Experimental Study on Steel Plate Reinforced Concrete Shear Walls with Joint Bars

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ABSTRACT

This paper describes the experimental study on the bond splitting strength between concrete and joint bars confined with the steel plates, and the behavior of the steel plate reinforced concrete shear walls anchored to a reinforced concrete base slab with joint bars under horizontal loading. For these objects, two type tests were carried out. One was an axial pulling out test, the other was a horizontal loading test. By the axial pulling out test, it is found that the previous evaluation equation for the bond splitting strength can be applied. By the horizontal loading test, it is found that the steel plate reinforced concrete shear walls have good ductility, and the relationship between shear force and deflection of this structure can be evaluated by the method which is developed based on the previous method. These studies enable the steel plate reinforced concrete shear walls with the joint bars to be applied to actual structures.

INTRODUCTION

The steel plate reinforced concrete is an effective and useful structural material to replace reinforced concrete. By switching to factory fabrication of steel plates, construction period and improvement in quality can be expected. The steel plate reinforced concrete consists of surface steel plates and concrete. They are jointed with studs. To apply the steel plate reinforced concrete to shear walls, a joint system between the steel plate reinforced concrete shear walls and the reinforced concrete base slab should be necessary. On the study [title : Study on A Steel Plate Reinforced Concrete Structure for Nuclear Power Plants], three type joint systems (shown in Fig.1) were investigated. The first type is a system of inserting steel plates into the base slab (steel plate inserting system). The second one is a system with anchor bolts (anchor bolt system). On this system, the stress of the steel plates is transferred to the base slab with the anchor bolts which are set at the leg portion of the steel plates. The last one is a system with the joint bars (joint bar system). On this system, first, the stress of the steel plates is transferred to concrete of walls with the studs. Secondly, the stress of concrete is transferred to the joint bars by the bond between the joint bars and concrete. Finally, the stress of the joint bars is transferred to the base slab. On this system, tie bars are set within the wall to avoid the bond splitting failure, and on this case, the evaluation equation for the bond splitting strength was obtained. The region where tie bars are set is the lower portion of the wall where the joint bars are set. Of these three systems, the joint bar system is the most convenient. Because, on the steel plate inserting system, the steel plates interfere with reinforcement bars in the base slab, and on the anchor bolt system, too many parts require to be welded.

On the joint bars system, the more convenient system was developed. On this new system, the partitioning steel web plates are set instead of tie bars. The partitioning steel web plates are set at narrower region than tie bars. The concept of this system is shown in Fig.2. This paper describes the experimental study on the bond splitting strength between concrete and the joint bars confined by the partitioning steel web plates, and the behavior of the steel plate reinforced concrete shear walls anchored to the reinforced concrete base slab with the joint bars under horizontal loading. The objects of this study are to confirm the bond splitting strength between concrete and the joint bars confined by the partitioning steel web plates, and the restoring force characteristics and the ultimate strength of the steel plate reinforced concrete shear walls with the joint bars under horizontal loading. For these objects, two type tests were carried out. One was an axial pulling out test, the other was a horizontal loading test. On the axial pulling out test, the bond splitting strength of the double layer joint bars was also investigated.

THE AXIAL PULLING OUT TEST

In this chapter, the bond splitting strength between concrete and the joint bars confined by the partitioning steel web plates is investigated. To confirm the bond splitting strength, the axial pulling out test was carried out.

Specimens

Specimens and material characteristics are shown in Table 1, and the shapes of specimens are shown in Fig.3. The joint bar is D38 (deformed bar, diameter:38mm). The number of the joint bars in the first layer is three. This test consists of two parts. In the first part test, the adaptability of the evaluation equation (Eq.(1)) for the bond splitting strength in the case of using tie bars shown in reference 1) to that of the case of using the partitioning steel web plates was roughly confirmed. In the second part test, the bond splitting strength was investigated in detail.

The parameters of this test are the anchoring length of the joint bars, length of the partitioning steel web plates, the number of the joint bars, and the number of the joint bars layer (one or two). The length of the partitioning steel web plates is obtained from Eq.(2). The partitioning steel web plate ratio (p_w) in Eq.(2) is same as tie bar ratio in Eq.(1). The object of this test was to confirm the bond splitting strength, therefor it was planned so that the failure modes of all specimens except 2S-40/11-72 were bond splitting. The bond splitting strength was estimated by Eq.(1). However, Eq.(1) is an equation for the lower limit strength, so the value obtained from Eq.(1) was divided by 0.9. Because it is found that the average of the previous experimental results can be evaluated by this method. Eq.(1) shows that the bond splitting strength decreases in proportion to the distance between the joint bars and the surface steel plates. In the case that the layer of the joint bars was double, first, the strength of the joint bar in each layer was obtained from Eq.(1), secondly, the strength of the joint bars in the first layer was reduced by multiplying a value (α) obtained from Eq.(3). Eq.(3) is shown in reference 2). The name of each specimen is as follows;

2S-30/5-38

- 38 — the length of the partitionning steel web plate (cm)
- 5 — the partitioning steel web plate ratio(%) x 10
- 30 — the anchor length of the joint bars (a multiple of the joint bar diameter)
- 2 — the number of the joint bars in the second layer

$$\tau_u = 0.31\, m\, (0.267 - 0.017\, a + 13.5\, p_w)\sqrt{\sigma_B} \qquad (1)$$

$$p_w = 2t\, \ell_w / (B\, \ell_p) \qquad (2)$$

$$\alpha = \sum T_1 / (\sum T_1 + \sum T_2) \qquad (3)$$

m : breadth of specimen (B) / total length of diameter of all joint bars
a : length between surface steel plate and joint bars / diameter of a joint bar
p_w : partitioning steel web plate ratio
σ_B : concrete compressive strength (N/mm^2)
t_w : thickness of partitioning steel web plate (mm)
α : reduction factor of the bond splitting strength of the joint bars in the first layer
T_1 : axial tensile force of the joint bars in the first layer (kN)
T_2 : axial tensile force of the joint bars in the second layer (kN)
others are shown in Fig.3.

Loading and Measurement Method

The concept of the loading system is shown in Fig.4. The edges of the joint bars are tensile loading points, and the edges of the surface steel plate are reaction points. Tensile force was loaded with oil jacks.

The relative displacement between the joint bars and concrete surface of the loading side, the displacement between the surface steel plates and concrete surface of the reaction side, and strains of joint bars, surface steel plates, and studs were measured.

Test Results

The test results are shown in Table 2. Except 0S-40/3-35 and 2S-40/11-72, the maximum load of all specimens was determined by the bond splitting failure. The partitioning steel web plates ratios of 0S-30/5-38,0S-30/5-51and 0S-30/5-72 are same. The maximum load of these three specimens are nearly equal, therefor it is found that specimens which the partitioning steel web plate ratios are equal have the same bond splitting strength.

The failure states of 0S-30/5-38 and 3S-30/10-38 are shown in Fig.5 and 6. Fig.5 is the surface state, and Fig.6 is the inner state. The inner state was observed after cutting specimens. Large cracks occurred along the joint bars, and in the case of the double layer joint bars, the largest cracks occurred along the joint bars in the first layer. So, it is found that the bond stress of the joint bars in the first layer is larger than that of the joint bars in the second layer. Fig.6 shows that the final collapse state is the corn failure of concrete enclosed by the surface steel plates and the partitioning steel web plates after the bond splitting failure occurring along the joint bars.

Investigation of Test Results

The strain distributions of the surface steel plate and the joint bar of 0S-30/5-38 are shown in Fig.7. It is found that the strain distribution is almost linear, so the bond splitting stress can be calculated by dividing the joint bar axial tensile force by the circuit length and anchoring length of the joint bars.

The relationship between the bond splitting strength obtained from this test divided by a square root of concrete compressive strength and the partitioning steel web plate ratio is shown in Fig.8. In this figure, the previous experimental results and calculated results obtained from Eq.(1) are shown. It is found that the test results are larger than the calculated results obtained from Eq.(1), and the tendency of variation in the test results corresponded with that in the calculated results.

It was estimated before testing that if the layer of the joint bars was double, the bond splitting strength of the joint bars in the first layer was smaller than that of the single layer joint bars. The relationship between the reduction factor of the bond splitting strength obtained from this test and the tensile force ratio of the joint bars in the first layer to the all joint bars is shown in Fig.9.

In this figure, the calculated results obtained from Eq.(3) is shown. This figure shows that the estimation is right, and the reduction factor can be evaluated by Eq.(3).

This test enables the bond splitting strength of the joint bars confined by the steel plates to be evaluated.

THE HORIZONTAL LOADING TEST OF SHEAR WALLS

In this chapter, the behavior of the steel plate reinforced concrete shear walls using joint bars confined by the steel plates under horizontal loading is investigated. To confirm the ultimate strength and the restoring force characteristics of the steel plate reinforced concrete shear walls, the horizontal loading test was carried out.

Specimens

Specimens and material characteristics are shown in Table 3, and the shapes of specimens are shown in Fig.10. The number of specimens is four (No.1 - 4). On No.1,2, the joint bar ratio of the web wall is 1.25%. This ratio was determined so that the shear friction strength obtained from ACI-318 was above the design load. On No.3,4, the joint bar ratios of the web and flange walls were determined so that specimens collapsed without slipping at the leg portion, because the failure states of No.1,2 were determined by slipping failure at the leg portion after the joint bars in flange wall yielded.

Loading and Measurement Method

The concept of loading system is shown in Fig.11. The shear span ratio (M/QD; D: whole length of wall) of No.1 is 1.23, that of No.2,3 is 0.70, and that of No.4 is 0.62. No.4 was loaded axial stress (1.47 N/mm^2), and loaded reverse over turn moment, because this specimen was absolutely made to collapse without slipping at the leg portion. Th reverse over turn moment was loaded in proportion to the shear force.

The relative horizontal deflection between the top of the base slab and the bottom of the loading slab, the relative vertical deflection of the flange wall, the relative horizontal deflection between the top of the base slab and the leg of the wall (the slipping deflection), and the strains of each steel plate, joint bars and studs were measured.

Test Results

The test results are shown in table 3. On No.2 and No4, the shear yielding of the web surface steel plates occurred before the maximum load. On No.1 and No.2, after the joint bars in the flange wall yielded, the slipping failure between the wall and the base slab occurred. The maximum load of both specimens was determined by this slipping failure. But the maximum load of No.1 was smaller than that of No.2. So, it is found that the slipping strength is relative to moment loaded at the slipping surface. On No.3, after the joint bars in flange wall yielded, concrete crushed at the leg portion. On No.4, yielding of the joint bars did not occur, and the maximum load was determined by the shear failure of the web wall.

The relationships between shear force and horizontal deflection are shown in Fig.12. Except No.4, each specimen showed good ductility.

Investigation of Test Results

The restoring force characteristics and the ultimate strength of the steel plate reinforced concrete shear walls were investigated. The horizontal deflection is defined as the total of bending deflection, shear deflection and rotation deflection caused by the joint bars slipping out from the base slab. The relationship between moment and curvature of section is evaluated based on the assumption that the section holds flat plane after deflection occurs. This relationship is modeled to a tri-linear curve. The first point is bending crack point, the second point is bending yielding point, and the final point is bending failure point. The relationship between shear force and shear deflection is also modeled to a tri-linear curve. The first point is shear crack point, the second point is shear yielding point of the web surface steel plate, and the final point is shear failure point. In this paper, the shear deflection is investigated in detail. The second and final points are calculated as follows.

The second point (Q_y, γ_y) is shear yielding of the web surface steel plate. Q_y is calculated based on the truss theory and von Mises yield criterion as follows (Eq.(4)-(11)).

$$Q_y = {}_cQ_y + {}_sQ_y \quad , \quad {}_cQ_y = {}_c\tau_w t_w \ell_w \quad , \quad {}_sQ_y = {}_s\tau_s P_w t_w \ell_0 \tag{4}$$

$${}_s\sigma_x{}^2 + {}_s\sigma_y{}^2 - {}_s\sigma_x {}_s\sigma_y + 3{}_s\tau^2 = \sigma_p{}^2 \tag{5}$$

$${}_s\sigma_x = \frac{A + \frac{B}{C}}{{}_xK_sP}\,{}_c\tau \tan\theta \quad , \quad {}_s\sigma_y = \frac{A + BC}{{}_yK_sP}\left(\frac{{}_c\tau}{\tan\theta} - \sigma_0\right) \quad , \quad {}_s\tau = {}_c\tau \frac{G_s}{G_{ns}} \tag{6}$$

$$\left(\frac{1}{{}_xK} - \frac{1}{{}_yK}\right)\cos^4\theta - 2\left(\frac{1}{{}_2K} + \frac{1}{{}_xK}\right)\cos^2\theta + \frac{1}{{}_2K} + \frac{1}{{}_xK} = 0 \tag{7}$$

$$G_{trs} = \frac{1}{\dfrac{1}{{}_2K\cos^2\theta\sin^2\theta} + \dfrac{\tan^2\theta}{{}_xK} + \dfrac{1}{{}_yK\tan^2\theta}} \tag{8}$$

$${}_2K = 0.7E_c \quad , \quad {}_xK = A + \frac{B}{C} + {}_xK_w + K_f \quad , \quad {}_yK = A + BC + {}_yK_w \tag{9}$$

$$A = \frac{{}_sP_s E_s}{1 - {}_s\nu^2} \quad , \quad B = A_s\nu \quad , \quad C = \frac{B - (A + {}_yK_w)\tan^2\theta}{B\tan^2\theta - A - {}_xK_w - K_f} \tag{10}$$

$${}_xK_w = {}_wP_{x\,w}E_{xs} \quad , \quad {}_yK_w = {}_wP_{y\,w}E_{ys} \quad , \quad K_f = \frac{aE_cI_e\ell}{t_wh_0^{\,4}} \qquad a = 22.5 : \text{joint bars system} , a = 360 : \text{others} \tag{11}$$

${}_sP$: surface steel plate ratio
${}_wP_x$, ${}_wP_y$: horizontal and vertical partitioning steel web plate ratio (if they are set)
${}_sE_s$, ${}_wE_{xs}$, ${}_wE_{ys}$: young's modulus of each steel plate (N/mm²)
G_s: shear stiffness of surface steel plate (N/mm²)
${}_s\nu$: poisson's ratio of surface steel plate , σ_p: yielding strength of the surface steel plate (N/mm²)
σ_B : concrete compressive strength (N/mm²)
E_c: young's modulus of concrete (N/mm²)
I_e: geometrical moment of inertia of flange wall (effective width of the flange wall = $2\,{}_ft_w + {}_wt_w$) (mm⁴)
${}_wt_w$: wall thickness of the web wall (mm)
${}_ft_w$: wall thickness of the flange wall (mm)
ℓ: the distance between the center points of both flange walls (mm)
ℓ_0: the distance between the inner sides of both flange walls (mm)
ℓ_w: the distance between the outer sides of both flange walls (mm)
h_0: the distance between the top of the base slab and the bottom of the loading slab
σ_0:axial stress (N/mm²)

The truss stiffness of concrete after cracking is obtained from Eq.(8). The angle of the principal compressive direction is obtained from Eq.(7). In Eq.(11), K_f is the stiffness of horizontal spring modeled the restraint effect by the flange walls, the loading slab and the base slab.

The second point shear distortion (γ_y) is obtained from Eq.(12).

$$\gamma_y = \frac{{}_c\tau}{G_{trs}} - \frac{\sigma_0}{{}_yK\tan\theta} \tag{12}$$

The final point (Q_u, γ_u) is shear failure. Q_u is obtained from Eq.(13), and γ_u is obtained from Eq.(14). In Eq.(13), h is the distance between the top of the base slab and the loading point.

$$Q_u = Q_y + \frac{1}{2}\tan\theta_{u\,w}t_w\ell_w\left(\lambda\sigma_B - \frac{2_c\tau}{\sin 2\theta_u}\right) \quad , \quad \tan\theta_u = \sqrt{\left(\frac{h}{\ell_w}\right)^2 + 1} - \frac{h}{\ell_w} \quad , \quad \lambda = 0.7 - \sigma_B/200 \tag{13}$$

$$\gamma_u = \frac{6}{1000} \tag{14}$$

The relationship between the ultimate bending moment and the slipping strength is evaluated by Eq.(15). The slipping strength without bending moment is obtained from Eq.(16).

$$\left(\frac{M_u}{M_{mu}}\right)^2 + \left(\frac{Q_u}{Q_{sl}}\right)^2 = 1 \tag{15}$$

$$Q_{sl} = A_{all}[14.1 + 0.8(p_{j\,j}\sigma_y + \sigma_0)] \tag{16}$$

M_{mu}: ultimate bending moment without slipping
Q_{sl}: slipping strength without bending moment
A_{alll}: entire horizontal area
p_j: area ratio of all joint bars to entire horizontal area, ${}_j\sigma_y$:yielding strength of joint bars

The relationships between the shear force and the shear distortion which are obtained from calculation are shown in Fig.13 as compared with the experimental results which include the previous experimental results shown in reference 3). The calculated results corresponded well with the experimental results.

CONCLUSION

A new joint system between the steel reinforced concrete walls and the reinforced concrete base slab with the joint bars and the partitioning steel web plates was developed. To investigate the bond splitting strength between the joint bars and concrete and the structural characteristics of the steel reinforced concrete shear walls with the new joint system under horizontal loading, the axial pulling out test and the horizontal loading test were carried out.

By investigating these test results, the evaluation methods for the structural characteristics of the steel plate reinforced concrete shear walls based on the previous methods were developed. The calculated results from these evaluation methods corresponded well with the test results. Therefore, these studies enable the steel plate reinforced concrete shear walls with joint bars to be applied to actual structures .

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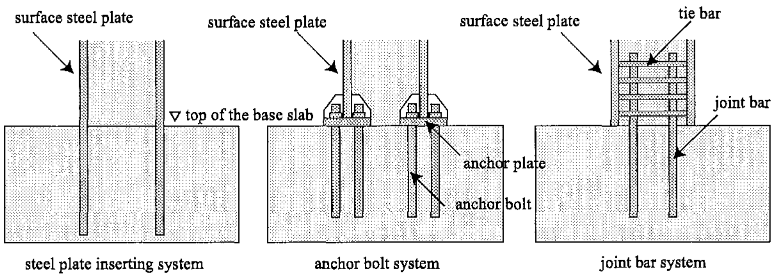

Fig.1 Three Joint System Types

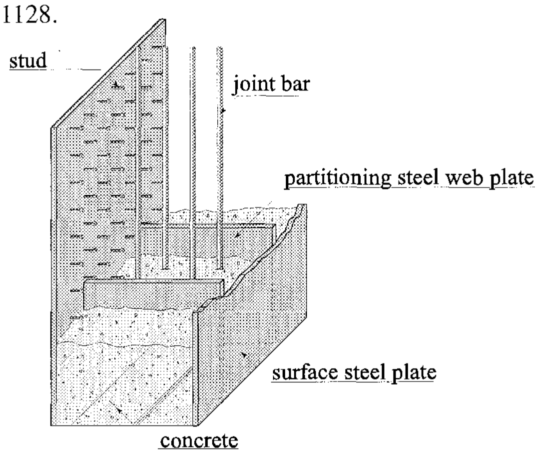

Fig.2 Concept of new Joint Bar System

Table 1 Specimens and Material Characteristics

specimens		section (B×D)	ℓp	joint bar layer	joint bar number	partitioning steel web plate pw (%)	partitioning steel web plate ℓw (mm)	σB (N/mm²)	surface steel ts (mm)	surface steel σy (N/mm²)	web steel tw (mm)	web steel σy (N/mm²)	joint bar σy (N/mm²)
part 1	0S-30/9-70	800 × 700	30d$_b$	1	6	0.91	700	34.9	5.90	380	5.90	380	531
	0S-30/5-35					0.45	350	34.8	5.90	380	5.90	380	
	0S-40/3-35		40d$_b$			0.32	350	34.7	5.60	390	5.60	390	
part 2	0S-30/5-38		30d$_b$			0.52	380	33.5	6.27	384	6.27	384	529
	0S-30/5-51					0.49	513		6.27	384	4.33	430	
	0S-30/5-72					0.49	722		6.18	387	3.08	291	
	1S-30/7-38	800 × 900		2	8	0.73	380	37.7	8.80	381	8.80	381	
	2S-30/8-38				10	0.80	380		9.64	338	9.64	338	
	2S-40/11-72		40d$_b$		10	1.14	722		9.64	338	9.64	338	
	3S-30/10-38		30d$_b$		12	0.97	380		11.59	386	11.59	386	

σB : concrete compressive strength, ts, tw : thickness of plate, σy : yielding strength, db : diameter of joint bar

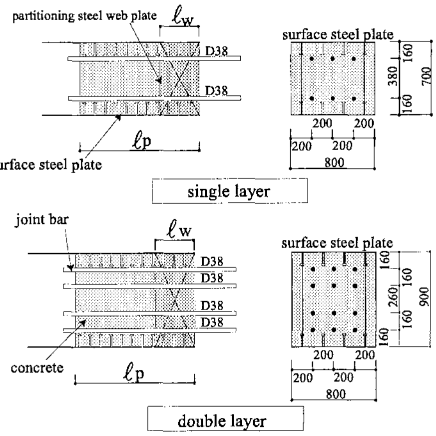

Fig.3 Shapes of Specimens

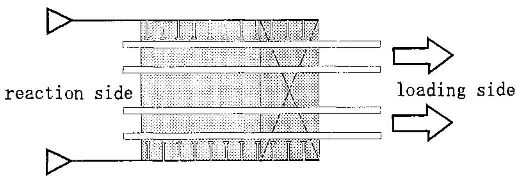

Fig.4 Concept of Loading System

Table 2 Test Resuls

specimens	crack load (kN)	surface plate yielding load (kN) exp.	surface plate yielding load (kN) cal.	joint bar yielding load (kN) exp.	joint bar yielding load (kN) cal.	maximum load (kN) exp.	maximum load (kN) cal. bond strength first layer	maximum load (kN) cal. bond strength second layer	maximum load (kN) cal. bond strength (first layer)	maximum load (kN) cal. stud** yieldin-g stength	failure mode
0S-30/9-70	1480	3500	3590	-	3640	3530	3790	-	3790	2270	bond splitting
0S-30/5-35	1470	-	3590	-	3640	3330	3070	-	3070	2880	bond splitting
0S-40/3-35	1480	3520	3500	3650	3640	(3650)	3800	-	3800	4450	joint bar yielding
0S-30/5-38	1372	-	3870	-	3620	3220	3120	-	3120	4380	bond splitting
0S-30/5-51	1372	-	3870	-	3620	3050	3070	-	3070	3920	bond splitting
0S-30/5-72	1470	-	3840	-	3620	3140	3070	-	3070	3460	bond splitting
1S-30/7-38	1960	-	5370	-	4820	3920	4900	4330	3680	6620	bond splitting
2S-30/8-38	1764	-	5190	-	6040	3830	6320	5650	3790	6620	bond splitting
2S-40/11-72	2160	5390	5190	-	6040	(5700)	9690	9010	5810	8020	steel plate yielding
3S-30/10-38	2160	-	7160	-	7240	4510	8150	6390	4080	6620	bond splitting

**stud yielding strength : $0.6n(0.5 \cdot {}_{sc}a \sqrt{\sigma_B E_c})$

n : number of studs

${}_{sc}a$: section area of stud (mm^2),

Ec : young's modulus of concrete (N/mm^2)

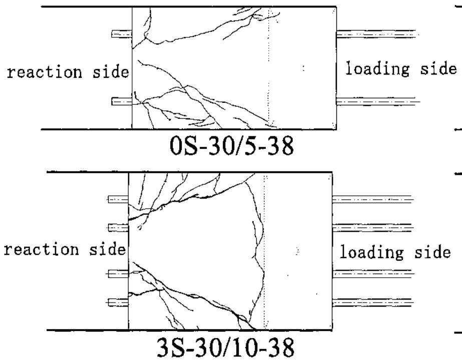

Fig.5 Failure State (Concrete Surface)

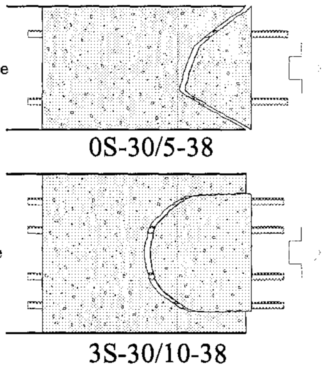

Fig.6 Failure State (Inner Concrete)

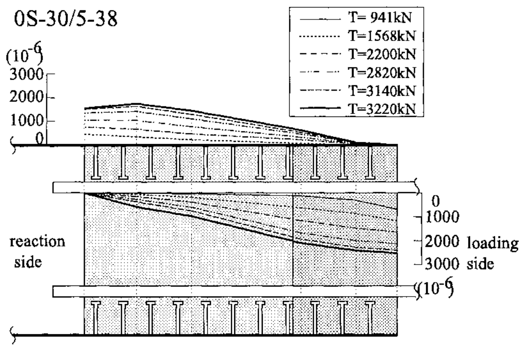

Fig.7 Strain Distributions of Joint Bar and Surface Steel Plate

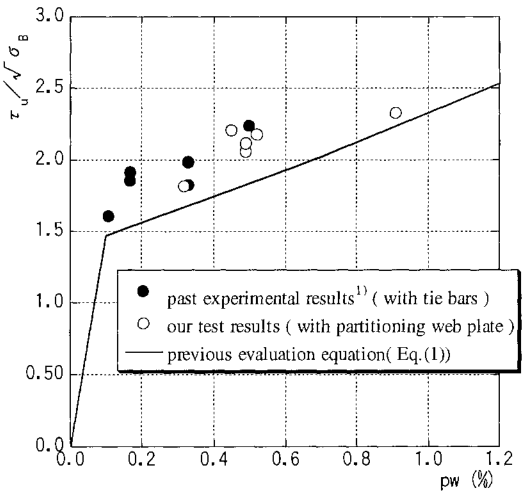

Fig.8 Evaluation for Bond Splitting Strength

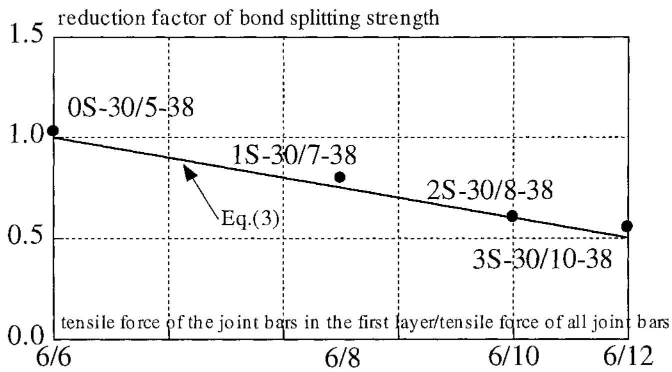

Fig.9 Evaluation for Reduction Factor of Bond Splitting Strength (Double Layer Joint Bars)

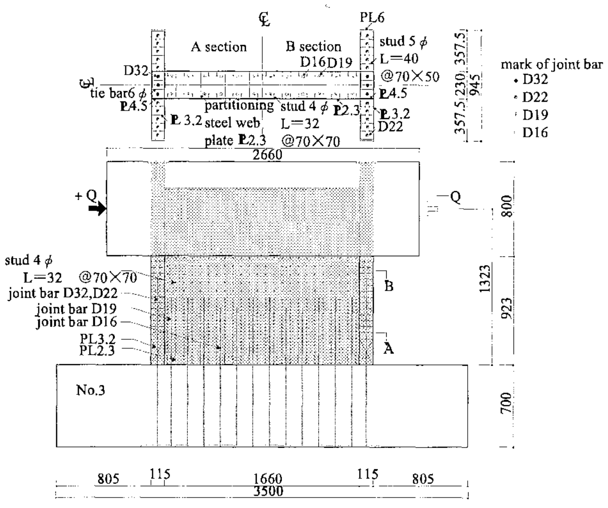

Fig.10 Shape of Specimen

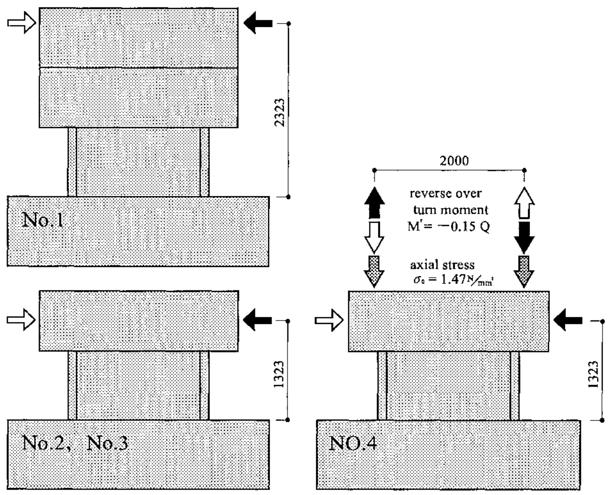

Fig.11 Concept of Loading Systems

Table 3 Specimens, Material Characteristics and Test Results

	joint bar		material characteristics (N/mm²)			test results (kN)					failure mode
	flange	web	concrete σ_B	joint bar σ_y	steel plate σ_y	bending crack load	shear crack load	shear yielding load	bending yielding load	maximum load	
NO.1	13−D22	24−D16	41.1	D16 : 521	PL2.3 : 375	+320 −277	+672	− −	+2480 −2470	+3390	bending yielding→ slipping failure
No.2			44.6	D22 : 500	PL3.2 : 373	+634 −614	+821	+3920 −3830	+4020 −3930	+4750	shear yielding →bending yielding →slipping failure
No.3	10−D22	8−D16 16−D19	31.9	D16 : 509 D19 : 515	PL2.3 : 416	+496 −494	+1282 −1313	− −	+4510 −4310	+4670	bending yielding→ shear failure
No.4	3−D32	8−D19 16−D22	36.1	D22 : 521 D32 : 514	PL3.2 : 270	+977 −984	+1446 −1767	+4700 −4740	− −	+4950	shear yielding →shear failure

σ_B : compressive strength, σ_y : yielding strength

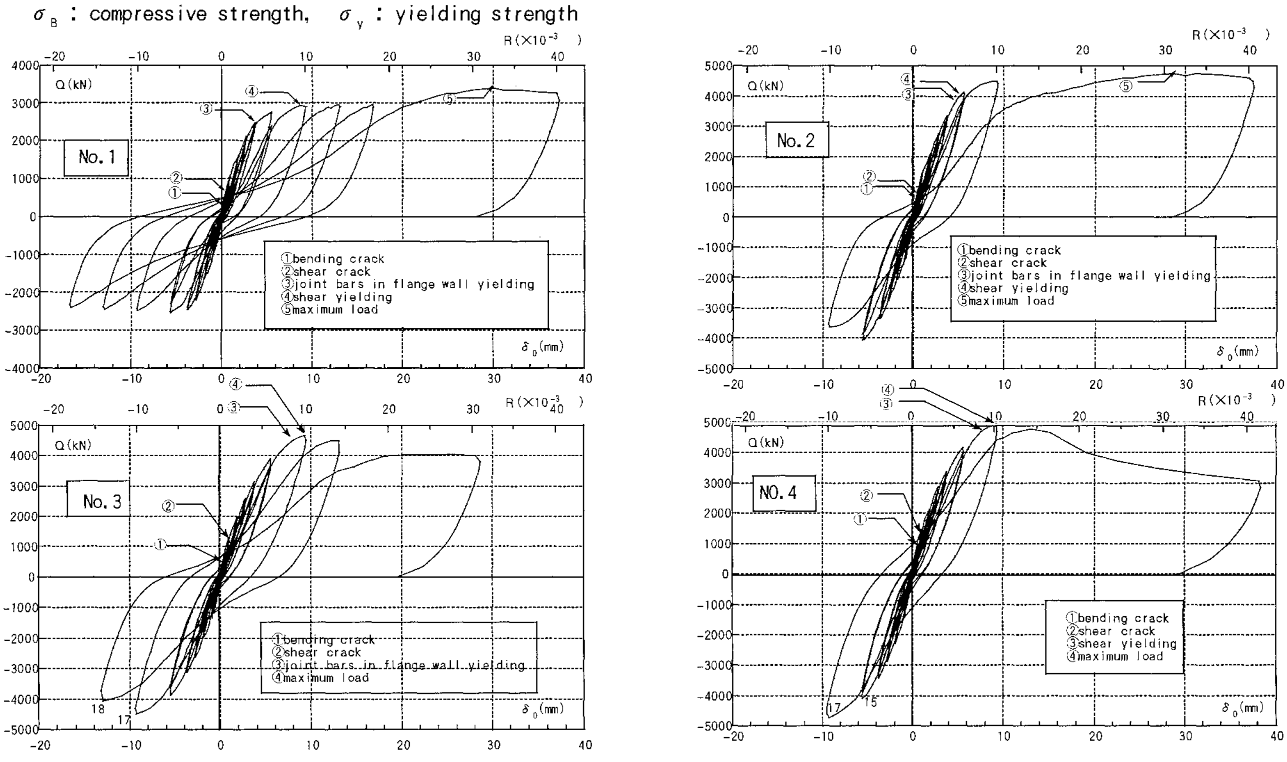

Fig.12 Relashionships of Shear Force and Horizontal Deflection

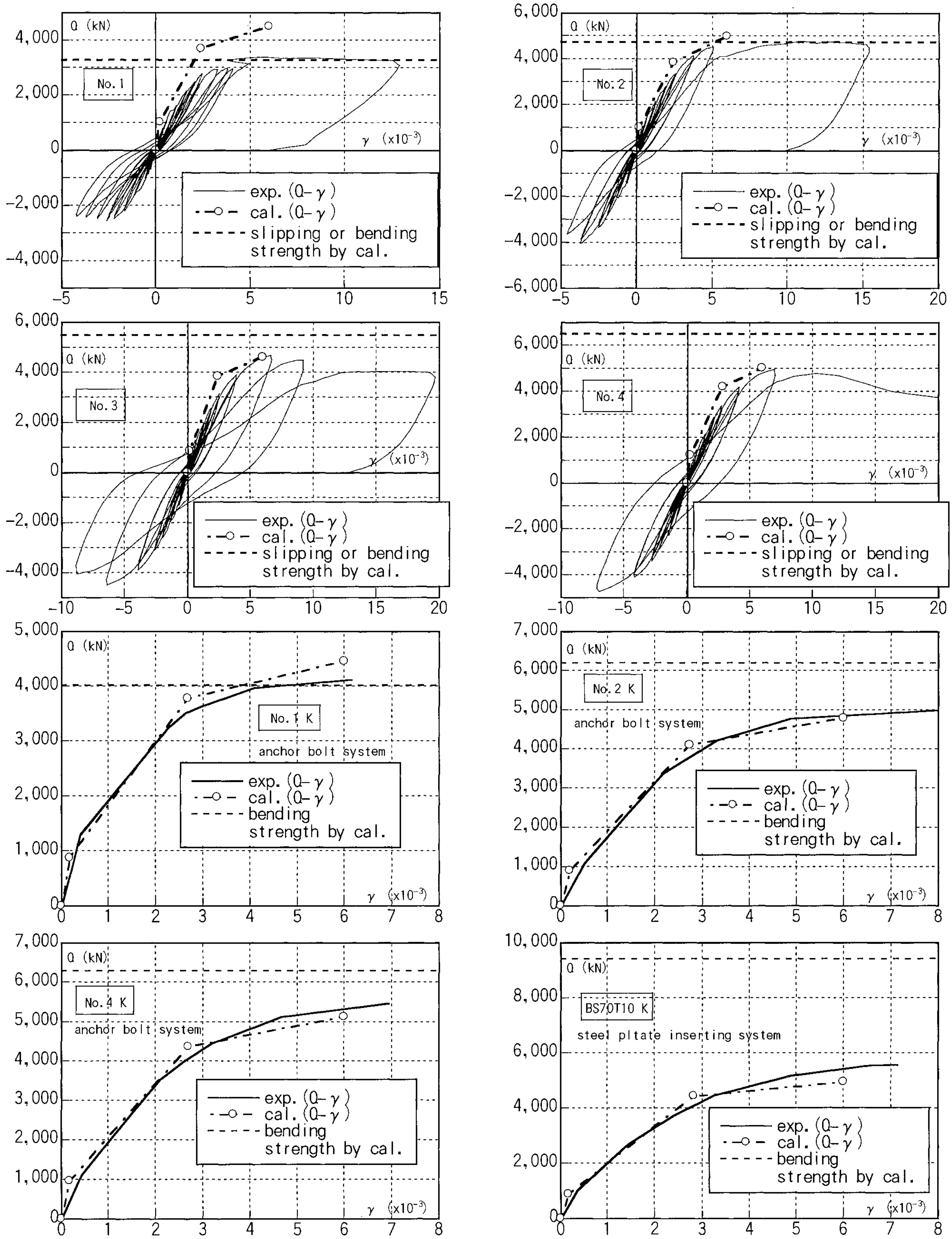

Fig.13 Correlation of Test and Calculated Relationships between Shear Force and Shear Distortion