Original Paper

Basic Study on Connection of Precast Concrete Units

*By Hiroshi Matsushima**

Synopsis: The Precast-block Method has many advantages for design and construction of the concrete structure, but the subjects about connection of the concrete units are unsolved yet.

This paper is discribed about strength and deformability of the concrete member of two units connected by steel. This study is based on the data which were obtained when I undertook the task of teaching the students of CHALMERS TEKNISKA HÖGSKOLA in Sweden.

1. Purpose of Study

The purpose of this study is to get the basic data about connection of the precast concrete units by steel. Especially it aims to clear the influences of type, diameter, yield strength of the steel, and concrete strength on the load carrying capacity and deformability of the jointed concrete members.

Notations:

A_s: cross sectional area of steel (cm^2)

P: tensile force (KN)

W_c: crack width of artificial crack, or deformation of jointed section (mm)

σ_{sy}: yield strength of steel (MPa)

σ_u: maximum strength of steel (〃)

σ_c: compressive strength of concrete (〃)

σ_{tu}: tensile strength of concrete (〃)

2. Test Method

2.1 Specimens

The shape and dimensions of the specimens are shown in **Fig. 1**.

A steel plate (1 mm thick) is placed at the center of the specimens, and separated two concrete units are connected by one bar.

Series of the specimens are shown in **Table–1**, which vary for the type, diameter, yield strength of connecting bars and strength of concrete.

The shapes of end anchorage of the connecting bars are shown in **Fig. 2**, and dimensions r and c are shown in the above table.

2.2 Materials — steel and concrete

Strength of steel and concrete are shown in **Table–2**.

2.3 Methods of loading and measuring

Load is pure tension and loading system is as follows.

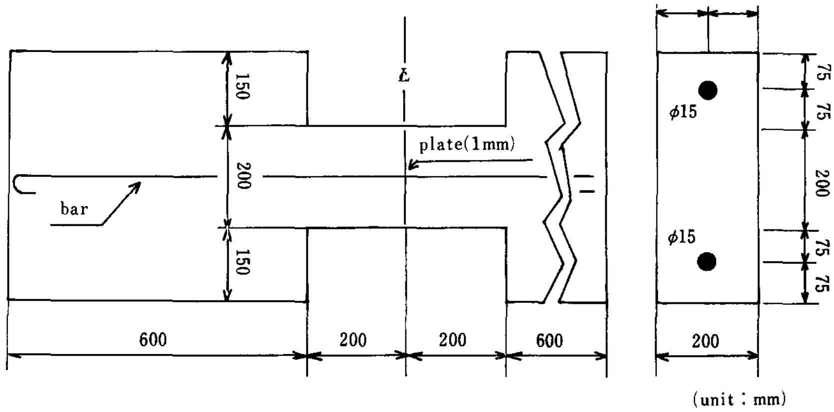

Fig. 1 Dimensions of specimen

* Professor, Dept. of Civil Engineering, Doctor of Engineering.

Table-1

steel		concrete	No.	—	r(mm)	C(mm)
K_s40	ϕ 8	K250	1	*	17	52
	ϕ 10	K150	12	*	33	90
		K250	2	*	17	50
			14	*		
			16	*	33	50
	ϕ 12	K250	3	*	18	62
			18	*		
			20	*		
	ϕ 16	K150	13	*	17	68
		K250	4	*		
			19	*		
		K400	15	*		
	ϕ 20	K250	5	*	17	95
K_s60	ϕ 10	K250	8	*	34	80
	ϕ 16		9	*		
S_s26	ϕ 10	K250	6	*	33	80
	ϕ 16		7	***	33	100
P_s50	ϕ 10	K250	17	*	30	54
		K400	11	*	33	85
strand	ϕ 9	K250	10	**	—	—

r and C are shown in **Fig. 2**
*, ** and *** are the types of shapes of anchoraged end shown in **Fig. 2**

Specimens are placed on the rollers and they are free to remove horizontally.

Load, deformation at the central jointed section and strain of steel at eight or ten points are measured and recorded automatically. Strain gauges for the steel are placed as shown in **Fig. 4**.

3. Test Results

3.1 Data of load, deformation and steel strain

These data are measured and recorded automatically at many load levels of constant load increment. These are shown in appendix. (Appendix is omitted in this paper.) Some examples of graphs which show the relations between these data are **Figs. 5~7**.

About the load and deformation of some specimens, there are differences between the direct measured data and automatically described graphs. This is due to the loading method that load increased to ultimate but repeating up and down sometimes. About these specimens, graphs are described mainly by the automatical

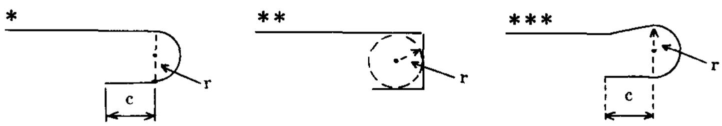

Fig. 2 Shape of anchoraged end

Table-2

steel					concrete			No.
	ϕ (mm)	σ_{sy}	σ_u	E_s		σ_c	σ_{tu}	
K_s40	8	446.219	679.625	0.211	K250	29.617	2.922	1
	10	463.871	654.127	0.209	K150	29.225	3.217	12
					K250	29.617	2.922	2
						36.384	3.736	14
						41.386	?	16
	12	453 083	650.204	0.210	K250	29.617	2.922	3
						31.186	2.824	18
						32.265	2.462	20
	16	485.447	745.332	?	K150	29.225	3 217	13
					K250	29.617	2.922	4
						31.186	2.824	19
					K400	51.781	3.913	15
	20	480.543	745.332	?	K250	29.617	2.922	5
K_s60	10	627.648	756 120	0.212	K250	35.109	3.403	8
	16	?	?	?				9
S_s26	10	349.129	457 006	0.193	K250	35.109	3.403	6
	16	327.554	467.794	0.190				7
P_s50	10	502.444	626.962	0.192	K250	32.265	2.462	17
					K400	51.781	3.913	11
strand	9	2030.049	2069.277	?	K250	30.500	2.962	10

(unit: σ, E. $.10^6$......MPa)

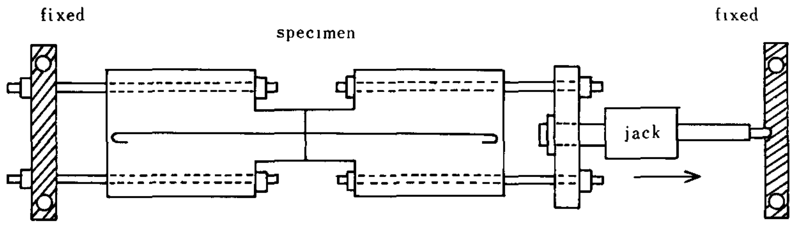

Fig. 3 Loading system

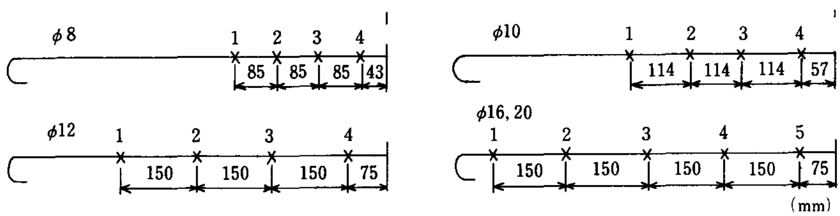

Fig. 4 Strain gauges

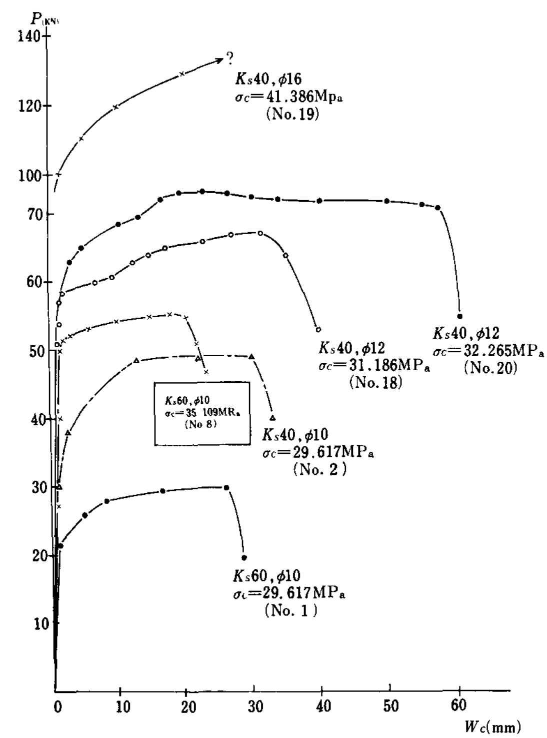

Fig. 5 P—W_c curves (K_s40, 60)

described one.

3.2 Progress to failure of the specimens

The pattern of the load-deformation curve can be seen clearly from **Fig. 8** which is summarized from **Figs. 5~7**, for the specimens with K_s40, K_s60 and S_s26 bars. For the specimens with P_s50 bars, this pattern is not so much clear as K_s specimens. For strand-specimens it may be possible to consider the same pattern by defining the points of B, C and D in **Fig. 7**.

In this figure, A — B stage is the "semi-monolithic stage" in which the specimen shows the similar behavior as the monolithic member. C — D stage is the "separate or non-monolithic state" in which the two units have separated and jointed member has lost its continuity.

Some examples of the load-deformation-steel strain curve are shown in **Figs. 9~11**, for K_s40, K_s60 and S_s26 specimens. Another graph is **Fig. 12** which shows the steel strain at each gauge point for various load levels.

a) A — B stage

The load-deformation curve is almost linear. Slight bond damage may begin near the connecting section and slowly progress to inside. **Figs. 9~12** show that in the early stage, strain of steel is low and this strain at center is not so higher than of inside. At the load near point B, steel strain only near the connecting section increases rapidly, but it is of course below the yield strength.

At point B, the bar resists against the tensile force by bond, and the end anchorage does not effective yet. **Fig. 12** shows this behavior.

b) B — C stage

A — B and C — D stages are comparatively stable. This B — C stage is a transition area which rapidly

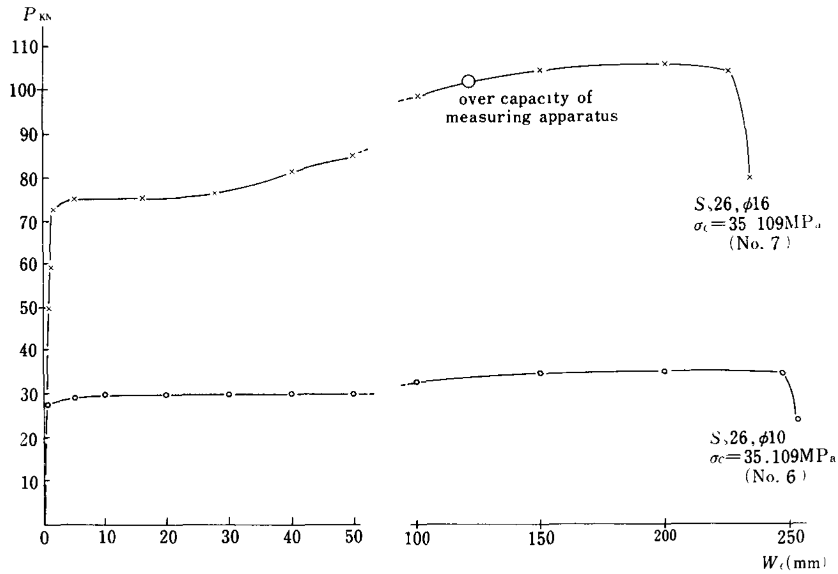

Fig. 6 P—W_c curves (S_s26)

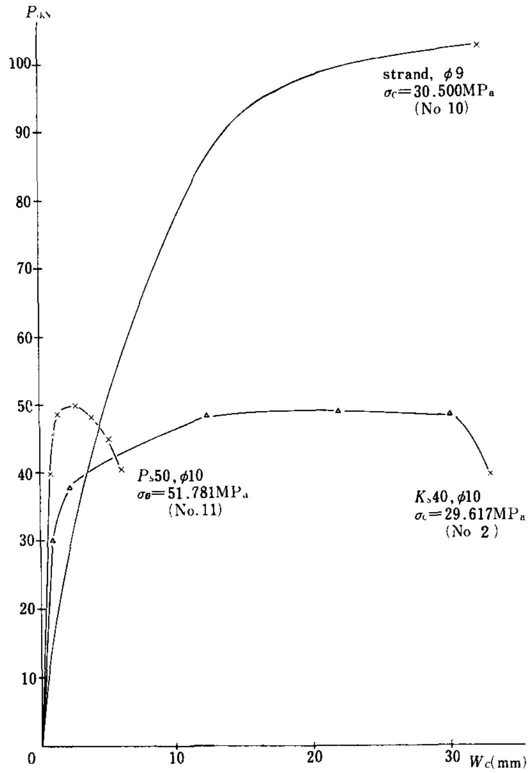

Fig. 7 P—W_c curves $\begin{cases} \text{strand} \\ P_s50 \\ K_s40 \end{cases}$

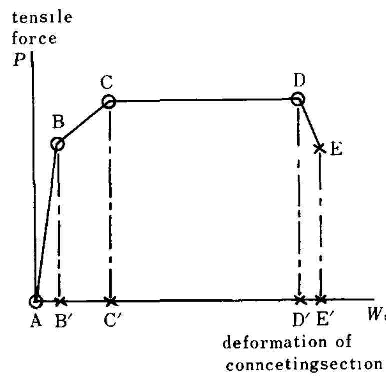

Fig. 8 Pattern of P—W_c curve

progresses and unstable. This may be due to the abrupt bond failure at point B.

At point B, deformation increases and stiffness falls remarkably. Between the points B and C, steel in both side of center yields for some length. This shows that bond has almost disappeared there. But during the end anchorage is complete, bond is not so much damaged and steel strain is small far from the center as shown in **Fig. 12.**

c) D — E stage

Fracture of steel begins and ends at points D and E respectively. Progress to ultimate state is abrupt. For the purpose of estimate the load carrying capacity and

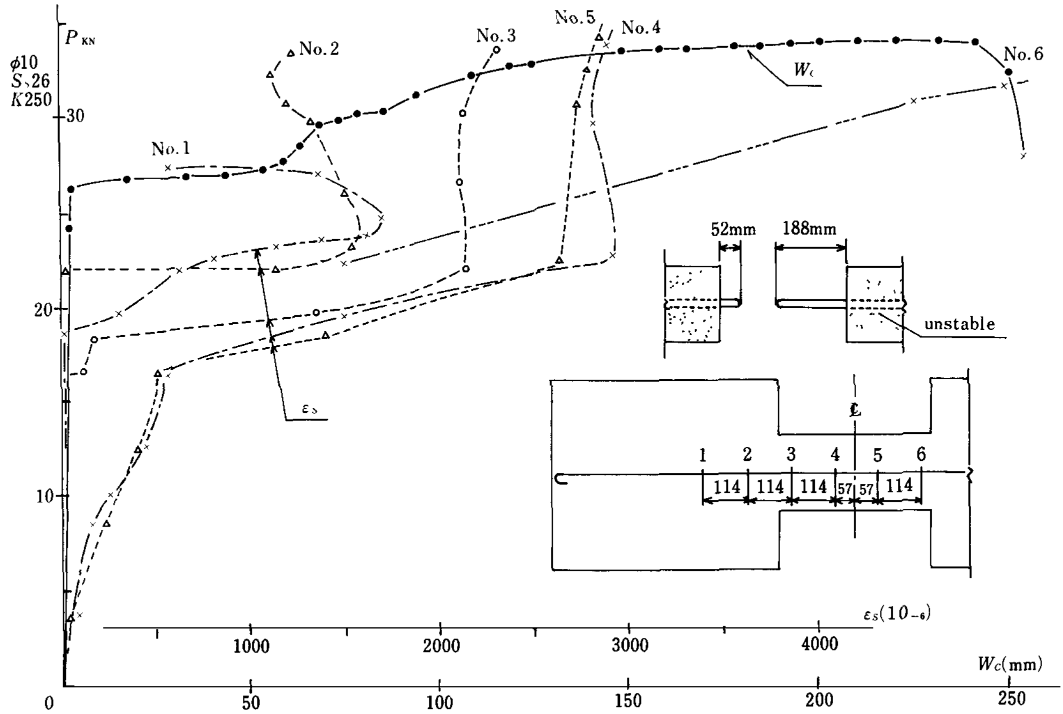

Fig. 9 $P—W_c—\varepsilon_s$ (No. 6)

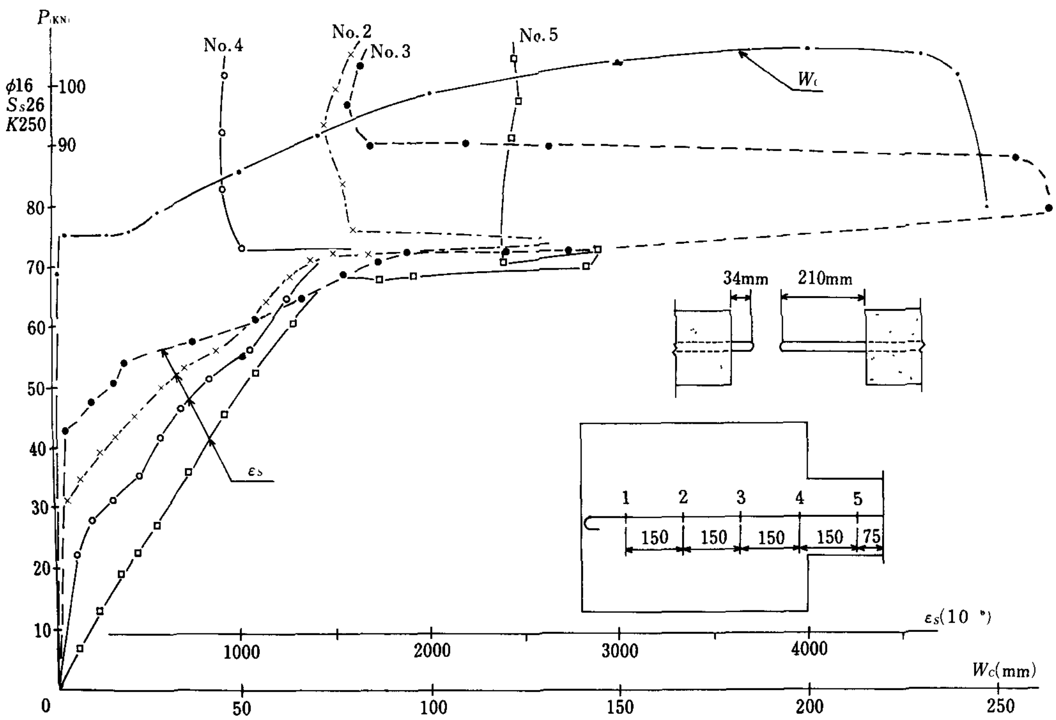

Fig. 10 $P—W_c—\varepsilon_s$ (No. 7)

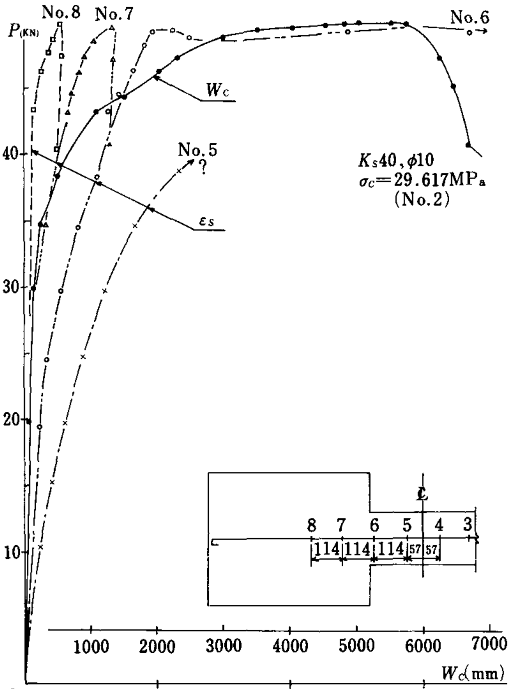

Fig. 11 P—W_c—ε_s (No. 2)

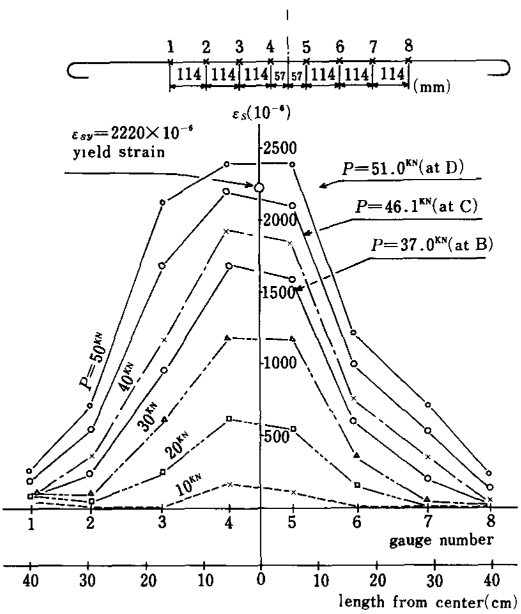

Fig. 12 Steel strain at each point

deformability of the member, it is proper to neglect this stage.

4. Considerations about the Test Results

The jointed member that is connected by steel under pure tension has the above mentioned failure progresses. Based on this behavior, the criterions required for connection will be considered. And then the influences of the test conditions on those criterions will be analyzed to get basic data for the design and future study.

4.1 Criterions required for connection of units

Under certain range of load which is larger than the design load considered safety factor, the jointed member is desirable to have the same properties as the monolithic one. After losing unification under overload, the member should have large deformability without decreasing the load carrying capacity.

From these points of view, the required criterions will be considered for each stage as follows:

a) A — B stage

To get monolithic property of the jointed member, $\tan\theta_1$ (**Fig. 13**) should be large: initial stiffness is large,

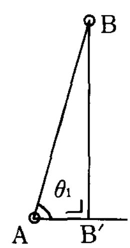

Fig. 13 A→B

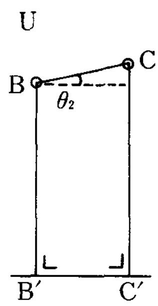

Fig. 14 B→C

so under design load the member shows the similar behavior to the monolithic one. And B — B′ is large: monolithic range is wide.

b) B — C stage

It is desirable that the member transits from A — B stage slowly to C — D stage, so that mild redistribution of stress in the structure can be done. For this purpose, strain energy and stiffness ($\tan\theta_2$) are desirable to be large.

c) C — D stage

To get large deformability of the member keeping the load not to decrease, strain energy is necessary to be large and $\tan\theta_3$ (**Fig. 15**) is desirable not to be less

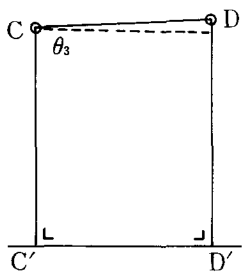

Fig. 15 C→D

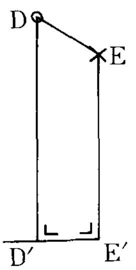

Fig. 16 D→E

than zero.

d) D — E stage

Load carrying capacity is suddenly lost at point D, so this stage may be neglected.

Jointed member of the concrete units is desirable to have all above criterions. But it is not easy to satisfy all these at the same time, and this fact will become clear by the following analysis. So, according to the design conditions the better connecting method shall be chosen.

In the following, influences of some test conditions on the above criterions will be consideres.

4.2 Influences of types of steel

These influences are analyzed by the test data showing in **Table-3**. K_s40 and K_s60 steel have not only different σ_{sy} but also ε_w, so these may be considered as another types.

Table-3

steel		concrete		No.
ϕ 10	S_s26	K250	35.109*	6
	K_s40	K250	35.796	2, 14, 16
	K_s60		35.109	8
	P_s50		32.265	17
		K400	51.781	11
	strand	K250	30.500	10

*: unit......MPa

In **Fig. 17**, load-deformation curves for these specimens are shown, and then **Fig. 18** can be described climinating differences of steel yield strength from **Fig. 17**. In **Fig. 18**, vertical axis P is changed taking K_s40 (σ_{sy}=463.871 MPa) as standard Namely,

$$P\times\frac{\sigma_{sy}(K_s40)}{\sigma_{sy}(\text{each one})}\times\frac{A_s(K_s40)}{A_s(\text{each one})}$$

note: A_s of standard (ϕ 9) is assumed to

be equal to $\frac{0.9^2\pi}{4}\times 0.8\ \text{cm}^2$

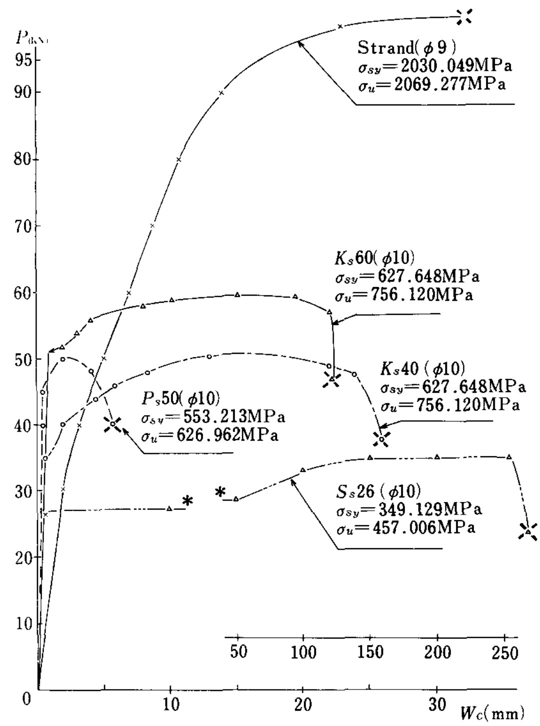

Fig. 17 P—W_s curves (K_s, S_s, P_s, Strand)

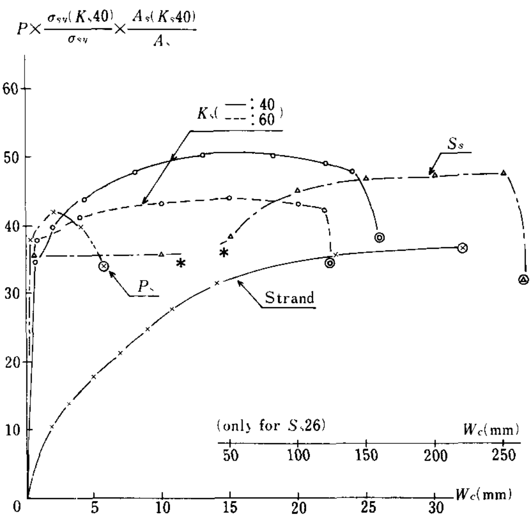

Fig. 18 P—W curves (clininating difference of E_{sy})

These small differetice of concrete strength (except for No. 11) is negligible as shown in 4.4.

Fig. 18 and another graphs show many differences about the shapes of load-deformation curve and deformability for each specimen with various type of steel as follows.

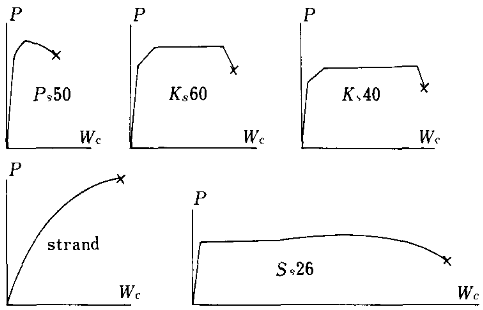

Fig. 19 Types of $P—W_c$ curves

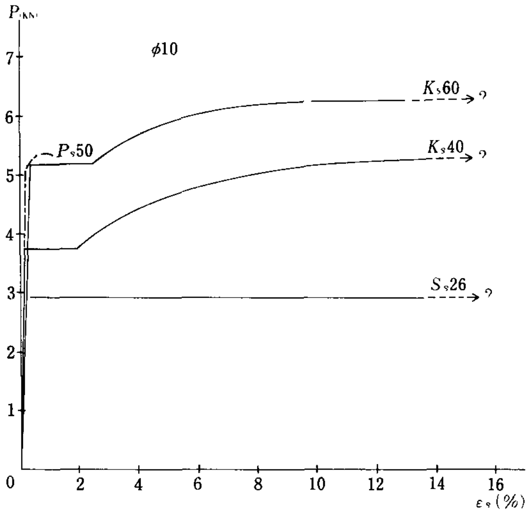

Fig. 20 $P—\varepsilon_s$ relations of bars (ϕ 10)

These differences are mainly due to the stress-strain relationship of each steel, and influence of bond properties of each steel may be also reflected. For reference, $P — \varepsilon_s$ curves are shown in **Fig. 20**.

Influences of types of steel are considered as follows.

a) A — B stage

On stiffness the differences cannot be found between the specimens with K_s, P_s and S_s steel. Strand is inferior in bond properties and Young's modulus is comparatively small, so stiffness of this specimen is small.

Load at point B is defined according to yield strength of steel.

b) B — C stage

Stiffness is smallest of the specimens with S_s26, and this is due to the property of this steel. Prominent differences can not be found between the specimens with other steels.

c) C — D stage

Specimens with P_s50 have few strain energy, because P_s steel is very brittle.

K_s specimens can keep load carrying capacity and strain energy is large. This is due to large deformability and good bond property of this steel.

Strand specimen has increasing load carrying capacity till the end of this stage and strain energy is also large.

S_s specimens don't show decrease of load carrying capacity. Deformation of this is very large. This is due to the fact that bond is broken for almost all length of the bar, and bar's elongation becomes large under high stress level.

But in this test, deformation of S_s specimens is too large. This is because of failure of end anchorage of steel judging from the next facts.

(1) after failure of the specimen, pullout of steel can be observed as shown in **Fig. 21**.

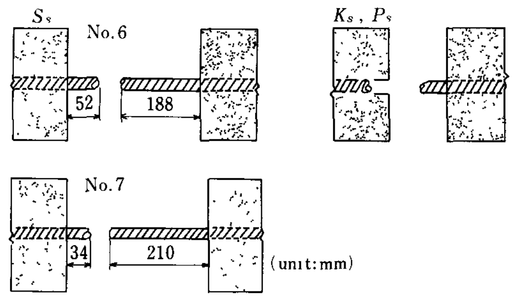

Fig. 21 Pull-out of steel (S_s26)

(2) for more than certain load level, steel strain doesn't increase and keeps almost constant. Differences of this behavior between S_s and K_s specimens can be seen in **Figs. 9～11**.

From above considerations it can be concluded as follows.

K_s specimen has good properties in all stages.

S_s specimen has the same stiffness in A — B stage as other's, and has large strain-energy in C — D stage. But the stiffness in B — C stage is very small and end anchorage is apt to failure because of its inferior bond property.

P_s specimen has few strain-energy in all stages, so this steel is unsuitable for connecting.

Of strand specimen, stiffness is small in A — B stage but large in B — C stage. In C — D stage, it has large strain-energy with increasing load carrying capacity.

Table–4

	steel				concrete			No.
	ϕ (mm)	σ_{sy}	σ_u	E(10^6)		σ_c	σ_{tu}	
K_s40	8	446 219	679.625	0.211	K250	29.617	2 922	1
	10	463.871	654.127	0.209	K150	29.225	3 217	12
					K250	29.617	2.922	2
						36.384	3 736	14
						41.386	?	16
	12	453.083	650 204	0 210	K250	29.617	2.922	3
						31.186	2.824	18
						32 265	2 462	20
	16	485.447	745.332	?	K150	29.225	3 217	13
					K250	29 617	2 922	4
						31.186	2 824	19
	20	480.543	745 332	?	K250	29.617	2.922	5

unit: σ, E……MPa

This steel can give the jointed member ductile behavior. It may be possible to get high stiffness in A — B stage by using together with K_s steel.

4.3 Influences of diameter of steel

This is considered about the following specimens which have the same conditions except for diameter.

Of these specimens, load and deformation at each point B, C, D and E are shown in **Table–5**.

Using these values, stiffness, strain-energy for each stage and load at B can be shown as in **Table–6**.

To compare stiffness and strain-energy between the specimens with different diameter bars, specimen with ϕ 12 bar is taken as standard. The ratios of these are in **Table–7**, considering these of ϕ 12 specimens are equal to 1.00.

The relations between these ratios and the diameters of bar (also, changing to the ratio of cross sectional area) shown graphically, ratio of stiffness in **Fig. 22**, of strain energy in **Fig. 23** and for each stage in **Figs. 24~26**. Judging from these figures, change of stiffness is as

Table–5

			B	C	D	E
1	ϕ 8	P (KN)	22 0	28 0	30 0	24.0
		W_c(mm)	1.0	8.5	26.2	28.3
12	ϕ 10	P (KN)	37.8	46.5	50.0	36.7
		W_c(mm)	1.31	8.21	29.2	34.8
2		P (KN)	32.0	47.5	49.2	40.0
		W_c(mm)	1.0	12.0	30.0	33.0
14		P (KN)	33.0	48 2	48.5	41.2
		W_c(mm)	1 25	10.21	23.0	25.6
16		P (KN)	37.0	46.1	51.0	25.95
		W_c(mm)	1.29	8.49	19.8	25.5
3	ϕ 12	P (KN)	57.5	66 0	69.3	64 0
		W_c(mm)	3.2	14 0	47.0	52.0
18		P (KN)	53.9	65 5	67.2	53.0
		W_c(mm)	1.05	17 35	31.4	38.8
20		P (KN)	59.6	70.0	67.5	55.1
		W_c(mm)	1.0	16 3	57.8	60.4
13	ϕ 16	P (KN)	—	—	—	—
		W_c(mm)	—	—	—	—
4		P (KN)	87.8	—	—	—
		W_c(mm)	1.9	—	—	—
19		P (KN)	90.0	128.0	—	—
		W_c(mm)	0.9	20.0	—	—
5	ϕ 20	P (KN)	108 0	—	—	-
		W_c(mm)	1 3	—	—	—

Table–6

		A——B			B——C		C——D
		stiffness (KN/mm)	strain energy (KN·mm)	load at B (KN)	stiffness (KN/mm)	strain energy (KN·mm)	strain energy (KN·mm)
ϕ 8	1	22.00	11.00	22.00	0 80	187.50	513 30
ϕ 10	2	32.00	16.00	32.00	1 41	437.25	870.30
	12	28.85	24.76	37.80	1.26	290.84	1012.77
	14	26.40	20.63	33.00	1.65	467.10	514.69
	16	28.68	23.87	37.00	1 26	299.16	599.11
	mean	28.98	21.32	34.95	1.40	373.59	749.22
ϕ 12	3	17.97	92.00	57.50	0.79	333.45	2232.45
	18	51.33	28.30	53.90	0.71	973 11	932 22
	20	59.65	29.83	59.65	0.68	991.82	2855.88
	mean	55.45	29.07	57.02	0.73	982.47	2587.73
ϕ 16	4	47.20	81.65	87.80	—	—	—
	19	100.00	40.50	90.00	1.99	2081.90	—
	mean	73.60	61.08	88.90	1.99	2081.90	—
ϕ 20	5	83.72	69.60	108.00	—	—	—

Table-7

steel		A———B			B———C		C——D
ϕ	A_s	stiffness	strain energy	load at B	stiffness	strain energy	strain energy
8	0.48 cm²	22.00	11.00	22.00	0.80	187.50	513.30
	0.44*	0.44**	0.38	0.39	1.10	0.19	0.20
10	0.79 cm²	29.98	21.32	34.95	1.40	373.59	749.22
	0.72	0.54	0.73	0.61	1.92	0.38	0.29
12	1.09 cm²	55.49	29.07	57.02	0.73	982.47	2587.73
				1.00			
16	1.91 cm²	73.60	40.50	90.00	1.99	2081.90	?
	1.75	1.33	1.39	1.58	2.73	2.12	?
20	3.14 cm²	83.72	69.60	108.00	?	?	?
	2.88	1.51	2.39	1.89	?	?	?

$*: \dfrac{0.48\ \mathrm{cm}^2(\phi 8)}{1.09\ \mathrm{cm}^2(\phi 12)}=0.44$, $**: \dfrac{22.00\ (\phi 8)}{55.49\ (\phi 12)}=0.40$

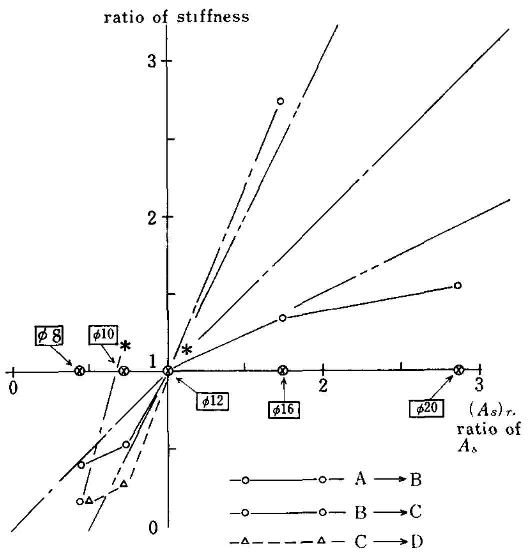

Fig. 22 Stiffness—A_s (by ratio)

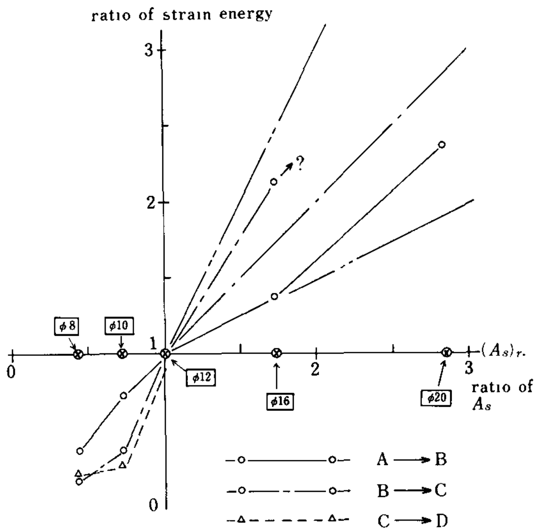

Fig. 23 Strain energy—A_s (by ratio)

follows.

$(A_s)_r$.: ratio of cross sectional area of bar to that of ϕ 12 one

a) increase for large diameter bar,

A — B stage: in proportion to $(A_s)_r.\times 0.5$

B — C stage: 〃 to $(A_s)_r.\times 2.0$

C — D stage: no data

b) decrease for smaller diameter bar,

all stages: in proportion to $(A_s)_r.\times 2.0$

About strain energy,

a) increase for larger diameter bar,

A — B stage: in proportion to $(A_s)_r.\times 0.5$

B — C stage: in proportion to $(A_s)_r.\times 1.0$

b) decrease for smaller diameter bar,

A — B stage: in proportion to $(A_s)_r.\times 1.0$

B — C, C — D } stage: 〃 to $(A_s)_r.\times 2.0$

Namely, when the jointed member with ϕ 12 bar is taken as standard, stiffness and strain energy of the member with smaller bar decreases in proportion to $\Delta A_s\times 2.0$ in all stages. On the contrary, increase for the member with larger bar is proportional to $(A_s)_r.\times 0.5$ in A — B stage and to $(A_s)_r.\times 2.0$ in B — C stage.

Accordingly, it is not proper to use smaller bar than ϕ 12, because not only these specimen has small stiffness and strain energy but also this decreasing ratio is high. when the larger bar is used, in A — B stage the large loading capacity at B can be obtained, and transition from point B to C desirably becomes slowly. About C — D stage, few data could be obtained to analyze, but the conclusions may be the same.

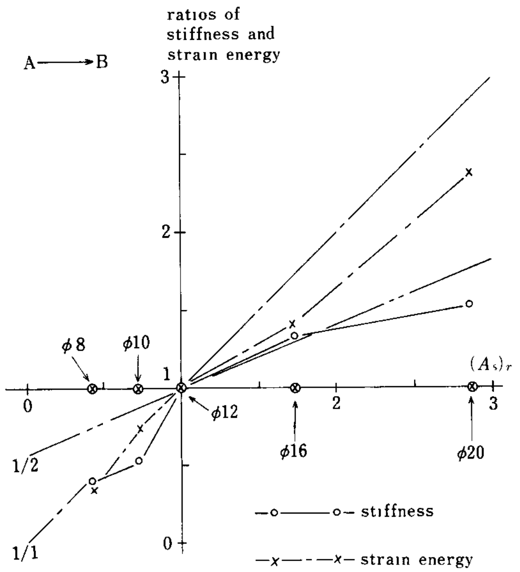

Fig. 24 Stiffness, Strain energy —A_s (by ratio) (A→B)

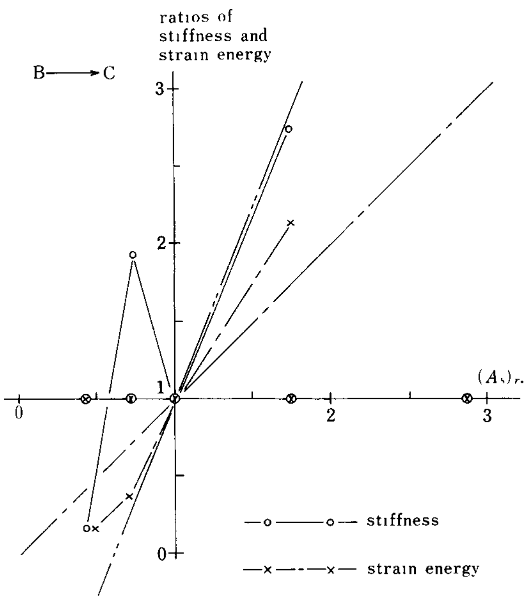

Fig. 25 Stiffness, Strain energy —A_s (by ratio) (B→C)

These considerations are true only for specimen with single bar. For the same steel cross sectional area, plural bars have larger bond strength and will show different influences.

4.4 Influences of strength of concrete

The specimens which have the same conditions expect

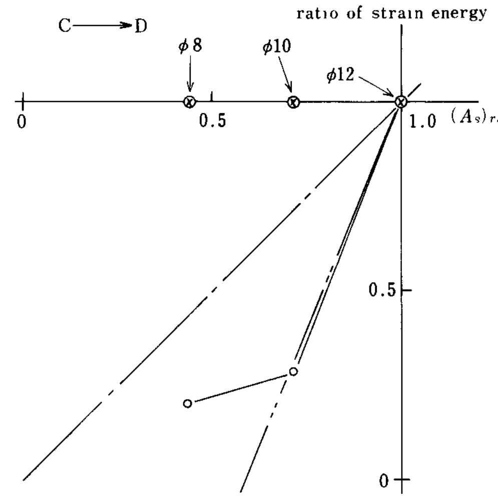

Fig. 26 Stiffness, Strain energy —A_s (by ratio) (C→D)

for concrete strength are shown in **Table–8**.

The influence of concrete strength on connection is mainly due to difference of its bond strength. Bond strength is said to be proportional to square root of its compressive strength, then in the group of ϕ 12 (K_s40), difference of bond strength is only 4% ($\sqrt{32.3/29.6}=1.04$), so this group shall be neglected. From the results of examination, differences could not be found. The group of ϕ 16 (K_s40) can not be examined because these specimens failed too early to get the data.

In the group of ϕ 10 (K_s40), the specimens for $\sigma_c=36.384$ MPa and 41.386 MPa may have 10% and 20% increase of bond strength respectively taking $\sigma_c=29.421$ MPa one as standard. ($\sqrt{36.4/29.4}=1.11$, $\sqrt{41.4/29.4}=1.19$)

a) Progress of bond failure

When the bar embedded in concrete is pulled and bond between concrete gradually damaged, stress that is carried by steel becomes larger and strain also increases. Then, progress of bond failure can be observed by increasing steel strain.

Strain at various points of the bars of specimens of ϕ 10 (K_s40) group are shown in **Figs. 27~30** at some load levels.

Using these graphs, the steel length can be known for which length strain of steel has reached more than certain level. For example, these graphs show that steel strain is more than 200×10^{-6} for 19 cm length of

Table–8

	steel				concrete	No.
	ϕ (mm)	σ_{sy}	σ_u	E(10^6)	σ_c	
K_s40	10	463 871	654 127	0.209	29.225 29.617 36.384 41.386	2, 12 14 16
	12	453 083	650 204	0 210	29.617 32.265	3, 18, 20
	16	485.447	745.332	?	29.225 31 186 51.781	4, 13, 19 15
P_s50	10	553 213	626.962	0.192	32.265 51.781	17 11

(unit: MPa)

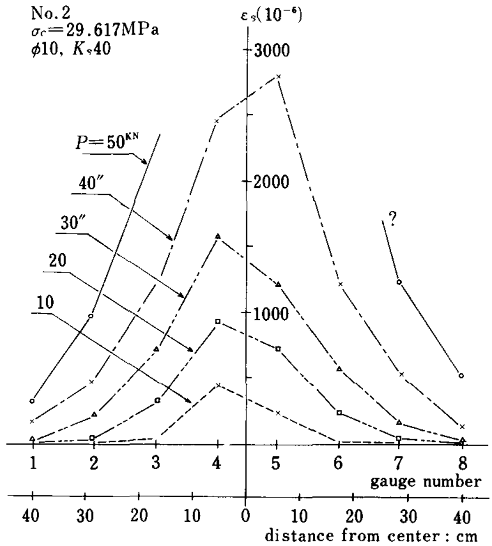

Fig. 27 ε_s at each strain gauge

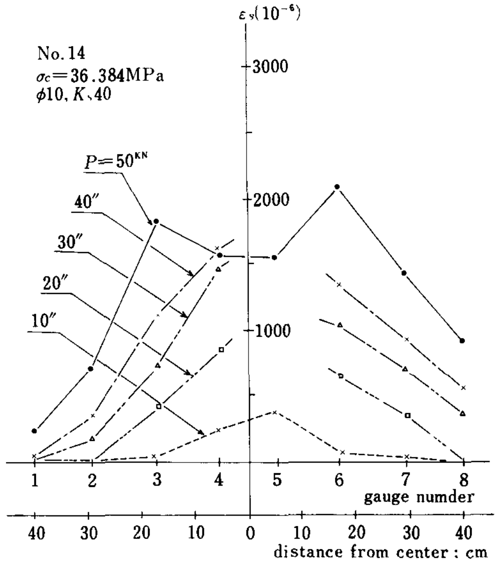

Fig. 29 ε_s at each strain gauge

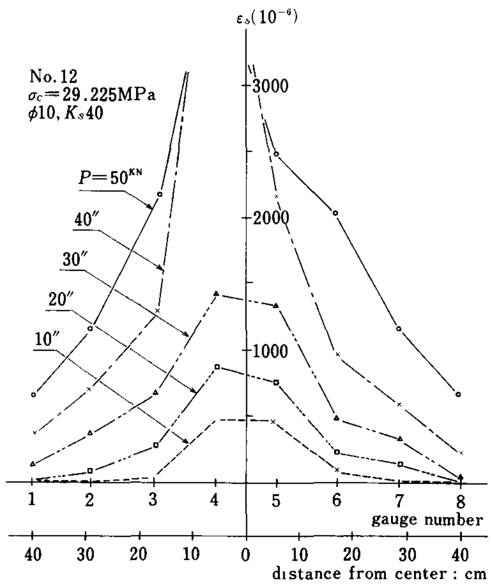

Fig. 28 ε_s at each strain gauge

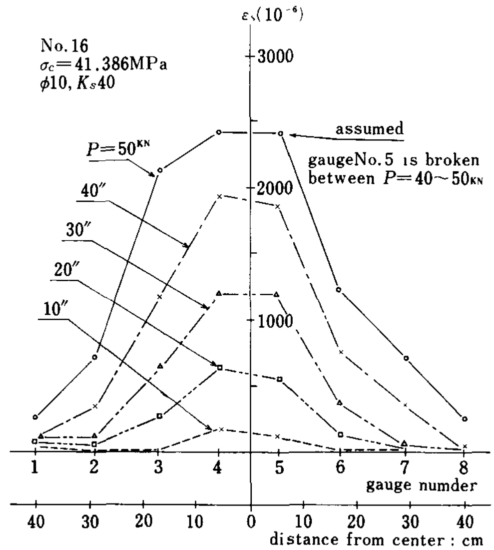

Fig. 30 ε_s at each strain gauge

bar (No. 2, σ_c=29.617 MPa) and zero cm (No. 16, σ_c=41.386 MPa) under tensile force P equal to 10 KN.

From these graphs, length of the bar at some strain levels for each load is measured and is shown in **Table–9–1~2.**

In **Fig. 31** the relations between steel strain and steel

Table–9–1 (unit: cm)

P(KN)	σ_c / $\varepsilon_s(10^{-6})$ \ No.	29.225~29.617 MPa 2	12	mean	41.386 MPa 16
10	200	19	27	23	0
	400	4	14	9	0
	600	0	0	0	0
20	200	42	42	42	36
	400	28	31	30	22
	600	20	19	20	8
	800	9	7	8	0
	1000	0	0	0	0
30	200	57	69	63	50
	400	46	51	49	39
	600	36	35	36	36
	800	29	28	29	24
	1000	23	21	22	17
	1200	14	16	15	6
	1400	8	8	8	0
	1600	0	?	0	0
40	200	80	80	80	70
	400	64	73	69	56
	600	55	60	58	47
	800	49	49	49	39
	1000	42	39	41	34
	1200	35	33	34	29

Table–9–2 (unit: cm)

P(KN)	σ_c / $\varepsilon_s(10^{-6})$ \ No.	29.225~29.617 MPa 2	12	mean	41.386 MPa 16
40	1400	31	29	30	23
	1600	28	26	27	19
	1800	25	24	25	14
	2000	22	21	22	0
	2200	18	19	19	0
	2400	14	17	16	0
	2600	7	16	12	0
	2800	0	?	0	0
50	200	80	80	80	80
	400	79	80	80	74
	600	74	80	77	64
	800	68	74	71	55
	1000	61	65	63	49
	1200	56	57	57	42
	1400	53	51	52	38
	1600	49	46	48	34
	1800	?	41	41	31
	2000	?	35	35	29
	2200	?	26	26	22
	2400	?	23	23	11
	2600	?	19	19	0
	2800	?	16	16	0
	3000	?	13	13	0
	3200	?	?	?	0

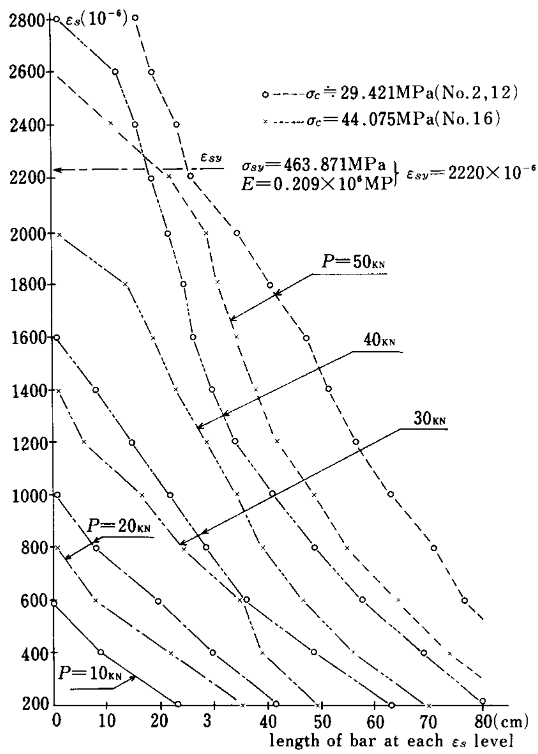

Fig. 31 E_s—l

length are shown at various load levels. This shows that high concrete strength makes progress of bond failure slowly, so it has influences on the behavior of jointed member.

High strength of concrete has another profit to make end anchorage sure, because bond failure does not extend inside too deep.

b) Behavior of jointed section

Stiffness and strain energy at each stage for these three graphs are shown in **Table–10.**

In **Table–11,** these values are shown with ratio by taking the values of the specimen (σ_c=29.421) MPa as standard.

The specimens are small in number, especially we have only one specimen for σ_c=36.384 MPa and 41.386 MPa respectively. So, the definite conclusion cannot be obtained about differences of the behavior. Only one conclusion is that high strength concrete makes strain energy in C — D stage smaller. This is because of the fact that deformation of the jointed member results from elongation of connecting steel, and high strength concrete makes bar's length at high stress level

Table-10

σ_c (MPa)	No.	A——B stiffness (KN/mm)	A——B strain-energy (KN·mm)	A——B load at B (KN)	B——C stiffness (KN/mm)	B——C strain-energy (KN·mm)	B——C load at C KN	C——D strain-energy (KN·mm)	C——D load at C KN
29.225	2	32 00	16.00	32.00	1.41	437 25	47.5	870.30	49.2
29 617	12	28.85	24.76	37.80	1 26	290.84	46.5	1012.77	50.0
	mean	30.43	20.38	34.90	1.34	364.05	47.0	941.54	49.6
36.384	14	26.40	20 63	33.00	1.65	467.10	49.6	514.69	49.0
41.386	16	28.68	23.87	37.00	1.26	299.16	46 1	599.11	46.0

Table-11

σ_c(MPa)	A——B stiffness (KN/mm)	A——B strain-energy KN·mm	A——B load at B KN	B——C stiffness (KN/mm)	B——C strain-energy KN·mm	B——C load at C KN	C——D strain-energy KN·mm	C——D load at C KN
29 421				1 00				
36.384	0.87	1.01	0.95	1 23	1 28	1.06	0 55	0.99
41.386	0 94	1 17	1 06	0 94	0.84	0.98	0.64	0.93

shorter. To get large deformability, it should take the bar long between the jointed section and end anchorage. Further study is necessary to clear these problems.

5. Problems to Study in Future

This basic experimental study should be continued to clear the following many unsolved problems.

(1) Theoretical analysis is necessary to know the general equations for calculating load carrying capacity, deformability and so on of the jointed member, considering a analytical model.

(2) The test was suggestive and valuable for me, and was carried out systematically with good schedule. And I want to continue the supplemental test considering the following points.

a) Number of specimens should be about five for the same test condition.

b) Data about the group which have different concrete strength are insufficient. Concrete strength must have some influences on the behavior of the jointed member.

c) Tests prefer to do about the specimens mainly with larger diameter and different embedded length of bars.

d) Examine should be carried out also about the specimens with plural bars, because plural smaller bars must have large bond strength than single one with the same cross sectional area.

This is the basic study. Using the proper jointed methods which are obtained here, the study should be continued especially about behavior under large deformed condition (membrane action) with large specimens, and also under some other kinds of load.

6. Conclusions

Precast concrete jointed member is desirable to have monolithic property under comparatively small load level including design load, and to have wide load range of this monolithic state.

Under exceeded load, the member loses remarkably its unification and deformation becomes large. In this stage it should have large deformability. And the former state is desirable to transit slowly to the latter state for the mild progress of redistribution of forces in the structure.

The following conclusions are obtained in this test using the specimens connected two concrete units with single bar situated at the centroid of the cross section, and the specimens are under pure tension.

(1) The progress to failure of the specimens is shown in **Fig. 8**. This behavior is recognized especially for K_s and S_s specimens. In this figure, A — B stage is the semi-monolithic area and C — D stage is the characteristic state for jointed members, in which two units are separated and superiority or inferiority of the connecting methods can be observed distinctly. B — C stage is the transition area between these two stages and generally under unstable condition.

(2) Influences of type of steel

S_s specimens have small load at B due to low yield strength of S_s26 steel, and drop of stiffness at point B is remarkable. Under exceeded load, bond failure spreads for almost all length of the bar, and the anchoraged end is subjected to load directly. The end of bar should be anchored firmly.

K_s specimens have statisfactory properties for all above mentioned stages. K_s40 steel may have larger ultimate strain ε_u than K_s60, so in C — D stage K_s40 specimens have larger deformability than K_s60 ones. To get large strain energy in C — D stage, it will be better to use K_s40 steel but using larger cross sectional area.

P_s specimens are not proper for jointed members, because deformability is very small in B — C — D stages. This is due to brittle property of P_s steel.

Strand specimens have a weak point that these initial stiffness is small, but drop of stiffness is mild to the ultimate state and they have large deformability. This is excellent for ductile connecting method. It may be worth to examine the method of using strand with K_s or S_s steel together.

(3) Influences of diameter of steel

In many cases, the bar larger than ϕ 12 may be used for connection, so the specimen with ϕ 12 bar is taken as standard in this analysis.

Of the specimens with smaller than ϕ 12 bar, drop of stiffness and strain energy in all stages is proportional to $(A_s)_r.\times 2.0$.

Of the specimens with the larger bars, load at point B becomes larger and increase of stiffness in B — C stage is proportional to $(A_s)_r.\times 2.0$. These are good properties for connection. When larger amount and plural number of bars are used, good influences due to increasing of bar's area and bond strength can be obtained.

(4) Influences of strength of concrete

Definite conclusions can be obtained because of lack of the test data. But it is clear that high strength concrete has a tendency to make deformability in C — D stage smaller. This behavior may be due to the fact that bond failure progresses to inside slowly, so elongation of the bar is small. But end anchorage of the bar is confirmed even under the exceeding high load level. To get large deformability of high strength concrete member, it may be better to use many number of small bars and take their long embedded length.

7. Acknowledgement

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