프리캐스트 콘크리트 판구조의 비선형 해석

Nonlinear Analysis of Precast Concrete Wall Structures

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요 지

본 연구에서는 반복하중을 받는 프리캐스 콘크리트 판구조의 비선형 거동을 예측할 수 있는 해석방법을 제시하고자 한다. 프리캐스트 콘크리트 판은 탄성유한요소로 이상화하고, 벽판이 교차하는 접합부는 비선형 스프링요소로 모델링한다. 특히, 접합부에서 발생하는 전단, 압축과 인장거동을 묘사할 수 있도록 압축-인장 요소와 전단요소를 개발하고 각 스프링 요소의 강도와 강성은 기존연구자들에 의해 제시된 연구결과를 이용하여 구축한다. 구축된 모델을 비선형 해석프로그램인 DRAIN-2DX에 적용시켜 프리캐스트 콘크리트 판구조의 비선형 이력특성을 예측한다. 제안된 방법의 적합성을 평가하기 위하여 기존에 실험된 실험체를 대상으로 비선형 해석을 실시하고 그 결과를 비교하였으며, 그 결과 강도, 강성, 에너지 소산성능 및 횡변위 등에 대하여 실험결과와 해석결과가 좋은 대응을 보이는 것으로 나타났다. 이로부터 제안된 방법을 이용하여 대형콘크리트 판구조체의 비선형 이력특성을 적절히 예측할 수 있는 것으로 보여진다.

핵심용어 : 해석방법, 프리캐스트 콘크리트 판구조, 접합부, 비선형 스프링요소

Abstract

The objective of this paper is to propose an analysis technique to predict the behavior of PC wall structures subjected to cyclic load. While PC wall panel is idealized by finite elements, the joints at which PC walls are connected each other are idealized by nonlinear spring elements. Axial and shear spring elements are developed for simulating shear, compression and tension behaviors of joints. The strength and stiffness of each spring elements are presented from the previous research results and incorporated into the computer program of DRAIN-2DX. The proposed analysis technique is evaluated by analyzing specimens previously tested and comparing with those. On the strength, stiffness, energy dissipation and lateral drift, analytical results show good agreements with test results. This means the proposed technique is effective to predict the response of the PC wall structures.

Keywords : *analysis technique, PC wall structures, joints, nonlinear spring elements*

1. Introduction

The seismic behavior of Precast Concrete (PC) wall structures depends on the characteristics of both horizontal and vertical joints. It is generally accepted that earthquake-induced damage

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occurs in the joints, while the panels remain essentially elastic. The inelastic behavior of joint, thus, has to be suitably modeled in the analysis. The objective of this paper is to propose an analytical modeling technique to simulate the hysteretic behaviors of PC wall structures subjected to cyclic loads.

2. Numerical Modeling

The joints of PC wall structures are idealized as inelastic nonlinear springs in which the slippage, crushing, and rocking motion can be considered. Whereas wall panels are modeled using linear elastic finite elements.

2.1 Modeling Assumption

- Precast panels remain linear elastic.
- All nonlinear and inelastic behavior occurs in the connection regions.
- The diaphragm effects of slab are ignored except computing the elastic module of horizontal joints.
- The foundation is rigid.

2.2 Horizontal Joint

2.2.1 Compressive Strength

The normal compressive force in precast concrete wall structures is transmitted from an upper wall to a lower one through the horizontal joint which consists of floor slab panels and mortar as shown in Fig.1. Transmission of compressive force in precast concrete, thus, is effected by mortar or cast-in-situ concrete layers. An experimental result shows that the failure pattern of horizontal joint of PC wall structure can be classified as failure of the joint and the wall. When the compressive strength of joint mortar is less than that of PC elements, the compressive strength of horizontal joint is

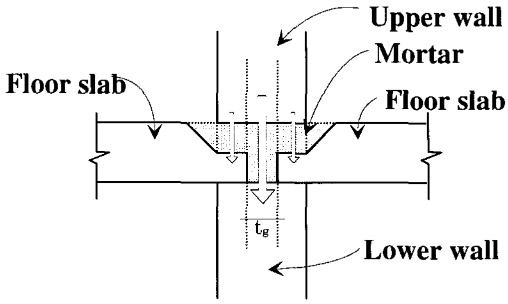


Fig. 1 Compressive force transmission in horizontal joint

governed by the compressive failure of joint, otherwise by the splitting failure of joint mortar. This concept is well considered in the formulas proposed by Seo (1994) as shown in Eq. (1) to (3). The formulas contain the parameters, such as the compressive strength ratio between joint and PC panels, grout width in joint. In this paper, those equation are used to predict the compressive strength of horizontal joint.

$$P_U = \phi f_{gh} t_g L \frac{1}{K}$$

$$(f_{gh} \le f_s \text{ : compressive failure of joint}) \quad (1)$$

$$P_U = \phi \frac{f_s \times t \times L}{5(1 - t_g/t)} \frac{1}{K}$$

$$(f_{gh} > f_s \text{ : splitting failure of walls}) \quad (2)$$

$$K = \frac{1}{(\frac{f_s}{f_{gh}})[(\frac{t}{t_g}) - 1] + 1} \le 1 \quad (3)$$

where, P_u : ultimate compressive strength of horizontal joint, ϕ : strength factor, f_{gh} and f_s: compressive strength of mortar and PC element respectively, t_g : width between slabs, L : length of horizontal joint, t : thickness of wall.

The compressive force-deformation curve of horizontal joint has been well defined by Karsen (Bangash, 1984) as shown in Fig. 2. In this

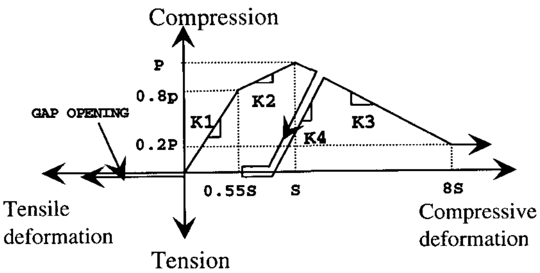


Fig. 2 Compressive spring element

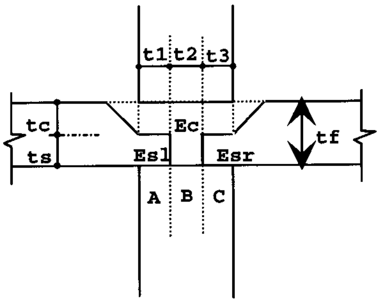


Fig. 3 Elastic module of compression joint

study, the curve is used for model of compressive spring since the detail of the horizontal joint will be analyzed is the same to the one on which Karsen applied.

In the Fig. 2, the stiffness of horizontal joint subjected to compression can be calculated by using effective elastic module, $E_{c,eff}$, as shown in Eq.(4). Also K_2 and K_3 can be obtained by knowing K_1. The unloading stiffness, K_4, is generally considered as the value same to K_1.

$$K_1 = \frac{E_{c,eff}\,A}{L} \tag{4}$$

where $E_{c,eff}$; effective elastic module, A ; area of wall section, L ; length of horizontal joint.

The elastic module of horizontal joint subjected to compression force should be evaluated in order to take in the effect of drying shrinkage and diaphragm in analysis. Eq. (5) shows a formula considering the those effects suggested by Lewicki (1985).

$$E_{c\ eff.} = \frac{t_1 + t_2 + t_3}{\frac{t_1}{E_A} + \frac{t_2}{E_B} + \frac{t_3}{E_C}} \tag{5}$$

$$E_A = E_C = \frac{t_c + t_s}{\frac{t_c}{(2/3)E_C} + \frac{t_s}{E_{sl*}}} \tag{6}$$

$$E_B = \frac{2}{3} E_C \tag{7}$$

In Eq.(6) and Eq.(7), the coefficient 2/3 indicates the effect of drying shrinkage. The elastic module of horizontal joint is increased since the diaphragm effect of floor slabs confines the joint. Eq.(8) shows the increased elastic module, E_{sl*} suggested by Lewicki (1985).

$$E_{sl*} = \frac{t_1 + 0.25t_f}{t_1} E_{sl} \tag{8}$$

where E_{sl} ; elastic module of PC slab element

2.2.2 Shear Slip

The shear resistance of the horizontal joint has been modeled by nonlinear springs with the elasto-plastic hysteretic curves as shown in Fig. 4. The shear behavior of the joint is generally explained by the shear friction mechanism as shown in Eq.(9). In this study, the friction coefficient, μ, is assumed as 0.4 like that Brankov (1977) did for cyclic loads. The bond force of concrete is ignored.

$$V_S = C + \mu(N + A_{sh} f_{yh}) \tag{9}$$

where, V_S : shear force, C : bond force, N : axial force, μ : friction coefficient, A_{sh} and f_{yh} ; area and yield strength, respectively of vertical reinforcement in horizontal joint.

To find exact shear stiffness of the horizontal shear joint is not easy since the stiffness varies

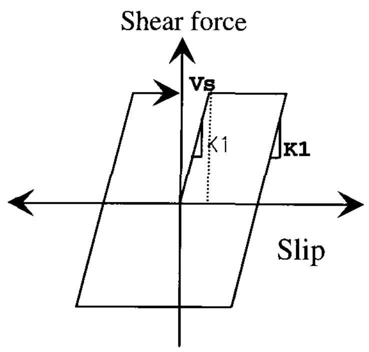


Fig. 4 Shear slip of spring in horizontal joint

along the length. However, in CEB-FIP model code (1990), the average shear stiffness of horizontal shear joint for PC wall structure is suggested for analysis as Eq. (10). This formula can be applied for calculating the shear stiffness of horizontal joint.

$$K_s = 50\frac{V_S}{A_j} \qquad (10)$$

where A_j : area of horizontal joint

2.3 Vertical Joint

Vertical joints of PC wall structures are designed to connect horizontally the walls and to resist the vertical deflection of the walls. Generally the keyed edges are often used on vertical connections to increase shear resistance through interlocking of the keys. The connection fails by shearing or crushing of the keys, or diagonal tension cracks in the joint mortar. The residual shear resistance after failure of the keys depends on both the friction of keys and the dowel action of the reinforcing bar crossing the joints. While the addition of reinforcement across a keyed joint greatly improves its ductility after failure of the keys, the amount and detail of reinforcement increase the residual shear resistance.

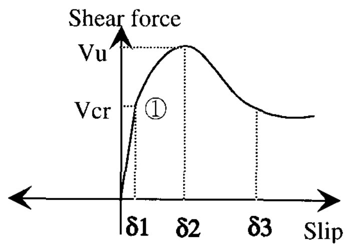


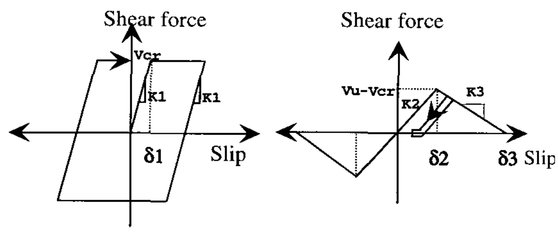
Fig. 5 Force-displacement curve of keyed joint

Table 1 Slippage of the shear key

	δ_1	δ_2	δ_3
shear slip (mm)	0.05~0.07	0.5~1.5	5

Fig. 5 shows the force-displacement relationship of the vertical joint. Initially the behavior of the vertical joint is on elastic status. With the increasing of the slippage, the shear strength goes up to the V_u, however, at beyond V_u, the strength decreases. These relationship between shear force and slippage has been well suggested by Matsuzaki (1989) as shown in Fig. 5 and Table 1. In this study, the slippages of vertical joint at yield, ultimate and failure are idealized using the values of Table 1.

Fig. 6 shows the hysteretic curve of the vertical joints for shear under the cyclic loads. This can be idealized as a nonlinear spring with the simple friction element and shear key element. The maximum value of V is usually considered to be equivalent to the ultimate shear resistance of joint.



(a) Simple friction element (b) Shear key element

Fig. 6 Shear slip of spring in the vertical joint

The formulas suggested by Seo (1994) are used in this paper to compute the ultimate shear strength of vertical joint. Eq.(11) to Eq.(13) show those equations.

$$V_{cr} = 0.09 A_k f_{gv} \tag{11}$$

$$\eta = \frac{A_{sv}\, f_{yv}}{A_k\, f_{gv}} < 0.097$$

$$V_u = \phi\left(A_k f_{gv} \sqrt{(0.494 - 0.955\,\eta)\,\eta\, 0.955}\right) \tag{12}$$

$$\eta = \frac{A_{sv}\, f_{yv}}{A_k\, f_{gv}} \geq 0.097$$

$$V_u = \phi(0.119\, A_k f_{gv} + 0.764\, A_{sv} f_{yv}) \tag{13}$$

$$f_{yv} \leq 2400\, kg/cm^2$$

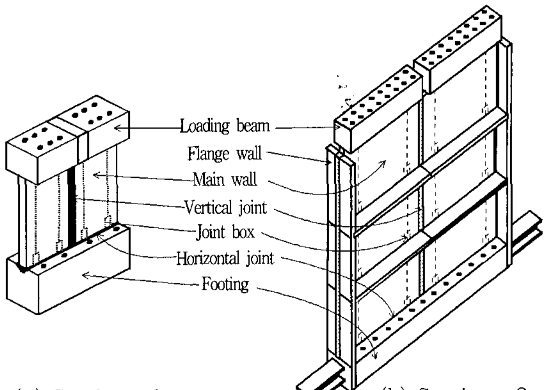
where V_{cr} ; shear strength at crack, V_u ; ultimate shear strength, A_{sv} and f_{yv} ; area and yield strength, respectively, of shear reinforcement in vertical joint. A_k ; area of shear key, f_{gv} ; compressive strength of mortar in vertical joint.

2.4 Panel Idealization

The panel is idealized using plane stress element with four node and eight degree of freedom in this paper. In many analyses, it may be reasonable to idealize a panel as a single structural element in which the overall extensional, flexural, and shear stiffness of the panel are modelled. The panel is assumed to have three uncoupled primary modes of deformation. The element stiffness is defined by its rigidity in extension, bending, and shear, respectively.

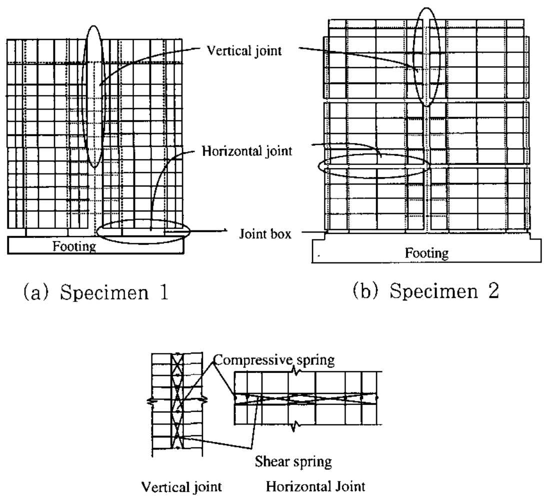
3. Comparison of Results

To evaluate the effectiveness of the modeling technique proposed in this paper, the analytical results are compared with the experimental results (Seo et al., 1992; Park et al., 1994). The test specimens are 1/2 scaled PC wall subassemblages which represent scaled-down models of the lower part of a 24 story PC wall. PC panels of the specimens are connected horizontally by shear key and loop bar, linked vertically by joint box which is made of steel plates. The layout of test specimens are as shown in Fig. 7.



(a) Specimen 1 (b) Specimen 2

Fig. 7 Layout of test specimens



(a) Specimen 1 (b) Specimen 2

(c) Idealized joint

Fig. 8 Modeling of specimen

Lateral force is loaded cyclically to the test specimens subjected to constant axial loads. Three story specimen failed at the horizontal

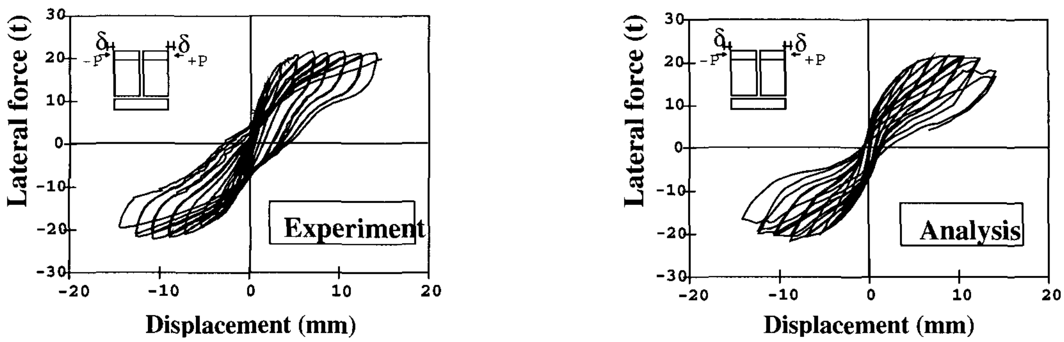


Fig. 9 Load-displacement curve of specimen 1

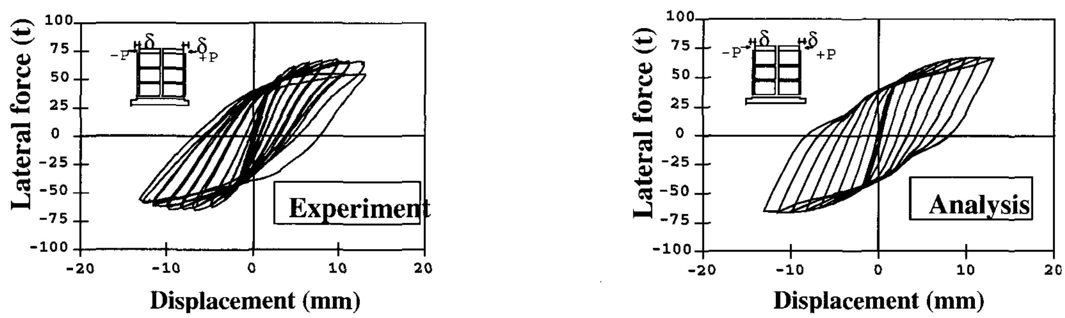


Fig. 10 Load-displacement curve of specimen 2

Table 2 Comparison of test results

Content		$1\delta_y$			$3\delta_y$			$5\delta_y$			$7\delta_y$		
		(1)	(2)	(1)/(2)	(1)	(2)	(1)/(2)	(1)	(2)	(1)/(2)	(1)	(2)	(1)/(2)
Specimen 1	Strength(t)	12.5	11.8	1.06	20.6	20.1	1.02	21.1	21.2	0.99	21.1	21.0	1.01
	Stiffness(t/cm)	79.1	77.1	1.03	38.0	36.6	1.04	24.4	26.0	0.94	17.0	16.8	1.02
	Energy(t. mm)	12.0	6.0	1.50	75.9	61.0	1.26	144.3	117.0	1.23	228.7	160.1	1.43
Specimen 2	Strength(t)	38.6	38.5	1.00	61.3	58.0	1.06	66.6	62.4	1.07	66.7	66.6	1.00
	Stiffness(t/cm)	236.5	223.9	1.06	131.2	124.3	1.06	83.7	81.5	1.03	60.3	59.0	1.02
	Energy(t. mm)	19.0	40.0	0.50	219.0	347.5	0.63	627.4	764.7	0.82	965.2	1214.0	0.80
(1) Result of experiment, (2) Result of analysis													

and vertical joints with diagonal compression cracks in walls, whereas the failures of one story model was concentrated at the horizontal joints with rocking motions.

The load-displacement curves of analytical results are compared with those of the experimental results as shown in from Fig. 9 to Fig. 10. Analytical results are in a good agreement with those of the experimental results. Table 2 shows the maximum strength, stiffness and energy dissipation of each cycle on test models. Even though the energy dissipation by analysis is little bit far from that of experimental results, the differences between the analytical and experimental results on the strength and stiffness are less than 7%.

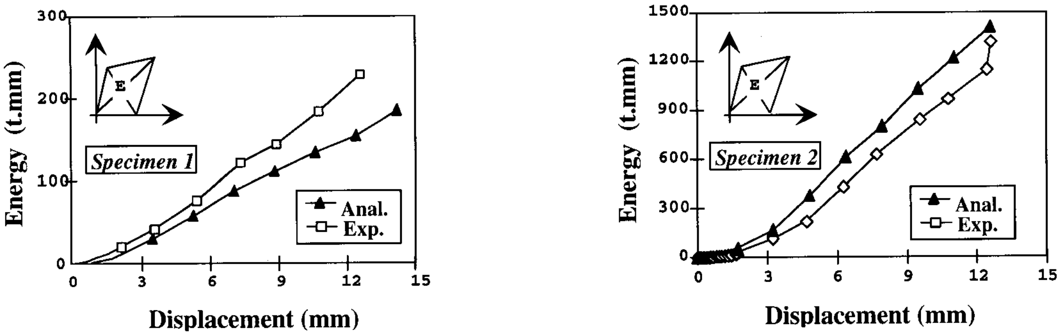


Fig. 11 Energy-displacement curve

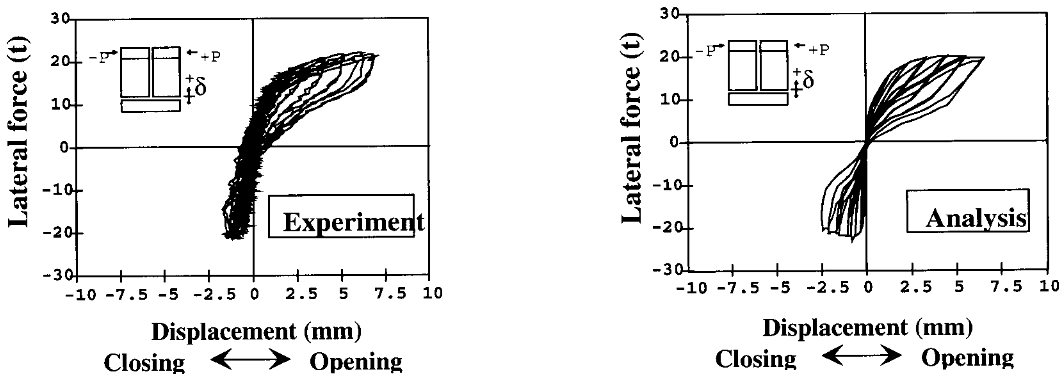


Fig. 12 Rocking displacement of specimen 1

4. Conclusion

An analytical modeling technique is proposed to predict the hysteretic behavior of PC wall structures subjected to cyclic loads. The effectiveness of the modeling technique was evaluated and it was shown that inelastic analysis using the proposed modeling technique was useful to predict the nonlinear response of the PC wall structures subjected to cyclic loads. However, in order to apply the proposed analysis technique to evaluating the nonlinear response of a building, works for modeling many spring elements are still required. For this, more simple technique is necessary to be developed.

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