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Displacement Based Design of Ordinary Moment-Resisting Composite Frames

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Abstract

The use of moment-resisting frames as the lateral force-resisting system for low to moderate rise buildings in Australia is examined from the perspective of displacement-based design. The design is likely to be governed by a Level 3 earthquake, one with a 2% chance of exceedance in 50 years, with the corresponding performance level typically being that of collapse prevention. These frames are flexible, and for some site conditions are likely to respond in the elastic range, even under the Level 3 earthquake. A case study has been taken of a composite frame using concrete-filled steel tubes as the columns. The shortcomings of the usual AS1170.4 design procedure have been examined in detail and a new displacement-based approach has been proposed for design.

Keywords: moment-resisting frames, displacement-based design, band beams, concrete-filled steel tubes

1. INTRODUCTION

The design process for a case-study ordinary moment-resisting composite frame is investigated in this paper, with careful consideration given to seismic loads and displacements. Composite frames are commonly used in Europe, Japan and China, mainly due to their enhanced speed of construction and structural efficiency as described in CIDECT 5 (Bergmann et al. 1995). In an on-going research project, unique moment-resisting beam/column connections are being developed that fit in well with Australian practice, since they are simple to construct, and require shop-welding and field-bolting rather than extensive welding on site (Yao et al. 2008). This paper ties in with that overall project and looks at one possible system that could be useful in the Australian construction environment.

In recent years the field of earthquake engineering has seen a shift towards the displacement based design method (Priestley et al. 2007). Several fundamental flaws exist within the current force-based methods that are widely used. Of particular interest are the discrepancies between the natural period assumed in AS1170.4-2007 (Standards Australia 2007) and those calculated analytically. Drift predictions obtained using the AS1170.4 force-based approach are shown to be unrealistic when compared with the displacement demand predictions from a code-compatible displacement response spectrum.

2. MODEL BUILDING DESCRIPTION

The building modeled is a five storey, four bay composite framed building. The columns are concrete filled square hollow sections, while the beams are UBs with a 120mm reinforced concrete slab supported by metal sheeting such as bondek.

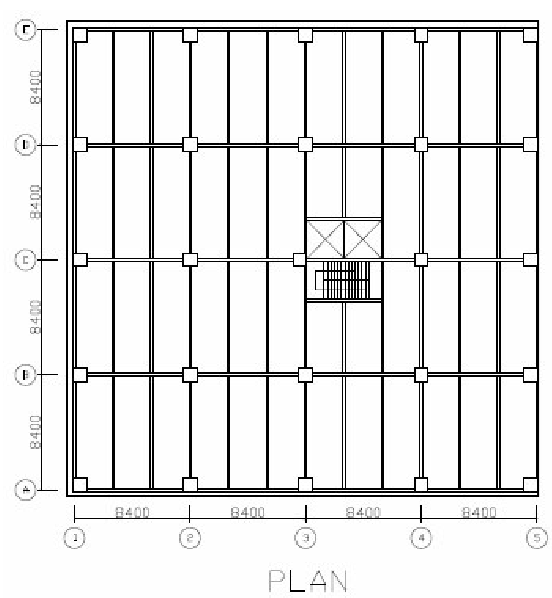


Figure 1: Plan of a five storey building

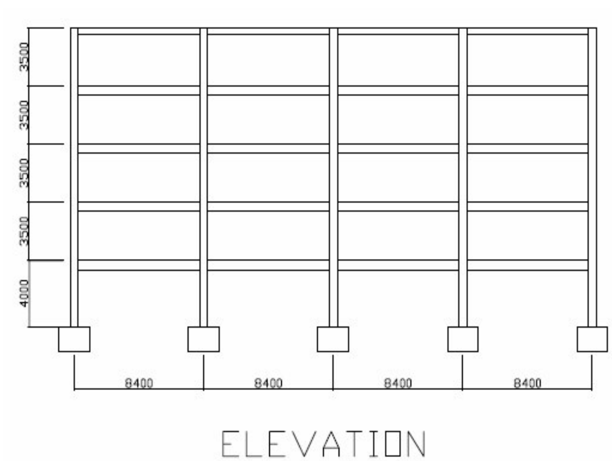


Figure 2: Elevation of the building

Each bay is 8.4m wide in each direction. The storey height is typically 3.5m with the exception of the first floor which is 4m. The arrangement can be seen in Figures 1 and 2. The building has two perimeter moment resisting frames in each direction. It is only in the perimeter frames that the connections are designed to be moment-resisting and the contribution from the other frames to the lateral force-resisting system will be ignored. The frames are assumed to be fixed at the base (due to the assumed presence of a rigid perimeter basement wall) and to be located on a site with class B soil in Melbourne or Sydney.

The moment resisting frame connections modeled are composite blind bolted connections with T-stubs (Yao et al. 2008). A 2D analysis was undertaken using the structural analysis software package Spacegass (Integrated Technical Software 2010).

3. LOADS

3.1 Dead load and live load

The gravity loads applied to the building are typical loads for office type buildings. The dead loads used were 1.0kPa, 0.5kPa for façade, plus self weight, while the live load used was 3.0kPa

3.2 Earthquake loads – Using the Australian Standards

Earthquake loads were determined in accordance with AS1170.4 (Standards Australia 2007) using the equivalent static force method for return periods of both 500 and 2500 years. This method involves calculating the base shear force using equation (1) below. The base shear is then distributed up the height of the building in accordance with equation (2) as shown in table 1.

$$V = [k_p Z C_h(T) S_p/\mu] W_t \tag{1}$$

The following parameters were used; probability factor k_p = 1.0 and 1.8 (for return periods of 500 and 2500 years respectively), hazard factor Z = 0.08, natural period T = 1.2s (equation (4) below), spectral shape factor $C_h(T)$ = 0.73, structural performance factor S_p = 0.77 and ductility factor μ = 2. Therefore for a return period of 500 years V = 0.022W_t or 2.2% of the seismic weight and for a return period of 2500 years V = 0.040W_t or 4.0% of the seismic weight, W_t.

$$F_i = k_F V \tag{2}$$

Where $k_F = W_i h_i^k / \sum(W_j h_j^k)$ (3)

$$T = 1.25 k_t h_n^{0.75} \tag{4}$$

Where k_t = 0.11 for moment resisting steel frames and the coefficient k used in equation (3) is equal to 1.35, based on a period of1.2 seconds calculated using equation (4).

Since the lateral force-resisting system consists of two spandrel moment resisting frames in each perpendicular direction, half of the building seismic mass is used in calculating the earthquake lateral load applied to each frame as shown in table 1.

Table 1: AS1170.4 Seismic Loads acting on one frame for 500 year return period

Storey, i	**Dead Load, G (kN)**	**Live Load, Q (kN)**	**Seismic Weight, W_i (kN)**	**Height, h_i (m)**	**h_i^k**	**$W_ih_i^k$**	**k_F***	**Seismic Force, F_i (kN)**	**Deflection, d_{ie} (mm)**
5	2822	847	2822	18.0	49.5	139713	0.33	121	75.7
4	2822	1693	3330	14.5	36.97	123125	0.29	106	68.0
3	2822	1693	3330	11.0	25.46	84797	0.20	73	54.3
2	2822	1693	3330	7.5	15.18	50563	0.12	44	36.3
1	2922	1693	3330	4.0	6.5	21641	0.05	19	16.5
TOTAL			16144			419840	1.0	363	

*k_F is the proportion of the base shear going to any particular level

Similarly, the forces for the 2500 year return period were calculated by using a scaling factor of 1.8. These forces were then factored up by 1.05 to account for torsional effects. Since this is a preliminary design this has been assumed as appropriately accurate at this stage. Additionally, biaxial bending has been considered by applying 100% of the force in one direction and 30% of the force in the orthogonal direction as per AS1170.4.

A return period of 500 years must currently be considered when determining the ultimate earthquake design load for a building of this importance level (Level 2) in accordance with the Building Code of Australia (ABCB 2009). This corresponds to a k_p factor of 1.0 in Table 3.1 of AS1170.4 (Standards Australia 2007). However, as discussed in section C3.1 of the commentary to AS1170.4 (Standards Australia 2007), designing for a return period of 2500 years is considered internationally to result in a more uniform risk of collapse when comparing regions of low to moderate seismicity with those of high seismicity. This is because the 2500 return period event is likely to be greater than the 500 year return period event by a larger factor in regions of low to moderate seismicity than in regions of high seismicity. Hence, in the U.S. and Canada, the philosophy that has been adopted is to design for collapse prevention (a state of non-collapse or near collapse) for the maximum level of ground shaking that could reasonably be expected to occur at a site, the MCE or Maximum Considered Earthquake, which is determined using a return period of 2500 years.

The performance objective for the 500 year return period event used in AS1170.4 could be regarded as damage control in that there is a limit placed on interstorey drifts. This limit of 1.5% of the storey height is quite strict compared with the 2% or 2.5% damage control limits for a 500 year return period event commonly used in displacement-based design (Priestley et al. 2007). In regions of high seismicity the performance objective for the 2500 year return period event is typically that of collapse prevention and is usually achieved by using capacity design principles.

Additionally the frame was checked for structural robustness including the application of 2.5% of G + Ψ_cQ at each level and 5% of G + Ψ_cQ at the connections. Neither of these cases governed the design.

3.3 Wind loads

The wind loads were calculated in accordance with AS1170.2. For region A1, non cyclonic conditions and a 500 year return period on a terrain category 3 site, the following wind paramters have been calculated; regional wind speed V_R = 45m/s, serviceability wind speed V_s = 37m/s, wind direction multiplier M_d = 1.0, terrain/height multiplier $M_{z,cat}$ = 0.94, shielding multiplier M_s = 1.0, topographic multiplier M_t = 1.0, external pressure coefficient C_{pe} = 0.7, -0.5, internal pressure coefficient C_{pi} = -0.2, 0 and combination factor K_c = 0.8. This resulted in wind pressures of; p = 0.52 kPa windward and p = 0.29 kPa leeward. These loads were then calculated and applied at each floor based on tributary area.

3.4 Combination load cases

The combination load cases investigated were; 1.2G + 1.5Q, G + 0.3Q + EQ and 1.2G + WL.

4. MEMBER PROPERTIES

The composite members were modeled in Spacegass by adjusting the stiffness of each element. Eurocode 4 (CEN 2004) and CIDECT 5 (Bergmann et al. 1995) have been used to determine the strength and stiffness of the composite sections. This was then verified using the software compbeam (Onesteel 2010). It was assumed that the beam would crack over the supports and would remain uncracked throughout the rest of the beam. A constant effective width was assumed and calculated in accordance with Eurocode 4. The connections were modeled as semi-rigid with a stiffness of $16EI_B/L_B$ based on research currently being undertaken. This was achieved in Spacegass by applying springs to the beam element ends. The columns have been assumed to be fixed against rotation at the base as discussed previously.

5. ANALYSIS

Spacegass was used to perform a linear elastic analysis on the frame. From this the deflection, bending moments, shear forces and axial forces were assessed. Using the design actions obtained and the drifts, member sizes were selected. Following this a dynamic frequency analysis was run to determine the period of the structure. The key analytical results have been summarized in Table 2. Based on the results shown in Table 2 the member sizes were determined as 350x8 SHS concrete filled columns with composite 410UB54 beams. It can be seen that the drift limit required for AS1170.4 is the governing design criteria, although the serviceability wind drift is also close to being exceeded. Interestingly the 2500 year return period earthquake forces (which are not required to be considered in the BCA for this type of building (Importance Level 2)) do not govern the design. They are calculated in accordance with AS 1170.4 and incorporate a μ/S_p factor of 2.6.

Table 2: Results from linear analysis

Load Case	Maximum Bending Moment			Drift, d_i ($d_{ie} x \mu/Sp$)	Drift Criteria	Governing Criteria
	Beam		Column			
	Negative	Positive				
1.2G + 1.5Q	212kNm	90kNm	113kNm			Strength
G+0.3Q+EQ_{500} (code)	166kNm	135kNm	217kNm	60 mm	60 mm*	Drift
G+0.3Q+EQ_{2500} (code)	298kNm	243kNm	390kNm	108 mm		Strength
1.2G + WL_u	112kNm	135kNm	177kNm			Strength
WL_s				7 mm	8 mm	Drift

** Denotes governing criteria*

The flexibility of the frame is such that the yield displacement is high. Experimental studies carried out at the University of Melbourne (Yao et al. 2010) have indicated a large reduction in stiffness when the connection experiences slip between the beam flanges and the stem of the T-stub. The likely design approach will be to design the connections to be "partial strength", reaching yield before either the beams or columns at the joint, and the slip moment will be regarded as the effective "yield" of the connection. It will be designed to occur when the column reaches 80% of its yield moment (The yield moment is assumed to be approximately 80 % of the ultimate moment). From the Spacegass elastic analysis the yield displacement of the equivalent substitute structure has been estimated to be 134 mm. This is a lower bound estimate and is based on first yield of the columns at the fixed base. It is interesting to note that this is much higher than the maximum drift demand calculated using the code-compatible displacement spectra (26 mm as shown in the code based displacement spectra seen in figure 3b) and is even greater than 1.8 times this value which is supposed to correspond to the 2500 year return period event, suggesting that the building will not experience yield in these events.

6. RESULTS/ DISCUSSION

Based on the acceleration spectra provided in AS1170.4 the displacement response spectrum has been produced as seen in figure 3(b). From the graph it can be seen that for a period of 1.2 seconds, which is defined by equation 6.2(7) in AS1170.4, the displacement demand is 21 mm. Even the maximum possible displacement demand is small, only 26 mm.

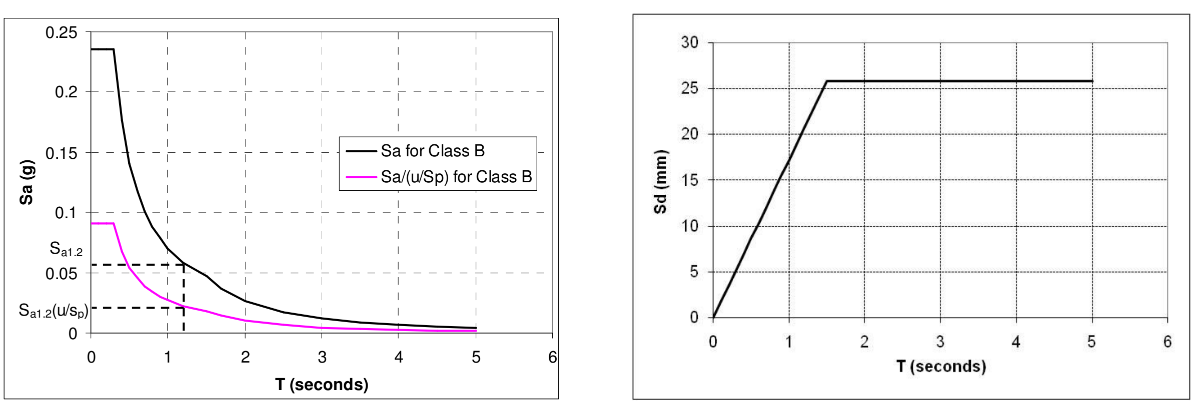
The intention of the AS1170.4 standard is to limit the interstorey drift demand to less than the drift capacity, i.e.1.5% of the storey height. However the method used to determine the displacement of the structure due to the 500 year return period earthquake is shown below to be seriously flawed when applied to this case study. The problem encountered here is that the frame actually has a natural period of 2.9 seconds (found using a dynamic frequency analysis in Spacegass) and hence a stiffness of $(1.2/2.9)^2$ times the stiffness of the 1.2 second structure that was used to calculate the base shear force in the code-based method. The AS1170.4 method is illustrated in Figure 3. In Figure 3(a) the elastic response spectrum stipulated by AS1170.4 is shown for Class B site conditions. The spectral acceleration corresponding to a period of 1.2 seconds is reduced by μ/S_p in the AS1170.4 method and this value, $S_{a1.2}/(\mu/S_p)$

(= 0.022g in this case study) is used to find the base shear. The base shear is then distributed up the height of the building using equation (2) resulting in the force distribution shown in Table 1 (the F_i values). The elastic displacements, d_{ie}, given in Table 1 are calculated by applying these forces to the actual frame in the Spacegass model. These displacements are then factored up by μ/S_p and are intended by AS1170.4 (Standards Australia 2007) to represent the displacements due to the 500 year return period event.

The important point that is being made here is that the displacement demand calculated using the AS1170.4 method is clearly impossible for this structure. Using the substitute structure approach the effective displacement would be said to be approximately 156 mm (the displacement at 2/3rds of the height) which is much higher than the maximum possible demand of 26 mm from the AS1170.4 (Standards Australia 2007) compatible displacement spectra. These discrepancies suggest that an alternative method for design should be sought.

7. PROPOSED DISPLACEMENT-BASED METHOD FOR DESIGN

The direct displacement-based method advocated in (Priestley et al. 2007) relies on displacement spectra being available for a given site. However, there is considerable controversy attached to the derivation of these spectra which is primarily associated with the estimate of the corner period, i.e. the period at which the spectra changes from “constant velocity” to “constant displacement”. For example, the corner period estimated by (Faccioli et al 2004) as reported in (Priestley et al. 2007) consistently predicts higher values of the corner period than those stipulated in AS1170.4 (2007) which is based on recommendations in (Lam et al 2000). For example, in the case of a 500 year return period event in Australia (taken to be that due to a 7.0 Moment Magnitude earthquake producing a 0.08g PGA (effective peak ground acceleration) at a firm ground site, (Faccioli et al 2004) would predict a corner period of 4.25 seconds and a displacement demand at the corner period of 103 mm. These values are both approximately three times higher than the displacement spectra compatible with the acceleration spectra defined in AS1170.4 (Standards Australia 2007). An evaluation of the corner period provisions is presented in Lumantarna et al. (2010) which is published in these proceedings.



(a) (b)

Figure 3(a) – AS1170.4 acceleration response spectra for Class B site.
Figure 3(b) – AS1170.4 compatible elastic displacement spectra

In the design method proposed here, the AS1170.4 (Standards Australia 2007) acceleration spectra and the compatible displacement spectra have been used. A quick step-by-step summary of the method is given below:

1) Design for code load combinations in the usual manner. In this case study the earthquake forces would still be estimated using the AS1170.4 estimate of the natural period, i.e. 1.2 seconds, since this attempts to take into account the presence of non-structural components that tend to stiffen the structure. The artificial reduction factor of μ/S_p would also be used, simply to be consistent with the current design method, although ductility appears to be largely irrelevant in this design.
2) Drift due to serviceability level wind would be checked and the structural member sizes would be resized as necessary.
3) Determine the fundamental elastic period of the frame and, assuming a substitute SDOF structure vibrating in the first mode, use the period to estimate the displacement demand from the elastic displacement response spectra (with 5% damping). This provides a more conservative estimate of the displacement demand than using the period of 1.2 seconds which was calculated in accordance with equation 6.2(7) in AS1170.4.
4) Obtain an estimate of the yield displacement and compare this with the displacement demand. As discussed above, in this case study the frame would not have yielded under the 500 year return period earthquake, nor the 2500 year return period earthquake.
5) Calculate the equivalent elastic stiffness, K_e, using the estimated effective period, T_e, and the equivalent mass, m_e. Use this to find the base shear force (simply multiply the effective stiffness by the displacement demand). Given that the frame remains elastic under the 2500 year return period event, the base shear force for the 2500 year event will be higher than that corresponding to the 500 year event. In this case study the effective stiffness, K_e, is estimated to be 13200 kN/m where $K_e = 4\pi^2 m_e/T_e^2$. The calculation of the equivalent mass, m_e, is based on the assumption of a linear displacement distribution (Priestley et al 2007). The effective period, T_e, will simply be the elastic value of 2.9 seconds. For a 500 year event the base shear force is 340 kN and, for the 2500 year event it is 620 kN. These correspond closely to the code estimates and there is no need to redesign.
6) The roof displacement can be estimated by multiplying the displacement demand by 1.5. For example for the 2500 year return period earthquake the roof displacement is estimated to be 70 mm, giving 14 mm per floor if the drifts are evenly distributed. Even this is considerably less than the code drift limit of 1.5% of the storey height, i.e. 60 mm.

8. CONCLUSIONS

A four bay, five storey composite framed building has been modeled in Spacegass and designed in accordance with Australian Standards. It was found that;

- The 1.5% drift limit imposed on drifts estimated using the method outlined in AS1170.4 was the governing criterion in the design of the frame elements.
- Further investigation revealed that the method used in AS1170.4 to determine the drifts due to the 500 year return period earthquake event resulted in a gross overestimate of these values. The displacement demand of a single degree of freedom equivalent structure corresponding to the case study frame on a Class B site was estimated to be 26 mm using a code-compatible displacement spectrum, whereas the AS1170.4 method estimate of displacement demand was 156 mm.
- The errors in the code estimate of displacements were largely due to the discrepancy between the code-estimated natural period of the frame and the actual period of the frame.
- The case study frame did not experience yield under the 500 year return period event and even remained in the elastic range under the 2500 year return period event.
- An alternative displacement-based design method has been outlined in this paper.
- Further work is needed to investigate the appropriateness of this method for different site conditions. The large discrepancy between different proposed displacement design spectra such as those proposed by (Faccioli et al 2004) and those that are compatible with the AS1170.4 needs to be investigated.

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