LOCALIZED CLADDING RESPONSE AND IMPLICATIONS FOR SEISMIC DESIGN

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SUMMARY

Overall and localized response models for cladding are presented in a study of the potential influence of heavy precast concrete cladding on the dynamic properties and seismic response of a medium highrise office building. Cladding was found to alter interstory drift and framing member forces in the supporting framework. Current recommendations for isolation of cladding through use of slotted connections may not perform as intended.

INTRODUCTION

Claddings on highrise buildings form a first line of defense for the structure against environmental loadings such as wind or earthquake. However, building designers usually treat the curtain wall as nonstructural, and often leave the choice of cladding and its connections to the architect and cladding manufacturer. Aesthetic considerations usually play a major role in selection of a building's facade, but the potential structural stiffening effect of heavyweight claddings (eg., precast concrete, brick, or granite) must be studied, as well, for lateral motions.

A number of previous studies (Refs. 1-6) have suggested that cladding may be used as a participating structural element for control of wind and earthquake motions in modern highrise construction, resulting in substantial economies. However, the prevailing design philosophy at present in the United States, as expressed in the design recommendations of the Prestressed Concrete Institute (7), is to isolate precast panels from interstory drift motions. Slots and oversized holes in clip angle connections are suggested as ways to limit force transfer into panels, but sliding connections may be rendered ineffective by poor construction practice and lack of inspection, or connection deterioration with time.

The principal objective of the present study was to investigate the possible role which heavy claddings might play in the lateral response of a medium highrise office building for moderate earthquake ground motions. First, an overall response model for the facade is presented in which the cladding stiffness between stories is represented as a piecewise linear

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force-drift relationship. This model is used to study overall building interstory drift response to ground motion. Then, a localized response model for a small portion of the facade is studied to determine framing member and panel connection force levels for code-specified drift, and the implications for design of cladding are investigated.

Background

In previous studies, the authors have presented linear (1,2), incremental failure (3), and hysteresis (8,9) models for precast cladding to account for the lateral stiffening contribution of the precast concrete facade on a 24-story steel frame building of core construction. Plan and elevation views of the structure and a portion of the facade are presented in Figs. 1 and 2. The cladding, consisting of highly contoured precast concrete vision panels (12 ft. x 5 ft. x 1.5 ft., 3.7 m x 1.5 m x 0.5 m) (Fig. 2), is supported by a light steel framework on the exterior of the structure. Four wedge-insert-type connections per panel are used to fasten the cladding to the supporting spandrel members.

Previous models for cladding originated with the linear model in which the cladding was represented as a tridiagonal stiffness matrix (see Ref. 9). Cladding stiffness, expressed in terms of an interstory shear stiffness parameter V, was added to that of the core frames and exterior framing supporting the facade. By comparing both ambient and forced vibration test results, from field tests conducted at the site of the case study building, with analytical values from the three dimensional finite element model of the building, the value of $V = 625$ kips/inch (1.1×10^5 kN/m) was determined. Parameter V represents the interstory shear stiffness of the row of 12 panels and their connections on one face of the building between any two successive floor levels.

In the present study, a slotted connection model is introduced to represent the design case in which slots are typically specified in clip angle attachments to isolate cladding panels from lateral building distortions. The efficacy of slotted connections in protecting brittle facade components is explored below.

SLOTTED CONNECTION MODEL

The Prestressed Concrete Institute (PCI) recommendations (7) for support of nonstructural cladding panels are summarized in Fig. 3. As will be shown in the localized cladding response studies below, this arrangement of slotted connections does not completely isolate the panels from lateral drift motions as intended. Therefore, a modified PCI support conditions case, referred to as the slotted connection model, was conceived and is shown in Fig. 4. The slotted connection model was implemented in the form of a piecewise-linear force-drift plot (Fig. 5) for the row of cladding panels between story levels on one face of the building in place of the linear interstory drift model described above. The support conditions depicted in Fig. 4 were felt to match PCI intended behavior more closely than the PCI recommended support conditions, since force transfer into panels can be drastically reduced at low interstory displacement levels. When relative interstory drift was less than the allowable drift, Δ, a small stiffness, arbitrarily set at one-tenth of the full interstory shear stiffness (V = 625 kips/inch = 1.1 x 10^5 kN/m) of an entire row of panels at a story, or V = 62.5 kips/inch (1.1 x 10^4 kN/m), was provided to account for friction in the panel connections. The full interstory shear stiffness of V = 625 kips/inch (1.1 x 10^5 kN/m) was employed when the interstory drift exceeded Δ. No failure of panels or connections was assumed in this model and piecewise-linear elastic behavior was assumed throughout. A 5% eccentricity was specified between centers of mass and rigidy to produce an eccentric mass model for these studies in accordance with UBC provisions (10).

The equations of motion were integrated step-by-step using direct linear extrapolation with the trapezoidal rule. The force-deformation relationship for the cladding was updated at each story and on each face of the structure at the end of each time step on the basis of story drift values on each face.

Response Studies

Five allowable interstory drift (Δ) conditions were considered: (1) Δ=0 (linear case, V = 625 kips/inch = 1.1 x 10^5 kN/m; (2) Δ=0.18 in (0.5 cm, 0.125% h, h=story height); (3) Δ=0.36 in (0.9 cm, 0.25% h); (4) Δ=0.72 in (1.8 cm, 0.5% h); (5) Δ=∞ (unclad case, V=0). Peak interstory drift are summarized in Table 1 for 14 seconds of the 1940 El Centro ground motion, N-S component, input in each of the structure directions. Story drift envelopes for conditions (1) and (4) above are presented in Figs. 6 and 7 for ground motion input in the rigid frame and braced frame structure directions (see Fig. 1), respectively, for each structure face.

In general, the above results demonstrate that the performance of the structure model is distinctly different when cladding effects are included. For example, in Fig. 6 the peak story drift values for face 4 (see Fig. 1), which is parallel to the direction of ground motion, are substantially reduced when the slotted connection model is used. However, in the braced direction (Fig. 7), peak drift values are increased for the slotted connection model compared to the linear case (i.e., condition (1) above). Hence, in the rigid direction, structure dynamic properties are altered sufficiently by addition of cladding to reduce overall structure sensitivity to this particular ground motion loading, while the reverse is true for the braced direction. Response values for intermediate drift cases (i.e., conditions 2, 3 and 4 above) are bracketed by the fully clad (condition 1) and unclad (condition 5) cases. Finally, inspection of Table 1 values demonstrates that, in all cases, peak interstory drifts on faces parallel to the direction of ground motion exceeded allowable drift values. As a result, for the earthquake loading considered, connection slot dimensions must also be greater than allowable drifts if complete isolation of panels is to be accomplished.

LOCALIZED RESPONSE MODEL

Localized cladding response was investigated with the frame-panel model of Fig. 8 in which panels were considered flat and perfectly rigid, spandrel and column framing was linearly elastic, and panel-to-frame clip angle connectors were modeled as piecewise-linearly elastic springs. Panel-to-panel contact was neglected but additional spring supports (referred to as continuity springs) were positioned at connection points on the top and bottom spandrels to simulate the effect of panels in stories above and below. Spandrel-to-column connections were AISC Type 2 and were treated as semi-rigid (rotational stiffness = 8 x 10^4 kip-in/radian, 9 x 10^3 kN-m/radian). Spandrels were W18x35 and columns W10x49 steel shapes (A36).

The lateral stiffness of the panel-frame system, defined as V, was computed by imposing lateral displacements at top corner joints in Fig. 8 and determining the required reaction forces at these joints. The interstory shear stiffness of the unclad frame (V = 11.53 kips/inch, 2.0 x 10^3 kN/m), determined in a separate analysis, was deducted from that of the clad frame to obtain the interstory shear stiffness contribution of cladding panels and connections only. The connection spring stiffnesses were adjusted in steps and cladding lateral stiffness compared to the value of 625 kips/inch (1.1 x 10^5 kN/m) obtained for a row of panels on one structure face only. A row of panels on one face corresponds to 12 bays of cladding resulting in a cladding lateral stiffness value of 52.08 kips/inch (9121 kN/m) per bay. A best match was obtained when connection stiffnesses were set at $K_x = K_y = 360$ kips/inch (6.3 x 10^4 kN/m) which yielded V = 63.69 - 11.53 = 52.16 kips/inch (9134 kN/m). For continuity springs, K_y = 360 kips/in.

An interstory drift of 0.72 in (1.8 cm), as required by UBC (10), was applied to the panel-frame model. This drift value represents approximately 55% of the peak drift computed for the 1940 El Centro NS record applied to the overall structure model. Maximum connection (spring) forces and framing member forces were computed and are compared in Table 2 for the uniform spring (i.e., all connection spring stiffnesses = 360 kip/in, 6.3 x 10^4 kN/m) and PCI support condition (Fig. 3) cases. Connection springs in the model which corresponded to directions free to slide in Fig. 3 were given very low stiffness values.

The PCI support condition case with connection stiffnesses of 360 kips/inch (6.3 x 10^4 kN/m) in the non-sliding directions resulted in an interstory shear stiffness of V = 248 kips/inch (4.3 x 10^4 kN/m). Apparently, the lateral stiffness contribution of cladding is not totally negated by the support system of Fig. 3. When this more flexible PCI case was compared to the uniform spring stiffness case, most values were comparable or slightly lower except in the case where sliding was permitted. Computed moments and shears in the frame members due to drift only were found to be well within their capacities, computed on the basis of AISC-specified allowable stresses, for the loading considered, but may be excessive if combined with other loading effects. Connection force levels were observed in all instances to either approach or considerably exceed the ultimate capacity of 5 to 15 kips (22.2 to 66.7 kN) typical of cladding panel connections (7).

Finally, Table 2 also contains maximum connection and framing force values for the modified PCI or slotted connection model case shown in Fig. 4. Now, V is reduced to almost zero and all connection forces due to interstory drift are predicted to be zero. Spandrel shear is substantially reduced while column and spandrel moments are only slightly affected.

CONCLUSIONS

The above studies were performed to determine the effects of cladding on overall structure lateral stiffness, on interstory drift due to earthquake ground motion, and on connection force levels in a frame-panel system subjected to interstory displacements. Results of the studies of overall structure response using the slotted connection model demonstrated that cladding stiffness can alter peak interstory drift values substantially for selected ground motion cases. For the localized response model investigation, the above results showed that the modified rather than the PCI recommended support conditions were best suited for use when connection forces must be kept at low levels for interstory racking of brittle facade components. Further studies of heavy cladding, including laboratory tests of panels and connections, will lead to refinements in the cladding models and to an improved understanding of force transfer to cladding due to interstory floor motions. Ultimately, studies of cladding behavior are expected to result in better procedures for design of panels and connections and improved performance of cladding in earthquakes.

ACKNOWLEDGEMENTS

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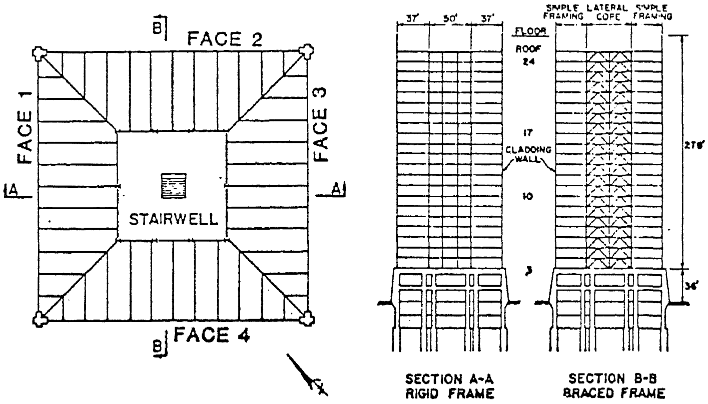


Fig. 1 Plan and Structural Elevations

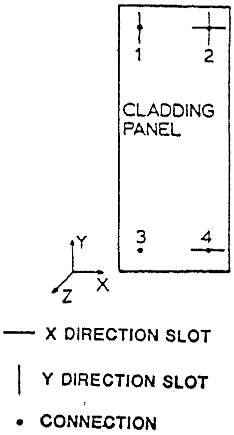


Fig. 3 PCI Panel Support Conditions

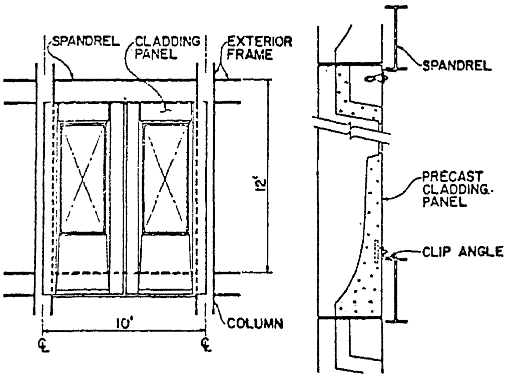
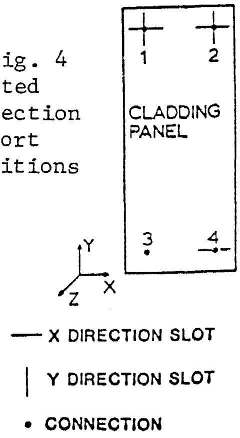


Fig. 2 Precast Panels and Connections

Fig. 4 Slotted Connection Support Conditions



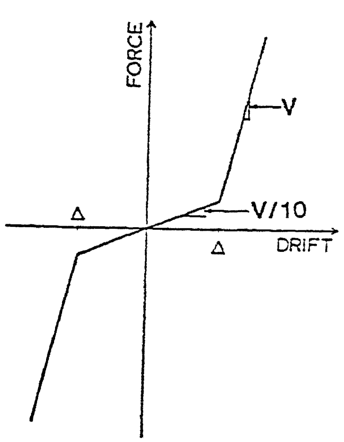


Fig. 5 Force-Drift Relation for Slotted Connection Model

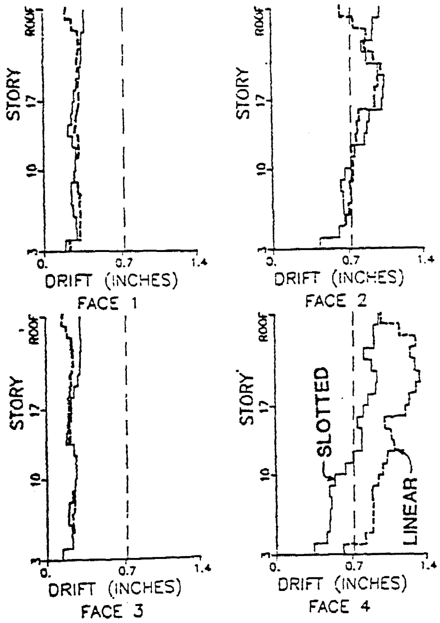


Fig. 6 Peak Story Drift Envelopes for 1940 El Centro NS in Rigid Direction (parallel to Faces 2 and 4)

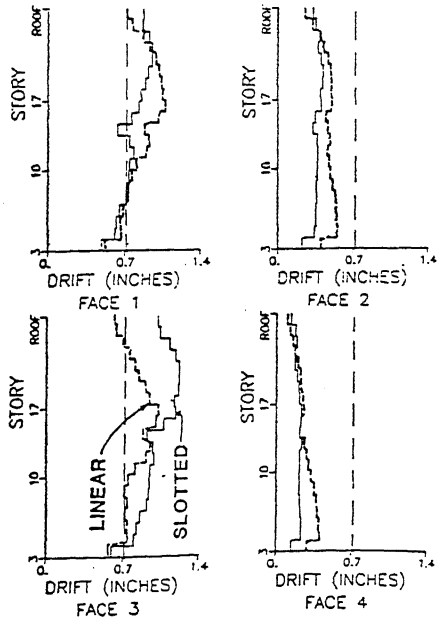


Fig. 7 Peak Story Drift Envelopes for 1940 El Centro NS in Braced Direction (parallel to Faces 1 and 3)

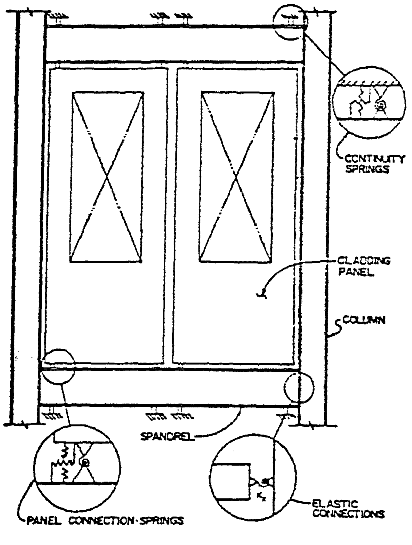


Fig. 8 Localized Response Model for Cladding

Table 1. Peak Interstory Drift for Slotted Connection Model

Ground Motion Input Direction (1)	Allowable Drift, Δ (inches) (2)	Peak Relative Interstory Drift (inches)			
		Face 1[a] (3)	Face 2 (4)	Face 3 (5)	Face 4 (6)
Braced	0.0[b]	1.1(16)	0.5(5)	1.0(16)	0.4(4)
	0.18	0.9(16)	0.5(16)	1.3(16)	0.3(4)
	0.36	0.9(20)	0.6(5)	1.3(19)	0.3(4)
	0.72	1.0(21)	0.4(19)	1.2(20)	0.2(12)
	∞[c]	1.0(21)	0.4(19)	1.3(18)	0.2(22)
Rigid	0.0	0.3(4)	1.0(19)	0.3(11)	1.3(19)
	0.18	0.4(21)	1.2(22)	0.3(4)	1.1(16)
	0.36	0.5(24)	1.0(23)	0.3(10)	1.0(18)
	0.72	0.4(23)	1.0(18)	0.3(22)	1.0(24)
	∞	0.5(24)	1.3(24)	0.4(24)	1.2(24)

[a] Story at which peak drift occurred is shown in parentheses.

[b] Clad case (V = 625 kips/inch).

[c] Unclad case (V = 0).

Note: 1 inch = 25.4 mm.

Table 2. Connection and Member Forces for Localized Response Model

Case (1)	Uniform Spring Stiffness(V=625)* (2)	PCI (V=248)* (3)	Slotted (V=0) (4)
1. Shear Stiffness per bay (kip/in)	63.69	32.15	0.
2. Max. horiz. force (kip) (bottom connection)	9.50	7.37	0.
3. Max. vertical force (kip) (bottom connection)	29.44	29.89	0.
4. Max. horiz. force (kip) (top connection)	9.54	7.37	0.
5. Max. vertical force (kip) (top connection)	37.03	0.	0.
6. Max. spandrel shear (kip)	28.45	21.04	6.17
7. Max. spandrel moment (in-kip)	316.52	305.90	300.25
8. Max. column moment (in-kip)	305.61	305.90	300.25
9. Max. vertical displ. (in) (spandrels)	0.0038	0.0036	0.0035

*unit = kips/in, 1 kip = 4.46 kN, 1 in = 2.54 cm