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Lateral strength and plastic deformation of R.C. railway frame structure

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ABSTRACT: A static reversed cyclic loading test and an elastoplastic frame analysis are carried out on a one-bay two-story reinforced concrete frame structure which is a one third scale model of the typical railway viaduct commonly constructed in Japan. In order to obtain rational and simple seismic design concept, the yield capacity and the relation in toughness between the structure and members are studied.

1 INTRODUCTION

The new seismic design code for the railway concrete structures in Japan [1] adopted the limit state design method. Considering the toughness of structure in design, it ensures the seismic safety against the earthquake of large magnitude which is assumed to occur nearly once in the lifetime of railway structure that is estimated approximately 100 years.

In the design code, the seismic safety is assessed by examining whether the yield capacity of the structure and its displacement toughness factor, which ensures stable sustenance of the yield capacity, are larger than the design lateral inertia load and corresponding displacement ductility factor respectively. It is stipulated to evaluate the displacement toughness factor of member using an equation proposed by Ishibashi et al. [2] which is derived based on some experimental results whose specimens cover most of the typical dimensions of railway concrete members. In this evaluation, the member is defined to have yielded the fiber tension reinforcing bar reaches the yield point. For the statically determinate structures like bridge piers, the above-mentioned evaluating method may be directly used in the design. However, the following items shall be primarily studied for the statically indeterminate structures like frame-type viaducts:

1. the definition of yield capacity as a structure;
2. the relation between the displacement toughness factor of members and the required (design) displacement ductility factor for a structure; and
3. the behavior of members in a structure whose toughness factors are quite different from others.

2 SCOPE OF THIS STUDY

In this paper, the definition of yield capacity and the relation in ductility factor between structure and member are studied through a series of loading tests on one-bay two-story reinforced concrete frame specimens and a static elastoplastic analysis.

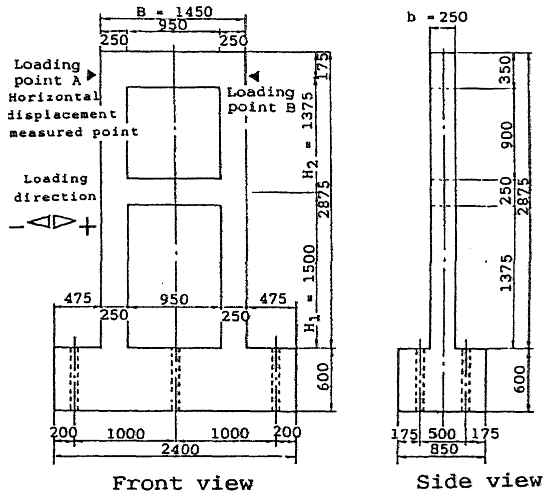


Fig. 1 Test specimen

3 OUTLINE OF TEST PROGRAM

3.1 Specimen

Three one-third scale one-bay two-story reinforced concrete frames (F1, F2 and F3) modeling the section in transverse direction of an actual railway viaduct were designed and constructed as specimens. The shape and dimension of the specimen are shown in Fig.1 and Table 1.

The reinforcement of each specimen was varied and designed to produce the first plastic hinge at the lower-story beam, the lower-story column and the upper-story beam in F1, F2 and F3, respectively. The section sizes of members, however, are kept constant for all specimens. Larger stiffness was given to the upper-story beam than that of columns, modeling the fact that the top-story of railway viaduct usually has floor slab to support rail track. Amounts of transverse

Table 1 Dimensions and reinforcement of test specimens

Specimen Number	Dimension H	H_1	H_2	B	Cross section Beam (h×b)	Column (h×b)	Reinforcement Location	Longitudinal A_{s1}	ρ_t	f_{sy}	Transverse A_{s2}	S	ρ_w	f_{wy}	f_c
1							Upper beam	D10×5	0.45(1.30)	374	D10	75	0.76	374	27.5
							Lower beam	D10×5	0.53(1.83)	374	D10	80	0.76		
							Column	D13×5	1.17(3.24)	392	D10	60	0.95 (Upper)		
												75	0.76 (Lower)		
2	2875	1500	1375	1450	Upper :350×250	250 × 250	Upper beam	D13×5	0.80(2.32)	392	D10	75	0.76	374	31.9
							Lower beam	D13×5	1.17(3.24)	392	D10	60	0.95		
					Lower :250×250		Column	D10×5	0.53(1.83)	374	D10	75	0.76 (Upper)		
												100	0.53 (Lower)		
3							Upper beam	D6×5	0.20(0.58)	396	D10	100	0.57	374	25.8
							Lower beam	D13×5	1.17(3.24)	392	D10	60	0.95		
							Column	D10×5	1.17(3.24)	392	D10	60	0.95 (Upper)		
												75	0.76 (Lower)		

Unit : mm

ρ = Reinforcement ratio
f = Strength(MPa)
s = Pitch(mm)

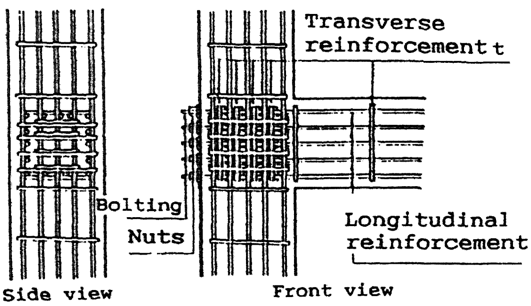


Fig. 2 Beam-column joint reinforcement details

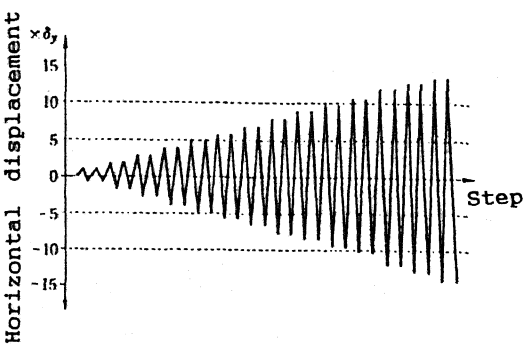


Fig. 3 Loading history

reinforcement of all members are determined using the displacement toughness evaluation equation [2]. The toughness factor was set at 8.

The longitudinal reinforcement ratio, the transverse reinforcement ratio (hoop and stirrup), the yielding point of reinforcing bar and the compressive strength of concrete cylinder are shown in Table 1. Only the areas of bars placed at top or bottom in the section were taken into account as the longitudinal tensile reinforcement area (A_{s1}).

The beam-column joint reinforcement details are shown in Fig.2. The spacing of transverse reinforcement is set half that of hoop and stirrup in adjacent members. A mechanical anchorage of longitudinal reinforcement of beams and columns is adopted using plates and nuts.

3.2 Loading method

The reverse cyclic loading was applied from the sides of specimen at the beam-column joints in the upper-story. The lateral displacement of specimen was also measured at this point. The typical loading history is shown in Fig.3. After confirming the first cracking strength and positive and negative yield capacities, the positive and negative maximum displacements in a loading cycle were controlled based on the yield displacements. The maximum displacements were increased monotonously in steps of integral multiple of yield displacements. The loading and unloading were repeated twice in a step. The yield capacity was defined as the load causing a tensile strain of 2000 μ in any of the longitudinal reinforcements.

Table 2 Loads and displacements at yielding and maximum strength

Specimen Number		Commencement of crack		Yielding		Maximum			Sequence of hinge formation			
		P_{cr}	Location	P_y	δ_y	P_{max}	δ	δ/δ_y	$1\delta_y$	$2\delta_y$	$3\delta_y$	$4\delta_y$
1	+	20.0	Column Lower beam	91.4	13.9	146	55.6	4.0	f	e a b	i ℓ	k j
	−	−16.0		−88.5	−13.7	−142	−67.2	5.0	e	f b a ℓ	a j	ℓ k
2	+	21.0	Lower beam	85.2	12.9	119	52.0	4.0	a	i j c	b f	d
	−	−8.0		−78.4	−14.2	−110	−42.9	3.0	b	a d j e	c i	
3	+	17.5	Column Lower beam	87.4	13.8	148	55.2	4.0	k	ℓ a b	e	
	−	−15.0		−72.0	−11.1	−131	−44.4	4.0	ℓ	k b	a	e f

Unit P:(kN)
δ:(mm)

4 TEST RESULTS

4.1 Load-displacement relation

The cracking strength, positive and negative yield capacities and yield displacements, the maximum capacities and their displacements, and the sequence of plastic hinge formation at the ends of members of three specimens are shown in Table 2. Positions of plastic hinge formation shown in the table correspond to the indexes shown in Fig.4. The load-displacement hysteresis curves are shown in Fig.5. From them, the following results are obtained;

1. Considerable increase in lateral capacity after the yield capacity is reached is observed in all the specimens.
2. Large toughness is assured in all the specimens because the decrease in the capacity after attainment of the maximum capacity is mild.
3. Large energy dissipating ability is sustained up to the end of testing in all the specimens.

4.2 Rigidity

The tangential rigidities between displacement steps K_i (see Fig.6) up to the maximum capacities of three specimens and the tangential rigidities at the yielding of the lower-story column K_c are shown in Fig.7.

The yield rigidities (the rigidity at yield capacity) of all the specimens are almost the same ($0.65*10^5$ MPa). Moreover, K_c is found to be approximately a mere 10% less than K_1. So it is supposed that a fairly large rigidity is sustained until the formation of plastic hinges in the lower-story columns.

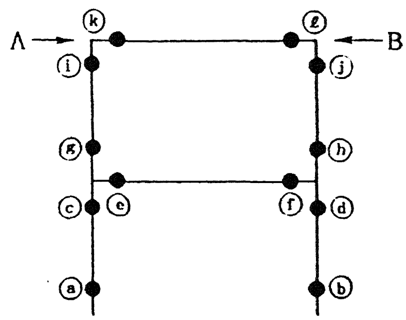
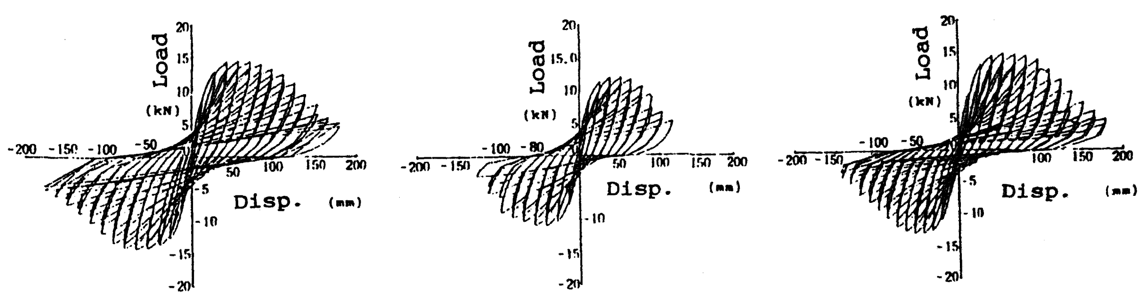


Fig. 4 Expected plastic hinge locations



(a) Specimen 1 (b) Specimen 2 (c) Specimen 3

Fig. 5 Load versus horizontal displacement

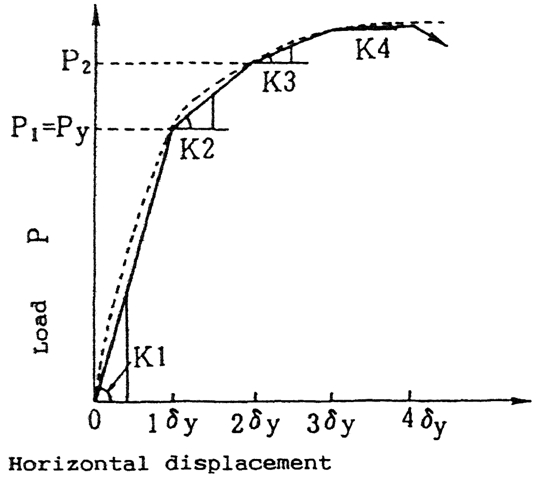


Fig. 6 Secant rigidities for each loading step

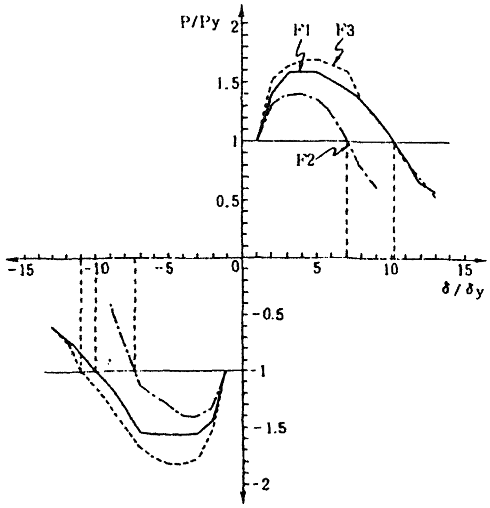


Fig. 8 Envelope of P-δ hysteresis curve

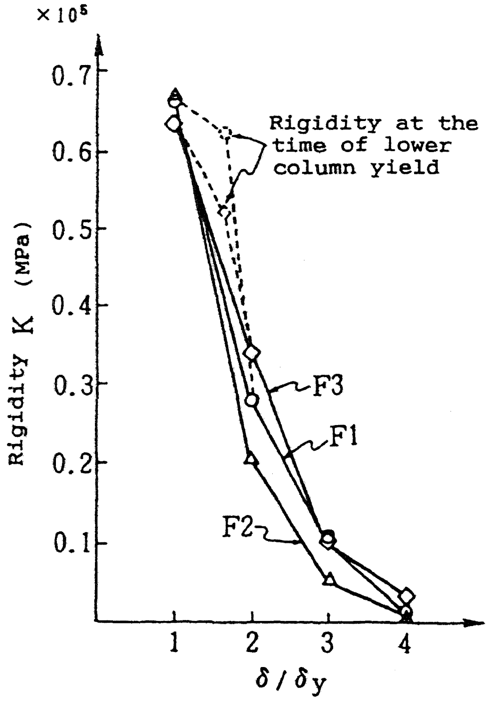


Fig. 7 Measured rigidity

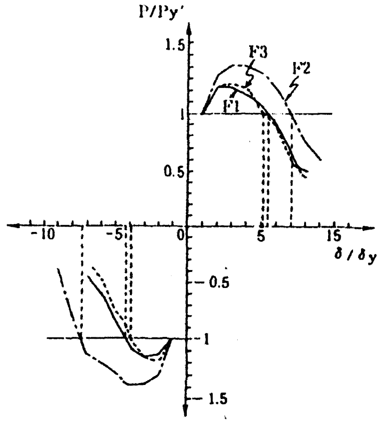


Fig. 9 Modified envelope of P-δ hysteresis

4.3 Re-definition of yield capacity of structure

The envelope curves of load-displacement hysteresis, where each envelope curve is normalized using respective yield capacity and yield displacement of each specimen, are shown in Fig.8. From these curves, the displacement toughness factors are obtained as 10, 7 and 10 for specimens F1, F2 and F3, respectively.

Surplus capacities and large toughness are observed with specimen F1 and F3 whose first plastic hinges are formed in beams. This surplus is ignored in the conventional design, resulting in uneconomical structures. Therefore, taking into account the sustenance of rigidity before column yields and its large toughness, re-definition of structure yielding may be possible.

Newly defined normalized envelope curves are shown in Fig.9. From the diagram, specimens F1 and F3 are found to still retain a sufficient displacement toughness factor of approximately 5.

5 ANALYSIS OF TEST STRUCTURE

The newly defined yield capacity of structure needs an assessment of how far the yield strength of beam can be decreased in the allowable range without requiring a

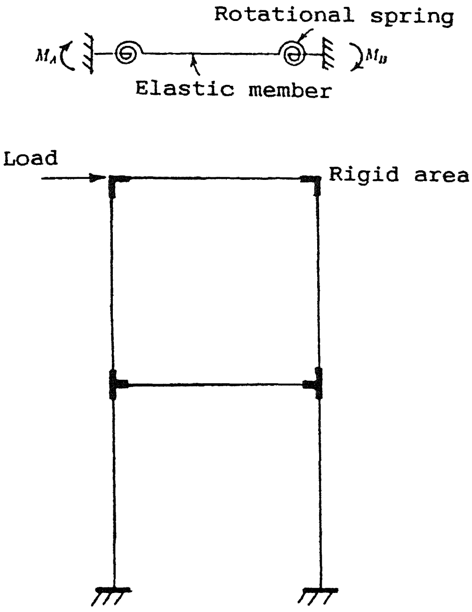


Fig. 10 Analytical model

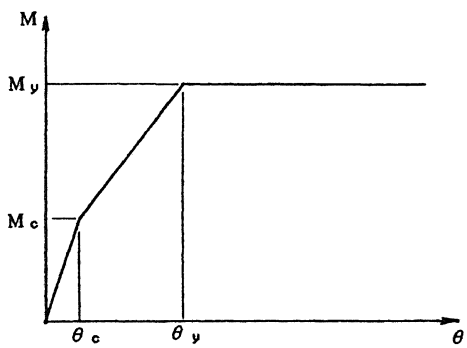


Fig. 11 Assumed moment-curvature relation

large displacement toughness factor or concentration of deflection. A static elastoplastic analysis was performed for the assessment.

5.1 Analysis and modeling

Every member of a frame was idealized as an elastic element with two nonlinear rotational springs at the two ends (see Fig.10).

Yield moments and curvatures of springs (see Fig.11) are computed using an idealized stress-strain relationship for the concrete and for the reinforcing steel, taking into account the effect of diagonal cracking and the effect of pullout of bars from joints and bases. Beam-column joints are assumed as rigid zones.

The dimensions of specimen F1 are used as the reference values in the analysis. Only the yield moment of lower-story beam was varied. The ratios of yield moments to the reference moment are noted as the strength ratio α.

5.2 Assessment of ductility

The design code [1] basically requires a toughness factor of four ($4\delta_y$) for concrete structures, so the ductility factors of the beam and column in lower-story at the time when the displacement ductility factor of the frame reaches 4 are computed. The displacement ductility factor of structure is computed using the lateral displacement at the beam-column joint of upper-story referring to the displacement at the time when the lower-story column yields. The displacement ductility factors of members are computed using the relative transverse displacement between the center of members and the end of members referring to the yielding displacement of respective members (see Fig.12). From the analysis, the following results are obtained (see Fig.13):

1. In the case that the strength ratio of beam in lower-story decreases, its ductility factor increases proportionally to the reduction of its yield moment. However, the rate of increment remains mild provided the strength ratio is not less than 0.85.
2. In the same case, the difference between the displacement ductility factor of structure and that of column of lower story remains a mere in 20%.
3. On the contrary, in the case that the strength ratio of beam in lower-story increases, the ductility factor of column in lower-story increases. The rate of increment, however, continues to be mild provided the strength ratio is less than 1.3.

6 CONCLUSIONS

The knowledge obtained through this research is summarized below.

1. For the low-story frame structure as railway viaduct, the displacement toughness factor of structure may be evaluated referring to the lateral displacement of a member in top-story and its yield displacement which is defined to take place when the column in base story produces a plastic hinge.
2. In the case of a weak beam system, which means that the first yielding occurs in lateral member, the displacement toughness factor may be almost equivalent to that of the columns in lower-story.
3. Within the range of strength ratios α from 0.85 to

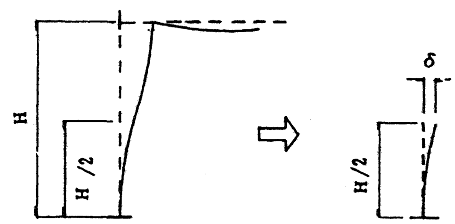


Fig.12 Converted displacement ductility of member

Table 3 M and θ values used in analysis

Case number	Strength ratio α	Lower beam				Lower column				Upper column			
		M_y	θ_y	M_c	θ_c	M_y	θ_y	M_c	θ_c	M_y	θ_y	M_c	θ_c
1	1.5	90.6	8.09										
2	1.25	75.5	7.24										
3	1.0	60.4	6.36	6.25	0.114	60.9	7.25	6.5	0.171	60.8	6.42	6.5	0.122
4	0.85	51.3	5.82										
5	0.70	42.3	5.33										

M : KN·m
θ : rad($\times 10^{-3}$)

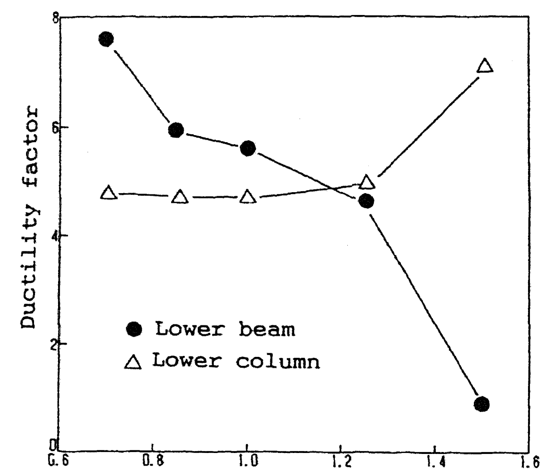


Fig.13 Strength ratio versus ductility factor

1.3, the concentration of deformation is avoidable when a slight extra toughness is provided to the weak member.

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