EARTHQUAKE-RESISTANT DESIGN OF KYOTO TOWER BUILDING

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ABSTRACT

The earthquake-resistant design for the Kyoto Tower Building in Kyoto City introduced new and interesting problems in structural engineering. The paper describes the general philosophy and practical ideas adopted in the design of the skeleton and super-structure of this building. A detailed description of subsoil conditions, mechanical properties of structural materials and the way of assembly of structural members is given.

INTRODUCTION

The building and tower structure, Kyoto Tower Building, being in the final stage of construction as of the end of June, 1964, is located in front of the Japan National Railway's Kyoto Station. The structure is consisting of the 9-story commercial building with 3-floor basement and a tower of 5-decks for TV broadcasting and for sightseeing. The shape of the tower is illustrated in Photo. 1 and Fig. 2. It is a huge tubular stressed-membrane, inside of which are occupied by two sets of elevators and helical staircases. The foundation surface occupied by the building is 2,230 square meters. The weight including the foundation structure and 90% live load, is roughly estimated to be 35,400 tons: therefore the unit load at the foundation slab elevation is 14.5 ton/sq. m.

The building is supported on a rigid reinforced-concrete mat foundation resting on a firm gravel layer. The depth to the bottom of the foundation slab is 11.6 meters below ground surface elevation. The foundation and retaining walls have been waterproofed to obtain effective use of buoyant forces. But the buoyant forces were not taken into consideration for the design of mat foundation.

PRELIMINARY STRUCTURAL PLANNING

The contemporary structural design practice for 8- or 9-story office buildings in Japan may be summarized as follows.

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Because of economy and building code regulations which strongly require fire-proof property of structural members, ferro-concrete construction, with steel sections as reinforcement, have been overwhelmingly popular for the buildings of this size: relatively few buildings have concrete skeletons with reinforcing bars. The structural analyses for the buildings have generally been carried out on the basis of moment-resisting frames with or without shear bearing walls.

Since the lateral force requirements in Japan are strict in comparison with those enforced in other countries, the structural members of Japanese buildings generally had to have considerably large size or dimensions. The volume of the structural members times the specific weight of 2.4 for reinforced concrete or 2.5 for steel and reinforced concrete construction gives a tremendous amount of dead weight which is not desirable for the foundation design as well as the design of structure itself. It is no doubt that a larger weight or mass of structures induces a larger lateral forces or base shear in the event of earthquakes.

Therefore, for the design of Kyoto Tower Building, every effort has been made to eliminate unnecessary dead weight of the structure as much as possible. For this purpose, light weight concrete was widely used and high strength steel was unanimously adopted though the unit price of the high strength steel was a little higher than ordinary structural steel. The light weight concrete of the specific weight of 1.6 or so was used for floor slabs, concrete partitions, beams and columns on every floor above the ground surface. It has been estimated that the decrease in the total dead weight of the building is about one quarter in comparison with the buildings of this size with ordinary reinforced concrete.

Coarse aggregate of the light weight concrete used was volcanic gravel of the size ranging from 5 to 20 millimeters in diameter. This was obtained from Sakurajima Volcano in Kagoshima, Japan. However, it was well known that the light weight concrete in general shows considerably low strength. The 4-week compressive test specimens have broken at the static loads ranging from 120 to 180 kg/cm^2. Undoubtedly, therefore, it was not possible to expect a good cooperation of the light weight concrete with high strength steel nor a reliable bond stress between the concrete and steel. The reasons why we have chosen the light weight concrete for this building are merely that the concrete is one of the legally authorized fire-proof material and that the concrete will be effective for preventing local buckling of steel sections even though it is not enough compressive strength comparable with the high tensile steel.

As illustrated in Figs. 1 and 2, the Kyoto Tower Building will be utilized jointly by various enterprises such as a hotel, a social club, rental office rooms, show rooms, shopping facilities, and branch offices of bank and trust companies. Demands for the function and necessary facilities or equipments of electric and water supply are different for each of the building elements. The architectural planning was therefore a very difficult problem. In order to make architectural planning much easier, and to cope with the alteration of partitions in the future, it was attempted that no shear walls were designed in the structure above the ground surface, since the shear bearing walls in most cases confine the freedom of architectural planning if they were expected to play an effective role. All static and dynamic loads acting upon the building and the tower have therefore been assumed to be taken care of by the steel skeleton alone.

DETAILS OF STRUCTURAL ANALYSIS

Data on which the structural design of the Kyoto Tower Building is based are tabulated herein. Unit weights and dimensions of the tower and the building structure are listed in Table 1 with the design live loads. The information on the allowable unit stresses for the structural materials is given in Table 2.

The design lateral forces of earthquake has been approved by the Kyoto City Authority to adopt a seismic coefficient with 10% reduction from the value specified for reinforced concrete buildings with ordinary subsoil conditions. It is not the purpose of this paper to discuss or to make a complaint against the numerical value of the seismic coefficient, but the seismic coefficient adopted in this case was 0.18 for the height of the building above the ground level up to 16 meters. The coefficient was added by 0.009 for each additional 4.0 meters in height. The 10% reduction in the seismic coefficient was by virtue of the subsoil conditions and was based on the Japanese Ministry of Construction Notification No. 1074 enacted on July 25, 1952[1]. As shown in Fig. 3, the composition of the subsoil below the level of the bottom of the foundation slab is found to be mostly gravel layers.

Design wind pressure for the tower and the building was determined by multiplying the coefficient C of wind force and the velocity pressure calculated from the following equation;

$$q = 120\sqrt[4]{h} \tag{1}$$

where h = Height above ground level in meter, and
q = Velocity pressure in kg. per sq. m.

Coefficient C was taken to be 0.7 for the tower as it has a circular cross-section.

A number of loading conditions have been considered for the design of the tower. Design stresses at certain locations of the cylindrical membrane of the tower were compared for the cases of earthquake, wind pressure acting upon the tower in the direction parallel to the wind direction, and the effect of the Kármán vortices inducing a vibratory action on the tower in the direction normal to the wind.

Calculations of design stresses for earthquake and wind pressure were made on the basis of statics along the line of usual design practice, whereas the effect of Kármán vortex was checked by basing upon the empirical formula[2]

$$P_d = \frac{1}{16} V_c^2 C_D \tag{2}$$

$$V_c = \frac{N \cdot D_m}{S} \tag{3}$$

$$C_D = \frac{H}{D_m} - 9 \tag{4}$$

where
P_d = Uniformly distributed, lateral force in the direction normal to the wind,
V_c = Critical wind velocity,
C_D = Dynamic coefficient of wind force,
N = Natural period of vibration of the tower, (1/sec.),
D_m = Reference diameter of the tower, (m),
S = Strouhal number, assumed to be 0.18, and
H = Height of the cylinder, (m).

The natural period of vibration of the tower is, of course, associated with many factors such as the weights and stiffnesses of the tower itself, the building and the foundation. The subsoil conditions also are not independent on it although this does not seem to participate in too much. However, the comparison of the weight and stiffness of the tower with those of the building indicated that there will be no serious discrepancy of the value of the fundamental period of vibration of the tower if we assume the tower is fixed to a rigid foundation. Stodola-Vianello's method was used to find the fundamental period of vibration of the tower which would undergo a combined flexural and shearing deformation. The height of the tower was divided into 10 equal intervals and the calculation was made iteratively for the intervals.

The critical wind velocity for the Kármán vortex phenomenon was found to be 51.0 m/sec. This indicates that if the wind velocity will be kept constant at that value the tower will be oscillating at the resonant condition due to Kármán vortices.

Since the tower has a tubular cross-section, the stresses due to the shearing, flexural and ovalling were computed by using the theory of elasticity, and the working stresses for the membrane of the thicknesses presumed were checked as shown in Table 3. The steel shell is further reinforced with horizontal and vertical stiffners welded inside the membrane to prevent local buckling.

At the bottom of the tower the stream of internal forces is gradually concentrated into a framework which has eight legs of hollow box-sections of rectangular shape. All legs are pinned to the building. Anchorage of the tower to the roof of the building was made at the eight points, and for each point a total of seven anchor bolts of a three-inch diameter are spaced adequately. The static as well as dynamic loads are directly transmitted through the legs to the main columns of the building.

The structural analysis of the building frames for the permanent load was made by the moment-distribution method, and the points of inflection of the columns due to the lateral load were estimated by the method proposed by Dr. Kiyoshi Muto[3]. An example of the results of the analysis is shown in Fig. 4.

DESIGN OF STRUCTURAL MEMBERS

Based on the results of the structural analysis, cross-sections of the beams and columns of Kyoto Tower Building were designed as shown in Figs. 5(a) and 5(b). All members were so-called built up or fabricated from steel plates into desirable shapes of I-type by using submerged arc welding. Weldable steel plates were mostly used for the structural members. For columns and at the ends of beams, high strength steel plates SM50B, specified by the Japan Industrial Standards G3106, whose breaking stress has been tested to be more than 50 kg/mm^2, were used, whereas the ordinary structural steel SS41, specified by the Japan Industrial Standards G3101, was utilized for the inner half spans of the beams since the working stress there was far less than those at the ends of the beams and since the structural steel SS41 has been tested to find that it has a pretty well weldability.

Joints of members for erection were designed on every beam at the locations about 1/6 of the span length apart from both ends. Erection joints for columns were spaced at every two or three stories. All joints were riveted at the field, whereas the fabrication of structural members and the joints of beams and columns was made at the shop either by using the "Union Melt" automatic welder or by hand weld. High tensile, friction bolts were partly employed for field erection joints of columns.

A typical configuration of beam-column joints is illustrated in Fig. 6, in which we can see the details of the flanges and the web plates of the structural members getting together and the locations where it is not possible to employ the submerged arc, automatic welding. Before deciding the most desirable shape of welding joints, trial tests have been carried out on the strength, workmanship, necessary working time and the inevitable deformation of plates due to heat of welding. Finally, the shape of welded joints, as shown in Fig. 6, was proved to be satisfactory for our purpose.

The stressed-membrane of the tower is also welded at the site by using carbonic acid gas semi-automatic welders. In this case, much attention was paid to the facilities and welding conditions, and before that a series of fatigue tests were carried out to find the desirable shape of grooves and to confirm the strength required. The shape of grooves for the joints of the stressed-membrane is shown in Fig. 7.

SUPPLEMENTARY REMARKS

The slab of the roof floor of the Kyoto Tower Building was reinforced further with horizontal X-bracings. This was done on the basis of making the roof floor slab a monolith to distribute the base shear of the tower uniformly in the events of earthquake and wind. Horizontal thrusts at the points of legs of the tower were to be carried by the tie-bars arranged on the level slightly above the roof floor.

Quite a large number of steel beams of the building had to have circular holes spaced at approximately equal distances on the neutral axis of the web, through which the pipe lines and ducts for air-conditioning can penetrate. Undoubtedly, the holes introduce the problem of stress concentration. Stress concentration is expected also at the small openings or at the corners of large openings in the tower structure. Therefore, every spot where the stress concentration is expected was stiffened with square bars or plates welded. In most cases, notches of a small radius have been avoided.

BIBLIOGRAPHY

1) Structural Standards Committee, Architectural Institute of Japan: "A.I.J. Structural Standards", pp. L-4 -- L-7, 1960.

2) Structural Standards Committee, Architectural Institute of Japan: "Structural Standards for Steel Stacks", Journal of the Architectural Institute of Japan, Vol. 79, No. 941, pp. 367-368, June 1964.

3) Kiyoshi Muto: "Seismic Analysis of Reinforced Concrete Buildings", Proceedings of the World Conference on Earthquake Engineering, Berkeley, California, pp. 34-1 -- 34-18, June 1956.

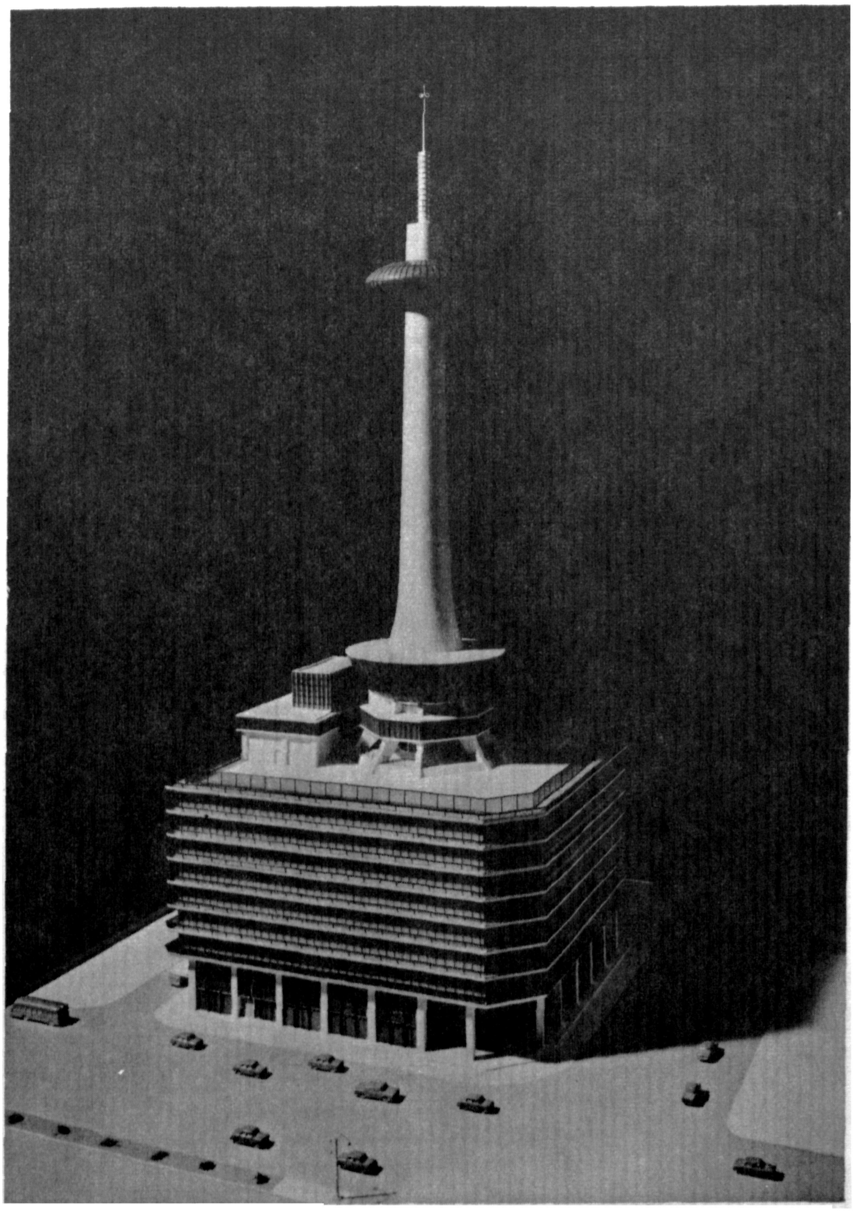


Photo. 1 Kyoto Tower Building

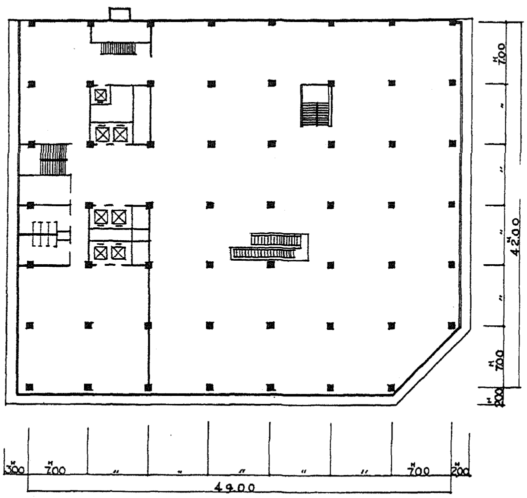


FIG.1–Typical Floor plan. of the Kyoto Tower Building

Note : For FIG.2 see page 529

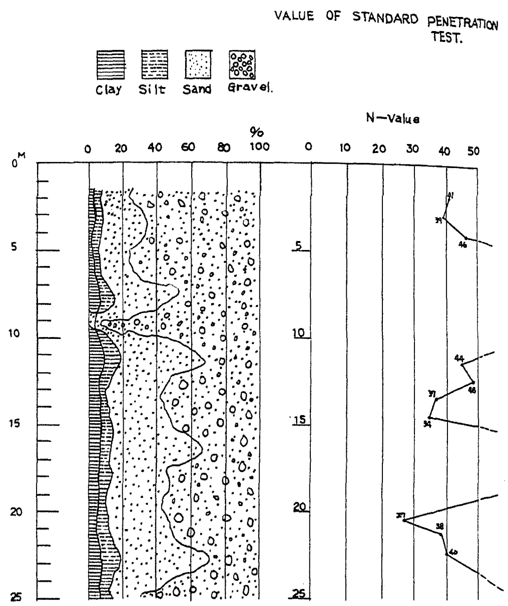
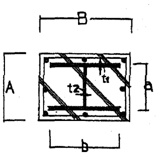


FIG 3–Subsoil Conditions and the N-value of the Standard. penetration Test.

Note : For FIG.4 see page 530

COLUMN



Section. of Column

(Hoop 9φ @ 300)

FL	A	B	a	b		t_1	t_2	
9	530	750	350	570	UE	19	16	8-D13
					C	16	"	
					LE	19	"	
8	"	"	"	"	U	19	"	"
					C	16	"	
					L	22	"	
7	"	"	"	"	U	22	"	"
					C	19	16	
					L	25	22	
6	580	800	400	620	U	25	"	"
					C	22	"	
					L	28	"	
5	"	"	"	"	U	28	"	"
					C	22	"	
					L	28	"	
4	"	"	"	"	U	28	"	"
					C	22	22	
					L	28	25	
3	580	850	400	670	U	28	"	"
					C	25	"	
					L	32	"	
2	680	950	500	770	U	32	"	"
					C	25	"	
					L	32	"	
1	"	"	"	"	U	32	"	8-D13
					C	25	"	
					L	32	"	
B1	1150	1150	"	"	U	32	"	20-D25
					C	25	"	
					L	28	"	
B2	1150	1150	"	"	U	28	"	38-D25
					C	25	"	
					L	"	"	

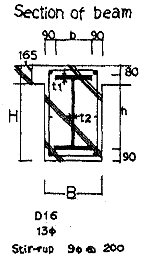
Abbreviation.

UE Upper end of Column
C Center of "
LE Lower end of Column
D Deformed-bar

FIG 5(a)–List of Cross-section of Columns---Unit: mm.

BEAM

Section of beam



FL	B	H	b	h	LO	t_1	t_2	Notation
R	500※1 400※2	720	300※1 220※2	550	E	22 16	6	SM 50B
					C	16	6	SS 41
9	450	720	270	550	E	19	6	"
					C	16	6	
8	"	"	"	"	E	19	6	"
					C	"	"	
7	"	820	"	650	E	16	9	"
					C	"	"	
6	"	"	"	"	E	19	9	"
					C	"	"	
5	"	"	"	"	E	22	9	"
					C	25	"	
4	500	"	320	650	E	22	9	"
					C	"	"	
3	"	920	"	750	E	22	12	"
					C	"	"	
2	"	"	"	"	E	25	12	"
					C	"	"	
1	550	"	370	"	E	25	12	"
					C	"	"	
B1	"	"	"	"	E	16	12	"
					C	"	9	

Abbreviation

FL Floor
LO Location
E End
C Center
※1 Upper flange
※2 Lower "

FIG.: 5(b)

List of Cross-section of Beams.---Unit: mm.

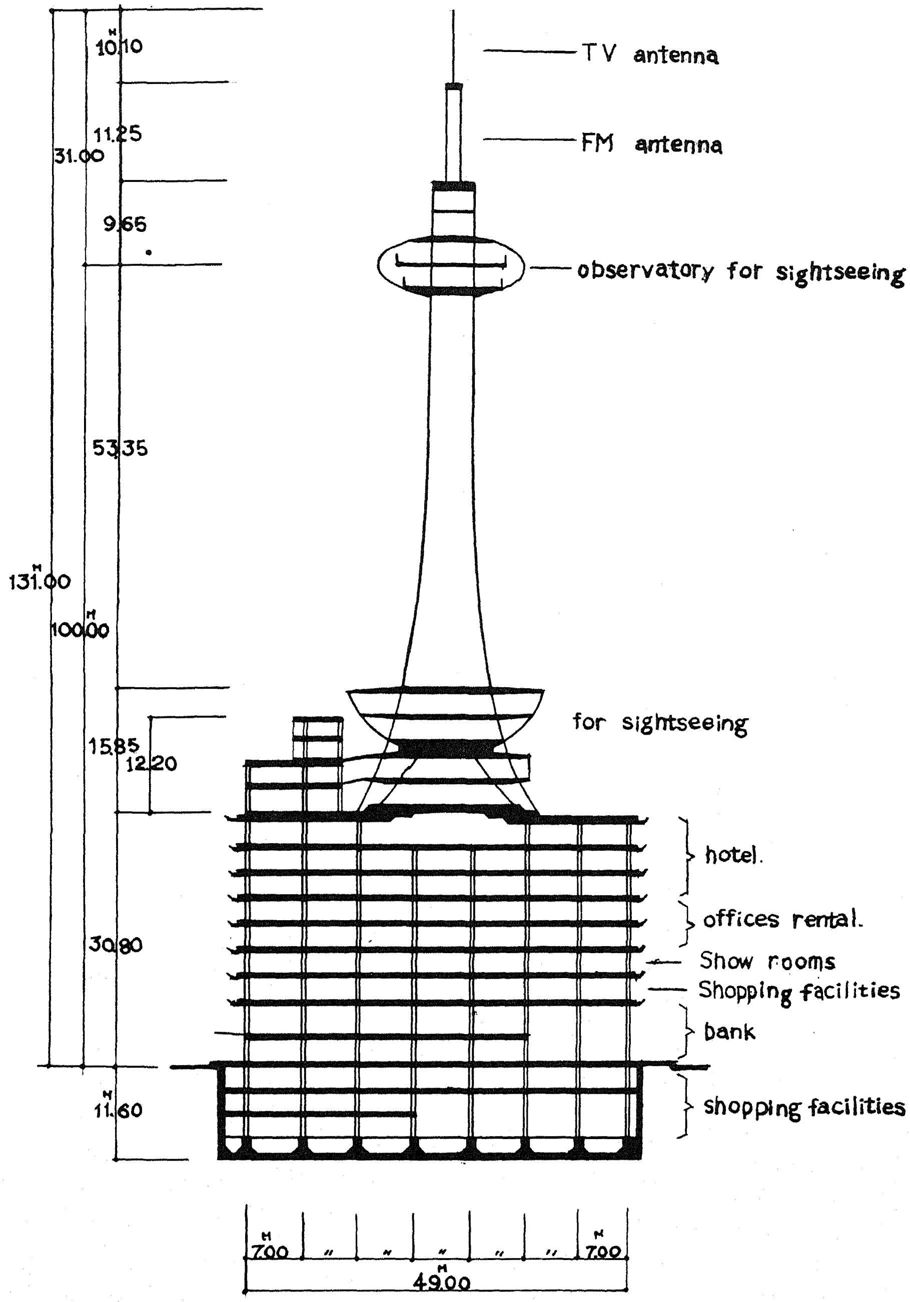
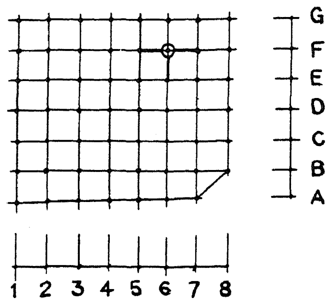
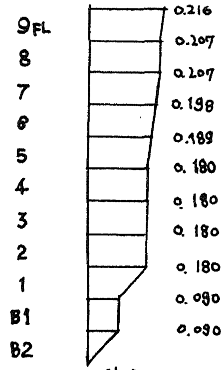


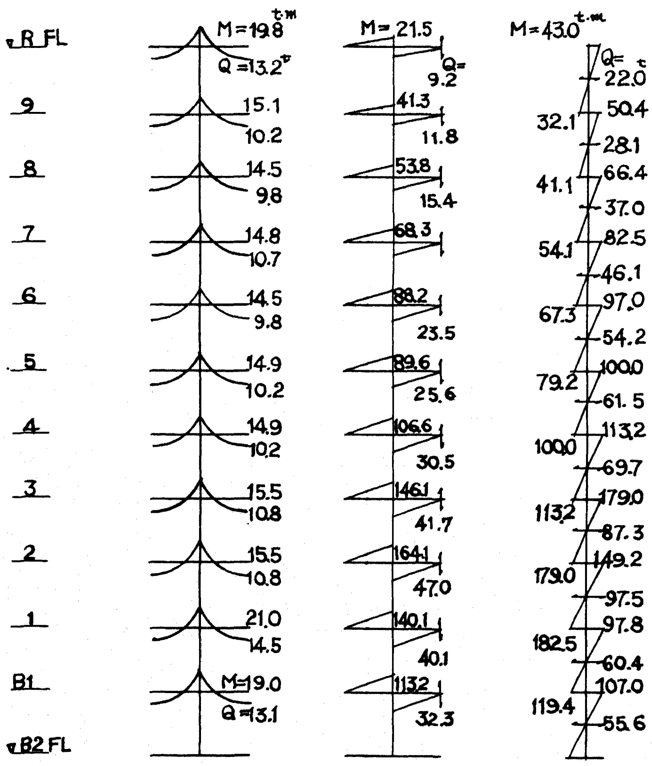
FIG 2 – Section of the Kyoto Tower Building



(a)
Layout of the Column Shown.



(b)
Distribution of Seismic Coefficient.



(c) (d) (e)

(c) Bending Moment Diagram for Beams Under Permanent Load.
(d) 〃 for Beams Under Lateral Load.
(e) 〃 for Column Under Lateral Load.

FIG. 4 – Design Stresses in the Structural Members.

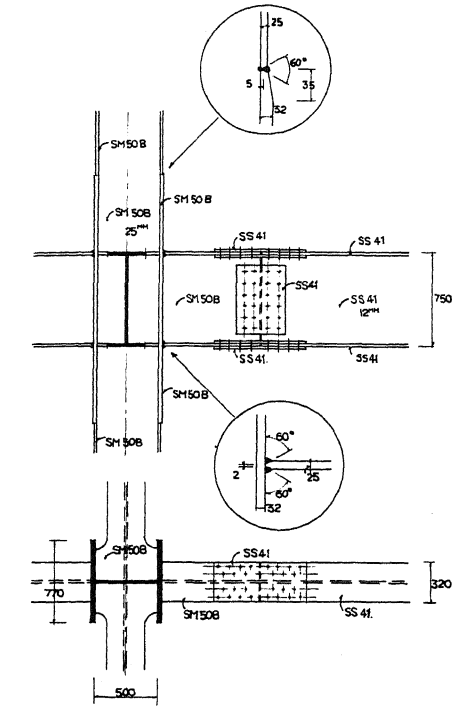


FIG.6-Details of Beam-column Joints and Erection Joints of Beams---Second Floor Beam--- Unit:mm.

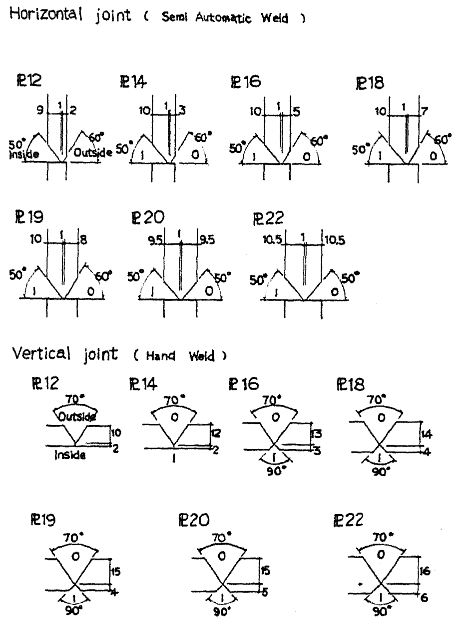


FIG.7 Details of Welded Joint at the Stressed-membrane of the Tower.

TABLE 1

Unit: kg/cm^2

	Roof			Penthouse Roof			Hotel & Office		
	D.L.	L.L.	T.L.	D.L.	L.L.	T.L.	D.L.	L.L.	T.L.
Slab	675	300	975	555	180	735	425	300	725
Frame	"	240	915	"	130	685	"	180	605
Lateral Load	"	130	805	"	60	615	"	80	505

	Shop			First Floor			B-1 Floor		
	D.L.	L.L.	T.L.	D.L.	L.L.	T.L.	D.L.	L.L.	T.L.
Slab	425	300	725	610	300	910	500	300	800
Frame	"	240	665	"	240	850	"	240	740
Lateral Load	"	130	555	"	130	740	"	130	630

TABLE 2

Unit: kg/cm^2

	Permanent Loading					Temporary Loading
	Tension	Compression	Bending	Shear	Bearing	
Steel SS41, SM41	1600	1600	1600	900	3000	1.5 times the value for permanent loading
Steel SM50	2200	2200	2200	1300	4100	
Rivet SV41	1600	—	—	1200	—	
Bolt SS41, SM41	1000	—	—	1200	—	
Steel Bar SS39	1600	1600	—	—	—	
Deformed Bar SSD49	2000	2000	—	—	—	
Concrete F_c= 180	6	60	—	6	—	Double the value for permanent loading
Light Weight Concrete F_c= 120	4	40	—	4	—	
Shop Weld SS41 Butt	1400	1400	1400	800	—	1.5 times the value for permanent loading
Shop Weld SS41 Fillet	800	800	800	800	—	
Shop Weld SM41 Butt	1600	1600	1600	900	—	
Shop Weld SM41 Fillet	900	900	900	900	—	
Shop Weld SM50 Butt	2200	2200	2200	1300	—	
Shop Weld SM50 Fillet	1300	1300	1300	1300	—	

TABLE 3

Location No.	Cross-section r (cm)	t (cm)	A (m^2)	Z (m^3)	Earthquake (t/cm^2)	Wind Parallel (t/cm^2)	Wind Normal (t/cm^2)
10	275	1.2	0.207	0.285	0	0	0
9	275	1.2	0.207	0.285	0.135	0.149	0.136
8	275	1.2	0.207	0.285	0.443	0.428	0.394
7	283	1.2	0.212	0.301	0.750	0.655	0.528
6	290	1.4	0.255	0.369	0.790	0.670	0.674
5	300	1.6	0.302	0.454	0.790	0.670	0.738
4	310	1.8	0.350	0.544	0.790	0.670	0.803
3	321	2.0	0.404	0.610	0.782	0.685	0.880
2	350	2.2	0.480	0.850	0.681	0.602	0.800
1	410	2.2	0.565	1.160	0.708	0.640	0.728
0	500	2.2	0.680	1.730	0.695	0.635	0.600