An Innovative Tie System for Improving the Monolithic Behavior of Masonry In-filled Reinforced Concrete Frames (INFILTIE)

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ABSTRACT

CONCERT- Japan was an ERA-NET project under FP7 aimed at promoting an effective and coordinated science, technology and innovation (STI) cooperation between European countries and Japan. INFILTIE is an outgrowth of this call. A three-way cooperative research program was crafted and initiated in 2013 between the Scientific and Technological Research Council of Turkey (TUBITAK), the Federal Ministry of Education and Research (BMBF) of Germany and Japan Science and Technology Agency (JST). With partnering entities from Turkey, Germany and Japan the issue that is tackled is the reduction of damage during earthquakes to infill masonry walls that are enclosed within reinforced concrete building components.

Partial or complete collapse of infill walls affects the lateral load resistance and seismic performance of RC frames, which may determine whether the building stands or collapses during an earthquake. In-plane (IP) damage is related to the drift of the frame that encloses the wall, but its out-of-plane (OOP) capacity is influenced both by the IP damage and the OOP acceleration that walls undergo. One remedy is to provide an easily installed positive connection between the masonry infill and the surrounding frame so that the IP capacity is not degraded and the OOP capacity prevents falling out of the masonry part. This is achieved through a simple connector shaped like a dog bone that is connected to a slotted metal on the column. A flat steel plate laid flat on a joint course of the masonry wall to the columns. The name coined for this innovative system is INFILTIE.

Four one-half scale specimens were tested in the IP direction in Turkey. The parameters that were investigated were the amount and placement style of the ties to allow comparison with the empty frame or the infilled frame without ties. A total of eight one-fourth scale specimens were tested on a shake table in Japan to understand the resistance of the infill panels to OOP simulated earthquake accelerations. Extensive analytic modeling was done in Germany to understand the interaction between IP and OOP damages and the way in which these can be related to the installation of the ties. The paper provides an overview of the experimental program in Turkey, and presents the findings to date. It is concluded that INFILTIE provides significant enhancement for the OOP capacity.

Key Words: infill wall, reinforced concrete frame, in-plane response, out-of-plane failure, joint ties

1. INTRODUCTION

A recurring sight in post-earthquake field surveys in many countries in the Mediterranean basin relates to the poor performance of infill walls in reinforced concrete framed buildings that fall outwards following excessive drifts in their own plane. Several visual frames in Figure 1 illustrate the problem.
Infill walls are not the passive inertial structural components that most design procedures assume them to be. They inject stiffness into the framing, play a major role in altering the forces that columns must resist and are usually the most visible mirrors of drift because they tend to crack at very low drift values. Much research has been conducted for a better understanding of the interaction between the frames and infill walls but because of the unavoidable variations in material properties, workmanship and construction details an exhaustive modeling has proved to be elusive (e.g., Hashemi and Mosallam, 2007; Drysdale et al., 1999). Whereas codes or guidelines such as Eurocode, ACI 530 or ASCE 7 recommend that infill walls should be taken into account the anticipated improvement in performance is not easily quantified, nor are they confirmed by field experience.

Most investigations to date have focused on the in-plane (IP) role played by walls (Canbay et al., 2003; Pujol and Fick, 2010; Preti et al., 2012) but the need to examine the interaction between the IP and the out-of-plane (OOP) behavior has caught the attention of many others (e.g., Meisl et al., 2007; Mosalam and Günay, 2015).

2. DETAILS OF THE EXPERIMENTAL PROGRAM

The investigation reported here has dealt with the challenge of improving the IP strength and ductility of simplified infill frames while enhancing their OOP capacity with a simple bed joint tie that is inserted while the wall is assembled. The prototype is a single-story, one span, one-half scale substructure cut out from a typical frame in a building shown in Figure 2. The centerline height of the frame was 1400 mm, and its span 2500 mm. Part of the slab in the upper level was included in the model.

Four specimens were tested under reversed horizontal loading. The bare frame in Figure 2(b) was used for understanding the strength and ductility of the reinforced concrete component of the assembly. The next three
specimens were built with an infill wall and tested as in Figure 3(a), one without any horizontal bed joint to quantify the enhancement provided by the wall alone and two with two different arrangements of the joint reinforcement in the bed joints shown in Figure 3(b). Constant axial loads causing 0.3f_{ck} stress in the columns were maintained. The top girder was loaded with steel plates that amounted to 11 kN/m uniform load. Specimen details are shown partially in Figure 4.

2.1. Material Properties
Concrete strength f_{ck} varied from 22.3 MPa (Specimen 2) to 34.6 MPa (Specimen 3). The 8 mm longitudinal reinforcement had f_{yk} = 410 MPa. Standard hollow clay masonry units with 185x100x95 mm size were cut in half and laid with their cavities in the horizontal direction. They had a strength of 10 MPa based on gross area parallel to cavities. Their prism strength was measured as 1 MPa. ASTM standards were observed in mixing and testing the mortar in the walls. It had an average strength of 1.9 MPa.

2.2. Conduct of Tests
The experiments were conducted with the specimens placed in a test rig as in Figure 3(a), and subjected to IP cyclic load reversals up to 0.04 drift, proceeding at intervals of 0.005. Their recorded behavior was monitored through 60 channels of transducer output. The sequential images arrayed in Figure 5 provide the graphical description of the tests from specimen preparation to onset of force application.

Specimens 3 and 4 were fitted with the tie system shown in Figure 3(b). Whereas in Specimen 3 the ties were laid in every other horizontal bed joint and connected at both ends to closed U-shaped profiles attached to the two columns as in Figure 5(b), they were attached at only one end in Specimen 4 for easier installation. In Figure 5(c) it is seen that the tie in the second mortar bed from the base is free at one end. Successive ties were staggered with respect to which end was connected positively to one of the adjoining columns.

The manner of the tests permitted only measurement of the role played by the ties in the IP direction. The capability of the laboratory test setup and the loading protocol that was followed did not permit a direct assessment of the role they played in the OOP direction.
3. MEASURED IP RESPONSE AND CORRELATION WITH CALCULATED STRENGTH

The global response to horizontal loads applied at the girder level of the four specimens is best described in terms of their hysteretic load-drift curves. Unit column end rotations, joint distortions, wall deformations and reinforcement strains were also measured but in the interest of compactness of this narrative such curves will not be displayed here. The overall hysteretic response from all specimens is summarized in Figure 6(a). Figure 6(b) is the envelope of these curves, averaging force in either direction over the drift to which it applies.

The strength of the frames alone for all specimens derives from the capacity of the column sections shown in Figure 4. Translating the measured material properties into nominal frame capacity it becomes possible to infer the contribution of the infill walls to the IP capacity. The specimens all had yielding at the ends of the columns, so the moment capacity corresponding to the applied axial column force permits a conversion of column capacity to lateral capacity. The column strength is shown in Figure 7 which has two parts: the interaction curves and the moment-curvature curves for all specimens. The average column section moment capacity is 26.7 kN.m and column length 1.43 m so the frame alone provides a nominal strength of about 74.8 kN, matching well the measurement in Figure 6(b). The wall components in Specimens 2-4 showed first evidence of cracking at a
normalized drift of 0.005 or 7 mm. In Table 1 we provide the measured strength and secant stiffness for each specimen at this value.

Distinct planes of slippage formed in the walled specimens for continued increasing cyclically imposed displacements. Brick destruction accompanied this. In Specimen 2 the cracks also progressed through header courses, but for Specimens 3 and 4 the slippage occurred along the ties because vertical cracks were impeded by the steel in the horizontal joints. Figure 8 shows the cracking development for Specimen 3 at three drift values.

Table 1. Summary of Measured Response

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strength, kN</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>74</td>
<td>7 700</td>
</tr>
<tr>
<td>2</td>
<td>114</td>
<td>14 300</td>
</tr>
<tr>
<td>3</td>
<td>122</td>
<td>16 700</td>
</tr>
<tr>
<td>4</td>
<td>96</td>
<td>11 860</td>
</tr>
</tbody>
</table>

The graphical information in Figure 6 and the tabular values entered in Table 1 require careful interpretation. Differences caused by the infill-ties are embedded in the curves for Specimens 3 and 4. It is apparent that the continuous ties for Specimen 3 have added both strength and stiffness, but Specimen 4 displays an unexpectedly weak response. The specimen capacity is below the plain wall Specimen 2 which is counter intuitive. It seems to have been an outlier.
Only first-order estimates of the additional strength provided by the walls and the ties are needed to reconcile measurements and calculations. The vertical stress transmitted to the wall from the girder can be neglected. The total area of horizontal joint is equal to the product of the wall thickness (100 mm) and length (2300 mm) = 2.3e5 mm². With measured wall shear strength capacity from coupons = 0.13 MPa, the wall capacity is 29.9 ≈ 30 kN. The drift at which this capacity is mobilized is smaller than that at which the frame strength reaches its maximum value. So for Specimen 2 the capacity is found to be 74.8 + 30 = 104.4 kN, which is some 10 percent less than the measurement of 114 kN. No tests were carried out to quantify the bond strength of the smooth steel ties to the mortar, but assuming it to be 0.013 MPa and ignoring the strain incompatibility we can arrive at an estimate of the additional horizontal tension capacity as follows. Tie length (2260 mm) x width (80 mm) x bonded surface for each tie (2) x number of ties (3) x unit stress (0.013 MPa) yield an estimated incremental strength of 14 kN. The sum is 119 kN and for Specimen 3 the measured strength was 122 kN, representing a good match. In Specimen 4 one end of each tie was unconnected, so the bond cannot have been as good.

These estimates may appear to be crude models but they are surprisingly representative of experimental measurements. If we subtract the cyclic capacity of the frame alone (Specimen 1) at each cyclic drift value from those of Specimens 2 and 3 then we can isolate the contribution of the wall. This notion is exploited in Figure 9. The jagged nature of the curves derives from the fact that interpolation is required to match the displacement value of Specimen 1 exactly with any of Specimens 2 and 3. The estimated capacity enhancements of 30 kN and 44 kN are reflected in the measurements.
OOP CAPACITY

For the IP response, the behavior is driven by drift, a displacement quantity. In the OOP direction the situation is not the same because drift plays a minor role for that sense of loading. It is driven instead by normal “pressure” or floor level acceleration in building frames. An order-of-magnitude estimate for the OOP capacity of the infill can be worked out by using FEMA-356 (2000) provisions and a procedure similar to Dawe and Seah (1990). It is formulated below. The properties assumed for the materials are all measured quantities.

- \( \rho_d = 65 \text{ kg/m}^2 \): Unit mass of infill clay masonry
- \( f_{ck} = 29.5 \text{ MPa} \): Characteristic concrete strength
- \( E_c = 29 \text{ 500 MPa} \): Modulus of elasticity for concrete
- \( \nu_c = 0.15 \): Assumed Poisson ratio for concrete
- \( G_c = 12 \text{ 800 MPa} \): Shear modulus for concrete
- \( t_w = 100 \text{ mm} \): Wall thickness
- \( h_w = 1300 \text{ mm} \): Wall height
- \( l_w = 2300 \text{ mm} \): Wall length
- \( h_c = 1430 \text{ mm} \): Column height
- \( I_c = (200)^4/12 = 133 \text{ 000 mm}^4 \): Column moment of inertia
- \( J_b = 2 I_c = 265 \text{ 000 mm}^4 \): Column torsion constant
- \( I_b = 2.55e8 \text{ mm}^4 \): Girder moment of inertia
- \( J_b = 2 I_b = 5.1e8 \text{ mm}^4 \): Girder torsion constant
- \( f_m = 1 \text{ MPa} \): Wall compressive strength

\[
\begin{align*}
\alpha &= \frac{1}{h_w} [E_c l_c h_w^2 + G_c l_c t_w h_w]^{0.25} = 7.3 N^{0.25} \\
\beta &= \frac{1}{h_w} [E_c l_b t_w^2 + G_c l_b t_w l_w]^{0.25} = 34.9 N^{0.25} \\
q_{oop} &= 4.5 f_{ck}^{0.75} t_w^2 \frac{\alpha}{t_w} + \frac{\beta}{h_w} ] = 4.8 \text{ kPa}
\end{align*}
\]

This estimate of 4.8 kPa for the OOP capacity for the wall is based on the procedure by Dawe and Seah (1990). We can derive a second estimate using the FEMA-356 procedure.

\[
h_w = 1300 \text{ mm} \\
t_w = 100 \text{ mm} \\
h_w / t_w = 13 \text{ which implies } \lambda_2 = 0.044 \text{ (FEMA-356, Table 7.11)}
\]

\[
q_{oop,\text{min}} = \frac{0.7 f_m \lambda_2}{h_w / t_w} = 2.37 \text{ kPa}
\]

Lower limit = Mean – Standard deviation

FEMA-356, Table 7.2 \( \rightarrow \) multiplier = 1.3 which implies \( q_{op} = 2.37 \times 1.3 = 3.08 \text{ kPa} \). The range for OOP capacity therefore is obtained as \( 3.08 < q_{oop} < 4.8 \text{ kPa} \). Recalling that the unit mass for the wall is 65 kg/m\(^2\) these pressures correspond to the following constant spectral (or floor) accelerations:

\[
47.4 \text{ m/s}^2 < S_s < 73.8 \text{ m/s}^2 \text{ or } 4.83 \text{ g} < S_s < 7.52 \text{ g}
\]

It must be recalled that these high spectral accelerations depend on the mass distribution of the scaled-down specimens that have been described here. It is applicable only to Specimen 2. When the steel bands in Specimens 3 and 4 are considered as smeared reinforcement effective against bending in the horizontal direction the range

\[
\]
of plausible accelerations that would exhaust the capacity of the wall is estimated as $6.93 \text{ g} < S_a < 9.63 \text{ g}$. The wall panel with plate frequencies in the range of 10 Hz or more is a source of local vibration modes, so in a full-size building with frequencies of 2 Hz or lower, the OOP failure of a wall is possible only when the IP drift weakens it to a degree when its capacity in that direction is substantially degraded. An interactive heuristic hypothesis for the failure of the wall needs to be developed.

**AN INTERACTION MODEL**

The development of cracking on account of IP drift must reduce the OOP capacity of the wall. Similarly a frame with an infill wall that has first been cracked on account of OOP pressure applied to it must show a reduction in its IP capacity. This fundamental concept is pictured in Figure 10. If IP and OOP damage ensure mutual reduction of capacity then a failure surface that places these variables on its axes must be convex downward, with intersections describing the virgin capacities of the wall in the absence of prior damage on account of the other type of loading. We will adopt the hypothesis advanced by Kadysiewski and Mosallam (2009) as the basis of the development that follows.

Let $P_{IP, O}$ denote the capacity of the wall in response to drift and $P_{OOP, O}$ the capacity to normal pressure. With $P_{IP}$ and $P_{OOP}$ standing for the simultaneous “loads” in these directions that co-exist during the response to any ground motion then

$$\left( \frac{P_{IP}}{P_{IP, O}} \right)^{1.5} + \left( \frac{P_{OOP}}{P_{OOP, O}} \right)^{1.5} \leq 1$$

The power 1.5 is the result of curve fitting on the calculations that have been derived from fiber models. The key to applying Equation (5) to the measurements of the series of tests that have been reported in this paper is embedded in Figure 6(b). The difference of capacity between any specimen with a wall and Specimen 1 is attributable to the wall that degrades with increasing in-plane drift. The largest such difference must be $P_{IP, O}$ and the differences at smaller drifts the discrete values of $P_{IP}$. Let us choose to write $P_{OOP, O}$ in terms of acceleration expressed as m/s$^2$. These have been calculated for each specimen. Then solving Equation (5) for $P_{OOP}$ we obtain

$$P_{OOP} = P_{OOP, O} \left[ 1 - \left( \frac{P_{IP}}{P_{IP, O}} \right)^{1.5} \right]^{0.67}$$

In Figure 11 the interaction diagrams for all walled specimens are drawn as succinct summary. Specimens 2 and 3 are seen to conform to the failure hypothesis advanced in Equation (5) because their curves are both convex downward, confirming that damage in one direction inexorably diminishes the capacity in the perpendicular direction. Specimen 4 shows difference both because its $P_{IP, O}$ is smaller than that for Specimen 2 and because its convexity is violated, though for what appears to be a limited range. Again, transporting data from Figure 6(b) we can infer that for drifts larger than 0.02 an OOP acceleration of at least 2 g is required for wall failure. This estimate applies to the scaled models reported in this paper. For heavier and larger walls, possibly with various openings in them, the critical acceleration is likely to be smaller, causing the abject failures that Figure 1 shows.
DISCUSSION AND CONCLUSIONS

The improvement in OOP capacity provided by the infill ties is clearly shown in Figure 10 by the superior strength of Specimen 3. The measured poorer-than-expected response of Specimen 4, however, dampens the optimistic impulse in the interest of generalization that this limited observation might encourage. While providing positive connection to the side column at only one end of the ties leaves them vulnerable to cyclic in-plane displacement effects, it is still counter-intuitive that it should have inferior IP capacity to Specimen 2. It only confirms the difficulty of ensuring uniform quality in masonry construction that may have plagued other research programs.

INFILTIE appears to be an easily installed remedy to inject robustness into the OOP capacity of infill walls. The experimental program we have described in this paper has not considered any openings or piers in the walls because the tie needs to be connected at both of its ends. Only minimum joint reinforcement (the percentage provided in the specimens was 0.37) is then sufficient to ensure wall stability, and will help reduce the damage in the structural system.

The scale of the specimens tested in the experimental program was 1:2, so the rapid reduction of the wall mass will play a major role in interpreting the findings here, requiring adjustments of the critical accelerations that will endanger the OOP stability of real walls. A full appreciation of INFILTIE requires a broader program involving static and dynamic testing.

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A patent (No. TR 2012 01485 B) has been awarded in August 2015 to Gülkan and Güneş by the Patent Institute of Turkey for a device similar to the INFILTIE described here.
REFERENCES


