THERMOMASS® Connector System for Concrete Sandwich Walls
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Composite Technologies Corporation

Introduction

THERMOMASS® connectors are manufactured and marketed by Composite Technologies Corporation (CTC) at the following address:

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Boone, Iowa  50036-0950

THERMOMASS® connectors are fiber composite anchors used in the construction of insulated concrete sandwich panels. The connectors provide a distributed fastening system that ties two concrete layers together through an insulation layer.

The following document provides a description of Thermomass connectors, a summary of the exhaustive testing of the connectors, and calculations that provide verification of the connectors’ ability to withstand environmental loading.

Applications

1. General

Thermomass fiber composite connectors are used in the construction of horizontally or vertically cast concrete sandwich walls. Horizontally cast applications include plant-cast precast panels and site-cast tilt-up panels. Vertically cast applications include plant-cast modular precast units and formed-in-place structures.

The connectors pass through insulation sheets that are cast between two or more layers of reinforced or prestressed concrete (Fig. 1). Connector ends bond to the concrete and transfer tensile and shear forces between the separate concrete layers. Because the connectors have low thermal conductivity and the insulation layer is installed throughout the width and length of each wall, the completed sandwich wall has excellent insulation qualities.
Thermomass connectors are formed from a fiber composite comprising 76 percent glass and 24 percent vinyl ester polymer by weight evaluated per ASTM D3172. A high impact, alkali-resistant polymer collar is injection molded to the connector at the center portion and produces a holding and sealing section at the insulation layer. Tension tests per ASTM D3039 show that the connector material has a tensile strength of 120 ksi (827 MPa). Accelerated-aging tests show that the connector material is resistant to alkali attack for over 100 years.

Insulation thickness can range from a minimum of 1 in. (25mm) to a maximum of 10.5 in. (265mm). Insulation panels are pre-cut using either hot-wire cutters or using power equipment as required for wall dimensions, window and door openings, and mitered corner joints. Connectors are inserted in pre-drilled, 7/16 in. (11mm) diameter holes in insulation boards.
It can be shown that Thermomass connectors provide low restraint of relative movement due to thermal and shrinkage effects. They therefore allow the construction of large panels with no control joints.

It is common to design the sandwich wall by assuming that the interior layer of concrete resists wind seismic and/or roof loads, while the exterior layer provides only physical protection for the insulation. However, it must be noted that the exterior concrete layer and the sandwich panel connectors themselves will be subject to forces arising from normal wind and seismic loading, temperature gradient within the layers, and temperature differential between the layers. These internal panel forces can be determined using sandwich beam theory as well as by using the stiffness method or the finite element method. This will be described in more detail in a following section.

2. **Horizontally Cast Systems**

In horizontally cast applications, connectors are typically placed at 16 in. (406mm) on center. The fiber composite core of the connector is supplied with dovetail ends that act as wedge anchors in the concrete (Fig. 2 and 3). The polymer collar includes a restraining flange, which rests at the insulation face, as well as corrugations, which wedge inside the holes in the insulation.

In wall systems in which the thickness of one layer of concrete is 2 in. (51 mm) to 2.5 in. (63 mm), connectors with 1.5 in. (38mm) embedment are specified (Series 15 (MS)). In wall systems in which the thicknesses of both layers of concrete are 2.5 in. (63 mm) or more, connectors with 2 in. (51 mm) embedment are specified (Series 20 (MC)). The total connector length is adjusted for the insulation thickness in order to provide the required embedment in each layer of concrete (Fig. 3).

![Thermomass Connector for Horizontally Cast Applications](image)

**Fig. 2. Thermomass Connector for Horizontally Cast Applications.**
3. Vertically Cast Systems

In vertically cast applications, connectors are placed at 12 in. (305mm) on center over the full wall area. The fiber composite core of the connector is normally supplied with two to four notches at each end to provide anchorage in the concrete. However, dovetail anchorage similar to that provided for horizontally cast systems can be provided where higher anchorage forces are required. Special corrugations on the collar section serve as snap rings that position and secure separate snap locks, which provide restraint at each face of the insulation layer during concrete placement. The core is fabricated to a length that is 1/8 in. (3mm) shorter than the specified wall thickness. The connectors therefore serve to position the insulation system inside the wall during concrete placement.
Installation
1. Installation drawings and instructions are shipped to the job site with the connectors. Drawings show insulation panel and connector layout for each sandwich panel. Instructions provide guidelines on placement of insulation and consolidation of concrete around connectors.

2. Plant-cast or site-cast (tilt-up) panels are cast in horizontal forms. The initial layer of reinforcing is positioned in the form. Concrete is then placed and leveled to the design thickness. Insulation is placed over the plastic concrete, connectors are inserted in the pre-drilled holes in the insulation, and the concrete around the connectors is consolidated. Consolidation can be effected by vibrating the connector, vibrating the form, or by “walking” the insulation surrounding each connector. The second layer of concrete and its reinforcing is then placed on top of the insulation layer. In plant-cast construction, this operation normally occurs immediately after the insulation is placed. In site-cast construction, this operation normally occurs after the first layer of concrete has hardened.

3. Modular pre-cast or in cast-in-place concrete walls are cast in vertical forms. Before placement, a snap lock is slid over one end of each connector rod. The rod is inserted through pre-drilled holes in the insulation. A second snap lock is then slid over the free end of the connector rod and snapped into place. After all snap locks are fully seated against an insulation board, the assembled board is inserted into the concrete form, reinforcing steel is installed in each concrete layer, and the forms are installed.

Fig. 4. Connector Assembly for Vertically Cast System.
Concrete is placed on both sides of the insulation board and is consolidated using standard pencil vibrators inserted in the concrete.

Quality Control
Thermomass Series 15 (MS) and Series 20 (MC) connectors are listed by Southwest Research Institute (SwRI) of San Antonio, Texas (Southwest Research 2000). The NES and ICBO ES recognize SwRI as a testing laboratory and quality assurance inspection agency.

Existing Product Approvals
2. Deutsches Institute Für Bautechnik (German Institute for Civil Engineering). Approval Number: Z-15.2-144.
3. Centre Scientifique et Technique du Batiment (CSTB), Commission chargée de formuler des Avis Techniques (Commission charged to formulate Technical Opinions), Avis Technique 1/99-750.
4. ÉPÍTÉSÚGYI MINÖSÉGELLENÖRZÖ INTÉZET (Institute for Quality Control of Building), Budapest, Approval Number A-161/1994.

Discussion of Test Reports
1. Southwest Research Institute
   The SwRI report (Southwest Research 2000) includes test data on the behavior of Thermomass connectors when subjected to static and cyclic loading as well as information on the performance of the connectors when subjected to fire. All connectors tested were MS type, with 1.5 inch nominal embedment. It should be noted that CTC’s MC type connectors have 2 inch nominal embedment—the data for the MS connectors are therefore conservative for all Thermomass applications.
   Tension and shear tests were conducted in general accordance with ASTM E488 and ICBO ES AC01 requirements. These tests not only required that the static capacity of the connectors be determined, but that the connectors would show less than a 20% degradation in strength after simulated seismic loading. It should be noted that the shear tests were conducted using a severe test configuration with a steel plate located 1 inch above the free surface of the concrete. This configuration produces higher shear forces and stresses than would be produced in a connector anchored in 2 layers of concrete and with insulation thickness of 2 inches or more. However, it allows cyclic shear testing to be conducted with a minimum of variables and test equipment.
   Fire tests were conducted in general accordance with ASTM E1512. Test connectors were embedded in 3 inches of concrete and were loaded with 300 lbs tension while being subjected to a standard fire time-temperature profile. The test was designed to verify the ability of the connector to hold a face veneer against the building during an internal fire (a likely post-earthquake event).
   In the SwRI tests, the connectors provided a mean static pullout capacity of 1642 lbs and a minimum mean single shear capacity of 513 lbs. The post-dynamic tension and shear capacities exceeded 80 percent of the static capacities, indicating that the Thermomass connectors are capable of resisting severe seismic loadings without major
loss of capacity. Finally, the fire tests indicated that the connectors resist over 1 hour of
fire temperatures, even when subjected to a high tensile force.

2. Iowa State University

A comprehensive series of tests were conducted at Iowa State University (ISU) to
establish the capability of the connectors (Wade et al. 1988), (Porter and Wade 1990),
and (Porter et al. 1992). The following is a discussion of the testing programs.

a. Pullout Strength

The initial series of tests included pullout tests of anchors at both normal and elevated
temperatures (Wade et al. 1988). These test results were used to establish the current
design recommendations, including spacing and embedment. In tests of 30 anchors with
2 in. (51mm) embedment, the average pullout capacity of the connectors was measured at
2550 lbs. (11.34 kN) in concrete with an average compressive strength of 3500 psi (24
MPa). Failures were of three types: concrete cone pullout, interlaminar shear in the
connector, or fracture at the net section of the connector. For connectors with 1.5 in.
(38mm) embedment, the average pullout capacity was measured at 1850 lbs. (8.23 kN).

b. Effect of Elevated Temperature

Two tests were conducted to determine the effect of elevated temperature on the
pullout capacity of the Thermomass connector. Test 1 was subjected to a load of 1050
lbs. (4.67 kN) and to increasing temperatures. Test 2 was subjected to a load of 350 lbs.
(1.56 kN) and to increasing temperatures. Test 1 failed in interlaminar shear after 235
minutes of loading, reaching a temperature of 200°F (93°C) at failure. Test 2 showed a
displacement of 0.009 in. (0.23mm) after 455 minutes and reaching a temperature of 233°
F (112°C). It should be noted that 350 lbs. (1.56 kN) is equivalent to hanging a 15 in.
(381 mm) thick, horizontal face layer from connectors at 16 in. (406mm) on center.

c. Alkali Resistance

The initial accelerated aging tests were used to show that the resins used in the
connectors serve to protect the glass fibers from alkali attack (Wade et al. 1988).
Notched test specimens were immersed in a lime-saturated bath heated to a temperature
of 122°F (50°C) for up to 52 weeks. Studies on glass-fiber-reinforced concrete (GRFC)
indicate that one day of immersion in this type of bath is related to 101 days of natural
aging for GRFC. The Thermomass specimens showed no significant change with respect
to time after the first week of exposure. The connectors are therefore anticipated to be
capable of surviving over 100 years of exposure within concrete installations.

Extensive additional work was conducted on connector specimens embedded in
concrete (Porter and Barnes 1993). Pullout, tension, interlaminar shear, compression, and
flexural properties were evaluated. Because data were obtained using concrete at various
ages, test loads were normalized for strength of the concrete. In all cases, the simulated
aging had little effect, if any, on any of the test specimens.

d. Shear Capacity

Push-out (shear) tests were used to evaluate the capacity of the connectors loaded in
shear. Specimens were constructed with three concrete layers and two insulation layers.
Load was applied until failure while displacement was monitored. The data show that the
connectors are capable of carrying the exterior layer of concrete with no fixed supports
such as ledges or ledger angles. A 2 in. (51 mm) layer of concrete weighs 25 psf (1.20
kN/m²). The weight of concrete supported by each connector will be a function of the
connector spacing. For connector spacing of 16 in. (406mm), the tributary area for each
connector is 1.77 ft$^2$ (0.16m$^2$), giving a total connector load of $25 \times 1.77 = 45$ lbs. (192 N). Tests show that at a shear load of 45 lbs. (200 N), the connector displacement is less than 0.05 in. (1.3mm).

e. Compression Capacity

Compression tests were used to evaluate the ability of the connectors to assist the insulation in carrying compressive loads between the concrete layers. Specimens were constructed with two 3 in. (76 mm) concrete layers and one 2 in. (51 mm) insulation layer. Four 6 in. (152 mm) connectors were installed in 32 in. (812 mm) square panels. Load and boundary conditions were established to allow the connectors to punch through the concrete layers. Test and analysis results show that, in a typical configuration, the connectors carry approximately 22 percent of the compression loads, while the insulation carries the remainder. The results also show that the combined insulation-connector system is capable of carrying a minimum of 1800 psf (86 kN/m$^2$) before punching or failure of the connector occurs.

f. Full Scale Panel Tests

Flexural tests were conducted on precast sandwich walls with mild steel reinforcement. Test specimens were 4 ft. (1219mm) wide by 20 ft. (6096mm) long and used insulation thicknesses of 2 in. (51mm) or 4 in. (102mm) between 3 in. (76mm) thick concrete layers. Panel 1 was in a 3”/2”/3” (76/51/76 mm) configuration and was tested using a push load. Panel 2 was in a 3”/2”/3” (76/51/76 mm) configuration and was tested using a pull load. Panel 3 was in a 3”/4”/3” (76/102/76) configuration and was tested using a push load. The 4 in. (102 mm) insulation thickness was built-up using two 2 in. (51mm) layers of insulation. Panels 1 through 3 used extruded polystyrene insulation. Connectors were placed 16 in. (406 mm) on center in both the longitudinal and transverse directions. Line loads were applied at the upper and lower quarter points, and the panel was horizontally restrained at each end. Load and deflection data were obtained.

Panel 1 displayed nearly constant stiffness up to its cracking load. Panels 2 and 3 displayed non-linear softening behavior before cracking. Because Panel 2 was a pull test, the bond between the concrete and the insulation was broken at a relatively low load. Panel 3 maintained bond between the concrete and the insulation; however, the low bond between the insulation layers allowed relative slip, also at a relatively low load. After cracking, the load carried by Panel 1 dropped off to nearly the same load as was carried by Panels 2 and 3. After cracking, all panels carried nearly constant load with increasing displacement up to failure at a centerline displacement of up to 6 inches (152mm).

The precast panel tests show that insulated wall panels can be easily constructed using the Thermomass connector system. The data show that partial composite action can exist, providing strength and stiffness exceeding values calculated based on the presence of two independent concrete layers. Large panel deflections, exceeding 6 inches (152mm), are possible with no apparent distress to the fiber composite connectors. The tests show that the connector design provides firm connection between the two layers of a sandwich panel and is sufficient to withstand handling and service forces.

Eight prestressed concrete flexural specimens were tested to failure. Specimens were 4’ (1219mm) wide by 20’ (6096mm) long. All panels had 3” (76mm) thick concrete layers. Panels 1, 2, and 6 through 8 had 16” (406mm) connector spacing, Panel 3 had 24” (610mm) connector spacing, and Panel 4 had 12” (305mm) connector spacing.
Panels 1 through 4 incorporated 2” (51mm) extruded polystyrene insulation; Panel 6 and Panel 7 had expanded polystyrene insulation with a 1 pcf (157 N/m$^3$) density; and Panel 8 had expanded polystyrene insulation with a 2 pcf (314 N/m$^3$) density. All panels had 4 prestressing strands in each layer. Panels were loaded using line loads applied at the quarter span points. Panels were instrumented with strain gages on the concrete and reinforcing steel and were tested using push or pull loads.

All panels displayed essentially linear behavior up to the cracking load, when the behavior became non-linear softening. The panels were very ductile, with a total centerline deformation of up to 10 inches (254mm). Using theoretical cracking loads calculated assuming non-composite action, it was determined that experimental cracking loads for specimens with extruded insulation were approximately 25% higher than anticipated. In all panels, the strain data indicate that the neutral axis was not located at the layer centroid. The neutral axis did move toward the centroid as loading increased, indicating a reduction in composite action with increasing load. Strain data were used to calculate the moment carried by each layer through pure flexure. The sum of the moments carried by each layer was subtracted from the external (applied) moment to obtain the approximate moment due to composite action. Although Panels 1 through 5 had composite moments that were as low as 10 percent, Panels 6 and 7 had composite moments that were between 50 and 70 percent for applied loads up to 8 kips (35.6 kN) (nearly 70 percent of the capacity). At no point was bond deterioration noted between insulation and concrete. Although Panel 8 had a maximum composite moment of 30 percent, it should be noted that it was subjected to several lift tests before flexural testing. The lift testing reduced the bond and contact area between the concrete and insulation at the time of the flexural test.

The panel tests show that the fiber composite connectors are capable of withstanding large deformations without failure. Panel capacity continues to increase after initial flexural cracking. The data indicate that, with extruded polystyrene insulation, composite action does exist at small displacements. However, with increasing load, the degree of composite action decreases. The data also indicate that flexural behavior of concrete sandwich walls can be significantly affected by the degree of bond between the insulation and the concrete. Because expanded polystyrene has a roughened texture, it provides a greater degree of composite action than extruded polystyrene.

**g. Low Temperature Fatigue Behavior**

An elemental series of tests was conducted to determine the effects of cold temperatures on the low-cycle fatigue behavior of Thermomass connectors (Porter and Barnes 1990). Shear specimens were constructed using 3 layers of concrete and 2 layers of insulation. Each insulation layer comprised two 3 in. (76mm) sheets of insulation for a total thickness of 6 in. (152mm). The imposed shear deformations were selected to simulate extreme temperature cycling of the exterior layer of a 42 ft (12.8m) tall, base-supported panel. The cyclic temperature differential between layers was selected as 160° F (89° C), resulting in a relative deformation of 0.45 in. (11.4mm). This differential was based on a building experiencing a maximum temperature range of 120° F a total of 2000 times over a 50 year period, combined with an interior operating temperature of -40° F (-40° C).

A total of 12 specimens were tested. During the tests, the outer two layers of concrete were cooled to -40° F (-40° C) and the inner concrete layer was displaced relative to the
outer two layers of concrete for 2000 cycles. The specimens were then loaded in shear to failure of the connectors.

All specimens withstood the extreme temperatures and the large deformation over the full 2000 cycles. Between 0 and 100 cycles, the load required to deflect the layers 0.45 in. decreased significantly. Although the authors speculated that local spalling of the concrete at the connector anchorage caused this reduction, it should be noted that the polystyrene collar on the connector does add to the initial stiffness of the Thermomass connector. At low temperature and large displacements, it is probable that the collar will crack and the connector stiffness will therefore decrease. Since the collar is not considered as a load-carrying portion of the connector, this loss is of no significance. For the remainder of the cycles, the load required to reach the maximum displacement remained nearly constant. During both the cyclic testing and the final load to failure, the connectors displayed an increase in stiffness with increasing deformation over a small deformation range. This increase is the result of the increased contribution of the insulation to resisting shear. As the displacement increases, the connector carries more tension load. This, in turn, is equilibrated by compression in the insulation. Friction at the insulation interface is therefore increased, resulting in increased stiffness. All specimens failed at shear loads exceeding 650 lbs. (2890 N). The failure mode was fracture at the net section.

h. Connector Orientation

An extensive test program was conducted to evaluate connector behavior as a function of connector orientation relative to the insulation layer (Porter et al. 1992). Test panels were 48 in. by 96 in. (1219 mm by 2438 mm) with two 3 in (76 mm) layers and insulation of 2 (51 mm) to 4 in. (102 mm) thickness. In order to evaluate the effect of bond, panels were constructed with and without polyethylene sheets between the insulation and the concrete. Three test types: A, B, and C, were conducted. Type A specimens had 18 connectors installed normal to the plane of the insulation. Type B specimens had a combination of 18 normal and 3 or 4 angled connectors. Type C specimens had angled connectors only. Push-off tests were used to induce shear in the insulation and connector system.

Push-off tests showed that the most efficient orientation for angled connectors was at an angle of 30 degrees relative to the insulation with the connector installed such that it was subjected to weak axis bending. (In the weak axis bending orientation, the connector anchorage is fully engaged in concrete on both sides of the connector. In the strong axis bending orientation, one of the anchor surfaces on the connector will have reduced cover and will tend to spall the concrete at the anchorage).

With 2 in. (51 mm) extruded polystyrene insulation, the average shear load at 1/16 in. deflection for Type A specimens was 3916 lbs. (17.4 kN). This is an average connector load of 217 lbs. (967 N). Type B push-out specimens with 4 angled connectors required nearly 3 times the load as Type A specimens to reach a 1/16 in. (1.6 mm) relative deflection. The average shear load at failure for Type A specimens was 6941 lbs. (30.8 kN). This is an average shear force of 385 lbs. (1.71 kN) per connector. Type B specimens had peak loads nearly 3.9 times those of Type A specimens. At failure, the average deflection for Type A specimens was 0.24 in. (6.1 mm) while the average deflection for Type B specimens was 0.35 in. (8.9 mm).
With 4 in. (102 mm) extruded polystyrene insulation, the average shear load at 1/16 in. deflection for Type A specimens was 2255 lbs. (10.0 kN). This is an average connector load of 125 lbs. (555 N). Type B push-out specimens with 4 angled connectors required 3.6 times the load as Type A specimens to reach a 1/16 in. relative deflection. The average shear load at failure for Type A specimens was 13050 lbs. (58.0 kN). This is an average shear force of 725 lbs. (3222 N) per connector. Type B specimens had peak loads nearly 1.9 times those of Type A specimens. At failure, the average deflection for both Type A and Type B specimens was 0.58 in. (14.8 mm). It is important to note that, even with 4 in. (102 mm) insulation, the average connector load at 1/16 in. deflection (125 lbs.) is more than the self-weight of a 5 in. thick face layer affecting each connector.

i. Mechanical Properties
The tensile modulus of elasticity, and flexural modulus of elasticity were evaluated on individual connector specimens. The connectors were shown to have tensile and flexural modulus values of 7200 ksi (49000 N/mm²) and 4800 ksi (33000 N/mm²), respectively.

3. Construction Technologies Laboratory
Three flexural tests were performed on 8 ft x 4 ft x 10 in. (2438 mm x 1219 mm x 254 mm) sandwich panels with 2 in. (51mm) extruded polystyrene insulation (Johal 1984). Specimens were vertically cast with Thermomass connectors at 12 in. (305mm) centers in each direction. The concrete had a nominal maximum aggregate size of 3/4 in. (19mm) and a slump of 4 in. (102mm).

Test procedures conformed to ASTM E72, “Standard Methods of Conducting Strength Tests of Panels for Building Construction.” Specimens were tested in the horizontal position and were subjected to quarter point loading with a 90 in. (2286mm) span.

Cracking in the bottom layer and slip at the upper layer-to-insulation interface initiated at loads corresponding to midspan moments of 2.2 kip-ft/ft of width (9.8 kN•m/m) and 2.9 kip-ft/ft (13 kN•m/m), respectively. The average maximum midspan moment was 4.5 kip-ft/ft (20 kN•m/m). For comparison purposes, the moment capacity based on full composite action would be 4.0 kip-ft/ft (17.6 kN•m/m). Also for comparison, the moment at slip at the upper layer-to-insulation interface corresponds to the calculated moment capacity for an 8 in. (203 mm) solid concrete wall with similar reinforcement and cover. The report clearly shows that the Thermomass connectors can be used in the construction of vertical cast sandwich panels and that the flexural behavior is excellent.

4. Underwriters Laboratories, Inc.
Jensen and Sanchez (1989) evaluated the performance of a concrete sandwich wall incorporating the THERMOMASS® connector system in a small-scale fire test per UL 263. The test panel comprised 2-inch (51mm) and 5-1/2 inch (140mm) layers of concrete with a 2-inch (51mm) layer of extruded polystyrene. Thermomass connectors were installed at 16 inches (406mm) on center. The 5-1/2 inch (140mm) layer of concrete was exposed to the test fire.
During the test, the unexposed surface showed no signs of cracking or separation. Also, the exposed surface showed no signs of cracking and showed no signs of separation from the exposed surface. After 240 minutes of fire exposure, the test sample was subjected to a hose stream for 45 seconds at 45 psi (0.31 N/mm²). No projection of water beyond the unexposed surface was observed.

The specified average and maximum limiting temperatures were not reached during the fire exposure. At the end of the four-hour test, the interior of the interior concrete layer reached 738°F (390°C) in the area surrounding a fiber composite connector. The surface temperature at the interface of the concrete and insulation was 404°F (207°C). The average external surface temperature at the 2-inch (51-mm) layer of concrete was 105°F (41°C). It is important to note that the auto-ignition temperature for the THERMOMASS® connector, as measured using ASTM D1929, is 900°F (482°C) minimum. This is well above the temperatures observed during the four-hour test and indicates that the system has significant potential for even more severe exposures.

5. University of Kaiserslautern

The following subsections summarize reports required for a product approval in Germany (Deutsches Institut für Bautechnik 1993, 1999). Thermomass connectors have been marketed in Germany by DEHA Ankersystem GmbH & Co. under the trademark DEHA-TM-Verbundsystem. Tests were conducted at the University of Kaiserslautern.

a. Panel Shear

Two shear tests were performed to determine the load-deformation behavior of a panel system (Ramm 1991a). Test specimens were subjected to static load applied as shear. Specimens were constructed with one 60mm (2.4 in) insulation layer and two 70mm (2.8 in.) concrete layers. Specimens were 800mm (31.5 in) wide and 160mm (6.3 in) long and contained 8 connectors. One specimen was constructed with insulation only, while the other specimen was constructed with a polyethylene film between the insulation and the structural layer.

The maximum shear load for Specimen 1, with insulation only, was 4 kN (899 lbs.) per connector, while the maximum shear load for Specimen 2, with both insulation and film, was 3 kN (674 lbs.) per connector. The maximum shear deformation was approximately 7mm (.3 in) for both specimens. Examination showed that connectors had flexural-tension failures. No concrete failures were noted. The test results confirm the results reported by Wade, Porter, and Jacobs (1988).

b. Pullout and Shear Capacity

Ten tensile tests were conducted on 400mm (15.7 in) by 400mm (15.7 in) specimens with two-170mm (6.7 in) concrete layers and one-60mm (2.4 in) insulation layer (Fig. 5) (Ramm 1991b). Each specimen had one connector at the centroid of the 400mm (15.7 in) square area. Four shear tests were conducted on 400mm (15.7- in) by 800mm (31.5 in) specimens with one-170mm (6.7 in) center layer and two-70mm (2.8 in) exterior layers. Three of the shear specimens had 60mm (2.4 in) of extruded polystyrene insulation between each concrete layer, while one of the shear specimens had 60mm (2.4 in) air gaps between each concrete layer. One of the specimens with insulation was prepared with an oiled surface on the insulation. A total of 4 connectors were installed in each
shear specimen, with two per shear area. Concrete strengths ranged from 31 to 32 Mpa (4500, 4640 psi).

In all tensile tests, the maximum load was limited by failure of the concrete. The minimum failure load was 10.5 kN (2361 lbs.), the maximum failure load was 14.5 kN (3260 lbs.), and the average failure load was 12.6 kN (2833 lbs.) at an average displacement of 8mm (0.3 in).

In all shear tests, the maximum load was limited by failure of the anchors. The maximum connector load for the two systems with normal insulation surfaces were 4.5 kN (1012 lbs.) (Test 1) and 4.0 kN (899 lbs.) (Test 2) at 13.8mm (0.54 in) and 13.4mm (0.53 in) displacements, respectively. The maximum connector load for the system with oiled insulation was 3.5 kN (787 lbs.) at 9.8mm (0.39 in) displacement, and the maximum connector load for the system without insulation (Specimen 4) was 2.2 kN (495 lbs.) at 6.2mm (0.24 in) displacement. After examination of the test specimen, it was determined that Specimen 4 had insufficient embedment of the connector and that the data were not valid.

The maximum loads confirm the results obtained by Wade, Porter, and Jacobs (1988). The deformations obtained in the tension tests were 8mm (0.3 in), compared with 13mm (0.51 in) in the ISU tests. However, the ISU test data included deformation of the load inserts, while the current report excluded deformation of the load inserts. The shear deformations obtained in Tests 1 and 3 agree with the data from the ISU tests.

c. Tension, Shear, and Combined Tension/Shear

Ramm (1992a) further evaluated tension, shear, and combined tension/shear behavior. Tension tests were conducted on 400mm x 400mm (15.7 x 15.7 in) specimens with concrete layer thicknesses of 170mm (16.7 in) and an insulation thickness of 60mm (2.4 in) (Fig. 5). One anchor was placed at the centroid of each specimen. Tests Z1-Z10 used class B15 concrete (15 MPa cube strength, 12 MPa cylinder strength (1740 psi)). Tests Z18-Z27 used class B55 concrete (55 MPa cube strength, 45 MPa cylinder strength (6530 psi)). Tests Z1-Z10, Z18-Z19, and Z22-Z24 had 51mm (2 in) anchor embedment, while tests Z20-Z21 and Z25-Z27 had 38mm (1.5 in) anchor embedment.
Shear tests were conducted on 400mm x 800mm (15.7 x 31.5 in), three-layer specimens (Fig. 6) Outer concrete layers were 70mm (2.8 in) thick, while the inner concrete layer was 140mm (5.5 in) thick. Tests S01 through S14 were cast with insulation, while tests S15 through S21 were cast without insulation. Concrete strengths ranged from 19 to 37Mpa (2755, 5365 psi). Insulation or air gaps were 40, 60, or 100mm (1.6, 2.4, or 3.9 in).
Combined tension/shear tests used 400mm x 800mm (15.7 x 31.5 in), three-layer specimens similar to the shear test specimens. Tests K1 through K9 included insulation, while tests K10 through K18 had air gaps between the three concrete layers. Tension levels from 2.9 kN (652 lbs.) to 8.1 kN (1821 lbs.) were applied to panels with insulation, while tension levels from 1.7 to 4.8 kN (382, 1079 lbs.) were applied to panels without insulation. In all tension/shear tests, the shear load was increased to failure while the tension level was held constant.

Tension tests for anchors with 51mm (2 in) embedments had average failure loads of 12.5 and 16.5kN (2810, 3709 lbs.) for class B15 and B55 concrete, respectively. Anchors with 38mm (1.5 in) embedment had an average failure load of 14.5kN (3260 lbs.). Three failure modes: fracture, interlaminar shear, and concrete pullout were noted. Shear specimens with insulation had average failure loads of 4.2, 4.6, and 5.2kN (944, 1034, 1169 lbs.) with insulation layer thicknesses of 40, 60, and 100mm (1.6, 2.4, and 3.9 in) respectively. Shear specimens without insulation had average failure loads of 1.6, 1.4, and 0.9kN (359, 315, and 202 lbs.) with air gaps of 40, 60, and 100mm (1.6, 2.4, and 3.9 in) respectively. Under combined shear and tension loads, the specimens showed that shear capacity decreases with increasing tension load. Interaction between shear and tension is approximately linear (Fig. 7).
These tests verify earlier tension and shear test data produced at Iowa State University. The shear interaction results verify that the connectors are capable of carrying significant forces, even without the benefit of insulation and that a linear interaction between shear and tension is appropriate for design.

![Tension-Shear Interaction for Thermomass Connectors in 60 mm Styrofoam Insulation](image)

**Fig. 7 Tension-Shear Interaction for Connectors with 60mm Insulation (Ramm 1992a).**

d. **Long-Term Shear**

Three-layer test specimens were subjected to long-term dead load applied as shear (Ramm 1992b). Specimens were constructed with 60mm (2.4 in) air layers. Load magnitudes ranged from 0.6 to 1.0kN (135 to 225 lbs.) (41% to 74% of the short-term capacity) and were applied with weights. Initial deformations ranged from 3.0 to 8.1mm (0.12 to 0.32 in). Specimens sustained the applied loads over times ranging from 5 to more than 2500 hours. Deflections and time-to-failure were monitored.

Long-term-load to short-term-capacity \(\left(\frac{Q_t}{Q_o}\right)\) ratios were assigned to each test. Two specimens subjected to a 1.0kN load \(\left(\frac{Q_t}{Q_o} = 0.74\right)\) failed after at least 5 hours. The initial deformations were 7.6 and 8.1mm, while the maximum deformation at failure was 14.5 mm. Four specimens subjected to 0.9 kN loads \(\left(\frac{Q_t}{Q_o} = 0.63\right)\) failed after at least 15 hours. The initial deformations ranged from 5.8 to 7.7 mm, while the maximum deformation at failure was 12.4 mm. Four specimens subjected to 0.7 kN loads \(\left(\frac{Q_t}{Q_o} = 0.52\right)\) failed after at least 560 hours. The initial deformations ranged from 4.6 to 6.5 mm, while the maximum deformation at failure was 14.9 mm. Two specimens subjected to 0.6 kN loads \(\left(\frac{Q_t}{Q_o} = 0.41\right)\) did not fail at the time of the report, but survived over 1500 hours of loading. The initial deformations ranged from 3.0 to 4.7 mm, while the maximum deformation at 1500 hours was 6.9 mm.
A load-durability diagram was generated with duration of loading on the abscissa (0.1 to 10000 hours on a logarithmic scale) and the \( Q_t/Q_o \) ratio on the ordinate (Fig. 8). Using the data, the following load-durability relationship was derived:

\[
\frac{Q_t}{Q_o} = (t/t_o)^{-1/15}; t_o = 0.1h
\]

This can be used to predict the time to failure for a specific load ratio:

\[
(t/t_o) = (Q_t/Q_o)^{-15}
\]

Using the load-durability curve obtained from the tests, a very conservative estimate of the ability of the connectors to carry the dead load of a face layer can be calculated. For example, with an anchor spacing of 400mm x 400mm (15.7 x 15.7 in) and a face layer thickness of 100mm (4 in), the long-term load on a connector is approximately \( Q_t = 25 \times 0.1 \times 0.4^2 = 0.40\text{kN} \) (90 lbs.). Using a conservative 20% reduction in load bearing capacity due to alkaline influence and using the previously obtained test results, the connectors have a predicted short-term capacity of \( Q_o = 0.8 \times 1.4 = 1.1\text{kN} \) (247 lbs.). Therefore, \( Q_t/Q_o = 0.4/1.1 = 0.36 \) and \( t = 0.1 \times (0.36)^{15}/(365 \times 24) = 44 \text{ years} \). It must be noted that this is based on the extremely conservative assumption that the insulation has no long-term contribution to the connector capacity. Since the insulation will, in fact, carry significant shear loads, the connectors are fully capable of carrying an even thicker face layer over a long design life.

![Creep Failure Under Shear Load With a 60 mm Gap and No Insulation](image)

**Fig. 8 Time to Failure as a Function of the Shear Load without Insulation.**
e. Cyclic Tension

Cyclic tension tests were conducted on 3 specimens with 51mm (2 in) embedments and 3 specimens with 38mm (1.5 in) embedments (Ramm 1992c). The maximum load amplitude was selected based on a temperature differential of 5K (41°F) from the outside to the inside of a face layer of 100mm (4 in) thickness. This amplitude is given by

\[ \Delta Z = 2E_c \alpha \Delta T d_1^2 / 8 \]  

where  
- \( E_c \) = modulus of elasticity of the concrete layer  
- \( \alpha \) = coefficient of thermal expansion for the concrete layer, and  
- \( \Delta T \) = design temperature differential over the exterior layer thickness  
- \( d_1 \) = exterior layer thickness

This provides \( \Delta Z = 4kN \). A minimum load of 2 kN (450 lbs.) was selected based on wind suction. 25000 cycles were applied to 400x400 (15.7x15.7 in) specimens with 60mm (2.4 in) insulation and a single connector.

Average peak tensile capacities were 15.8 kN (3552 lbs.) and 15.7 kN (3530 lbs.) for connectors with 51 and 38mm (2 and 1.5 in) embedments, respectively. These capacities are 96 and 108 percent of the static capacities determined from earlier tests. In summary, the data show that cyclic tension loads have a negligible effect on the connectors.

f. Alkali Resistance

Tension and shear tests were conducted on specimens deposited in a saturated calcium hydroxide solution (pH=12) at 50°C (122°F). Specimens had the same dimensions as previously constructed tension and shear specimens (Ramm 1992c).

Tensile specimens subjected to 171 days in the alkaline solution had average tensile capacities of 16.0 kN (3600 lbs.). This is 111% of the previously determined capacity of specimens not subjected to alkaline attack. Shear specimens subjected to 171 days in the alkaline solution failed at average shears of 4.2 kN (944 lbs.), or 91% of the previously determined capacity of specimens not subjected to alkaline attack.

The tests show that the connectors are only affected slightly by the alkaline environment present in concrete. These tests provide significant information on the long-term effects of concrete on the connectors. Because the saturated solution was heated, its effects are accelerated. Studies have shown that each day in such an environment is equivalent to over 100 days in concrete at ambient temperatures. The connectors therefore are capable of surviving over 47 years in concrete with only slight deterioration.

Ramm (1993a) conducted an additional evaluation of the alkali resistance of the connector structure. Two-layer tension specimens and three-layer shear specimens were placed in a saturated calcium hydroxide solution at 50°C (122°F). Specimens were removed after 2, 4, 6 and 8 months and tested in tension and in shear. Concrete strengths ranged from 62 to 69 MPa (8990 to 10005 psi) for tension specimens and 26 to 38 MPa (3770 to 5510 psi) for shear specimens. Both tension and shear specimens were cast with insulation.
Tension specimens failed in three possible failure modes—fracture at the origin of the dovetail notch, interlaminar shear, and concrete pullout. The average tension capacity of specimens subjected to 214 days in the heated alkaline solution was 15.5 kN (3485 lbs.), or 107% of the previously measured capacity of specimens not subjected to the alkaline bath. The average shear capacity of specimens subjected to 214 days in the heated alkaline solution was 4.0 kN (899 lbs.), or 87% of the previously measured capacity of specimens not subjected to the alkaline bath. Using the current and previous test results the average capacity of specimens without insulation was estimated by deducting the contribution of the insulation to the measured capacity. The estimated shear capacity of the aged connector was found to be 0.8 kN (180 lbs.), or 57% of the connector capacity measured for specimens not subjected to the alkaline bath.

A 214-day exposure to a saturated lime bath results in some degradation of strength. It must be noted, however, that based on other studies, the test exposure is equivalent to a 60 year exposure and no further degradation is to be anticipated. It also must be noted that, even neglecting the contribution of the insulation to the shear capacity of the connector, the connector capacity is still well above that required to carry a 100mm (4 in) face layer.

g. Cyclic Shear

Three-layer test specimens were subjected to cyclic shear loads under displacement control (Ramm 1992e). Specimens were constructed with 60mm (2.4 in) insulation or 60mm (2.4 in) air layers. The amplitudes of the displacements were selected based on a 70K (126°F) temperature difference, Δϑ, between the structural and exterior layers on 12m (39.4 ft) long panels. The displacement range is therefore given by

\[ u = \alpha \Delta \vartheta L / 2 \]

\[ = 10^{-5} (70)(12000) / 2 \]

\[ = 4 \text{mm}(0.16 \text{ in.}) \] (0)

Based on stiffness data obtained in previous tests, a static deflection of 2mm was selected as representative of the initial dead-load deformation of a face layer. Test specimens S31 through S33 and S37 through S39 were subjected to 25000 cycles of deformation from 2 to 6mm (0.08 to 0.24 in) (increasing deformation). Specimens S34 to S36 were subjected to 25000 cycles of deformation from +2 to –2mm (0.08 to –0.08 in) (reversing deformation).

Increasing deformation produced the most significant effects on connectors. Average shear capacities after 25000 cycles of deformation were 2.8 kN, 4.8 kN, and 1.2 kN (629, 1079, and 270 lbs.) for insulation-layer specimens subjected to increasing deformation, insulation-layer specimens subjected to reversing deformation, and for air-layer specimens subjected to increasing deformation, respectively.

Even after severe cyclic deformation, the connectors continue to have shear capacities that exceed the weight of a suspended face layer. For example, a 70mm (2.8 in) face layer will produce a maximum shear of 0.27 kN (61 lbs.), while the tests show that the connector capacity will remain over 4 times this amount.

Three-layer test specimens were subjected to cyclic shear deformation under displacement control (Ramm 1993b). Specimens were constructed with 60mm (2.4 in.) air layers. The initial deformation for all specimens was 2mm (0.08 in.), based on the
static deformation of a connector supporting a 70mm (2.8 in.) face layer. The amplitudes of the imposed displacements were selected based on a 70K (158°F) temperature difference between the structural and exterior layers on 7 and 12 meter (23 and 39 ft) long panels, resulting in 2.5 and 4 mm (0.1 and 0.16 in.) incremental displacements, respectively. After 25000 cycles, the pullout capacities of the anchors were determined. The average pullout capacities after cyclic loading were compared with pullout capacities under static load only.

Specimens subjected to a 2.5mm (0.1 in) amplitude incremental displacement had static tensile capacities of 9.8 kN (2203 lbs.), while specimens subjected to a 4.2mm (0.17 in) incremental displacement had static tensile capacities of 3.2 kN (719 lbs.). These values are 59 and 19 percent of the tensile capacities of statically loaded specimens. In each test, the failure mode was a fracture at the origin of the dovetail notch in the connector.

3.2 kN (719 lbs.) is 12 times the weight of a 700 mm (28 in) face layer, and is 10 times a 2 kPa (40 psf) wind suction. The connectors maintain adequate capacity even after long-term fatigue loading due to temperature change. It must also be noted that a 70 K (126°F) temperature difference is not realistic for concrete sandwich panels with 60 mm of insulation. A more realistic value is 35 K (63°F).

**Discussion of Variations in Product Installations on Fire Resistance**

Typical applications of the Thermomass connectors are described above. As is discussed, concrete and insulation thickness can vary between and within projects. Although the described fire test was conducted on a small-scale panel, the test results clearly show that the Thermomass® test panel more than satisfied the fire requirements for a nonbearing wall.

Standard methods can be used to calculate the fire rating for wall configurations other than those included in the test. For example, the PCI Design Handbook (PCI 1992) describes the calculation for the effectiveness of a sandwich panel using the equation

\[
R = (R_1^{0.59} + R_2^{0.59} + R_3^{0.59})^{1.7}
\]

where \( R \) = the fire endurance of the composite assembly in minutes, and \( R_1, R_2 \) and \( R_3 \) are the fire endurance values for the individual layers in minutes. PCI Figure 9.3.2 can be used to find the fire endurance of concrete according to type and thickness. Since the melting point of cellular polystyrene of one inch or greater is around 400 to 600°F (200 to 316°C), any additional thickness has a negligible resistance value. PCI Section 9.3.6.5 therefore assigns a resistance of five minutes to this type of insulation.

**Connector Behavior Under Load**

Thermomass connectors are subject to forces resulting from lifting and from service conditions (temperature, gravity, wind, and seismic effects). These effects are discussed in detail in the following sections.

In most applications, the standard configuration of Thermomass connectors is more than adequate to carry the load. Detailed calculations are therefore not warranted. Because the Thermomass connectors are unique, however, this information is provided as
a supplement to the empirical justification provided by the extensive testing program. It is also provided to allow verification of unusual panel configurations or loading.

1. **Lifting Conditions—Tension Forces**

   A fundamental concept that is discussed above is the need to maintain isolation between the two layers in a sandwich panel. The use of full-thickness concrete or the use of steel connectors that penetrate the insulation system is strongly discouraged, since it can create a number of problems. These problems include 1) thermal bridging, which can lead to energy waste and formation of condensation, 2) cracking, which results from the introduction of localized rigid connections that restrain shrinkage movement, and 3) bowing, which results from shrinkage accompanied by composite action induced by rigid connections near the panel ends. The Thermomass connectors are designed to eliminate the need for full-thickness concrete or steel connections.

   In order to avoid these problems, it is important to tie the two concrete layers in a sandwich panel with Thermomass connectors only. During lifting, therefore, only the Thermomass connectors will exist to pull the lower layer of concrete from the casting surface. One of the most important features of the Thermomass connector is its ability to safely provide this function.

   Analytical and experimental studies by Wade et al. (1988) show that the force in an individual connector increases as the distance from the lift point decreases. Analytical studies on 16 in. (410 mm) wide panels with thin structural layers show that if the lift point is directly above a connector, the connector force will be approximately 50 percent of the lifting force. These studies also show that the two connectors on either side of the lift point each can carry 25 percent of the lifting force. Although the study was conducted on 16 in. (410 mm) wide panels models, it is reasonable to assume that, in wider panels with thin structural layers, the force applied at each lifting insert could be carried by only 4 connectors.

   In a typical tilt-up installation, the face layer is cast first. The face layer can typically be 3 in. (76mm) thick and will weigh 38 psf (1.8 kPa). Even with a release agent present on the casting surface, it is reasonable to expect a modest bond between the surface and the lower layer of concrete. A typical assumption is that the bond will create a 25 psf (1.2 kPa) suction force that will act in addition to the weight of the lower layer. The total force that must be lifted by the connectors is therefore approximately 65 psf (3.3 kPa). Based on this distributed force, the load carried by each lift point can be calculated.

   For example, for a 20 ft tall by 10 ft wide (6096mm x 3048mm) panel rotated about its base using two lifting inserts at 5’-9” (1753mm) from the top of the panel, the lifting forces will be approximately 4300 lbs. (19 kN) at each insert. Distributed over 4 connectors, the lifting force will be approximately 1100 lbs. (4.8 kN) in each of the four Thermomass connectors immediately surrounding the insert.

   It should be noted that this is an extremely conservative analysis. In actual panels, the lifting force is not applied to only four connectors at each lift point. As the relative stiffness of the structural layer increases, more connectors will participate.

2. **Service Conditions—Shear**
During service, the Thermomass connectors must transfer gravity, wind, and thermal loads from the face layer to the inner layer. The behavior of Thermomass connectors subjected to shear loading has been quantified in a mechanical model proposed by Ramm and described by Gastmeyer (1997). Using their notation, the total shear force, \( Q_{A,G} \), carried by a connector and its surrounding insulation is given by

\[
Q_{A,G} = Q_D + Q_{A,A},
\]

where \( Q_D = \) the portion of the force carried in the insulation (Dämmung) and \( Q_{A,A} = \) the portion of the force carried in the connector (Anker).

![Load Carrying Mechanism for Thermomass Connectors (Ramm 1993c and Gastmeyer 1997).](image)

Fig. 9. Load Carrying Mechanism for Thermomass Connectors (Ramm 1993c and Gastmeyer 1997).
As shown in Fig. 9, shear load in the connector/insulation system is the sum of shear in the insulation and in the connector. The shear in the insulation initially is a function of bond between the concrete and the insulation (Region 1). After bond is broken, shear in the insulation is carried by friction, which is influenced by the tension force and the resulting compression force in the insulation (Region 2). In addition, the connector carries shear in bending (Region 3). The force carried in the connector will be a function of the relative displacement, \( u \), between the two concrete layers, and the flexural stiffness of the connector. Ignoring shear deformations and assuming that the connector is rigidly connected to the concrete at each end, \( Q_{A,A} \) will be given by

\[
Q_{A,A} = \frac{12E_{ab}I_A}{d_A^3}u
\]

(0)

where \( E_{ab} \) = the modulus of elasticity of the connector in bending = 30000 N/mm\(^2\) (4350 ksi), \( I_A \) = the average moment of inertia of the connector = 245 mm\(^4\) (0.0006 in\(^4\)), and \( d_A \) = the effective length of the connector. Distance \( d_1 \) is a function of the insulation thickness, \( e \), and the embedment length for the connector, \( d_e \):

\[
d_A = e + \frac{2d_e}{3} \left[ 1 - \frac{1}{1 + d_e/e} \right]
\]

(0)

The force carried in the insulation is a function of the bond condition between the insulation and the concrete. With bond, \( Q_D \) will be given by

\[
Q_D = \frac{G_DA_A}{e}u
\]

(0)

where \( G_D \) = the shear modulus for the insulation = 4 MPa (580 psi) and \( A_A \) = the effective area of the insulation surrounding each connector = \( ab \) (for connectors at 406 mm (16 in.) on center, \( A_A = 164,836 \text{ mm}^2 \) (248 in\(^2\))). After the bond between the insulation and the concrete is broken, the insulation will continue to carry shear through friction. As the connector undergoes a displacement \( u \), the connector is placed in tension, resulting in a compressive force in the insulation. This force can be approximated using the kinematics of the connector-insulation system (Fig. 10). Incorporating friction, \( Q_D \) will be given by

\[
Q_D = \frac{\mu E_D A_A}{2ed_A}u^2
\]

(0)

where \( E_D \) = the modulus of elasticity for the insulation = 9 MPa (1305 psi) and \( \mu \) = the coefficient of friction between the concrete and the insulation = 0.3.
During handling and erection, bond exists between the concrete and the insulation through most of the panel length. For this case, the shear force vs. differential-shifting stiffness incorporating bond, $C_{B,I}$, can be approximated by combining Eq. (7) and (9):

$$
C_{B,I} = \frac{Q_{A,G}}{u} = \frac{G_D A_A}{e} + \frac{12 E_{Ib} I_A}{d_A^3}.
$$

After the panel has been placed in service and has been subjected to cyclic deformation resulting from temperature changes, the bond between the concrete and the insulation may be broken at the ends of the panels. The shear force vs. differential-shifting behavior without bond, $C_{B,II}$, can be approximated by combining Eq. (7) and (10):

$$
C_{B,II} = \frac{Q_{A,G}}{u} = \frac{\mu E_D A_A}{2ed_A} u + \frac{12 E_{Ib} I_A}{d_A^3}
$$

$$
= \sqrt{\frac{\mu E_D A_A}{2ed_A} Q_{A,G} + \left(\frac{6 E_{Ib} I_A}{d_A^3}\right)^2} + \frac{6 E_{Ib} I_A}{d_A^3}
$$

As shown by Gastmeyer (1997), Eq. 11 and 12 can be used to determine the stresses in the face layer of concrete by combining the stiffness relationships with classical sandwich panel solutions.
3. Service Conditions—Tension

As shown by Gastmeyer (1997), direct tension force is induced in the connector by the combined effect of wind suction and temperature gradient through the protective layer (Fig. 11). The total direct tension force, $Z_A$, is given by

$$Z_A = Z_{A,w} + Z_{A,\Delta T}$$

(0)

where $Z_{A,w}$ is the tension force induced by the wind and is given by

$$Z_{A,w} = 1.13 A_A w,$$

(0)

and where $Z_{A,\Delta T}$ is the tension force induced by a temperature gradient through the face layer and is given by

$$Z_{A,\Delta T} = \frac{\alpha_r \Delta T}{8b / E_{ab} a d_1^2 + 6ad_1 / C_z b A_A} + \frac{\alpha_r \Delta T}{8a / E_{ab} b d_2^2 + 6bd_2 / C_z a A_A}.$$  

(0)

Fig. 11. Wind and Temperature Gradient Loading (Gastmeyer 1997)
Safety Factors Per the German Approval (Deutches Institute 1993, 1999)

Using data from the studies conducted in the U.S.A. and Germany, Ramm (1993c) tabulates allowable forces for Thermomass connectors. These allowable forces are based on average failure loads reduced using global safety factors, $\gamma$. Separate global factors are calculated for tension, and shear loading. Each includes partial safety factors that account for material variability, $\gamma_m$, modeling uncertainties, $\gamma_{SYS}$, and load variability, $\gamma$. The partial safety factor for material variability, $\gamma_m$, is given by

$$\gamma_m = e^{(\alpha R \beta - k)\nu}$$

where $\alpha_R$ = the structure factor, $\beta$ = the safety index, $k$ = the fractile factor, and $\nu$ = the coefficient of variation for the test results. For the Thermomass connectors, Ramm used $\alpha_R = 0.8$ based on combinations of effects with a normal distribution, $\beta = 4.74$, $k = 1.645$ for the 5% fractile, and $\nu = 0.2$ based on a statistical analysis of the conducted tests. Using these values, $\gamma_m = 1.5$.

The partial safety factor that accounts for installation conditions, $\gamma_{SYS}$, varies with the loading type, the installation safety as a function of errors and tolerances, and the level of industry experience with the product. For tensile loading, Ramm recommended $\gamma_{SYS} = 1.5$. For shear loading, concrete connectors are typically not sensitive to installation errors. $\gamma_{SYS}$ can therefore be set equal to 1.0. However, in German practice, $\gamma_{SYS}$ was set equal to 1.5 for both tension and shear. As allowed in Eurocode applications, load variations can be considered using $\gamma = 1.35$.


In the SwRI tests, the mean ultimate pullout force for the connectors was found to be 1642 lbs. The minimum mean ultimate shear capacity of the connectors was found to be 513 lbs with a standard deviation, based on five tests, of 22.6 lbs.

Based on § IV of ICBO ES AC 15, the characteristic strengths for the connectors are determined as follows:

1) For pullout capacity, which is governed by concrete failure, the design value is given by

$$\text{Design Value} \leq \text{(average maximum strength)} \times (1/\text{SF})$$

where $\text{SF} = 1.7 = \text{the strength design Safety Factor for concrete}$. This safety factor is appropriate since pullout capacity is limited by the concrete strength. Also, although the tension forces induced by environmental conditions are relatively small, a high safety factor in tension forces will provide a greater overall safety factor for combined loading. From the SwRI tests (Southwest Research 2000) the average maximum tension strength, $Z_m$, is 1640 lbs.
The Design Tension Strength, $\phi Z_A$, is therefore given by

$$\phi Z_A \leq 1640 / 1.7 = 965 \text{ lbs} = 4290 \text{ N}$$

2) The shear capacity is governed by the connector shear capacity. The design value is therefore calculated using the safety index procedure, where

Design Value $\leq \text{(average maximum strength)} \times \exp(-0.75BV)$

where $B =$ Safety Index $= 3.5$, $V = CV_i \geq 0.10$, $V_i =$ Coefficient of variation of the average maximum strength, $C = 2.0 - 0.1n$, with $V = CV_i \geq 0.10 =$ number of identical specimens. The value of $n$ for this equation may range from 5 to 10. In this case, $n = 5$.

The SwRI tests provided an average maximum shear strength $= Q_m = 513$ lbs (2282 N) and a standard deviation based on five tests $(n = 5) = \sigma = 22.6$ lbs (101 N). $C = 1.5$ and $V_i = \frac{\sigma}{Q_m} = \frac{22.6}{513} = 0.044$. Therefore, $V = 0.1$, and the Design Shear Value, $\phi Q_{AG}$, is given by

$$\phi Q_{AG} \leq 513 \times \exp(-0.75 \cdot 3.5 \cdot 0.1) = 513 \cdot 0.769 = 395 \text{ lbs} = 1755 \text{ N}$$

**B. 1997 Uniform Building Code**

**1. Seismic Forces**

Per UBC §1633.2.4.2, exterior nonbearing non-shear wall elements shall be designed to resist the forces per Formula (32-1) or (32-2). Per §1633.2.4.2.5, fasteners shall be designed for the forces determined by Formula (32-2) using $R_p = \text{the component response modification factor} = 1.0$, and $a_p = \text{the in-structure component amplification factor} = 1.0$.

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r}\right) W_p \quad \text{(UBC Formula (32-2))}$$

except that

$$F_p \geq 0.7 C_a I_p W_p \quad \text{and}$$

$$F_p \leq 4 C_a I_p W_p \quad \text{(UBC Formula (32-3))}$$

where $C_a = \text{the seismic coefficient per UBC Table 16-Q}$, $I_p = \text{the importance factor per UBC Table 16-K}$, $h_x = \text{the element attachment elevation with respect to grade}$, $h_r = \text{the structure roof elevation with respect to grade}$, and $W_p = \text{the weight of the component}$. For sandwich wall panels with distributed connectors, $W_p$ is a distributed load over the area of
the panel. In Seismic Zone 4, $C_a$ can be a function of $N_a$, the near-source factor per UBC Table 16-S.

It should be noted that typical wall panel connections as envisioned in §1633.2.4.2.5 have little or no redundancy and must survive enforced compatibility between a flexible building structure and a stiff wall element. In contrast, sandwich panel connectors are highly redundant and seismic loads induced in sandwich panel connectors are, in fact, internal forces. It could therefore be argued that sandwich panel connectors should not be held to the same standard as wall panel connections and that $R_p = 3.0$ is more appropriate. However, for purposes of this report, the conservative $R_p = 1.0$ is used.

For most buildings, $N_a = 1.0$ and $I_p = 1.0$. Using these values and using $C_a \leq 0.44 N_a$, the seismic force can be calculated for typical panels.

The seismic force applied to a vertically spanning wall panel is the average of the forces calculated for the base and roof elevations. The force at the base elevation is the larger of Formula (32-2), using $h_s = 0$ and $h_r = 1$ and Formula (32-3), using $F_p = 0.7 C_a I_p W_p$. The force at the roof elevation is the smaller of Formula (32-2), using $h_s = 1$ and $h_r = 1$ and Formula (32-3), using $F_p = 4 C_a I_p W_p$.

The largest seismic shear forces will be induced in the connectors at the upper and lower ends of such a sandwich panel and will result when the two concrete layers are accelerated in phase, normal to the plane of the panel (in effect, $W_p = \text{the weight of both concrete layers are accelerated}$). No seismic tension force will be induced in the connectors under this condition. The largest seismic tension forces in the connectors (at any location in the panels) will be induced when the two concrete layers are accelerated out of phase, normal to the plane of the panel. For calculation of tension forces, $W_p$ will be the weight of the thinnest concrete layer (since the bending stiffness of the thinner layer will not be sufficient to resist acceleration of the thicker layer). A shear force will be induced in the connectors under this condition, but the magnitude will be proportional to the difference in the weights of the thickest and thinnest concrete layers, or $(d_2-d_1)ab$.

2. Wind Forces

Per UBC §1620, building elements shall be designed to resist the design wind pressures in accordance with Formula (20-1).

$$P = C_e C_q q_s I_w \quad \text{(UBC Formula (20-1))}$$

where $C_e = \text{the combined height, exposure and gust factor coefficient per UBC Table 16-G}$, $C_q = \text{the pressure coefficient per UBC Table 16-H and can conservatively be taken as 1.2 for a corner panel with a tributary area greater than 100 ft}^2$, $q_s = \text{the wind stagnation pressure at a height of 33 ft per UBC Table 16-F}$, and $I_w = \text{the wind importance factor per UBC Table 16-K}$.

3. Temperature Induced Loading

In U.S. practice, temperature loading is typically left up to the discretion of the engineer of record. In order to maintain consistency with the German approval (Deutches Institut (1993, 1999)), it is recommended that a temperature differential (between layers) of 30K and a temperature gradient of 5K is used for typical panel configurations. For
applications in which the interior design temperature indicates larger temperature differential is possible (for example, in cold storage buildings), it is recommended that differentials be calculated based on the design interior temperature and the average maximum monthly temperature for the building site.

C. Analysis

1. Long-Term Shear Loading

For long-term (gravity induced) shear forces alone, the design strengths per the German approval are used. These design strengths incorporate both strength reduction factors and load factors. Therefore no load factors are applied to long-term dead load calculation. That is, the allowable loads are compared directly with the weight of the face layer.

The permissible gravity load on an individual connector is a function of the insulation thickness. The maximum value is 350 N for a 40 mm insulation layer and the minimum value is 200 N for a 100 mm insulation layer. Intermediate values can be interpolated.

Table 2 provides the maximum thickness of the face layer of concrete for a given insulation thickness based on a connector spacing of 406 mm (16 in.) in each direction. For a given insulation thickness, if the face layer thickness exceeds the maximum indicated value, the connector spacing must be reduced.

<table>
<thead>
<tr>
<th>Insulation Thickness, e, mm (in.)</th>
<th>Maximum Face Layer Thickness, d, mm (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>38 (1.5)</td>
<td>89 (3.5)</td>
</tr>
<tr>
<td>51 (2)</td>
<td>76 (3)</td>
</tr>
<tr>
<td>76 (3)</td>
<td>63.5 (2.5)</td>
</tr>
<tr>
<td>102 (4)</td>
<td>51 (2.0)</td>
</tr>
</tbody>
</table>

The German approval is limited to insulation thickness up to 4 inches. In American practice, however, up to 10.5 inches of insulation has been used. In many applications, the combined insulation and connector system has been used to suspend the face layer, even with large insulation thickness.

In tilt-up panels, temperature-induced deflections are a function of the support condition for the face layer. For face layers supported by the connectors only and with backer rod and elastomeric sealant at the foundation, the face layer is allowed to expand and contract about the centroid of the panel surface. For face layers supported at the foundation elevation, all temperature-induced movement of the face layer is constrained at the base. Therefore, temperature-induced shear forces are double those that would occur with connector-only support. Wind and seismic-induced shear forces will not be affected by the gravity support condition (this will be shown with a stiffness method analysis).
Based on the German approval and successful application in the United States, it is proposed that two support conditions be considered. For thin insulation (less than 4 inches), it is recommended that the connectors carry the weight of the face layer. For thick insulation, (4 inches and greater), it will be acceptable to provide support at the base of the face layer.

Although it is common for designers to specify grout for base support, it must be noted that this provides not only vertical constraint, but also can provide constraint against horizontal movement of the face layer. Temperature or shrinkage-induced horizontal movement of the face layer will therefore be restricted—potentially causing vertical cracking of the face layer near the base of the panel. It is therefore recommended that the base support be provided using discrete shim stacks, a continuous joint-filler, or a continuous layer of insulation that can act as a bond breaker. Spandrel panels must have support provided by stiff shear connectors or by discrete ledge angles.

2. Short-Term Shear Loading—Closed-Form Analysis

Gastmeyer’s (1997) closed-form solution was developed for the calculation of the concrete forces induced by temperature and wind effects on sandwich panels (Fig. 12). With modification, this solution can be used to determine the connector shear forces induced by temperature and wind effects, as well as by seismic effects.

In the calculation of the internal layer and connector forces, the moments of inertia for the individual layers, \( I_f \), and the full section, \( I \) are required (note that \( I \) is often referred to as the composite moment of inertia for the sandwich panel). \( I_f \) and \( I \) are given by

\[
I_f = I_{f1} + I_{f2}
\]

\[
= \frac{b}{12} \left( d_1^3 + d_2^3 \right)
\]

and

\[
I = \frac{b z^2 d_1 d_2}{d_1 + d_2} + I_f
\]

where \( b \) = the unit width of the panel (For convenience, \( b \), can be set as the connector spacing in the direction perpendicular to the span.), \( z \) = the distance between the centroidal axes of the two layers, and \( d_1 \) and \( d_2 \) = the thicknesses of the exterior and interior concrete layers, respectively (Fig. 12 and 13). The distance \( z \) is given by

\[
z = e + (d_1 + d_2)/2
\]

Additional parameters required are the areas of the layers, \( A_1 \) and \( A_2 \), and the distance from the centroidal axis for the external layer to the centroidal axis for the composite section, \( z_1 \). The areas will be given by

\[
A_1 = b d_1
\]

and
\[ A_2 = bd_2 \]  

The distance \( z_1 \) will be given by

\[ z_1 = \frac{zd_2}{d_1 + d_2} \]  

Note that the above relationships apply to flat concrete layers. Ribbed structural layers, with internal or external ribs, can also be modeled by using the correct relationships for the centroid locations, areas, and moments of inertia as required.

In the evaluation of concrete stresses, Gastmeyer proposed that the connector shear stiffness, \( C_{BB} \) (per Eq. 12) is a function of both the bending stiffness of the connector and the shear stiffness of the insulation material. The shear force carried in the insulation is limited by the sliding friction of the insulation across the concrete surface, with a coefficient of friction, \( \mu \), equal to 0.3.

However, the characteristic shears (design values) used for the connectors have been obtained from tests on isolated connectors and do not include the contribution of the insulation. Therefore, for calculation of the connector forces resulting from temperature effects and from forces applied normal to the plane of the panel, the connector force must be determined by subtracting the insulation component from the calculated total force. It is simpler, however, to directly calculate the connector force using the connector stiffness alone, \( C_B \), (which is simply a re-statement of Eq. 7):

\[ C_B = \frac{12 \cdot E_{ab} \cdot I_A}{d_A^3} \]  

The solution to the differential equations for the connector forces will be based on an auxiliary value, \( \omega \), given by

\[ \omega = \sqrt{\frac{C_B \cdot (1 + \frac{1}{A_1} + \frac{z^2}{A_2 + I_f})}{k \cdot E_B \cdot a \cdot A_1 \cdot z_I \cdot I_f}} = \sqrt{\frac{C_B \cdot \frac{z_I}{A_1}}{k \cdot E_B \cdot a \cdot \frac{z_I}{A_1} \cdot I_f}} \]  

The shear forces induced in the connectors by normal wind or seismic forces are given by

\[ Q_{AK} = A_A \cdot K \cdot \left( \frac{L}{2} - x_H \right) \cdot \frac{A_1 \cdot z_I}{I} \cdot \left[ 1 - \frac{\sinh \left( \frac{\omega}{2} \left( \frac{L}{2} - x_H \right) \right)}{\omega \cdot \left( \frac{L}{2} - x_H \right) \cdot \cosh \left( \frac{\omega}{2} \right)} \right] \]  

where \( K \) = the uniformly distributed wind or seismic force applied normal to the plane of the panel, in N/m² or in psf (the subscript “AK” is replaced with “AW” for wind and “AE” for earthquake), \( L \) = the panel span, \( A_f \) = the tributary area surrounding each connector = \( a \cdot b \) and \( x_H \) = the distance from the panel support to the critical connector.

**Thermomass® Connector System for Concrete Sandwich Walls**
The shear forces induced in the connectors by temperature differential between the concrete layers are given by
\[
Q_{A\Delta\theta} = b \cdot \alpha \cdot \Delta \theta \cdot k \cdot E \cdot \frac{(A_1 \cdot z_i \cdot I_f)}{z \cdot I} \cdot \omega \left( \frac{\sinh \left( \frac{\omega \cdot (L - x_h)}{2} \right)}{\cosh \left( \frac{\omega \cdot L}{2} \right)} \right)
\]

The shear force induced in the connectors by the weight of the face layer is given by
\[
Q_{Ag} = A \cdot d_1 \cdot \gamma_c
\]

The shear forces induced by normal forces and by temperature differential are functions of the connector location within the span of the panel and will be maximum at the ends of the panel. For simplicity, \(x_H\) is assumed to be equal to \(a/2\). The shear forces induced by the weight of the face layer are distributed equally between the connectors.

Fig. 12. Differential Element of Sandwich Panel (Gastmeyer 1997)
3. Short-Term Shear Loading—Stiffness Method Analysis

Using the stiffness method, the support condition at the base of a panel can be modeled using a compression spring. In order to model support at the foundation, the compression spring can be assigned a stiffness = 100000 N/mm. In order to model no support at the foundation, the compression spring can be assigned a stiffness = 0.1 N/mm.

The sandwich panel itself can be represented using a commercially available stiffness method program by creating elements representing the two concrete layers and the connectors (Fig. 14 and 15). The elements representing the connectors may be defined as having the same modulus of elasticity as used in the closed-form solution. However, the
element length will be the distance $z$. Therefore, the connector element moment of inertia used in the stiffness analysis, $I_{conn}$, must be set equal to

$$I_{conn} = \frac{z^3}{d^3} I_A$$  \hspace{1cm} (0)

Fig. 15. Stiffness Model Superimposed on Concrete Sandwich Panel.

In Table 3, results from a closed-form analysis are compared with results from a stiffness method analysis. The example panel is 10.15 m tall with a 64 mm face layer, 102 mm insulation, and 200 mm structural layer. It can be seen that the analysis methods show good agreement for panels with either type of face layer support.

Table 3. Maximum Connector Forces from Stiffness Model and Closed-Form Solution.

<table>
<thead>
<tr>
<th>Force Type</th>
<th>Stiffness Model</th>
<th>Closed-Form Solution</th>
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<td>Stiff Spring</td>
<td>Soft Spring</td>
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<tr>
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<tr>
<td>Temperature</td>
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</table>

Schematic panel models under various loading conditions are shown in Fig. 16-21. Fig. 16 and 17 show gravity loading with a soft spring support at the base (suspended face layer). Fig. 18 and 19 schematically show the effect of temperature differential for a
soft or stiff base support, respectively. Fig. 20 shows loading normal to the panel span, while Fig. 21 shows the deflected shape of the panel under normal loading.

Note that, although the temperature-induced connector (and therefore, force) is doubled by the provision of base support, the gravity-induced shear in vertically spanning panels is reduced to zero. Therefore, the total short-term shear is reduced. Also, even though the temperature-induced shear is doubled, the base support has no significant effect on the deformation of the panel under normal loading (Table 3 and Fig. 21). It is also important to note that for insulation thicknesses of 4 inches and greater, the increased bending length of the connector reduces the induced shear due to temperature and earthquake bending. The softer connector is therefore subject to lower temperature and seismic shear forces than would exist in panels with thinner insulation.

Fig. 16. Model with Gravity Loading on Face Layer.

Fig. 17. Deflected Shape of Model with Gravity Load on Face Layer and Soft Base Support.
Fig. 18. Deflected Shape of Model with Differential Temperature Loading (Soft Spring Support for Face Layer).

Fig. 19. Deflected Shape of Model with Differential Temperature Loading (Stiff Spring Support for Face Layer).

Fig. 20. Model with Uniformly Distributed Normal Loading.
4. Short-Term Loading—Load Combinations

Because the characteristic strengths for the connectors are developed using AC 15, short-term load factors per UBC §1612.2.1 are used (UBC 1997).

Sandwich panels can be installed with the panels in a vertical (commonly known as tilt-up panels) or in a horizontal span condition (typically known as spandrel panels). In tilt-up panels, the maximum connector shear will be the algebraic sum of the gravity-induced shear, the maximum temperature-induced shear, and the maximum wind or seismic-induced shear. In spandrel panels, the maximum connector shear will be the vector sum of the gravity, temperature, and wind/seismic shears.

Per §1612.2.1, load combinations are given by:

\[
Q_{A1} = 1.4Q_{Ag} + 1.2Q_{A\Delta \theta} \quad \text{(UBC Formula (12-1))} \\
Q_{A2} = 1.2Q_{Ag} + 0.8Q_{Aw} + 1.2Q_{A\Delta \theta} \quad \text{(UBC Formula (12-2))} \\
Q_{A3} = 1.2Q_{Ag} + Q_{AE} + 1.2Q_{A\Delta \theta} \quad \text{(UBC Formula (12-2))} \\
Q_{A4} = 1.2Q_{Ag} + \left(\frac{d_2 - d_1}{d_2 + d_1}\right)Q_{AE} + 1.2Q_{A\Delta \theta} \quad \text{(UBC Formula (12-2))}
\]

\[
Z_{A1} = 1.2Z_{A\Delta T} \quad \text{(UBC Formula (12-1))} \\
Z_{A2} = 0.8Z_{Aw} + 1.2Z_{A\Delta T} \quad \text{(UBC Formula (12-2))} \\
Z_{A3} = Z_{AE} + 1.2Z_{A\Delta T} \quad \text{(UBC Formula (12-2))}
\]

In this case, \(Z_{Aw}\) and \(Z_{AE}\) and the wind suction and seismic acceleration forces and are assumed to be applied to the face layer only. \(Z_{A\Delta T}\) is the tension force induced by local bending of the face layer when subjected to a temperature gradient of 5K (9R) through the thickness.
Combined shear and tension forces are limited using the linear interaction relationship:

\[ R = \frac{Q_A}{\phi Q_{AG}} + \frac{Z_A}{\phi Z_A} \leq 1.0 \]  

The linear interaction is checked for the following combinations of shear and tension:

\[ R_1 = \frac{\max(Q_{d1}, Q_{d2})}{\phi Q_{AG}} + \frac{Z_{A2}}{\phi Z_A} \]  

\[ R_2 = \frac{Q_{d3}}{\phi Q_{AG}} + \frac{Z_{A1}}{\phi Z_A} \]  

and

\[ R_3 = \frac{Q_{d4}}{\phi Q_{AG}} + \frac{Z_{A3}}{\phi Z_A} \]  

5. Short-Term Loading—Example Analyses

Example analysis results are summarized in Table 3. For simplicity, all wind loads were based on 110 mph (49 m/s) wind speeds, Exposure D, and 40 ft panel height. All seismic loads were calculated using \( a_p = 1.0, R_p = 1.0, N_a = 1.0, I_p = 1.0, \) and \( C_a = 0.44 N_a \).

The results show that, for common panel configurations, the connector spacing can be maintained at 406 by 406 mm (16 in. by 16 in.). For example, a 10 m tall panel with a face layer of thickness 63.5 (2.5 in.) has a controlling interaction value \( R_2 = 0.95 \) when used with 51 mm insulation (2 in.). Interestingly, insulation thickness has a dramatic effect on connector shear. For example, a 14 m (46 ft) tall panel with the same face layer thickness has a controlling interaction value \( R_2 = 0.95 \) when used with 63.5 mm (2.5 in.) insulation. Further, a 14 m panel with a 76 mm (3 in.) face layer has a controlling interaction value \( R_2 = 1.00 \) and would still be appropriate. In contrast, as the insulation thickness decreases, the effect of temperature on the connector becomes significant. For example, a 5 m panel with a 51 mm face layer thickness and 38 mm insulation has a controlling \( R_2 = 0.88 \) with a 406 mm by 406 mm connector spacing.

Example calculations for the first panel in Table 4 are included as an Appendix.
Table 4. Example Results for Tilt-Up Panel Analyses.

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<th>$a$ mm</th>
<th>$b$ mm</th>
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<th>$E$ kPa</th>
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References


Appendix—Example Calculations

**Thermomass® With UBC '97 Thermal, Wind, and Seismic Forces**

**Span**

\[ L := 12 \text{ m} \quad L = 39.37 \text{ ft} \]

**Anchor grid field**

- Vertical spacing \[ a := 406 \text{ mm} \quad a = 15.984 \text{ in} \]
- Horizontal spacing \[ b := 306 \text{ mm} \quad b = 12.047 \text{ in} \]

**Layer Data**

- Facing layer thickness \[ d_1 := 63.5 \text{ mm} \quad d_1 = 2.5 \text{ in} \]
- Structural layer thickness \[ d_2 := \frac{L}{50} \quad d_2 = 0.24 \text{ m} \quad d_2 = 9.449 \text{ in} \]
- Insulation layer thickness \[ d_D := 51 \text{ mm} \quad d_D = 2.008 \text{ in} \]

Note that \( e \) is a reserved variable in MathCAD. \( d_D \) is therefore used in its place.

**Wind Data**

- Basic Wind Speed per Fig. 16-1 \[ S := 110 \text{ mph} \quad S = 177 \frac{\text{km}}{\text{hr}} \]

**Seismic Data**

- Coefficient per Table 16-Q \[ C_a := 1.0 \]

- \[ N_a := 1.0 \]

- \[ C_a := 0.44 N_a \]

- \[ R_p := 1.0 \]

- \[ a_p := 1.0 \]

- \[ I_p := 1.0 \]

**Temperature Data**

- Temperature difference between concrete layers \[ \Delta \theta := 30 \text{K} \quad \Delta \theta = 54 \text{R} \]

- Temperature gradient across the facing layer \[ \Delta T := 5 \text{K} \quad \Delta T = 9 \text{R} \]

**Miscellaneous Data**

- Reduction factor for effect of cracking on concrete stiffness (\( k=1 \) for uncracked members).

- \[ k := 1 \]

- Distance from end of panel to anchor with maximum shear

\[ X_H := 203 \text{ mm} \quad X_H = 7.992 \text{ in} \]

**Face wythe support**

- Include facing layer weight in total shear calculations? \[ W\text{flag} := 1 \]

- Input \( W\text{flag} = 0 \) for No (base supported facing layer).

- Input \( W\text{flag} = 1 \) for Yes (suspended facing layer).

- Is the wind span in the vertical direction?

\[ S\text{flag} := 1 \]

- Input \( S\text{flag} = 0 \) for horizontal span.

- Input \( S\text{flag} = 1 \) for vertical span.
Concrete layer properties

Coefficient of thermal expansion
\[ \alpha_T := 10^{-5} \text{K}^{-1} \quad \alpha_T = 5.56 \times 10^{-6} \text{R}^{-1} \]

Elastic modulus
\[ E_b := 34000 \frac{\text{N}}{\text{mm}^2} \quad E_b = 4.931 \times 10^6 \frac{\text{lbf}}{\text{in}^2} \]

Unit weight
\[ \gamma_b := 24\,1000 \frac{\text{N}}{\text{m}^3} \quad \gamma_b = 153 \frac{\text{lbf}}{\text{ft}^3} \]

Strength
\[ f_b := 35 \frac{\text{N}}{\text{mm}^2} \quad f_b = 5.076 \times 10^3 \frac{\text{lbf}}{\text{in}^2} \]

Insulation layer properties

Friction coefficient
\[ \mu = 0.3 \]

Short-term elastic modulus
\[ E_D := 9 \frac{\text{N}}{\text{mm}^2} \quad E_D = 1305 \frac{\text{lbf}}{\text{in}^2} \]

Short-term shear modulus
\[ G_D := 4 \frac{\text{N}}{\text{mm}^2} \quad G_D = 580 \frac{\text{lbf}}{\text{in}^2} \]

Anchor properties

Bending elastic modulus
\[ E_{Ab} := 30000 \frac{\text{N}}{\text{mm}^2} \quad E_{Ab} = 4.35 \times 10^6 \frac{\text{lbf}}{\text{in}^2} \]

Tensile elastic modulus:
\[ E_{Az} := 40000 \frac{\text{N}}{\text{mm}^2} \quad E_{Az} = 5.8 \times 10^6 \frac{\text{lbf}}{\text{in}^2} \]

Cross sectional area
\[ A := 50.5 \text{mm}^2 \quad A = 0.078 \text{in}^2 \]

Embedment
\[ d_e := \begin{cases} 51\text{mm} & \text{if} \quad d_I \geq 63.5\text{mm} \\ 38\text{mm} & \text{otherwise} \end{cases} \quad d_e = 51\text{mm} \]

Average moment of inertia
\[ I_A = 243.5 \text{mm}^4 \quad I_A = 5.85 \times 10^{-4} \text{in}^4 \]

Anchor grid area
\[ A_A := a \cdot b \quad A_A = 1.242 \times 10^4 \text{mm}^2 \]

Anchor bending length
\[ d_A := d_D + \frac{2d_e}{3} \left(1 - \frac{1}{1 + \frac{d_e}{d_D}} \right) \quad d_A = 68\text{mm} \]
**Wind Loads Per the Uniform Building Code**

**Combined Height, Exposure, and Gust Factor Coefficient Per Table 16-G**

\[ C_e := 1.62 \]

**Pressure Coefficient for Corner Panel with Tributary Area > 100 square feet per Table 16-H**

\[ C_q := 1.5 - 0.3 \]

**Wind Importance Factor per Table 16-K**

\[ I_w := 1.0 \]

**Wind Stagnation Pressure**

\[ q_s := 0.613 \frac{kg}{m^3} \cdot \frac{S^2}{m^2} \]

\[ q_s = 1 \times 10^3 \frac{N}{m^2} \]

\[ q_s = 31. \cdot \]

*Re: ASCE 7*

**Wind suction per Eq. (20-1)**

\[ w := C_c C_q q_s I_w \]

\[ w = 2.882 \times 10^3 \frac{N}{m^2} \]

\[ w = 60.2 \]

**Seismic Loads Per the Uniform Building Code**

**Unit weight of both concrete layers**

\[ W_p := (d_1 + d_2) \cdot \gamma_b \]

\[ W_p = 152. \]

**Seismic Force per 1632.2 (Function)**

\[ F_p[a_p, C_a, I_p, R_p, h_x, h_r, W_p] := \frac{a_p C_a I_p}{R_p} \left( 1 + 3 \frac{h_x}{h_r} \right) W_p \]

**Min/Max Seismic Force per 1632.2 (Function)**

\[ F_{pM}[K, C_a, I_p, W_p] := K C_a I_p W_p \]

**Force at Base of wall:**

\[ h_x := 0 \]

\[ h_x := 1 \]

\[ F_{po} := F_p[a_p, C_a, I_p, R_p, h_x, h_r, W_p] \]

\[ F_{po} = 67 \frac{lbf}{ft^2} \]

**Force at Roof:**

\[ h_x := 1 \]

\[ F_{pr} := F_p[a_p, C_a, I_p, R_p, h_x, h_r, W_p] \]

\[ F_{pr} = 268 \frac{lbf}{ft^2} \]

**Min/Max Seismic Force**

\[ F_{pMIN} := F_{pM}[0.7, C_a, I_p, W_p] \]

\[ F_{pMAX} := F_{pM}[4, C_a, I_p, W_p] \]

**Mean Seismic Force**

\[ E := 0.5 \left( \max[F_{po}, F_{pMIN}] + \min[F_{pr}, F_{pMAX}] \right) \]

\[ E = 8012 \frac{N}{m^2} \]

\[ E = 167 \frac{lbf}{ft^2} \]
Permissible Shear Force under long-term effects (per Zulassung)

\[d_i := \begin{cases} 40 \text{ mm} \\ 100 \text{ mm} \end{cases} \quad Q_j := \begin{cases} 350 \text{ N} \\ 200 \text{ N} \end{cases}\]

\[zulQ_{AD} := \text{interp}[d_i, Q_j, d_i]\]

\[zulQ_{AD} = 323 \text{ N} \quad zulQ_{AD} = 73 \text{ lbf}\]

Shear load on anchor due to weight of facing layer

\[Q_{ag} := f\left[ W_{lag}, a \cdot b \cdot d_i, \gamma_b, 0 \text{ N} \right]\]

\[Q_{ag} = 189 \text{ N} \quad Q_{ag} = 43 \text{ lbf}\]

Test 1 := if\{Q_{ag} < zulQ_{AD} \text{ "OK", "NG"}\} \quad \text{Test 1 = "OK"}

Characteristic Strengths (Design Loads) based on Section IV of AC 15.

Design Shear Load Under Short Term Combined Effects

\[Q_m := 513 \text{ lbf} \quad Q_m = 2282 \text{ N}\]

\[n := 5 \quad B := 3.5\]

\[SD := 22.6 \text{ lbf}\]

\[V_i := \frac{SD}{Q_m} \quad V_i = 0.044 \quad SD = 101 \text{ N}\]

\[C := 2.0 - 0.1 \cdot n \quad C = 1.5\]

\[V := \max\{C \cdot V_i, 0.1\} \quad V = 0.1\]

\[\phi Q_{AG} := Q_{ag} e^{(-0.75B \cdot V)} \quad \phi Q_{AG} = 1755 \text{ N} \quad \phi Q_{AG} = 395 \text{ lbf}\]

Design Tension Force Under Short Term Combined Effects

\[\phi Z_A := \frac{1640 \text{ lbf}}{1.7} \quad \phi Z_A = 4291 \text{ N} \quad \phi Z_A = 965 \text{ lbf}\]
**Cross-sectional Values**

**Distance between concrete layer centroids**

\[ z := d_D + \frac{d_1 + d_2}{2} \]

\[ z = 202.75 \text{ mm} \]

**Facing layer Moment of Inertia**

\[ I_1 := \frac{b}{12} \left( d_1^3 \right) \]

**Structural layer Moment of Inertia**

\[ I_2 := \frac{b}{12} \left( d_2^3 \right) \]

**Sum of layer moments of inertia**

\[ I_f := I_1 + I_2 \]

\[ I_f = 3.59 \times 10^8 \text{ mm}^4 \]

**Composite moment of inertia**

\[ I := \frac{b \cdot z^2 \cdot d_1 \cdot d_2}{d_1 + d_2} + I_f \]

\[ I = 9.907 \times 10^8 \text{ mm}^4 \]

**Unit Area for facing layer**

\[ A_1 := b \cdot d_1 \]

\[ A_1 = 1.943 \times 10^4 \text{ mm}^2 \]

**Unit Area for structural layer**

\[ A_2 := b \cdot d_2 \]

\[ A_2 = 7.344 \times 10^4 \text{ mm}^2 \]

**Distance from composite centroid to centroid of facing layer**

\[ z_1 := \frac{z \cdot d_2}{d_1 + d_2} \]

\[ z_1 = 160.329 \text{ mm} \]
**Anchor Tensile Force**

**Tensile rigidity**

\[ C_Z := \frac{E_A}{d_D + d_e} \]

\[ C_Z = 1.98 \times 10^4 \frac{N}{\text{mm}} \]

**Tension from temperature gradient in facing layer**

\[ Z_{\Delta T} := \frac{\alpha_T \Delta T}{8 \cdot b + 6a \cdot d_1} + \frac{\alpha_T \Delta T}{C_Z \cdot b \cdot A_A} \]

\[ Z_{\Delta T} = 458 \text{ N} \]

**Maximum Anchor Shear Force**

(for calculation of connector stiffness)

**Reduced total shear**

\[ Q_{AG} := \phi Q_{AG} \left( 1 - \frac{Z_{\Delta T}}{\phi A} \right) \]

\[ Q_{AG} = 1568 \text{ N} \]

**Shear rigidity of connector only for calculation of connector shear forces.**

\[ C_B := \frac{12 \cdot E_{Ab} \cdot I_A}{d_1^3} \]

\[ C_B = 279 \frac{N}{\text{mm}} \]

**Auxiliary value**

\[ \omega = \frac{C_B}{k \cdot E_{eb} \cdot a} \left( \frac{1}{A_1} + \frac{1}{A_2} + \frac{z^2}{I_f} \right) \]

\[ \omega = 0.0602 \text{ m}^{-1} \]

**Alternate Statement for Auxiliary value**

\[ \omega = \frac{C_B}{k \cdot E_{eb} \cdot a} \left( \frac{z I}{z^2 A_1' I_f} \right) \]

\[ \omega = 0.0602 \text{ m}^{-1} \]
Anchor Shear Force

Wind suction shear in anchor at location $X_H$ from end of panel

$$Q_{Aw} := A_w E \left( \frac{L}{2} - X_H \right) \frac{A_f z_j}{I} \left[ 1 - \frac{\sinh \left( \frac{\omega \left( \frac{L}{2} - X_H \right)}{2} \right)}{\omega \cosh \left( \frac{\omega L}{2} \right)} \right]$$

$Q_{Aw} = 279 \text{ N}$

Temperature-differential shear in anchor at location $X_H$ from end of panel

$$Q_{A\Delta\theta} := b \cdot \alpha_f \cdot \Delta \theta \cdot k \cdot E_b \cdot \frac{A_f z_j I}{z I} \left[ \frac{\sinh \left( \frac{\omega \left( \frac{L}{2} - X_H \right)}{2} \right)}{\omega \cosh \left( \frac{\omega L}{2} \right)} \right]$$

$Q_{A\Delta\theta} = 350 \text{ N}$

Double $Q_{A\Delta\theta}$ for base-supported face in vertical-span panel

$$Q_{A\Delta\theta} := \text{if} \left( W\text{flag}, Q_{A\Delta\theta}, \text{if} \left( S\text{flag}, 2 \cdot Q_{A\Delta\theta}, Q_{A\Delta\theta} \right) \right)$$

$Q_{A\Delta\theta} = 350 \text{ N}$

Weight of facing layer induced shear at every connector

$$Q_{Ag} := \text{if} \left( W\text{flag}, A_w d_1 \cdot \gamma_b, 0 \cdot N \right)$$

$Q_{Ag} = 189 \text{ N}$

Seismic-induced shear imposed by panel seismic force normal to plane of panel

$$Q_{AE} := A_E E \left( \frac{L}{2} - X_H \right) \frac{A_f z_j}{I} \left[ 1 - \frac{\sinh \left( \frac{\omega \left( \frac{L}{2} - X_H \right)}{2} \right)}{\omega \cosh \left( \frac{\omega L}{2} \right)} \right]$$

$Q_{AE} = 776 \text{ N}$
Factored Connector Shear Loads Per '97 UBC

Case 1--Full Gravity and Temperature on Panel

\[ Q_{A1} = \frac{S_{flag}}{1.4Q_{Ag} + 1.2Q_{A\Delta\theta}} \sqrt{(1.4Q_{Ag})^2 + (1.2Q_{A\Delta\theta})^2} \]

\[ Q_{A1} = 685 \text{ N} \quad Q_{A1} = 154 \text{ lbf} \]

Case 2--Full Gravity, Wind and Temperature on Panel

\[ Q_{A2} = \frac{S_{flag}}{1.2Q_{Ag} + 0.8Q_{Aw} + 1.2Q_{A\Delta\theta}} \sqrt{(1.2Q_{Ag})^2 + (0.8Q_{Aw} + 1.2Q_{A\Delta\theta})^2} \]

\[ Q_{A2} = 870 \text{ N} \quad Q_{A2} = 196 \text{ lbf} \]

Case 3--Full Gravity, Earthquake and Temperature on Panel

\[ Q_{A3} = \frac{S_{flag}}{1.2Q_{Ag} + Q_{AE} + 1.2Q_{A\Delta\theta}} \sqrt{(1.2Q_{Ag})^2 + (Q_{AE} + 1.2Q_{A\Delta\theta})^2} \]

\[ Q_{A3} = 1422 \text{ N} \quad Q_{A3} = 320 \text{ lbf} \]

Case 4--Full Gravity, Net Earthquake (Out of phase), and Temperature on Panel

(Net \( Q_{AE} = \text{Acceleration of Structural Wythe Less Acceleration of Face} \))

\[ Q_{A4} = \frac{S_{flag}}{1.2Q_{Ag} + Q_{AE} + 1.2Q_{A\Delta\theta}} \sqrt{(1.2Q_{Ag})^2 + (Q_{AE} + 1.2Q_{A\Delta\theta})^2} \]

\[ Q_{A4} = 1098 \text{ N} \quad Q_{A4} = 247 \text{ lbf} \]

Factored Connector Tension Loads Per '97 UBC

Case 1--Temperature Gradient in Face Wythe Only

\[ Z_{A1} = 1.2Z_{A\Delta T} \quad Z_{A1} = 550 \text{ N} \]

Case 2--Wind Suction + Temperature Gradient In Face Wythe

\[ Z_{Aw} = 1.13 \cdot w \cdot a \cdot b \]

\[ Z_{A2} = 0.8Z_{Aw} + 1.2Z_{A\Delta T} \quad Z_{A2} = 873 \text{ N} \quad Z_{A2} = 196 \text{ lbf} \]

Case 3--Earthquake "Suction" + Temperature Gradient In Face Wythe

\[ Z_{A3} = 1.0E \cdot a \cdot b \cdot \frac{d_1}{d_1 + d_2} + 1.2Z_{A\Delta T} \quad Z_{A3} = 758 \text{ N} \quad Z_{A3} = 170 \text{ lbf} \]
Wind–Combined Tension and Shear

\[ Q_A := \max (Q_{A1}, Q_{A2}) \quad Z_A := Z_{A2} \]

\[ Q_A = 870 \text{ N} \quad Z_A = 873 \text{ N} \]

\[ R_1 := \frac{Q_A}{\phi_{QAG}} + \frac{Z_A}{\phi_A} \]

\[ R_1 = 0.699 \]

Seismic–Combined Tension and Shear

Normal (face wythe and structural wythe accelerated in same direction)

\[ Q_A := Q_{A3} \quad Z_A := Z_{A1} \]

\[ Q_A = 1422 \text{ N} \quad Z_A = 550 \text{ N} \]

\[ R_2 := \frac{Q_A}{\phi_{QAG}} + \frac{Z_A}{\phi_A} \]

\[ R_2 = 0.939 \]

Out of phase (face wythe accelerated in opposite direction of structural wythe)

\[ Q_A := Q_{A4} \quad Z_A := Z_{A3} \]

\[ Q_A = 1098 \text{ N} \quad Z_A = 758 \text{ N} \]

\[ R_3 := \frac{Q_A}{\phi_{QAG}} + \frac{Z_A}{\phi_A} \]

\[ R_3 = 0.802 \]