Lessons Learned

problems and solutions encountered by practicing structural engineers

Hybrid Masonry Connections and Through-Bolts

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As voluminous as our codes have become, they do not provide guidance to all situations. Students in structural design classes often ask the professor, “What is the procedure I need to follow?” The question they should be asking is “What is the underlying behavior?” Only then will they begin to develop engineering judgment and be able to correctly implement code provisions in their designs. Experienced engineers know that there are many instances in which engineering judgment is needed to move a design forward; the code allows for judgment based on a “rational analysis.” The underlying structural behavior informed by fundamental engineering principles can be used to build on a code provision. An engineer’s willingness to provide a client a design based on a rational analysis will be subject to the perceived risks and benefits associated with the design. The following are lessons learned from a limited number of tests performed on structural connections between steel and masonry elements of hybrid masonry seismic structural systems.

The goal of this article is to help practitioners gain a better understanding of the behavior of through-bolted masonry connections so that they can appropriately implement existing code provisions into designs prior to more data being developed.

Hybrid Masonry Overview

Hybrid masonry was introduced as a structural system concept in 2007. The system is composed of a structural steel frame and reinforced concrete masonry panels. Hybrid masonry offers a design alternative to braced frames and moment-resisting frames that are appropriate for low and mid-rise construction. It is best suited for cases where a structural steel framing system and masonry walls would naturally be chosen due to structural and architectural efficiency.

Hybrid masonry includes three distinct types of load transfer, which are shown in *Figure 1*. In Type I Hybrid Masonry (*Figure 1a*), steel connectors transfer in-plane shear between the steel frame and the top of the masonry panel. These connectors can be either rigid link plates or ductile fuse plates. The connectors do not transfer any vertical load to the masonry wall, but their design can have a significant influence on the overall performance of the system. In Type II and III Hybrid Masonry (*Figure 1b* and *Figure 1c*), headed studs are used to transfer shear from the beam and/or columns to the masonry panel. Vertical load is also transferred directly through contact from the beam to the top of the masonry panel.

In its simplest form (Type I), hybrid masonry consists of reinforced concrete masonry panels connected to the surrounding steel frame such that story shears can be transferred from the floor beams to the masonry. The masonry panel is constructed in-plane with the steel frame, supported on the floor beam or foundation below the panel. Steel connector plates between the masonry panel and the floor beam above the panel transfer only horizontal story shears (*Figure 1a*). The masonry does not make contact with the upper beam or columns other than through-bolts in vertically slotted holes in the connector plates, which are designed either as ductile “fuse” or elastic “link” connectors. The structural masonry panel acts as a surrogate-bracing member and can be reinforced both vertically and horizontally to resist in-plane and out of plane lateral forces.

Hybrid Masonry Types II and III are designed to transfer both shear and vertical load from the steel beam to the top of the masonry panel and, in the case of Type III, to transfer shear between the panel and the steel columns (*Figure 1b* and *Figure 1c*). Shear transfer is achieved through the use of headed studs welded to the beam and columns and embedded in the grouted cells, or formed bond beams, of the concrete masonry panel. All inelastic activity is focused in the masonry panel, while the steel frame and headed studs are designed with an overstrength factor to remain elastic during a design level seismic event.

Through-Bolt Connectors

The performance of hybrid masonry is highly dependent upon the performance of the connectors. Therefore, much of the research on hybrid masonry

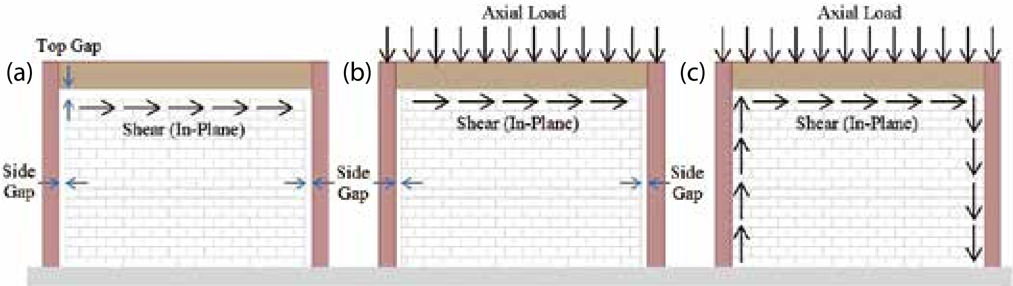

Figure 1. Hybrid Masonry Systems: (a) Type I; (b) Type II; (c) Type III.

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has uncovered some important information on the performance of masonry connectors that are not directly addressed in the masonry standard or building codes. Due to space considerations, this article will only address through-bolted connections. However, Part 2 of the article, in an upcoming issue of STRUCTURE, will address hybrid masonry steel plate connectors and headed studs in bond beams. The practicing engineer, whether designing hybrid masonry or conventional reinforced masonry, will find that added information useful.

Local Failure of Masonry at Through-bolts

For the hybrid masonry system to function correctly, it is essential that through-bolt connections between the masonry panel and the connection plates are able to transfer the required load without premature failure. However, many engineers also use through-bolts for conventional masonry construction. Local masonry failure mechanisms caused by a horizontal point load introduced into a bond beam via through-bolts is not addressed by any code. Is it appropriate to extrapolate existing provisions for anchors which load one face of the wall to the through-bolts? A test setup which monotonically loaded a single through-bolt toward the end of a CMU wall was used to estimate a lower bound capacity of the masonry. *Figure 2* shows one of the wall specimens after two tests. Damage on the right was caused when a through bolt in the second cell from the end of the wall was loaded towards the right. Damage on the left was caused by the load applied to the left at the side plates and through-bolt shown. Details of the test setup, results and discussion can be found in an upcoming TMS Journal article titled *Capacity of Masonry Loaded by Through-Bolts in Double Shear* and other reports and conference proceedings.

Through-bolt test data statistics.

Limit State	Low P_{cr}/P_n	High P_{cr}/P_n	Mean P_{cr}/P_n	COV
Masonry Breakout TMS 402-13	0.402	0.723	0.583	0.164
Masonry Crushing TMS 402-13	1.029	2.659	1.652	0.330
Bearing TMS 402-13	0.883	2.282	1.500	0.299
Shear of Unreinforced Masonry TMS 402-13	0.729	2.459	1.525	0.345
Shear Loading of Anchors ACI 318-14	1.232	2.152	1.708	0.150

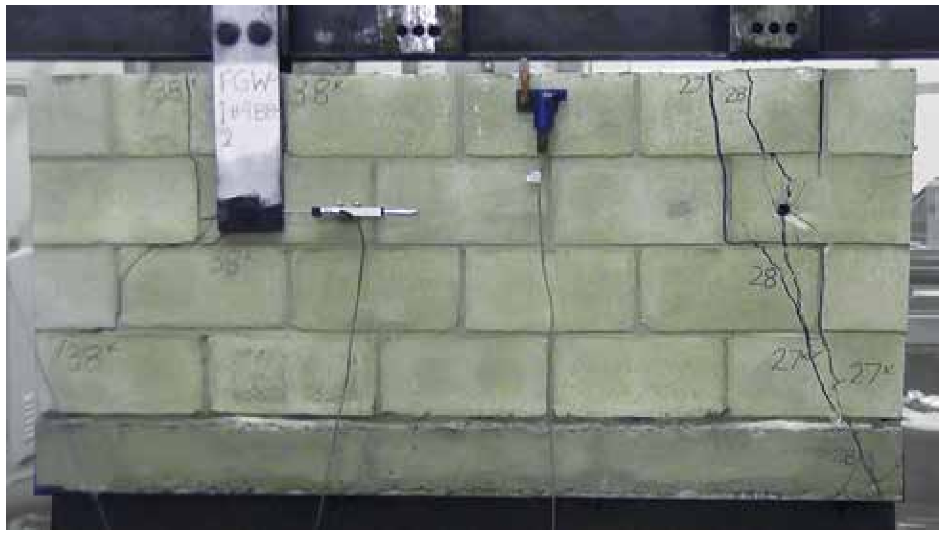

Figure 2. Through-bolt test specimen.

The results were compared with code specified limit states from TMS 402-13 and ACI 318-14 which could potentially be used with engineering judgment. The TMS 402-13 limit states of masonry breakout at anchors, masonry crushing at anchors, bearing, and shear were considered. The ACI 318-14 limit state of shear loading of anchors was also considered. The formulation for each limit state can be found on the table on the following page. *Figure 3* illustrates each limit state as implemented for through-bolts.

Figure 4 shows a plot of the predicted capacity of each limit state considered, versus the test data corresponding to the first cracking load observed during testing. Points to the left of the diagonal line indicate a conservative under-prediction of the capacity while points to the right indicate an overprediction. The data series labeled Maximum Test Load shows the reserve or additional strength beyond the first cracking for each specimen. The TMS 402-13 limit state equation for masonry breakout clearly and consistently overpredicts the capacity. Indeed, in most cases, the prediction also exceeds the maximum test load which is shown in the figure and indicates

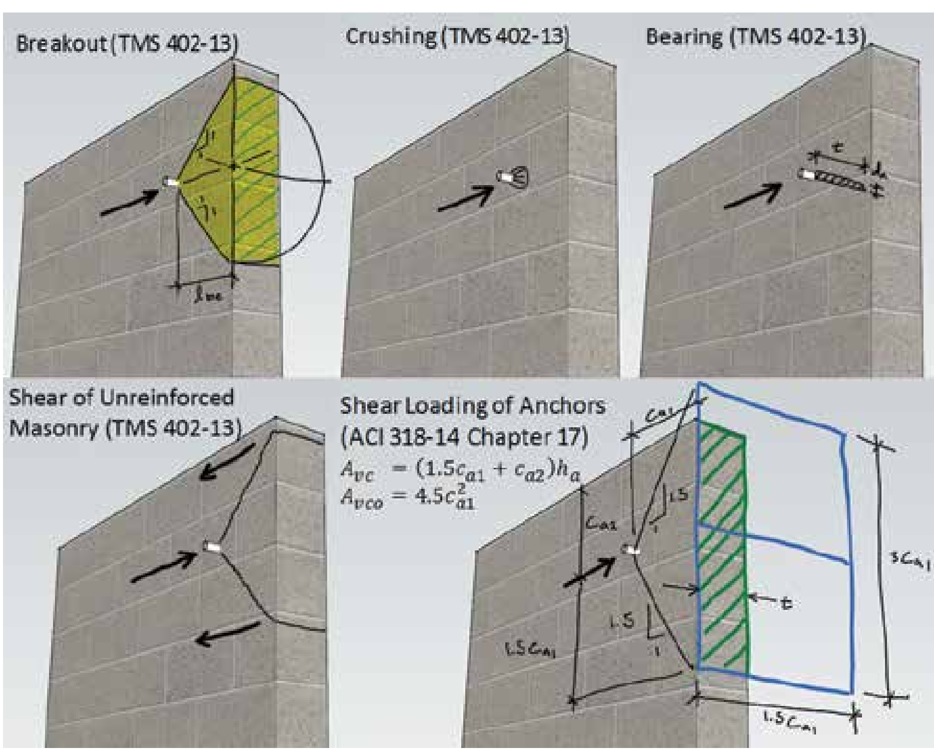

Figure 3. Limit State Geometry.

Table 1. Limit States evaluated for Through-Bolt Application with assumptions.

<table>
<tr><th>Limit State</th><th>Code Definitions</th><th>Additional Assumptions</th></tr>
<tr><td>Masonry Breakout at Anchors (TMS 402-13)
$B_{vnb} = 4A_{pv}\sqrt{f'_m}$ (US; f'_m in psi)
Where $A_{pv} = \pi l_{be}^2/2$</td><td>A_{pv} = projected shear area on the masonry surface of one-half of a right circular cone, in^2. The failure surface is oriented at 45 degrees from the axis of the cone.
l_{be} = anchor bolt edge distance.</td><td>Due to the narrow wall section the projected shear area can be approximated by a rectangular area extending above and below the level of the through-bolt at a 45 degree angle unless limited by the extent of grouting:
$A_{pv} = 2l_{be}t$, where t is the wall thickness.</td></tr>
<tr><td>Masonry Crushing at Anchors (TMS 402-13)
$B_{vnc} = 1050\sqrt[4]{f'_m A_b}$ (US; f'_m in psi)</td><td>A_b = anchor bolt cross-sectional area, in^2. Crushing occurs at the surface of the masonry block immediately adjacent to the anchor.</td><td>Crushing occurs at both sides of the wall simultaneously, so the total load applied to the through-bolt should be compared to two times the value determined by the given crushing formula.</td></tr>
<tr><td>Bearing (TMS 402-13)
$C_n = 0.8f'_m A_{br}$
Where $A_{br} = A_1\sqrt{A_2/A_1} \leq 2A_1$</td><td>$A_{br}$ = bearing area, in^2.
A_1 = loaded area not greater than the bearing elements size.
A_2 = supporting bearing area. It is the base of a pyramidal frustrum which must be wholly contained within the masonry. Each side of the frustrum is sloped at 2 transverse to 1 longitudinal in the direction of loading.</td><td>Since the through-bolt crosses the surface of the masonry wall, $A_{br} = A_1 = td_a$ no additional increase in bearing area is allowed by the code. t is the wall thickness and d_a is the diameter of the anchor (through-bolt). The supporting bearing area, A_2, cannot increase beyond the limits of the material.</td></tr>
<tr><td>Shear of Unreinforced Masonry (TMS 402-13)
$V_n = 3.8A_n\sqrt{f'_m}$ (US; f'_m in psi)
$\leq 90A_n + 0.45\ N_u$
$\leq 300\ A_n$</td><td>A_n = net cross-sectional area of a member, in^2.
N_u = compressive force acting normal to the assumed critical section associated with the applied shear force, V_u.
The limits shown are those associated with running bond.</td><td>Two shear cracks propagating from the location of the through-bolt toward the end of the wall was assumed. The resulting net area is defined by the distance between the through-bolt and the last vertical reinforcing bar in the wall, and the thickness of the wall. Another more conservative approach is to neglect any tensile capacity of the masonry above the through bolt. This alternative is also justified since the load path would transmit the shear force down to the bottom of the wall.</td></tr>
<tr><td>Shear Loading of Anchors (ACI 318-14)
$V_{cb} = \frac{A_{Vc}}{A_{Vco}}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$
V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete. Least of the following equations:
$V_b = (7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a})\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$
$V_b = 9\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$
c_{a1} = distance from the center of the anchor shaft to the edge of concrete in the direction of the applied shear or as modified to account for narrow sections.
c_{a2} = distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, in.
d_a = outside diameter of anchor, in.
l_e = load bearing length of anchor for shear, in.
λ_a = modification factor to reflect the reduced mechanical properties of lightweight concrete in certain concrete anchorage applications.</td><td>A_{Vc} = projected area of the failure surface on the side of the concrete member at its edge, for a single anchor or a group of anchors.
A_{Vco} = maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing, or depth of member. A_{Vco} is taken as $4.5(c_{a1})^2$
$\Psi_{ed,V}$ = factor to modify shear strength of anchors based on proximity to edges of concrete members.
If $c_{a2} \geq 1.5c_{a1}$, then = $\Psi_{ed,V} = 1.0$
If $c_{a2} < 1.5c_{a1}$, then $\Psi_{ed,V} = 0.7 + 0.3c_{a2}/1.5c_{a1}$
$\Psi_{c,V}$ = factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement.
$\Psi_{h,V} = \sqrt{1.5C_{a1}/h_a}$ = factor used to increase shear strength of anchors located in concrete members with thickness less than $1.5c_{a1}$.</td><td>For the through-bolt case, l_e was assumed to be the thickness of the wall, 7⅝ in. This results in the second formula for V_b controlling for through-bolts with a diameter greater than ½ inch.
The light weight concrete is not a factor: $\lambda_a = 1$.
The masonry compressive strength, f'_m, was used in place of the concrete compressive strength, f'_c.
$\Psi_{c,V}$ = 1.4 uncracked concrete at service loads. We are using this formula to attempt to predict first cracking. Otherwise using a value of 1.2 for cracked concrete with reinforcement of a No. 4 bar or greater between the anchor and the edge is appropriate.</td></tr>
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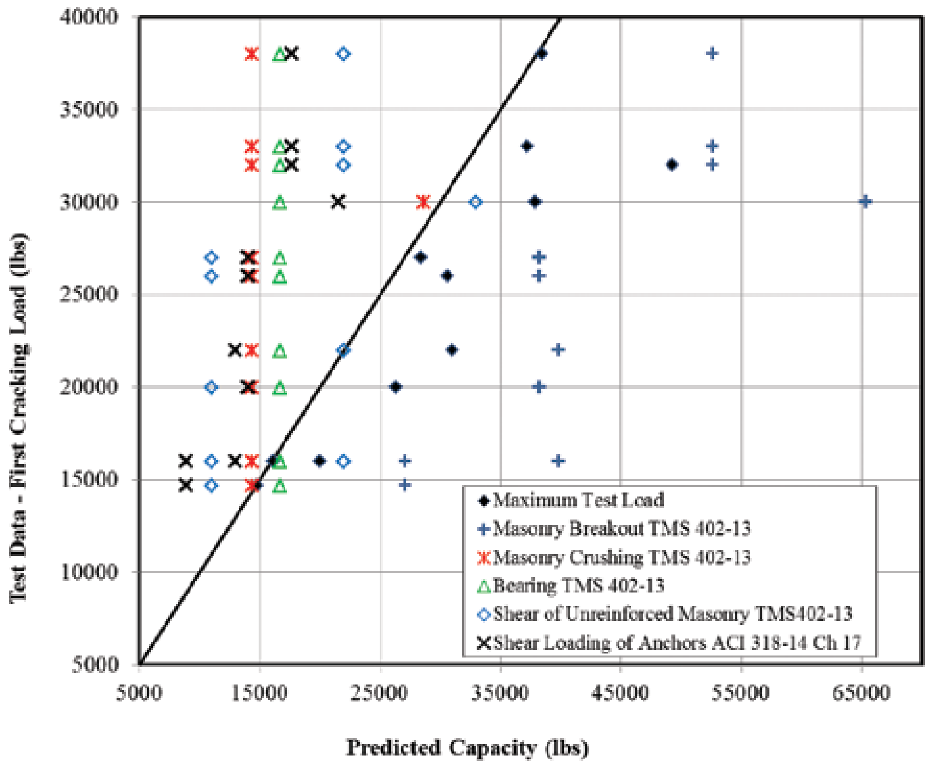

Figure 4. Test data versus predicted capacity from various specifications.

the additional observed strength beyond first cracking during the tests. The TMS limit states of masonry crushing at anchors and bearing are both a function of the bolt diameter and number of bolts – they do not appear to limit the connections capacity. Prediction using shear of unreinforced masonry generally was less than the actual tested capacity; however, it was an unconservative prediction in two cases. The ACI shear loading of anchors limit state provided a consistently conservative prediction. This data is not definitive especially considering the limited number of tests; however, it is instructive to know that the masonry breakout limit state equation is severely overpredicting capacity.

For each limit state evaluated, the *Table* gives the low, high and mean ratio of the first cracking load to the predicted nominal capacity (P_{cr}/P_n) as well as the coefficient of variation. Values of P_{cr}/P_n less than 1.0 indicate that the predicted nominal capacity is unconservative. The ACI shear loading of anchors and the TMS masonry breakout of anchors limit states have a significantly lower coefficient of variation compared to the other limit states. The observed failures are consistent with the theoretical failure shown in *Figure 3*.

Conclusions

Practitioners who use through-bolt connection details described in this article will not be able to find code language or limit states that directly address the behavior, boundary conditions and loading which can make these connections cost effective for hybrid masonry systems. They must rely on engineering judgment and should consider the following information.

Local Masonry Failure at Through-Bolts

- Based on the limited test data, TMS 402-13 masonry breakout of anchors and ACI 318-14 shear loading of anchors appear to provide the best correlation to the likely failure mechanism of masonry when a through bolt is installed near an edge. However, only ACI 318-14 shear loading of anchors can be used unaltered – it will provide a conservative result (the mean failure of the tests performed is 171% of the predicted nominal capacity).
- The TMS 402-13 masonry breakout of anchors limit state should be reduced significantly prior to it being applicable to predict the capacity at these connections. The mean failure of the tests performed is 58% of the predicted capacity which means that, even after using the strength reduction factor of 0.60, there will still be a 50% probability of failure at the design level load.
- Engineering judgment is required to incorporate these into a design. Further testing is needed to justify any specific code provisions being adopted for these connections.▪