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JOINTING PRECAST CONCRETE MEMBERS

TO A COMPOSITE RIGID FRAME

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Synopsis

This is an experimental study on jointing a precast concrete girder to precast prestressed concrete piles in a composite rigid frame. This paper (1) proposes a new method of jointing a girder to precast concrete hollow columns, (2) clarifies the behavior of jointed corner, (3) proposes a method to analyze a composite frame by means of "Joint ductility coefficient", (4) clarifies necessary prestressing force for giving the same behavior under the working load as in monolithic rigid frame, together with other details related thereto, and (5) clarifies the effect of the unevenness of the column heads on the behavior of a composite frame.

Keywords: bond (concrete to concrete); composite structure; concrete pile; facial irregularity; frame; joint; precast; rigidity; rotation.

Introduction

With proper use of precast members for both structural girder and column and adoption of appropriate jointing system for these members, it will be possible to construct a concrete composite rigid frame with less time and work as well as minimizing undesirable effect of construction work such as interference in traffic flow. Notwithstanding the above-mentioned merits of using precast members are getting wider recognition, the practical use of composite rigid frames in Japan is very limited because of the unsolved structural behavior of such frames and the lack of recognized method of jointing girders to columns as well as the traditional preference of cast-in-place concrete structures. A rare example of composite rigid frames is the elevated railway bridge of Arakawa-Higashi on Sobu Line of Japanese National Railways, in which precast box girders and precast box columns were jointed by prestressing force. The effective use of precast members on the market such as precast spun concrete piles of high quality is considered to have greater merits of economy in constructing composite rigid frames.

This paper aims at the examination of structural problems in designing composite rigid frame basing on the extensive study of elasto-plastic behavior of the frame made by jointing girder directly to column through prestressing system as well as the study of mechanical behavior of the anchorage of prestressing steel in the concrete at midheight of column.

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1 Anchorage of Prestressing Steel at Midheight of Columns in Jointing Precast Concrete Members

A new anchorage method of prestressing steel for jointing is investigated for the purpose of using it in constructing a composite frame by utilizing as column prestressed concrete piles on the market. The proposed method is to anchor the prestressing steel in the concrete pre-placed at midheight of column. This method has several advantages as follows: Field work will be only prestressing work in jointing; the error in the position of holes in girders may be absorbed by flexibility of the prestressing steel; stressing work will be simpler; length of steel will be shorter; corrosion of steel may be prevented by grouting.

Before proposing this type of "midheight anchorage" method, pull-out tests and push-out tests were conducted in order to investigate the behavior of the anchorage zone, especially the ultimate capacity.

1.1 Specimens and Test Procedures

Outside diameters of precast concrete piles used in the tests were 20 cm with 5 cm thick, 30 cm with 5 to 6 cm thick and 70 cm with 11 cm thick. Qualities of concrete, dimensions of piles, inside surface condition of piles and amount of prestress are shown in Table 1.

For small piles with diameters of 20 cm and 30 cm, a prestressing bar with diameter of 22 mm anchored in concrete placed inside the pile was pulled out by universal type testing machine with 100-ton capacity. For large piles with diameter of 70 cm, concrete inside the pile was pushed out through concrete cylinder as shown in Fig. 1.

1.2 Failure Mode of "Midheight Anchorage" Zone

In Fig. 2 an example of the relationship between average bond stress and free end slip is shown. In the cases of PD1 and PD2 shown in this figure, no visible crack was observed on the surface of piles even after the slip exceeded 1 mm and the bond between the inside concrete and the pile concrete seemed to be destroyed. Longitudinal cracks observed after the occurrence of 2 or 3 mm of slip, and anchorage zone of the piles was perfectly destroyed after the development of lateral cracks between the longitudinal cracks. These specimens were considered to have failed due to the destruction of bond between pile and the inside concrete.

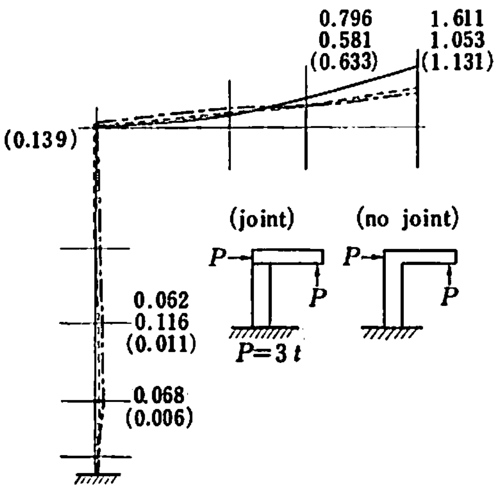
In Fig. 2 the results of PD12 and PD13 are also shown. In these specimens longitudinal splitting cracks were observed at about 0.2 mm free end slip, and simultaneously the slip began to increase very rapidly toward failure. This type of failure was considered to be tensile failure of concrete pile due to the development of longitudinal cracks before the occurrence of large slip.

As mentioned above, there are two types of failure modes in this type of anchorage. In case of small piles whose thickness is relatively large compared to the inside diameter bond failure is prominent, while tensile failure is prominent in large piles whose thickness is relatively small.

ERRATA

Pages		
23	4 th line from the bottom	C 4 specimen ⟶ 5
49	9 th column in Table 3	$\sigma_f \longrightarrow \overline{\sigma_f}$
49	9 th column in Table 3	2280 ⟶ 2200
149	16th line from the bottom	bridges of large scale ⟶ bridges of comparatively large scale
150	21st line from the top	93.000m ⟶ 92.000m
153	4 th line from the bottm	elastic construction ⟶ contraction
162	31st line from the top	consists ⟶ consist
162	36th line from the top	extend ⟶ ectended
163	the bottom line	permiability ⟶ permeability
164	6 th line from the top	permiability ⟶ permeability
164	20th line from the top	permiability ⟶ permeability
165	23d line from the top	form ⟶ forms
166	the top line	Table 1 ⟶ 2
166	2nd line from the top	Table 2 ⟶ 3

189 **Fig.—11** Deformation of angle model for testing (joint or no-joint)



—·—·— Observed value (no-joint)
——— Observed value (joint)
------- Theoretical value in parentheses
(Unit : mm)

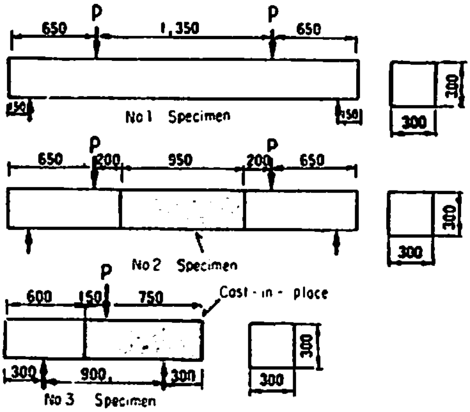
192 Table-5 Loading Table

Structure for Testing		Design-Load (t)	Design-Cracking Load (t)	Ultimate-Load (t)
Angle model	Solid-type (no-joint)	3.0	5.0(10.0)	11.0(27.5)
	Box-type (no-joint)	3.0	5.0(8.0)	11.0(14.8)
	Solid-type (joint)	3.0	5.0(5.0)	11.0(12.8)
	Box-type (joint)	3.0	5.0(5.0)	11.0(12.4)
Rigid-Frame	Vertical-Downward	18.0	28.0	63.0
	Horizontal	10.0	18.0	32.0(42.0)

Condition: Design-Cracking Load $\sigma_c = -25\,\mathrm{kg/cm^2}$

( )… observation value

265 Fig.— 5



266 the bottom line No. 16, Sept. 1970 ⟶ No. 18, 1969

1.3 Ultimate Capacity of "Midheight Anchorage" Zone

Effect of bond development length on the ultimate capacity of anchorage zone failing by tensile cracking of piles was investigated. The loads at failure due to circumferential tensile stress were almost the same in spite of the difference in bond development length as shown in Table 1 (PD14 to PD16). This result indicates that even in the case of the tensile failure there exsists a limit in effective development length.

The effect of the inside concrete quality is also important even in the case of tensile failure as shown in Table 1 (PD13 vs. PD14). The ultimate capacities of the specimens with heavy laitance on the inside surface of concrete piles (PD9 to PD12) are 40 to 60 percent of those with such treatment as to clean up the inside surface of concrete piles.

The effective length of concrete pile to resist the tensile stress occurring in anchorage of prestressing steel, depends on development length, degree of the treatment on the bonding face, concrete quality and so forth. The effective bond development length is considered to be about one half of the inside diameter of piles. From this assumption about 15 percent of pull-out force is recognized to be equal to total tensile force acting on the pile with roughened inside face, and this was found to be able to be applied to the cases of small piles as 20 cm to 30 cm diameter.

In case of bond failure the failure mechanism is rather simple, and the ultimate capacity may be the product of bond strength and development area. Due to the three axial compressive condition of the inside concrete, the bond strength may be larger than that in usual condition.

1.4 Reinforcement in "Midheight Anchorage" Zone

It is obvious that the ultimate capacity of anchorage zone will be controlled by the smaller one of tensile failure capacity and bond failure capacity. As the prestressed concrete piles prescribed in Japan Industrial Standards generally have only small amount of spiral reinforcement to be used for this type of columns, large piles have a tendency to reach the ultimate capacity with tensile failure mode. Therefore, in order to anchor the prestressing steel in the concrete pre-placed inside the piles, it will be necessary to provide with enough amount of spiral reinforcement to the anchorage zone. The required amount of reinforcement may be calculated by the above procedure to a certain degree. For example, spiral reinforcement of 10 mm diameter with 15 cm spacing is necessary, in order to ensure the ultimate capacity of 270-ton pull-out force for 70 cm diameter pile. Although further study on the effect of reinforcement is necessary, this type of anchorage system is considered to be efficient in practical use.

2 Elasto-plastic Behavior of a Composite Frame

Loading test of large frame models as shown in Fig. 3 was carried out, and structural behavior of a precast concrete composite frame, especially relation between loading and rigidity of the joint, was investigated in detail. Besides, the effect of supplemental reinforcement and so force on the rigidity of a joint is also investigated based on test results of π-shaped specimens as shown in Fig. 4.

2.1 Frame Models and Test Method

Large scale frame models were made. Each frame was formed by jointing a precast girder to two or three precast prestressed concrete columns, which have 30 cm outside diameter ring section with 4 cm thick. These precast girders to be jointed were of three kinds of sections. Precast columns were buried in the foundation girder which has 50 cm x 35 cm cross section. Foundation girder is reinforced so as to match to be buried in bearing.

In jointing a girder to column, the midheight anchorage method of prestressing bars mentioned in 1 was used, and so anchored 22 millimeter prestressing bar was strained after being run through the hole pre-made in a girder, placing fresh plaster paste between the two faces. Jointing forces were varied to three kinds such as 2 tons, 5 tons, and 10 tons respectively. Vertical load was applied at each span center until the strain of reinforcement reached 1000×10^{-6}.

Relative rotation of girder axis to column axis, deflection of span center of girders and strain in each part of the frame were measured. In the case of three column frame girder with 30 cm x 16 cm section horizontal load was applied at an end of a girder until the bottom moment of prestressed column reaches the crack developing moment.

2.2 π-shaped Specimens and Testing Method

Besides the large scale frame models, π-shaped models, which are analogous to a joint part of girder and column, were tested to investigate the factors affecting rigidity of joints in detail. These members were jointed by straining a prestressing bar after hardening of cement paste which was placed between the jointing faces to adjust the unevenness. The factors investigated were grouting around prestressing bars, location of prestressing bars, and supplemental reinforcement. Horizontal load was applied by 10-ton capacity oil jack to a specimen placed on rollers as shown in Fig. 4. At each state of loading, detachment of jointing faces of girder and column, and strain on prestressing bars, reinforcing bars and concrete were measured.

2.3 Distribution of Moment in a Composite Frame

From the test results, as shown in Fig. 5, it became clear that the jointed corner could be considered to be rigid when the working moment was small, but the rigidity of the jointed corner decreased rapidly due to the detachment of the column surface from the girder when the moment exceeded a certain value. This value was corresponding to the moment when the fiber stress of the column section got reduced down to zero. In Fig. 6, measured distribution of moment and claculated distribution of moment were shown in comparison to the case of vertical loading of 12 tons at each span center of girders. Twelve-ton can correspond to the load when tensile stress of the bar reaches 1400 kg/cm^2 by the calculation of ordinary rigid frame analysis. This figure clearly shows that the 40 percent decrease of moment at the joint and 15 percent increase of moment at the span center due to the effect of relative rotation of the girder axis to the column axis, which is measured as 5.0×10^{-4} radian.

From these results, it came to be clear that the structural characteristic of a composite frame could be found in the decrease of rigidity of joint with the increase of working moment at a corner followed by the variation of moment distribution from that of monolithic rigid frame. That is, to the action of vertical load, moment is distributed as if the stiffness of columns were reduced and to the action of horizontal load, moment is distributed as if the stiffness of girder were reduced.

The characteristic distribution of moment of a composite frame is shown to be calculated by considering a joint as a structural spring, whose ductility coefficient changes nonlinearly with amount of axial force and moment. As an example of the calculation, the calculated distribution of moment taking the joint ductility coefficient of 8×10^{-6} radian/ton·cm is also shown in Fig. 6.

2.4 Rigidity of a Joint

As was mentioned in 2.3, variation of rigidity of column girder joint gives considerably large effect on the distribution of moment of a composite frame. This effect is obviously calculated if the variation of rigidity get cleared. Therefore, the factors influencing the rigidity of a joint were investigated in detail.

The factor investigated first was the stiffness ratio of column to girder. The measured rigidity of the joint of the column with girder whose dimension was varied to three kinds as stated in 2.1 and accordingly stiffness ratio was varied to three kinds, was studied and besides this, the rigidity of the two column frame and three column frame were compared with, as the stiffness ratio of three column frame was twice as large as that of two column frame. The experimental rigidity of the joint was determined so that the assumed value for the joint ductility coefficient gives agreement to the measured moment distribution with the calculated one. From the tests the stiffness ratio is not recognized to have any substantial effect on the measured rigidity of the joint. Fig. 7 shows the relation between the measured corner moment and the measured joint ductility coefficient. This figure shows that there is no substantial difference of the relation between the cases of two column frame and the case of three column frame though the stiffness ratio of one is twice as large as that of the other.

However, this figure shows slightly smaller decrease of the rigidity in three column frame than in two column frame. This is because of the effect of larger amount of axial force in the case of three column frame due to one half span length of two column frame. This figure shows also that there is no substantial difference of the relation in the case of horizontal loading action. However, the rate of decrease of the rigidity get increased in the case where larger amount of moment is working compared to the case of vertical loading action. This is also considered to be due to the smaller amount of axial force.

After all, it comes to be clear that the important factors affecting the rigidity of a column-girder joint are the shape of column section and amount of prestressing force, besides the amount of axial force and working moment at the corner. Therefore, as a next step, the effects of such factors as grouting around prestressing tendons and their location were investigated.

Fig. 8 shows the relation of working moment and measured rotation by 4 cm length contact gauge for each joint shown in the same figure. The figure shows that the rigidity of a joint increases remarkably when grouting is done around prestressing bars or location of prestressing bars are selected to the vicinity of outer faces or supplemental reinforcement are added. For example, the relative rotation of C2 specimen decreased to the amount less than one half of that of C4 specimen, whose dimension and prestressing force of 50 kg/cm^2 are completely equal to those of C2, in the state of large amount of moment. Moreover, the relative rotations of C2 in which prestressing bars were located in the vicinity of outer faces and that of C3 in which supplemental reinforcing bars of four D16 mm were added, decreased to from 1/2 to 1/5 of that of C1 in which a prestressing bar was placed at the center of the section in spite of the same amount of prestressing force in all specimens.

Fig. 9 shows an example of the calculated rotation of joints basing on the following assumption together with the measured one.

(1) The relation of rotation of girder axis to column axis in a composite frame is essentially equal to that of a prestressed concrete beam, and the rotation calculated by the same method as that used in getting bending deformation of a prestressed concrete beam, is denoted as ϕ_{cal}.

(2) The rotation of a joint is equal to ϕ_{cal} until compressive stress of outer fiber of tension side of the section is reduced down to zero. For the conveniency, moment working in this state is called as balanced moment.

(3) When amount of working moment exceeds the balanced moment, the bond between reinforcing bar and concrete or prestressing bar and grout is lost and deformation of rotation concentrates to the joint. The length of bondless range is assumed as constant, 15 cm in these cases.

Fig. 9 shows that in the area where working moment is comparatively small, the calculated rotation is small in all joints compared to the measured rotation, while the measured rotation exceeds the calculated rotation in the area where amount of moment is larger.

This is considered to be due to the fact that the length of bondless range of bars is smaller than 15 cm during the state when working stress in bars is relatively small and in the loading state where working stress in bars is larger, the length becomes greater than 15 cm. As an assumption of 15 cm is taking for the length of bondless range, the calculation method is consequently very rough. However this analysis just aims to give rough estimate of the rigidity of joints.

Tomei toll road elevated bridges constructed in Komaki area used cast-in-site concrete girders instead of precast members though they used precast spun concrete piles as columns. As an example of applying the above-mentioned method in calculating the rigidity of the joint, distribution of moment was calculated assuming that the precast girders were used and jointed to the columns by prestressing force. An an example, the case of uniform jointing prestress of 40 kg/cm^2 and uniform loading on the girder is shown in Fig. 10. In this figure the calculated distribution of moment is shown in comparison to that of monolithic frame for the load of 90 ton/m which produces very close to the design moment. The distribution of moment is almost the same as that of monolithic frame in spite of the slight amount of rotation of 3.0×10^{-5} radian. In the same figure, the case of loading as large as 180 ton/m which is almost equal to the estimated ultimate load of the girder is also shown. In this case, the rotation of the girder axis to the column axis of 15×10^{-5} radian is calculated and because of this rotation, amount of moment at the joint is reduced to 85 percent of that of monolithic frame and moment at the span center or at the point on the central column increases by the amount of 4 percent of that of monolithic frame. In accurate frame analysis, it is necessary to take into consideration the fact of reduction of moment of inertia due to the development of bending crack, besides the reduction of rigidity of joints. At present state, the method of a composite frame analysis mentioned above is considered to give a rational basis in designing a precast concrete composite frame.

3 Effect of Irregularity of a Column Head on the Distribution of Moment of a Precast Rigid Frame

One of the problems to be solved in using precast concrete composite frames, is the effect of irregularity of a column head on the moment distribution of a frame. No matter how carefully precast concrete columns are made, it is impossible for all columns to be completely of the same height and for faces of all column heads to be perfectly even. Therefore, the effect of irregularity on the behavior of precast composite frame was investigated. Irregularities of jointing members can be classified into two groups according to the effect on the structural property of a precast composite frame. One is the difference in height of columns and the other is unevenness of faces of column heads or subsurfaces of girders.

If there is difference in column height, even after jointing a girder to a higher column, there still remains some distance between the girder and a column of smaller height. Therefore, jointing a girder to this column will give rise to bending moment which is a product of span length and amount of prestress introduced until the subsurface of a girder touches column heads. As this arm length is large, the difference in height produces large amount of internal stress in jointing members. This stress can be calculated relatively accurately.

On the other hand, in the case of irregularity in unevenness of the face of a column head or the subsurface of a girder, at least a point of both surfaces is in touch each other. The distance between that point and the location of prestressing tendons is relatively small. Therefore, this irregularity does not give rise to so large amount of internal stress as the case of difference in column height.

However, the irregularity of this kind affects considerably the moment transmitting property of a joint. If this effect is quantitatively clarified, it is possible to form the precast rigid frame directly jointing girder to column. It is obvious that analysis method of rigidity of a joint mentioned in 2.4 can be thereto applied to get quantitatively the effect of this irregularity, once the amount and distribution of prestress and shape of column section are known. Type A in Fig. 11 is a model of irregularity simplifying such condition where only the inner portion as marked in the figure touches the girder with its prominence. Type B is that of the case where one half of the face touches the girder. For the comparison, the case of no irregularity, Type C, was likewise analyzed.

Therefore, in constructing precast rigid frame, adoption of direct jointing method is considered to be effective if the irregularity be controlled as slight as possible and the ultimate strength of the structure be increased by the amount of undesirable effect of irregularity analyzed by the abovementioned method.

On the other hand, there is another method for countervailing the irregularity. This method consists in pre-patching column head with irregularity absorbing material. The use of ratex mortar which is produced by mixing rubber ratex with cement mortar is considered to match to the purpose. According to the test results, it was possible to reduce Young's modulus of ratex mortar to 0.8×10^5 kg/cm^2 by using ratex on the market. It is also shown that with the placement of the mortar, the irregularity of 1 millimeter can be absorbed by the cushion function of the mortar. For the irregularity exceeding 1 millimeter, however, such a method as using block of ratex mortar and grouting after the absorption of large irregularity by full utilization of plasticity of the blocks is recommendable as an effective measure.

Conclusion

For the construction of concrete composite rigid frames, jointing precast concrete girder and column by prestressing is considered as a method for shortening construction period and minimizing undesirable effect of construction work such as interference in traffic flow. Using large scale composite frame models and π-shaped specimens, elasto-plastic behavior of a composite rigid frame was examined together with the mechanical behavior of an anchorage system of prestressing steel. Within the limit of experiments, following conclusions were obtained.

(1) Structural characteristic of a composite frame composed of precast concrete girder and column could be found in the decrease of the jointed corner. The rigidity of the corner decrease rapidly due to the detachment of the column heads from the girder when the moment exceeds a certain value. This value corresponds to the moment when the fiber stress of the column section gets reduced down to zero. Therefore, it is recommended to give enough jointing prestress so as to retain the fiber strain compressively under the working load in order to give the same structural characteristic as in a monolithic rigid frame.

(2) Due to the reduction in rigidity of the jointed corner, moment in a composite frame is distributed as if the stiffness of columns were reduced to the action of vertical load, and to the action of horizontal load the moment is distributed as if the stiffness of girder were reduced. It is indicated that this effect can be analyzed by introducing an idea of a structural spring at the corner, i.e. "joint ductility coefficient".

This ductility coefficients depend on the shape of column section, amount of prestressing force, grouting around prestressing tendons and their locations, besides the amount of axial force and working moment, and the effect of these factors on the coefficient can be estimated roughly. Rigidity of a jointed corner can be increased effectively by grouting around prestressing tendons, locating the tendons to the vicinity of outer faces and adding supplemental reinforcement, besides giving large amount of prestressing force for jointing.

(3) Facial irregularities of jointing members can be classified into two groups, i.e. height of columns and faces of column heads and subsurfaces of girders, according to the effect of the behavior of a precast composite frame. The difference of column height produces large amount of internal stress at the time of jointing members, and this effect could be calculated. The effect of the unevenness of the column head or the subsurface of girder could be analyzed by the similar procedure as mentioned in (2).

(4) Failure mode of a proposed anchorage method of prestressing steel in concrete at midheight of a hollow column is classified into two sorts i.e. bond failure between the inside concrete and the outside concrete and tensile failure of the outside concrete. If prestressed concrete piles of large diameter are used as precast columns, the anchorage zone should be reinforced against the tensile failure. The amount of reinforcement can be estimated roughly. Thus, this type of anchorage system is worthy of note especially in the case of relatively long columns.

Acknowledgement

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Table 1 Specimens and test results

	Properties of piles						Comp. Strength of inside kg/cm²	Test results				
	D_o cm	D_i cm	l_o cm	Stress kg/cm²	Comp. strength kg/cm²	Inside surface		Fail. mode	P_u ton	Ult. bond stress	Cal. of P ton	$\frac{P_u}{P_{cal}}$
PD1	20	10.8	5	0	585		360	Bond	7.9	47		
PD2	20	10.8	10	0	585	shear	360	Bond	13	39		
PD3	20	10.8	10	0	585	key	440	Bond	20	(60)		
PD4	30	18.2	20	0	585		440	Tens	31		25.3	1.23
PD5	30	19.6	10	0	585		440	Bond	20	32		
PD6	30	18.0	10	80	565		350	Tens	26		25.5	1.02
PD7	30	18.0	20	80	565		350	Tens	26		25.5	1.02
PD8	30	20.0	10	80	565		350	Tens	20		23.6	0.85
PD9	70	50	25	60	500		220	Tens	60		114	(0.53)
PD10	70	50	25	60	500	heavy	305	Tens	47		114	(0.41)
PD11	70	50	35	60	500	laitan-	305	Tens	68		114	(0.60)
PD12	70	50	45	60	500	ce	305	Tens	71		114	(0.62)
PD13	70	50	27	60	500		210	Tens	75		114	(0.66)
PD14	70	50	27	60	500	rough	520	Tens	118		114	1.04
PD15	70	50	35	60	500		520	Tens	93		114	0.82
PD16	70	50	45	60	500		520	Tens	110		114	0.97

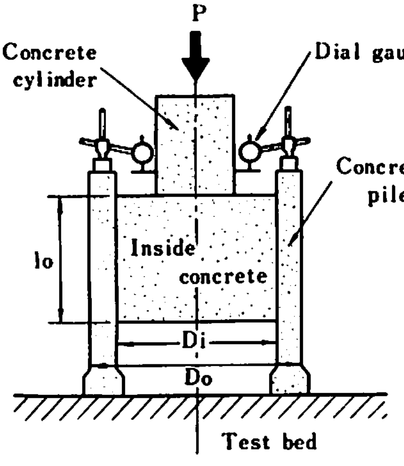

Fig.1 Test procedure of push-out test

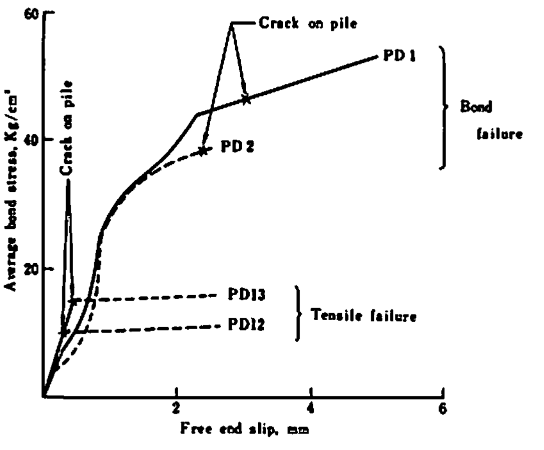

Fig.2 Relation between bond stress and free end slip

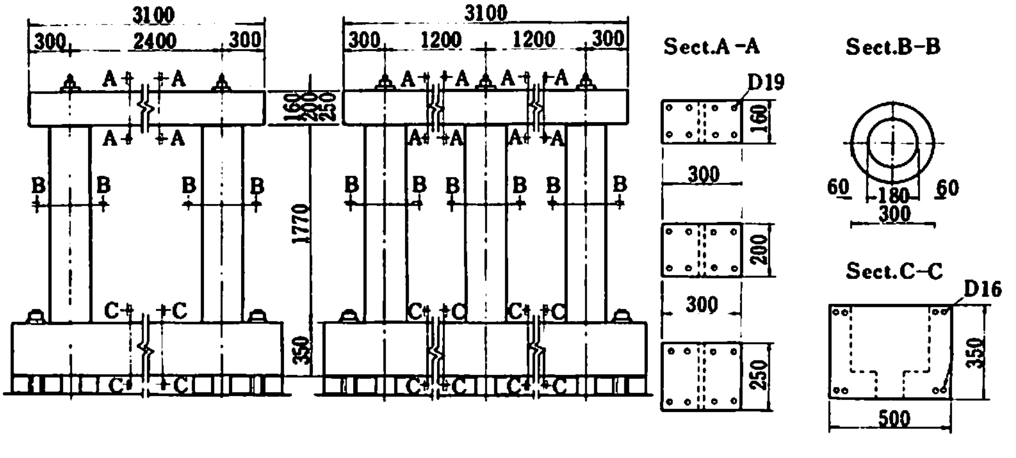

Fig.3 Large scale models of composite frames

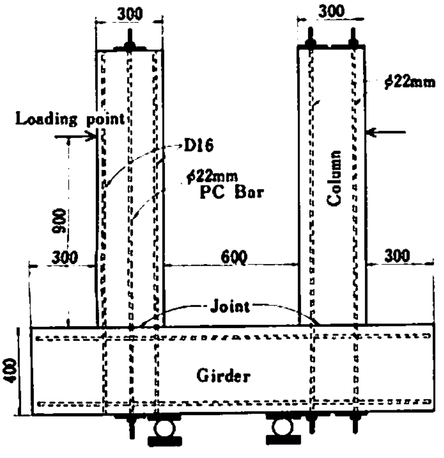

Fig.4 Test procedure of π-shaped specimens

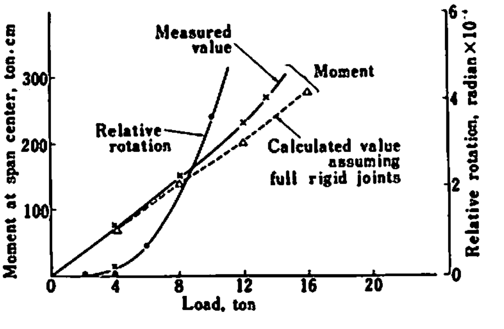

Fig.5 Moment at span center of a composite frame and relative rotation of the joint

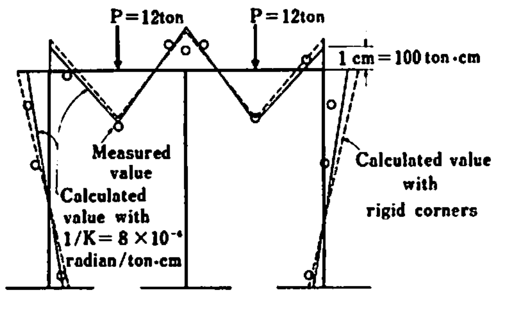

Fig.6 An example of moment distribution in a composite frame

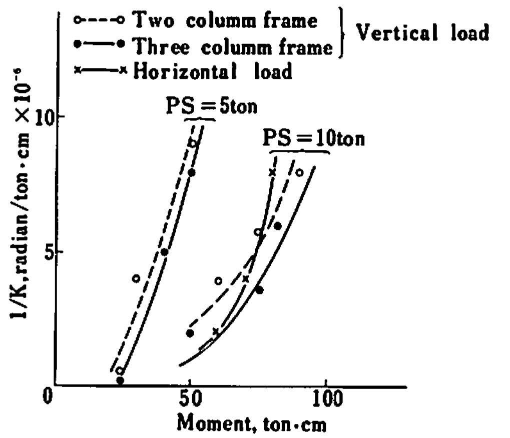

Fig.7 "Joint ductility coefficient"

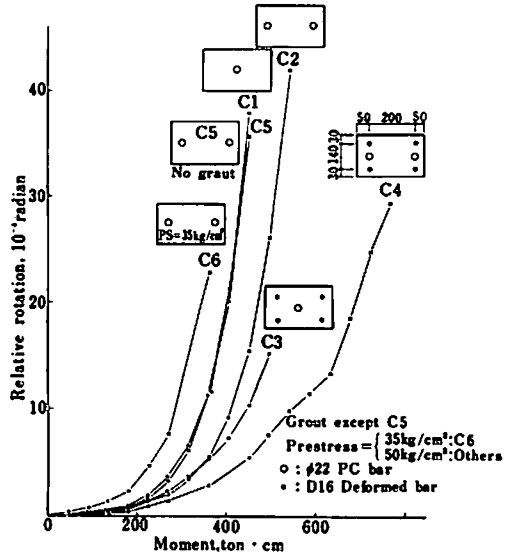

Fig.8 Measured relative rotation in various types of joints by 4 cm measured length

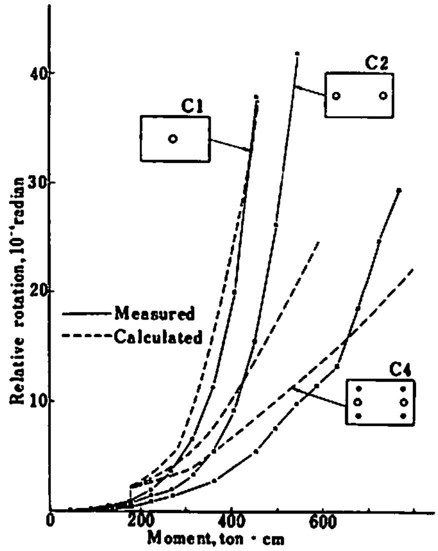

Fig.9 Calculated relative rotation by the method in 2.4

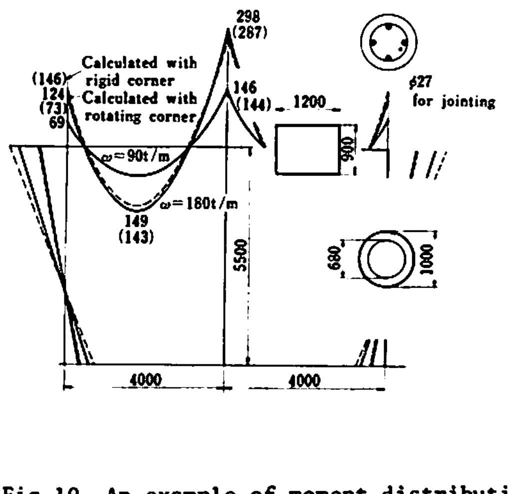

Fig.10 An example of moment distribution in a composite frame with rotation calculated by the method in 2.4

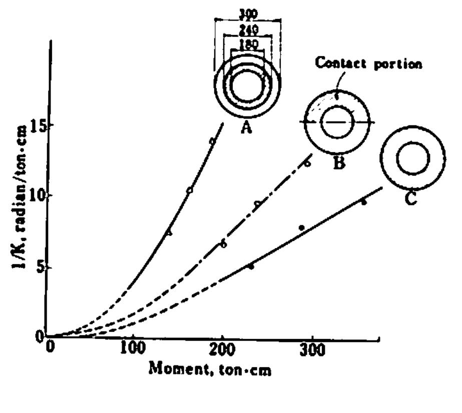

Fig.11 Effect of irregularity of column head on the "Joint ductility coefficient" 1/K