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Behavior of the connection of R/C column with steel beams stiffened by reinforced concrete at beam ends

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ABSTRACT: Tests of five interior and two exterior beam-to-column connections which consist of the reinforced concrete columns and the steel beams were carried out putting an emphasis upon investigation of affects with different detail of steel beams and shear stress levels in the connections. From the discussion of the test results, it could be pointed out that the connections of such composite structures obtained more ductile deformations than that of reinforced concrete structures. Although the ultimate shear stress of the connections exceeded to 0.3 times of compressive strength of concrete which were adopted in the Japanese regulation regarding the connection of reinforced concrete structures, strength decay was not observed.

1 INTRODUCTION

Recently in Japan, many researches and the experimental works have been completed on the composite structures which consist of reinforced concrete columns and steel beams. It was regarded that the composite structure has two big advantages in the design and in the construction work. At the first such structure makes it possible to design the long span using the steel beam and acquire high horizontal stiffness of the structure with reinforced concrete columns. At the second it is also possible to save labor and shorten duration of the construction works due to not casting concrete in the beam. The detail of composite structure is much complicate particularly in the connection. Many details were proposed by the construction companies and some of them were applied to actual practice. In the representative detail of the proposed composite structure, the connection panel concrete is covered around by steel plates, and the shear strength of that is calculated based on the strength requirements of the Standard for the Steel Reinforced Concrete Structures edited by Architecutural Institute of Japan (A.I.J.). A technical committee on steel-concrete mixed structures was organized by the Japan Concrete Institute (J.C.I.) constituted of members from universities and companies in the field of structural engineering for buildings and public works. The committee provided for the technical report and the proceedings on the symposium of composite structures 1991.

In this paper, we propose such composite structures as consist of reinforced concrete columns and steel beams, whose ends are stiffened by the reinforced concrete beam

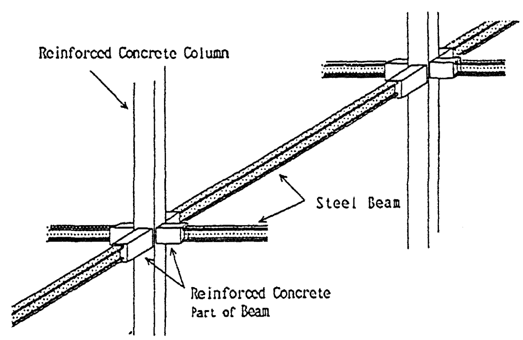

Fig.1 Composit Structure

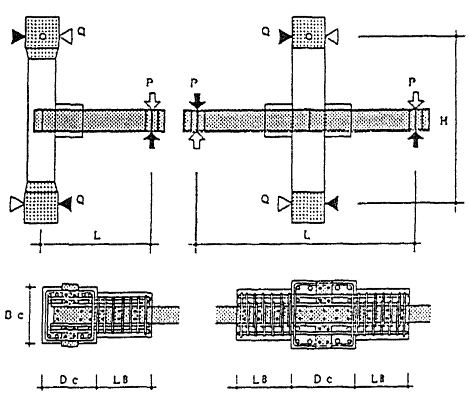

Fig.2 Dimensions of Specimen

(called R/C beam) to develop the yield zone onto the baked steel beam far from the connection as shown in Fig.1. We also present

Table 1 Components of Specimens

SPECIMEN	C1	C3	C4, C5, C6	T1, T2
Span L (mm)	4000	4125	2750	2063
Height H (mm)	3000	3000	2000	3000
Column Section (mm) Reinforcements Hoops	540 x 540 12-D22 2-D10 @75	600 x 600 12-D22 2-D10 @75	330 x 330 12-D22 12-D25 2-D10 @50 2-D10 @50	500 x 500 12-D25 2-D10 @60
Beam Section (mm) Reinforcements Stirupps Steel	360 x 540 4-D22 4-D22 2-D10 @ 175 BH-346x174x6x9	440 x 600 4-D19 4-D19 4-D10 @70 BH-400x150x6x14	270 x 400 2-D19 2-D16 4-D16 4-D22 2-D16 4-D16 2-D10 @80 4-D10 @50 BH-250x100x4.5x9 BH~250x100x6x16	400 x 600 4-D19 4-D22 4-D19 4-D22 2-D10 @70 @55 BH-400x150x6x14
Connection Hoops	2-D10 @75 6-sets	2-D10 @75 5-sets	2-D10 @100 3-sets	2-D10 @75 5-sets

Table 2 Material Properties

unit (kg/cm^2)

		C1	C3	C4	C5, C6	T1, T2
D22	Yield Strength Maximun Strength Young's Modulus	3760 5390 1600×10^3	4210 5900 1760×10^3	3840 5950 1850×10^3		4080 5810 2000×10^3
D19	Yield Strength Maximun Strength Young's Modulus	4220 5660 1740×10^3	3920 5630 1710×10^3	3910 5580 1790×10^3		3990 5620 1920×10^3
D16	Yield Strength Maximun Strength Young's Modulus	—	—	3600 5510 1800×10^3		—
D10	Yield Strength Maximun Strength Young's Modulus	3960 5440 1660×10^3	3270 4860 1710×10^3	3220 4940 1580×10^3		3550 4750 1940×10^3
Flange	Yield Strength Maximun Strength Young's Modulus	3220 4800 1720×10^3	2770 4680 2000×10^3	3170 4610 2280×10^3	2760 4950 1930×10^3	3670 5560 2040×10^3
Web	Yield Strength Maximun Strength Young's Modulus	3900 4830 1710×10^3	3380 4850 2140×10^3	2040 4080 2010×10^3	3510 4380 2140×10^3	3120 4580 2020×10^3
Concrete Compressive Strength		200	261	330		288

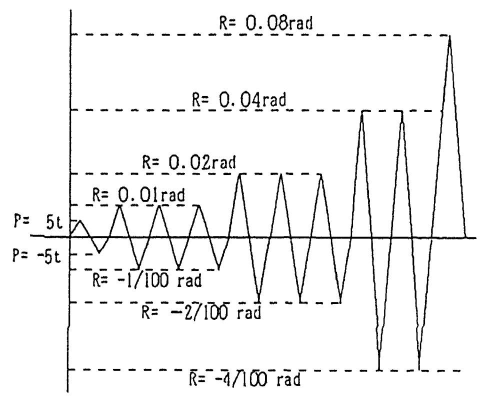

Fig. 4 Loading History

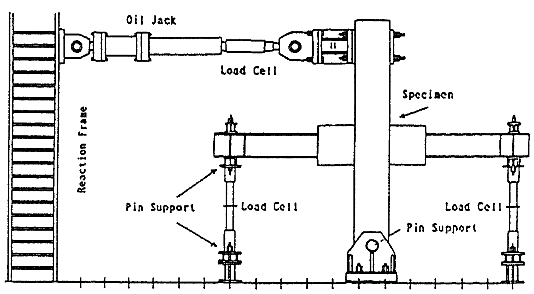

Fig. 3 Loading set up

herein experimental works using several specimens with interior and exterior beam-to-column connections putting an emphasis upon that the flexural strength of steel beams will govern shear failure of the connection.

2 TEST SPECIMEN AND TESTING METHOD

2.1 Specimens

Five specimens (C1, C3, C4, C5, C6) of interior beam-to-column connection and two specimens (T1, T2) of exterior beat-to-column were constructed and tested. In these tests we had three parameters, the detail of the steel beam in the connection and the flexural strength of the steel beam and that of the R/C beam. Two steel beams of C1 were inserted into both sides of the column about a quarter of the column depth and connected each other with a gusset plate. The thickness of the gusset plate was same as the steel beam web. The steel beams of C3, C4, C5 and C6 were embedded through the column and perpendicular steel beams were installed in the connection. Specimen C3, C4, C5 and C6 had different ratios of the flexural resistance of steel beam to that of R/C beam, and their shear stress levels in the connection related with the whole flexural strength of the beam. The longitudinal reinforcements of the R/C beam passed through the connection and were anchored at the ends of each R/C beam with hooks for C1 and with the anchor plate for C3, C4, C5 and C6. Regarding of specimens T1 and T2, the steel beams were inserted into the column by the same length as depth of steel beam. The perpendicular steel beam was installed only for T1, and the amount of the longitudinal reinforcements of R/C beam in T2 was more than in T1. The anchorage of longitudinal reinforcements is U-shaped anchoring in the column. The dimensions and components of specimens and the

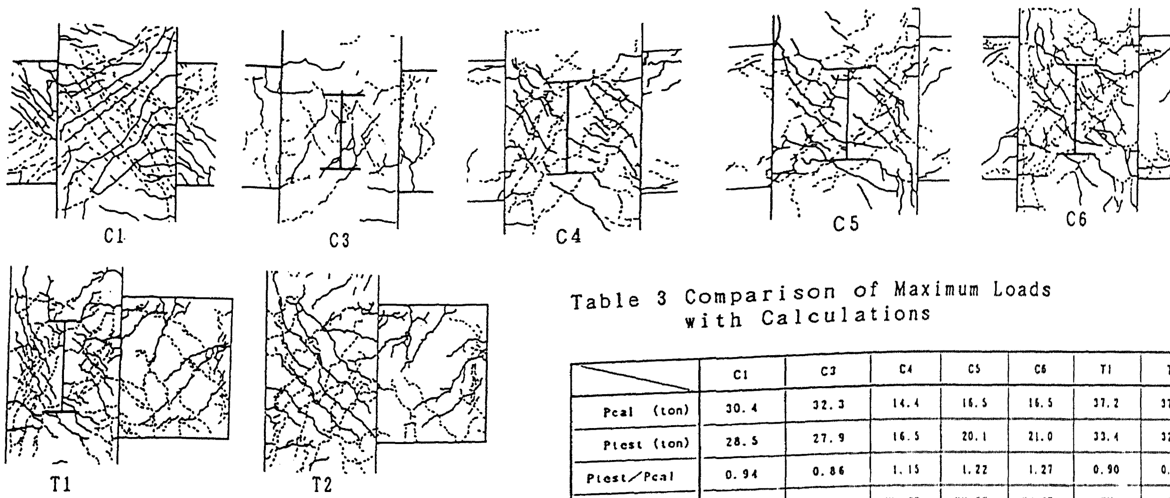

Fig. 5 Final Crack Patterns

Table 3 Comparison of Maximum Loads with Calculations

	C1	C3	C4	C5	C6	T1	T2
Pcal (ton)	30.4	32.3	14.4	16.5	16.5	37.2	37.2
Ptest (ton)	28.5	27.9	16.5	20.1	21.0	33.4	32.7
Ptest/Pcal	0.94	0.86	1.15	1.22	1.27	0.90	0.88
Failure	BY, JF	BY	BY, CF	BY, CF	BY, CF	BY	BY

Failure BY : The steel beam yielded CF : The connection concrete failed in shear

mechanical properties of materials were shown in Fig. 2, Table 1 and Table 2, respectively.

2.2 Testing method

Lateral cyclic loads were alternatively applied to the top of column using a hydraulic jack shown in Fig. 3, and the story displacement angle (=R) was controlled. The first cycle applied within elastic range, and after the second cycle, one cycle of R = ±0.005 rad, three cycles of R = ±0.01, ±0.02 rad. and two cycles of R = ±0.04 rad. were applied in plastic range. At last, the story displacement angle was made to reach 0.08 rad. The loading history was programmed as shown in Fig. 4. Columns of specimen C3, T1 and T2 were subjected to constant axial load 180 ton, 30 ton and 30 ton respectively during the test.

The displacement of the beam, the column and the connection were measured by electric displacement transducer, and strain in steel beams and reinforcements in beams and columns were measured using strain gages.

3 TEST RESULTS

3.1 Crack pattern and failure

Fig. 5. shows final crack patterns of each specimens.

The flexural cracks occurred at the critical section of the beam and small shear cracks appeared along the diagonal line of the connection panel by R = 0.005 rad. for all specimens.

For specimen C1, shear cracks in the connection grew up widely and propagated into the upper and lower columns while increasing of displacements. Many shear cracks were observed in the R/C beam and finally, cover concrete in the upper side of the beams and in the center of connection panel and in the corner of the columns spalled off. The test was discontinued, when the local buckling occurred in the steel beam the test was discontinued.

For specimen C3, concrete in the connections and the R/C beam was not damaged so much severely in the stage of story displacements more than R = 0.01 rad., since the previous shear crack developed a little and small number of new cracks occurred and concrete crashing was not observed.

For specimens of C4, C5 and C6 many shear cracks were observed in the connection. In particular the diagonal cracks along the line from compression corner of the connection to web fillet of perpendicular steel beam grew up widely. During the cycle of R = ±0.04 rad., concrete crushed in the connection.

The flexural cracks in the specimens of T1 and T2 appeared more than interior connection specimens. In the connection panel of T1 having perpendicular steel beams, the shear cracks concentrated in the outside area of the connection and inclined more vertically than the shear cracks in the inner area divided by the perpendicular steel beams. For the specimen T2 not having perpendicular steel beams, several long diagonal shear cracks occurred under both the positive and the negative cycles. Crash of concrete and failure in R/C beam were not observed through the test in both specimens.

The comparison of maximum loads with calculation is shown in table 3. The maximum loads were calculated based on the ultimate strength required in the standard for structural calculation of steel reinforced concrete structure. The maximum loads of C4, C5

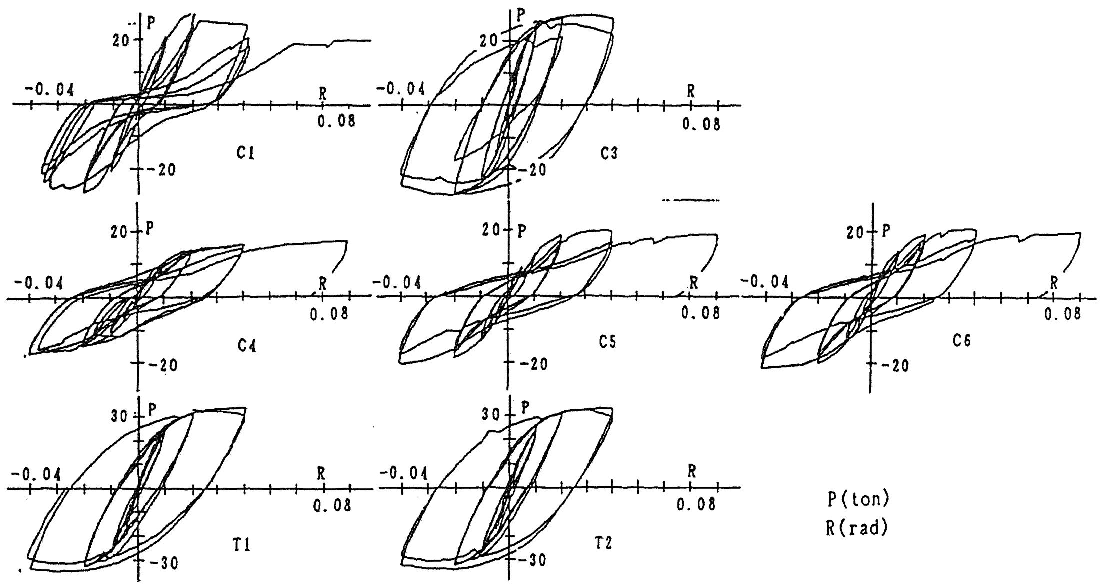

Fig. 6 Relationships between Load and Story Displacement Angle

and C6 exceeded the calculation, but those of C1 did not reach to calculation due to *several shear failure of the connection.* The maximum loads of C3, T1 and T2 did not also rise to calculations because of failure of bending in beam.

3.2 Deformation characteristics

The curves of loads versus story displacements angle for all specimens are shown in Fig. 6. The pinched hysteresis was exhibited and the maximum loads were recorded at R = 0.02 rad. and strength decay was observed remarkably for specimen C1.

Specimen C3, T1 and T2 show good spindle shapes of loops such as a steel frame structure. The strength reduced at the second cycle of R = 0.04 rad. The reductions were caused by local buckling of the steel beam near to the connection with R/C beams, without failure in the connection.

Specimen C4, decayed strength in the same displacement cycle, but showed comparatively better hysteresis than C1, and at the displacements of R = 0.08 rad got maximum strength during tests.

For specimen C5 and C6 , the maximum strength of each specimen was recorded at the peak load in the displacements R = 0.04 rad., and the peak loads of R = 0.08 rad went down to 95 % of their maximum strength, when the severe shear failures occurred in the connections but did not affect so much to the strength decays and the hysteresis loops of the structures.

Fig. 7 shows the relationships between the shear stress and the shear deformation angle in the connections of all specimens. The shear stress was calculated by following *equation:*

$$\tau p = ((P1+P2)L/jB - Q)/Dc/tp \qquad (1)$$

P1, P2 : reaction of left or right beam
L : distance from support of the beam to column face
jB : lever arm of beam
Dc : column depth
tp : effective thickness of panel (mean width of beam and column)

Dc multiplied by tp is indicated the effective area of the connection which is adopted in the design guideline for earthquake resistant reinforced concrete building based on ultimate strength concept of A. I. J.

For specimen the shear stress in the connection raised up until 60 kg/cm2 at R = 0.02 rad., and the shear deformation continue to increase in the cycle of R = 0.04 rad. For specimen C3, the maximum shear stress was about 50 kg/cm2 and the maximum shear deformation was smaller than C1. The maximum shear stresses are 80, 95, 100 kg/cm2 for specimen C4, C5 and C6, respectively. Increase of deformation can be observed at R = 0.04 rad cycles in the curve of specimen C4. As the shear stress got higher, the shear deformation got lager. As for specimen T1 and T2, the maximum shear stresses were almost in the same level, but the shear deformation of specimen T1 with perpendicular steel beams was rather lager than that of specimen T2 without perpendicular steel beams.

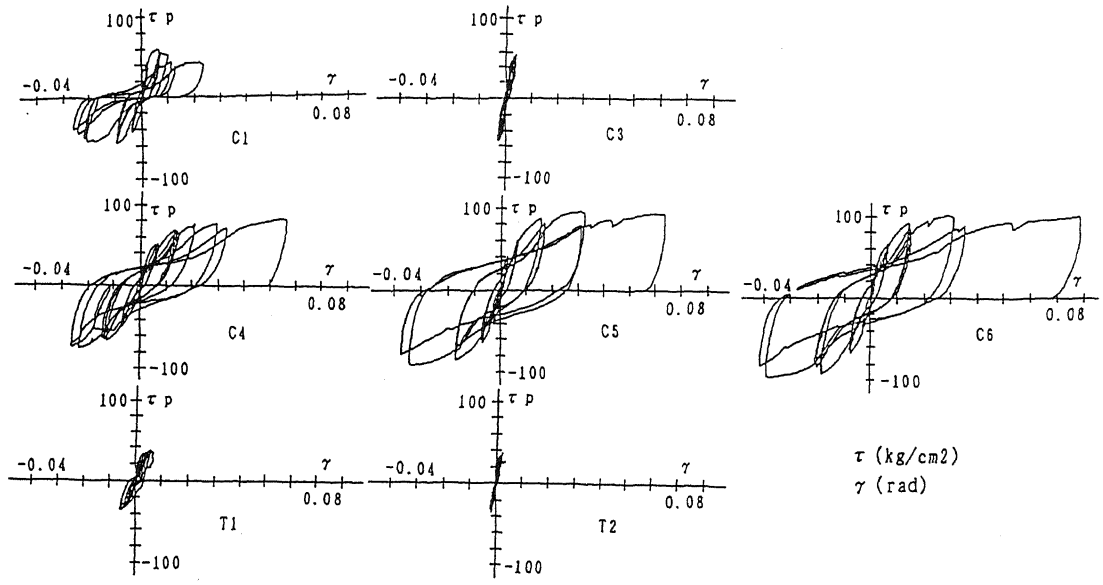

Fig.7 Relationships between Shear Stress and Shear Deformation Angle

3.3 Strain of steels and reinforcements

Fig.8 shows strain distribution of the flanges of steel beams and the longitudinal bars of R/C beams for specimen C4, C5 and C6. Steel beams yielded at the boundaries between steel beams and R/C beams in the cycle of R = 0.01 rad., and in the progress of cycles the yield zone propagated within the R/C beam, and at the ultimate stage, the yield strain extremely concentrated at the boundary for specimen C4 and at the column faces for specimen C5 and C6.

The longitudinal bars of specimen C4 and C6 remained elastic before R = 0.04 rad. and only for specimen C5, strain at the column face reached to yield at R = 0.04 rad. Strain of the longitudinal bars of three specimens indicated large increment more than yield strain at the ultimate stage.

Fig.9 shows strain of hoops in the connection. Hoops (No.2, 3, 4) located among upper and lower flanges of the steel beam yielded at R = 0.04 ~ 0.08 rad. Other hoops (No.1, 5) almost remained to be elastic or yielded only at the ultimate stages.

4 SHEAR STRESS AND DEFORMATION IN THE CONNECTIONS

Fig.10 shows the relationships between the ratio of the shear stress in the connection to compressive strength of cylinder concrete test ($\tau p/\sigma B$) and the shear deformation (γp) at the first peak load of each cycle for exteerior connection specimens. Data of R/C specimens, WJ6 whose connection failed in shear and WJ2 whose beams failed in bending, are plotted in the same figure. It is indicated obviously that the maximum shear deformations of the connections of those reinforced concrete specimen were smaller than that of our composite specimens. In the design guideline above mentioned, the $\tau p/\sigma B$ is limited less than 0.3. The shear stress of specimen C1 reached about $0.3\sigma B$ but strength was deteriorated remarkably. In this specimen the steel beam was not embedded through the connection and did not have the perpendicular steel beam. The strength of WJ6 was also deteriorated after shear stress went up to $0.3\sigma B$. The shear stress of specimen C6 raised up to $0.3\sigma B$ as C1, but strength decay was not observed and shear deformation gradually went up large. It was considered that the composite frame in which the steel beam was continuously developed through the connection had more ductile deformation capacity than the reinforced concrete frame.

5 CONCLUDING REMARKS

Following findings were obtained from the test results.

The steel beam stiffened by reinforced concrete at the end of beam could be made to yield at the baked steel beam in the displacement more than R = 0.01 rad. As the ultimate stage, the yield zone of steel propagated within the R/C beam.

From the test results of specimen C6, although the connection concrete failed in shear, the strength in the connection raised up to $0.3\sigma B$ which adopted the ultimate shear strength of the connection of A.I.J., and did

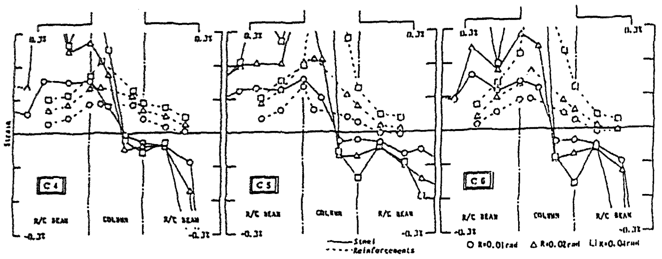

Fig. 8 Strain Distributions

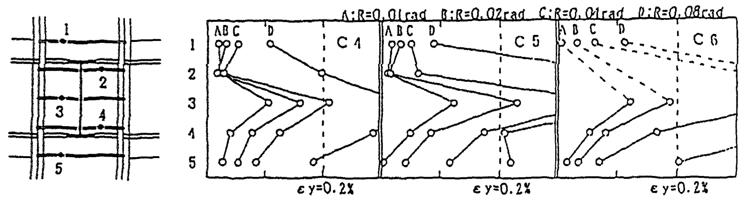

Fig. 9 Strain in Reinforcements in the Connection

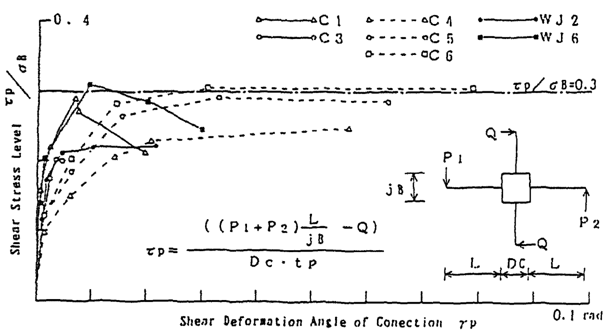

Fig. 10 $\tau p/\sigma B - \gamma p$ Relationships

not deteriorate remarkably.

When the steel was embedded through the connection and perpendicular steel beams were provided in the connection, the shear deformation capacity of the connection would get lager, it is regarded consequently that the composite structure is capable of generating more ductile deformation in the connection than that of reinforced concrete structure.

The maximum strength of the specimen failed in the connection was superior to the ultimate strength of the connection calculated based on the standard of the steel reinforced concrete structure.

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