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VI c

Utilisations nouvelles, comprenant les constructions sous-marines et flottantes

Neue Anwendungen einschliesslich Unterwasserbauten und schwimmende Konstruktionen

New Applications including submerged and floating Structures

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Precast Pretensioned Girder Bridges with Continuous Situ-cast Decks and Diaphragms

Ponts à poutres précontraintes d'éléments préfabriqués avec tablier continu et diaphragmes coulés sur place

Vorgespannte, vorfabrizierte Balkenbrücken mit an Ort hergestellten durchlaufenden Fahrbahnplatten und Querträgern

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1. Introduction

Utilization of simply supported precast pretensioned girder bridges in Japan started about in 1955. Since then a large number of this kind of the bridges with about 20m-spans or less have been constructed.

In highways the precast girder bridges were used relatively large in number in the early days, but it was found that many expansion joints were not desirable for riding quality and smooth passage of vehicle, in addition to difficulties of assuring good maintenance of highways. Therefore instead of precast girders the situ-cast continuous bridges have been often employed. But recently precast girders have been again approved more excellent with respect to productivity and quality control. And the method has been developed which enables to construct continuous girder bridges with precast members by casting in situ the decks and diaphragms over intermediate piers.

Thus, in planning multi-span viaducts of about 20m-spans in the urban areas, priority is now usually given to this type of bridges.

This paper reports how to connect precast girders over intermediate piers, the design concept and the flexural test results.

2. Construction method of girder connection for continuity

In Japan there are the standardized precast pretensioned beams available for simply supported beam bridges. The cross section of beam with, say, about 20m-spans is shown in Fig.1. And the bridge is composed of the beams placed in position at spacing of 1.05m and prestressed transversely through the diaphragms and slabs.

In connecting the precast beams the similar method is used.

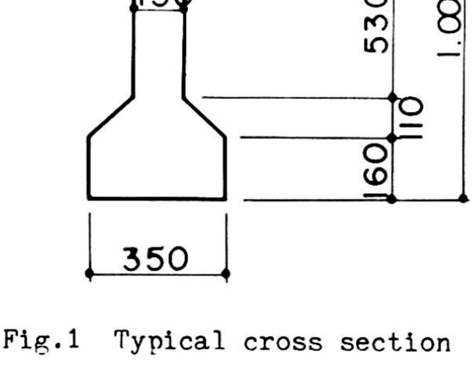

Fig.1 Typical cross section

Details of the method are as fol-

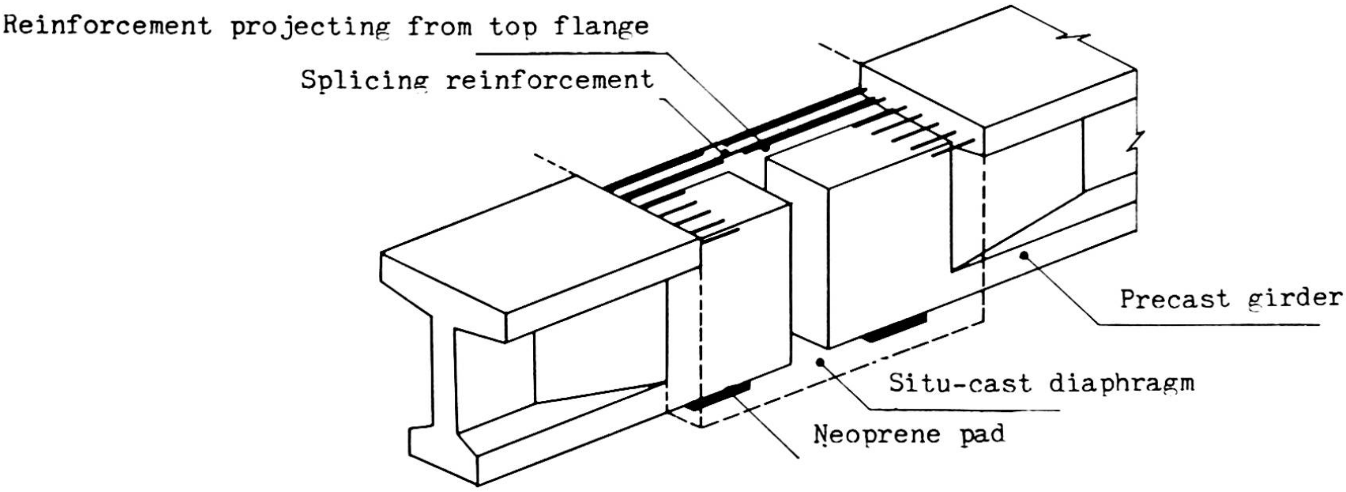

Fig.2 Schematic view of continuity zone

lows; The end of precast girders is designed as schematically shown in Fig.2. Continuity reinforcements are arranged longitudinally in the upper flanges of precast girders and along the projecting length the additional splicing reinforcements are lapped. Then concrete is placed in situ into fairly massive diaphragms including both ends of the precast girders. Then by using lateral prestressing steels provided in the diaphragms the adequate prestress is transferred to the connection zone.

The bearing shoes used temporarily for precast girders remain left behind in order to serve for the continuous girders when the connection has been completed. Therefore there exists a pair of shoes located slightly apart on each pier top.

3. Design concept on continuity

The amount of continuity reinforcement over piers may be not always enough compared to that of conventional continuous girders, because , as described before, the reinforcement for continuity are placed only in top flanges.

The zone of continuity tends to crack at lower load level than the other zones do and so it can be said that the former has the lower flexural rigidity. It is considered better to analyse this type of bridge as a group of simply supported beams, even though it behaves almost as continuous girder. In this respect it is different from conventional continuous girder.

A pair of bearing shoes are used by taking into consideration the fact that the connection zone can be constructed with more ease and that the safety of bridge could be assured even if the continuity of girder is completely lost. And a pair of bearing shoes, which is usually specified to use neoprene, is expected to work together alike one shoe because of their elastic deformation in vertical direction. It is found desirable by the analysis that the intensity of elastic constant of neoprene pads is less than 350 t/cm. This condition may be always satisfied when the pads are designed conventionally to correspond to the reactions and deformations of simply supported beams. The amount of continuity reinforcement that carries the loads (dead loads such as pavement and handrails, and live loads) imposed on the bridges after the completion of continuity, is calculated by the conventional reinforced concrete theory on the assumption that there exists only one bearing shoe on each pier top.

Generally, in case of spans about 20m long, deformed reinforcing bars of 22 mm dia. are placed at spacing of 10cm, so that the calculated stress of bars is less than 1600 kg/cm^2.

It is expected that the secondary bending moment will take place due to creep of precast girders, resulting tensile stresses in the lower parts of girder sections in case of standardized precast girders. That is, it works to relieve the negative moment acting on the zone of continuity, and furthermore the secondary deformation due to creep is estimated to be less than the deformation of simply supported girders. Because of this reason the moment due to creep is not fully taken into consideration. However, some amount of reinforcement is provided in the lower part of the diaphragms in the direction of bridge axis in order to resist this creep moment.

Thus the precast pretensioned girder bridges with situ-cast continuity zone over piers constructed in this fashion is substantially of simply supported beam system, although progressively taking into consideration the incompleteness of continuity.

This system has both the advantage of simplisity in construction like simply supported beam system and the same good riding quality as assured in continuous beam system.

4. Test results

As described above, this system of bridge can be considered as continuous beam system from riding quality, even though it has incomplete continuity zone from view of design concept.

Consequently, the zone of continuity should have sufficient flexural rigidity so that pavement on both sides of it can be kept in good condition under service load.

A series of tests were carried out to make sure the point as indicated above. The main purpose of these tests is to observe how the test beams crack under service load or so, and to pursuit their behaviors until failure.

Three test beams prepared were 9m long with the same cross section, reinforcement and continuity zone at center of spans as those of actual beams. The method of bending test of beams is shown in Fig.3.

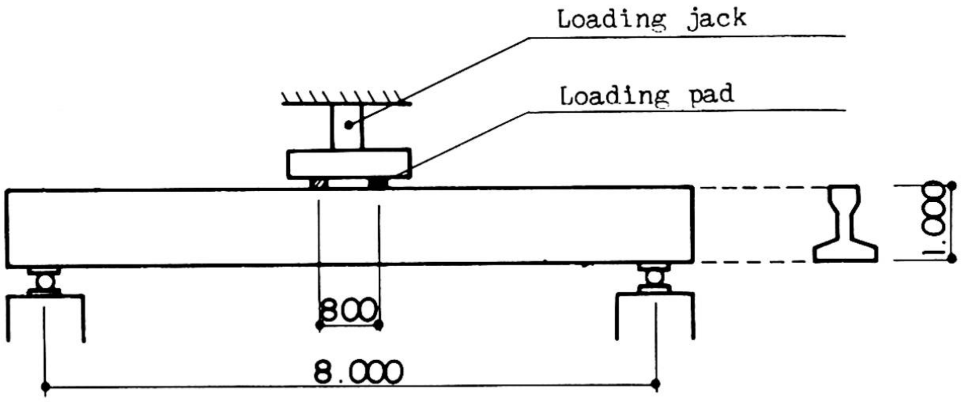

Fig.3 Loading method

The test results are as follows;

(1) Cracks were observed, firstly at both construction joints in decks, then in decks between both joints and finally in decks outside them. The load level at which cracks were observed at each stage were clearly different.

Now, the service load of the test beams is designated as P-1800, which means the load at which the stress of continuity reinforcement has reached 1800 kg/cm^2 as calculated by the conventional theory, while the allowable stress of reinforcement in actual beams is specified to 1600 kg/cm^2.

The initial crack at construction joints in top flanges occurred before the stress of reinforcement reached 1800 kg/cm^2. But the crack width remained almost unchanged thereafter, and at P-1800 the width was less than 0.1mm. At

P-3000 (the load at which the stress of reinforcement has reached the nominal yield stress 3000 kg/cm^2) the width was less than 0.2mm. In the zone between the two construction joints, no crack was found even at P-1800, and first crack occurred at the calculated stress level of 2000 to 2400 kg/cm^2.

Then the load was removed. When the beams were reloaded, the crack width in the zone between both construction joints was 0.1mm at P-1800, and became rapidly as large as 0.4mm at P-3000. This crack developed and caused the beams to fail finally as shown in Fig.4. Outside the both joints, there existed no crack before P-3000.

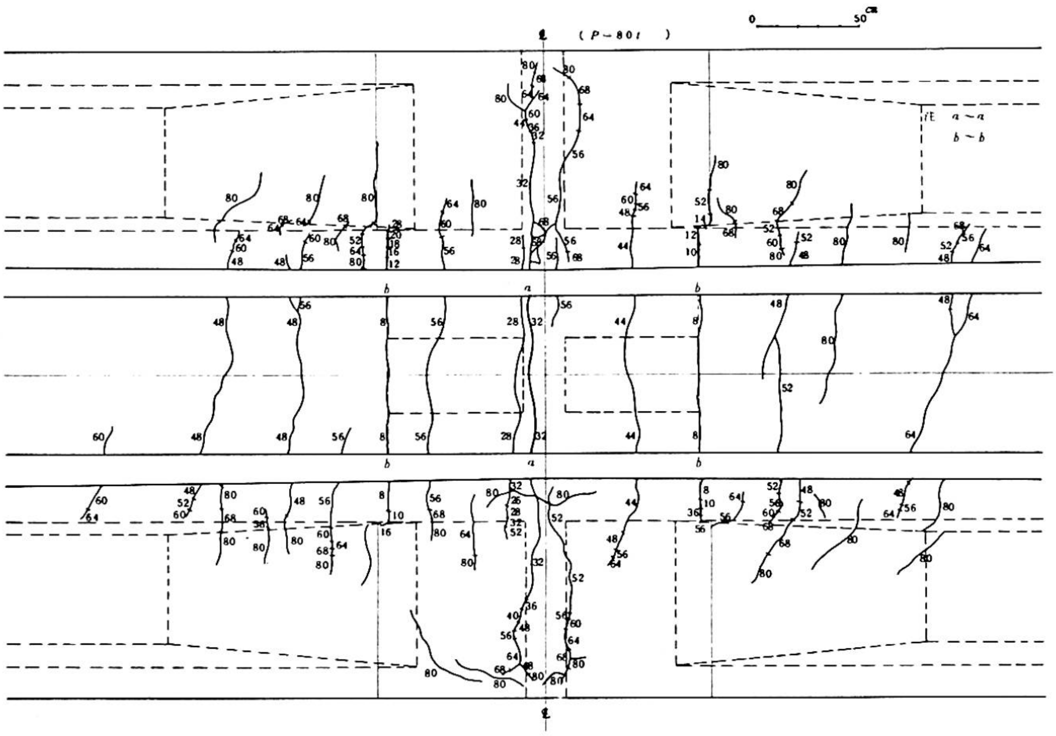

Fig.4 Typical final crack pattern

(2) The reduction in flexural rigidity at the zone of continuity as load increased was estimated by measuring deflections of test beams. The crack at construction joint in top flanges gives little influence on the flexural rigidity of beams. The flexural rigidity of test beams began to decrease after the cracks occurred in the middle part between both joints at the load higher than P-1800 as indicated in Fig.5. Until the test beams failed any slippage was not observed in the splicing reinforcement having lap lengths of over 30 times dia. each along the reinforcement projecting from top flange of girders.

The test beams failed due to excessive compression of top flange after the tip of crack between both joints had reached almost the top fiber. No abnormal behavior to be attributed to the performed continuity was observed throughout these tests.

The description above is the excerpts from the test results, which show that the zone of continuity does not give any worse influence on pavements at service load level and at the same time that not any special behavior were observed which might be resulted from the performed continuity. Consequently it is concluded that the precast girder bridge with this kind of the continuity works completely well.

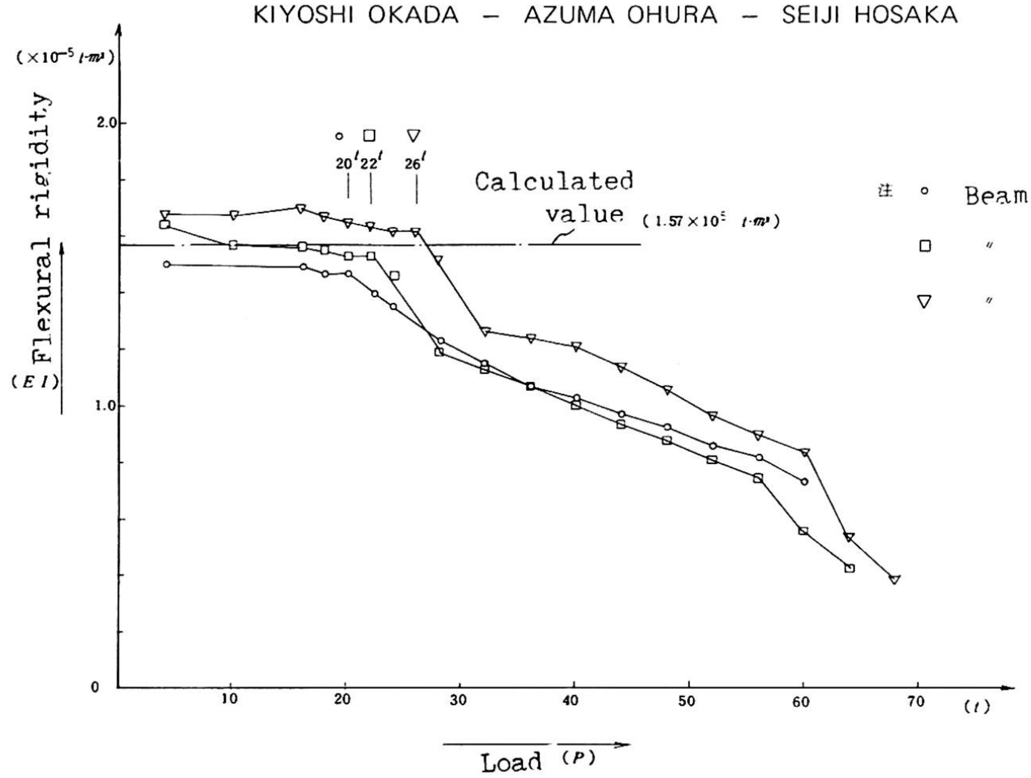

Fig.5 Reduction of flexural rigidity estimated by deflections measured

5. Remarks

A large number of viaducts have been constructed in urban areas in Japan. The engineers concerned must pay attention for the people living along the job sites to the shortening of construction periods and various pollution problems during construction.

From this point of view, the utilization of factory-made precast beams as reported here is to be approved to a large extent. The authors are happy if this report may give any help to the engineers concerned.

SUMMARY

The precast pretensioned girder bridges with continuous situ-cast decks and diaphragms over intermediate supports are the structures which behave actually as indeterminate continuous beams, but which are assumed to consist of determinate simply-supported beams with respect to structural analysis. Consequently they are completely different on this point from conventional continuous beam bridges. The method of girder connection for continuity, the design concept and the flexural test results are presented.

RESUME

Les ponts à poutres précontraintes d'éléments préfabriqués, avec tablier continu et diaphragmes coulés sur place sont des structures qui se comportent en fait comme des poutres continues indéterminées, mais qui sont considérées dans le calcul comme des poutres simples statiquement déterminées. En ce qui concerne l'analyse, ces ponts sont donc complètement différents des ponts conventionnels à poutres continues. La méthode d'assemblage des poutres pour assurer la continuité, la conception du projet et les résultats des essais de flexion sont présentés.

ZUSAMMENFASSUNG

Vorgespannte, vorgefertigte Balkenbrücken mit an Ort hergestellten durchlaufenden Fahrbahnplatten und Querträgern verhalten sich praktisch wie durchlaufende statisch unbestimmte Träger. In der Berechnung wird allerdings statisch bestimmte Lagerung als einfacher Balken vorausgesetzt. In dieser Beziehung unterscheiden sich solche Brücken demnach von normalen konventionell hergestellten Tragwerken. Die Verbindung der Balken über den Zwischenstützen, die grundlegenden Entwurfs-Gedanken sowie die Ergebnisse eines Biegeversuchs werden dargestellt.

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Precast Continuous Span Structures for Highway and Urban Bridges Made of Completely Prefabricated Segments

Poutres continues en béton armé pour les ponts routiers et urbains en voussoirs préfabriqués

Durchlaufende Stahlbetonüberbauten für Autobahn- und Stadtbrücken aus vorfabrizierten Fertigteilen

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A now developed continious span construction is an economical solution for multiple - span bridges and viaducts. with spans 21-63 m long. It is a precast, prestressed slab beam structure (abbr PRK) made of segments with maximum prefabrication. This construction is mounted span by span with a movable scaffold carrier.

There is a difference between slab-beam types of structure widely used abroad in continious in-situ constructions and precast ones.

Every PRK span consists of a number of composite segments laid out in length and covering the whole carriageway in width. PRK cross-section looks like two longitudinal bulky beams with their associated slabs above. This kind of structure is simple in shape, easy in fabrication, as the bulky beams are good for winding the cables and casting the concrete. At the ends of each segment there are cross stiffeners 50-60 cm high. (See Fig. 1,2). Such a design solution increased greatly the lateral rigidity of a segment necessary for three dimensional behaviour of the structure when in service, transportation and installation. It led to modification of the scheme of three-dimensional behaviour of a carriageway slab because now it is thinner and less voluminous in comparison with slab in in-situ constructions and here it behaves like a small span. The area of segments to be glued is extended. Therefore sticking is more secure.

All technological advantages of slab-beam cross-sections are inherent in PRK. But the PRK unlike analogical solutions in the west has better characteristics than typical multiple stiffened prestressed span structures in use due to positive features mensioned above.

PRK continious span consists of segments glued between each other and fastened by tendons running through ducts. Every span is nothing but a section, middle sections being of the same length as a span. One extreme section is shorter, the other longer than a span. Adjacent sections are coupled in-situ at a mean distance from a pier equal to 0.2 length of the span and fixed with cables. For transportation of segments by rail ways their dimension in longitudinal plane is taken within 3 m. Their weight is to 45 t. The PRK is designed in such a way that makes it possible to fabricate segments of different width and height in one and the same formwork. But the space between main beams and the width of the middle slab are uniform. For different spans and carriageways the beam webs and slab cantilevers may differ.

The upper surface of segment slabs has a cross fall for drainage. Water-proof insulation may be laid down in shops. Under field conditions waterproofed is only glued joints. PRK types of segments are easy to apply in structures for crooked and skewed portions of highway.

All major operations as for fabricating PRK superstructure are made at works. The rate of prefabrication is about 98%. In-situ operations comprise:

concrete casting of section joints ;
epoxygluing of segment joints;
tension of high-strength cables;
paving with bituminous concrete and
laying down a part of water-proof insulation.

Continious superstructure is assembled span by span from one end of a bridge. A span structure is assembled section by section on the movable scaffold carrier (Fig.3).

As a rule scaffold carriers do not occupy the space under a bridge, because span structures repose on them by their lateral stiffeners.

Thanks to segment features one can use the positive qualities of carriers, when located underneath as well as more effectively glue segment joints, prestress the reinforcement and improve the interaction between span structure and movable scaffold carrier (Fig.2).

Main peculiarity of a movable scaffold carrier lies in the fact that the heavy transshipment crane necessary for assembling segments of the next section does not stand on it (Fig. 3). The segments within the next section are transported by rail on the upper strips of the scaffold carrier. That is the main difference between this crane carrier and movable carriers widely used abroad. It has a considerably less weight of metall per 1 m^3 of concrete of a deck.

The movable scaffold carrier looks like two metall girders made of high-strength steel. It consist of welded sections, each 21 t. of weight. On a construction site these sections are tied into a single structure with high-strength bolts. The scope of this carrier is to mount spans from 33 to 63 m long.

There are winches here for hauling segments.

On the launching nose there is a cart (load capacity 20 t) for erecting next pier. Inside and outside the girders there are special gangways adjusted up and down. Besides that there is a suspended cart with jacks for tensioning tendons. The scaffold carrier moves along a deck on special carriages fixed to permanent piers. Camber is regulated by rails.

Fig 3 illustrates schematic erection of a span structure. The scaffold carrier is assembled on the approaches to the bridge on one of its slopes and pushed forward into the first span.

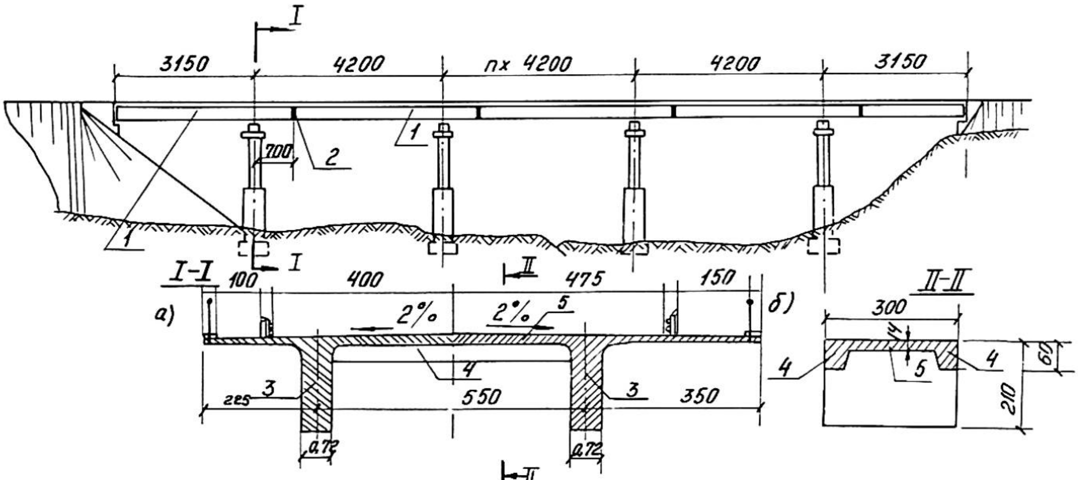

Fig 1

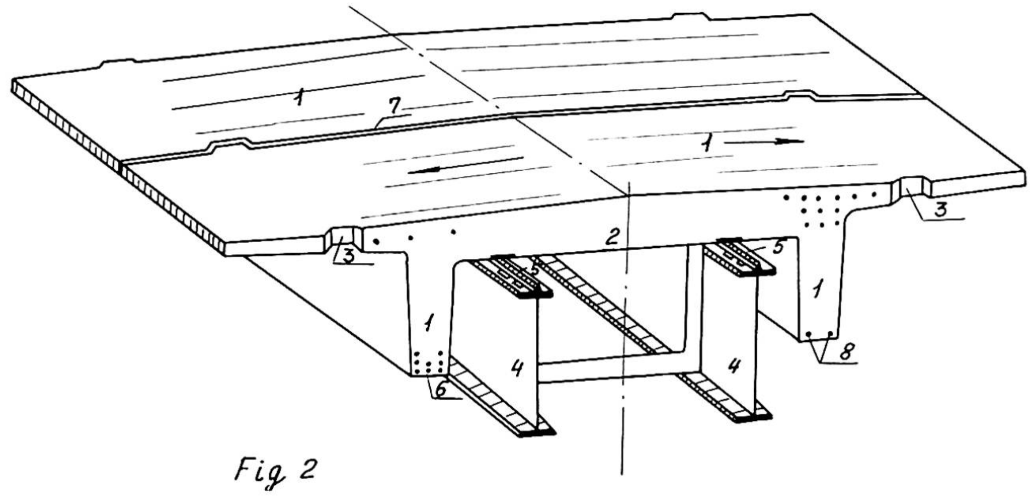

Fig 2

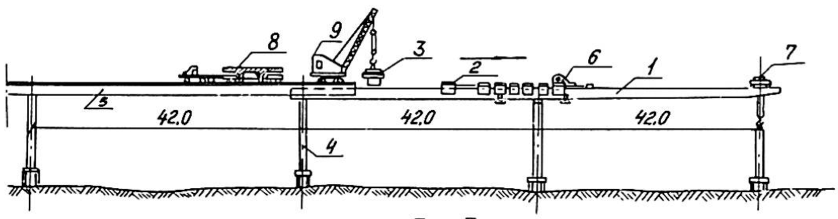

Fig. 3

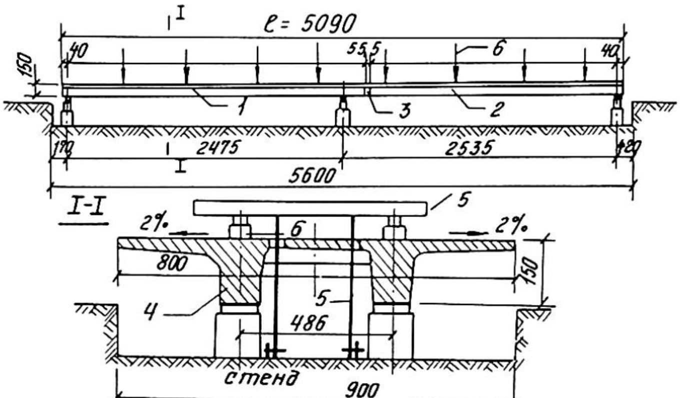

Fig 4

The scaffold carrier being in such position, the crane for hauling segments is installed at its front over the breast wall . The first available segment of span structure is transported on a trailer under a transshipment crane which then haults it on the rails. This done, the block is pulled along rail with the help of a winch in its design position. When all the segments of a span are assembled together on the carrier they are coupled and prestressed.

The section being completed, the scaffold carrier is descended and removed to the next span with hoisting jacks fixed on the piers. Next spans are assembled in the same way with the only difference that the transshipment crane in this case is installed on the outside panels of the assembled span.

A superstructure with spans 2x25 m was assembled on the special power test stand with the purpose of investigating and developing fabrication technology, transportation and assembly of continious construction of PRK type. Experimental slab-beam spans were made of segments of natural dimensions: length 2m, width 8 m, height 1.5 m. Weight of each block is 18.5 m.

Experimental deck was composed of two sections: 1st section of 15 blocks, 2nd section of 10. Blocks were fabricated by printing method and their production technology turned out to be very effective.

Their labour capacity happened to be lower than that of box beams. Transportation and mounting of experimental span structure. were made very successefully and with little labour expenditure.

The precast experimental span structures are supposed to be continued with static and pulse loads. Their stress-strain states at that are investigated at different stages until they are collapsed. (Fig.4).

Nowadays with the help of this method a bridge with spans 42x2x63+42 m is under construction and a number of other structures are scheduled to be erected.

SUMMARY - A new structure of a continuous span (abbr. PRK) is assembled on a movable scaffold carrier of original construction. An experimental continuous span with dimensions 2x25 m is tested under movable load on a specially designed test stand in order to examine the technology of fabrication and assembly and also to investigate the stress-strain state of the new construction.

RESUME - La structure nouvelle d'une poutre continue de pont (en abrégé PRK) est exécuté à l'aide d'un chariot d'étayage mobile de conception originale. Le poutre de 2x25 m de portée est sollicitée par une charge mobile en laboratoire, en vue d'élaborer une technologie de montage et d'étudier l'état contrainte-déformation de la structure nouvelle.

ZUSAMMENFASSUNG - Eine neue Methode, durchlaufende Ueberbauten von Brücken herzustellen, besteht in der Montage von Fertigteilen auf einen speziell entwickelten Gerüstwagen. Ein speziell hergestellter Versuchswagen dient dazu, Fertigungs- und Montagemethoden zu prüfen und den Spannungs-Verformungs-Zustand eines Versuchsträgers von 2x25 m Spannweite unter beweglichen Lasten zu ermitteln.

Industrialized Apartment Buildings Composed of Steel Frame and Precast Concrete Panels

Construction industrialisée d'appartements avec une ossature métallique et des panneaux préfabriqués en béton

Industriell hergestellte Wohnbauten, bestehend aus Stahlskelett und vorfabrizierten Deckenplatten

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1 PREFACE

In recent years in Japan, the convergence of the population into cities and their surrounding districts is remarkable. This has created the need for effective use of urban area and for increase in the number of housings. Consequently the development of the industrialized system for construction of tall apartment buildings are gaining demand. The research on the structural system of these buildings was carried out mainly from the aseismic point of view, because Japan is one of the worst earthquake countries in the world. Thus, the structural system composed of combined steel frame and precast concrete panels was newly developed, and group of apartment buildings 7 stories high, were constructed for the first time by this system in 1966. Thereafter this system and its variations have played the main roles for the construction of tall apartment buildings using industrialized process. Currently, the tallest example is 21 stories high (See Fig. 7), and the total number of dwellings constructed by these systems is roughly estimated to be about fifty thousand.

This paper deals with the structural characteristics of the apartment buildings constructed by said system on the basis of seismic tests and analyses, and also reference is made to some of the structural problems particular to these buildings.

2 STRUCTURAL SYSTEM

Fig. 1 shows the outline of the structural system of a 14 story apartment building on which structural researches were performed. In this building, lateral forces are resisted respectively by frame structures mainly composed of structural steel frame in longitudinal direction and by seismic shear walls in transversal direction. The strong axis of the H shaped steel columns are arranged in longitudinal direction of the building so as to resist the lateral force effectively.

In course of the construction, steel columns prefabricated to 3 stories length are erected. Next, steel beams encased in concrete with or without curtain walls and precast concrete shear walls embracing steel tie beams or tie plates at the top are jointed to the steel columns respectively in longitudinal and transversal directions, and then precast concrete floor slabs are laid on the beams and walls. After reinforcing and forming of the columns, concrete is then placed.

Steel beams are connected to steel columns by welding and bolting as shown in Fig. 2. Precast shear walls are jointed by bolting, welding and shear connectors. Precast shear walls with steel braces and concrete floor slabs poured at the site are applied in some cases.

A revised system was developed recently in which considerable portion of the seismic forces in logitudinal direction of the building is resisted by reinforced concrete, and consequently several tall apartment buildings were constructed by this system which was successful in saving considerable construction costs. (See Fig. 3)

3 METHOD OF ASEISMIC DESIGN

Seismic forces prescribed by the Japanese Building code were used for the aseismic design of these buildings. Structural members were determined by the Structural Standard of Architectural Institute of Japan. In longitudinal direction of the building, the structural design of steel frame was made to resist the bending and shear, while reinforced concrete of the columns was designed to carry the axial forces. In transversal direction of the building shear walls ware designed to resist the seismic force and the columns on both sides of the walls were designed to resist the over-turning of the building. As for the new type joints, load tests on joint specimens were performed and the allowable strength of the joints were properly determined from test results.

Because the structural system was a new development, load tests of the structural specimens including main connectors and joints were carried out to confirm the structural characteristics of the building, and the test results were fed back into the structural design. Observation of the dynamic behavior of the building was made by forced vibration tests. Finally, the aseismic safety of the building was confirmed through earthquake response analyses based on the test results.

4 STRUCTURAL TESTS

Static load tests of the structural specimens were performed to investigate the aseismic characteristics of the building shown in Fig. 1.

(1) Structural characteristics of the shear wall structure

A shear wall specimen of reduced 1/2 scale was prepared for the load test. The specimen represented two stories of the shear wall structure in transversal direction of the building. The shear wall of one story was composed of two precast concrete panels and a steel tie beam was provided at each floor level. The joints of the panels were formed with a combination of shear keys of concrete and stud dowels. The specimen was fixed to the testing floor at the bottom of the columns, and repeated lateral forces were applied alternately to the specimen at each floor level.

Fig. 4 shows the relation between the average shear stress and the shear deformation of the lower wall. The maximum strength of the specimen was 3.8 times as large as the design load and the joints possessed adequate strength and stiffness throughout the test. The deformation of the specimen at the maximum shear stress was about 4×10^{-3} rad..

(2) Structural characteristics of the frame structure

The load tests on the specimen of reduced 1/2 scale, which represented a two span-two stories unit frame with curtain walls, were performed to investigate the structural characteristics of the structure in longitudinal direction of the building. The repeated lateral forces were applied alternately to the ends of the columns of the specimen which was supported at the ends of the beams. Fig. 5 shows the load-deflection curve of the specimen. From this figure the following remarks can be obtained:

a. The curve showed stable and ductile characteristics which were caused by the yielding in bending at the ends of steel beams. The residual displacements of the specimen caused by 1.5 times design load or less were very small.

b. The maximum strength of the specimen was 2 times or more of the design load and its stiffness was considerably large.

c. The stiffness of the specimen as calculated on the assumption that wing walls and column behave as one body coincided fairly well with the observed one.

5 VIBRATION TESTS AND ANALYSIS

The building shown in Fig. 1 is supported on drilled steel piles at average depth of about 25 meters below ground level in the relatively soft layers of clay and sand. The bearing capacity of the soil and the length of the piles beneath the basement vary considerably according to location.

The dynamic properties of the building were investigated by forced vibration tests using a vibrator. The test results are shown at the top of Table 1 and in Fig. 6. It is observed from Fig. 6 that transversal vibration characterized by the deformation of the shear walls and floor slabs in their planes was created. In the fundamental translational vibration in transversal direction of the building, the ratio of displacement at the top of the building caused by the swaying and rocking motion was 66%. In contrast with this, the vibration of the structure in longitudinal direction was of typical shearing type.

Considering the dynamic characteristics observed from the vibration tests, dynamic analysis in transversal direction of the building was performed of a two dimensional structural model characterized by the correlated stiffness matrix of shear wall and floor slab in their planes and supported on the base with the conditions obtained from the tests. The analyses were in good correlation to the observed results as shown in Fig. 6.

The vibration test results of buildings of the same type structure are also summarized in Table 1. The following remarks may be derived from these test results:

a. The displacement due to rocking and swaying motions of the building was predominant in transversal displacement at the top of the building, especially in buildings supported by long piles driven in the soft layers of soil. In these cases the proportion of above mentioned displacement in total reached about 70 percent in maximum.

b. Fundamental natural periods of buildings 30 meters high were distributed in the range of 0.5 to 0.65 sec. in transversal direction, and 0.3 to 0.75 sec. in longitudinal direction respectively. The wide range variation of the latter periods may be attributed to the variety of types of the curtain walls fixed to the structural frames.

c. It should be noted that the period of torsional vibration (fundamental vibration of floor slabs) of slender buildings are close to the fundamental period of translational vibration of shear walls (fundamental translational vibration) and that the period of the vibration of the floor slabs in their planes (secondary vibration of floor slabs) was over 0.2 sec..

d. As for the relationship between natural periods and damping factors, it was observed that the damping becomes larger with the decrease of the period.

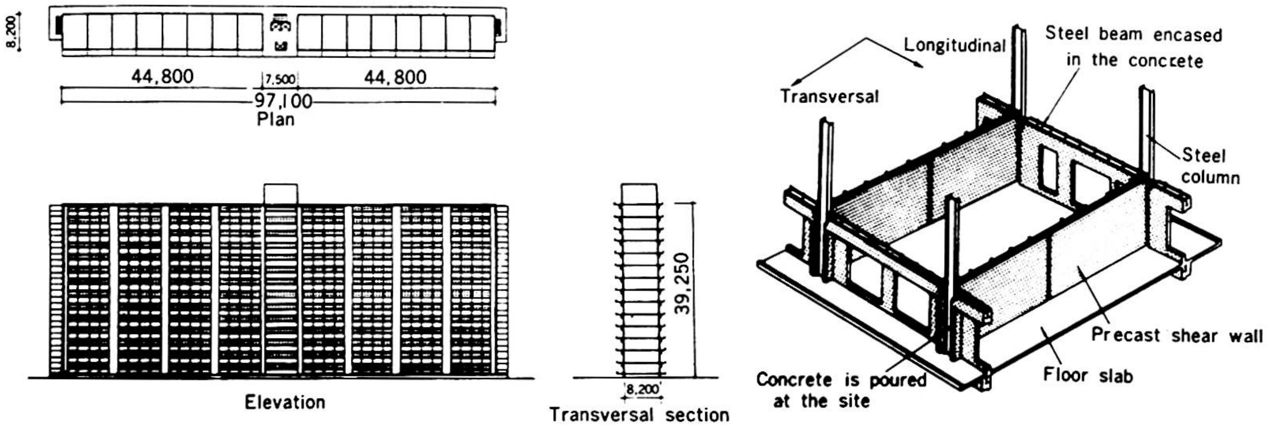


Fig. 1 Outline of a 14 story apartment building and its basic structural system

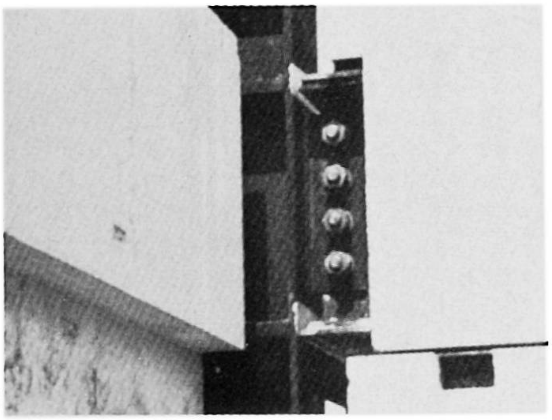


Fig. 2 Beam-column connection

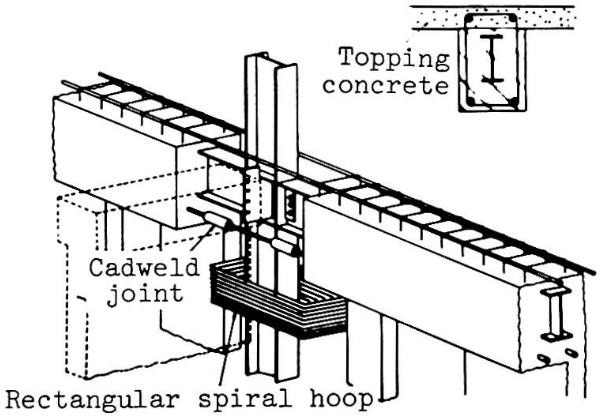


Fig. 3 Revised structural system

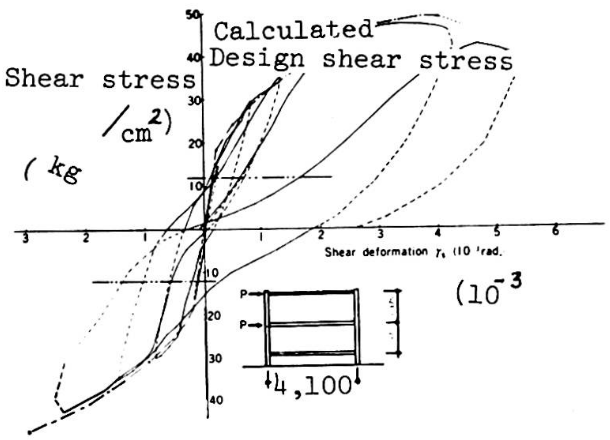


Fig. 4 Shear stress-shear deformation curve of shear wall specimen

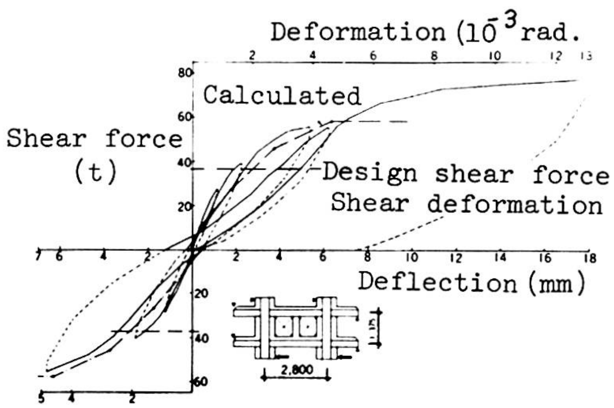
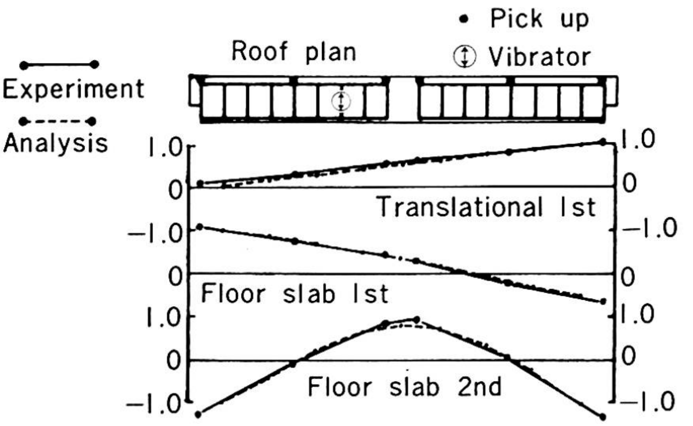


Fig. 5 Load-deflection curve of frame specimen

Table 1 Vibration test results

Bulg.	Dimension of building: Width (m)	Height (m)	Height (Story)	Length (m)	Period (s) & damping factor (%): Transversal: Translational 1st	Translational 2nd	Floor slab 1st	Floor slab 2nd	Longitudinal: 1st	2nd	Supporting	Proportion of sway & rocking
(a)	8.2	38.5	14	102.8	0.66 2.0	—	0.55 1.7	0.28 1.4	0.46 3.0	0.15 12.0	25m Steel pile	66%
(b)	8.5	29.6	11	92.7	0.48 2.0	—	0.43 1.7	0.25 2.5	0.31 3.7	0.11 —	17m Steel pile	50% or more
(c)	7.8	30.1	11	78.0	0.59 2.4	0.075 7.1	0.51 2.9	0.22 4.5	0.44 4.4	0.073 3.6	30m Steel pile	70%
(d)	10.74	30.5	11	59.4	0.54 2.2	0.095 4.8	0.54 2.2	0.22 3.9	0.74 2.2	0.21 8.4	Loam	50%
(e)	17.88	61.6	21	52.0	0.85 1.4	0.20 5.7	0.69 1.3	0.11 6.5	0.66 1.6	0.22 5.5	11m Concrete pile	25%
(f)	20.0	29.7	11	129.6	0.62 3.0	—	0.59 2.5	0.41 3.5	0.47 —	0.133 3.5	40m Steel pile	68%
(g)	7.79	29.7	11	80.4	0.59 1.9	—	0.54 1.7	0.21 2.9	0.76 1.2	0.28 2.5	25m Steel pile	67%



Vibration modes

Fig. 6 Results of experiments and analyses on the vibration of the building in transversal direction

Fig. 7 A tall apartment building constructed by the industrialized system

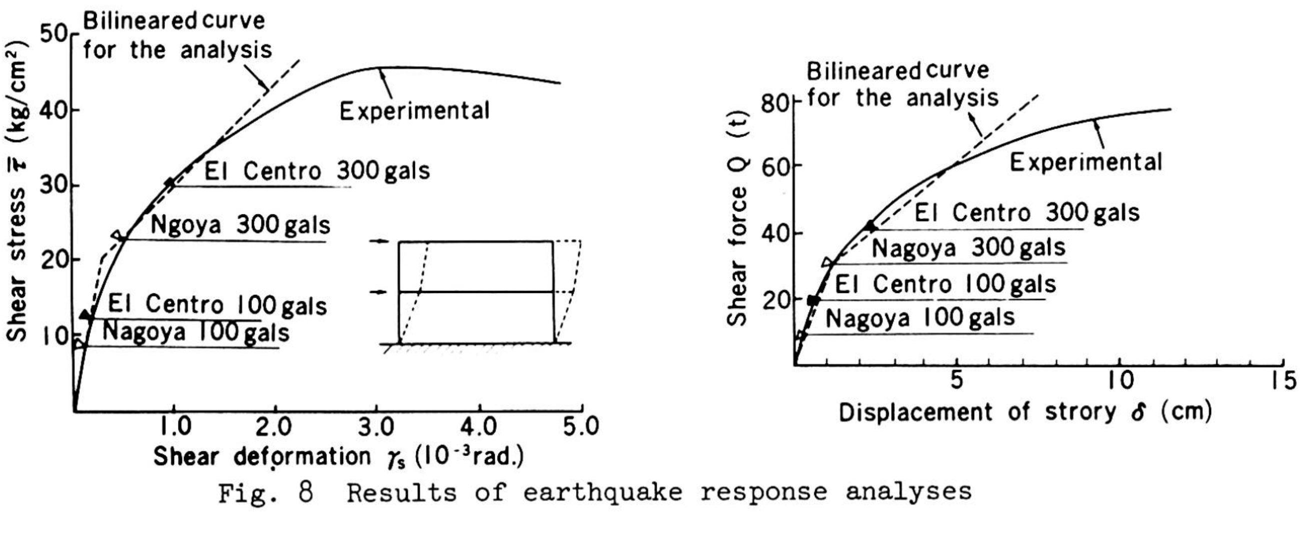


Fig. 8 Results of earthquake response analyses

6 EARTHQUAKE RESPONSE ANALYSIS

The earthquake response analyses of the building shown in Fig. 1 were made to confirm its aseismic safety. In transversal direction, a unit (1 span) of the building was treated as a flexural and shearing vibration model consisting of 14 lumped masses with rocking and swaying motion at the base. In this model, its flexure was assumed to be elastic and its shear to be bilinear. These properties were determined from the test result of the shear wall specimen. In longitudinal direction, the same unit was treated as a shearing vibration model fixed at the base. Bilineared characteristics of the model in shear were determined from the test results of frame specimens. In the analyses 5 % dumping factor was used, and El Centro and Nagoya earthquake accelerograms with modified maximum intensity of 100 gals and 300 gals each were applied to the models as the imput earthquake motions.

The results of analyses are shown in Fig. 8, in which the maximum response shear forces are marked on the load-deflection curves used in the analyses. It is considered that the superstructure of the building has sufficient earthquake resistance in both longitudinal and transversal directions.

7 CONCLUSIONS

From the tests and analyses mentioned herein, the tall apartment buildings constructed by the systems composed of steel frame and precast concrete panels are considered to have adequate aseismic safety. In other words, these buildings are considered safe from destructive damage which might result in loss of human lives during severest earthquakes, provided that the joints of the prefabricated structural members are normally constructed.

It is desired that further investigations will be performed on the seismic analysis method for the total structure which consists of superstructure, foundations, soils and piles, and that the data for more reasonable evaluation of aseismic safety of such buildings will be accumulated.

The authors wish to thank Dr. K. Muto for his leadership on this research and developement and Messrs. T. Tsugawa, S. Bessho and K. Ishii, Research Engineers of the Kajima Institute of Construction Technology, for their collaboration in the tests and analysis reported herein.

SUMMARY

An industrialized structural system for tall apartment buildings which consists of steel frame and precast concrete panels was developed in Japan in 1966. Thereafter, many apartment buildings have been constructed by this system and its variations. This paper deals with load tests of structural specimens, earthquake response analyses and vibration tests of the apartment buildings constructed by this system and also refers to some structural problems which are particular to these buildings.

RESUME

Un système de construction pour les habitations de grande hauteur a été développé au Japon en 1966, à partir d'une ossature métallique et de panneaux en béton préfabriqués. De nombreux bâtiments ont été réalisés avec ce système et ses variantes. Des essais de charge sur spécimens, une analyse du comportement sismique, et des mesures de vibration ont été réalisés sur des bâtiments de ce type. Quelques problèmes particuliers propres au système sont ensuite exposés.

ZUSAMMENFASSUNG

Ein Baukastensystem für Wohnhochhäuser, bestehend aus Stahlrahmen und vorfabrizierten Deckenplatten wurde 1966 in Japan entwickelt. Seither wurden viele Bauwerke nach diesem Verfahren erstellt. Hier wird über verschiedene Versuche an Bauelementen, über das Tragverhalten bei Erdbeben, über das Schwingungsverhalten sowie über weitere, für das Bausystem typische Probleme berichtet.

The Tokyo Port Tunnel

Le tunnel du port de Tokyo

Der Tunnel unter dem Hafen von Tokio

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Head, Bay Shore Route Construction Bureau
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1. Introduction

The Tokyo Port Tunnel, a submerged tunnel to which dynamic response analysis and other new technical devices were applied, is located at the First channel of the Port of Tokyo. The tunnel is 1,035m long and the ventilation buildings with cast-in-place tunnels together is 290m. The total length of the cross-tunnel is thus 1,325m. The undersea tunnel consists of 9 prefablicated reinforced concrete elements, each 115m long, 37.5m wide, 8.8m high, as shown in Fig. 1. The ventilation tower measures 48m in height above the ground and 25m in depth underground. The horizontal section is a square shape of about 40m. The soil condition at the construction site is shown in Fig. 2. The surface layer is 43m deep at the west land part, 28m at the sea-bottom and 49m at the east land part.

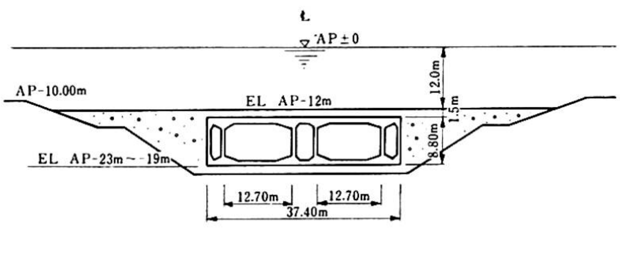

Fig-1 Standard cross section of Tokyo Port Tunnel

2. Earthquake response analysis and earthquake resistant design.

The predominant period of the ground inferred from the results of micro-tremor observation and empirical prospecting is 1.6 sec at the west side, 1.0 sec at the sea-bottom part and 1.8 sec at the east land part, as for the surface layer above Tokyo Gravel layer (base layer, GL-40m ~ -50m). The major part of the surface layer is composed of a very soft clayey soil having an N-value of 0 to 5.

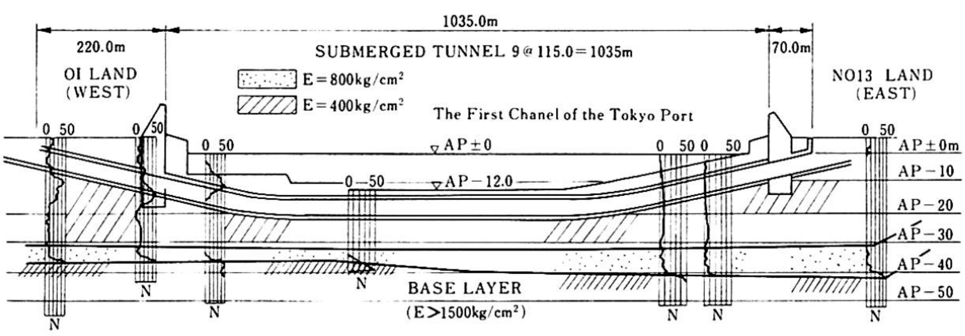

Fig-2 Geologcal map of the site of the Tokyo Port Tunnel

2-1 Earthquake response analysis

Earthquake response analysis on this tunnel was carried out by using the mathematical model which was jointly developed by Dr. OKAMOTO, Dr. TAMURA and HAMADA. And the following calculation and analysis were made by them and the Metropolitan Expressway Public Corp.

Horizontal vibrations are analytically divided into two directions along the tunnel axis and prependicular to it as shown in Fig. 3.

The ground from the west side to the east side, 1,325m long in total, was divided into 46 slices of 30m in thickness, and the distance between adjacent masses was taken as 30m.

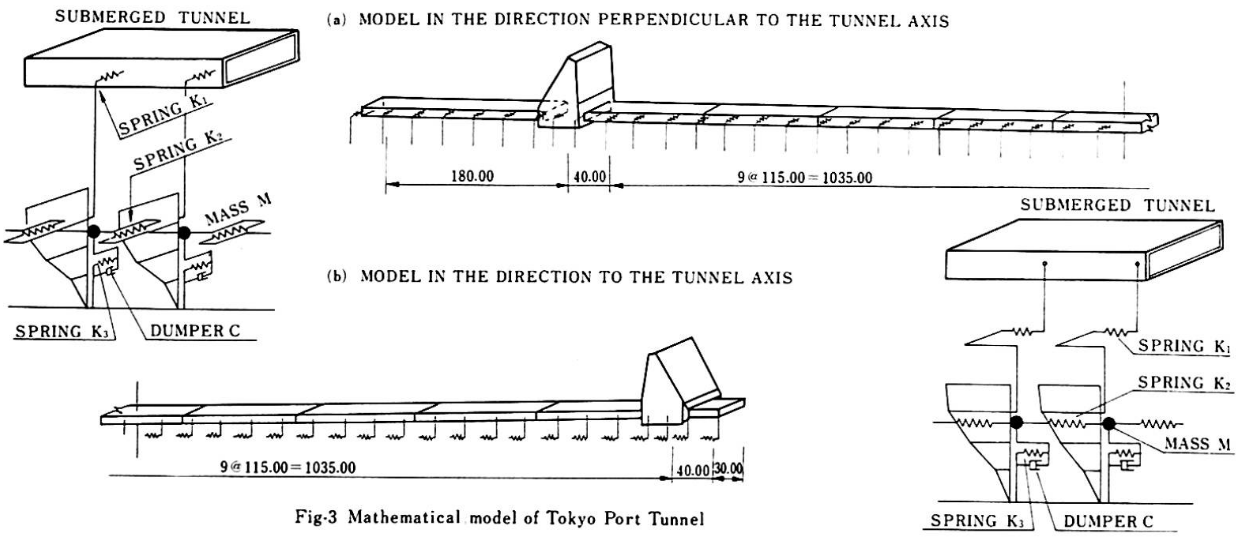

Fig-3 Mathematical model of Tokyo Port Tunnel

Responses in two directions were calculated mainly'by using EL Centro NS 1940 and Aomori NS 1968 (Off-Tokachi Earthquake) records as in-put seismic waves, the latter having comparably longer period when compared with the former. As a result of calculations, the followings were learned:

1) The distribution shapes of maximum values of bending moment, shearing force, axial force and displacement of the tunnel are all very similar each other, but the magnitudes of stress and displacement differ considerably depending on the input seismic waves.
2) Changes in dynamic characteristics of the ground along the tunnel axis, such as at the slope of the ground, give great influence on the stress in the tunnel during earthquake.
3) Relative displacement between the tunnel and the ground is smaller for vibration in the lateral direction than in the axial direction.

In the mathematical model, dynamic characteristics of the ground was represented by the one mass-one spring system equivalent to the fundamental shear vibration of the ground. To clarify the influence of the higher order mode of the ground motion, response displacement analysis were performed with Aomori NS and EL Centro NS, taking natural modes up to the third order of the ground into account. The result showed only several percent difference at most, so it was made clear that the displacement of the ground at the site could surely be represented by that of the mathematical model.

2-2 Effects of joints between Tunnel Elements

From the fact that stress distributions along the tunnel axis were very similar regardless of different type of input seismic waves, it became possible to develop an earthquake resistant design method for the submerged tunnel. For this purpose trial calculations were made for several cases.

In Fig. 4, case 1 shows a case that neither hinged joints nor spring joints are constructed, in case 2 hinged joints are constructed at points 5 and 32, and in case 3 at points 5 and 28. In case 4, spring joints are inserted for all the joints between tunnel elements.

For input seismic waves, Aomori, NS, 1968, maximum acceleration being modified to 100 gals, was adopted in analysis. From a result of the calculations, the following were made clear:

1) The hinged joint is effective to make bending moment and shearing force diminish, but its range of effectiveness is limited within about 100m from the joint for this tunnel.
2) Hinged joints make the tunnel flexible more, however restriction against deformation diminishes, and a small increase of stress is caused, as a result, at a portion fairly apart from the hinged point.
3) When spring joints are constructed between all elements, bending moment, shearing force and axial force decrease by nearly 50%.
4) Only a small increase is observed for the displacement of the tunnel by the insertion of hinged or spring joints. When hinged joints or spring joints are to be constructed, much care should be exercised in determining their positions.

2-3 Effects of Joints between Tunnel and Ventilation Tower

In the case of this Tunnel, the site of ventilation towers were so planned as to locate at the both ends of the tunnel on its axis. Since the ventilation tower shows different behavior during earthquakes, it was expected that large stress may be generated in the tunnel around the terminal joints. In this connection, three cases were considered. In case 1, the tower and the tunnel are made movable independently each other, in case 2 are connected by means of hinges (only for the lateral vibration), and in case 3 are rigidly connected.

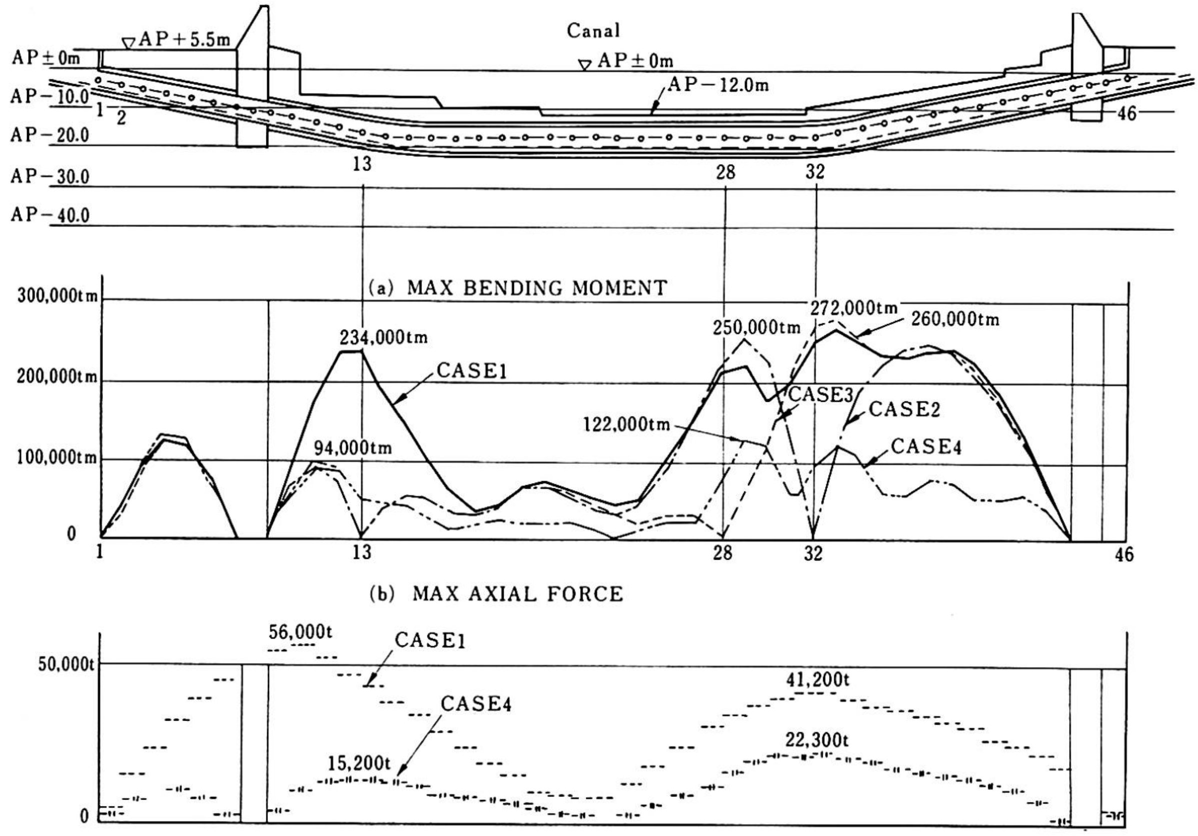

Fig-4 Influence of hinged joints and free joints on stresses of the tunnel

As easely understood from Fig. 5, considerably large bending moments are generated near the ventilation tower, and steeply decrease with the distance from the ventilation tower and same tendency goes for axial force too. The maximum bending moments caused at points more than 150m apart from the ventilation tower are almost same for all cases.

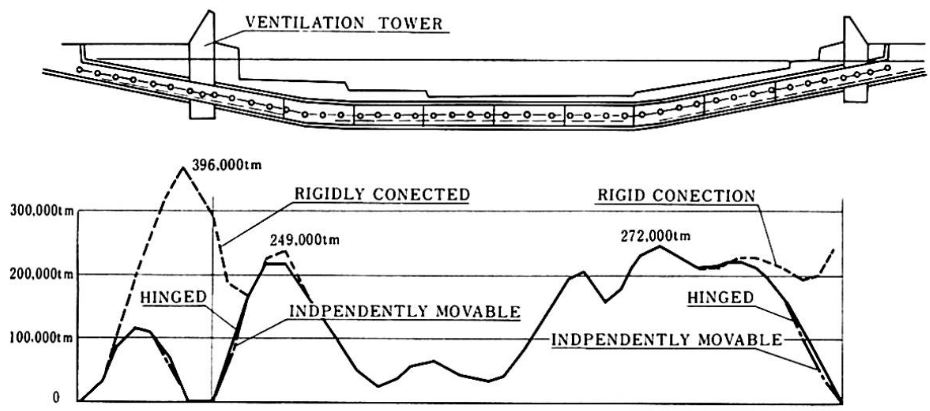

Fig-5 Influence of joints between the tunned and the ventilation tower on berding moment of the tunnel

Similarly, Fig. 5 indicates that extent of about 250m from the tower is under influence regarding axial force.

In both cases of the hinged and free joints, the values of the maximum bending moment are equal, and considerably small when compared with the values in the case of rigid connection. From the above-mentioned results, it is recommended that terminal joint between the tunnel and the ventilation tower should be made as flexible laterally and movable axially as possible in order to reduce stress concentration near the tower to the minimum.

2-4 Joint Structure actually applied

Fig. 6 shows the joint details actually applied at the first joint and the joint is able to move ± 4 cm horizontally and + 4 cm axially toward the approach tunnels, but the movement toward the sea bottom side is restrained under 10 cm by the function of SEEE Cables specially installed between the ventilation building and adjoining element.

Fig. 7 shows ordinary joint details. Utilizing the joint space, both horizontal and vertical shearing key, and the second water proofing, etc. are installed, and the upper side of the joint space is covered with specially devised Ω type steel plate against bending moment, axial force and shearing power which is generated at the primary stage of earthquake motion.

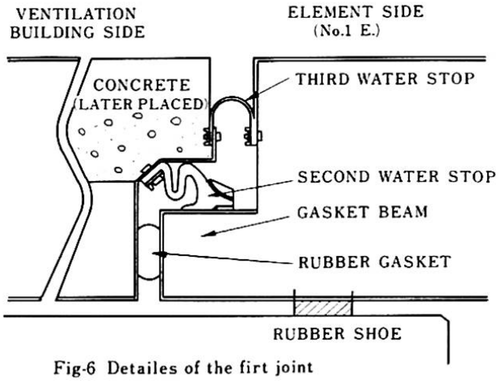

Fig-6 Detailes of the firt joint

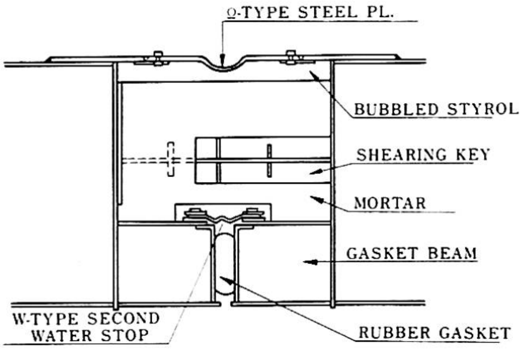

Fig-7 Detailes of the Flexible joint

3. Motar Grouting

Up to this time, what is called "Sand Jetting Method" has been used in the treatment of foundation of ordinary submerged tunnel with big cross section. However in this tunnel, mortar grouting method was newly developed. This method has such a great merit that the process of filling up the space between undersurface of a element and sea bottom is not disturbed by weather or sea condition, and also grout mortar itself has wide variety in choice.

3-1 Bentonite Mortar

The strength of grout mortar for the foundation of element is good enough with low strength. So much importance was rather attached to how to fill up the space as completely as possible. Several kinds of grout mortar were studied, for example, bentonite mortar, air bubble mortar and silt mortar. The bentonite mortar was adopted in a result.

The strength comes down by using bentonite as admixture, however it became possible to raise the filling up ratio with effect of the improvement of workability in lean mix mortar, prevention of material segrigation and continuity of fluidity for long hours. The following experiments were carried out prior to execution, and data for actual execution were obtained.

(A) Bentonite mortar proportion test
(B) Small sized grouting test
(C) Large sized (same as actual size) grouting-test

As a result of experiments, it was confirmed that more than 80% at minimum can surely be filled up.

Following is a standard proportion of bentonite mortar.

					per $1 m^3$
Cement	Bentonite	Water	Sand	Rital	Flow Value
150kg	37.5kg	647kg	750kg	1.5kg	14 – 18 sec

In the actual execution, the strength of bentonite mortar were fairly scattered, however, its strength was $\sigma 7=1.46$, $\sigma 14=2.49$ and $\sigma 28=3.42 kg/cm^2$ on the average.

3-2 Mortar Grouting

Prior to grouting, the clearance between under-surface of a element and sea bottom was measured just under all grouting holes by using mortar level meter (super sonic wave detector). Approximate volume of mortar to be grouted was calculated by data obtained.

The order of grouting was principally carried out in accordance with the chart shown in Fig. 8. According to the mortar flow test, the mortar does not flow uniformly in all directions, but in some direction, showing something like belt with 2 – 3 m width, and the mortar flows in other directions similarly, when the initial flow stops. Referring to this mortar flow behaviour, the mortar level meters were arranged at the suitable holes.

In order to prevent grout mortar to leak outside the projected plane of a element, mortar stoppers were applied, namely long mortar bag in the direction perpendicular to the tunnel axis, and crushed sand stone buried at the both sides of a element in axial direction.

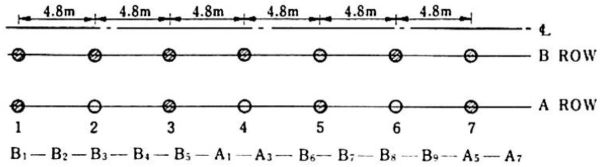

Fig-8 Order of mortal grauting

Effectiveness of grouted mortar was confirmed by comparing a theoretically estimated value of settlement of a element with actual settlement generated when the vertical jacks which were provided at the both sides of the top end of a element were released.

4. Countermeasure for Consolidation Settlement

In a result of the past several years' survey, settlement of alluvial layer supporting the tunnel, was thought to arise in future, being caused by silt layer's consolidation itself and the drop of underground water level in the diluvial gravel layer, mainly around eastern half of the tunnel. This drop of water level was caused by over pumping up for industrial use, but this underground water level is maintained at -7 m for these years by the severe government regulation.

In a result of comprehensive study, the settlement of the ground was estimated -10 cm at the center, -30 to 40 cm at the eastern end and negligible at the western half of the tunnel respectively. In this circumstances, the followings were considered for countermeasures.

1) replacing silt layer with sand
2) driving sand piles
3) to promote consolidation by preloading
4) rising up the underground water level
5) application of pile foundation

These methods were studied from such points of views as cost, construction period, construction process and effects of application. However the point of application was how to secure a continuity of the supporting condition to the elements.

For this purpose, a pile system which can follow the ground settlement to some extent and finally get settled on a ideal vertical line was studied. In ordinary piles, such as friction piles and semi-supporting piles which are both able to give a certain degree settlement, it is very difficult to secure prearranged settlement calculated from the settlement line.

In this circumstance, a pile, which functions as supporting pile ultimately by the operation of some divice put on the top of a pile, when the element reaches prearranged settlement, was developed.

This special device is shown in Fig. 9.

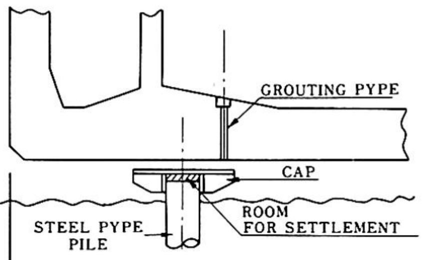

Fig-9 Special cap and steel pile

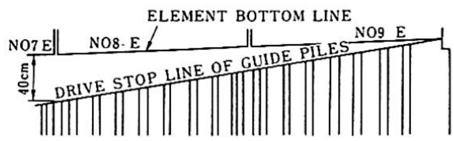

Fig-10 Pile arrangement in the axial direction

This cap, made from steel plate, is placed on the pile top, when steel pipe pile is driven. It is not necessary for a pile to be driven to accurate level position, as far as the cap with proper settlement allowance is placed on regular position because the space between the undersurface of element and sea bottom is filled by grout mortar.

Piles were cross sectionally so positioned just under grouting holes that grout mortar can surely fill up the space.

Piles were set from the end element (the 9th element) up to the quarter point of the 7th element, in such a manner that the settlement allowance changes in a straight line along the axis with 40 cm just under the joint between 7th and 8th element and no margine under the terminal joint, taking such all feasible conditions into consideration as allowable deformation of element, flexibility of joint and the development process of ground settlement, etc.

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6. Summary

The anti-earthquake design and construction of the submerged tunnel on the soft ground are presented. The analytical technique conducted herein is widely applicable to the earthquake resistant design of submerged tunnel. The seismometer observation now being carried out will provide the data to improve the method of analysis.

New technique of grouting into the bottom of the tunnel operated inside of the elements seems to be used widely in off-shore structures.

Newly developed capped piles foundation is also a good countermeasure against ground settlement.

SUMMARY

The anti-earthquake design and construction of the submerged tunnel on the soft ground are presented. The analytical technique conducted herein is widely applicable to the earthquake resistant design of submerged tunnel. A seismometer observation now being carried out will provide the data to improve the method of analysis. New technique of grouting into the bottom of the tunnel operated inside of the elements seems to be used widely in off-shore structures. Newly developed capped piles foundation is also a good countermeasure against ground settlement.

RESUME

L'étude anti-sismique et la construction d'un tunnel immergé sur des fondations compressibles sont présentées. La méthode analytique employée s'applique facilement à l'étude de la résistance aux tremblements de terre. Les observations actuellement réalisées par sismographe donneront des informations permettant d'améliorer cette méthode analytique. Il semble possible d'utiliser pour des ouvrages d'art martimes, la nouvelle technique d'injection de coulis de mortier sous tunnel à partir de l'intérieur des éléments. La nouvelle technique de fondation par pieux avec têtes élargies semble aussi une bonne solution contre les tassements importants.

ZUSAMMENFASSUNG

Ueber die Untersuchung des Verhaltens gegenüber Erdbeben von abgesenkten Tunnels auf nachgiebigem Baugrund wird berichtet. Die verwendete Berechnungsmethode eignet sich gut für die Ermittlung des Tragwerkwiderstandes gegenüber Erdbeben. Im Gang befindliche Messungen mit Seismografen werden Ergebnisse liefern, die eine Verbesserung der Berechnung gestatten. Die neu entwickelte Methode, vom Innern der Tunnel-Elemente aus den Zwischenraum zwischen Bauwerk und Baugrund zu injizieren, eignet sich ganz allgemein für überflutete Bauwerke. Ebenso erscheint die Verwendung von Pfahlfundationen mit vergrösserten Pfahlköpfen als gutes Mittel gegen Baugrundsetzungen.