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EXPERIMENT ON SLIP STRENGTH OF HORIZONTAL JOINT OF PRECAST CONCRETE MULTI-STORY SHEAR WALLS

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ABSTRACT

A multi-story shear wall has often discrete joints without cotter reinforcements at the vertical joints between the wall panels and the surrounding cast-in-place columns in Japan. In order to investigate the restrictive effect of joints on earthquake resistant behaviors of the precast concrete multi-story shear walls, a shear failure type experiment is performed on seven multi-story precast concrete walls having the various combinations of the presence or absence of cotter and connecting bar ratio at the joint. Based on the results of this experiment, the resistant mechanism of a precast concrete multi-story shear wall and a design formula considering slip strength of horizontal joints are proposed. Comparing the design formula with the results of the experiments the resistant mechanism is shown to be appropriate for the practical design.

KEYWORDS

joint; precast concrete; resistant mechanism; shear failure; sliding displacement; slip strength; wall panel

INTRODUCTION

In Japan, precast shear walls have become popular because they may be constructed more quickly than cast-in-place reinforced concrete shear walls and, moreover, owing to lack of construction workers, new construction methods have been developed. The precast shear wall in which wall panels with beam are set into cast-in-place columns as considered in this paper has been used. A precast shear wall of this type has discrete joints at the vertical joints between the wall panels and the cast-in-place columns and at the horizontal joints between the adjoining wall panels. It is expected that the resistant mechanism of precast shear walls with such discrete joints may be considerably different from that of cast-in-place reinforced concrete shear walls. In the case of precast shear wall, it is very important that the resistant mechanism of each element is not evaluated independently. For example, the sliding displacement in horizontal joint is related to not only the frictional resistance due to inclined compressive struts of cracked wall panels, but also the shear resistance of compressive cast-in-place columns. The object of this paper is to make clear the resistant mechanism resulted from the slip failure and to propose the practical design method of horizontal joints.

SUMMARY OF EXPERIMENT

The structural specifications of the experiment are given in Table 1. Fig. 1 shows the shape and bar arrangement drawing of specimen 1, which is considered to be the standard one.

Table 1. Structural specification of experiment.

	Vertical joint				Horizontal joint	
	Cotter	Cotter bar	Intermediate beam main bar	Horizontal connecting bar	Cotter	Dowel bar
	Area ratio (%)	Area (cm^2)	Area (cm^2)	Bar ratio (%)	Area ratio (%)	Bar ratio (%)
No.1	0.5	1.00	2.07	1.66	0.00	0.44
No.2	0.5	1.00	2.07	1.66	0.00	0.00
No.3	0.5	0.00	2.71	1.47	0.00	0.44
No.4	0.5	0.00	2.71	1.47	0.00	0.00
No.5	0.5	0.00	2.71	1.47	0.49	0.44
No.6	0.5	0.00	2.71	1.47	0.49	0.00
No.7	0.5	1.00	1.28	1.24	0.00	0.44

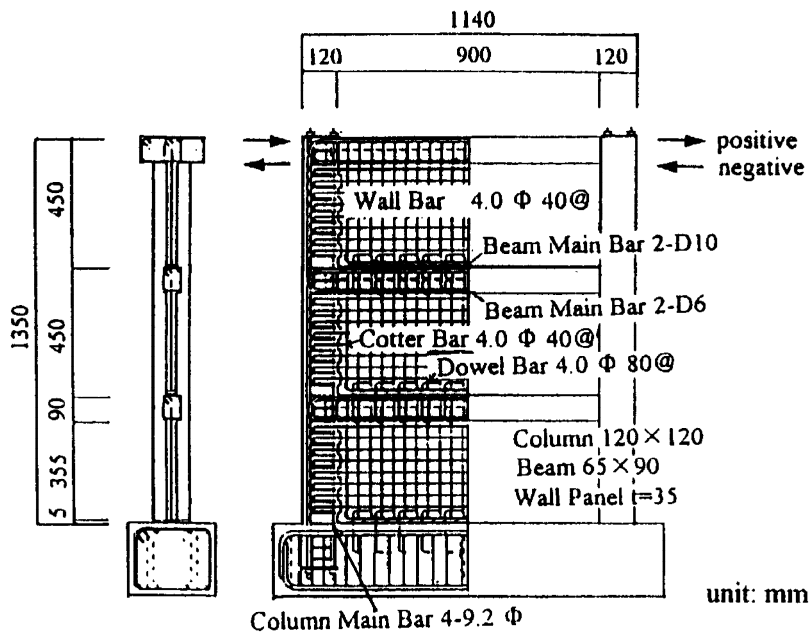


Fig.1. Shape and bar arrangement.

The specimen, a three-story, single-span precast shear wall, is about 1/8 as large as an actual one. The wall of each story forms a T-shaped section composed of intermediate beam and the wall panel. The top intermediate beam is used to apply as a loading beam. The wall panels with the intermediate beam on each story are connected to each other by horizontal joints, and to the cast-in-place columns by vertical joints. There are seven specimens. The parameters are the quantity of horizontal connecting bars in vertical joints, and the presence or absence of dowel bars and cotters in horizontal joints. The area of horizontal connecting bars in the vertical joints in Table 1 is the total area of cotter bars and intermediate beam main bars. The specimens aiming the slip failure in the horizontal joints are designed to be failed in not flexure but shear failure. Loading is performed by applying constant axial load using a hydraulic jack and alternating horizontal load controlled by rotation angle R ($=1,2,4,6,\cdots 25\times 10^{-3}$ rad). Horizontal load is applied by equivalent compressive force and tensile force to the left and right of the loading beam simultaneously. Displacements are measured by high sensitive displacement meters, such as horizontal displacement at the top of columns and the relative sliding and opening displacements in the vertical and horizontal joints. Strains of bars are measured by strain gauges.

EXPERIMENTAL RESULTS

Failure Process

Figure. 2 shows the final failure condition for the four specimens having a remarkable failure pattern. Specimen 1,4,5,7 show pattern of the shear failure of wall as a whole, of the shear failure of column, of the peeling off of wall panel, of the opening in vertical joint, respectively.

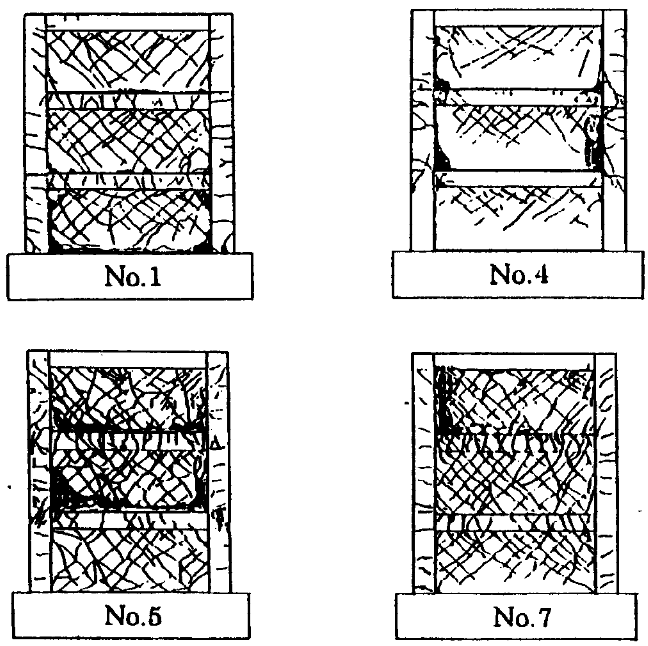


Fig. 2. Final failure condition.

The failure processes of specimen 1 as the pattern of prototype are; 1) cracks are initially formed through the horizontal joints at horizontal load Q=2.5 tf, rotational angle R=0.12×10^{-3} rad and through vertical joint at Q=4.3 tf, R=0.33×10^{-3} rad, 2) bending cracks are then formed in the side columns at Q=4.5 tf, R=0.34×10^{-3} rad, 3) shear cracks appear in the wall panel at Q=5.6 tf, R=0.56×10^{-3} rad, 4) tensile cracks occur in the intermediate beams at Q=15.3 tf, R=6.0×10^{-3} rad, 5) maximum load is reached at Q=17.6 tf, R=8.1×10^{-3} rad, 6) peeling off of wall panel are found at Q=17.2 tf, R=9.8×10^{-3} rad, 7) ultimate load is reached at Q=10.3 tf, R=25.0×10^{-3} rad and final failure mode is said to be the composite failure of shear wall composed of peeling off of the wall panels and sliding in the horizontal joint.

Failure characteristics of specimens are summarized;
1)cracks of wall panels of the specimen with cotters in the horizontal joints in spite of the presence or absence of dowel bars in the horizontal joints are density.
2)cracks of wall panels of the specimen without both of dowel bars and cotters in the horizontal joints and cotter bars in the vertical joints do not extend and failure mode of this specimen is shear failure of columns.
3)cracks conditions of wall panels of the specimen with dowel bars in the horizontal joints do not change by the presence or absence of cotter bars in the vertical joints.
4)failure mode of the specimen without adequate quantity of the intermediate beam main bars is opening

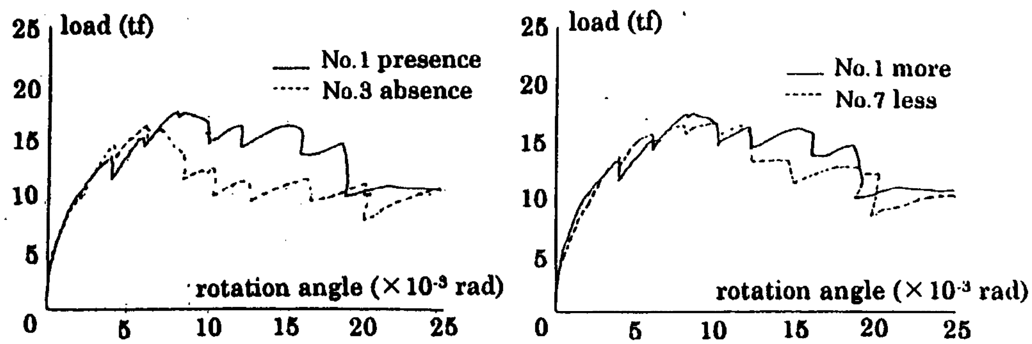
failure of the vertical joints.

Strength

Table 2 shows the experimental maximum load Q_{max} and the calculated shear load Q_S as cast-in-place shear walls. In this table, the maximum load is taken during positive loading. In this experiment, all specimens are designed to be failed in not flexure but shear failure. So the calculated flexural load is omitted in Table 2. The highest ratio of the maximum load to the calculated shear load is 0.80 for specimen 5 of which four sides of the wall panels are restricted as cast-in-place shear walls. The highest maximum load is 18.2 tf for specimen 5 and lowest one, 14.0 tf for specimen 5. While specimen 5 has both of dowel bars and cotters in the horizontal joints, specimen 4 has not dowel bars and cotters in the horizontal joints. Therefore, the restriction of the horizontal joints is said to play important role to the maximum load. Comparing the maximum load of specimen 1 to that of specimen 7 of which quantity of the intermediate beam main bars is less than that of specimen 1, the difference between them is found to be 1 t. So the intermediate beam bars has influence on the maximum load.

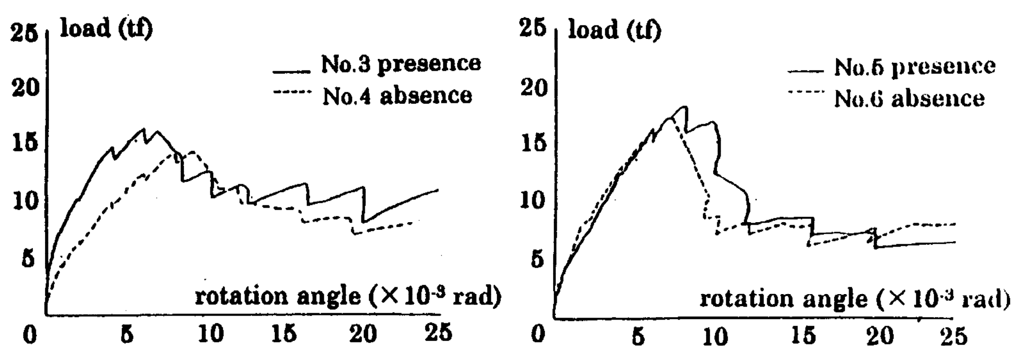
Relationship between Rotation Angle and Load

The Comparisons of the envelope curves of the relationship between the rotation angle and the load during the positive loading are shown in Fig. 3 for the presence or absence of cotter bars and dowel bars and more or less quantity of the intermediate beam main bars.



(a) cotter bar in vertical joint

(b) quantity of intermediate beam main bar



(c) dowel bar in horizontal joint, without cotter bar

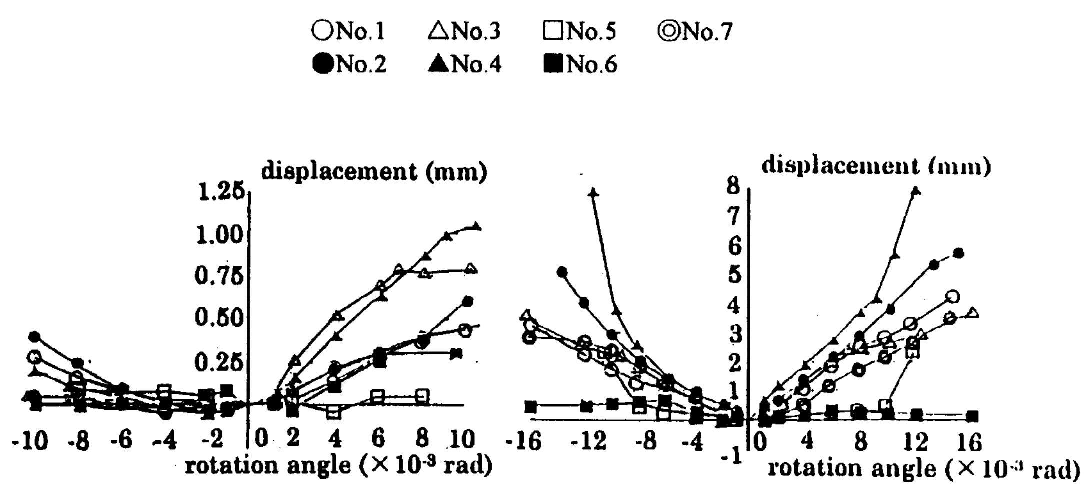
(d) dowel bar in horizontal joint, with cotter bar

Fig. 3. Relationship between rotation angle and load.

Comparing the envelope curve of specimen 1 with cotter bars to that of specimen 3 without cotter bars in Fig. 3(a), the rigidity until maximum load of them have no difference but the load after maximum load of specimen 3 without cotter bars fall rapidly. The comparison for the more or less quantity of intermediate beam main bars is shown in Fig. 3(b). Fig. 3(b) shows that there is almost no change in both of rigidity and load except for the fall in load after maximum load of specimen 1 with less quantity of intermediate beam main bars. From the Fig. 3(c) for the comparison of the presence or absence of dowel bars in specimens without cotters in the horizontal joints, specimen 4 without dowel bars has the lower initial rigidity and more fall in load after maximum load than those of specimen 3 with the dowel bars. The comparison of the presence or absence of the dowel bars in specimens with cotters in the horizontal joints is shown in Fig.3 (d). Specimen 6 without dowel bar shows sudden fall in the load after the maximum load accompanying the collapse of cotters, the specimen 5 with dowel bars, gentle fall in the load.

Sliding Displacement in Joint

Figure. 4(a) shows the relationships between the sliding displacement and the rotation angle at the second-story vertical joint. Specimen 1 and 2 with cotters in the vertical joints and restrained along the both sides of the wall panel show almost identical behaviors during the positive and negative loading. On the other hand, in the case of specimen 3 and 4 which have no cotter bars, the sliding displacement during the positive loading (tensile side columns) are greater than they are during the negative loading (compressive side column). It should be added that the behavior of the opening displacement at vertical joints is almost identical to the behavior of the sliding displacement. Fig. 4(b) shows the relationship between the sliding displacement and the rotation angle at the second-story horizontal joint. The opening displacement at the horizontal joint is almost nothing, so they are omitted. Specimen 1 and 3 with dowel bars but no cotters in the horizontal joints show smaller sliding displacements than those of specimen 2 and 4 without dowel bars and cotters. Specimen 5 with cotter and dowel bars and specimen 6 with cotter, but without dowel bars in the horizontal joints scarcely shows sliding displacement up to maximum load. It is obvious that the presence or absence of dowel bars in the horizontal joints significantly influence the behavior of sliding in the horizontal joints.



(a)for vertical joint (b) for horizontal joint

Fig. 4. Relationship between sliding displacement and rotation angle.

Strain of Intermediate Beam Main Bar

Figure. 5 shows the relationship between the strain of upper main bars at right end of the intermediate beam and rotation angle. From the comparison among specimen 1,2,3 and 4 which have equal quantity of the intermediate beam main bars, specimen 1 and 3 with dowel bars show smaller strain of the intermediate beam main bars than that of specimen 2 and 4 without dowel bars. The difference of the strain of the intermediate beam main bars between specimen 5 with both of cotters and dowel bars in the horizontal joints and specimen 6 with cotters, but without dowel bars is scarce. So it is concluded that the presence or absence of dowel bars do not influence the strain of the intermediate beam main bars up to maximum load if cotters are provided.

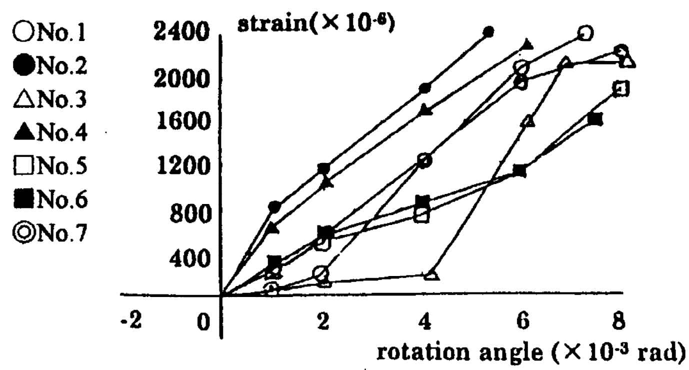


Fig. 5. Relationship between strain of upper main bar of intermediate beam and rotation angle.

SLIP STRENGTH OF HORIZONTAL JOINT

Resistant Mechanism by Compressive Strut

Based on results of this experiment, the author assumed the following assumptions for the resistant mechanism at the sliding failure of the horizontal joints.

1)The remainder horizontal shear force subtracted the shear force Q_{TS} of the tensile column from the shear force Q is transferred to the compressive diagonal wide strut which is supported at the horizontal and the vertical joint at the wall panel corner of each story as shown in Fig. 6.

2)The horizontal component N_{DBH} and the vertical component N_{DBV} of the compressive force N_{DB} of the strut supported at the horizontal joint become the shear force and the normal force respectively. The horizontal component N_{DCH} and the vertical component N_{DCV} of the compressive force N_{DC} of the strut supported at the vertical joint become the normal force and the shear force respectively.

3)The horizontal component N_{DCH} of the compressive force of the strut supported at the vertical joint is balanced to the restoring strength T_B of the intermediate beam main bars.

4)The horizontal component N_{DH} of the compressive force of the strut is balanced to the sum of the slip strength Q_{WH} of the horizontal joint of the wall panel and the shear strength Q_{CS} of the compressive column.

5)The vertical component N_{DV} of the compressive force of the strut is balanced to the slip strength Q_{WV} of the vertical joint integrated with the intermediate beam.

The slip strengths of the horizontal and the vertical joint of the wall panel are calculated by the formula (Mochizuki et al., 1981) proposed by the author and the shear strength of the compressive column, by the formula (B.C.J. 1981) given in Guide for Structural Calculation. As shown in equation (1) and (3), shear force Q can't be computed explicitly, because the terms in the right of equation (1) and (3) are the function of the unknown shear force Q. The method used to compute the bearing strength involves assuming the shear force and performing a serial approximation of the bearing strength.

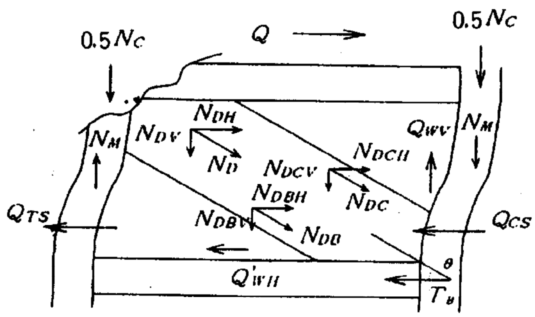


Fig. 6. Resistant mechanism.

$$Q=N_{DH}+Q_{TS} \tag{1}$$

$$N_{DCH} \leqq T_B \tag{2}$$

$$N_{DH} \leqq Q_{WH}+Q_{CS} \tag{3}$$

$$N_{DV} \leqq Q_{WV} \tag{4}$$

Q; shear force
N_D; compressive force of strut
N_{DH} (N_{DV}); horizontal (vertical) component of compressive force of strut
N_{DBH}(N_{DBV}); horizontal (vertical) component of compressive force N_{DB} of strut supported in horizontal joint
N_{DCH}(N_{DCV}); horizontal (vertical) component of compressive force N_{DC} of strut supported in vertical joint
Q_{WH} (Q_{WV}); slip strength in horizontal (vertical) joint of wall panel
Q_{CS} (Q_{TS}); shear strength of compressive column (tensile column)
T_B; tensile strength of intermediate beam main bar
N_M; additional longitudinal force of side column due to horizontal load
N_C; vertical load (compression is positive)

Computed Results

The comparison of the maximum load Q_{max} of the test and the shear load Q calculated by the proposed method are shown for the verification of the proposed mechanism in Table 2.

Table 2. Comparison of maximum load with calculated value.

	Experimental value	Calculated value				Failure mode
	Maximum load Q_{max} (tf)	Slip load Q (tf)	Q/Q_{max}	T_B/N_{DCH}	Shear load Q_S (tf)	
No.1	17.6	[redacted]	1.10	1.21	22.8	A_1
No.2	16.0	[redacted]	0.92	1.18	23.7	A_1
No.3	16.2	[redacted]	1.19	1.64	22.8	A_1
No.4	14.0	[redacted]	1.04	1.55	24.9	A_2
No.5	18.2	25.3	1.39	1.46	22.7	A_1
No.6	17.2	[redacted]	1.24	1.51	22.7	A_1
No.7	16.7	[redacted]	1.11	0.78	21.5	A_3

The failure modes are classified in the peeling off failure of wall panel A_1, the shear failure of the compressive column A_2 and the separating failure of the compressive column A_3 by the additional failure accompanying sliding in the horizontal joint. The bearing load of the specimen is determined by the smaller value between the slip load of the horizontal joint Q and the shear load Q_s of the shear wall as a whole. The failure of the specimens which the tensile strength T_B of the intermediate beam main bars is less than the horizontal component N_{DCH} of the compressive force of the strut supported at the vertical joint is thought to be the separating failure of the compressive column. The shaded values in the Table 2 show the bearing load specified from the calculation. The failure mode specified by the calculated values except for specimen 5 correspond to the failure mode in the test. The computed values except for specimen 6 show the good agreement with the maximum load. Both specimen 5 and 6 have cotters in the horizontal joint of which the scale are too small to resist shear force. So the divergence between the computed results and the experimental results of specimen 5 and 6 are caused by the realistic cotters in the horizontal joints. The difference of the ratio of the computed values to the experimental ones are less than 19 percent except for specimen 5 and 6. The proposed evaluation method of slip load of the horizontal joint is appropriate and is concluded to be used as the practical design method.

CONCLUSIONS

From above results, following conclusions are obtained.

1)The maximum load of precast concrete shear walls are influenced by the combination of restrained conditions of the horizontal joints and the vertical joints.

2)The ultimate load of precast concrete shear walls are influenced by the restrained condition containing dowel bars in the horizontal joints.

3)The rigidity of precast concrete shear walls up to the maximum load is not influenced by the presence or the absence of dowel bars for the specimens with cotters in the horizontal joints.

4)While the specimens with cotter bars in the horizontal joints shows gentle fall in load after maximum load, the specimens without cotter bars, sudden fall in load but restore the tendency falling in load afterwards.

5)The slip load of the horizontal joint of precast concrete shear walls with cotters in the vertical joint but without cotters in the horizontal joint is evaluated by the resistant mechanism of compressive strut proposed in this paper.

REFERENCES

B.C.J. (1981). *Guide for Structural Calculation*. pp. 110-111

Mochizuki,S.,E.Makitani and T.Nagasaka (1911) Ultimate shear strength of vertical connections between precast concrete wall panels considering dowel action and restraint compression. *Transaction of A.I.J.*,Vol. 424, pp.11-22.