not excessive.

The reduction of column design moments in a bent may result in loss of lateral load carrying capacity in that bent. Usually there will be only one column in a bent that will qualify for reduction of design moment. Because of the possible strength loss, when beam hinge overstrength capacities are being developed, according to G6.2 (c) the moment reduction allowed should not exceed 10% of the sum of the column design moments, taken before the application of moment reduction at the same level for all columns of the affected bent.

CG7.0 DESIGN SHEAR FORCES

CG7.1 The shear forces are estimated from a probable but critical moment gradient along the column. For upper storey columns it is assumed that when the maximum design

moment is developed at one end, one half of that intensity is simultaneously developed at the other end. When a capacity reduction factor of 0.85 is introduced, the ideal column shear strength required will be a little less than that given by Eq. (G-9). The shear force is considered to be unnecessarily overestimated by this equation in cases where the point of contraflexure along the column is situated substantially away from the midheight. For such situations Eq. (G-10) is intended to give the design shear. Eq. (G-10) will govern the design shear whenever the ratio of column end moments is such that $\psi_0 M_{\text{code,min}} / \psi_0 M_{\text{code,max}} < 0.5\omega$.

At the base of a first storey column, hinging with considerable plastic rotations must be expected. For this reason the value of $\psi_0 M_{\text{code,max}}$ for these columns is replaced by $M_{0,\text{col}}$, the flexural overstrength capacity of the section, allowing for the axial load on the column that is consistent with the loading considered.

CG7.2 When the analysis for code loading indicates that points of contraflexure are close to column midheight and the relevant ω factor is small, Eq. (G-13) or Eq. (G-14) will need to be applied to ensure that the shear force, resulting from code loading and magnified by the overstrength factor, ψ_0 , can be resisted. The term $M_{\text{col, reduced}} / M_{\text{col}}$, obtained from Eq. (G-8), allows for a reduction of the design shear in columns which are subjected to small axial compression or to net axial tension, and in which yielding at a low shear load is permitted. In all other columns the above ratio will be unity.

EXAMPLE OF APPLICATION

To illustrate the application of some of the steps involved in evaluating column actions, consider an isolated two-bay subassembly of a multistorey frame, shown in Fig. G.7(a). The assembly represents a typical upper floor of a one-way frame with a fundamental period of $T_1 = 0.95$ secs.

The gravity load, consisting of the load combination $D + 1.3L_R$, results in the moment pattern, M^G , shown in Fig. G.7(b), where the more important values in moment units are also recorded. The earthquake loading and the resulting bending moments for the columns and the beams are shown for the same subassembly in Figs. G.7(c) and G.7(d). The arrows indicate the direction of the static lateral loading on the frame that is responsible for the particular moment pattern.

The moments due to gravity and earthquake loading that were derived from an elastic analysis are subsequently superimposed and the cases for each of the two directions of lateral loading are shown in Fig. G.8. It is seen that the peak values, resulting from the elastic analysis, can be considerably reduced if the beam and column moments are suitably redistributed. The details of redistribution for this particular frame, as indicated by the four steps, were reported previously⁽¹⁾.

It will be assumed that these moment diagrams were used in determining the

flexural reinforcement for the beam and that every effort was made not to provide excess reinforcement. A comparison of the negative and positive moments to be resisted and the dependable flexural capacities provided by the chosen top and bottom flexural reinforcement is made in Table G.I. For example the bottom steel in span B-C could not be sufficiently curtailed near the centre column so that a dependable moment of resistance of 61 instead of 48 units, with reference to the column centre line resulted, (i.e. 27% excess).

The flexural overcapacities of the relevant sections are simply $1.25/0.9 = 1.39$ times the dependable capacities actually provided. These are also shown in Table G.I.

The beam overstrength factor ψ_0 , as defined in Section G.3, can now be readily derived. For example the overcapacity at Column A is 140 units in both directions. As Fig. G.7(d) shows the flexural strength required by the code specified lateral loading is only 100 units. Hence

$$\psi_0^{\vec{A}} = \psi_0^{\overleftarrow{A}} = 140/100 = 1.4.$$

The sum of the required flexural strength at the centre column, associated only with the lateral load acting from left to right, is from Fig. G.7(d) $110 + 80 = 190$ units. The sum of the corresponding flexural overstrengths provided is however from Table G.I $170 + 85 = 255$ units.

$$\text{Hence } \psi_0^{\vec{B}} = 255/190 = 1.34$$

$$\text{Similarly } \psi_0^{\overleftarrow{B}} = (170 + 125)/(110 + 80) = 1.55$$

$$\psi_0^{\vec{C}} = 100/80 = 1.25$$

$$\psi_0^{\overleftarrow{C}} = 140/80 = 1.75$$

The total required earthquake moment input from beams into columns is from G.7(d)

$$\Sigma \Delta \vec{M}_0^E = 100 + (110 + 80) + 80 = 370 \text{ units} = \Sigma \Delta \overleftarrow{M}_0^E$$

The corresponding overstrength moment input from the beams, as finally detailed is, however, in one direction

$$\begin{aligned} \Sigma \vec{M}_0^E &= \Sigma \psi_0 \Delta \vec{M}_0^E = 1.4 \times 100 + 1.34(110 + 80) \\ &+ 1.75 \times 80 = 534.6 \text{ units} \end{aligned}$$

and in the other direction

$$\begin{aligned} \Sigma \overleftarrow{M}_0^E &= 1.4 \times 100 + 1.55(110 + 80) + 1.25 \times 80 \\ &= 534.5 \text{ units,} \end{aligned}$$

i.e. 1.44 times that required for code loading.

The dynamic magnification factor, ω , for a period of $T_1 = 0.95$ seconds is from Eq. (G-1)

$$\omega = 0.6 \times 0.95 + 0.85 = 1.42 > 1.2$$

Hence the design moments for the three columns, with respect to the beam centre lines are from Fig. G.7(c) as follows:

90

$$\vec{M}_{\text{col,A}} = 1.42 \times 1.40 \times 58 = 115 \text{ units}$$

$$\overleftarrow{M}_{\text{col,A}} = 1.42 \times 1.40 \times 58 = 115 \text{ units}$$

$$\vec{M}_{\text{col,B}} = 1.42 \times 1.34 \times 113 = 215 \text{ units}$$

$$\overleftarrow{M}_{\text{col,B}} = 1.42 \times 1.55 \times 113 = 249 \text{ units}$$

$$\vec{M}_{\text{col,C}} = 1.42 \times 1.25 \times 46 = 82 \text{ units}$$

$$\overleftarrow{M}_{\text{col,C}} = 1.42 \times 1.75 \times 46 = 114 \text{ units}$$

These column moments would then be further reduced in accordance with Eq. (G-7) because the critical section is not at the beam centre line.

REFERENCE

- (1) Paulay, T., "Moment Redistribution in Continuous Beam of Earthquake Resistant Multistorey Reinforced Concrete Frames", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 4, Dec. 1976, pp.205-212.

TABLE G.I

A COMPARISON ON MOMENT REQUIREMENTS AND
FLEXURAL STRENGTHS PROVIDED

At	Sense of Moment	Moments		
		Required for code load	Dependable provided	Overcapacity provided
A	Negative	102	101	140
	Positive	98	101	140
BA	Negative	122	122	170
	Positive	88	90	125
BC	Negative	122	122	170
	Positive	48	61	85
C	Negative	102	101	140
	Positive	58	72	100

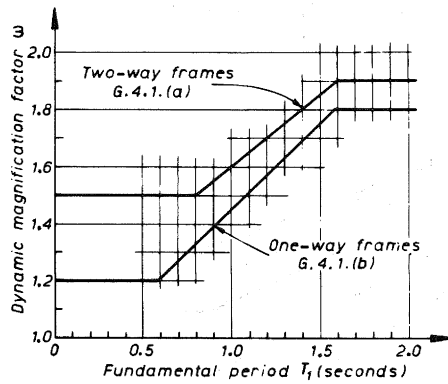


FIGURE G.1: THE DYNAMIC MAGNIFICATION FACTOR

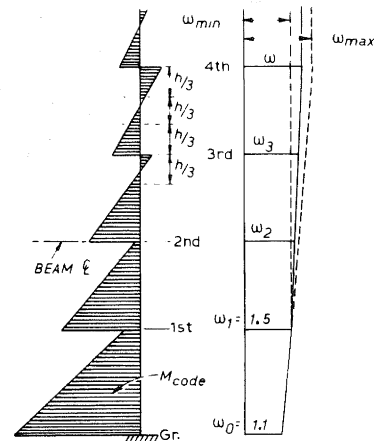


FIGURE G.2: THE VARIATION OF ω WHEN THE POINT OF CONTRA-FLEXURE IS OUTSIDE THE MIDDLE THIRD OF COLUMNS IN A TWO-WAY FRAME.

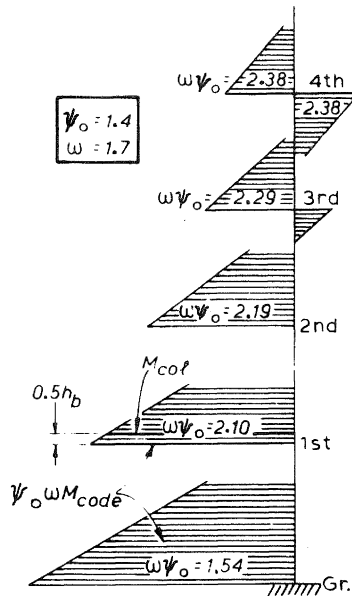


FIGURE G.3:
DESIGN MOMENTS
FOR THE COLUMN
SHOWN IN FIG. G.2.

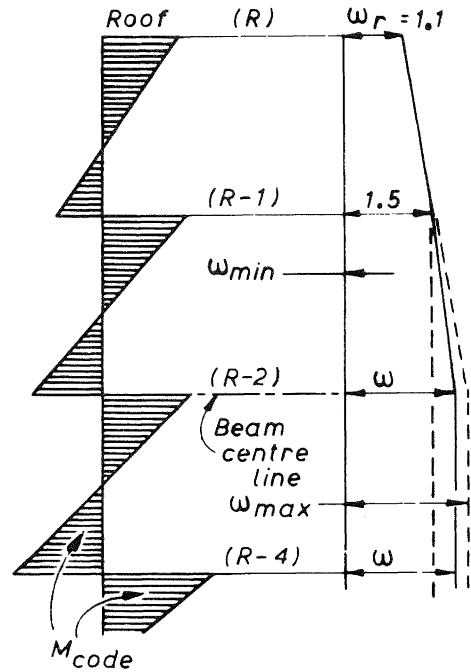


FIGURE G.4: THE INTERPOLATION
OF THE DYNAMIC MAGNIFICATION
FACTOR AT UPPER STOREYS OF A
TWO-WAY FRAME.

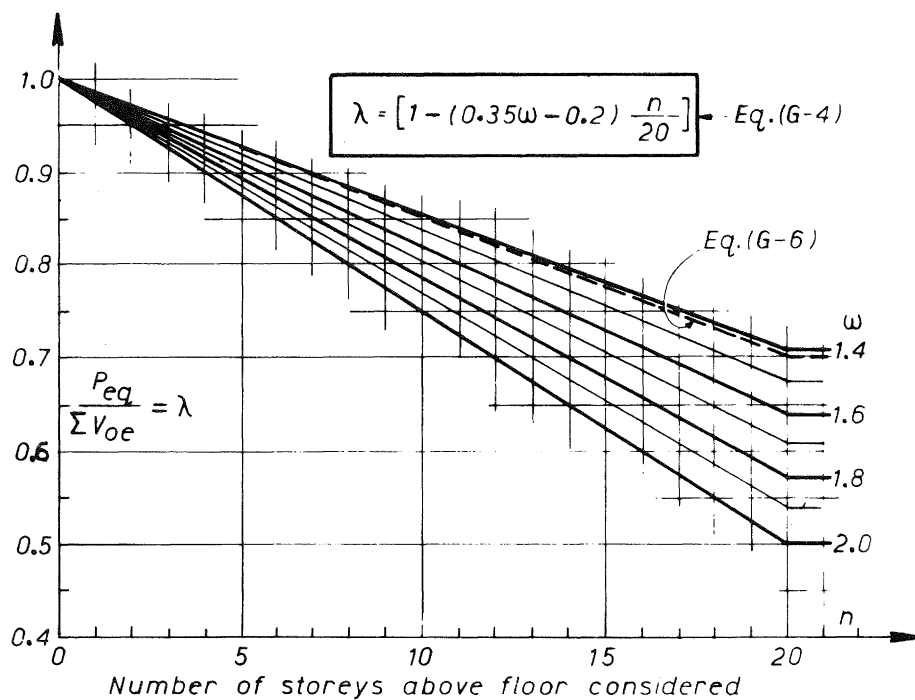


FIGURE G.5: THE REDUCTION OF
EARTHQUAKE INDUCED DESIGN
AXIAL LOAD WITH STORY NUMBER.

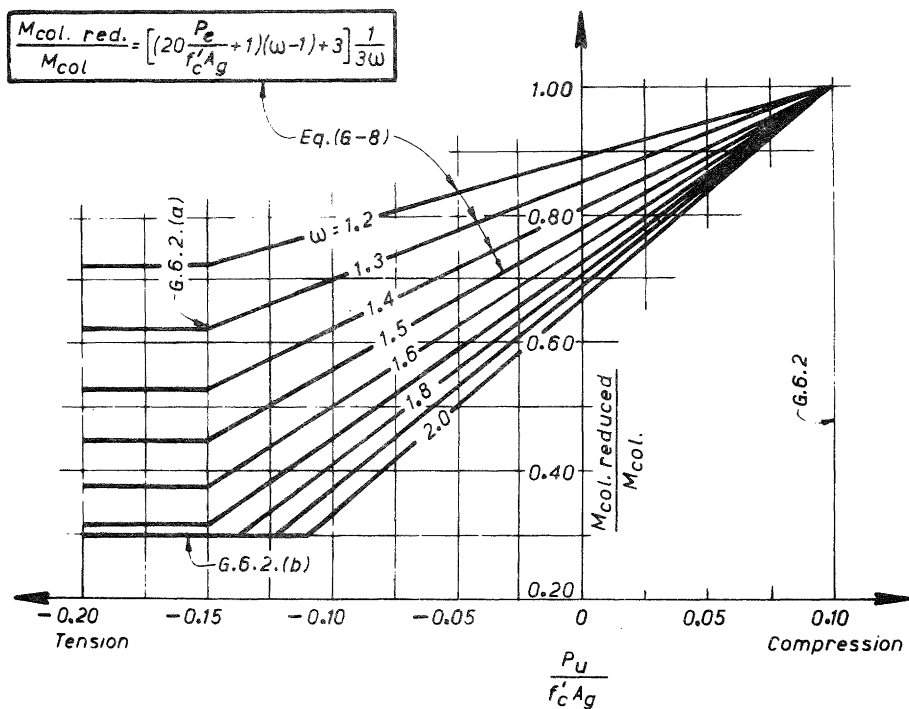


FIGURE G.6: REDUCTION OF DESIGN MOMENTS IN COLUMNS.

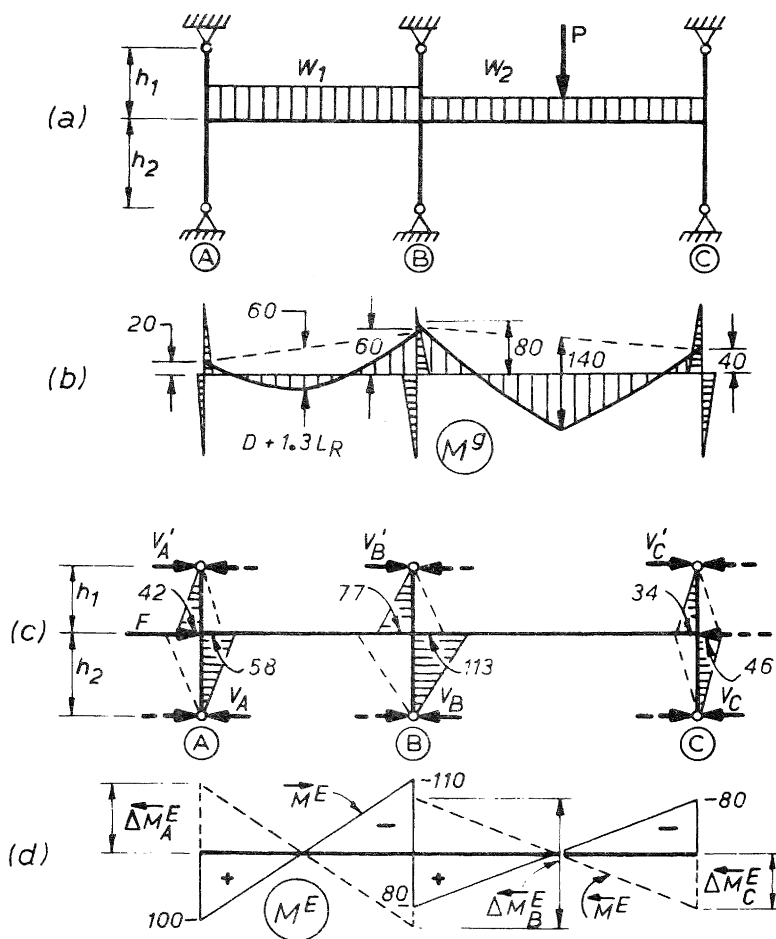


FIGURE G.7: AN EXAMPLE FRAME SUBASSEMBLY AND THE GRAVITY AND EARTHQUAKE MOMENT REQUIREMENTS.

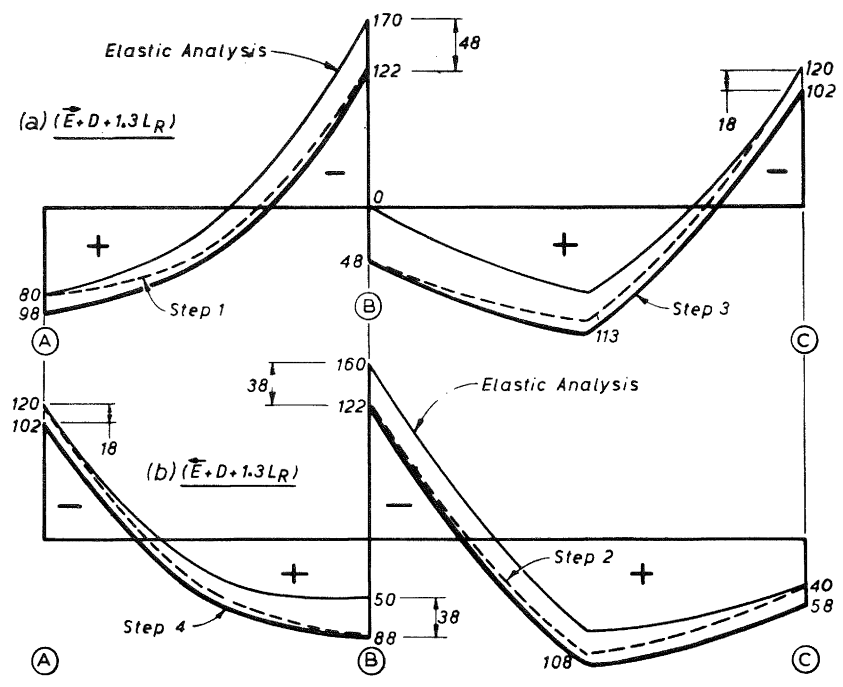


FIGURE G.8: THE REDISTRIBUTED FINAL DESIGN MOMENTS FOR THE LOAD COMBINATION $(E + D + 1.3L_R)$.