EARTHQUAKE-RESISTANT DESIGN OF TUBULAR BUILDINGS

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ABSTRACT

Structural design of high rise buildings susceptible to seismic excitation is occasionally performed in design offices. Regardless of the type of structural system used for a particular building located in an earthquake zone, the interstory drift must be within a permissible limit, the structure should attract the minimum magnitude of seismic forces and the building must possess sufficient ductility and hence sufficient energy absorption capacity such that it can withstand inelastic deformations without a catastrophic structural failure. These requirements are generally met by most tubular buildings because of the ductile moment-resisting characterstics of these building systems. This paper presents a comprehensive design procedure for the aseismic design of steel tubular buildings and illustrates it with a design example. A simplified procedure for a preliminary design based on a so-called 'stick' model is suggested. Some important observations are made on the results of the seismic analysis of the 'full' 3-D model of the building and its reduced stick model.

INTRODUCTION

Current trends in tall building design point to tubular structures, the mechanics and behavior of which have been developed by Khan (1,2). A large number of skyscrapers in the USA have been built using the tubular principle. Most of the ultra highrise buildings today are built on Khan's principle or a variation of it. A basic feature of Khan's work was to make very efficient exterior tube configurations carry the lateral loads imposed on tall buildings by wind or earthquake instead of assigning this role to the less efficient interior frames. Tubular structures seem to be a viable solution for economical and efficient aseismic design of tall buildings because of the ductile behavior under inelastic load reversals occurring during severe seismic occurrences and the substantial reduction in lateral drift. A desirable requirement for ductile moment-resisting frames is the strong column-weak beam concept (3). The majority of the columns in the frames in the direction of the seismic input (i.e., web frames) usually should satisfy the strong column-weak beam criteria. However, because of the inherent load resisting properties of tubular structures, where flange frames behave differently from the web frames, this requirement need not be strictly complied with for the entire tube. A tubular structure is thus a logical structural system for the tall buildings exposed to seismic hazards. Since gaps and uncertainties in our present state of knowledge do exist in the field of earthquake engineering and in view of the fact that a complete correlation between the available theoretical research and practical design aspects is yet to be achieved (4), engineers and architects are usually hesitant to build tall buildings in seismic zones beyond traditional height limits. This paper presents a comprehensive design procedure for tubular buildings located in seismic zones. By no means this is the only way by which the design may be accomplished. It can hardly be overemphasized that the judgement and common sense of the design engineer must always prevail while selecting design criteria and making design decisions.

SOME EARTHQUAKE-RESISTANT DESIGN ISSUES

A tubular structure is generally efficient for buildings about 40 stories and higher. Evidently, a dynamic analysis is warranted for a structure of this height. A linear elastic analysis is somewhat inaccurate by virtue of assuming purely elastic behavior which may or may not be true for a severe seismic disturbance. A non-linear dynamic analysis of structures for earthquakes is quite desirable. However, because of the slow progress of development of computer codes and the designer's natural resistance to accept more complex analysis methods, linear elastic dynamic analysis is more common in design offices.

Despite the limitations of the response spectrum method of analysis, this method is often used in design offices because of the relative simplicity of the method and the reduced cost involved as compared to the time history analysis.

The design of tall buildings is usually done for the gravity and wind loading and subsequently checked for the earthquake loading by an appropriate dynamic analysis method. In the following, the basic steps of the earthquake-resistant design of a tubular steel building are presented. This procedure is felt to be complete, but may need to be updated and modified as further research results on the subject become available.

THE DESIGN PROCESS

The design of a steel tubular building essentially includes the following steps:

1. Size the columns for gravity loads (dead plus reduced live loads) assuming an allowable steel stress of 0.5 F_{yc}, where F_{yc} is the column yield stress. Assume this value at 0.4 F_{yc} for corner columns to allow for biaxial moments. Size spandrel beams for stiffness using Manney-Goldberg type Equation (5). While sizing spandrels, make adjustments to ensure that for all joints in each floor, the stiffness ratio $\Sigma K_c/\Sigma K_b$ is approximately equal to unity within practical limits, where, $K_c = I_c/H$ (I_c = moment of inertia of column section; H = story height) and $K_b = I_b/L$ (I_b = moment of inertia of beam section; L = beam span).

Also, ensure that at every joint the condition $\Sigma M_{pc} \geqslant \Sigma M_{pb}$ is met, where, M_{pc} and M_{pb} are, respectively, the plastic moment capacities of columns and beams.

2. Analyze the idealized three dimensional structural model using a suitable computer program for gravity loads and wind loading. Plot wind load drift and shear lag effects for the wind directions considered. To ascertain the extent of tubular behavior, find the cantilever drift Δ_c by subtracting the deflection due to frame wracking, Δ_f, from the total lateral deflection Δ. The deflection Δ_f can be found from a supplementary analysis by either assigning infinitely large cross-sectional areas to the columns or restraining the vertical movements of the nodes (i.e., beam-column joints). A 60-80% cantilever drift is generally a good index of tubular behavior.

3. Check stresses in columns and spandrel beams for various load combinations and redesign for the new member forces. Spandrels are normally understressed but may have to be resized for stiffness. Study the interstory drifts, the shear lag effects and the stress patterns in the spandrels and adjust member sizes with a view to optimizing the structure. Reanalyze the structure and iterate the design until the building is tuned for the specified wind load drift limitation.

4. Perform a 3-D static earthquake analysis following the provisions of the Unified Building Code [UBC], (6), or any other applicable code [e.g., Ref. (7)] on the structure and check the lateral drift. If a dynamic analysis shows, however, that the corresponding base shear and overturning moment are greater than those by the static analysis, in many instances then the static analysis may not be required. This matter is subjective and a decision is to be made by the designer in consultation with the authority having jurisdiction.

5. Carry out a preliminary dynamic analysis on a reduced stick model having the equivalent shear and bending stiffnesses as for the prototype to obtain the fundamental period, base shear, overturning moment, building drift and the number of modes that should be considered for the final dynamic analysis (4). In Reference (4), torsional stiffness is not considered. For unsymmetrical buildings where centers of mass and rigidity are non-coincident, the torsional constant J_i at level i can be found for i = 1, 2,....,n levels from the relationship: $J_i = (T_i H_i)/(G\,\theta_i)$, where T_i is the story torsion, H_i is the story height, G is the shear modulus of elasticity and θ_i is the interstory angle of twist. From a wind load analysis where wind is applied at an eccentricity from the center of rigidity of the building, the terms T_i and θ_i can be readily found without additional analysis effort. The stick model can then be analyzed by inputting the cross-sectional area, shear areas in the two principal directions, torsional constant and the moments of inertia in the two principal directions, i.e., all the six parameters defining the complete properties of the cantilever beam. Revise member sizes if required after the preliminary analysis.

6. Select the critical earthquake directions and their combinations and run elastic response spectrum analysis (or a time history analysis) for the maximum probable and maximum credible earthquakes using a suitable computer program (ETABS, TABS 4.0, etc.). Check drifts and ductility demand ratios and adjust the member sizes based on the adopted design criteria. One approach is to define the ductility demand ratio μ as the ratio of the maximum computed moment at a critical section in the elastic structure, M_o, to the plastic moment capacity of the section, M_p, (8). For columns, also satisfy AISC Code Section 2.4 requirements, (9). Also, ensure that $\Sigma M_{pc} \gg \Sigma M_{pb}$ after reducing M_{pc} for the column axial loads using a standard formula such as in Reference (10).

7. If major changes have been made in member sizes, a final wind load analysis and a seismic analysis are desirable to 'fine tune' the structure. Also, a P-Delta analysis is required for all lateral load cases. Additional considerations are necessary to estimate the drift caused by the P-Delta effect and panel zone distortions. Panel zone deformations can often be minimized by using doubler plates in column webs.

8. Design web connections, panel zone stiffener plates, continuity plates and doubler plates as per AISC requirements and SEAOC recommendations (3,9). Use M_{pb} values for beam moments even though the beams do not develop plastic hinges. All joints where beams do not develop plastic moment M_{pb}, may qualify for waiver from full ductile design requirements from the authority having jurisdiction once a set of design criteria is established for such conditions.

DESIGN EXAMPLE

Consider the 58-story steel office building, the 'foot print' of which is shown in Figure 1. The building elevation and the adopted structural model are shown in Figure 2. The structural system utilizes the tube-in-tube concept. The geometry of the floors remains constant throughout the building height. Each floor is comprised of 3 in. (76mm) non-cellular metal deck and 2.5 in. (63.5mm) lightweight concrete and may be assumed to act as a rigid diaphragm.

Following the procedure outlined in the foregoing, member sizes were proportioned for the gravity and the wind loads as per Los Angeles Building Code (1980) using the SAPIV computer program (11). The structure was tuned for an interstory drift of H/400. It was noted that the maximum drift occurred when wind was applied on the diagonal face of the building, i.e., in the north-south (N-S) direction. In view of the somewhat inefficient tubular configuration of the building, the cantilever deflection was noted to be on the order 45% of the total drift in this direction. For the earthquake analysis, the general design criteria adopted were that member stresses would be checked for maximum probable earthquake (50 years return period). For the maximum credible earthquake (100 years return period), the structure was checked to assure that plastic hinges form in beams only. Columns were designed to remain elastic in all cases—a more conservative assumption as compared to the less severe requirement in some codes where plastic hinges are allowed to develop in columns, but should not howerver result in story mechanisms. Drifts were checked against specified limits for all cases. An accidental seismic torsion was considered for both static and dynamic analyses. Further, P-Delta effects were considered for all lateral load cases.

A static earthquake analysis by SAPIV was performed for K = 0.67. Masses were calculated by assuming 50% partition load and 10% of unreduced live load. A maximum permissible interstory drift of H/200 was adopted. The period of the building using UBC approximation was found to be 7.11 seconds. A dynamic analysis was than performed on a reduced stick model for the N-S direction using the maximum probable earthquake response spectrum input shown in Figure 3. The full details of this analysis are presented in a companion paper (4). The analysis indicated that about 30 modes should be considered for the 3-D structural analysis to get an accurate estimate of the base shear and member forces. However, to cut down the cost of this study, only 5 modes were considered for the analysis of the structure. A modal analysis was performed on the 3-D structural model adopting the mass lumping scheme shown in Figure 2 and using ETABS computer program (12). The first five modal periods are shown in Table 1. The fundamental period is noted to be 7.4 seconds, i.e., about 4% higher than the UBC approximation and about 5% higher than that found for the stick model. A response spectrum analysis was performed on the structure using the ETABS program for the maximum probable earthquake spectrum shown in Figure 3. The earthquake was input in the x and α directions. An earthquake response spectrum input in β-direction would be significant since this would cause the maximum torsional effects in the building. This is apparent from the fact that the centers of mass and rigidity were on the line of symmetry of the building along the N-S direction about 10 ft. (3m) apart. However, this was not pursued herein. Because of geometric and loading equivalence in the x and y directions, no analysis was performed for seismic input in the y direction.

A comparison of the base shear and the base overturning moment for the loading cases of wind, UBC seismic and maximum probable earthquake (on stick model and 3-D full model) is presented in Table 2.

In Table 2, note that, if higher modes would be considered, the base shear would increase in magnitude for full dynamic model. In the N-S direction, the wind load results in the greatest base shear as well as overturning moment. The large overturning moment caused by UBC seismic loading is primarily due to the single point load at the building top, F_t = 740 kips (3293 kN).

The deflection at the top of the building due to the maximum probable earthquake is 10.5 in. (267mm) for the full model and 11.5 in. (292mm) for the stick model. Since a tall building has a large fundamental period and the contribution of the higher modes to building deflection is negligible, it is not excited as much as a shorter building. It was also noted for the maximum probable earthquake that nowhere the moment M_o did exceed the plastic moment capacity M_p of either the beam or column sections. An analysis of the building for the maximum credible earthquake (100 years return period; Figure 3) by the ETABS program also revealed that the moment M_o was always less than M_p at all beam and column sections.

A P-Delta analysis for wind using the SAPIV program indicated that convergence was achieved in two cycles and the additional forces caused by the second order effects are within 10%. Since wind load criteria controlled the basic design of the structure, no P-Delta analysis was performed for the maximum probable and credible earthquakes. Note in this connection that P-Delta effects for the response spectrum method can only be qualitatively estimated since the SRSS (square root of the sum of the squares) deflections in fact constitute an envelope of maximum deflections only and therefore do not correspond to any particular deformation profile of the building under seismic excitation. All member sizes determined from the gravity and wind load criteria were found satisfactory for the adopted seismic design criteria. A final investigative analysis was conducted for the maximum probable earthquake using ETABS program by obtaining the total responses by the CQC (Complete Quadratic Combination) technique (13). The CQC values of forces and displacements were noted to be within 5% of the previously found SRSS values.

CONCLUSIONS

This paper presents a design procedure for the earthquake-resistant steel tubular buildings. Tubular structural system appears to perform well in seismic zones. For the example building presented, the stiffness of the structure rather than strength seems to predominantly govern the design. Wind load criteria rather than earthquake load criteria dictate the member sizes of the building. Since the building is essentially noted to remain elastic under both maximum probable and maximum credible earthquake excitations, full ductility criteria for the design of connections are not therefore critical for this structure.

Some areas where further research is immediately needed are: non-linear dynamic analysis of tubular buildings, simplified P-Delta analysis for earthquakes, panel zone deformations and ductile behavior of columns in the flange frame controlled by buckling rather than plastic hinge formation.

As more tubular buildings with different horizontal and vertical geometric configurations and aspect ratios are designed for earthquake loading in the future, a gradual increase in the present-day understanding of the response of such structures is expected. The author believes that with further research and knowledge in this area, more efficient and economical structural systems will evolve for highrise buildings located in seismic zones.

APPENDIX I - REFERENCES

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APPENDIX II - TABLES

TABLE 1 - Vibrational Mode Periods

Mode	Time Period (Seconds)	Predominant Direction
1	7.40	α (N-S)
2	5.95	β (E-W)
3	3.62	θ (Torsion)
4	2.46	α (N-S)
5	2.04	β (E-W)

Note: α and β are the principal axes of the building.

TABLE 2 - Base Shear (Kips) and Overturning Moment (Kip-ft.)

Base Forces	Wind		UBC Seismic	Max. Prob. (Stick Model)	Max. Prob. (Full Model)	
	x	α	x, α	α	x	α
Shear	3120	4360	2900	4015	4030	3600
Overturning Moment (x 10^3)	1601	2241	1687	996	1405	1060

Note: 1 kip = 4.45 kN: 1 kip-ft. = 1.35 kN.m

APPENDIX III - FIGURES

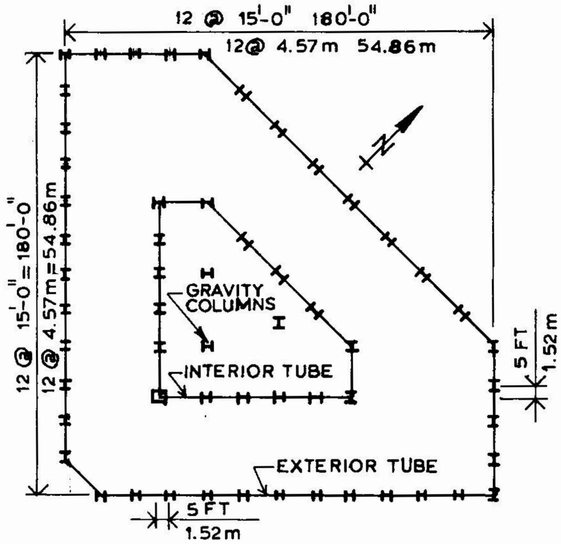

Figure 1: Building Plan

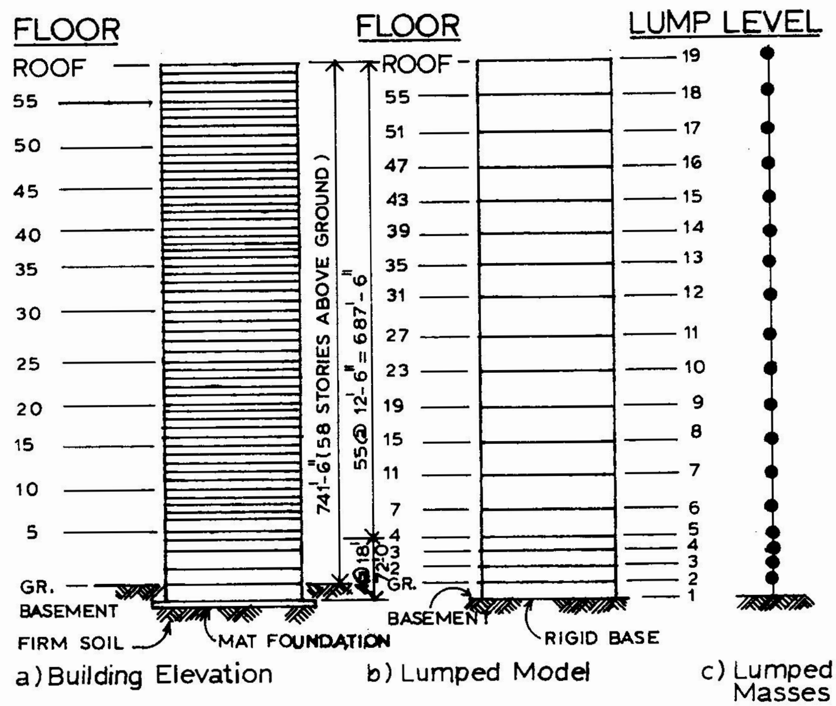

Figure 2: Building Idealizations (1 in. = 25.4mm; 1 ft. = 0.305m)

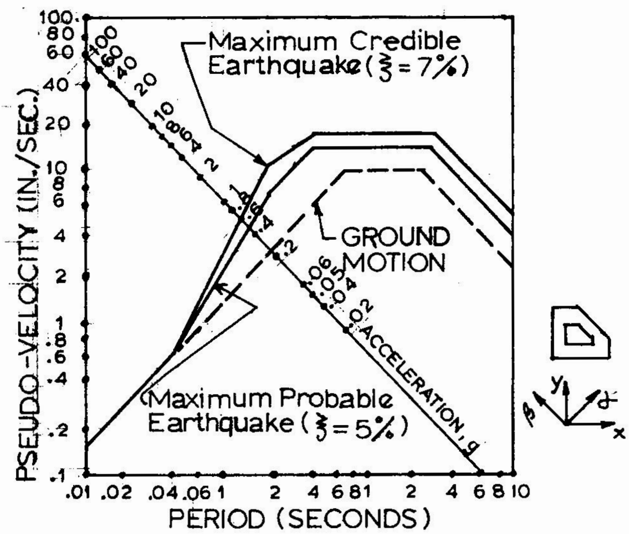

Figure 3: Earthquake Response Spectrum Input (1 in. = 25.4mm)