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Seismic performance of precast concrete column-to-column lap-splice connections

José F. Rave-Arango[a], Carlos A. Blandón[a,*], José I. Restrepo[b], Fabio Carmona[c]

[a] Department of Civil Engineering, EIA University, km 2 + 200 variante aeropuerto JMC, Envigado, Colombia
[b] Department of Structural Engineering, University of California, 9500 Gilman Dr, La Jolla, CA, USA
[c] Industrial Conconcreto, Cl 5 B# 21-24, Girardota, Colombia

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ABSTRACT

Implementation of precast construction for buildings requires connection techniques that can speed up the process by requiring only simple and easy on-site activities, while still guaranteeing satisfactory strength, stiffness and ductility. The construction method should reduce the use of formwork and temporary bracing, avoid aesthetic problems and be compatible with the lifting capacity available, but at the same time avert an increase of costs that could threaten the viability of using precast concrete. In the case of columns, the use of single story segments connected above and below the beam-column joints is an alternative that reduces the element weight and overcomes several problems that may appear when the column segments are connected at mid-height between stories. This alternative, however, requires splicing the reinforcement at a location where rotational demands may induce inelastic behavior, which is deemed inadequate by procedures stated in several design codes, unless proven otherwise. In response to this, a beam-column joint, splicing the reinforcement at the column ends, is herein proposed and tested under cyclic loading to evaluate its behavior and the possibility of using this technique to build moment resisting frame structures in seismic regions. The results from the precast column-to-column connection were compared with those from a similar cast-in-place unit with no splices. Results show that the behavior of both units are comparable, with just slight differences in the cracking, damage distribution and hysteretic behavior, so that the use of the proposed precast column-to-column connection may be considered appropriate.

1. Introduction

Precast concrete is accepted as an efficient construction method in various countries around the world, in recognition that under certain conditions this construction method can be more effective, economical and aesthetically pleasant than construction methods using cast-in-place concrete or structural steel. This includes countries in highly earthquake-prone regions, where precast concrete has been efficiently used to build low and high-rise buildings incorporating earthquake resistant structural walls and moment frames. These two structural systems can be designed either to emulate cast-in-place construction or to achieve low earthquake damage and re-centering properties [1]. Examples are already available of well-designed and built precast buildings, both emulative and low-damage, that exhibited an excellent seismic performance, including some cases reported during the 2010–2011 New Zealand swarm of earthquakes [2].

The main challenge in the design of earthquake resistant building structures incorporating precast concrete elements is in finding economical, practical and structurally sound methods for connecting the precast elements together [3] in such a way that, when subjected to seismic cyclic demands, these connections can, not only provide enough strength and stiffness, but also enable an appropriate inelastic deformation capacity and a stable hysteretic response of the structural system. In particular, long columns of precast moment frames would generally need to be spliced, with greater or lesser spacing depending on the crane lifting capacity, being the suitable locations of the splices either at mid-height between floors or at a face of the beam-column joints.

An important subject in considering the location of splices is their relationship with respect to the collapse mechanism expected for the system. In the design of special moment frames, that is, moment frames detailed to enable significant inelastic deformation capacity, codes make prescriptive recommendations to avoid the development of certain mechanisms during a rare but intense design earthquake. For example, there is a general agreement that a soft story mechanism should be precluded from forming in multi-story moment frame buildings, and

* Corresponding author.
E-mail address: carlos.blandon@eia.edu.co (C.A. Blandón).

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Notation

A_g	area of concrete cross-section
A_b	of longitudinal bar cross-section
$A_{h,j}$	energy for a cycle imposing a drift ratio j on the test unit
$A_{st,j}$	energy accumulated by an element linearly reaching point (D_{j+}, F_{j+})
b	section width
C_c	resisted by concrete in compression
D_{j+}, D_{j-}	and minimum drifts obtained when unloading from F_{j+}, F_{j-} with slopes K, K'
d_b	bar diameter
f_c'	concrete compressive strength
f_y	yield strength of reinforcement
F_{j+}, F_{j-}	and negative peak lateral strengths in a cycle
F_{max}	lateral force measured during test
h	section depth
K, $\acute{K}$	and negative initial stiffness for the first cycle
$M_{pr\ ACI}$	moment strength as prescribed by ACI 318-14 (2014)
$M_{pr,RR}$	moment strength as proposed by Restrepo and Rodriguez (2015)
P	force applied
$V_{pr\ ACI}$	lateral force capacity estimated from $M_{pr\ ACI}$
$V_{pr,RR}$	lateral force capacity estimated from $M_{pr,RR}$
x_c	from extreme compression fiber to point of application of force C_c
β	energy dissipation ratio
λ_{co}	strength-hardening factor
ρ_l	reinforcement ratio
ξ_{eq}	viscous damping ratio

codes include requirements to inhibit the development of such mechanism. Some codes, like the New Zealand code and those based on it, have stringent requirements to ensure the development of a weak-beam strong-column sidesway mechanism (Fig. 1a) [4]. These requirements permit the lap-splicing of the column longitudinal reinforcement at any given point, except at the bases of the columns where a plastic hinge is expected to form. Other codes, like ACI-318-14 [5] and those based on it, contain provisions that also aim at the development of a weak-beam strong-column sidesway mechanism, but recognizing that under dynamic conditions, an intermediate mechanism (Fig. 1b) may develop [6–8]. Such intermediate and other similar mechanisms can develop quite “unexpectedly” despite code provisions requiring that the column design moments at the faces of each joint be greater by a certain amount than the corresponding beam design moments. These mechanisms develop because during the dynamic response of the building to an earthquake input, the higher modes continuously move the inflection points of columns upwards and downwards in a way that cannot be captured in an equivalent static or modal analysis [9]. Coincidence between a lap-splice of the longitudinal reinforcement and the point where a plastic hinge forms, will constrain the spreading of plasticity and result in the development of large strains in the column longitudinal reinforcement at such location [10].

As a result, ACI 318-type codes favor lap-splicing of the columns longitudinal reinforcement at the center half of a column clear height. However, such splices can seldom be accomplished appropriately in precast concrete as they pose some practical drawbacks, like the necessity to schedule formwork and site activities both in between stories (for the column-to-column connections) and at the slab level (for the beam-to-column joints) and the difficulty to match the surface of the upper and lower spliced segments, which may result aesthetically inconvenient as the joint would be evident for the users even if tight tolerances are achieved. The alternative location of column-to-column connections right on top or below the beam-column joint, can eliminate the drawbacks of the mid-height location, but, while this option is suitable for frames designed in accordance with New Zealand-type standards, it is forbidden by codes like the ACI-318-14 in recognition that column hinging and concrete spalling may occur at a column end and compromise the splice. Application of such splices within these later type of building codes requires demonstration through laboratory testing that the spliced precast columns show an equivalent response to their cast-in-place counterpart.

This paper describes a precast column splicing method that makes use of corrugated steel grouted ducts at the column-joint interface. Experimental work was carried out on one precast concrete, one-way, interior, beam-column joint sub-assemblage (Fig. 1b) and on a similar cast-in-place sub-assemblage. The test units were built at a 2/3 scale and were tested using the quasi-static reversed cyclic loading protocol prescribed by ACI-374.1-05 [11]. Key response parameters obtained from each of the two test units are compared to assess the performance of the precast column-to-column connection.

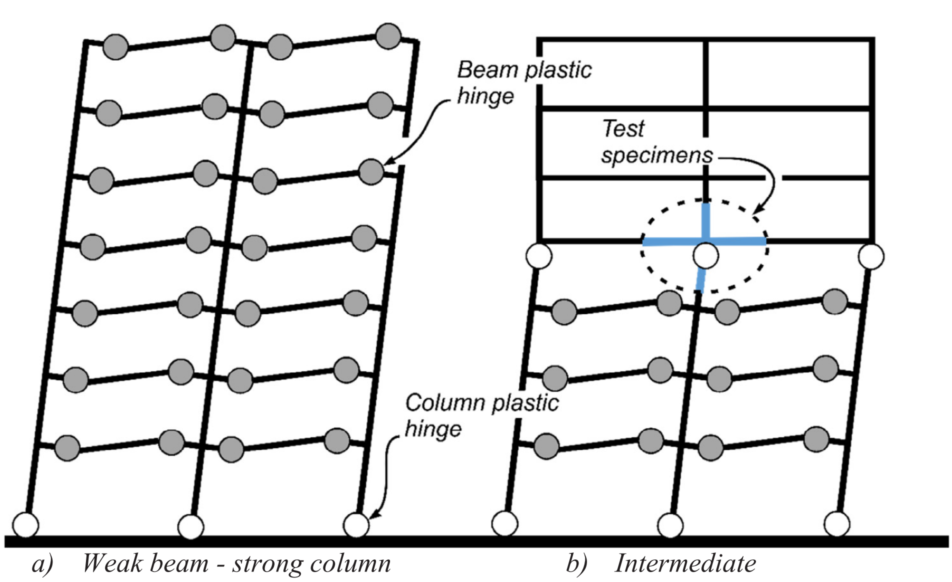

Fig. 1. Mechanisms of inelastic deformation for moment resisting frames.