Static and Repeated Load Tests on Precast Concrete Beam-to-Column Connections

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An experimental investigation was undertaken into the strength and deformation behavior of two types of precast reinforced concrete beam-to-column connections. Referred to in this paper as Types A and B, these connections are recommended by the PCI Committee on Connection Details and the Australian Prestressed Concrete Group for use in precast reinforced concrete building frames. A total of 18 half-scale interior connection models were designed, built, and tested to failure to evaluate their strength and ductility properties under static and unidirectional repeated loading. The comparative study shows that the two types of precast concrete connections performed satisfactorily in that their bending strengths are, without exception, higher than the monolithic connections. In addition, the ductility and energy absorbing capacities of the precast connections, generally, are superior to their monolithic counterparts.

Connection design is one of the most important considerations for the successful construction of precast reinforced concrete structures. The detailing and structural behavior of the connection affect the strength, stability, and constructibility as well as the load redistribution of the building under loads.

Although the PCI manuals[1,2] contain descriptions of approximately 40 beam-to-column connections fulfilling many functions, published test results are available for only a few of them. Reliable connection behavior can only be properly assessed by laboratory testing or proven performance in the field.

This paper presents a laboratory study of the strength and deformation behavior of beam-to-column connections suitable for use in precast reinforced concrete building frames. In all, 18 half-scale model connections were built and tested. They include:

- For the static load tests,[3,4] four monolithic models and four each of the precast connection Types A and B.[1,5]

• For the unidirectional repeated load tests,[4] two models each of the monolithic and precast connection types.

To make meaningful comparisons, the tests were undertaken in groups with the same controlled conditions.

TEST MODELS

The design of the models was based on the structural requirements of a proposed five-story reinforced concrete frame. The proposed frame, which is shown in Fig. 1, forms part of a tentative low-cost residential building system.[6] The two types of precast connections were designed according to recommended guidelines.[1,2,5,7] The reinforced concrete design and manufacturing process comply with the Australian Standard.[8,9]

In all, 18 half-scale models were fabricated, making six groups of two precast specimens (Types A and B) and one monolithic specimen. All models had the same dimensions but different groups had different concrete strengths and/or steel ratios. Of the models, 12 were tested under static loading. The remaining six were tested under unidirectional repeated loading.

Each model was identified by two letters and a number. The first letter, S or R, indicates static or repeated loading; the second letter, M, A, or B, represents monolithic construction or precast connection Type A or B. The number at the end identifies the different tensile reinforcing steel contents.

The use of half-scale models was considered appropriate because all structural details can be incorporated with ease. However, for Type A models, the bond lengths of the reinforcing bars at the connections were found to be inadequate. As a result, the longitudinal bars from the precast connecting beam were welded to the corresponding bars of the precast frame over a length of 100 mm (3.93 in.). In view of the comparatively large model scale, size effects are not believed to be significant if they existed at all.

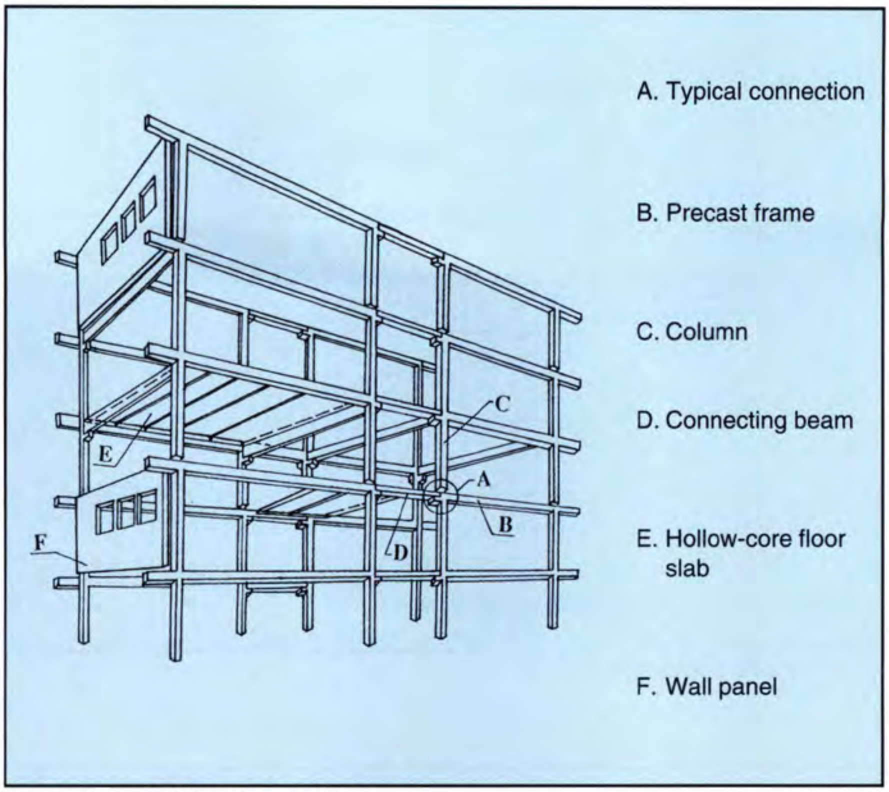


Fig. 1. Proposed five-story building.

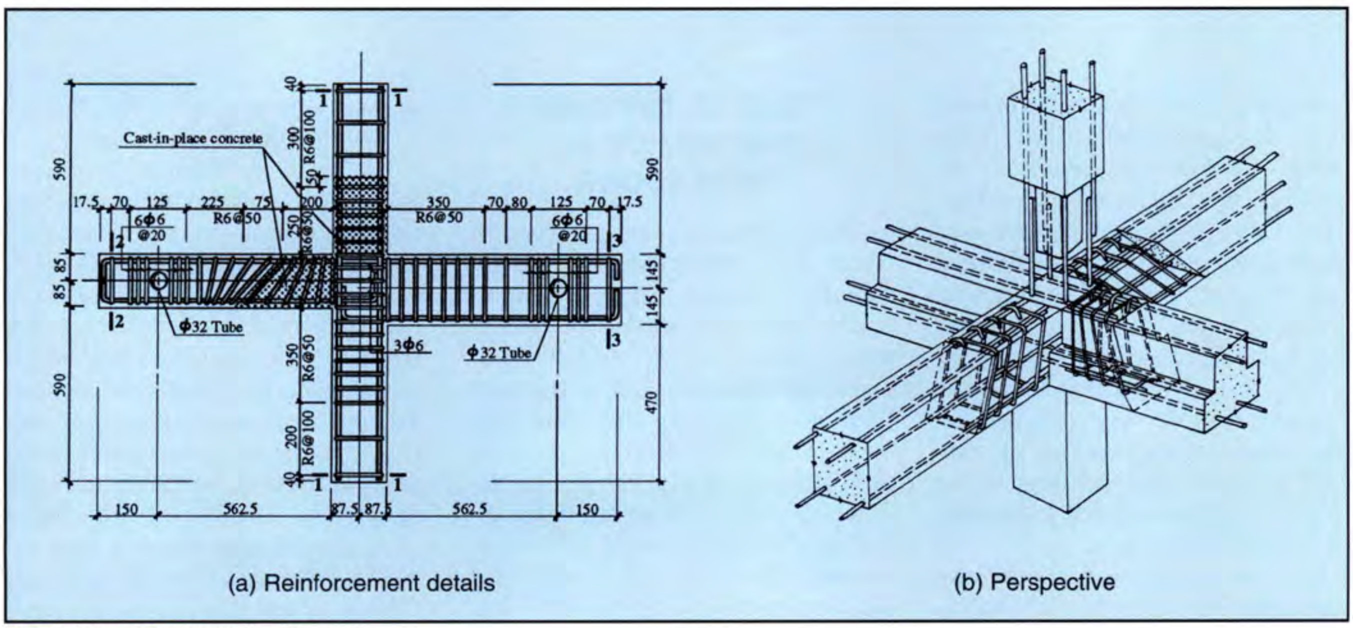
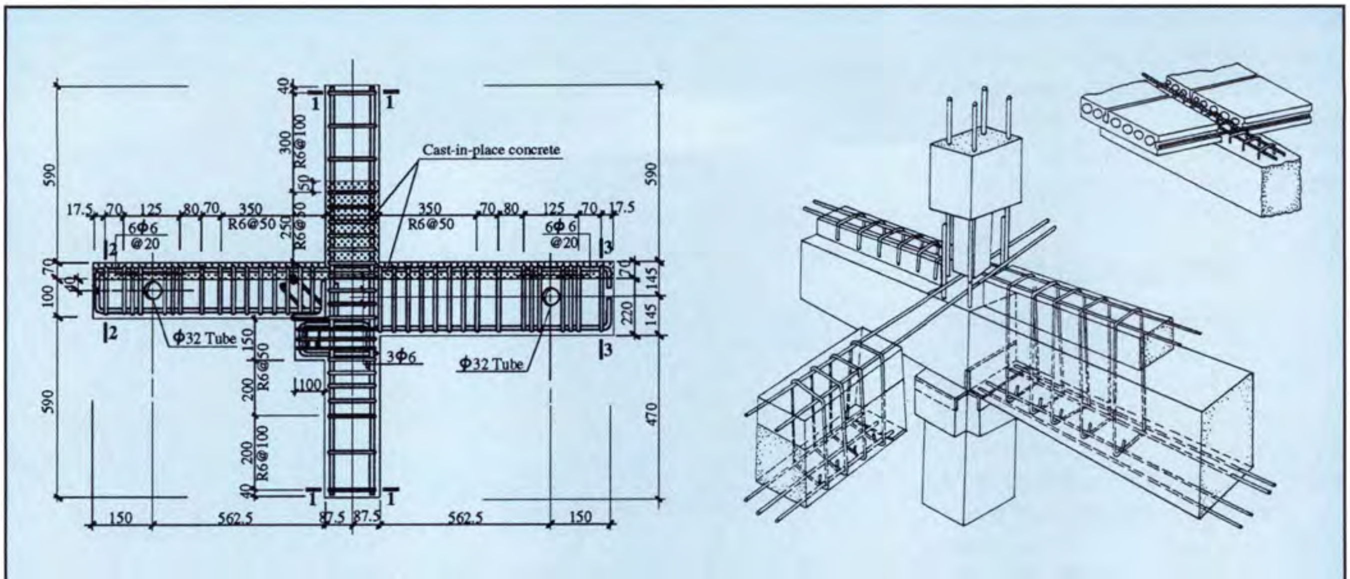
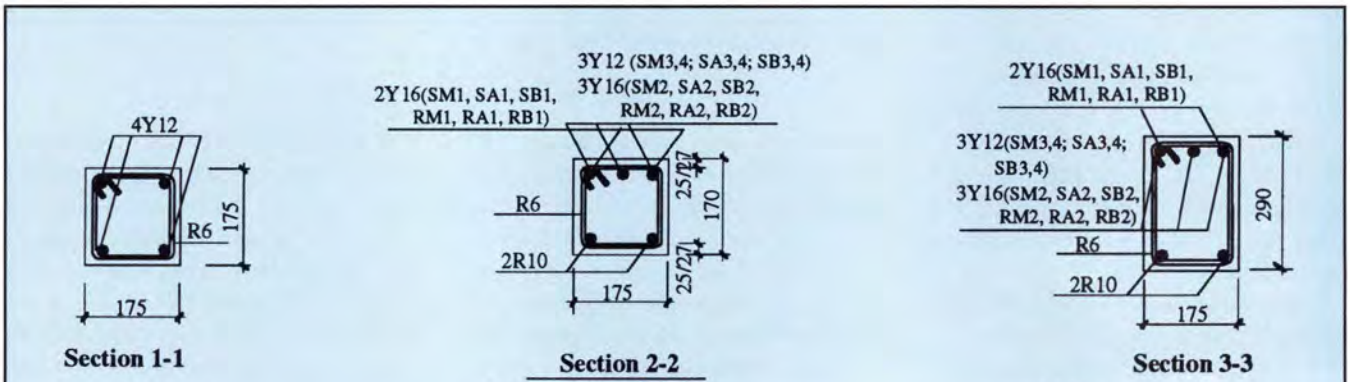


Fig. 2. Precast connection Type A.



(a) Reinforcement details (b) Perspective

Fig. 3. Precast connection Type B.



(a) Column (b) Connecting beam (c) Frame beam

Fig. 4 Cross-sectional details of beams and columns (refer to Figs. 2 and 3 and Table 1).

About 0.1 m^3 (3.53 cu ft) of concrete was required to cast each model. Commercial premixed concrete was used in the model construction. Cast-in-place concrete was mixed in the laboratory when assembling the components. The structural details of precast connection Types A and B are given in Figs. 2 and 3, respectively. Fig. 4, together with Table 1, summarizes the material properties and cross-sectional details of the connecting beams and columns of the models. Note that all the connecting beams are under-reinforced.[9] The construction processes for the precast connections are described in detail in other literature.[1,3,5]

TEST SETUP AND EXPERIMENTAL PROCEDURE

The loading apparatus consisted of three floor-mounted steel portal frames, as shown in Fig. 5. The required loads were applied by two hydraulic jacks, 1 and 2, with Jack 3 providing the balance for Jack 2. The loads were measured using "Interface" load cells [Model 1220-BF with 113.5 kN (25.5 kips) capacity]. The vertical deflection of the connecting beam directly under the loading point was measured by Dial Gauge 1, which had a maximum travel of 100 mm (3.93 in.). The tensile and compressive strains in the concrete were measured using 200 mm (7.87 in.) Demec® strain gauges.

The beam and column deflections and concrete strains were recorded manually at each load stage, until failure occurred. The load-deflection curves of the models under repeated loading were drawn with the aid of a Hewlett Packard plotter. The strains on the reinforcing bars at the connecting zone were measured using 10 mm (0.39 in.) electrical resistance strain gauges (Tokyo Sokki Kenkyujo, Type PL-10-11). The strain values were recorded using a Hewlett Packard 3054A Automatic Data Acquisition/Control System.

For every model test, an axial load,

Table 1. Details of beams and columns (refer to Fig. 4).

Groups	Name of specimens	Type of connections	Properties of beams										Reinforcements of columns				Precast concrete strength (MPa)	Cast-in-place concrete strength (MPa)
			Connecting beams					Frame beams										
			Reinforcement				Spacing of ties (mm)	Reinforcement				Spacing of ties (mm)	Cover (mm)	Area (mm^2)	f_{sy} (MPa)	Spacing of ties (mm)		
			Top		Bottom			Top		Bottom								
			Area (mm^2)	f_{sy} (MPa)	Area (mm^2)	f_{sy} (MPa)		Area (mm^2)	f_{sy} (MPa)	Area (mm^2)	f_{sy} (MPa)							
1	SM1	Monolithic	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	30	—
	SA1	A	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	30	59
	SB1	B	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	30	59
2	SM2	Monolithic	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	30	—
	SA2	A	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	30	59
	SB2	B	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	30	59
3	SM3	Monolithic	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	13	—
	SA3	A	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	13	67
	SB3	B	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	13	65
4	SM4	Monolithic	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	53	—
	SA4	A	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	53	78
	SB4	B	330	440	160	325	50/100	330	440	160	325	50/100	27	440	440	50/100	37	60
5	RM1	Monolithic	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	37	—
	RA1	A	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	37	67
	RB1	B	400	440	160	372	50/100	400	440	160	372	50/100	25	440	440	50/100	37	67
6	RM2	Monolithic	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	37	—
	RA2	A	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	37	67
	RB2	B	600	440	160	372	50/100	600	440	160	372	50/100	25	440	440	50/100	37	67

Note: 1 mm = 0.394 in.; 1 mm^2 = 0.00155 $in.^2$; 1 MPa = 145 psi.

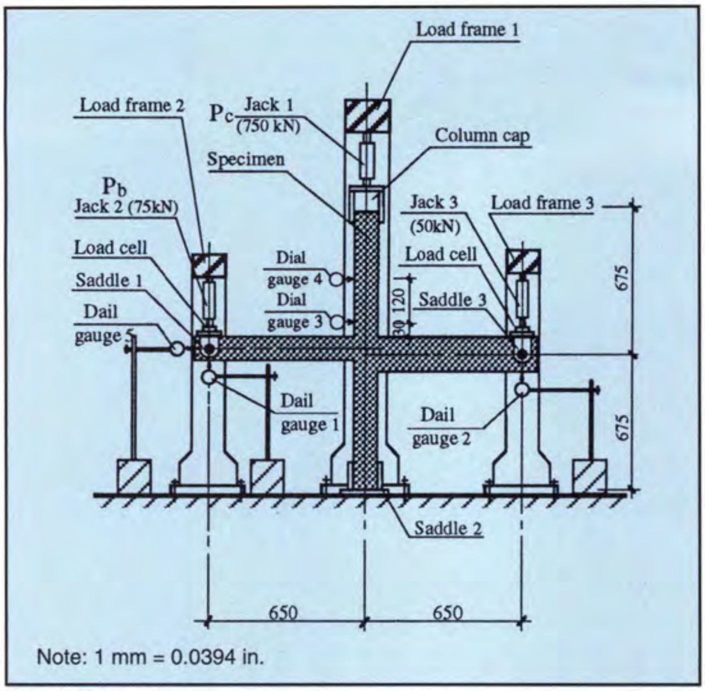


Fig. 5. Test setup.

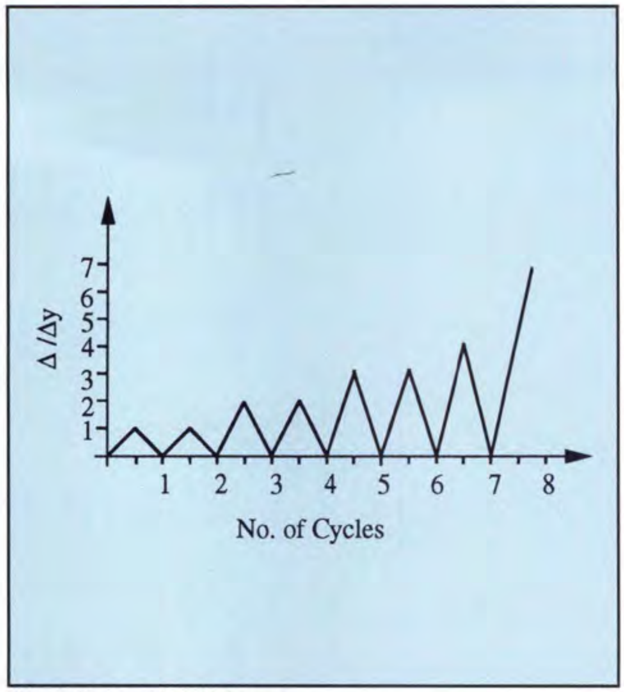


Fig. 6. Typical load history.

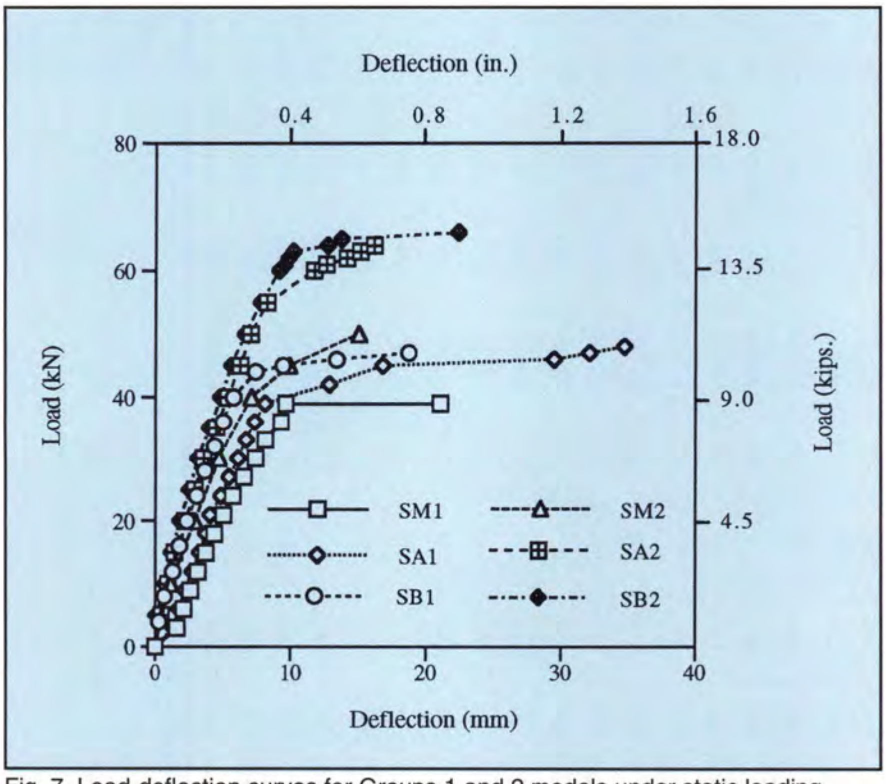


Fig. 7. Load-deflection curves for Groups 1 and 2 models under static loading.

P_c, was first applied at the top of the column. This load, which was equal to 10 percent of the design axial strength of the column, was kept constant throughout the test. Then a vertical load, P_b, was applied to the connecting beam stage by stage until failure of the model occurred. For the repeated loading test, the load P_b was controlled by the magnitude of the vertical deflection, Δ, measured at the tip of the beam (Dial Gauge 1, Fig. 5). The vertical tip deflection was increased in multiples of Δ_y, where Δ_y is the deflection at first yield. A typical load history diagram is shown in Fig. 6.

In between load applications, visual inspection and manual marking of cracks and crack propagation were carried out. For each test, the loading was continued until failure occurred. Failure was indicated by a marked increase in beam deflection accompanied by a rapid decrease in the vertical load P_b.

STATIC LOADING TEST RESULTS AND DISCUSSION

Flexural Strength

The flexural strength of the cantilever connecting beam is indicated by the ultimate load P_u. The values of P_u and the corresponding calculated ultimate loads, $P_{u,cal}$, are listed in Table 2.

It is clear that the flexural strengths of the precast concrete connecting beams were invariably greater than those of their monolithic counterparts. This was mainly due to the strength of the cast-in-place concrete being much greater than the concrete strength of the components and the corresponding monolithic models (see Table 2, Column 3). The overlapping of the longitudinal bars for Type A models also helped to increase the bending strength of the connections.

Similarly, for Type B models, the

compression bars were welded to the steel angle. This was in turn welded to the corbel. The resulting welded assembly tended to strengthen the compression zone of the connection.

Deformation and Ductility

The ductility behavior may be expressed as the ratio of the ultimate deflection, Δ_u, to the deflection at initial yield Δ_y. The values of Δ_u/Δ_y for all the models are shown in Table 2. The load-deflection curves for some of the specimens are presented in Fig. 7.

From these results, it can be observed that all the precast models possessed not only greater ductility but also higher stiffness than their monolithic counterparts. Further, while generally achieving a higher bending strength, the Type B precast connections were inferior to both the monolithic and Type A models in ductility characteristics as the tensile steel content increases.

Cracking Behavior

The crack propagations and the failure crack patterns of all the models were largely identical. The cracking loads of the precast models are mostly larger than their monolithic counterparts. Further discussion on the cracking behavior under static loading may be found elsewhere.[3,4] The failure crack patterns for Group 3 test models are shown in Figs. 8(a) to 8(c).

REPEATED LOADING TEST RESULTS AND DISCUSSION

Flexural Strength

The measured and predicted ultimate loads of the connecting beams are listed in Table 3. Similar to the static loading tests, the measured loads, P_u, of all the precast concrete models tested under repeated loading were greater than those of their monolithic counterparts. The reasons for the improved performance are the same as those responsible for the higher flexural strength of the precast models under static loading.

Table 2. Test results of specimens under static loading.

Group	Specimen	f'_c (MPa)	$P_{u,cal}$ (kN)	P_u (kN)	$\frac{P_u}{P_{u,cal}}$	Δ_y (mm)	Δ_u (mm)	$\frac{\Delta_u}{\Delta_y}$
1	SM1	30.26	37	39	1.05	8.07	21.06	2.61
	SA1	58.9 / 30.26	40	49	1.23	8.03	34.71	4.32
	SB1	58.9 / 30.26	41	52	1.15	5.74	18.71	3.26
2	SM2	30.2	52	52	1.00	7.05	15.02	2.13
	SA2	58.9 / 30.26	57	64	1.12	8.20	28.20	3.44
	SB2	58.9 / 30.26	57	66	1.14	9.1	22.38	2.46
3	SM3	13	27	27.8	1.030	10.08	23.88	2.37
	SA3	67 / 13	42	45.8	1.090	9.601	44.40	4.63
	SB3	65 / 13	43	35.33	0.820	9.55	20.65	2.16
4	SM4	53	40	44.9	1.120	8.611	28.42	3.30
	SA4	78 / 53	42	46.29	1.100	5.512	26.72	4.85
	SB4	60 / 37	50	54.71	1.090	9.61	28.89	3.01

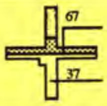
Note: 1 mm = 0.0394 in.; 1 MPa = 145 psi; 1 kN = 0.224 kips.
P_y = measured yield load
$P_{u,cal}$ = theoretical ultimate load
P_u = measured ultimate load
Δ_y = measured yield deflection
Δ_u = measured ultimate deflection

Deformation and Ductility

The ductility factors, Δ_u/Δ_y, of all the connecting beams are given in Table 3. By comparing Models RM2, RA2, and RB2, it is clear that both types of precast connections attained higher ductility than the monolithic specimens without strength degradation. However, for the models with a lower tensile steel content (Group 5), the monolithic connection (RM1) performed better than the precast models.

The load-deflection curves for the

Table 3. Test results of specimens under repeated loading.

Group	Specimen	f'_c (MPa)	P_y (kN)	P_u (kN)	$P_{u,cal}$ (kN)	$\frac{P_u}{P_{u,cal}}$	Δ_y (mm)	Δ_u (mm)	$\frac{\Delta_u}{\Delta_y}$	Energy absorption (kN-mm)
5	RM1		25	41	37.86	1.08	4.37	33.7	7.7	2832
	RA1		30	47.2	40.13	1.18	5.43	34.33	6.32	3275
	RB1		25	48	41.55	1.16	4.4	30.79	7.0	3061
6	RM2		40	59	53.86	1.1	6.63	38.06	5.74	4455
	RA2		45	68	57.86	1.18	7.04	41.15	5.85	6053
	RB2		44	70	59.11	1.18	6.6	40.66	6.16	5411

Note: 1 mm = 0.0394 in.; 1 MPa = 145 psi; 1 kN = 0.224 kips; 1 kN-mm = 0.00884 kips-in.
P_y = measured yield load
$P_{u,cal}$ = theoretical ultimate load
P_u = measured ultimate load
Δ_y = measured yield deflection
Δ_u = measured ultimate deflection

Group 5 models are shown in Figs. 9(a) to 9(c). From these curves and those for the Group 6 models,[4] the following can be observed:

- The load-deflection curves of the precast models are very similar to those of the monolithic ones. There was no premature failure occurring in any of the precast connections.
- Even though there was a residual deflection at the end of each cycle of loading, the flexural rigidity of each connection under subsequent loading was not significantly affected by the previous loading cycles.

Energy Absorption

The energy absorption capability of a beam-to-column connection may be taken as the area under the load-deflection curve. For all the specimens, the values of the cumulative energy absorbed are calculated. These values are listed in Table 3.

It is clear that both types of precast models performed better than the monolithic ones in absorbing energy. In addition, Type A models were superior to Type B models in this respect. This may be attributed to the fact that, for Type A connections, wider major cracks were developed at the connecting beam root (i.e., at the interface between the precast and cast-in-place concrete).

Cracking Behavior

The cracking and crack propagation characteristics of the connections under repeated loading are similar to those under static loading. A detailed discussion on crack patterns and failure modes is given elsewhere.[4]

CONCLUSIONS

Based on test results of 18 half-scale beam-to-column connection models, the following conclusions can be drawn:

1. Under both static and repeated loading, the precast connections attained a higher flexural strength than the monolithic connections.

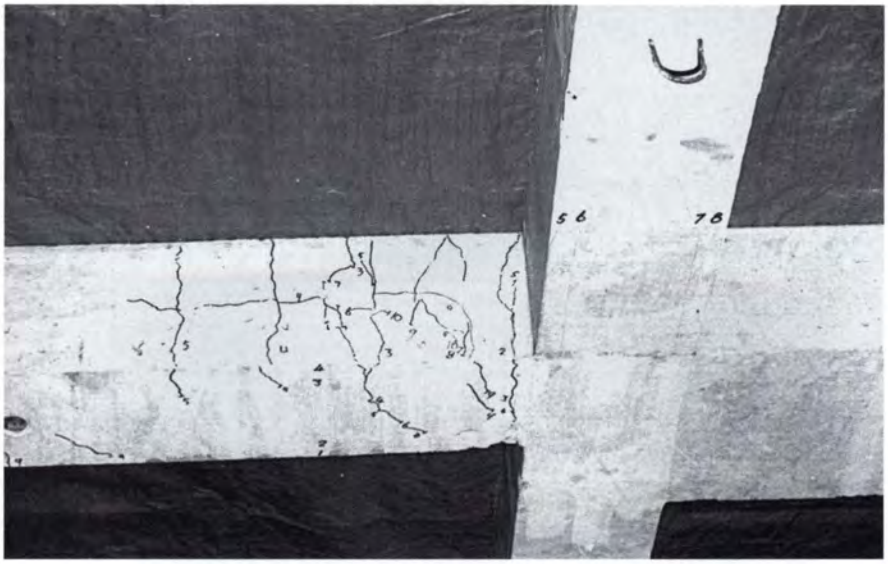
2. Under static loading, the ductility performance of Type B precast models is satisfactory when compared with that of the monolithic connections. In this respect, Type A connections are superior to Type B connections and the monolithic models.

3. Under repeated loading, the ductility characteristics of both types of precast connections are satisfactory, although Type B connections performed marginally better than Type A connections.

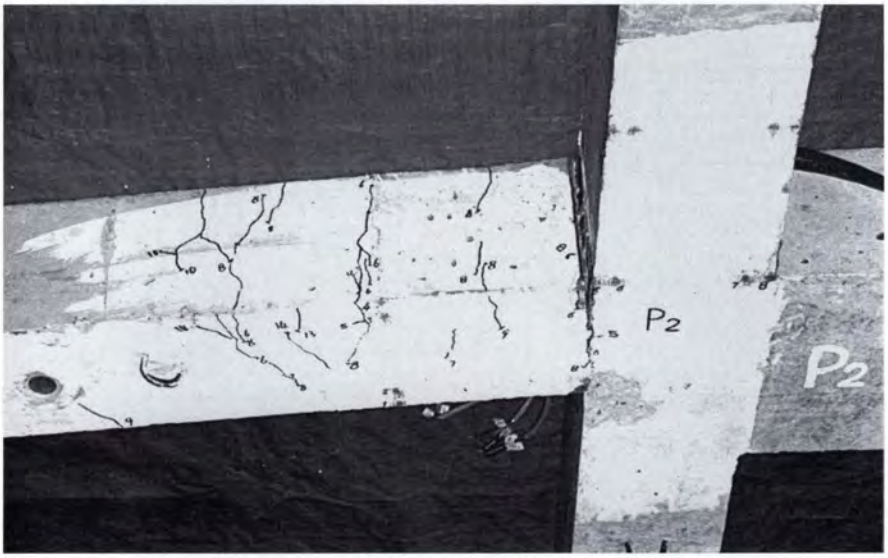
4. Both of the precast connection types, under repeated loading, possessed larger energy-absorbing capacities than the monolithic models.

RECOMMENDATIONS FOR FURTHER RESEARCH

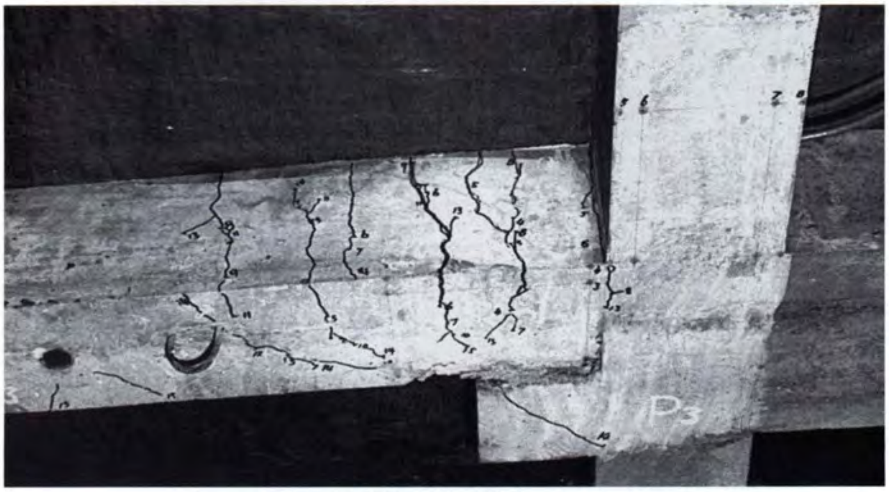
The present study indicates that the two types of precast connections are superior to their monolithic counterparts under static and unidirectional repeated loads. However, additional tests should be carried out under



(a) Specimen SM3

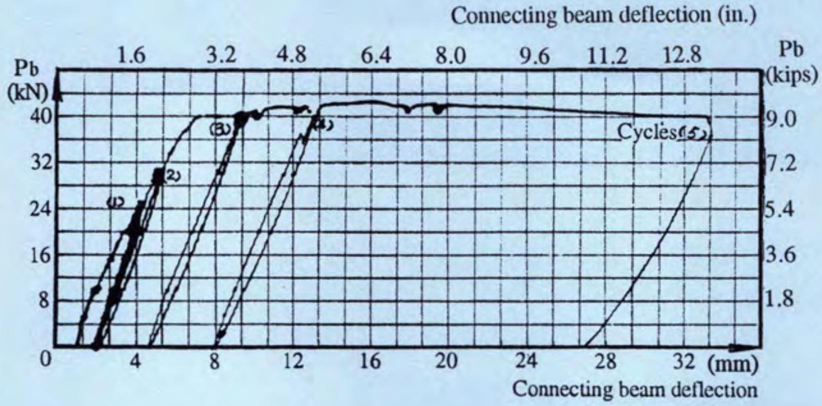


(b) Specimen SA3

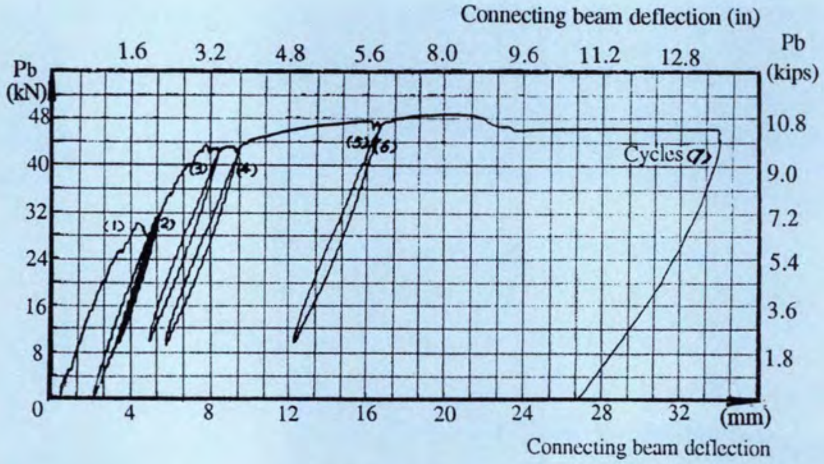


(c) Specimen SB3

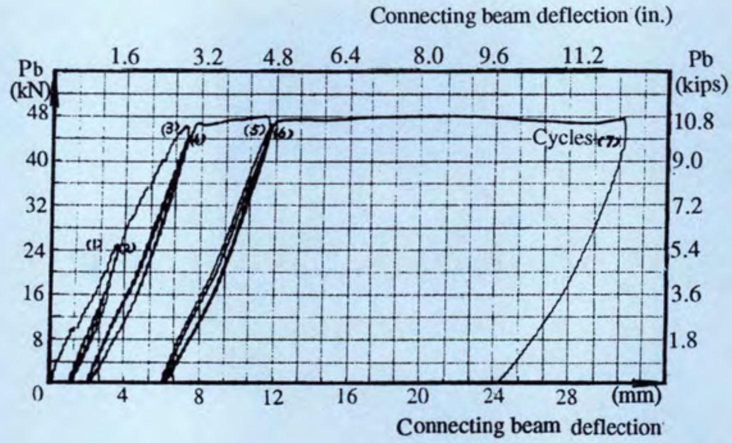
Fig. 8. Failure crack patterns for Group 3 models under static loading.



(a) Specimen RM1



(b) Specimen RA1



(c) Specimen RB1

Fig. 9. Load-deflection curves for Group 5 models under repeated loading.

cyclic loading conditions. This is particularly true for Type B connections where the welded assembly at the corbel is developed mainly to take compression. Cyclic loading causes tensile stresses that could lead to premature failure at the connection.

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