DESIGN OF MULTIPLE FRAMED TUBE HIGH RISE STEEL STRUCTURES IN SEISMIC REGION

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SUMMARY

This paper presents general procedures for design of multiple framed tube high rise steel structures in seismic region. The analytical methods and design procedures are outlined. Special considerations such as beam/column joints and member proportions are discussed. Three recent high rise projects completed using the multiple tubular concept are presented followed by the discussion on relative merits of multiple tubular systems.

INTRODUCTION

The "framed tube" system in high rise construction which has been widely used for structures in strong wind regions in the past two decades, usually consists of closely spaced wide exterior columns tied at each floor level with relatively deep spandrel beams, simulating a hollow tube.(1) This tubular concept is generally economically attractive and torsionally rigid, and also provides greater flexibility in space planning since most framed columns are located at the perimeter of the building.

The efficiency of the framed tube depends on plan dimension and aspect ratio. The loss of tubular efficiency in structures which are relatively long and narrow in plan is very apparent due to the shear lag effects.(1) Tubular efficiency can be improved substantially using the multiple tube concepts, as illustrated in Figure 1.

Multiple framed tube system consists of two or more framed tubes which can be various in form. The most common ones are rectangular and triangular. Three recent high rise projects completed using this system are presented.

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ANALYTICAL METHODS

All analyses were performed using a version of the three dimensional analysis computer program ETABS (Three Dimensional Analysis of Building System, Extended Version).[6][7][8]. The structural frames were modeled single 3-D space frame. Three independent degrees of freedom were permitted for each level; each floor was treated as a diaphragm rigid in its own plane.

All girders and columns of the multiple framed tube were modeled as discrete elements with nodal points at the centerline of intersecting elements. Full depths of members were used to describe the "rigid" panel zones at each node.

Critical sections for member force and stress calculations in the columns and girders were taken at the face of each rigid panel. However, the stiffness matrix formation considered only 50% of the full size of the rigid panel in order to closely approximate the actual behavior of the structure. Gravity loads (dead and live loads) were considered simultaneously with each of the following lateral load cases.

1. Wind Analysis: Wind forces prescribed in building codes (Uniform Building Code[3] or applicable city codes) perpendicular to two orthogonal faces of the structure were applied to the building frames. Overall drift and inter-story drifts were limited to 0.0025 times the overall building height and story height respectively.

2. Code Static Seismic Analysis: A set of equivalent static seismic forces were calculated based on the applicable codes using K = 0.67. These forces were applied to the building frames in two principal directions, each eccentric to the center of mass by an amount of 5% of the maximum building width. The drifts were limited to 0.0033H.

3. Three Dimensional Spectrum Analysis: Site response spectra including accelerations, velocity and displacements for lower level and upper level of earthquakes, commonly known as maximum probable and maximum credible earthquakes,[4] with damping factors of 2%, 5%, 7%, and 10% were generated by geotechnical consultants. The lower level earthquake has a probability of exceedance of 50% in 50 years. The upper level earthquake has a probability of exceedance of 10% in 100 years. The building frames were analyzed using CQC (Complete Quadratic Combination) method[7] and the drifts were limited to 0.0075H for lower level event and 0.015H for upper level event.[5]

DESIGN PROCEDURES

1. Wind Design: All framed members were checked per AISC[9] specifications on elastic design, with 33% increase permitted in the allowable stresses under combined loading of wind and gravity loads.

2. Seismic Design: All framed members were designed per AISC specifications for combined seismic and gravity loads and to satisfy the requirements for ductile moment resisting frames:

   a. Code Static Seismic Force: All framed members were designed with 33% increase permitted in the allowable stresses.

   b. Maximum Probable Spectrum: This is actually the design spectrum with loading combinations of DL + LL + EQ, and 0.8DL - EQ and the structure to remain elastic.

   c. Maximum Credible Spectrum: This spectrum is used to determine maximum building deflection and $P\Delta$ effects, as it relates to frame instability.

   d. Special Consideration: The corner columns to be designed to resist two directional seismic effects, i.e., 100% of the seismic force in the principal direction combined with 30% of the seismic force produced by the orthogonal components. For those columns, which are not a part of the frames, they were checked for allowable stress level, for DL + LL or DL + LL combined with $P\Delta$ moments with 33% increase in allowable stresses permitted, where Δ= 3/K times inter-story deflection from code static seismic forces or Δ= inter-story deflection from maximum credible excitation, whichever is greater.

   In building with ductile moment resisting space frames, strong column-weak beams concept is desirable, i.e., sum of the plastic moment capacities of the columns above and below the joint, when reduced for axial loads, is generally greater than the sum of the plastic moment capacities of the beams.[4] Hence, the proper balance of sizes between beams and columns at the beam/column joints are essential, especially in tubular building where deep framed beams are required for stiffness to meet drift criteria, but carry very little gravity loads. On the three multiple tubular buildings designed, 36" deep nominal wide flange members were used for framed columns and beams. By using high strength, grade 50 steel for framed columns, grade 36 steel for framed beams, and proportioning for columns two or three sizes heavier than the connecting framed beams, the total plastic moment capacities of the columns were about 40% greater than the plastic moments of the beams when reduced for axial loads. Continuity plates and doubler plate were provided at all framed joints to ensure full plastic capacities of the connecting beams are developed,[11] as shown in Figure 2.

DISCUSSION

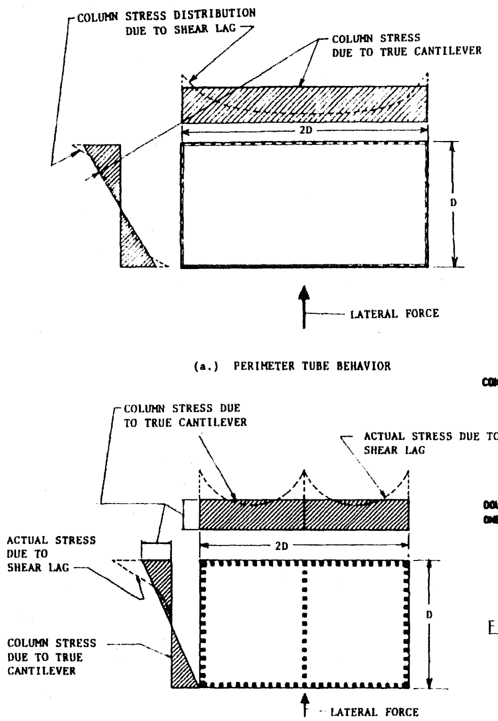
The Raymond Kaiser Engineers building in Oakland, Maquire Partners/Crocker Center, South Tower in Los Angeles and 345 California Street Building in San Francisco, California are the three recent projects using the multiple tubular concept. Each project represents a unique case because of the plan configuration of rectangular, rectangular/triangular with relatively wide face. The building height and plan dimensions are shown in Table 1. With this system, all framed columns are utilized efficiently. As shown in Figures 3, 4 and 5, gravity loads are distributed rather evenly through out the frames, regardless what the floor tributary areas on the columns might be. All framed columns, in each case, also participated well into the tubular action to provide fairly efficient framed tubes.

Seismic forces, static or dynamic, were the governing factors for framed member strength design. However, due to the wide building face, the drift requirement under wind loading in the transverse direction was the governing load case for overall frame stiffness.

Moreover, the member properties for an optimum framed tube generally requires that all framed members at a particular level are approximately of the similar size and weights. This creates very stiff frames in the longitudinal direction. The current building code procedures do not fully address this phenomenon and hence result in a non-conservative static lateral load.

In all above projects, the code static seismic forces are much smaller than that obtained from dynamic analyses, as shown in Table 1.

In conclusion, the multiple framed tube concept can be effectively utilized in the seismic region. Properly proportioned framed members will result economical structures consistent with the requirements of strength, stiffness and ductility prescribed in the current building codes and engineering practices.



(a.) PERIMETER TUBE BEHAVIOR

(b.) MULTIPLE TUBE BEHAVIOUR

FIG. 1 TUBULAR BEHAVIOUR DUE TO LATERAL FORCE

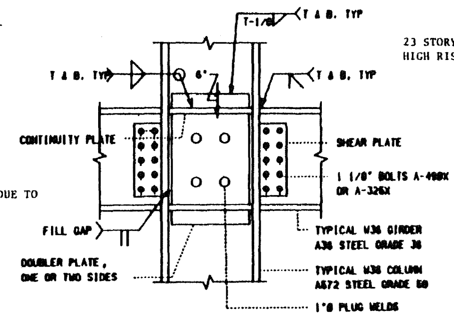
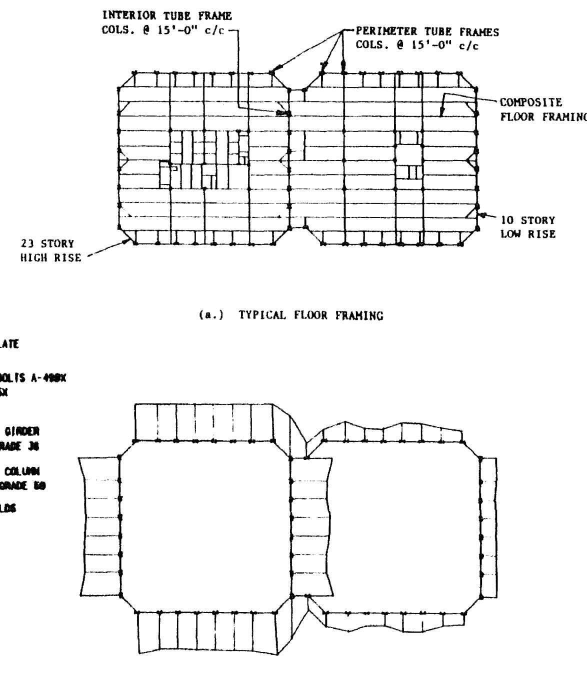


FIG. 2 TYPICAL BEAM/COLUMN JOINT DETAIL



(a.) TYPICAL FLOOR FRAMING

(b.) COLUMN AXIAL FORCE DISTRIBUTION - GRAVITY LOADS

FIG. 3 RAYMOND KAISER ENGINEERS BUILDING

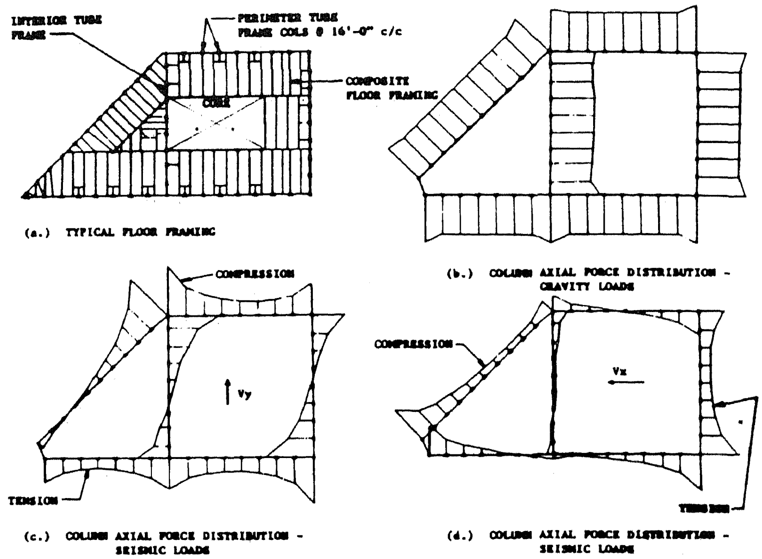


FIG. 4 MAGUIRE PARTNERS/CROCKER CENTER, SOUTH TOWER

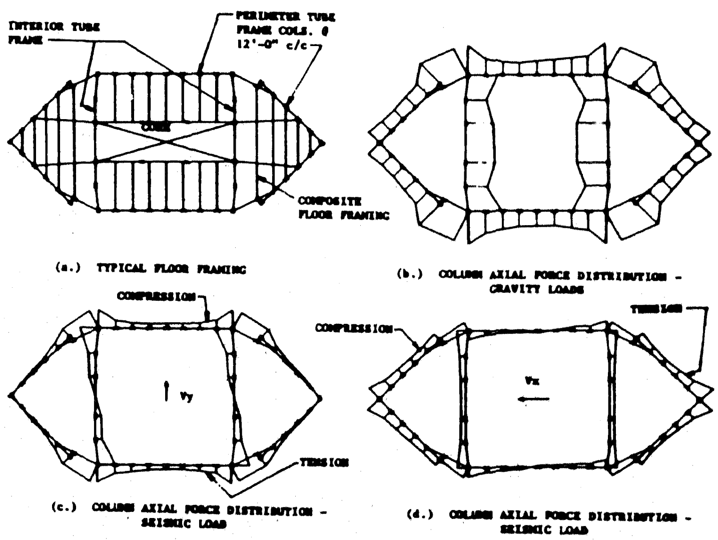


FIG. 5 345 CAILFORNIA STREET PROJECT

BUILDING DESCRIPTION

PROJECT	NUMBER STORY	TYPICAL STORY HEIGHT	BUILDING HEIGHT (ABOVE STREET)	FRAME COLUMN SPACING	PLAN DIMENSION		BUILDING PERIOD (SECONDS)	
					Y	X	Ty	Tx
RAYMOND KAISER ENGINEERS BUILDING OAKLAND, CA	25 HIGH RISE 10 LOW RISE	13'-0"	316'-0"	15'-0"	141'-6"	293'-0"	2.86	2.72
MAGUIRE PARTNERS CROCKER CENTER SOUTH TOWER LOS ANGELES, CA	49	13'-0"	635'-3"	16'-0"	134'-2"	268'-4"	5.64	4.28
345 CALIFORNIA ST. SAN FRANCISCO, CA	49	12'-10" & 11'-0"	599'-5"	12'-0"	100'-0"	227'-3 "	4.65	4.37

CODE EQUIVALENT STATIC SEISMIC FORCE

PROJECT	BASE SHEAR (KIPS)		MAX. BUILDING DEFLECTION (INCHES)		BUILDING ROTATION (RADIANS)		BUILDING DRIFT	
	Vy	Vx	Δy	Δx	Θy	Θx	$\Delta y/H$	$\Delta x/H$
RAYMOND KAISER ENGINEERS BUILDING OAKLAND, CA	2381	2857	8.94	7.51	0.000434	0.000664	0.00236	0.00198
MAGUIRE PARTNERS CROCKER CENTER SOUTH TOWER LOS ANGELES, CA	2498	2680	14.50	10.03	0.000220	0.000550	0.00201	0.00139
345 CALIFORNIA SAN FRANCISCO, CA	2305	2412	14.00	13.40	0.001306	0.001266	0.00195	0.00191

MAXIMUM PROBABLE SPECTRUM

PROJECT	BASE SHEAR (KIPS)		MAX. BUILDING DEFLECTION (INCHES)		BUILDING ROTATION (RADIANS)		BUILDING DRIFT	
	Vy	Vx	Δy	Δx	Θy	Θx	$\Delta y/H$	$\Delta x/H$
RAYMOND KAISER ENGINEERS BUILDING OAKLAND, CA	6153	6629	13.34	12.46	0.002437	0.000134	0.00352	0.00329
MAGUIRE PARTNERS CROCKER CENTER SOUTH TOWER LOS ANGELES, CA	4082	6531	15.46	15.56	0.001550	0.003330	0.00214	0.00216
345 CALIFORNIA ST. SAN FRANCISCO, CA	7600	8600	28.10	26.90	0.000074	0.000060	0.00392	0.00383

Y - Transverse Direction
X - Longitudinal Direction

TABLE 1 SUMMARY

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