building blocks

Precast/Prestressed Concrete Sandwich Walls

Information for the Structural Engineer...

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Precast sandwich wall panels have gained in popularity over the years, but some engineers are still not very familiar with them. Reasons for their increased use in place of more traditional materials include:

Speed of erection: A typical warehouse can be erected in a week. Cold weather is not a problem, since the panels are cast in a temperature-controlled environment and shipped to the site when needed.

Design flexibility: The casting procedure allows for a great variety of finishes and patterns, including inset brick and stone.

Thermal efficiency: Edge-to-edge, top-to-bottom rigid insulation can be used within a relatively thin wall, providing a high effective R value. The thermal mass of the concrete provides an added benefit by slowing heat transmission through the wall, flattening out temperature swings.

Competitive cost: Precast concrete walls can be a cost-effective alternative to masonry or tilt-up construction.

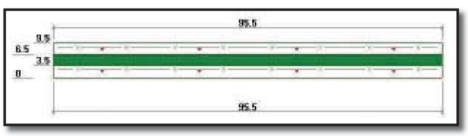

Figure 1: Typical sandwich wall section

Figure 2: Panels are cast in a long-line form

Figure 3: Panels are prestressed

What is a Precast Concrete Sandwich Wall Panel?

A typical panel has two outer concrete layers, or "wythes", separated by rigid insulation (*Figure 1*), and is cast in a long-line form in a plant (*Figure 2*). One or both wythes are usually prestressed to reduce cracking and improve performance (*Figure 3*). The panels are trucked to the job site and erected with a crane (Figure 4). Panels can carry roof and floor loads or just act as cladding (load-bearing vs. non-loadbearing, *Figure 5*).

There are different types of panel designs to consider:

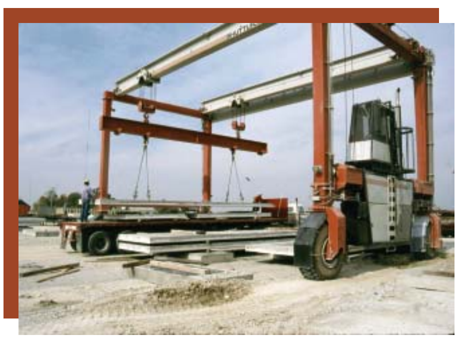

Figure 4a: Loading panels onto a trailer

Figure 4b: Non-bearing panels erected at the job site

Figure 5: Loadbearing panels carry a steel roof

Non-Composite

In non-composite panels, the concrete wythes act independently (*Figure 6a*). This design is used when a high insulation value is required, such as for a cooler or freezer building. The wythes are isolated by high-performance rigid insulation and are connected together solely by thermally non-conductive pin connectors (*Figure 6b*). The pins are proprietary, made of either a fiberglass and vinyl ester or polypropylene plastic or other non-conductive material.

Fully-Composite

In fully-composite panels, the wythes act together as a unit for full horizontal shear transfer. A typical composite panel is eight times stiffer, can take three times the stress without cracking and has twice the ultimate strength of a non-composite panel of similar thickness. Composite panels are typically less expensive to make than non-composite panels, can carry more loads and can be made taller and thinner. This is the most commonly used panel type. One drawback is that horizontal shear transfer between wythes is normally accomplished by adding zones of solid concrete and/or metal trusses (*Figure 6c*). These can create thermal bridges in the panel, reducing its effective "R" value. This is somewhat mitigated by the beneficial thermal mass effect of concrete, as noted earlier. A non-metallic truss system has also been recently developed which eliminates the thermal bridges, yet maintains composite action.

Composite panels often bow outward when exposed to direct sunlight, due to the temperature increase and subsequent expansion of the outer wythe. This characteristic is normal, but should be taken into account if panels are attached to, or butt up against, an intermediate floor near mid-height. On one project, the suspended ceiling track was attached directly to the precast panels at a southern exposure. When the sun hit the panels, they would bow out and ceiling tiles would fall out of the track. The tiles would be replaced, then fall out again the next day. This went on for a while until the precast manufacturer was called in and identified the problem.

Shrinkage cracking can occur in composite panels when one concrete wythe is more than twice as thick as the other wythe. The thin

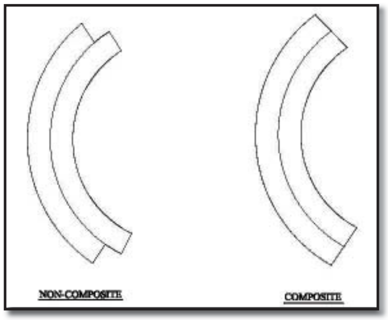

Figure 6a: Concrete wythe behavor

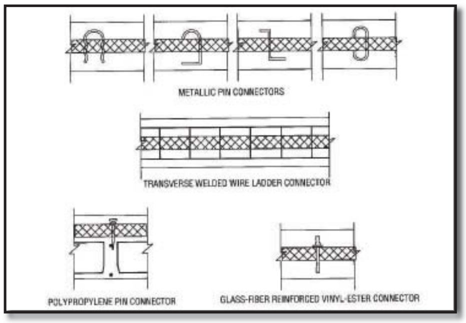

Figure 6b: Non-composite wythe connectors

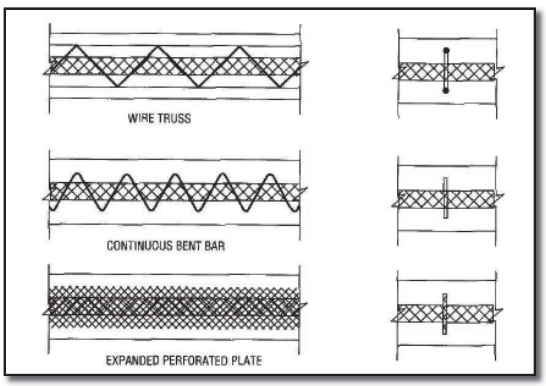

Figure 6c: Composite wythe connectors

wythe shrinks faster and acts like a canvas stretched on a frame. For this reason, wythes should be kept close to equal thickness for composite designs.

Partially Composite

Partially composite panels provide less than full shear transfer between wythes. They behave in a manner in-between composite and non-composite. The degree of composite action is determined by load tests performed by an independent testing lab. Proprietary partially-composite wall systems have recently become available which combine the high insulating value of non-composite panels with the strength and slenderness of composite panels. This is accomplished using nonconductive truss or bar connectors between the wythes for shear transfer. Partial composite action provides sufficient strength for most applications.

It is important to keep the distinctions between panel types in mind when determining a standard wall thickness for a particular project. Since the panels are cast in a form that is usually several hundred feet long, the thickness should ideally be the same for all the panels on a particular project, to avoid excessive setup costs. Occasionally a designer will select a panel thickness based on the assumption of full composite action, but then insist on thermal performance that can only be achieved with a thicker, more expensive, non-composite design. Work with local precast manufacturers to estimate a panel thickness for a particular project.

How are they Manufactured?

Solid Wet-Cast

The panel face that is exposed to weather is usually cast down-in-form. The outside face, therefore, has a steel form finish. The inside face can have a float finish (rough) or hard trowel finish (smooth). Reveals and form liners can be applied to the form to add architectural interest (*Figures 7, 8 and 9*). Projections from the face, such as a bullnose or cornice, are more difficult to achieve since the entire form has to be built up around the projection. An economical alternative is to cast the projection separately and connect it to the panel face later (*Figure 10*), or form it out of polystyrene insulation and Dryvit.

The panel width is limited to the precaster's form width, as well as shipping restrictions. Any panel more than 12 feet wide will require special handling (*Figure 11*), and panels wider than 14 feet can not be shipped easily by road, due to the lane width and minimum bridge height along the route.


Figure 7: Decorative reveal strips glued to form face


Figure 8: A formliner is used to create a pattern


Figure 9: Formliner and reveals used together

Hollow-Core Wet or Dry-Cast

The hollow-core wythe is placed down-in-form and is the main structural wythe for this non-composite system (*Figure 12*). The top wythe is thin, non-structural, and serves as

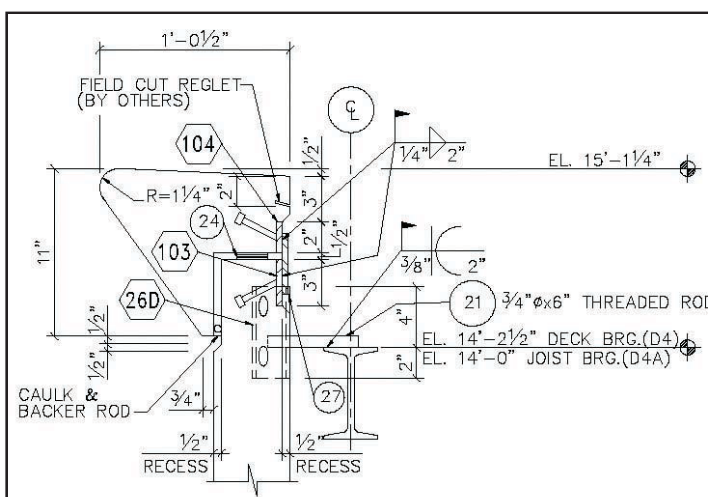

Figure 10: Bullnose cast separately

Figure 11: Extra-wide panel on a "Low-Boy" trailer

Figure 15: Temporary pipe braces anchored to "Deadmen"

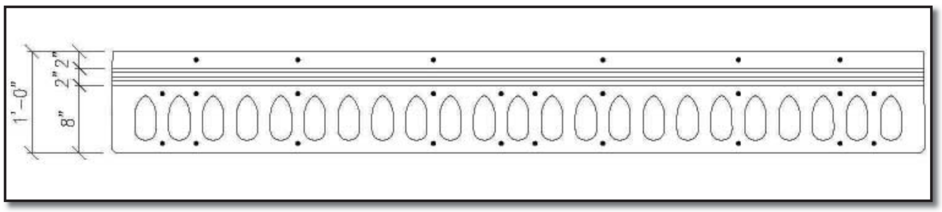

Figure 12: Hollow-core wall panel

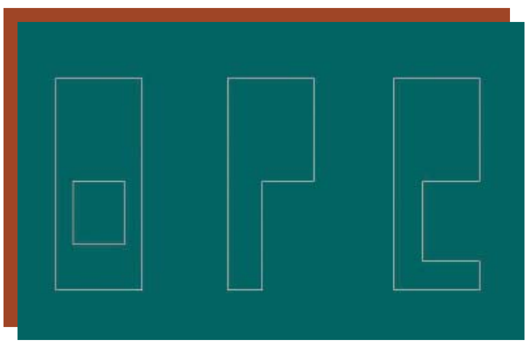

Figure 13: Punched opening Vs. "L" & "C" shaped panels

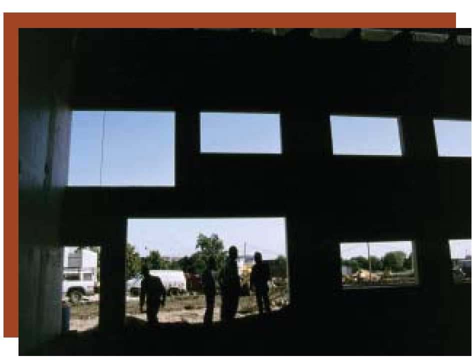

Figure 14: What were they thinking? How not to locate openings

the outside face, exposed to weather. Plastic pins anchor the top wythe to the bottom wythe. Wet-cast hollow-core uses inflatable diaphragms, or insulation, or gravel to create the voids. Dry-cast hollow-core requires a zero-slump concrete mix. The "plank" is extruded from a machine like pasta. Window and door openings are difficult to form with the dry-cast method, and these are usually sawcut in the field.

Manufacturing Tolerances

Specify PCI MNL 116 *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*. A common mistake is to specify PCI MNL 117, which is meant for architectural precast products. Precast wall panels are large and often load-bearing, and it is not practical to meet the fine tolerance standards in MNL 117. MNL 116 is the appropriate standard to use.

How are They Designed?

Design Responsibility

Due to the complexity of precast design and differences in standard products supplied by various precasters, the design responsibility for the precast components is usually taken by a licensed structural or professional engineer hired by the precaster.

If the building has precast exterior walls only, then the lateral load calculations are usually done by the Engineer of Record (EOR). He/she calculates and notes all loads applied to the panels on the contract Structural Set. The precast engineer is then responsible for the panel designs. For an all-precast building, such as a parking deck or prison, the lateral loads may be calculated by the precast engineer. The EOR would then design the foundation only, using loads provided by the precast engineer. The design responsibilities of the precast engineer should be spelled out in the General Structural Notes to avoid conflict.

Structural Planning

Use repetition to minimize the number of different forms required to produce the wall panels. Stick to a consistent panel width; 12-foot widths are usually most economical. Adjust the bay spacing to match a multiple of the panel width so that connections occur at the same place on the panels. For example, use 36- or 48-foot bays for 12-foot panels, and 40-foot bays for 10-foot panels. Panels with "punched" openings are easier to handle than L or C shaped panels (*Figure 13*). *Figure 14* is an example of how *not* to locate openings. The openings to the right are okay, but what's going on at the left?

Anticipate the erection sequence. It is preferable for the precast to be erected all at one time. If the precast erector has to come back later to place additional panels supported by steel framing, then it may be desirable to convert this steel framing to precast. Alternately, the precast erector may be able to erect a few steel beams to avoid another "move-in".

Handling Loads May be Most Critical

The forces a panel experiences when stripped from the form, then bounced around on the truck to the job site and then tripped up into place, usually exceed any in-place design loads due to gravity, wind or seismic. This is a good load test for a panel; if it survives the trip, it will surely do fine in service. Lifting inserts are placed in the panel face and edges at locations designed to minimize these stresses.

Brace Design

Temporary brace design is commonly based on ASCE 7 design loads, with the basic wind speed dialed down from a 50 year re-occurrence to a 5 year re-occurrence. Pipe braces have a factor of safety of 1.5 applied. The braces are usually anchored to augured concrete footings called "deadmen" (*Figure 15*) or, less often, to a continuous temporary strip footing. The deadmen resist uplift by their own weight plus some skin friction. In my experience, I have never heard of a deadman pulling out; though there have been instances where pipe braces have buckled due to extreme wind forces. In one case, vandals pulled out the pins used to adjust the brace length, sending the panels over.

Tabulate In-Place Loads

The precast engineer should consider the following load cases: Wind, seismic, gravity loads, earth pressure and differential temperature strains.

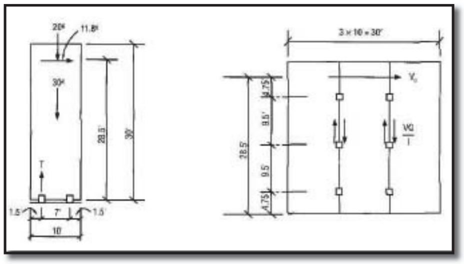

Figure 16: Shear wall connection design

Since these panels often exceed the maximum ACI height-thickness ratio of 25 to 1, slenderness effects need to be taken into account (also known as P-Delta effects). According to ACI 318, Section 14.2.7, a structural analysis is required. This involves adding the design loads, predicting the bow magnitude, then adding additional P-Delta moment due to the bow, and repeating until there is a convergence.

Check Stresses

There is usually a 7.5 to 12 $\sqrt{F'c}$ tension stress limit for service loads to minimize flexural cracking. The precast engineer needs to determine if the panel is cracked under factored load cases (tension exceeds 12 $\sqrt{F'c}$, the Modulus of Rupture). If so, then the Moment of Inertia should be reduced for the P-Delta analysis. Alternately, more prestressing strands can be added to control tension stresses.

Figure 17: Hung panels supported by bearing tubes & welded plates

Figure 18: Bearing tube cast into a panel

Check Deflections

According to ACI 318, Sect. 14.8.4, the maximum panel bow due to service load cases should be kept to less than L/150. The bow due to differential temperature strain should not be combined with the bow due to wind forces.

Ultimate Load Capacity

The precast engineer should use the appropriate ACI load combinations and factors, combined with a P-Delta analysis of secondary moments for load-bearing panels. In most cases the axial load, "Pu", will be very low on the interaction curve. What this means is that these panels act more like a beam on end instead of a column. Flexural tension almost always controls as the failure mode. According to ACI 318 Sect. 4.3.6 and 18.11.2.2, column ties are not required when flexural tension controls.

Figure 19: Continuors bearing angle for roof support

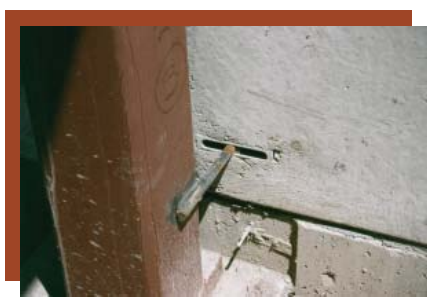

Figure 20: Slotted tie-back connections

Figure 21: Beam pockets replace exterior columns

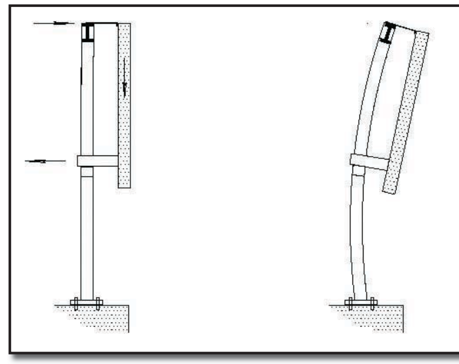

Figure 22: Flexible columns cause panel rotation

Shear Wall Design

Precast sandwich walls are often called upon to act as the shear walls for the building. As noted above, the lateral diaphragm loads to the panels should be supplied to the precast engineer by the Engineer of Record (with the possible exception being when the building is all-precast). The panel top connections should be capable of resisting at least 300 plf in shear or tension, as a minimum value.

Shear wall panels are usually designed to cantilever from their base (*Figure 16*). If this is not sufficient, several panels can be welded together to increase shear capacity. It is not good practice to con-nect more than a few panels together in a row, to avoid the possible buildup of shrinkage strains in the wall. For loadbearing walls, structural integrity provisions require that a minimum of two base connections be used per panel, and that each connection be capable of at least 10 kips nominal tensile strength (ACI 318 Sect. 16.5.1.3b).

If the prestress force in the walls is at least 225 psi, then no shear reinforcement is required, according to ACI 318 Sect. 18.11.2.3. If no prestress is provided, then a minimum transverse reinforcement ratio of 0.001 times the gross cross-sectional area of the panel should be used (ACI 318, Sect. 16.4.2).

Connection Design

Most precasters have a library of standard connections for use with AutoCAD. It helps to incorporate the standard connections used by the precasters who will be bidding a particular project. Many precasters will put a clause in their contract which gives them the option to substitute their standard connections for those shown on the structural contract set. These connections will be designed by the precast engineer. Some typical connections are shown in Figures *17 - 21*.

Continued on next page

Figure 23: Always use a cap flashing at the top of panel- It's missing here

Avoid These Problems

When hanging panels on steel framing, the stiffness of the steel framing should be designed to prevent excessive deflection or rotation. On one project, precast panels were attached to 4-inch wide steel tube columns, bearing on angle haunches. The tubes did not have enough stiffness to prevent the panels from rotating after erection (*Figure 22*). The gravity load reaction of the panel was eccentric to the column, which induced a moment that was apparently not considered in the column design. Diagonal braces were added to the columns to prevent further rotation.

Do not locate structural framing too close to the precast. Generally, at least one inch of space is required between the precast walls and other materials for erection tolerance. For non-loadbearing applications, steel column base plates will often interfere with the panels. The columns should be moved back sufficiently to allow the base plates to clear the panels.

Precast sandwich walls should always have a continuous cap flashing. Occasionally, a designer will show a precast sandwich wall exposed on top, without a flashing. Even though the insulation can be made to stop short of the top of panel, there will be a cold joint across the top panel edge which could allow water penetration (*Figure 23*). In addition, prestressing strands extending out the top of the panel are typically burned off and patched. The patches could come loose over time if not protected by flashing.▪

For Further Information

I recommend the Precast/Prestressed Concrete Institute report, *State-of-the Art of Precast/Prestressed Sandwich Wall Panels* (Publication JR 403, available at **www.PCI.org**).

Ed Losch is President of Losch Engineering Corp. in Palatine, Illinois. He is a licensed structural engineer and architect in Illinois, and a registered professional engineer in many other states. He is an active member of the Precast/Prestressed Concrete Institute, and is currently a member of the PCI Wall Panel Committee. He also served as co-editor and contributor to the PCI Precast Concrete Detailing Handbook.