Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 71 (1994)

Artikel: Design of a long-span multistory building above a railway station

Autor: Kaino, Takaya / Jinbo, Kenji / Kuriyama, Michio

DOI: https://doi.org/10.5169/seals-54138

Design of a Long-Span Multistory Building above a Railway Station

Projet d'un immeuble élevé enjambant une gare ferroviaire

Entwurf eines weitgespannten Hochhauses über Bahnhofsgleisen

Takaya KAINO
General Manager
East Japan Railway Co.
Tokyo, Japan

Kenji JINBO
Manager
East Japan Railway Co.
Tokyo, Japan

Michio KURIYAMA
Assistant Manager
East Japan Railway Co.
Tokyo, Japan

Hideaki YAMAOKA
Manager
East Japan Railway Co.
Tokyo, Japan

Teruyoshi IIMORI
Chief Structural Eng.
JR East Design Corp.
Tokyo, Japan

SUMMARY

The paper reports on the design for a 12-story, long-span building over the tracks of a railway station in the centre of Tokyo. The cores bearing the building loads on either side of the tracks will be supported by diaphragm-wall foundations, and the span between them of approximately 55 meters will support a structure with no footing beams. For the main framework trusses, arches, stay cables, and polygonal suspension plates were considered. Static and dynamic seismic designs were tested The results of the investigation verified that all of these construction methods satisfied criteria relating to factors such as deformation.

RÉSUMÉ

Les auteurs présentent le projet d'un immeuble de 12 étages enjambant une gare au centre de Tokyo. Les deux noyaux du bâtiment s'appuient de part et d'autre de la voie ferrée sur des fondations en caisson, tandis que la construction intermédiaire suspendue franchit la voie d'une seule portée de 55 m. L'article présente l'analyse structurale des charpentes en treillis, des arcs porteurs, des haubans et des structures polygonales suspendues, du point de vue de leur comportement statique et sismique. Tous les systèmes porteurs satisfont aux exigences imposées, entre autres les déformations maximales admissibles.

ZUSAMMENFASSUNG

Es wird von Studien für ein zwölfgeschossiges Gebäude berichtet, das im Zentrum Tokios einen Bahnhof überspannen würde. Die Gebäudekerne werden beidseits der Gleise auf Tragwänden gegründet, währenddem die zwischen ihnen eingehängte Konstruktion mit ca. 55 m Spannweite keine Zwischenstützen aufweist. Für die Tragkonstruktion wurden Fachwerke, Bögen, Schrägseile und polygonale Hängetragwerke auf ihr statisches und erdbebenresistentes Verhalten untersucht. Alle Tragsysteme erfüllten die gestellten Anforderungen u.a. maximal zulässige Deformationen.

1. INTRODUCTION

There are few long-span, multistory buildings in Japan at present, because of the effects of earthquake motion. However, there is increasing desire to make use of the narrow vacant sites that are common beside the groups of tracks close to Tokyo's railway stations. We at the East Japan Railway Company, together with representative construction companies, have studied design methods for a long-span, multistory building which has two core frames on sites on either side of the tracks of a certain station, with no footing beams.

During our studies of this long-span, multistory building, we examined several types of main framework to support the part of the building that bridges the tracks and connects the core frames on either side. These main framework types are super truss, arch, stay cable, and polygonal suspension plates. Details of our investigations are given in this paper.

2. OUTLINE OF STRUCTURE

2.1 Basic Structure and Geology

The building will be approximately 60 meters tall (12 stories), 70 meters deep (across the tracks), and 75 meters long (parallel to the tracks) as could be imagined in Fig. 1. The length of the span over the tracks will be approximately 55 meters, and the width of each core frame will be approximately 7 meters (Fig. 2). The foundations will be a diaphragm wall under each core frame, in diluvial deposits.

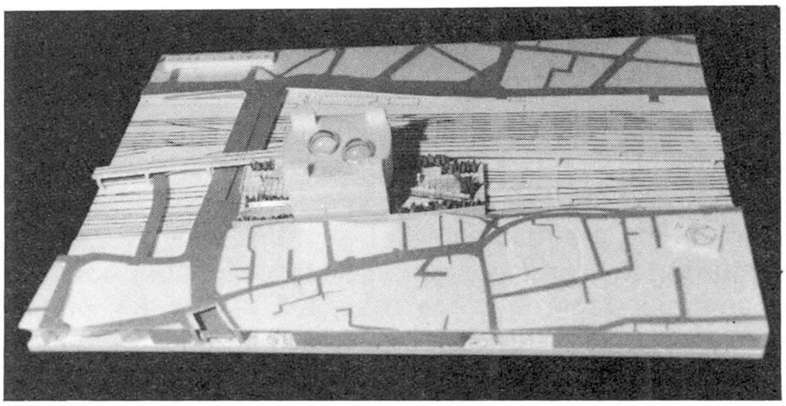


Fig. 1 Aerial photograph of proposed site

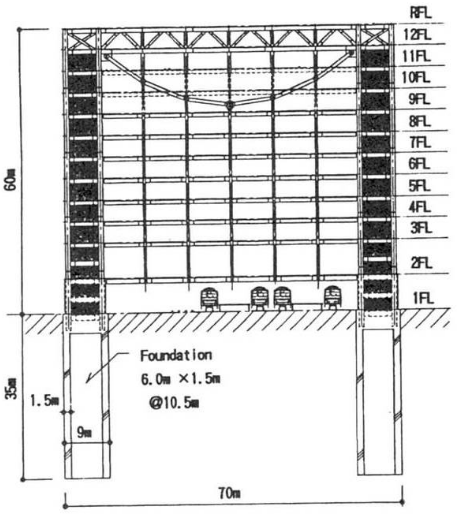
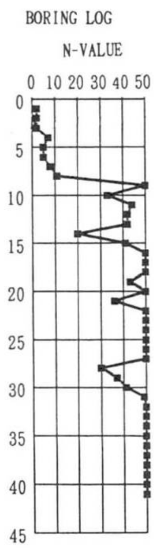


Fig. 2 Section through proposed building

The site that we surveyed is sandy to a depth of 3 meters from ground level, then consists of sand alternating with layers of clay and sandy gravel to a depth of 35 meters below ground level. Two proposals of a sandy layer down to 21 meters ground level or a sandy gravel layer down to 35 meters ground level were considered as the load-bearing subsoil of the foundations under vertical loading, but the biggest effect on horizontal displacement at ground level of the foundations during an earthquake is depth of setting of the foundations rather than changes in stiffness of the foundations, so the depth of foundations was taken to be 35 meters from the ground level.

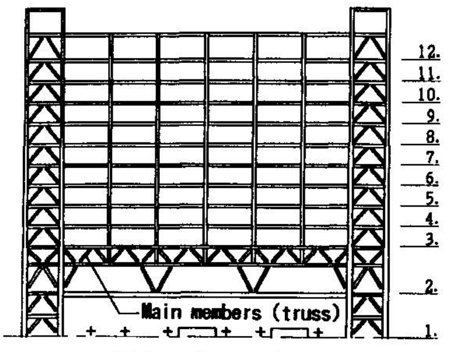


DEPTH (m)	SOIL TYPE	LAYER (m)	NUMBER OF BLOWS (BLOWS/30cm)	MODULUS OF DEFORMATION (MPa)
3.0	Sand	3.0	5	
7.5	Clay	4.5	8	0.306
15.0	Sand	7.5	40	0.275
18.0	Sand & Gravel	3.0	50	0.265
28.0	Sand	10.0	50	0.408
35.0	Clay	7.0	25	0.510
41.5	Sand & Gravel	6.5	50	

Table 1 Results of soil borings

2.2 Framework Characteristics

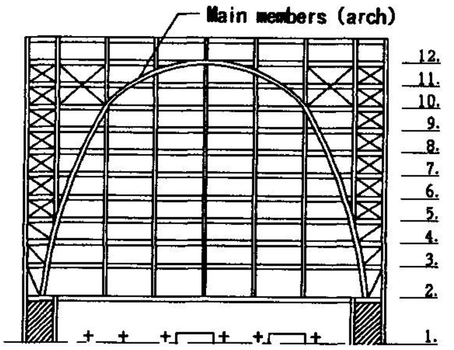
The characteristics of each type of framework that we investigated are described briefly below.



A. Truss framework

Making the first and second floors of super truss frames provides concentrated support for vertical loads. From the third floor upwards, the inner portions have a rigid-frame construction, with earthquake-resisting braces on the core frame portions on either side.

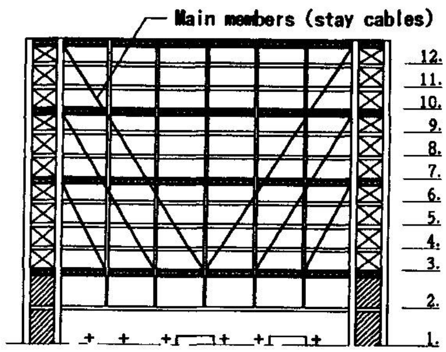
Main columns (1st floor):	Box 1200 × 1200 × 60 × 60 (SM490)
Intermediate columns:	BH 600 × 600 × 36 × 40 (SM490)
Main beams (2nd floor):	2BH 1000 × 500 × 40 × 40 (SM490)
Truss members:	2BH 600 × 500 × (25–32) × (32–60) (SM490)



B. Arch framework

An arch is characterized in bearing both vertical and horizontal loadings. Since arches have a large cross-section and are highly rigid, bracing is provided to resist reverse shear stresses, particularly in the upper floors.

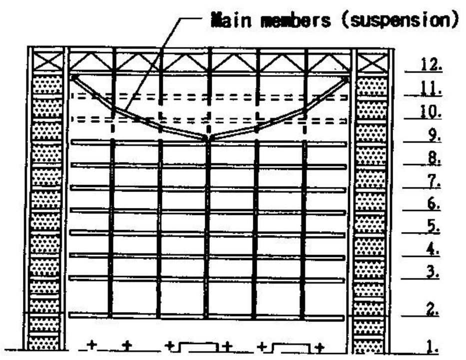
Main columns (1st floor):	B × D = 2000 × 8000 (RC)
Intermediate columns:	Box 900 × 700 × 40 × 40 (SM490B)
Main beams (2nd floor):	BH 1000 × 400 × 19 × (28–40)
	BH 1000 × 500 × 19 × 40 (SM490B)
Arch members:	I 1500 × 900 × 80 × 80 (SM570Q)



C. Stay-cable framework

This structure is characterized in that the part of the multistory building above the tracks is suspended on cables from the core frames at either side. Beams bear the compressive forces, and the cables together with the core frames acts as effective aseismic elements.

Main columns (1st floor):	B × D = 1500 × 1500
	Box 900 × 900 × 65 × 65 (SM490A)
Intermediate columns:	Box 600 × 600 × (16–40) (SM490A)
Main beams (3rd floor):	2BH 800 × 400 × 22 × 36 (SM490A)
Cable members:	2SPWC-367, 283, 301



D. Suspension framework

The suspended members bear only vertical loads in the part of the building above the tracks—they are not intended to bear horizontal loading. Therefore, where the suspension members intersect the beams and columns in the part of the building above the tracks, those beams and columns are paired to allow the suspension members to move freely.

Main columns (1st floor):	Box 1200 × 1000 × 80 × 80 (SM490A)
Intermediate columns:	Box 550 × 550 × 22 – 28 (SM490A)
	(Floors 2 to 8, 11)
	2-Box 250 × 250 × 25 (SM490A)
	(Floors 9 and 10)
Main beams (2nd floor):	BH 900 × 300 × 19 × 25 (SM490A)
Suspension members:	4PL-750 × 100 (HT80)

Fig. 3 Framework characteristics

3. ANALYSIS PROCEDURE AND CONDITIONS

The flow of the design procedure we followed is shown in Fig. 4.

3.1 Static Analysis

We subjected a two-dimensional frame model of the upper structure coupled with the diaphragm-wall foundations to linear stress analysis, applying vertical loading and static earthquake loading determined from results of preliminary response analysis, and investigated the effects of the sizes of members. We determined subsoil reaction coefficients of the diaphragm-wall foundations by comparing the results of several horizontal loading tests on this type of foundation and results obtained by independent finite element analysis.

3.2 Dynamic Analysis

As the initial step, we substituted the static analysis model into an equivalent spring-mass model for a linear response analysis, and determined that the model satisfied the criteria we had set. Two sets of earthquake wave data were input, El Centro 1940 (N–S) and Taft 1952 (E–W), with the maximum velocity being set to 25 cm/s (level 1).

In the next step, we introduced nonlinear characteristics into the members of the static analysis model and performed an incremental lateral loading analysis at two to three times the static earthquake forces. The resultant restoring force characteristics were obtained for each floor, we then performed nonlinear response analysis with the spring-mass model, and we verified that the design criteria were satisfied. In this case, the maximum velocity of the input earthquake waves was 50 cm/s (level 2).

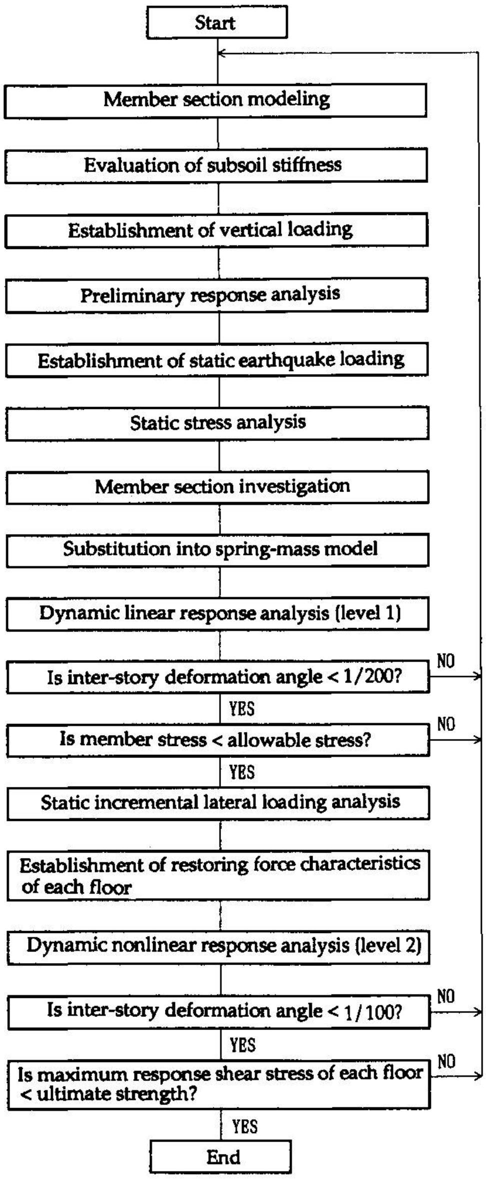


Fig. 4 Design flow

4. RESULTS

The results of the analysis are discussed below.

4.1 Static Analysis (Vertical and Horizontal Loading)

Vertical displacements of the beams under vertical loading at the center of the second floor above the tracks are listed in Table 2.

When the arch and suspension structures were subjected to vertical loading, the members subjected to axial forces resisted the loading, so these displacements were less than with the truss method in which the bending members provide resistance. These values were approximately half or less of the truss method. These values were also less than about 1/800 of the span across the tracks, so would cause no problems. Increasing the stiffness of the structural members is one way of making these displacements smaller that is common to all these methods. For each of these methods, it will be necessary to investigate the handling of displacements further, considering details such as the setting of beam camber during construction and the building sequence.

(cm)

	Truss	Arch	Stay Cable	Suspen-tion
Vertical Displacement	16.7	7.9	*6.5	6.8

* Live Load Only

Table 2 Magnitudes of vertical displacements (2nd floor)

Partial ratios of story shear forces during an earthquake are listed in Table 3. With the truss method, 60% to 70% of the earthquake force is taken by the core portions on ordinary floors. With the arch or stay cable method, their main members pass through even the ordinary floors, so the arch or stay members bear a large proportion of the story shear forces. In the arch method in particular, since the arch members in the upper stories are close to the horizontal, they can bear large shear forces of over 100% so that reverse shear forces can occur in the other parts of the framework. On the other hand, with the suspension method, the core portions and intermediate beams are pin-jointed, so that the suspension members have joints that do not bear any story shear forces, and thus the core portions bear virtually all of the story shear forces.

Static horizontal displacements of buildings of each method under earthquake loading are shown in Fig. 5. We performed analysis on the upper structure coupled with the diaphragm-wall foundation to 35 meters below ground level that all of these construction methods have in common, as described above. Movement of the diaphragm-wall foundation shows a tendency toward roughly rigid-body rotation that is common to all methods. Horizontal displacement of the tops of the foundations was 2.1 to 2.9 cm, and this value was verified to be sensitive to the subsoil reaction coefficient. The lack of footing beams connecting the tops of the foundations has a huge effect on the upper structure and the building's natural period. Since the sizes of most of the members are determined by the stresses they experience during vertical loading, in effect the upper framework becomes extremely strong, and thus the inter-story deformation angle can sufficiently satisfy the condition of no more than 1/200 radians. In the upper framework deformation mode, the characteristics of each method vary with differences in the main member arrangement and the story stiffness distribution.

Story	Truss			Arch			Stay Cable			Suspension		
	A	B	C	A	B	C	A	B	C	A	B	C
12.	30	70	0	25	75	0	-33	28	105	98	2	0
10.	57	43	0	-35	-27	162	25	19	56	75	25	0
8.	65	35	0	-23	20	103	21	19	60	83	17	0
6.	72	28	0	9	32	59	22	15	63	86	14	0
4.	79	21	0	20	39	41	29	13	58	87	13	0
2.	-18	0	118	38	27	35	99	1	0	95	5	0
1.	100	0	0	100	0	0	100	0	0	100	0	0

(*) A: Core frame, B: Intermediate column, C: Main member

(Unit:%)

Table 3 Story shear force partial ratios

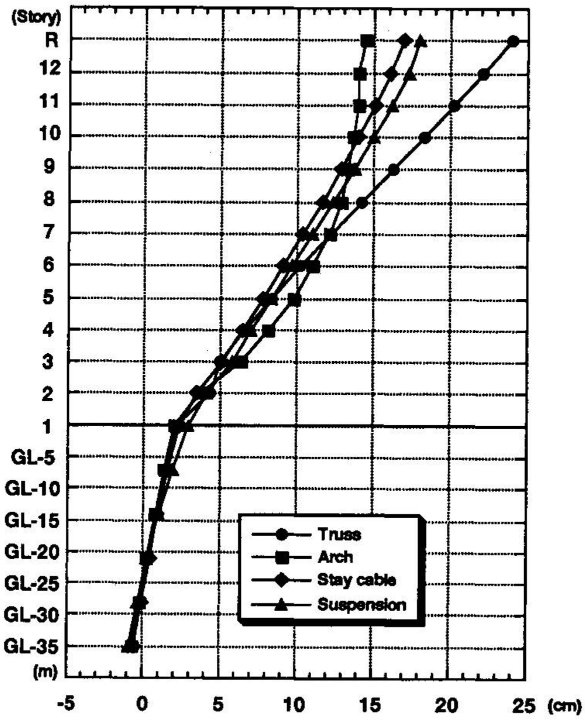


Fig. 5 Static horizontal displacements

4.2 Dynamic Analysis (Levels 1 and 2)

The primary natural periods for each structural method were within the range of 1.4 to 1.5 seconds, as shown in Table 4. In comparison with an ordinary building in the 60-meter-high class, this primary natural period is fairly long, because there are no footing beams. Looking closely at each natural period, it is clear that the arch and stay-cable methods, which impart horizontal stiffness to their members, have a slightly shorter period than the other two methods.

The dynamic analysis was done on a multiple mass model using the equivalent shear springs obtained as a result of static elasto-plastic analysis. In the first-floor columns, subsoil sway springs were considered.

Displacements of the tops of the diaphragm-wall foundations of each of the methods during response to 50 cm/s Taft (E-W) waves are shown in Table 5. There was scattering between the different methods, but displacements were within the range of 3 to 6 cm.

Truss	Arch	Stay Cable	Suspension
1.51	1.44	1.42	1.48

(Unit:sec)

Table 4 Primary natural periods

Distributions of inter-story deformation angle of each of the methods during response to 50 cm/s Taft (E-W) waves are shown in Fig. 6. It was verified that the design criterion of 0.01 radians was satisfied by each structural method. The suspension method tends toward an even distribution with height because the main members have joints that do not contribute to the horizontal stiffness. The other three structural methods exhibit the characteristics that are specific to those methods. For example, a singular point can be seen at the position of the second story in the truss method's case or the third story in the stay-cable method's case, in other words, at the position of that method's main members. With the arch method, a tendency toward decreasing deformation angle can be seen from the third floor upwards, as the horizontal stiffness of the arch increases.

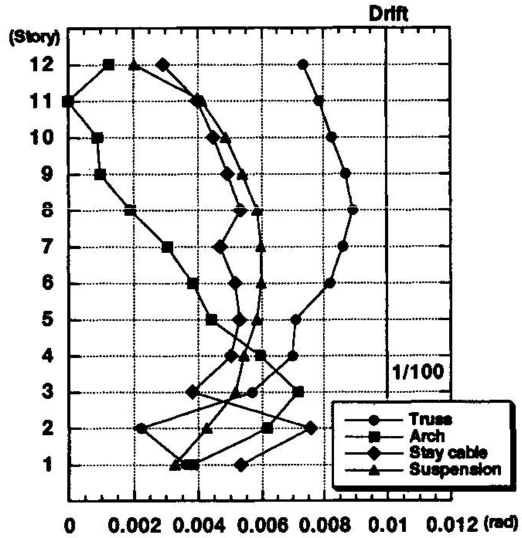


Fig. 6 Inter-story deformation angles

The distribution of story shear coefficients during response to 50 cm/s Taft (E–W) waves is shown in Fig. 7. With the truss method, the shear coefficient rises with floor in the upper framework, but this is due to the way in which the upper stories above the third floor become a rigid-frame structure.

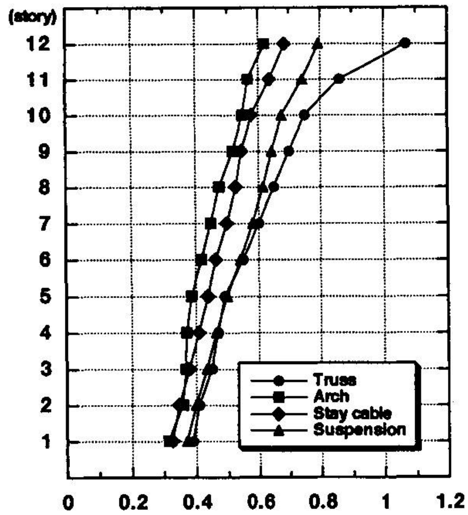


Fig. 7 Seismic story shear coefficients

5. CONCLUSION

Aseismic design in an earthquake-prone country such as Japan necessitates a fair amount of compromise and decision-making in the proposal of a structural framework that suits a certain design concept, when the design takes into consideration factors such as safety, economics, and ease of construction.

This paper has presented the results of our investigations into different structural forms that are aimed at creating a large-span building designed to make effective use of the vacant areas alongside railway tracks. It also clarified that there are some differences in efficiency and functionality of different structural methods, but that they can be implemented.

Truss	Arch	Stay Cable	Suspension
3,96	3,09	3,76	5,50

(Unit:cm)

Table 5 Displacement of diaphragm-wall foundation tops

Such a building would require rather more steelwork than an ordinary rigid-frame structure, but we have determined that it is possible to build a multistory structure in the space over the tracks in order to create a long-span building that does not impede the functions of the railway.

We intend to intensify our investigations in the future, to implement choices and decisions for an even better structural format.

6. ACKNOWLEDGMENTS

The contents of this paper were put together by the "Long-Span Building Design Implementation Investigative Committee" and the author would like to express his sincerest thanks to everyone involved, from committee chairman Jiro TAJIMA and vice-chairman Shigemi MACHIDA to the designers who participated from the construction companies Obayashi, Kajima, Shimizu, Taisei and Tekken ,and JRC.