- Technical Paper -

EXPERIMENTAL STUDY ON THE SEISMIC PERFORMANCE OF ASSEMBLED PRECAST HIGH-STRENGTH CONCRETE BEAMS

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ABSTRACT

Seismic performance of assembled precast high-strength concrete beams in high-rise buildings is discussed. Various connections, located at beams mid-span zone, were considered. They were of various forms and mechanisms, and supposed suitable for construction sites. Experimental test results confirmed the adequacy of such assembles. The beams proved to be ductile and failure occurred outside the connection zone similarly to monolithic ordinary beams. Assumed mechanisms were also pictured for the evaluation of the strength of different connections.

Keywords: Precast beam, connection, high-strength concrete, seismic performance, seismic test

1. INTRODUCTION

Precast concrete frame construction has been increasing because of potential benefits in construction speed and quality control. However, when it comes to high-rise buildings in high-seismic regions, such full precast construction is not used extensively and cast-in-place concrete is still sharing an important amount. Reducing cast-in-place concrete amount can be accomplished by conceiving particular joints and connections that would reproduce similar properties of monolithic ordinary elements. Dynamic characteristics, strength and energy absorption capacity of such particular connections and joints, thus, should be demonstrated by technical and test data to be in accordance with code requirements. Worldwide, various concepts have been studied analytically and experimentally where connections were particularly considered at element ends.

An experimental program to examine the behavior of precast beams provided with different connections and subjected to lateral loads was initiated. The objective of the program was to develop recommendations for the design of such particular connections suitable for use in regions of high-seismic risk where the amount of cast-in-place concrete used for connections was replaced by a slight amount of high-strength mortar. The connection basic concept uses bolts or sleeved crossed-bars or sleeved straight bars to connect the parts of precast beams and provide the required shear resistance to the applied seismic, dead and live loads. The connection is achieved at the mid-span of the beams. Test results of some beams are discussed in the coming sections. Also, an equation based on test results, is suggested for the evaluation of the ultimate strength of each presented beam.

2. OUTLINE OF BEAMS AND CONNECTIONS

Four half-scale precast beams are presented herein. They represent interior shallow beams in high-rise buildings. All designed beams were aimed to have a flexural behavior. The detailing and main reinforcement were almost the same in all beams, except at the connection zone. Each beam was precast into two overlapping or head-to-head cantilevers. All connections were conceived at the cantilever-tip zone. Two beams had connections with transversal bolts. One beam had sleeved-bars connection and another had sleeved crossed-bars connection. All beams were assembled with a 10mm-gap between the cantilever parts. The gap and other voids were grouted with very high-strength cement mortar.

2.1 Connection with bolts (No.1-1 and No.1-2)

The two cantilever-parts of beams No.1-1 and No.1-2 were joined using two layers of 3 non-tensioned bolts. The bolts were fastened just by hands in a way to keep a constant 10mm-gap for grouting. Beam No.1-1 was composed of two symmetric parts of L-shape. Beam No.1-2 was composed of one part of U-shape and another part of T-shape. When connected, the arm of the T-shape part was inserted between the arms of the U-shape part. Fig.1 and Fig.2 illustrate the geometric characteristics and detailing of both beams.

2.2 Connection with sleeved straight bars (No.2-1)

The two cantilever-parts of beam No.2-1 were joined using 8 sleeves distributed on three layers. The sleeves were grouted first, and then the space at beam mid-span that contains the sleeves was filled with concrete. Fig.3 illustrates the geometric characteristics and detailing of beam No.2-1.

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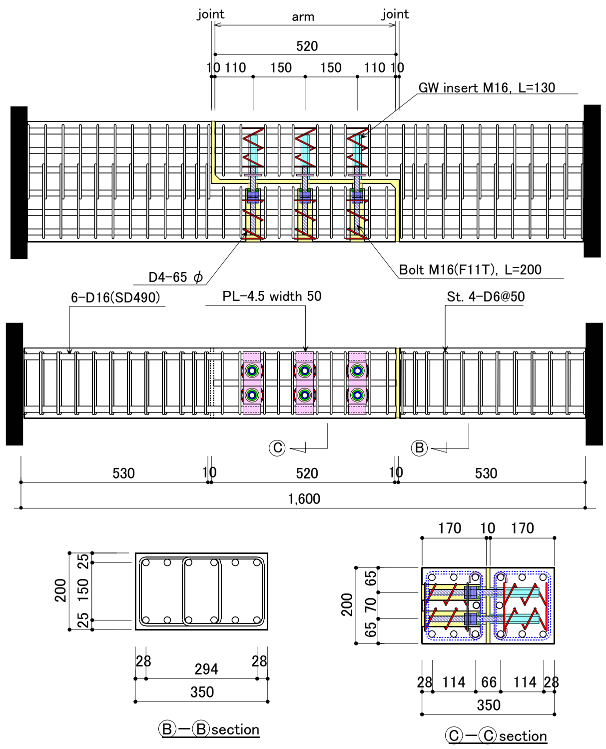

Fig.1 Outline of beam No.1-1

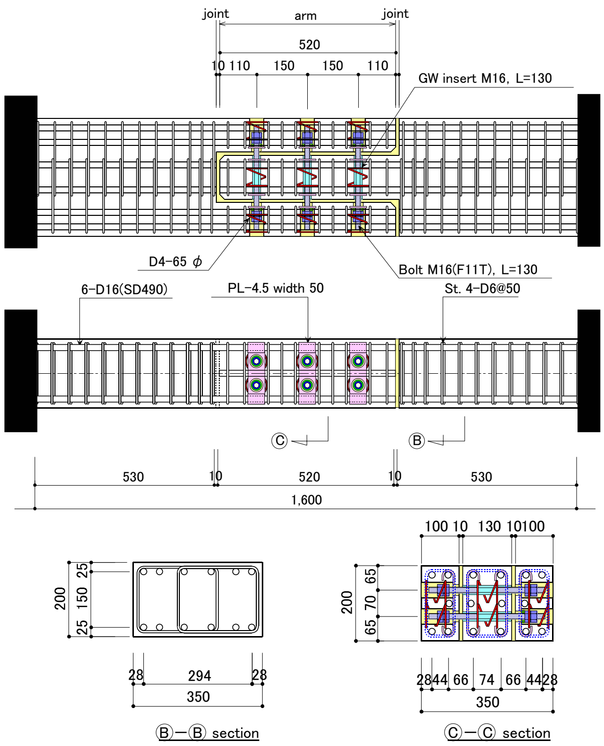

Fig.2 Outline of beam No.1-2

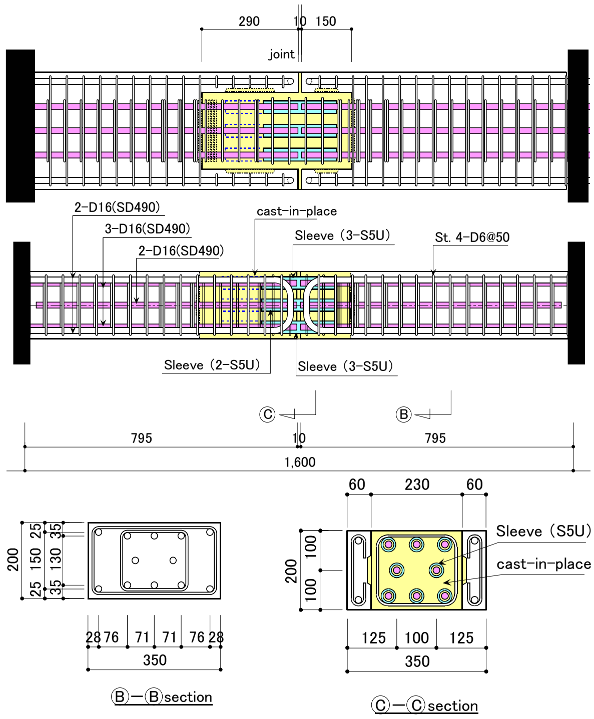

Fig.3 Outline of beam No.2-1

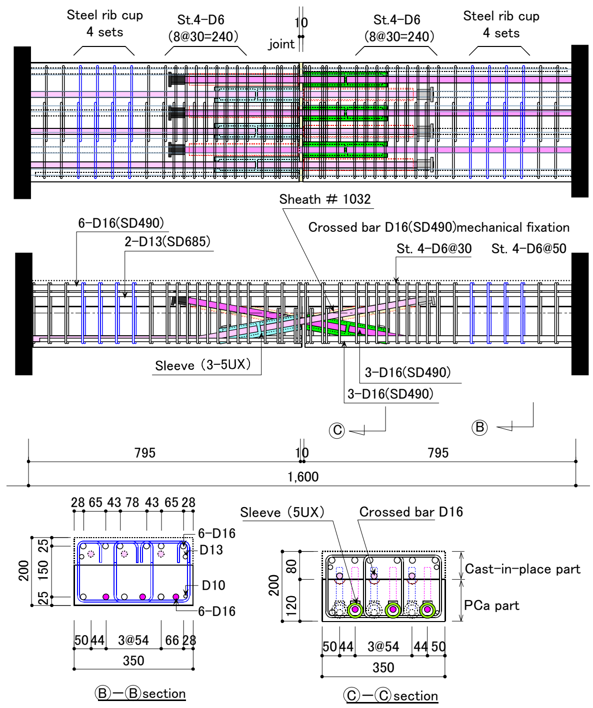

Fig.4 Outline of beam No.3-2

2.3 Connection with sleeved crossed-bars (No.3-2)

This beam was not fully precast. The lower part was first precast. The two precast cantilever-parts were joined using 6 sleeved headed bars. The sleeves were grouted first, then ducts and other voids were filled by mortar, finally, the upper part of beam was cast-in-place, covering the apparent crossed bars. Fig.4 illustrates the geometric characteristics and detailing of beam No.3-2.

3. TEST PROGRAM

3.1 Materials

(1) Reinforcing materials

Similar reinforcements were used in all beams. Longitudinally, D16 bars were used as main reinforcement, while transversally D6 stirrups were used with different arrangements for each beam. For detailing, some D13 bars were also added. As to the connection bolts, M16 bolts of F11T type (specific yield tensile strength equal to 1100 N/mm^2) and GW inserts (GW stands for the size and form) for M16 bolts were used. The properties of all reinforcements are shown in Table 1.

(2) Concrete

The specified concrete strength was 72MPa and the average compressive strength at 28 days was 86MPa. The specified cement mortar strengths were 80MPa and 120MPa, respectively, for grouting the gaps and sleeves, and the average compressive strengths at 28 days were 85MPa and 128MPa.

Table 1 Characteristics of reinforcements

Type	Elastic Modulus (kN/mm^2)	Tensile strength (N/mm^2)	Ultimate strength (N/mm^2)
D16 (SD490)	193	534	700
D13 (SD685)	190	665	913
D6 (SD785)	191	967	1199

3.2 Instruments and Loading Method

The beams were instrumented internally and externally. Strain gauges were installed on reinforcements at different locations, particularly, at hinge and joint zones. Displacement transducers were installed on beam faces to measure deformations at critical sections and connections.

As to loading, no load was applied axially. Laterally, all beams experienced the same loading protocol of an anti-symmetric double curvature bending. The beams were subjected to twice-repeated reverse-cycles with increasing amplitude, including some intermediary cycles of short amplitudes. The loading was displacement-controlled where the deflection angle amplitudes R were (by ratio of 1/1000): ±1.0, ±2.0, ±3.3, ±5.0, ±2.0, ±7.5, ±10.0, ±5.0, ±15.0, ±20.0, ±5.0, ±30.0, ±40.0 and ±50.0.

3.3 Estimated strength

The strength of each tested beam was evaluated according to AIJ standards and guidelines [1,2,3]. The flexural design yield strength [1] was evaluated simply by

$$Q_y = 2M_y/l, \qquad M_y = 0.9 a_t\, \sigma_y\, d \qquad (1)$$

where,

Q_y, M_y: yield shear force and yield moment
a_t : tensile reinforcement sectional area
σ_y : reinforcement design yield stress
d : beam section effective depth
l : beam length

As to the performance level of different connections, the strength was evaluated for both long-term and short-term loading conditions. The long-term loading of the connection zone was characterized mainly by the importance of the bending moment in the contrary to the short-term loading. The results of long term case would not be presented as long as the focus herein is on the short-term case. However, it is worth to mention that the evaluated flexural strength of the connections for long-term case proved to be suitable in the case of beam No.1-1 and beam No.3-2 for which long-term tests were conducted.

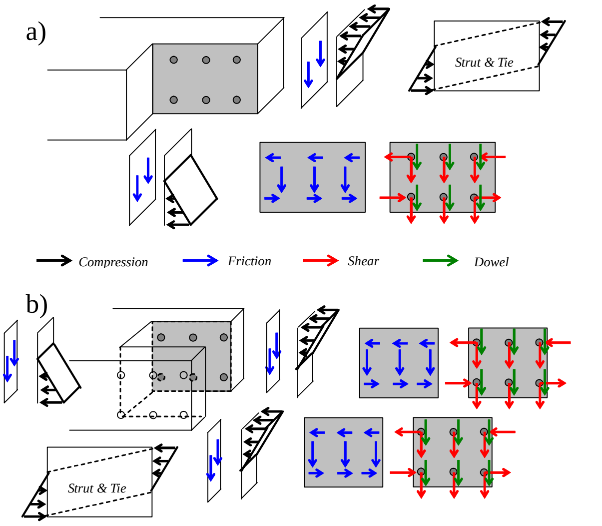

Fig.5 Assumed forces at connection of a one-arm beam No.1-1 (a) and a two-arm beam No.1-2 (b)

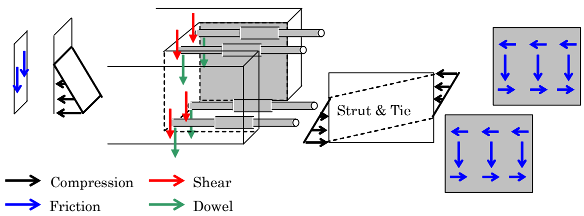

Fig.6 Assumed forces at connection of sleeved beam No.2-1

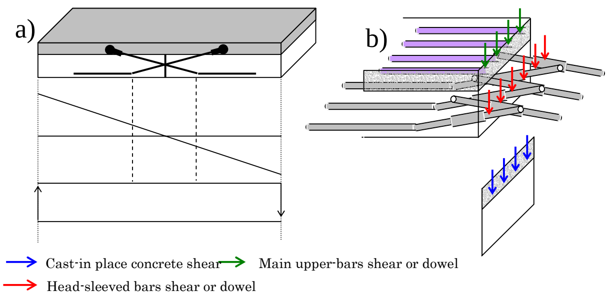

Fig.7 Moment and shear distribution (a), and assumed forces at connection of crossed-bars beam No.3-2 (b)

In this study, the strength of different connections was assumed developing proportionally through non-cumulative forces that are illustrated for each connection type in Fig.5, Fig.6 and Fig.7. Strength assumed coming from the strut-and-tie mechanism [2] that would develop along beam arms in beams No1-1 and No.1-2 or along the concrete middle bloc in beam No.2-1was evaluated by

$$V_u = b\, j_t\, \rho_w\, \sigma_{wy} \cot\phi + \tan\theta\,(1-\beta)\, b\, D\, \nu\, \sigma_B/2 \quad (2)$$

where,

b, D : beam section width and height
j_t : tensile reinforcement lever arm (=$7d/8$)

ρ_w, σ_{wy} : stirrups' ratio and design yield stress
ϕ : truss angle
$\tan\theta$: $[(x/D)^2+x]^{1/2}-x/D$
x : beam arm length
ν : concrete softening factor
σ_B : concrete compression strength
β : $(1+cot^2\phi)\ \rho_w\ \sigma_{wy}\ /\ (\nu\ \sigma_B)$

Shear strength Q_b [3] of bolts and their dowel action Q_d [3] were evaluated by

$$Q_b = n\ a_b\ \sigma_{by}\ /3^{1/3} \tag{3}$$
$$Q_d = n\ 1.65\ a_b\ [\sigma_B\ \sigma_{by}\ (1-\alpha^2)]^{1/2} \tag{4}$$

where,
n : number of bolts
a_b, σ_{by} : bolt section area and design yield stress
α : flexure ratio (0 for no combined flexure)

Shear strength Q_c [3] corresponding to crushing of concrete or grout around bolts, was evaluated by

$$Q_c = n\ \varphi_{S1}\ 0.5\ a_b\ [\sigma_B\ E_c\ ]^{1/2} \tag{5}$$

where,
a_b : bolt sectional area
φ_{S1} : load type factor (0.6 for short-term load)
E_c : concrete/grout elastic modulus

A cumulative strength was assumed for the crossed-bars connection beam. It was coming from the vertical component forces F_v of inclined bars and the ultimate shear strength of cast-in-place concrete F_c [2], as well as the dowel action of the upper longitudinal bars.

$$F_v = n\ a_s\ \sigma_{sy}\ \sin\gamma \tag{6}$$
$$F_c = 2\lambda\ b\ h\ \sigma_B^{1/2}\ /3 \tag{7}$$

where,
n : number of bars
a_s, σ_{sy} : bar section area and design yield stress
γ : bar inclination
λ : factor equal to 0.85
b, h : section width and height of concrete

Table 2 Evaluated beam connection strength (kN)

Beam	Q_y	$V_{u,b}$ $V_{u,a}$	Q_b	Q_d	Q_c
No.1-1	125.5	354.4 350.4	661.5	490.0	687.0
No.1-2	125.5	354.4 292.8	1322.9	980.0	1374.0
No.2-1	101.6	304.7 276.6	736.2	731.1	1434.4
No.3-2	125.5	304.7 621.9*	833.8[&]	908.8[$]	1669.2[#]

Note: $V_{u,b}$ = strut and tie model for the whole beam, $V_{u,a}$ = strut and tie model for the beam-arm or concrete block, * = included the vertical force component of inclined bars, shear force of upper longitudinal bars and concrete shear strength, [&] = included only shear strength of all steel bars, [$] = included only dowel force of all steel bars, [#] = included only concrete crushing strength around all steel bars

The evaluated strength of the tested beams is summarized in Table 2. Friction was not considered in the evaluation.

4. ELEMENT PERFROMANCE

All tested beams showed a very good performance where the behavior was completely of flexural type with a significant ductility. Shear force increased with increasing deflection angle and when yielding of longitudinal reinforcement was reached at beam ends the increase in shear strength became limited although the deflection angle increased consistently. No strength degradation was noticed and the ultimate level could not be reached. The dissipated energy also increased with cyclic loading amplitudes, and pinching, generally, was not present or slightly appeared at the end of loading or beyond deflection angle of 30/1000 for some beams.

4.1 Beam No.1-1

The tested beam experienced during loading different stages that are illustrated in Fig.8 where the force response related to the controlled deflection angle is presented. The beam mechanism was characterized by a flexural failure and truss action. First bending cracks at beam ends appeared followed by a crack at the joint at deflection angle 2/1000. Since then cracks appeared almost simultaneously in the connection zone and at the location where there was an abrupt change in the precast cross-section. Bending-shear cracks started at hinge zone soon after deflection angle 10/1000 followed by yielding of main bars at beam ends. Crushing of concrete was noticed after deflection angle 17/1000. Although many bending and bending-shear cracks developed at hinge zone, beyond deflection angle 30/1000 damage concentrated around the weak section, which corresponded to an abrupt change in the precast cross-section. Splitting of concrete near the joint was observed due to large axial deformation at the joint interface. Slight torsion along the arms was observed through the course of cracks and data records due to the grout deformation at the interface; however, at the end of testing, damage could be found neither on bolts nor on their surrounding grout. When the test was concluded and beam unloaded, it experienced a residual deflection angle of 24/1000. The damage undergone by the beam appears in Fig.9.

4.2 Beam No.1-2

The tested beam experienced during loading different stages that are illustrated in Fig.10 where the force response related to the controlled deflection angle is presented. The beam mechanism was characterized by a flexure failure and truss action. First, bending cracks at beam ends appeared followed by a crack at the joint at deflection angle 3.3/1000. Since then cracks appeared almost simultaneously at the location where there was an abrupt change in the precast cross-section for one-arm part or two-arm part. Later, after deflection angle 5/1000, cracks appeared in the connection zone. Bending-shear cracks started at hinge zone soon after deflection angle 7.5/1000 followed by yielding of main bars at beam ends soon after deflection angle 10/1000. Crushing of concrete was noticed after deflection angle 17/1000. Besides bending and bending-shear cracks that developed at hinge zone, damage concentrated, since deflection angle 15/1000, progressively around the weak section of one-arm part, which corresponded to an abrupt change in the precast cross-section and amplified considerably after deflection angle 30/1000. Splitting of concrete near the joint was observed due to large axial deformation at the joint interface. Slight torsion along the two-arm part arms was observed through the course of cracks and data records due to the grout deformation at the interface; however, at the end of testing, damage could be found neither on bolts nor on their surrounding grout. When the test was concluded and beam unloaded, it experienced a residual deflection angle of 23/1000. The damage undergone by the beam appears in Fig.11.

4.3 Beam No.2-1

The tested beam experienced during loading different stages that are illustrated in Fig.12 where the force response related to the controlled deflection angle is presented. The beam mechanism was characterized by a flexure failure and truss action. First, bending cracks at beam ends appeared followed by simultaneous cracks at the joint and at the small arms at deflection angle 2.5/1000. Yielding of main bars at beam ends occurred at deflection angle 10/1000, then soon after bending-shear cracks started at hinge zone. Crushing of concrete was noticed after deflection angle 16/1000. Due to the large deformation and very wide opening of the flexural cracks at the hinge zone, damage, though minor, was totally concentrated at beam ends. Concrete expansion due to shear was very limited and splitting was not observed. The connection seemed very rigid though a slight sliding of the concrete bloc containing the sleeves at large deflection angle when loading reached the final step. When the beam was unloaded, it experienced a residual deflection angle of 27/1000. The damage undergone by the beam appears in Fig.13.

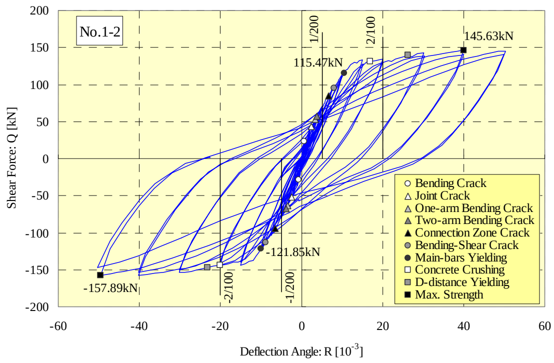

Fig.10 Load response of beam No.1-2

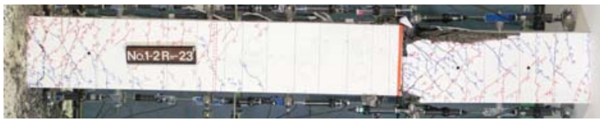

Fig.11 Final appearance of beam No.1-2

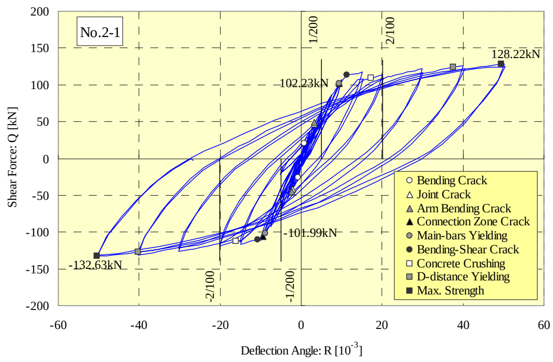

Fig.12 Load response of beam No.2-1

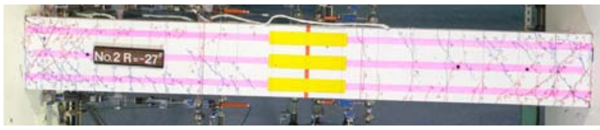

Fig.13 Final appearance of beam No.2-1

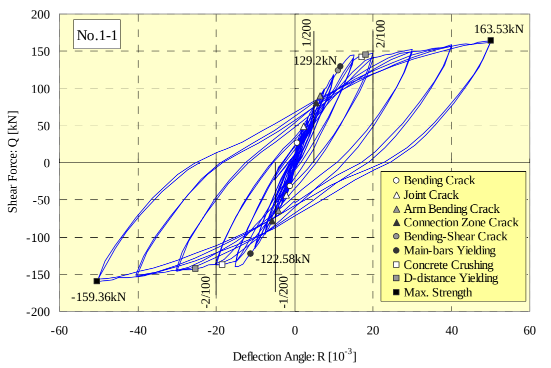

Fig.8 Load response of beam No.1-1

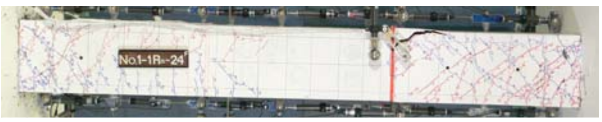

Fig.9 Final appearance of beam No.1-1

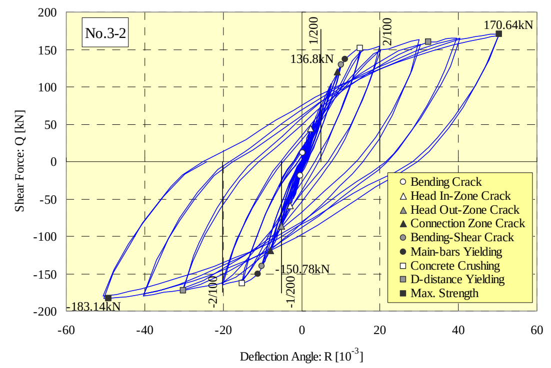

Fig.14 Load response of beam No.3-2

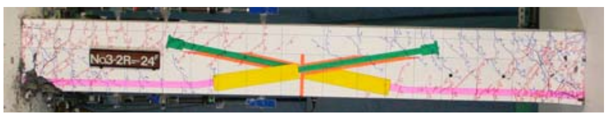

Fig.15 Final appearance of beam No.3-2

4.4 Beam No.3-2

The tested beam experienced during loading different stages that are illustrated in Fig.14 where the force response related to the controlled deflection angle is presented. The beam mechanism was characterized by a flexure failure and truss action. First, bending cracks at beam ends appeared followed by some cracks around the bars' head zone, and then a crack developed at the joint at deflection angle 10/1000. Bending-shear cracks started at hinge zone soon after deflection angle 10/1000 followed shortly by yielding of main bars at beam ends. Crushing of concrete was noticed at deflection angle 15/1000. Due to the large deformation and very wide opening of the flexural cracks at the hinge zone, damage was totally concentrated at beam ends. Concrete expansion due to shear was limited with some splitting occurring beyond deflection angle 30/1000. At the end of testing, no cracks could be seen between the concrete layers and no damage could be found around the connection zone. When the beam was unloaded, it experienced a residual deflection angle of 24/1000. The damage undergone by the beam appears in Fig.15.

4.5 Comparison of elements performances

From previous sections, the tested precast beams proved to be ductile and failure occurred outside the connection zone without considerable damage inside it. The behavior of the tested beams resembled to the behavior of similar ordinary beams. They did not experience any strength deterioration even at large deflection angles. The performance, in terms of strength, energy absorption, strains in reinforcement and damage concentration, was almost comparable for beam No.1-1 and beam No.3-2. The performance of beam No.2-1 was close to the performance of previous beams in all aspects. Its performance could be comparable to the other beams if the amount of longitudinal reinforcement was the same. The performance of beam No.1-2 was the lowest among all beams. The difference of precast sections at the one-arm part and two-arm part of the beam was the reason of the relatively weak performance when compared to other beams, and particularly to beam No.1-1. Finally, although test results confirmed the performance adequacy of the connections in beams No.1-1, No.2-1 and No.3-2, a preference would be toward the bolted beam No.1-1 due to the simplicity of the connection type, the reduced amount of cast-in-place concrete and short time of implementation, thus, resulting in a reduced construction cost.

The evaluation of beams' strengths did not bring full satisfaction, except for the yield strength that matched conveniently test results (Table 2 and Table 3). The large difference between test and evaluated results at connection zones seemed to be logic, as long as neither damage nor appearance of any failure mechanism could be observed.

Table 3 Test results and evaluated strength (kN)

Beam	$Q_{y,\,test}$	$V_{max\,,\,test}$	$Q_{y,\,cal.}$	$Q_{connection,\,cal.}$
No.1-1	129.2	163.5	125.5	350.4*
No.1-2	115.5	145.6	125.5	292.8*
No.2-1	102.2	128.2	101.6	276.6*
No.3-2	136.8	170.6	125.5	621.9*

Note: $Q_{y,test}$=test yield shear strength, $V_{max,test}$= maximum test shear strength, $Q_{y,cal}$=calculated yield shear strength, $Q_{connection,cal}$=calculated shear strength at connection, * = minimum of connection strength calculated values given in Table 2

5. CONCLUSIONS

The test carried out on assembled precast beams confirmed the adequacy of the selected assembles and connection types. The beams proved to be ductile, did not experience any strength deterioration even at large deflection angles and failure occurred outside the connection zone similarly to ordinary constructed beams. Their performance, in terms of strength-deflection angle relationship, energy absorption, strains in reinforcement and damage concentration, was relatively comparable. The abrupt variation of small cross-sections in beam No.1-2 with arms was the reason of the relatively weak performance when compared to other beams. Furthermore, although test results confirmed the performance adequacy of the connections in beams No.1-1, No.2-1 and No.3-2, a preference would be toward the bolted beam No.1-1 due to the simplicity of the connection, the reduced amount of cast-in-place concrete and short time of implementation, thus, resulting in a low erection cost.

The evaluation of the flexural strength of the assembled precast beams was appropriate. Therefore, the strength of the connections and the assumed mechanisms could not be checked as long as no particular damage or failure mechanism could be observed. The evaluated connection strength values were far higher than the maximum recorded strengths of beams.

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