Attachment E

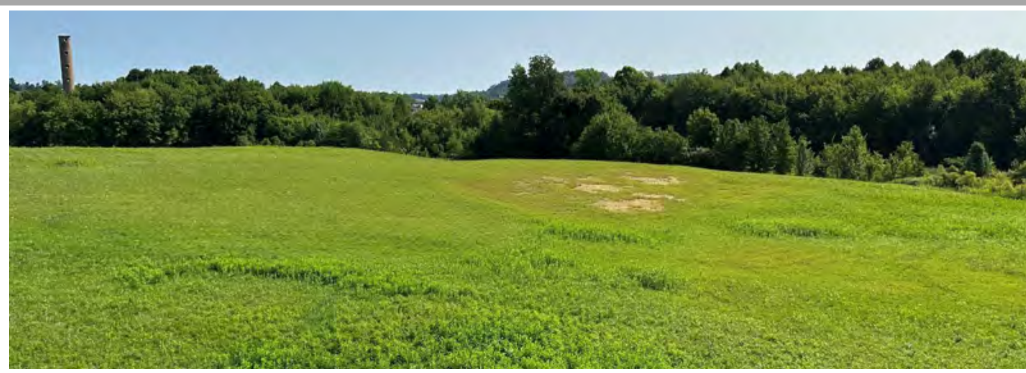
White | Pollard architects

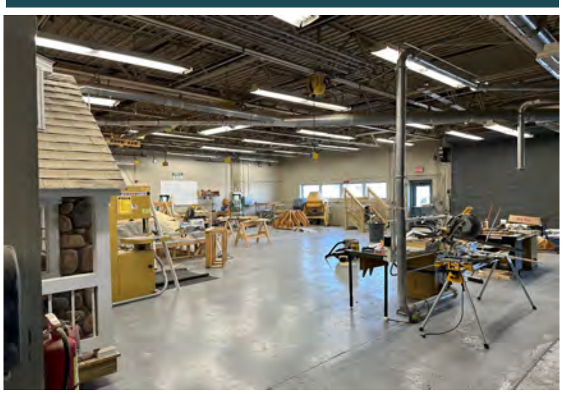
File #: 470-CAYX-SS70-00

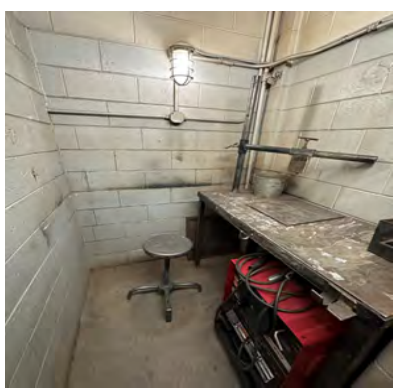
Final Report for

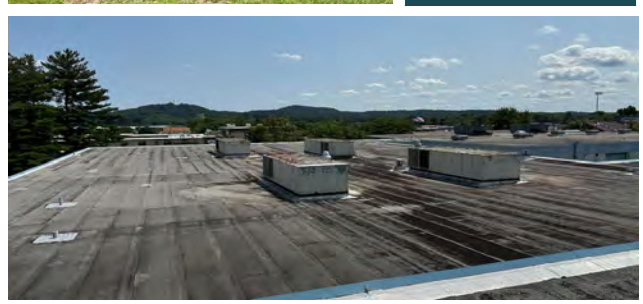
KCTCS - LAUREL SOUTH CAMPUS FEASIBILITY STUDY
SOMERSET COMMUNITY COLLEGE

KENTUCKY COMMUNITY AND TECHNICAL COLLEGE SYSTEM



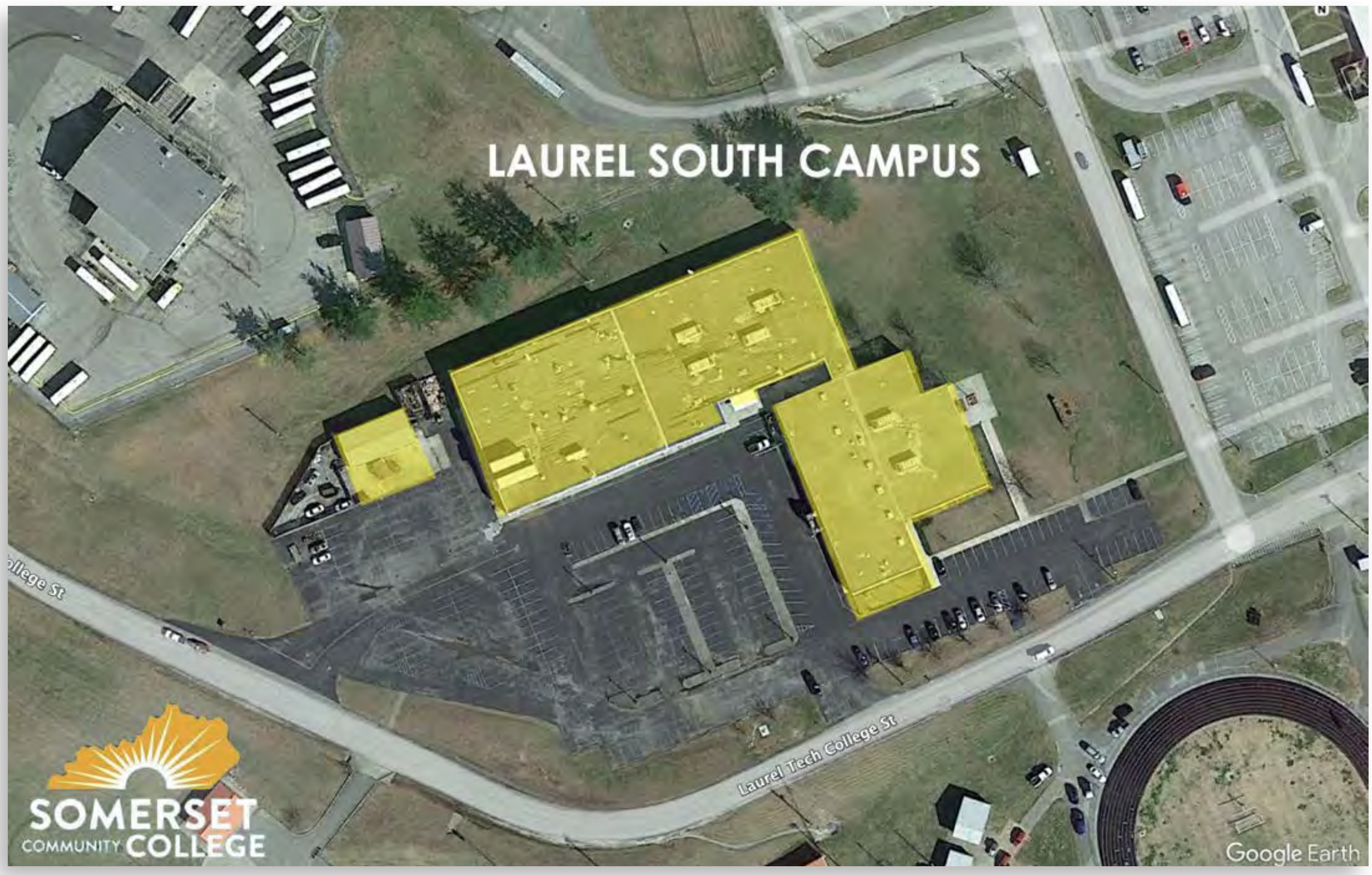






1 September 2023

WHITE | POLLARD architects, was contracted to provide an architectural survey of existing conditions at the Laurel South Campus of Somerset Community College WPa toured and documented the existing conditions on August 1, 2023. The 62,000 SF building at the Laurel South Campus (Building #1701) consists of an original masonry building built in the 1960s and a steel framed addition with masonry infill built in 1975. The masonry portions of the walls are white brick with CMU back-up. The facility currently houses several programs: Cosmetology, Criminal Justice, Auto Tech, Diesel Tech, Welding, HVAC, Carpentry, Electrical Construction, IMT and the Share and Care Center. Drawings of the building are included in Appendix A.



Existing Building Overview

The existing, roughly 62,000 SF, one-story building was constructed in the 1960s with an addition added in 1975. The exterior envelope is predominately white brick with a band of metal panels at the upper portion. The CMU back-up extends behind both the brick and metal panel and supports large trusses and a metal roof deck. Exterior windows are single-paned, steel showing signs of rust in many locations. Entry and secondary egress doors are hollow metal in hollow metal frames and are also rusted in many places. There are eleven large overhead coiling doors that are mostly uninsulated and in poor repair. The aging roof and mechanical systems are in need of replacement.

The roof is a modified bitumen system that is 23 years old. It has numerous water vapor blisters throughout the roof area and the granules are missing in large areas leading to degradation of the membrane where exposed to UV. The foil-faced flashing at the roof edges and mechanical curbs is damaged in many places and the roof edge itself is nearly flush with the roof. There are only four roof drains for the entire 1975 addition, a single drain over the connector, and nine for the original building. During our visit, WPa observed most of the drains were covered with pine needles and granules. It appears that these drains do not provide adequate drainage as we also observed clumps of pine needles washed up against the edge flashing around the perimeter of the roof where foil-faced coverings were damaged. This would provide ample opportunities for water to penetrate the wall cavities from the roof and allow the relief angles to rust (especially if those angles were not galvanized when installed). The roof replacement will require a full tear-off (membrane and existing insulation) exposing the metal deck. Per the Kentucky Building Code, this will require new insulation to be installed per the current IECC standards. This will increase the thickness of the roof system and require some re-design of the roof edge conditions to accommodate the increased thickness of the roof system. This will also affect the mechanical curbs as many of them are less than 8” tall currently.

During the site visit, WPa documented a consistent occurrence of instances where mortar was falling out of the wall in long pieces. Upon more detailed inspection, the phenomenon is occurring at rusted relief angles in the wall. This is due to water that is getting into the wall cavity (which is very small according to the drawings of the 1975 addition provided by the owner). There is also some deformation in the upper portions of the exterior walls at corners that is likely due to differential expansion and contraction – the original 1971 Building does not have any masonry expansion joints. Where the mortar or bricks are damaged the building is likely taking on more moisture which is affecting the wall integrity as well as the indoor air quality. Discoloration in horizontal bands at the relief angle locations supports this assumption.

The interior air is humid (57% - 60% measured during the site visit) and the interior finishes are suffering as a result. Of the nine restrooms in the building only two of them are accessible and all of them are undersized by current codes. Floors are a mix of VCT and carpet throughout and there is terrazzo in the corridors of the original building. Some VCT (particularly in the esthetics area is 8” tile and could contain asbestos. These should be tested. The acoustical ceiling tiles are sagging and discolored in most of the building spaces and many walls show signs of mildew and mold. Interior solid-core wood doors are in poor repair and have been modified repeatedly for changes in hardware. There is also inconsistency in door type throughout the building as many walls have been added through the years and doors have been added with whatever is available at the time. The interior needs to be repainted throughout and most floor finishes should be updated.

Throughout the building, lighting, lighting controls, HVAC systems and fire alarm controls and devices are outdated and inefficient. The rooftop units are over 40 years old and cannot keep up with the current needs of the building. While the building is equipped with an automatic fire suppression system, there have been numerous changes to the layout of the building spaces and to their uses so, the sprinkler system may not be providing the intended coverage in all areas at this time. This is also true of the HVAC design – while comparing the original construction documents to the observed spaces, there have been many alterations to the building plan over the years. When replacing the aging mechanical units, the ductwork layouts should be evaluated to ensure they are performing as intended.

The shop areas need upgraded exhaust systems. The welding, diesel tech, auto tech and carpentry areas need systems designed for those activities to ensure the safety of the students in these areas. The carpentry shop has an aging dust extraction system that needs to be replaced, the distribution of this system appears to be functioning as needed.

Boilers, Air Compressors and Generators are all in need of upgrades or replacement. The main switch gear is likely original to the building and should be replaced and brought up to current electrical codes.

In general, the building is dull, uninspiring and lacks of a clear entrance for visitors. A renovation of the building should include significant upgrades to the appearance and character of the building as well as repairs to aging elements. Fences and landscaping walls that are in poor condition should be repaired or replaced. The site also needs to have its 131,000 SF parking lot, with 262 parking spaces, resurfaced and re-striped. Any damaged curbs should be repaired and the design should prioritize accessibility.

As with many buildings constructed at that time, there are many issues with the design and construction that have led to the deterioration of the building. It was designed prior to the adoption of the Americans with Disabilities Act (ADA) and many features do not conform to the standards set forth in ANSI A117.1 Guidelines. Additionally, the building was designed before creation of energy conservation codes. There is little, if any, air-space or insulation in the exterior walls, the roof-top insulation is less than that required by the International Energy Conservation Code. The majority of restrooms are not accessible and the fixture count is significantly lower than that required by the 2017 Kentucky State Plumbing Law, Regulations and Code Book for higher education occupancies – especially in regard to the fixture counts for females (815 KAR 20:191 Minimum fixture requirements – Section 8).

Exterior Envelope

The exterior envelope presents several challenges and needs many improvements and repairs. Starting at the roof, which is showing clear signs that water is getting through the membrane, and including the drainage design of that roof which may be undersized or vulnerable to interference from nearby trees. If standing water is being blown to the roof edges where there are many damages to the flashing it is easy to understand how water is getting into the wall cavities. Those cavities are very small (a maximum of 3/8” between the 1½” rigid insulation and the back of the face brick) and can easily be blocked by mortar droppings or other debris. Without an adequate air space and weeps, moisture in the wall accumulates and damages bricks during freeze/thaw cycles, rusts unprotected steel lintels and angles, and moves through wall materials causing interior humidity and discoloration of exterior brick. In addition to the water issues at the perimeter, the existing windows and doors are in poor shape and should be replaced with more energy efficient elements.

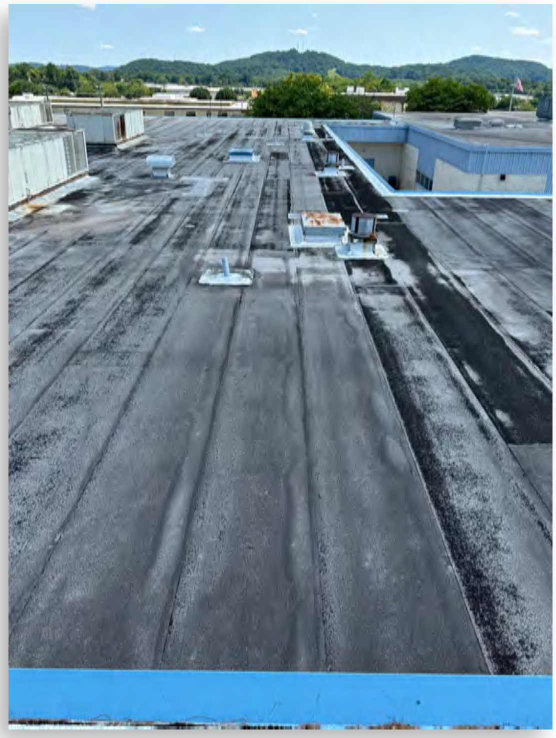
Roof Conditions

The current roof was installed in 2000 and is about 23 years old. There are no walk pads on the roof and there is significant loss of granules over the whole area. Flashings are compromised and the membrane is cracking in large areas.

During WPa's site visit, the roof drains were covered with pine needles and deep piles of roof granules. (this can be seen in the photo to the right.) Additionally, there are blisters in the roof membrane visibile beyond the drain.

This photo of the roof edge at the rear of the original 1971 building shows the cracking of the membrane along the roof edges where granules have been displaced (darker areas) and the edge flashing detail. The mechanical until seen on the ground below is the sawdust extraction system for the carpentry shop. This view also shows the condition of the existing parking lot.

In this image, looking down on the low bay portion of the building, the long blisters in the roof are clearly visible. These occur when moisture gets under the membrane and becomes vapor as the roof heats up. It is safe to assume that the insulation under the membrane is wet and no longer providing much R value to the building. A new roof will require a full tear-off.



This photo shows the expansion joint on the roof of the original building. The joint runs above the wall of the corridor separating the offices and classroom portion of the building from the Construction and HVAC shop areas on the back side of the building. In the corridor below there is a long crack in the terrazzo floor running the length of the corridor. It appears some settling has occurred beneath this wall.

The problem with the blocked roof drains was consistent on the roof of the 1975 addition. All four of the roof drains were covered in pine needles and surrounded by piles of granules. The discoloration of the roof where the granules have become dislodged, exposing the modified bitumen membrane to UV light is also apparent here. Note the mechanical curb under the unit in the background: with the addition of new roof insulation there will be little room to properly flash the curbs into the roof system. All roof penetrations will need to be extended to the new roof level.

The loss of roof granules is very evident where there is frequent foot traffic on the roof. The area around the roof access hatch is nearly bare. A variety of low mechanical curbs likely to be affected by the increased thickness of the roof system in the background can be seen.

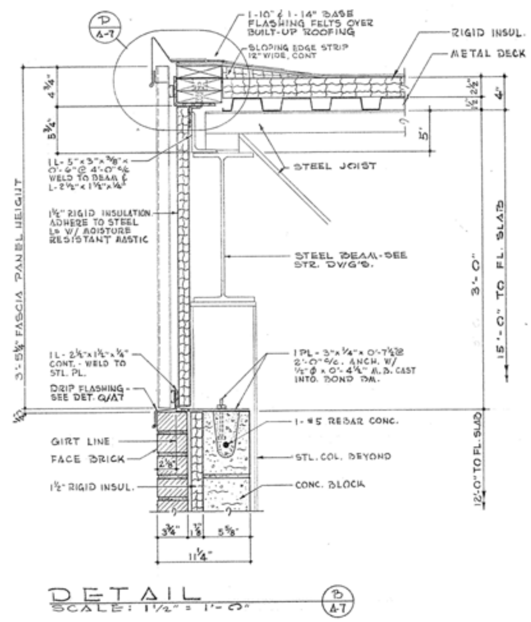
The foil-faced flashing at the roof perimeter is pulling up and damaged in many places. The gaps in the flashing where the material has stretched shown here, allow water to penetrate and enter the wall cavity

In other places, the flashing material is torn or missing. Here is appears to have snagged pine needles at the roof edge.

The damage to the foil-faced flashing is pervasive. WPa documented many conditions like the ones shown at roof edges, mechanical curbs and other features.

This overview of the original building's roof shows the state of the existing roof. Dark areas are places where the roof has lost granules. In the foreground extensive cracking of the membrane is visible. The low curbs on the mechanical unit on this portion of the roof is also visible. The roof drains on the original building do not seem to suffer from the presence of debris like the 1975 roof. The original building also has significantly more roof drains on its smaller area.

The detail at right is a typical roof detail from the construction documents for the 1975 addition. The detail shows the roof edge flashing and the 2½" of roof insulation called for in this system. Current IECC standards call for as much as 5" of continuous rigid insulation. It is also worth noting that the flashing is not designed to redirect water that has entered the wall system to the exterior. Any water that enters the wall from above will easily find its way into the wall cavity. (The construction documents have no details showing the installation of the relief angles.)



Windows and Exterior Doors

The exterior windows are steel with single paned, uninsulated glazing. WPa documented deteriorating glazing sealants, rusted frames and sashes, and damaged hardware aat numerous windows in the original 1971 building as well as the 1975 addition. Exterior doors are painted, hollow-metal doors and frames – some with sidelites at public entrances. The building has eleven large (10' x 10' or 10' x 12') over-head coiling doors that are uninsulated, rusting or in need of repair. In many places window and door lintels are rusting and should be inspected for structural integrity.

This is a typical steel window showing signs of rust in multiple places. These windows are uninsulated and not thermally broken allowing for thermal bridging that can cause condensation in the winter and summer. WPa recommends these windows be replaced with energy efficient ones. Note the discoloration in the bricks below the window and in a horizontal band at the left side near the window head. These are the signs that water is accumulating in the wall cavity.

This steel window has a damaged sash that does not seal properly. There are signs of rust in the gap at the top.

This mage shows the deterioration of the glazing sealant in this window. The bottom of the sash is also showing significant wear.

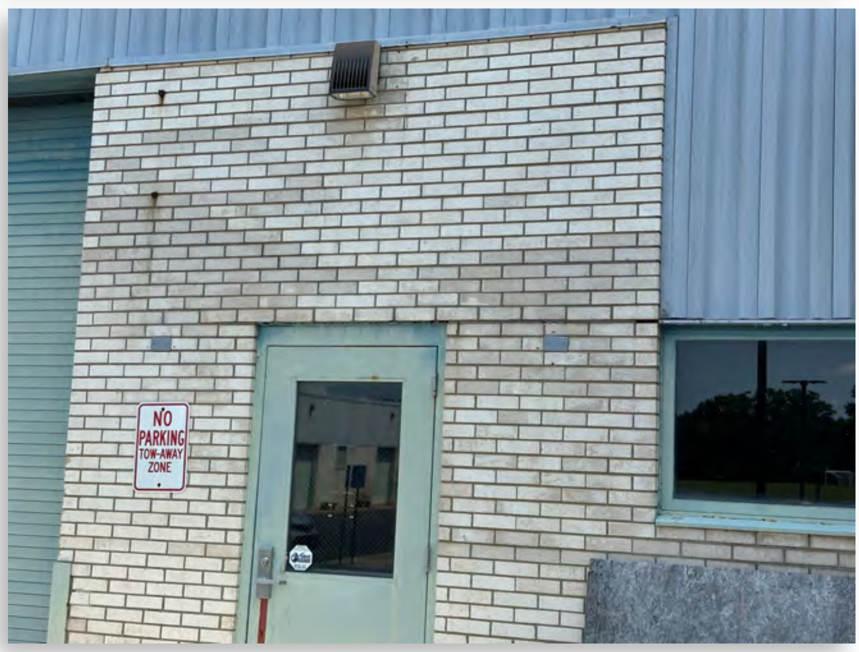
This image shows another window with rust in various places. This window also has a damaged sill that is allowing moisture into the wall cavity below the window. Note the gap between sill lengths under the middle sash.

This is the main entry to the Laurel South Campus. The two pairs of doors – each separated by a sidelite – are flush hollow-metal doors. In addition to being in bad shape, these doors do not provide an appropriate first impression to the campus.

This enlarged section of the previous image shows the damage to the doors and the infilled transoms above each door. Most outward-swinging exterior doors are also showing rust on the hinges as well as the bottoms of many frames. The configuration shown here is typical at most entries.

All exterior hollow-metal door frames have a 4” head. Many doors like these have been fitted for multiple hardware sets over the years and were not typically repainted to cover the locations of old hardware. The doors here at the end of the main corridor in the 1975 addition are in particularly good shape.

This door into the carpentry shop area shows bare metal where an old lockset was removed. There is rust on the panel at the lite kit and the lintel above the door is rusted. The discolored bricks above the lintel hint that water in the wall cavity has collected there, causing the lintel to rust and swell which is also causing the mortar joint to pop out. Water stains on the frame head also indicate that water has worked its way out below the lintel.



The over-head coiling garage doors typically extend to the top of the brick as shown here on the original 1971 building. The jamb on the left side of the door is showing rust which begins about 32” below the top of the brick. A line of discolored bricks to the left of the door suggests there is a relief angle there collecting water.

This detail of the same door shows a damaged sweep at the top of the opening as well as a portion of the metal panels that is not closed off properly at the bottom edge. The condition repeats in many places around the perimeter of the building. This may be due to differential expansion and contraction between the masonry and the panels.

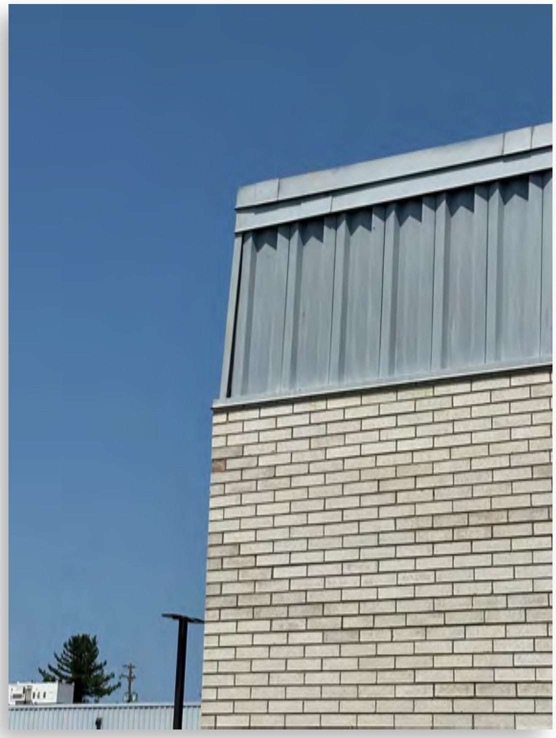
Exterior Envelope

The condition of the exterior envelope at Laurel South is curious. The original 1971 building and the 1975 addition were both designed by architect, L.E. Browning, and structural engineers, Hugh Dillehay & Assoc. so there is some continuity between the buildings. The original building was designed without expansion joints and the 1975 addition has them. Both projects use what appear to be relief angles at about 32"O.C. vertically around the perimeter of the building. These are not documented in any of the drawings.

During our site visit, WPa documented a lot of damaged mortar joints that regularly occurred every 32" vertically around the building. When inspected, the mortar joints were loose in between the bricks and falling out of the wall in many places. In other places, the rusted steel edges or angles were seen poking through the mortar. The presence of relief angles is common in buildings constructed in this time period. However, there is a growing opinion that they were designed and located incorrectly and often with greater frequency than was actually needed to relieve the loads in the masonry. In an article by Brian E Trimble in Structural Magazine (May 2009), he states that guidelines for relief angles require them at 30'-0" vertically in steel structures and not at all in CMU structures. That is considerably greater spacing than seen at Laurel South.

The fact that the steel angles are visible is also cause for concern as the proper detail for the angle provides a mortar joint under the angle leg, flashing, a weep above the angle and finished off with backer rod and sealant at the wall face. (BIA Technical Note 18A "Accommodating Expansion of Brickwork" – See Appendix C). With the presence of rusting and discoloration in the bricks, it is obvious that the flashing and weeping of the angles was not performed or has been hindered by the very thin air space in the wall cavity. It is possible that the angles were actually intended to act more like a masonry anchor in the wall.

The deformation of the upper, metal-panel-covered portions of the building is shown here at the South corner of the original building. This is the most pronounced instance and may be partially due to the lack of masonry expansion joints on this portion of the building. There are bricks in many places that have spalled faces like the one five courses down from the top of the brick area.



This photo of the parking lot side of the 1975 addition shows the expected expansion joint where the stresses in a brick wall are likely to cause damage to the bricks. However, this area again shows the issues with the relief angles 32” from the top of the brick (8 courses down) where the mortar has popped out of the joint. This issue with the relief angles is typical on all sides of the building.

Even with properly located expansion joints, the expansion and contraction in the building gis still causing bricks to spall and break. The image at right shows a broken brick on the right side of the expansion joint and two bricks above it with spalled faces.

This deformations in the brick are subtle in most places and difficult to document. However, in this photo the undulations in the brick surface are fairly clear. The familiar horizontal striping associated with the relief angles is also evident.

Laurel South Feasibility Study

KCTCS Somerset - 235 S Laurel Rd

London, Kentucky 40744

WPa attempted to photographically document the apparent bulging in some walls but were unable to make it apparent. Here, a level shows that the wall is slightly leaning outward.

This mortar joints are so deteriorated that the mortar can be easily removed by hand at lintels and relief angles. In the empty mortar joint the rusted lintel is clearly visible.

In the same area as the previous image, WPa found numerous fragments of mortar joints scattered on the sidewalk.

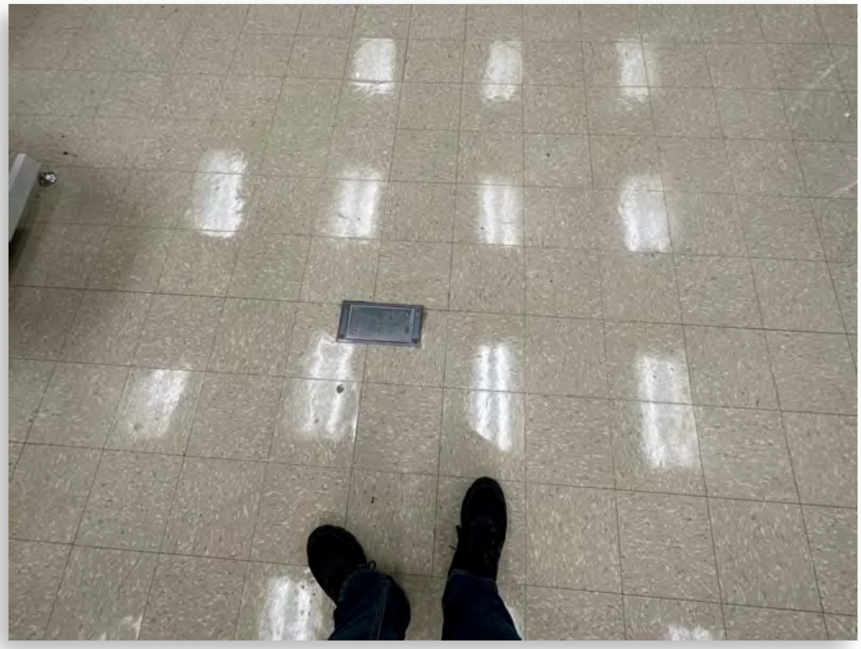
Interior Environment

The interior of the building is plagued by dated finishes, alterations to the plan, and damage due to humidity. Mechanical, electrical and plumbing systems are dated and in need of upgrading throughout the building.

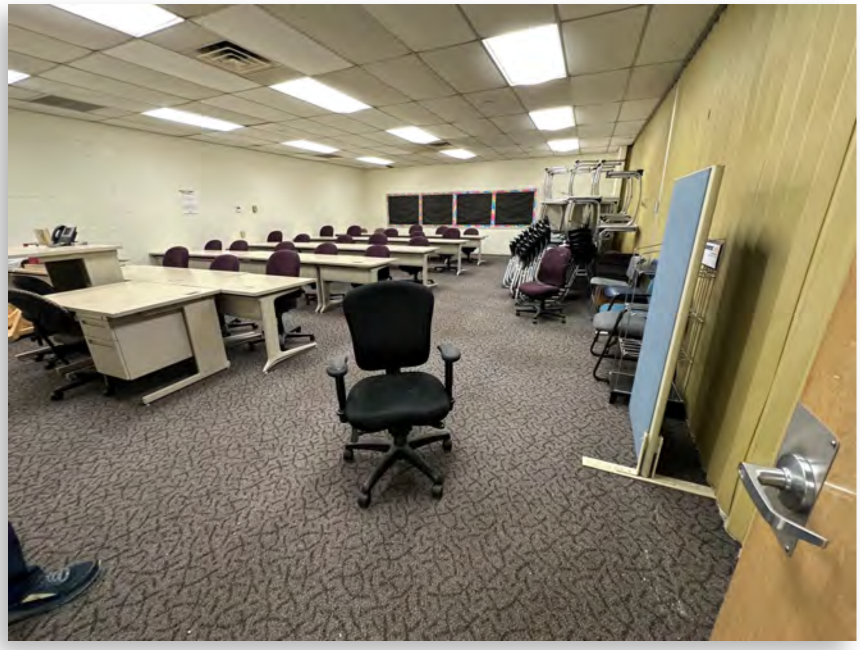
Interior Finishes and Doors

The main corridors in the 1971 building are terrazzo. The crack shown here runs the majority of the way down the wall separating the office/classroom spaces on the right from the open shop areas on the left. The building appears to be divided structurally along this line and there is an expansion joint on the roof directly above. This wall on the left side of the image is a bearing wall and has experienced some settling since the building was constructed.

The VCT flooring in the Esthetics area consists of 8” x 8” tiles. These tiles are typically associated with asbestos products and should be tested before any work in this area begins.



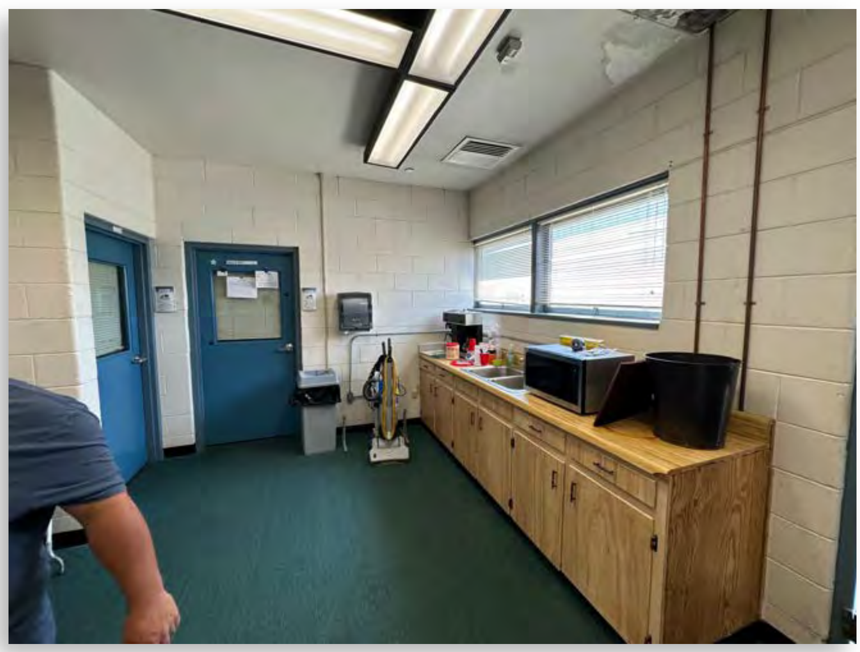
The interior finishes in many areas are dated. Ceiling tiles are sagging and discolored from the humidity and age. The accordion wall panel on the right of this image is in particularly bad shape. Many classroom spaces have no windows and poor lighting. Carpets are wall to wall and hold on to mildew smells, stains and wear in many areas.



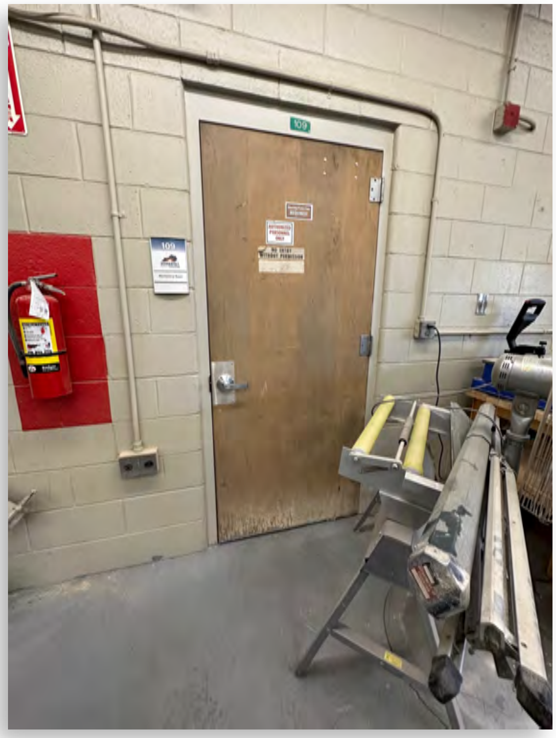
This area in the original building was renovated during the addition project in 1975. However, the sagging acoustical ceiling tiles, cut-pile carpets, and dated colors and finishes show the age of the space.

The interior doors on many spaces are in bad shape. Some doors are showing the effects of changes in humidity over 45 years, others have obvious water damage, and some are repurposed doors or doors that have been damaged through use. Interior doors need to be replaced to provide a consistent look and to ensure the right door is used for the space it serves.

These doors, in what is now the teacher's lounge, originally served spaces associated with the original cafeteria. These doors to offices should reflect the space they serve. This is another space that has been adapted to its current use. The carpet, casework and lighting are not appropriate for a teacher's lounge and should be updated.



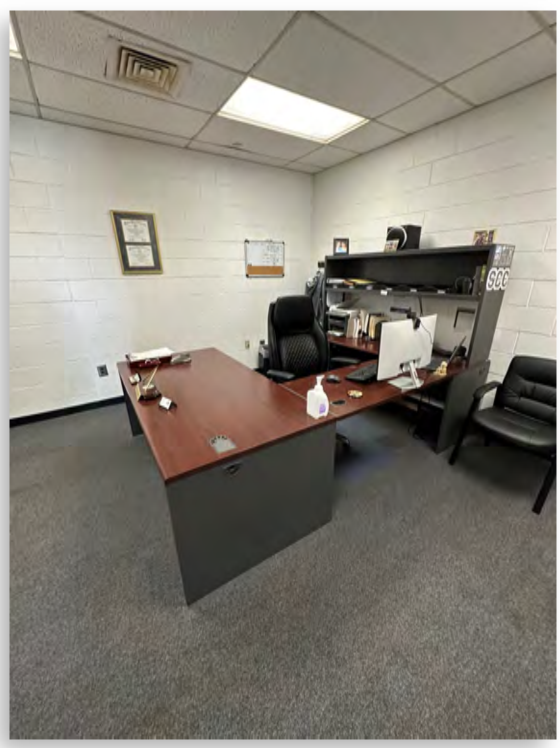
This door, in a shop area, has obvious water damage at the bottom. This is another example of doors that are inappropriate for their environment. Doors in areas like this should be hollow-metal and galvanized to protect them from elements.



This photo shows another door in an area that should have had a hollow metal-door based on the use of the space. This door in the autobody shop is also showing water damage at the bottom edges.

Even in areas that are less exposed to the elements and physical abuse, the doors are showing their age. This door into an office area has several chips in the latch-side edge and the veneer is starting to pull away in spots. This photo also shows the condition of the carpet in the offices at the door threshold where no transition strip was installed – or has been damaged and removed.

This is the office of the Dean of Business and Applied Technology. It has been converted from the reception area (1971) serving the offices behind it to a Book Room (1975) and back into an office. It still has the transaction counter and window serving the lobby in place.

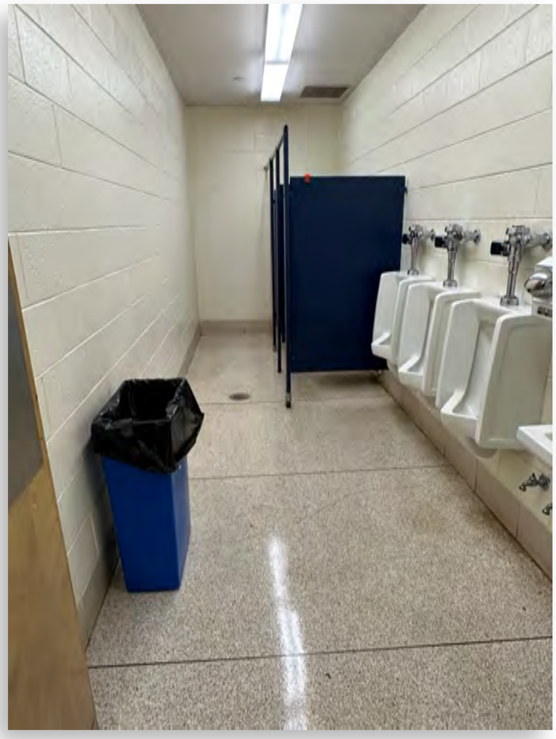


Restrooms

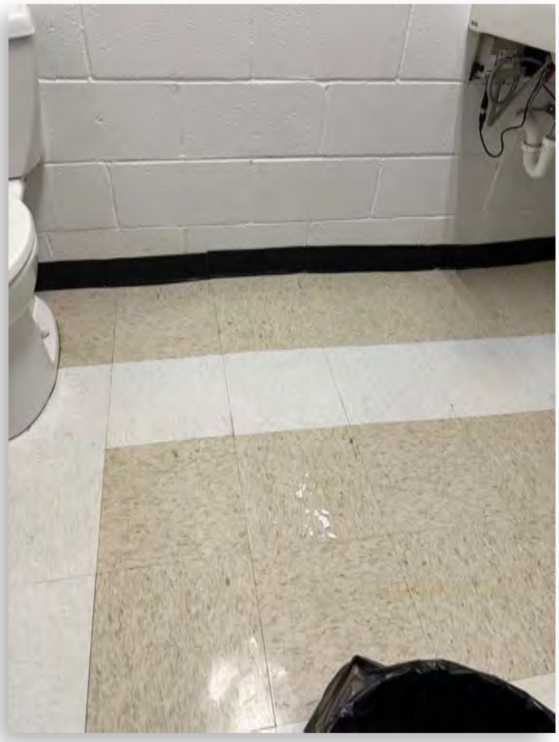
The sign to the left of the door explains that “accessible restrooms are available in Rooms 217A and 217B” at the other end of the building. The door here is 32” wide and leads into a series of switch backs to enter this unisex restroom. The other restroom that was originally paired with it has been converted into a lactation room to the right down the corridor. This building, designed and constructed long before the Americans with Disabilities Act was passed into law has very few accessible features. The fact that it was built with CMU walls and terrazzo floors makes changes to the restrooms very difficult. Moreover, by the current Kentucky Plumbing Code, the building should have 13 water closets for females in the design. The existing count falls far below this number. (1:25 water closets to females and 1:50 water closets to males).

This restroom is probably the most problematic in the building. In addition to opening into a tiny vestibule with another door, the restroom itself is only 6’-0” wide (see next image) and contains three water closets and three urinals (one of the largest in the building). The fact that it is served by two doors that are only 28” wide makes it uncomfortable for most people. The women’s restroom just to the right of this image has the same entry configuration but only serves a single person.

This restroom, behind the door in the previous image, is one of the largest restrooms in the building (fixture-wise). It contains half the required fixtures for males. At just 6'-0" wide it is almost impossible to make ADA accessible.



This unisex restroom in the faculty lounge was converted from a trashcan washing room that was only accessed from the outside and a janitor's area in the original kitchen off the cafeteria. There is a considerable deformation in the floor where a CMA wall was removed. At the time of WPa's visit to the site there was also an active leak in the ceiling of this room – water can be seen on the floor in the photo.



The accessible restrooms at 217A and 217B are actually in 217B and 217C according to the room plan provided to WPa. However, since there is no room signage outside the restrooms, it hardly matters. This photo pf the men's room shows that it meets the bare minimum required for accessibility: One toilet, two urinals and two sinks. The lighting and finishes are well cared for but minimal.

The women's room next door is similar in size and in its lack of lighting and minimal approach to finishes.

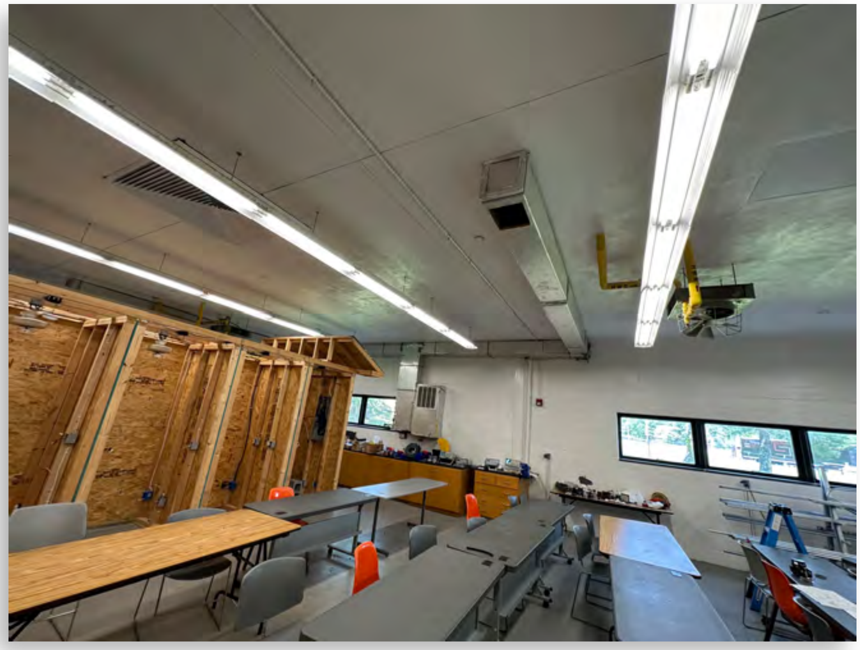
Building Systems

The building systems at Laurel South are over 40 years old. Rooftop units, plumbing, boilers, air compressors, generators, lighting, fire alarm and all the associated controls need to be upgraded. All upgrades need to take into account the various alterations to the building that have occurred since 1975. Space layouts have changed and the uses of some shop areas is also different from the original design. The Electrical shop is a good example. It was originally designed for masonry and has a variety of systems in place that are no longer needed. Classrooms, offices and lounges have been created from kitchens, storage rooms and cafeterias. The existing units (and the building envelope) were also never meant to keep up with the humidity levels inside the building or meet the current energy efficiency standards. The exhaust systems in most shop areas need improvements for safety and building upkeep. WPa's recommendations for improvements to the building systems are based on stated ages of the systems in place, recent experience with similar buildings of this time period, size and construction. There was no MEP consultant assigned to this study.

Mechanical & Electrical

The photo at right is a good starting place for a discussion about the mechanical systems in the Laurel South building. This mechanical unit is serving the former masonry shop which has been converted into the electrical shop. The HVAC needs changed and this is how those needs were met.

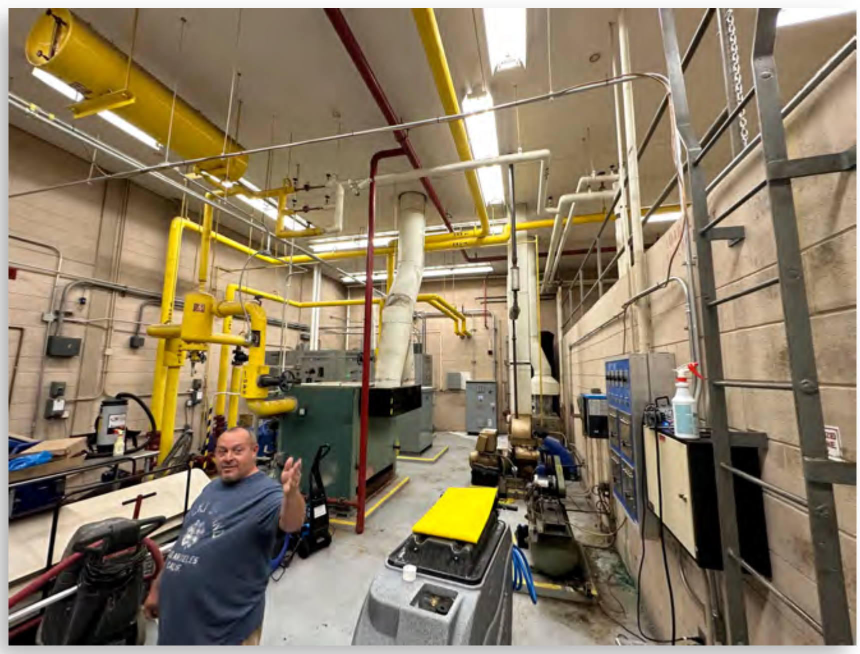
The photo shows the interior of the electrical shop and the surface mounted unit and ductwork connected to the condenser outside through the window. Shop areas were not designed with air conditioning originally. They were designed for heat only which was provided by a steam radiator with a fan (one can be seen on the ceiling to the right of the light fixture (yellow pipes) The A/C has been added later.



An overview of the mechanical room in the 1975 addition from the mezzanine over the restroom, showing the boilers, and the original switchgears in the background. The yellow piping supplies hot water to radiators with fans in the shop areas.

The photo shows the smaller boiler in the back of the mechanical room.

Another overview of the mechanical room in the 1975 addition.



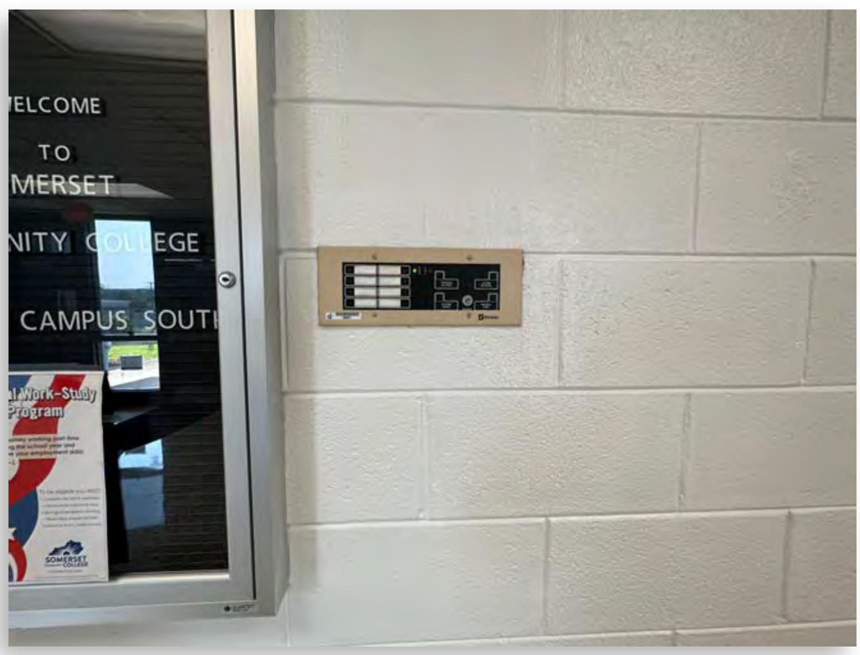
This image shows the mechanical room in the original 1971 building. The main switch gear and various air compressors are visible. Most electrical and mechanical equipment is original to the building.

This view is facing the opposite direction. A boiler is visible behind the switch gear.

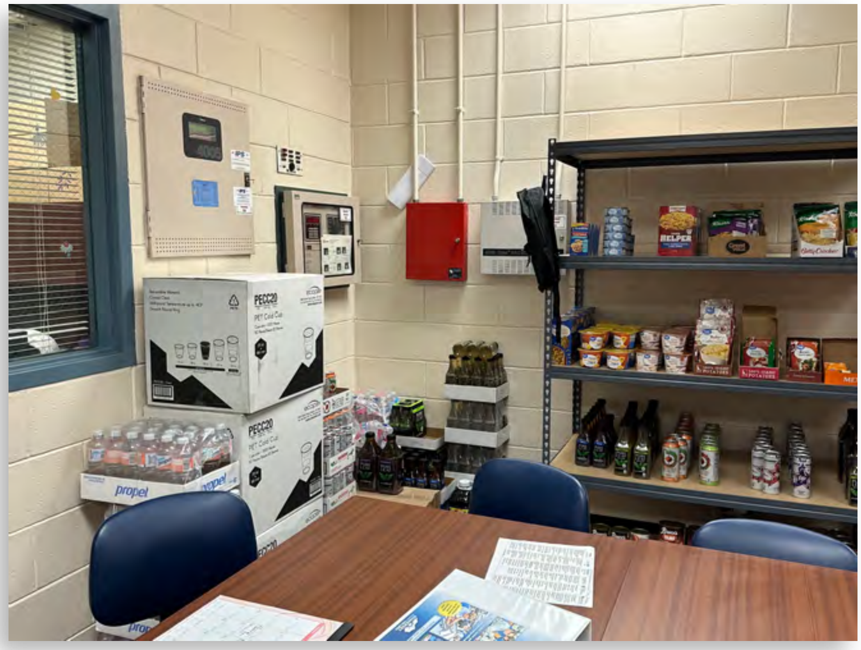
MEP Controls

Lighting and HVAC controls are outdated and should be upgraded with the new mechanical units.

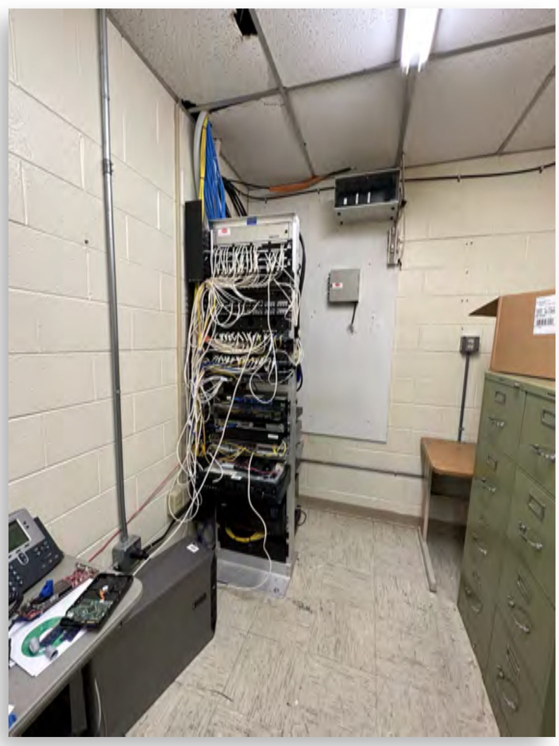
There are a variety of panels and devices associated with the fire alarm. Many of these are outdated and should be upgraded if no longer serviceable.



This photo shows the fire alarm and master clock panels against the back wall of the food pantry. The Simplex 2350 master clock system is no longer made. However some version of the Simplex 4005 Life Alarm and the 4009 IDNet NAC Extender appear to be available as recently as recently at 2021. WPa has been told that there have been no upgrades to the systems since the building was renovated in 1975.

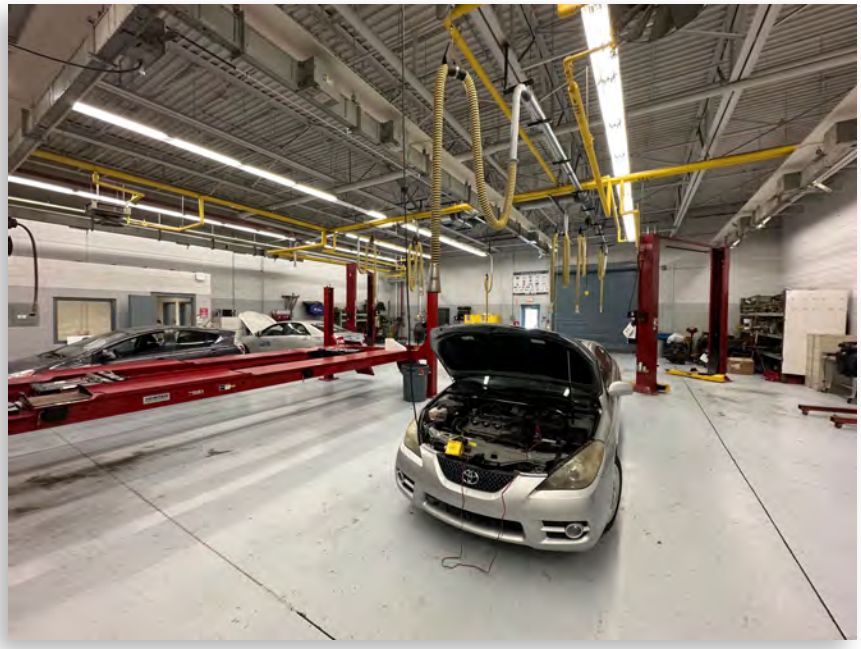


This view is of the communications room. This space was originally a first aid office. The building needs a communications / data room built for this purpose.

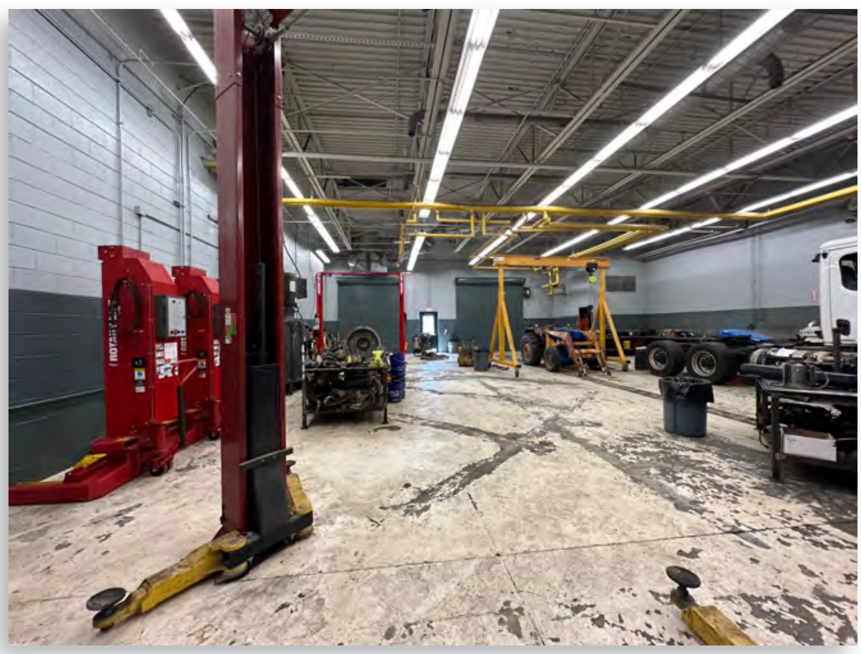


Exhaust and Venting

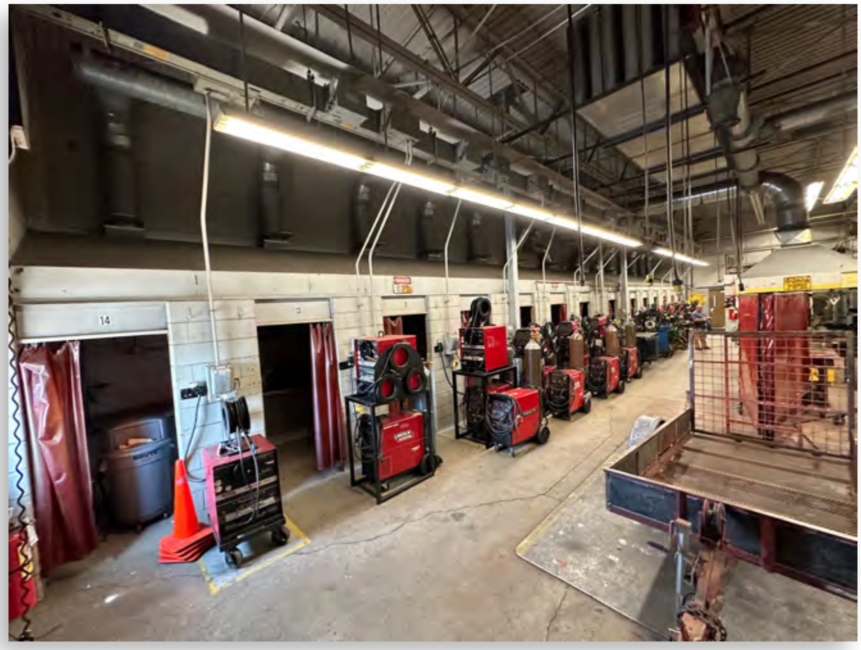
The Automotive Lab has a number of nozzles attached to an exhaust extraction .system. These systems are crucial when working on a running vehicle indoors. WPa was unable to find about the attachment protocols for the nozzles in place. The ones seen here did not appear to have clips to hold them in place on horizontal exhaust systems like those on the car shown.



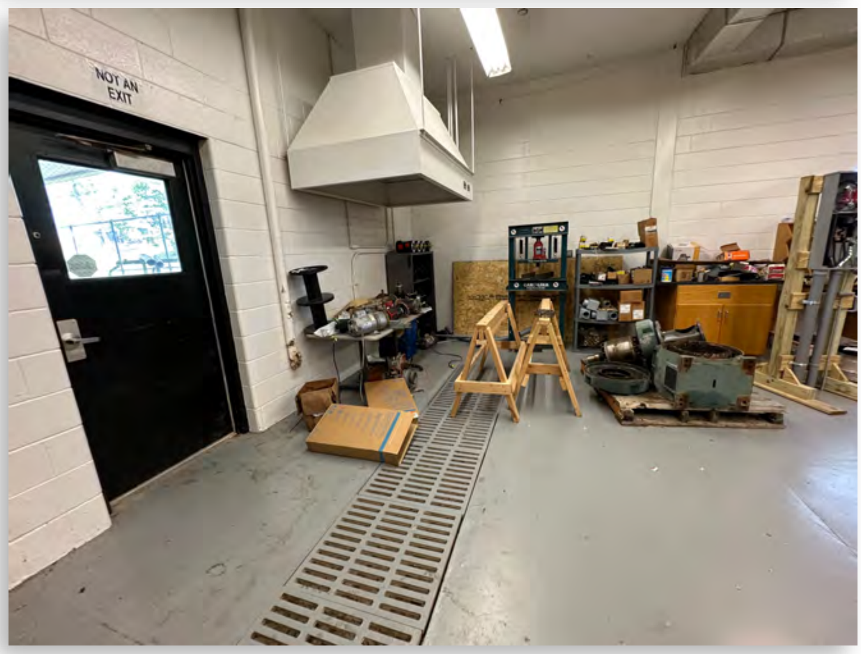
The Diesel Technology Lab, however did not appear to have any exhaust extraction systems. These shop spaces are provided with only heaters (fan-powered radiators) and are not conditioned spaces. The office spaces and classrooms have been equipped with a variety of A/C solutions from PTAC units to residential window units to provide cooling in these areas.



The Welding Technology Lab has an extensive exhaust system with a number of hoods throughout the space. However, the system itself appeared to be aging.



The Electrical Shop has an exhaust system in place. However, it is a remnant of the masonry shop which occupied the space in the original design of the building and it is no longer used.



Laurel South Feasibility Study

KCTCS Somerset - 235 S Laurel Rd

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The Construction Technology Lab utilizes a sawdust collection and extraction system to collect sawdust from floor inlets and at various machines. These units exhaust the sawdust and deposit it in the barrels seen below the units. This system is aging and needs to be upgraded.

Site Improvements

The site around Laurel South has a number of landscape walls and fences which mostly need to be repaired or replaced. Many masonry walls are falling apart, and the fenced areas need to be cleaned up and the fences replaced. Laurel South suffers from a lack of a clear entry to help visitors find a way in. Providing an clear entry can be done a number of ways, especially with regard to the small lobby one encounters once they are inside. While the design of a new entry is beyond the scope of this study, WPa has experience designing entries for similar facilities to make estimates of probable cost. The 131,000 SF parking lot needs to be resurfaced and restriped. Any damaged curbs or islands should be repaired as well. The current lot holds roughly 262 spaces. At the West end of the 1975 addition there is a detached maintenance building that stores supplies and equipment. This building needs to be replaced or reskinned and renovated at the very least.

The Laurel South Campus showing the parking lot and drives.

Brick landscape walls that are in bad shape need to be repaired or rebuilt depending on whether they are retaining walls or simply decorative like this one at the right.

This view of the maintenance shed from the roof of the 1975 addition shows the need for new fencing and walls as well as the condition of the asphalt parking lot. The building itself seems structurally sound but could use a new metal skin and some improved shelving and storage inside as well as upgrades to insulation were needed and any heating units.

The right side of the maintenance building contains mowers and supplies for maintaining the site.

The left side of the maintenance building contains supplies for the building and its upkeep.

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This fenced area is outside the HVAC shop and houses a collection of condensing units associated with the shop. Renovation of the building should address areas like this outside as well.

This retaining wall along the sidewalk leading to the main entrance is in disrepair. The wall has cracked and spalling bricks, damaged mortar joints and significant discoloration as it leads visitors up to the lobby.

Recommendations:

Based on the documents provided to WHITE|POLLARD architects, our on-site observations and research, we recommend the following items to be included in the renovation of the Laurel South Campus:

Exterior Envelope

The building exteriors need a number of improvements. The roof may be the most pressing of these. For a twenty-three year old roof, it is in pretty bad shape and there is evidence that water is getting under the membrane in many places – especially on the 1975 addition's roof. WPa recommends a full tear-off of the existing roof system to the deck and replacing it with IECC required insulation and a new SBS modified bitumen roof system. Alterations to the roof edges and new flashings around the perimeter will be required to respond to the deeper system thickness. Roof drains should be extended to the new roof level and equipped with overflow protection to prevent water from ponding on the roof when one or more of the four roof drains become blocked by debris from nearby trees. Most mechanical curbs will need to be altered as well (see MEP recommendations).

The existing, steel, uninsulated windows should be removed, the openings properly flashed, and new, insulated and thermally broken frames with insulated, low-e glazing units installed. Exterior doors at main entries to the building should also be replaced with more inviting aluminum storefront frames and entry doors. The storefront systems should also be thermally broken and insulated and include insulated glazing in the sidelites and doors. Doors that are not part of a main entry (at shop areas, for example) should be replaced with new hollow-metal frames and doors that are insulated, thermally broken, hot-dipped galvanized and weather-tight. Any damaged lintels should be repaired or replaced, all rust removed from existing lintels and repainted. New lintels should be hot-dipped galvanized and painted. All of the large, over-head coiling garage doors should be replaced with insulated ones with new motors and rails. Jambs and heads should be repaired and prepped to allow the new doors to seal properly when installed.

WPa recommends replacing the metal panels at the tops of the walls to make sure they are flashed properly and weather-tight. In areas where there appears to be horizontal movement in the top portions of the wall, a structural engineer should evaluate the cause and whether it is a item for continued concern or the remnant of building settling that has stopped.

The brick and mortar issues are tricky and there is no easy way to replace the rusting relief angles without removing the exterior brick and re-building it. The relieg angles could be replaced with hot-dipped galvanized angles at that time and the condition of the wall cavity insulation could be assessed, remediated and resolved before new brick in re-installed. It is possible that a new roof, new flashings and new metal panel skins at the top of the wall would halt the ingress of moisture into the wall cavities and prevent further rusting and expansion. Under those circumstances simply repairing the damaged mortar and brick could suffice but there is no guarantee that the rusting wouldn't continue to expand from some other cause. Regardless it is safe to assume that the wall insulation in the discolored areas is of little value at this time for energy efficiency.

Interior Environment

Most interior finishes need to be updated. Some elements like the acoustical ceilings are in such poor shape (sagging, discoloration, etc.) from exposure to humidity that they should be completely replaced. There are a variety of floor finished throughout the building and many of them are dated or not appropriate for a higher education building. The terrazzo floors are unique and tend to endure. There is no obvious need to remove them or cover them up. The variety of VCT colors and patterns should be standardized across the building. Vinyl sheet flooring should be removed and replaced with new VCT. Carpet tiles should replace the cut-pile carpets in office areas. Shop floors could benefit from a thorough cleaning and new epoxy coatings that are easy to maintain and clean. The entire building needs to be repainted.

Interior doors should be replaced with new solid-core wood doors with new hardware and locks at all locations where a solid-core wood door is appropriate. The shop areas where the interior environment is variable due to garage door operation, cleaning and other activities should be equipped with new, galvanized, hollow-metal doors. Most interior frames are likely in good shape but where there has been water damage to doors in shop areas, new, galvanized hollow-metal frames are also recommended.

The restroom problems are not easy to address as there are few opportunities to add new restrooms within the existing building footprint, and even fewer opportunities to modify existing restrooms to meet the ADA requirements for maneuvering and clearances. This could be addressed as part of a new entry project that extends the existing lobby and provides a focal point to the building. Restrooms included in that design could be sized to meet the ADA and the fixture shortages of the existing building. The existing restrooms need to be updated with new fixtures and accessories, as well as finishes.

The operable partitions in several classrooms are dated and should either be replaced with new operable acoustical panels to properly divide the spaces or be removed altogether. The Student and Faculty Lounges also need upgrading. Currently, both these spaces feel like an afterthought and do not really provide the type of experience one would expect from a lounge area.

The shop areas almost all have storage problems. Materials are stored haphazardly on mezzanine spaces accessed by ladders or spiral stairs which do not provide safe access to the areas. Railings around such areas are sometimes non-existent. In the original building, tool storage is lacking. In one instance, in the Carpentry Lab, tools were being hung on sprinkler pipes due to lack of suitable storage options. Tools and materials need to be stored in areas that are safe to access and provide security and easy access to the items.

Because the building has very little natural lighting, the need for quality artificial lighting is at a premium. The existing light fixtures should be upgraded to brighter LEDs with new lighting controls throughout the building. In addition to improving the interior environment, LED lighting reduces operational costs through energy efficiency and ease of maintenance.

Building Systems

The HVAC and power systems in the buildings are over 40 years old. On similar buildings, it has been recommended that all the mechanical units be replaced. This will also require some new systems like make-up air that have been incorporated into the mechanical code since the building was constructed. The ductwork layout should be carefully considered at that time to make sure spaces that have been altered since the initial design will be properly conditioned. The use of the hot-water radiant heaters in various shop areas should also be evaluated to determine if this is the best approach to heating these spaces today.

Much of the main power distribution equipment appears to be original to the building and may not meet current codes. WPa's cost estimate assumes that these elements will be upgraded as part of a major renovation.

The various exhaust systems in the shop areas need to be upgraded and modernized – or provided in the case of the diesel technology lab. The defunct exhaust system in the electrical shop should be removed if no longer needed.

Site Improvements

WPa recommends repairs and improvements to all landscaping walls and fences around the building. This will add to the much-needed sense of place at the Laurel South Campus.

The whole asphalt parking lot and associated drives need to be resurfaced and restriped.

The small maintenance building behind the 1975 addition houses a large amount of equipment for the upkeep of the building and grounds. It would improve the appearance of the building to have a new metal skin to match the new metal panels on the main building. The interior of this building could also use improved storage shelving and updated insulation and heating.

An improved entry addition is the most effective way to give the building a presence and establish a focal point for visitors. Additionally, a new entry addition provides an opportunity to easily add much needed restrooms for accessibility. WPa recommends seriously considering this approach to resolve multiple shortcomings of the building at the same time.

Other Considerations

An additional factor that will affect the cost of renovation are the user agency's request that the building remain occupied during construction. This will complicate the schedule and ability of contractors to work on systems that affect the whole building. HVAC, electrical, plumbing upgrades would affect large areas of the building at once. Other work could easily be phased. WHITE|POLLARD architects recommends that the work be divided into three parts according to the roof areas: the original building, the low bay portions of the 1975 addition and the high bay portions of the addition. The actual order, and extent, of each phase would require close coordination with an MEP Consultant as the upgrading of building systems will be a primary driver of the phasing. WPa's recent experience with long lead-time items, like major electrical components, will make the phasing even more difficult.

New Construction:

WHITE | POLLARD architects was also asked to estimate the cost of a new building of the same size, construction, and program to be located on a site at the Laurel North Campus.

Due to the natural slope of the site and the types of spaces it would contain, the building would need to be two-stories in areas with offices and more traditional classrooms. The shop areas would need to have taller clearances and could be located on the lower portion of the site to the East where they have access to the drives and parking areas for overhead doors. The area of the new building would increase to account for stairs and elevators to serve the two-story version of the original program.

Laurel South Feasibility Study

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Summary

The building at Laurel South has been in service for over 50 years. In those 50 years, codes have changed, programmatic needs have changed, and the building itself has changed. During our site visit, a number of issues were immediately apparent: the building was in need of a face lift, water was getting into the walls, and the building had been modified haphazardly over the years to try to keep up with the changing needs of the school. WPa was provided construction documents for the 1975 addition but no plans of the original building are known to exist – an issue that causes problems for the maintenance staff on a regular basis. To provide a meaningful renovation of the Laurel South building, more than finishes must be considered. There are design issues that are affecting the brick exterior and will likely continue affecting the brick exterior forever. Water has gotten into the walls and damaged the wall system (wet insulation, rusted angles, damaged mortar, cracked and broken bricks). As long as water continues to enter the wall the problems will persist. The building systems are as old as the building itself and while they have been maintained, they were not designed to do the job they need to do now. Energy Efficiency, Safety and Accessibility all place demands on the building and some are not easy to address. Systems like the roof will require complete removal and the new roof will have to meet current codes – this means thicker insulation and changes to the roof edge and mechanical curb design.

WPa has attempted to address all the issues we found during our visit in this report but, there are likely a number of issues still to be uncovered. WPa took over 700 pictures, measured humidity levels, researched the design of the building and observed every space. Our estimate addresses everything found in the limited time available.

Cost Estimates

The Cost Estimate for the proposed renovations is based on areas, dimensions and quantities derived from a building model constructed from available construction documents, plans and observed conditions at the site. Unit costs are based on a variety of sources, past experience with similar systems and building types and recent trends and are determined by a variety of elements that may not be explicitly mentioned in the item description. (See Appendix B)

Laurel South Renovation	$25,063,576.87.
New Building at Laurel North	$28,020,094.08

Comparison of renovation costs to cost of new construction 89.4%

WPa is providing the cost for new construction to provide a reference for the costs of renovating the existing building. The cost of new construction would be approximately $2,956,517.20 more than the estimated cost to repair the existing building. However, the costs associated with the temporary relocation of programs during repairs is difficult to estimate with a complex renovation that will occur while the building is partially occupied and in an economic climate where lead times on items can suddenly increase, causing delays. Additionally, the new construction is based on a building of similar size and program – a new building on the North Campus might have slightly different programmatic needs. The schematic design of a new facility was not part of the scope of this study.

WHITE | POLLARD architects has enjoyed the opportunity to work with you on this challenging project. Renovation projects are always difficult to quantify, and some issues can only be discovered once construction begins. With buildings that are in the process of failing, that unknown element can be even more unpredictable as one failing element may cause damage to other elements in places that are hard to observe during design. While WPa was tasked with assessing the needs of the existing building, it was important to keep those needs in context with the possibility of completely replacing the building rather than attempting to address all the programmatic challenges and construction issues inherent in buildings constructed at that time. Please feel free to contact us if clarifications are needed or if there are additional questions or concerns.

Respectfully Submitted,

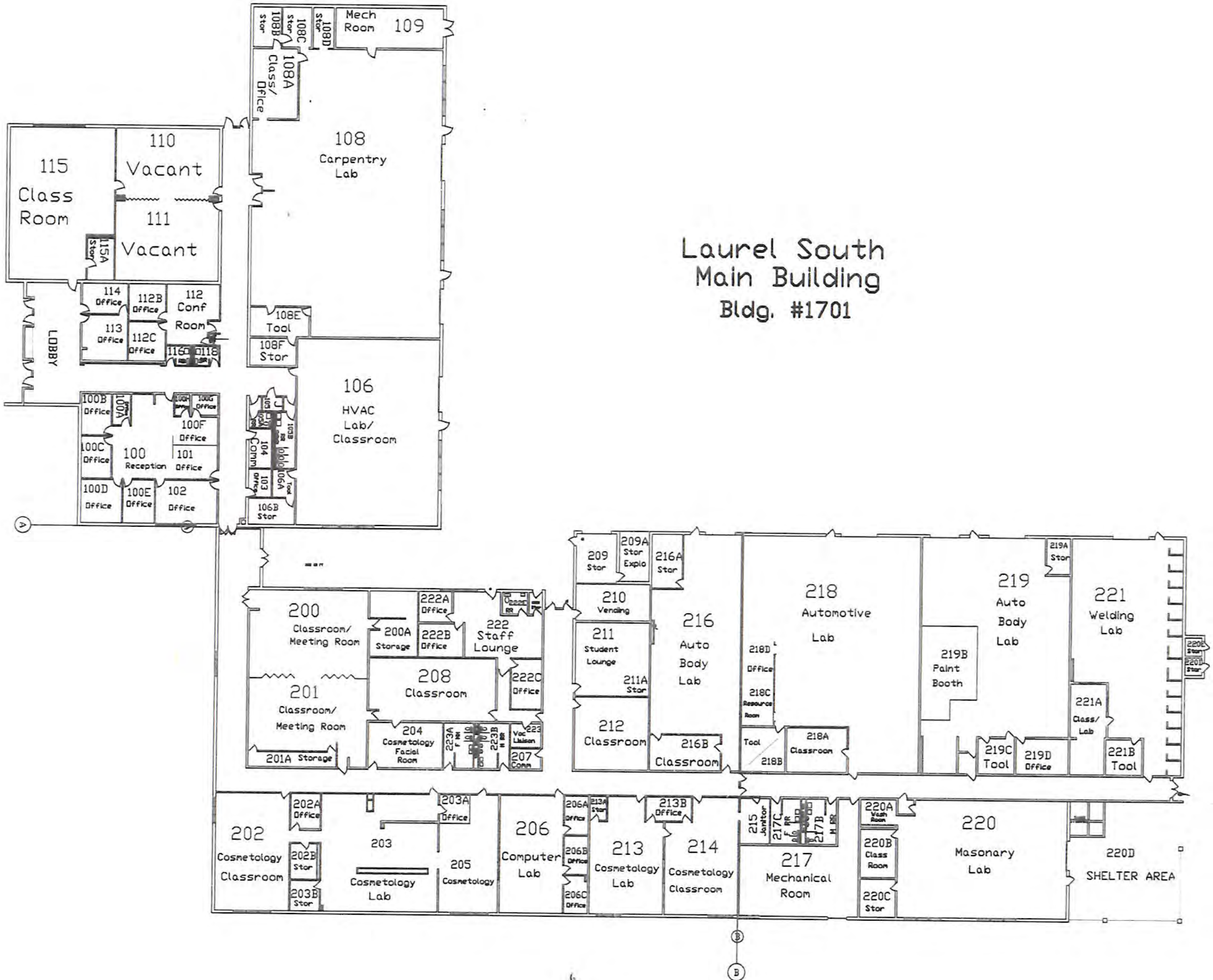
Kell D Pollard, AIA, NCARB, LEED AP

Steven M White, AIA, LEED AP

Appendix A

Existing Building Room Plan

Laurel South
Main Building
Bldg. #1701



Appendix B

Cost Estimates

COST ESTIMATE - RENOVATION

KCTCS - Somerset Community College -
Laurel South Renovation Study
470-CAYX-SS74-00

WHITE | POLLARD architects

I. Existing II-B Construction - Total Square Footage = 51,956 Square Feet

II. REPAIRS AND IMPROVEMENTS

			Description	Unit cost		x	Quantity		Total
	A		STRUCTURAL REPAIRS	unit cost		x	quantity		
		1.0	Structural Masonry Repairs	$ 14,880.00			1		$ 14,880.00
		2.0	Roof and Framing Modifications	$ 49,600.00			1		$ 49,600.00
		3.0	Foundation Repairs & New Footers for Brick	$ 167,748.51			1		$ 167,748.51
		4.0	Misc. Structural Items	20%					$ 46,445.70
			Structural Subtotal:						$ 278,674.21
	B		ARCHITECTURAL REPAIRS	unit cost		x	quantity		
X		1.0	New SBS Modified Bitumin Roof Install, Repair & Insulation	$ 32.76	/sf		62,100	SF	$ 2,034,396.00
		2.0	New Copings and Extended Parapets / Flashing	$ 94.21	/lf		1,600	LF	$ 186,913.04
		3.0	Energy Efficient Windows & Flashings (Blinds)	$ 1,566.31	/ea		46	ea	$ 72,050.08
		4.0	New Alum Entries, Doors & Sidelites (ADA & Security Card Readers)	$ 9,539.15	/ea		6	ea	$ 57,234.90
		5.0	Large Energy Efficient Windows & Flashings (Blinds)	$ 9,595.99	/ea		2	ea	$ 19,191.98
		6.0	New Acoustical Tile Ceilings and Grids	$ 9.40	/sf		28,812	SF	$ 270,832.80
		7.0	New Accessible Restrooms (8 WC, 4 Lavs, 4 Urin.)	$ 10,052.01	/ea		20	ea	$ 201,040.25
		8.0	New Interior Paint	$ 2.52	/sf		160,545	SF	$ 404,573.40
		9.0	Exterior Metal Panel Replacement	$ 17.42	/sf		6,485	SF	$ 112,987.94
		10.0	Brick Replacement (Mortar, Relief Angles, Exp Joints, CMU Repair)	$ 72.67	/sf		19,111	SF	$ 1,388,856.73
		11.0	Exterior Doors (non-entrances)	$ 2,820.00	/ea		13	SF	$ 36,660.00
		12.0	Epoxy Floor Coating in Shop Areas (prep floors)	$14.40	/sf		25,624	SF	$ 368,985.60
		13.0	New Operable Partitions at Classrooms (acoustical)	$ 107.88	/sf		744	SF	$ 80,262.72
		14.0	Upgraded Student and Faculty Lounge Areas	$ 26.04	/lf		1,532	lf	$ 39,893.28
		15.0	New Door Panels and Hardware for all rooms	$ 1,785.60	/ea		109	ea	$ 194,630.40
		16.0	Extend / Replace Roof Drains	$ 2,855.34	/ea		14	ea	$ 39,974.69
		17.0	New Entry / Extended Lobby (restrooms not incl.)	$ 217.12	/sf		2,200	sf	$ 477,672.80
		18.0	Upgraded shop storage areas & mezzanines	$ 20.80	/sf		901	sf	$ 18,743.95
		19.0	-	$ -	/ea		1	ea	$ -
		20.0	New Floor Finishes (excl. Corridors)	$ 6.61	/ea		36,861	ea	$ 243,621.72
		21.0	-	$ -	/ea		12	ea	$ -
		22.0	Temporary Classroom Trailers	$ 416,000.00	/ea		1	ea	$ 416,000.00
		23.0	Miscellaneous Labor & Material			x	30%		$ 1,435,421.13
			Architectural Subtotal:						$ 8,099,943.40
	C		MEP REPAIRS & IMPROVEMENTS	unit cost		x	quantity		
		1.0	Upgraded Exhaust, Ventillation and Extraction Systems	$ 2,059,200.00	/ea		1	#	$ 2,059,200.00
		2.0	Undate Existing Plumbing and Fixtures	$ 1,711,460.25	/ea		1	#	$ 1,711,460.25
		3.0	New HVAC and Makeup Air System	$ 3,617,237.52	/ea		1	#	$ 3,617,237.52
		4.0	Updated Electrical and Lighting	$ 2,567,192.93	/ea		1	#	$ 2,567,192.93
		3.0	Updated Communication and Data	$ 203,801.65	/ea		1	#	$ 203,801.65
		3.0	Electric Safety and Security	$ 866,159.15	/ea		1	#	$ 866,159.15
		4.0	Miscellaneous Labor & Material			x	20%		$ 754,132.05
			MEP Subtotal:						$ 11,779,183.55
	D		SITE IMPROVEMENTS	unit cost		x	quantity		
		1.0	New Parking lot surface and striping	$ 2,127.84	/sp		262	sp	$ 557,494.08
		2.0	Upgraded Maintenance Bldg	$ 49,600.00	/ea		1	ea	$ 49,600.00
		3.0	Miscellaneous Labor & Materials	$ -		x	20	%	$ 121,418.82

	Site Improvements Subtotal:				$	728,512.90
III.	ARCH ESTIMATE OF CONSTRUCTION COST SUBTOTAL (No OHP + GC) :				$	20,886,314.06
IV.	Contractors O & P + General Conditions					
	1.0 Contractor's Overhead & Profit + Federal Wage Rate		x	15%	$	3,132,947.11
	2.0 Contractor's General Conditions		x	5%	$	1,044,315.70
	TOTAL CONTRACTOR O&P + GENERAL CONDITIONS COST:				$	4,177,262.81
V.	ARCH TOTAL ESTIMATE OF CONSTRUCTION COST (IIA+V+VI):				$	25,063,576.87
	PROJECT COST PER SQUARE FOOT (Arch & Sitework):	$ 25,063,576.87	/	51,956 sf	$	482.40

COST ESTIMATE - NEW CONSTRUCTION

KCTCS - Somerset Community College -
Laurel South Renovation Study
470-CAYX-SS74-00

ESTIMATED COSTS FOR NEW CONSTRUCTION

I. Trade School Building with two-story classroom section and one-story high bay based on program of existing building

A	New Trade School Building on the North Campus (62,000 SF)		Unit Cost		Quantity		
	1.0	Cost of a New Trade School Building	$438.00	/SF	62700	SF	$27,462,600.00
	2.0	New Parking Lot and Drives	$2,127.84	/space	262	spaces	$557,494.08
	3.0		$0.00		0	LF	$0.00
	4						

Total Cost for New Construction	$28,020,094.08

Appendix C

Brick Industry Association, Technical Note 18A
Brick Industry Association, Technical Note 31B

Accommodating Expansion of Brickwork

Abstract: Expansion joints are used in brickwork to accommodate movement and to avoid cracking. This *Technical Note* describes typical movement joints used in building construction and gives guidance regarding their placement. The theory and rationale for the guidelines are presented. Examples are given showing proper placement of expansion joints to avoid cracking of brickwork and methods to improve the aesthetic impact of expansion joints. Also included is information about bond breaks, bond beams and flexible anchorage.

Key Words: bond breaks, differential movement, expansion joints, flexible anchorage, movement, sealants.

SUMMARY OF RECOMMENDATIONS:

Vertical Expansion Joints in Brick Veneer:
- For brickwork without openings, space no more than 25 ft (7.6 m) o.c.
- For brickwork with multiple openings, consider symmetrical placement of expansion joints and reduced spacing of no more than 20 ft (6.1 m) o.c.
- When spacing between vertical expansion joints in parapets is more than 15 ft (4.6 m), make expansion joints wider or place additional expansion joints halfway between full-height expansion joints
- Place as follows:
  - at or near corners
  - at offsets and setbacks
  - at wall intersections
  - at changes in wall height
  - where wall backing system changes
  - where support of brick veneer changes
  - where wall function or climatic exposure changes
- Extend to top of brickwork, including parapets

Horizontal Expansion Joints in Brick Veneer:
- Locate immediately below shelf angles
- Minimum ¼ in. (6.4 mm) space or compressible material recommended below shelf angle
- For brick infill, place between the top of brickwork and structural frame

Brickwork Without Shelf Angles:
- Accommodate brickwork movement by:
  - placing expansion joints around elements that are rigidly attached to the frame and project into the veneer, such as windows and doorframes
  - installing metal caps or copings that allow independent vertical movement of wythes
  - installing jamb receptors that allow independent movement between the brick and window frame
  - installing adjustable anchors or ties

Expansion Joint Sealants:
- Comply with ASTM C920, Grade NS, Use M
- Class 50 minimum compressibility recommended; Class 25 alternate
- Consult sealant manufacturer's literature for guidance regarding use of primer and backing materials

Bond Breaks:
- Use building paper, flashing, or 4 to 6 mil thick polyethylene sheeting to separate brickwork from dissimilar materials, foundations and slabs

Load-Bearing Masonry:
- Use reinforcement to accommodate stress concentrations, particularly in parapets, at applied loading points and around openings
- Consider effect of vertical expansion joints on brickwork stability

INTRODUCTION

A system of movement joints is necessary to accommodate the changes in volume that all building materials experience. Failure to permit the movements caused by these changes may result in cracks in brickwork, as discussed in *Technical Note* 18. The type, size and placement of movement joints are critical to the proper performance of a building. This *Technical Note* defines the types of movement joints and discusses the proper design of expansion joints for brickwork. Details of expansion joints are provided for load-bearing and non-load-bearing applications. Movement joints are typically included in the design of commercial and multistory structures and, although rare, must also be considered for residential structures.

TYPES OF MOVEMENT JOINTS

The primary type of movement joint used in brick construction is the expansion joint. Other types of movement joints in buildings include control joints, building expansion joints and construction (cold) joints. Each of these is designed for a specific application, and they should not be used interchangeably. It is important to understand the proper function of each movement joint, as improper application will prevent the joint from functioning properly and may result in damage to the masonry.

An expansion joint prevents cracking by separating brick masonry into segments, reducing the cumulative effects of movements caused by changes in temperature, moisture expansion, elastic deformation, settlement and creep. Expansion joints may be horizontal or vertical. The joints are formed by leaving a continuous unobstructed opening through the brick wythe that may be filled with a highly compressible material. This allows the joints to partially close as the brickwork expands. Expansion joints must be located so the structural integrity of the brickwork is not compromised.

A control joint creates a plane of weakness in concrete or concrete masonry construction that, in conjunction with reinforcement or joint reinforcement, causes a crack resulting from shrinkage to occur at a predetermined location in a straight line. A control joint is usually a partial depth indentation cut or formed into concrete or a vertical gap through a concrete masonry wythe that may be filled with inelastic materials. A control joint will tend to widen as the concrete or concrete masonry shrinks. Control joints must be located so that the structural integrity of the concrete or concrete masonry is not affected.

A building expansion joint is used to separate a building into discrete sections so stresses developed in one section will not affect the integrity of the entire structure. A building expansion joint extends through the entire wall assembly, other components of the building envelope, and the underlying structure, and is wider than a typical expansion or control joint in a masonry wythe.

A construction joint (cold joint) occurs primarily in concrete construction when construction work is interrupted. Construction joints should be located where they will least impair the strength of the structure.

EXPANSION JOINT CONSTRUCTION

Although the primary purpose of expansion joints is to accommodate expansive movement of brickwork, the joint also must resist water penetration and air infiltration. Figure 1 shows typical examples of vertical expansion joints. A premolded foam or neoprene pad that extends through the full wythe thickness aids in keeping mortar or other debris from clogging the joint and increases water penetration resistance. Fiberboard and similar materials are not suitable for this purpose because they are not as compressible.

Expansion joints should be formed as the wall is built, as shown in Photo 1. As expansion joints are formed, it is important to prevent mortar, ties or wire joint reinforcement from bridging the expansion joint. If this occurs, movement will be restricted and the expansion joint will not perform as intended. In some cases, vertical

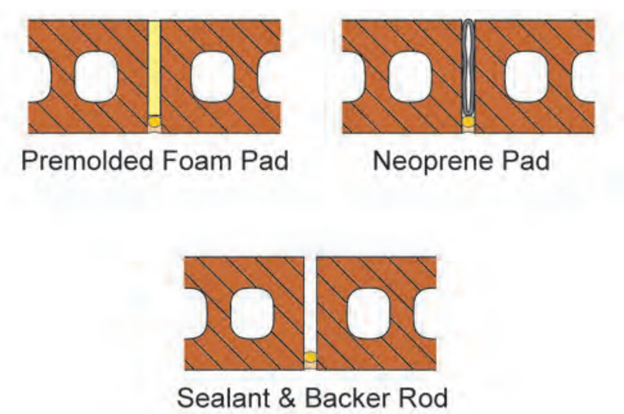


Figure 1
Vertical Expansion Joints

Photo 1
Vertical Expansion Joint Construction

expansion joints may be cut into existing brickwork as a remedial action. However, proper placement and alignment of expansion joints cut into completed brickwork are necessary to avoid small slivers of brick adjacent to the joint, as shown in Photo 2.

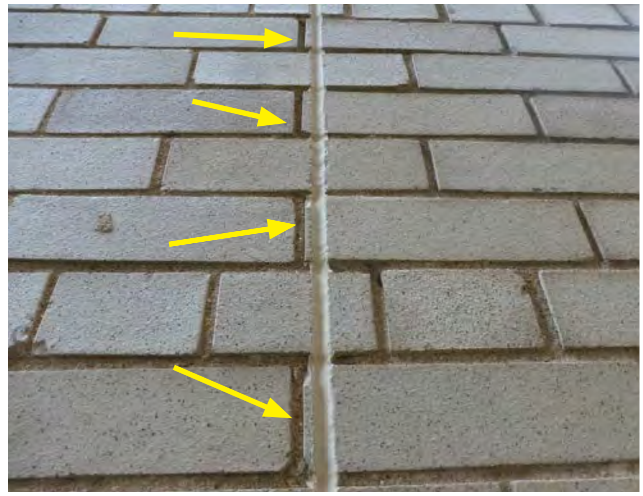


Photo 2
Poorly Aligned Remedial Expansion Joint

Sealants

Sealants are used on the exterior side of expansion joints to prevent water and air penetration. Many different types of sealants are available, although those that exhibit the highest expansion and compression capabilities are best. Sealants used with expansion joints should conform to the requirements of ASTM C920, *Standard Specification for Elastomeric Joint Sealants* [Ref. 1], Grade NS, Use M, and be sufficiently compressible, resistant to weathering (ultraviolet light) and bond well to adjacent materials. Grade NS specifies a non-sagging sealant applied to joints installed on vertical surfaces at temperatures between 40 and 122 °F (4.4 and 50 °C). Use M indicates a sealant that meets the requirements of the specification when tested on mortar specimens.

Sealant manufacturers should be consulted for the suitability of their sealants for expansion joint applications. Compatibility of sealants with adjacent materials such as brick, flashings, metals, etc., also must be taken into consideration. Silicone sealants are generally recommended for use on brickwork. Other sealant types that have been used successfully in brickwork include polyurethanes and polysulfides. Most sealants suitable for use in brickwork expansion joints meet an ASTM C920 Class 25, Class 50 or Class 100/50 rating. Class 25 and Class 50 ratings require the sealant to expand and contract by at least 25 percent or 50 percent of the initial joint width, respectively. The Class 100/50 rating requires the sealant to withstand 100 percent expansion and 50 percent contraction when tested for adhesion and cohesion. Sealants meeting Class 50 or Class 100/50 are recommended to minimize the number of joints. Many sealants require a primer to be applied to the masonry surface to ensure adequate bond. Field adhesion testing of the specified expansion joint sealants should be performed prior to construction for all substrates on the project in order to verify the sealant bond and determine the need for a primer.

Use a circular foam backer rod behind sealants to keep the sealant at a constant depth and to provide a surface to tool the sealant against. The backer rod should be sized approximately 25 percent larger than the joint width to provide the appropriate fit. The sealant must not adhere to the backer rod. The depth of the sealant should be approximately one-half the width of the expansion joint, with a minimum sealant depth of ¼ in. (6.4 mm).

VERTICAL EXPANSION JOINTS

Spacing

No single recommendation on the positioning and spacing of expansion joints can be applicable to all structures. Each structure should be analyzed to determine the full extent of movement expected. Accommodate these movements with a series of expansion joints. Determine the spacing of expansion joints by considering the amount of expected wall movement, the desired size of the expansion joint, and the compressibility of the sealant, backer and filler materials. In addition to the amount of anticipated movement, other variables that also may affect the size and spacing of expansion joints include restraint conditions, elastic deformation due to loads, shrinkage and creep of mortar, construction tolerances, and wall orientation.

The theory and equation for estimating the anticipated extent of unrestrained brick wythe movement are presented in *Technical Note* 18. Estimated movement is based on the theoretical movement of the brickwork attributed to each property and expressed as coefficients of moisture expansion (k_e), thermal expansion (k_t) and freezing expansion (k_f). As discussed in *Technical Note* 18, for most unrestrained brickwork, the total extent of movement can be estimated as the length of the brickwork multiplied by 0.0009. A derivative of this equation can be written to calculate the theoretical spacing between vertical expansion joints as follows:

$$S_e = \frac{w_j e_j}{0.09} \qquad \text{Eq. 1}$$

where:

S_e = spacing between expansion joints, in. (mm)

w_j = width of expansion joint, typically the mortar joint width, in. (mm)

e_j = percent compressibility of expansion joint material (least of sealant, backer and filler)

The expansion joint is typically sized to resemble a mortar joint, usually ⅜ in. (10 mm) to ½ in. (13 mm). The width of an expansion joint may be limited by the sealant capabilities. Compressibility of modern sealants in the 25 to 50 percent range is typical for brickwork.

Example. Consider a typical brick veneer with a desired expansion joint size of ½ in. (13 mm) and a sealant with 50 percent compressibility. Equation 1 gives the following theoretical expansion joint spacing:

$$S_e = \frac{(0.5\text{ in.})(50)}{0.09} = 278\text{ in. or }23\text{ ft, }2\text{ in. }(7.06\text{ m})$$

Therefore, the maximum theoretical spacing between vertical expansion joints in a straight wall would be 23 ft, 2 in. (7.06 m). This theoretical spacing does not take into account window openings, corners or properties of other materials that may require a reduction in expansion joint spacing. In most instances, it is desirable to be conservative when calculating spacing between joints, but it may be justifiable to exceed the theoretical maximum spacing based on engineering judgment. For example, calculations may result in a theoretical spacing of expansion joints every 23 ft, 2 in. (7.06 m), but the actual expansion joint spacing is set at 24 ft (7.32 m) to match the structural column spacing or a specific modular dimension. Vertical expansion joint spacing should not exceed 25 ft (7.6 m) in brickwork without openings and 20 ft (6.1 m) for brickwork with multiple openings.

Placement

The actual location of vertical expansion joints in a structure is dependent upon the configuration of the structure, as well as the expected amount of movement. In addition to placing an adequate number of expansion joints within long walls, consider placing expansion joints at areas of natural stress concentration, such as corners, offsets, openings, wall intersections, changes in wall heights, junctions, parapets, material transitions, deflection of supports and deflection of wood.

Corners. Walls expand toward their ends, which may cause distress where they intersect on one or both sides of a corner, as shown in Figure 2a. Place expansion joints near corners to alleviate this stress. The preferred location is within 2 ft (600 mm) of the corner on either side. This is because masons can typically reach about 2 ft (600 mm) around the corner from where they are working, so this is a convenient location for joint placement. An expansion joint should be placed within approximately 10 ft (3 m) of at least one side of the corner in either wall. The sum of distances from a corner to first vertical expansion joint in each wall should not exceed the spacing of expansion joints in a straight wall, as shown in Figure 2b. For example, if the spacing between vertical expansion joints on a straight wall is 25 ft (7.6 m), then the spacing of expansion joints around a corner could be 10 ft (3.0 m) on one side of the corner and 15 ft (4.6 m) on the other side.

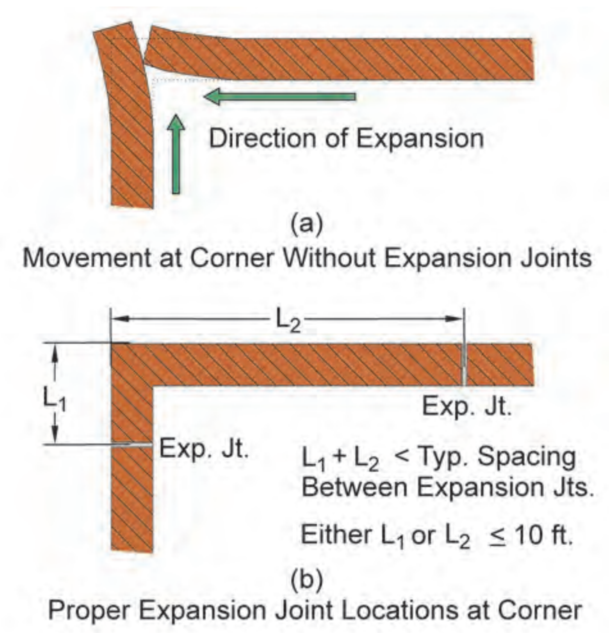


Figure 2
Vertical Expansion Joints at Corners

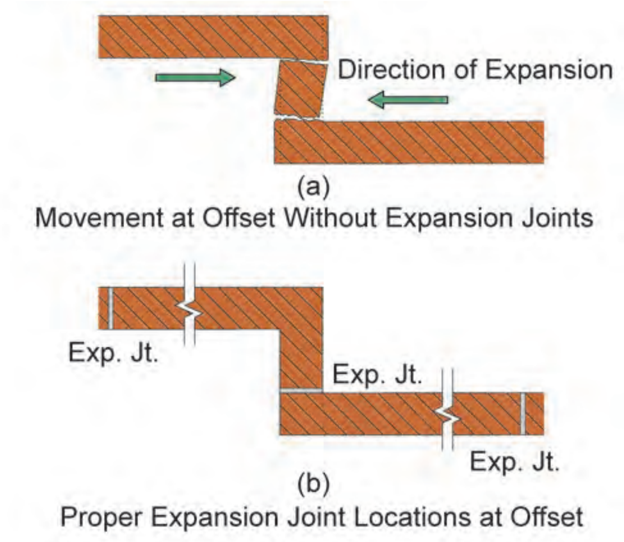


Figure 3
Vertical Expansion Joints at Offsets

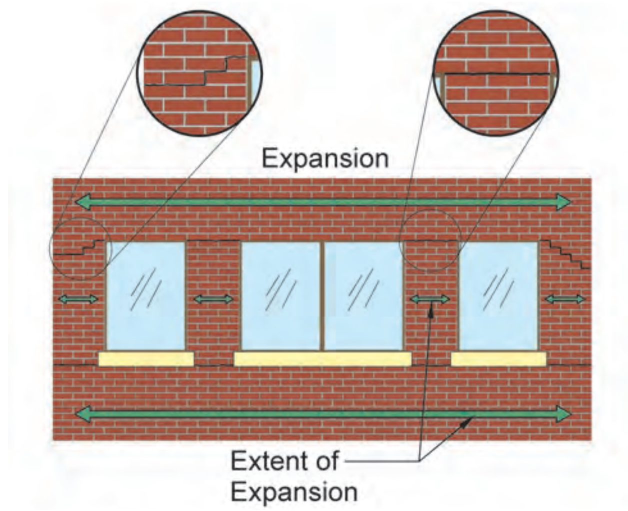


Figure 4
Cracking at "Punched" Windows

Offsets and Setbacks. As the ends of parallel walls expand, they tend to rotate the wall section that connects them, resulting in cracks, as shown in Figure 3a. Place expansion joints at the offset to allow the parallel walls to expand, as Figure 3b illustrates. Expansion joints placed at inside corners are less visible.

Openings. In structures containing "punched" windows and door openings at regularly spaced intervals, more movement occurs in the brickwork above and below the openings than in the brickwork between the openings. Less movement occurs along the line of openings since there is less masonry. This differential movement creates a stress concentration where the two sections of brickwork meet, which can cause cracks that emanate from the corners of the opening, as in Figure 4. This pattern of cracking does not exist in structures with continuous ribbon windows, as there are only spandrels of similar length and no smaller sections of brickwork between windows.

Window and door openings typically act as "natural" expansion joints and may govern or otherwise play a role in determining the placement of movement joints. Because of this, it is typically desired to place joints aligned with the edges of these openings; however, the feasibility of doing so will often depend on the size of openings in a given wall section, how the brickwork is supported above the opening and how the openings are aligned. Sealant joints are typically placed around the perimeter of the door or window frame to allow for movement between the door or window and the masonry rough opening (perimeter joints). These perimeter sealant joints may intersect with veneer expansion joints. For instance, when a vertical veneer expansion joint aligns with a window or door opening, the same sealant joint can serve as both the perimeter joint and the vertical expansion joint along the height of the window or door unit.

Where the masonry above an opening is supported by shelf angles attached to the structure, there are no impediments to placing a vertical expansion joint aligned with the jamb of the opening. In this case, the same sealant joint can serve as both the perimeter joint and the horizontal expansion joint along the width of the window.

If a loose laid lintel is used to support the brickwork above an opening, expansion joints can also be placed alongside the opening, as shown in Figure 5a. However, this configuration is more complicated to detail and construct. Because the lintel is not attached to the structure, it must be allowed to expand and contract independently of the brick. A slip plane should be formed by placing flashing above and below the angle. Mortar placed in front of the lintel is subject to cracking; thus, a backer rod and sealant should be used, as shown in Figure 5b. Because steel expands more than masonry, a ⅛ to ¼ in. (3.2 to 6.4 mm) space should be left at each end of the lintel. These measures form a pocket that allows movement of the steel angle within the brickwork. If a vertical expansion joint cannot be built in this manner, it is not recommended to place a joint alongside the opening.

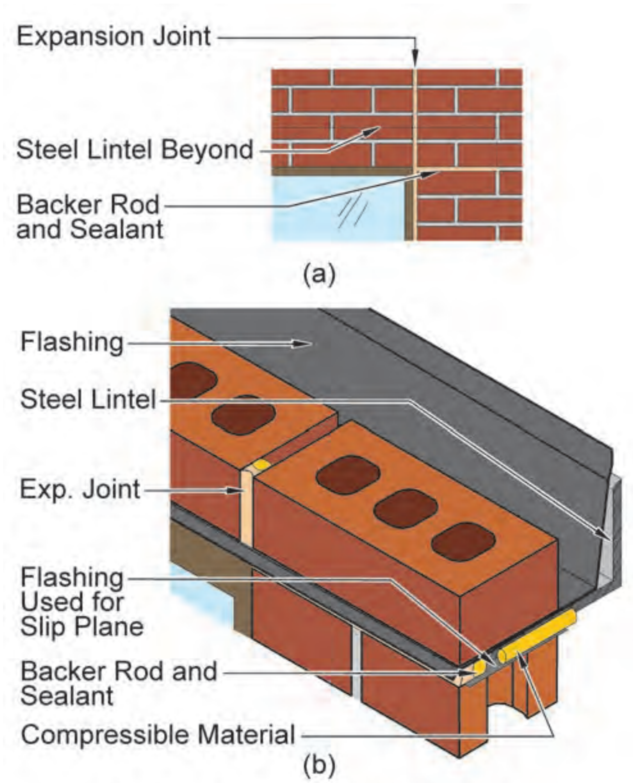


Figure 5
Expansion Joint at a Loose Lintel

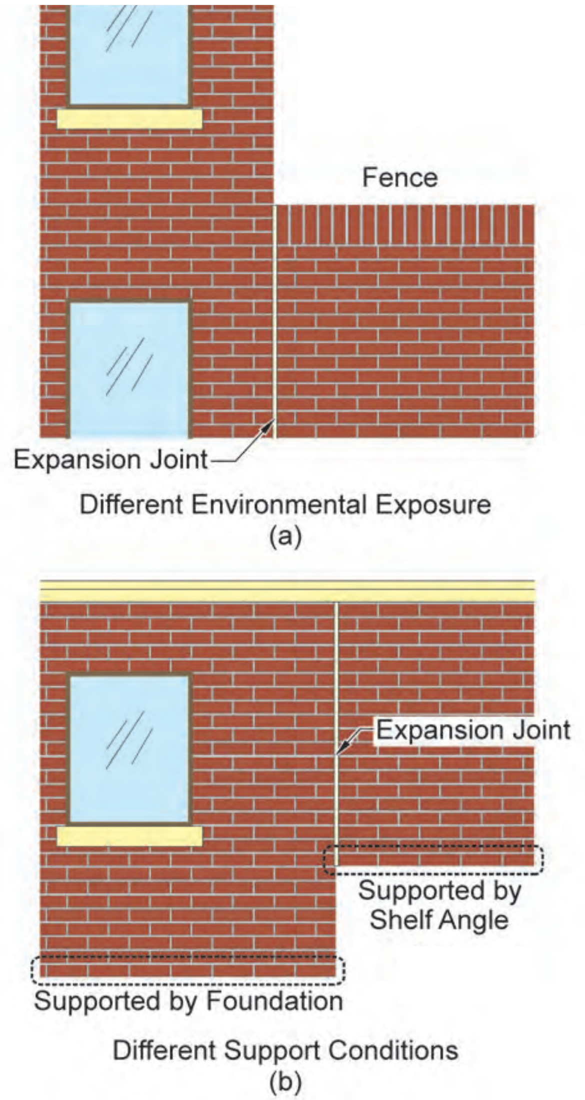


Figure 6
Expansion Joints at Junctions

Locating the expansion joint adjacent to the window when using a loose-laid lintel will influence the dead weight of the masonry bearing on the lintel. The full height and thus weight of the masonry above the opening should be assumed to bear on the lintel instead of the triangular-shaped load typically assumed for loose-laid lintels. See *Technical Note* 31B for more information about steel lintel design.

There are alternatives to placing vertical expansion joints adjacent to window and door openings. Particularly for openings with loose-laid lintels, shifting the joint past the end of the lintel will simplify detailing. A variation on this option is to place expansion joints halfway between the windows. Doing so provides a purposeful aesthetic but requires a sufficiently wide section of masonry between the openings, typically no less than 4 ft (1.2 m). When windows are too close together to permit an expansion joint between them, consider installing expansion joints at each end of the window group. In this case, joint reinforcing is recommended to be placed in the courses directly above and below the window group to reduce the risk of cracking. The joint reinforcing should be engineered.

Junctions. Expansion joints should be located at junctions of walls with different environmental exposures or support conditions. Separate portions of brickwork exposed to different climatic conditions with expansion joints since each area will move differently. An exterior wall containing brickwork that extends into a building's interior should have an expansion joint separating the exterior brickwork from the interior brickwork. Expansion joints should also be installed to separate adjacent walls of different heights to avoid cracking caused by differential movement, particularly when the height difference is very large. Examples are shown in Figure 6.

Parapets. Parapets require special treatment due to their differing configuration compared with a typical building wall. A parapet is exposed to moisture and the environment on three sides instead of one, which increases the amount of movement it can experience. A parapet also lacks sufficient dead load from brickwork above to restrain movement. Because of these conditions, additional accommodations for movement are required in parapets. It is recommended to extend all vertical expansion joints through the parapet and place additional parapet expansion joints approximately halfway between those running full height, such that the spacing between joints is no more than 15 ft (4.6 m) apart at the top of the parapet. These parapet expansion joints must continue to a horizontal expansion joint. Usually they will be terminated at the horizontal joint associated with the shelf angle at the roof level. If joint spacing of no more than 15 ft (4.6 m) cannot be achieved, widen the expansion joints. If additional parapet joints cannot be installed, continuous joint reinforcement should be installed at 16 in. (406 mm) o.c. vertically in the parapet.

Material Transitions. Many modern buildings incorporate a variety of cladding materials in their design, with multiple materials present on the same facade. Expansion joints should always be placed at the transitions between brick and non-masonry cladding systems to accommodate the movement of each material. Closure of the brick air space, flashing and drainage between cladding systems is necessary in many cases. Expansion joints are also required between brick and projecting elements such as pipes, vents and ducts. Refer to *Technical Note* 7 for more information about flashing requirements and recommendations at material transitions. In the case of horizontal joints, increased width to accommodate additional movement due to frame shrinkage may be required.

Masonry Infill. Expansion joints should be placed around masonry infill to isolate it from the surrounding structural frame. The expansion joint along the top course of the infill should accommodate the deflection of the beam, floor or roof system above.

Deflection of Support. Brickwork can be supported by a beam or floor, provided that the maximum deflection of that support is L/600. These spandrel sections of brickwork are subject to stresses from deflection of the support. Reduced spacing between expansion joints will permit deflection to occur without cracking the brickwork.

Support on Wood. *Building Code Requirements for Masonry Structures* (TMS 402) [Ref. 4] and most building codes allow anchored masonry veneer with an installed weight not exceeding 40 lb/ft^2 (1,915 Pa) and a maximum height of 12 ft (3.66 m) to be supported on wood construction, provided that a vertical expansion joint is used to isolate the veneer supported by wood from the veneer supported by the foundation.

Planning Expansion Joint Placement During Design

It can be difficult to decide where to begin when determining the placement of vertical expansion joints. Starting at a corner and placing joints at the typical spacing around the building perimeter is not a recommended approach. The following approach is suggested, which prioritizes placing joints at known areas of stress concentrations prior to considering maximum recommended spacing.

1. Place joints at transitions between brick and other cladding systems or structures.
2. Place joints at junctions such as changes in support conditions and interfaces between walls of differing height.
3. Place joints at all inside corners (offsets and setbacks) with brick on both sides.
4. Place joints near outside corners with brick on both sides, per Figure 2b.
5. Where possible, place joints near detailing such as quoins or reveals to minimize their appearance.
6. Based on whether the brickwork has openings, determine the maximum spacing between joints, and place additional joints where needed. It is often preferable to place joints closer together where they have minimal impact on architectural features rather than spacing them at the recommended maximum distance. Joints at closer spacing can be used to create an aesthetic layout.
7. Extend vertical joints through the parapet. When spacing between vertical joints in a parapet is more than 15 ft (4.6 m), either widen the joints or place additional parapet expansion joints as needed to achieve a maximum 15 ft (4.6 m) spacing.

Aesthetic Effects

Although expansion joints are usually noticeable on flat walls of masonry buildings, there are ways to reduce their visual impact. Architectural features such as quoins, recessed panels of brickwork or a change in bond pattern reduce the visual impact of vertical expansion joints. In some cases, it may be desirable to accentuate the location of the expansion joint as a design detail. This is possible by recessing the brickwork at the expansion joint or by using special-shaped brick units as shown in Photo 3.

Colored sealants that match the brick in running bond, or the mortar in stack bond, help to hide vertical expansion joints. Mason's sand also can be rubbed into new sealant to remove the sheen, making the joint blend in more. Expansion joints also are less noticeable when located at inside corners. Hiding expansion joints behind downspouts or other building elements can inhibit maintenance access and is not advised. Installing expansion joints to follow the masonry bond pattern (toothing or zipper joint) is not recommended. Their shape creates difficulty in keeping debris out of the joint during construction; such debris could interfere with movement. In addition, the articulated shape subjects the sealant to both shear and tension combined, which adversely affects the performance of the sealant.

Symmetrical placement of expansion joints on the elevation of buildings is usually most aesthetically pleasing. Further, placing the expansion joints in a pattern such that wall areas and openings are symmetrical between expansion joints will reduce the likelihood of cracking.

Photo 3
Accentuated Expansion Joint

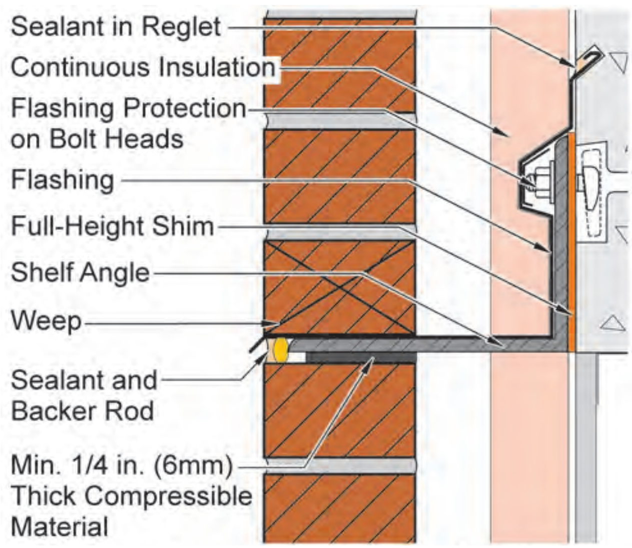


Figure 7
Horizontal Expansion Joint at Shelf Angle

HORIZONTAL EXPANSION JOINTS

Horizontal expansion joints are typically needed if the brick wythe is supported on a shelf angle attached to the frame or used as infill within the frame. Placing horizontal expansion joints below shelf angles provides sufficient space for vertical expansion of the brickwork below and deformation of the shelf angle and the structure to which it is attached. The joint is formed by leaving an unobstructed space, typically ¼ in. (6 mm) in height, or placing a highly compressible material beneath the angle, and a backer rod and sealant at the toe of the angle to seal the joint. Structures that support the brick wythe on shelf angles, usually at each floor, must have horizontal expansion joints under each shelf angle. Larger sized expansion joints may be required to accommodate the differential movement of taller story heights or where a shelf angle supports more than one story of brickwork. Figure 7 shows a typical detail of a horizontal expansion joint beneath a shelf angle.

If the shelf angle is not attached to the structure when the brick below it are laid, then any temporary shims that support the angle during installation must be removed after the shelf angle is connected. It is not necessary to interrupt shelf angles at vertical expansion joint locations. However, shelf angles must be discontinuous to provide for their own thermal expansion. A space of ¼ in. in 20 ft (6 mm in 6 m) of shelf angle length is typically sufficient. Bolt heads anchoring a shelf angle to the structure should be covered to decrease the possibility of flashing puncture.

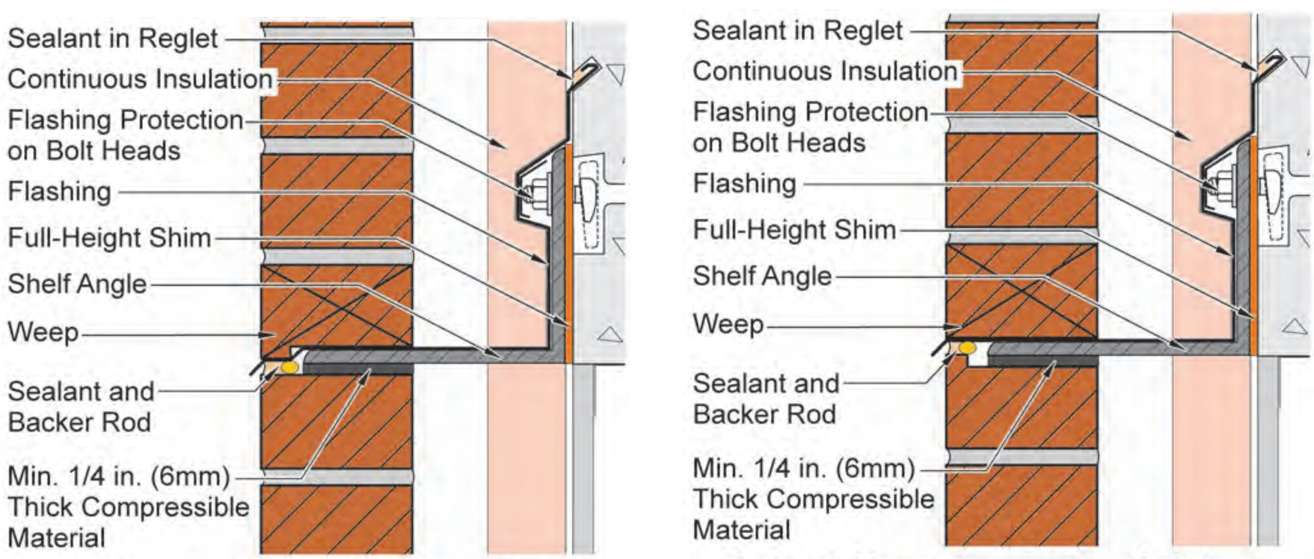
The size of the horizontal expansion joint should take into account movements of the brickwork and movements of the frame. Frame movements include both material and load-induced movements, such as deflections of the shelf angle; rotation of the horizontal leg of the shelf angle; and movement of the support from deflection, temperature change, shrinkage, creep or other factors.

When a large horizontal expansion joint is necessary, a lipped brick course may be used to allow movement while minimizing the aesthetic impact of the joint. To avoid problems with breakage, the height and depth of the lipped portion of the brick should be at least ½ in. (13 mm). When specifying the depth of the lip, keep in mind that at least two-thirds of the total thickness of the brick must bear directly on the shelf angle. No more than one-third of the thickness of the brick wythe is permitted to overhang the shelf angle. For lipped brick, this overhang dimension must include the depth of the lip. Lipped brick should be made by the brick manufacturer for quality assurance purposes.

Construction using lipped brick requires careful consideration of the frame movements noted previously. Allowance for adjacent material tolerances including the building frame should also be considered. Adequate space should be provided between the lipped portion of the brick and the shelf angle to ensure no contact. Contact should not occur between the lipped brick and the brickwork below the shelf angle or between the lip of the brick and the shelf angle, not only during construction, but also throughout the life of the building.

Lipped brick may be installed as the first course above a shelf angle, as shown in Figure 8a. Flashing should be placed between the shelf angle and the lipped brick course. Proper installation of flashing is made more difficult with lipped brick because the flashing must conform to the shape of the lip. This shape may be achieved with more rigid flashing materials or drip edges preformed to the shape of the lip. If the specified flashing materials are made of composite, plastic or rubber, then a sheet metal drip edge should be used. The practice of placing flashing one course above the shelf angle is not recommended, as this can increase the potential for moisture entering the course below.

Lipped brick also may be inverted and placed on the top course of brickwork located directly beneath a shelf angle with the lip oriented upward, as shown in Figure 8b. While installing an inverted lipped brick course allows the flashing of the brickwork above to maintain a straight profile through the brickwork, it also allows the lipped brick



Lipped Brick
(a)

Inverted Lipped Brick
(b)

Figure 8
Alternate Expansion Joint Detail

course to move independent of the shelf angle. As a consequence, there is an increased possibility of the shelf angle coming in contact with the lipped brick course, resulting in cracking at the lip. When a course of inverted lipped brick is installed, it is difficult, if not impossible, to install compressible material below the shelf angle, as well as to access and remove temporary shims supporting the shelf angle above.

Horizontal expansion joints are also recommended when brick is used as an infill material within the frame of the structure. Expansion joints must be provided between the top course of brickwork and the member above. Deflections of the frame should be considered when sizing the expansion joint to avoid inadvertently loading the brickwork.

STRUCTURES WITHOUT SHELF ANGLES

Some buildings with brick veneer construction do not support the brickwork on shelf angles. Low-rise buildings constructed with wood and steel stud framing and buildings with shear walls typically do not exceed prescriptive height limits for masonry veneer and do not need shelf angles to support the brickwork. The *TMS Code* prescriptively permits brick veneer with wood or steel stud framing to a height of 30 ft (9 m) to the top plate and 38 ft (12 m) to the top of a gable. However, there are no prescriptive height limits or intermediate support requirements for brick veneer with a rigid backing of concrete or concrete masonry. Such veneers may be supported without intermediate shelf angles to a recommended maximum height of about 50 ft (15 m), provided that the building is detailed appropriately for the differential movement and that the moisture drainage system is designed and constructed properly.

In these buildings, differential movement is accommodated by the anchor or tie system, window details, and detailing at the top of the wall. These details must provide independent vertical movement between the brickwork and the backing. Building components that extend into or through the brick veneer (windows, doors, vents, etc.) also must be detailed to allow independent vertical movement of the brick veneer and the component. The structural frame or backing provides the brick veneer with lateral support and carries all other vertical loads. The veneer is anchored by flexible connectors or adjustable anchors that permit differential movement. Allowance for differential movement between the exterior brickwork and the adjacent components should be provided at all openings and at the tops of walls. Vertical expansion joints also must be incorporated, as discussed in previous sections of this *Technical Note*.

Connectors, anchors or ties that transfer load from the brick wythe to a structural frame or backing that provides lateral support should resist movement perpendicular to the plane of the wall (tension and compression) but allow movement parallel to the wall without becoming disengaged. This flexible anchorage permits differential movement between the structure and the brickwork. Figure 9 shows typical methods for anchoring masonry walls to columns and beams. *Technical Note* 44B provides detailed information about masonry ties and anchors.

The size and spacing of anchors and ties are based on tensile and compressive loads induced by lateral loads on the walls or on prescriptive anchor and tie spacing requirements in building codes. *Technical Note* 44B lists recommended tie spacing based on application.

There must be sufficient clearance among the masonry elements and the beams and columns of the structural frame to permit the expected differential movement. The masonry walls may be more rigid than the structural frame. This clearance provides isolation between the brickwork and frame, allowing independent movement.

COMBINING MATERIALS

As discussed in *Technical Note* 18, brick have different movement properties compared with other building materials. When other materials are used in combination with brick, the movement properties of that building material (concrete, concrete masonry cast stone, etc.) must be considered. To reduce the potential for cracking in a multi-wythe wall of brick and concrete masonry, movement joints must be installed in each wythe to accommodate the differential movement between the materials. In this case, expansion joints are placed in the brick wythe, and control joints are placed in the concrete masonry, although they do not necessarily have to be aligned through the wall. Another way to separate wythes or bands of materials that express different movement properties is to install a bond break to allow each material to move independently.

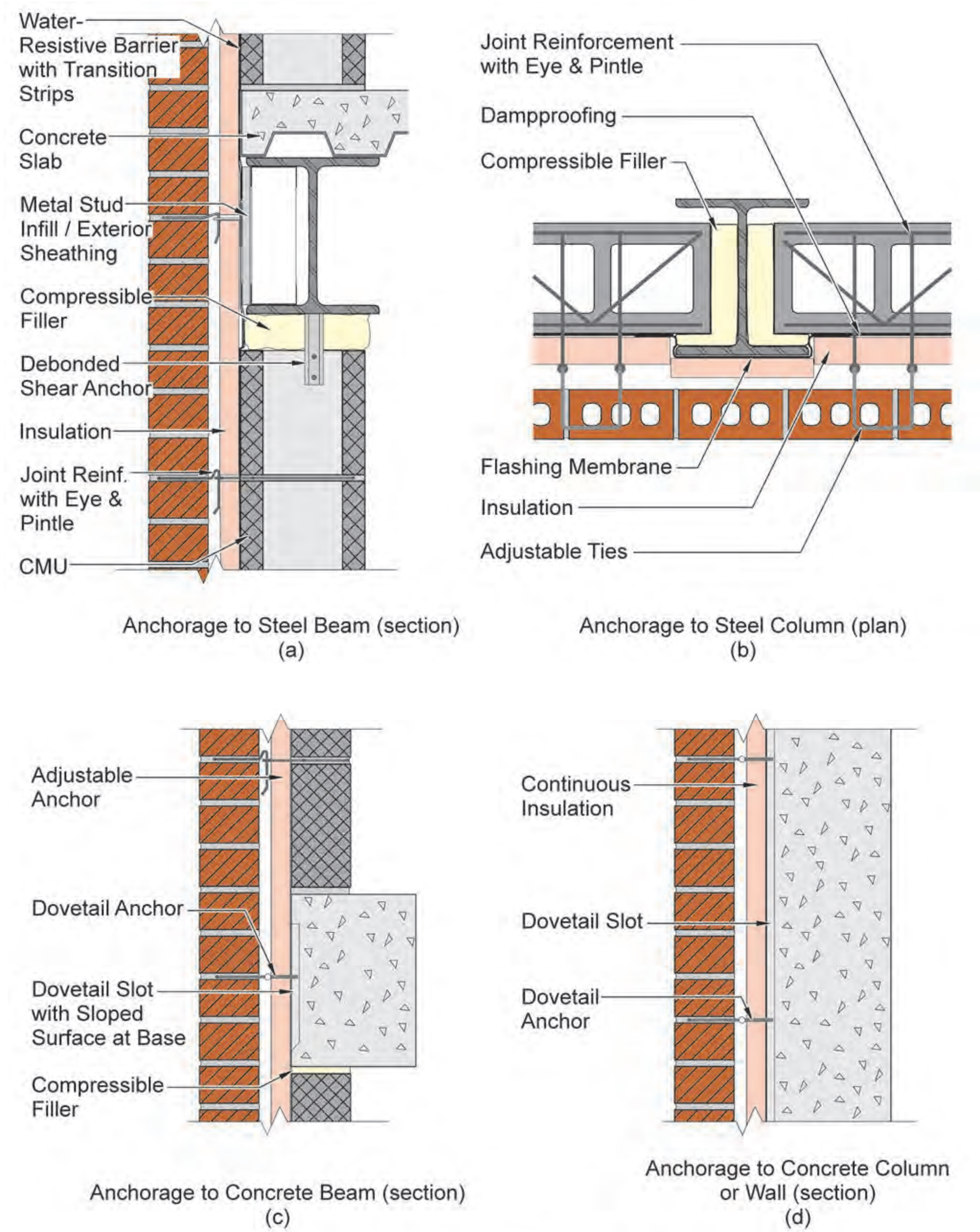


Figure 9
Flexible Anchorage to Beams and Columns

Bond Breaks

Concrete and concrete masonry have moisture and thermal movements that are considerably different from those of brick masonry. Floor slabs and foundations also experience different states of stress due to their loading and support conditions. Therefore, it may be necessary to separate brickwork from these elements using a bond break such as building paper, flashing, or 4 to 6 mil thick polyethylene sheeting. Such bond breaks should be provided between foundations and walls, between slabs and walls, and between concrete and clay masonry to allow independent movement while still providing gravity support. Typical methods of breaking bond between walls and slabs, and between walls and foundations, are shown in Figure 10.

When bands of clay brick are used in concrete masonry walls, or when bands of concrete masonry or cast stone are used in clay brick walls, differences in material properties may cause mortar joints or masonry units to crack. Such problems can be easily avoided by using bands of brickwork featuring brick of a different color, size or texture, or a different bond pattern. If, however, a different material is used for the band, it may be prudent to install a bond break between the two materials, provide additional movement joints in the wall, or place joint reinforcement in the bed joints of the concrete masonry to reduce the potential for cracking.

Breaking the bond in this way does not affect the compressive strength of the wall and should not affect the stability of the veneer wythe when anchored properly. The weight of the masonry, additional anchorage and the frictional properties at the interface provide stability. Sealant at the face of the joints between the different materials will reduce possible water entry. If the band is concrete masonry or cast stone, then additional control joints are recommended in the band. If the band is a single course, then there is a likelihood of vertical cracks at all head joints. These can be closed with a sealant. Bands of two or more courses should include horizontal joint reinforcement in the intervening bed joints, as shown in Figure 11.

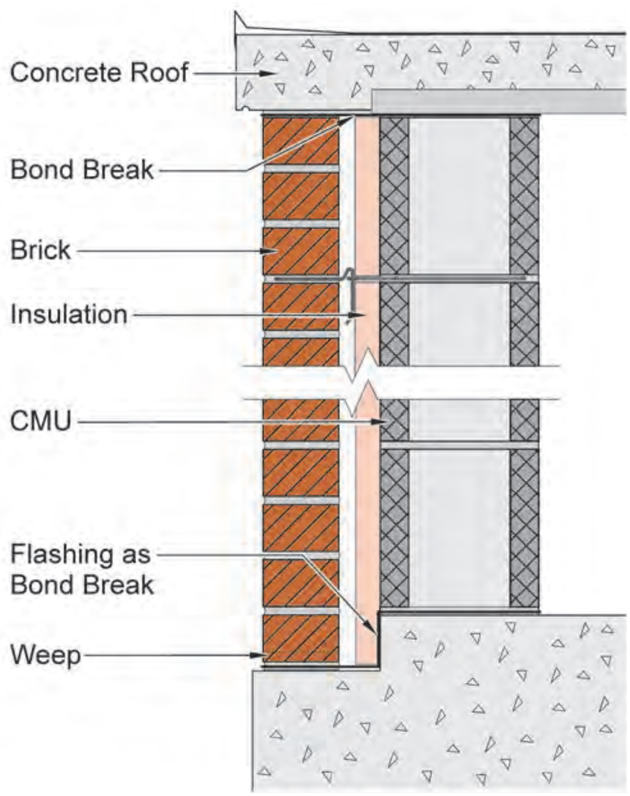


Figure 10
Bond Breaks in Cavity Wall

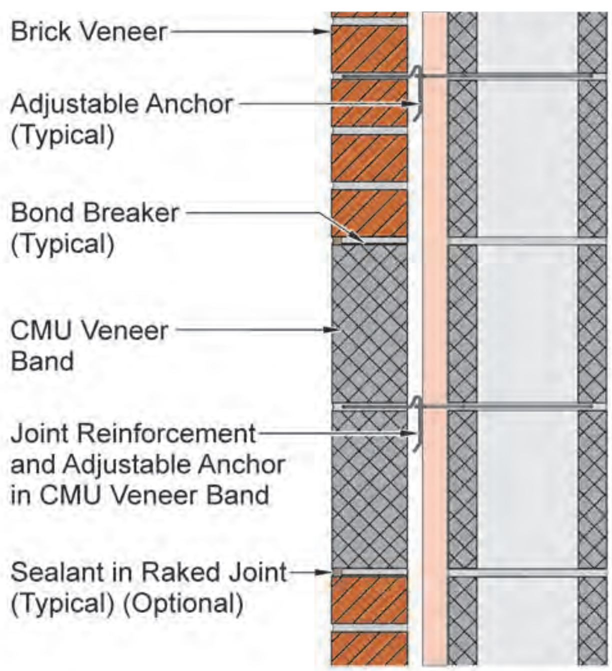


Figure 11
Multi-Course Concrete Masonry Band

LOAD-BEARING MASONRY

The potential for cracking in load-bearing masonry members is less than in non-load-bearing masonry members because compressive stresses from dead and live loads help offset the effects of any movement. Adding reinforcement at critical sections such as parapets, points of load application and around openings to

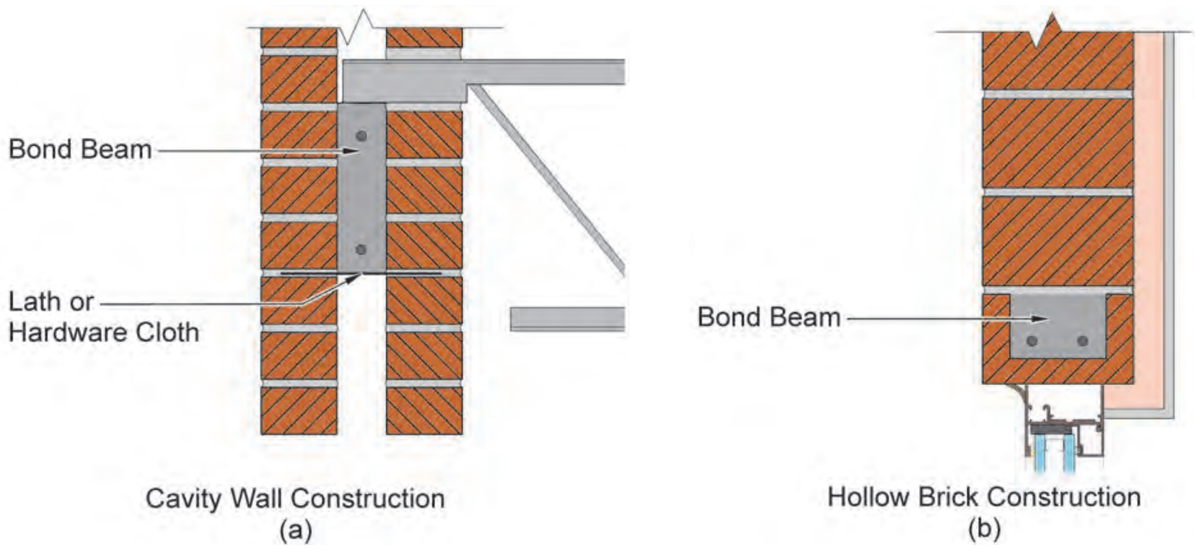


Figure 12
Bond Beams

accommodate or distribute high stresses will also help control the effects of movement. Reinforcement can be placed in bed joints or in bond beams, as shown in Figure 12. Historic load-bearing structures were not constructed with expansion joints. However, these walls were constructed using multi-wythe brick construction, unlike typical structures built today.

SUMMARY

This *Technical Note* defines the types of movement joints used in building construction. Details of expansion joints used in brickwork are shown. The recommended size, spacing and location of expansion joints are given. By using the suggestions in this *Technical Note*, the potential for cracks in brickwork can be reduced.

Expansion joints are used in brick masonry to accommodate the movement experienced by materials as they react to environmental conditions, adjacent materials and loads. In general, vertical expansion joints should be used to break the brickwork into rectangular elements that have the same support conditions, climatic exposure and through-wall construction. The maximum recommended spacing of vertical expansion joints is 25 ft (7.6 m). Horizontal expansion joints must be placed below shelf angles supporting brick masonry.

> *The information and suggestions contained in this* Technical Note *are based on the available data and the combined experience of engineering staff and members of the Brick Industry Association. The information contained herein must be used in conjunction with good technical judgment and a basic understanding of the properties of brick masonry. Final decisions on the use of the information contained in this* Technical Note *are not within the purview of the Brick Industry Association and must rest with the project architect, engineer and owner.*

REFERENCES

1. ASTM C920, Standard Guide for Use of Elastomeric Joint Sealants, *Annual Book of Standards*, Vol. 04.07, ASTM International, West Conshohocken, PA, 2018.
2. Beall, C., *Masonry Design and Detailing for Architects, Engineers and Contractors*, Sixth Edition, McGraw Hill Inc., New York, NY, 2012.
3. Beall, C., “Sealant Joint Design,” *Water on Exterior Building Walls: Problems and Solutions*, ASTM STP 1107, T.A. Schwartz, Ed., ASTM, West Conshohocken, PA, 1991.
4. *Building Code Requirements for Masonry Structures* (TMS 402-16), The Masonry Society, Longmont, CO, 2016.
5. “Building Movements and Joints,” Portland Cement Association, Skokie, IL, 1982.

Technical Notes
on Brick Construction
Brick Industry Association 11490 Commerce Park Drive, Reston, Virginia 20191

31B
REVISED

Reissued*
May
1987

STRUCTURAL STEEL LINTELS

Abstract: The design of structural steel lintels for use with brick masonry is too critical an element to be left to "rule-of-thumb" designs. Too little concern for loads, stresses and serviceability can lead to problems. Information is provided so that structural steel lintels for use in brick masonry walls may be satisfactorily designed.

Key Words: **beams** (supports); **brick**; **buildings**; **deflection**; **design**; lintels; **loads** (forces); **masonry**; structural steel; **walls**.

INTRODUCTION

A lintel is a structural member placed over an opening in a wall. In the case of a brick masonry wall, lintels may consist of reinforced brick masonry, brick masonry arches, precast concrete or structural steel shapes. Regardless of the material chosen for the lintel, its prime function is to support the loads above the opening, and it must be designed properly. To eliminate the possibility of structural cracks in the wall above these openings, the structural design of the lintels should not involve the use of "rule-of-thumb" methods, or the arbitrary selection of structural sections without careful analysis of the loads to be carried and calculation of the stresses developed. Many of the cracks which appear over openings in masonry walls are due to excessive deflection of the lintels resulting from improper or inadequate design.

This *Technical Notes* presents the considerations to be addressed if structural steel lintels are to be used. It also provides a procedure for the structural design of these lintels. For information concerning reinforced brick masonry lintels, see *Technical Notes* 17H and for brick masonry arches, see *Technical Notes* 31, 31A and 31C Revised.

CONSIDERATIONS

General

When structural steel lintels are used, there are several considerations which must be addressed in order to have a successful design. These include loading, type of lintel, structural design, material selection and maintenance, moisture control around the opening, provisions to avoid movement problems and installation of the lintel in the wall.

Types

There are several different types of structural steel lintels used in masonry. They vary from single angle lintels in cavity or veneer walls, to steel beams with plates in solid walls, to shelf angles in brick veneer panel walls. Most building codes permit steel angle lintels to be used for openings up to 8 ft 0 in. (2.4 m). Openings larger than this are usually required to have fire protected lintels.

Loose Angle Lintels. Loose angle lintels are used in brick veneer and cavity wall constructions where the lintel is laid in the wall and spans the opening. This type of lintel has no lateral support. Figure 1a shows this condition.

Combination Lintels. In solid masonry walls, single loose angle lintels are usually not capable of doing the job. Therefore, combination lintels are required. These combination lintels can take many forms, from a clustering of steel angles, such as shown in Figs. 1b and 1c, to a combination of steel beam and plates, as shown in Figs. 1d and 1e.

Angle Lintels - In solid masonry walls, it is usually satisfactory to use multiple steel angles as a lintel. These angles are usually placed back to back, as shown in Figs. 1b and 1c.

Steel Beam/Plate Lintels - In solid walls with large superimposed loads, or in walls where the openings are greater than 8 ft 0 in. (2.4 m), it may be necessary to use lintels composed of steel beams with attached or suspended plates, as shown in Figs. 1d and 1e. This permits the beam to be fully encased in masonry, and fire-protected.

Shelf Angles. In panel walls systems, the exterior wythe of brickwork may be supported by shelf angles rigidly attached to the structural frame. These shelf angles, in some cases, also act as lintels over openings in the masonry. This condition is shown in Fig.1f.

*Originally published in Nov/Dec 1981, this *Technical Notes* has been reviewed and reissued.

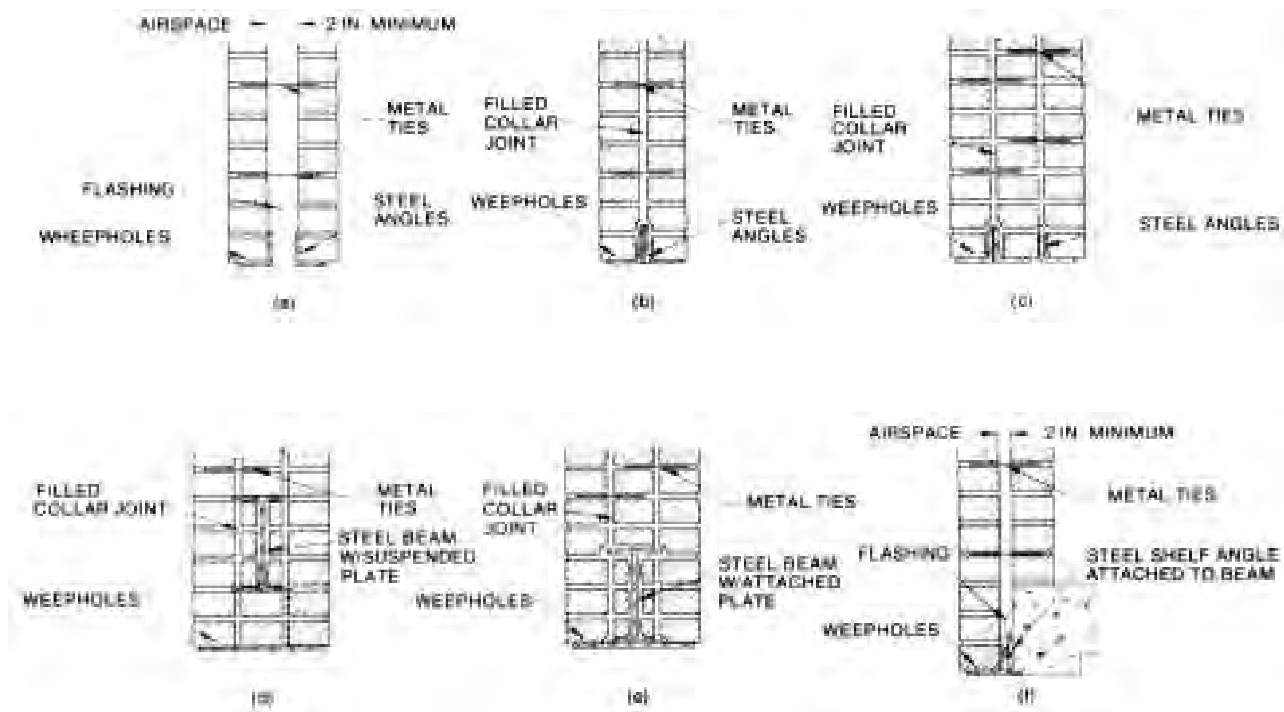


Fig. 1
Types of Structural Steel Lintels

Design

The proper design of the structural steel lintel is very important, regardless of the type used. The design must meet the structural requirements and the serviceability requirements in order to perform successfully. Design loads, stresses and deflections will be covered in a later section of this *Technical Notes*.

Materials

The proper specification of materials for steel lintels is important for both structural and serviceability requirements. If materials are not properly selected and maintained, problems can occur.

Selection. The steel for lintels, as a minimum, should comply with ASTM A 36. Steel angle lintels should be at least 1/4 in. (6 mm) thick with a horizontal leg of at least 3 1/2 in. (90 mm) for use with nominal 4 in. (100 mm) thick brick, and 3 in. (75 mm) for use with nominal 3 in. (75 mm) thick brick.

Maintenance. For harsh climates and exposures, consideration should be given to the use of galvanized steel lintels. If this is not done, then the steel lintels will require periodic maintenance to avoid corrosion.

Moisture Control

Proper consideration must always be given to moisture control wherever there are openings in masonry walls. There must always be a mechanism to channel the flow of water, present in the wall, to the outside.

Flashing and Weepholes. Even where galvanized or stainless steel angles are used for lintels in cavity and veneer walls, continuous flashing should be installed over the angle. It should be placed between the steel and the exterior masonry facing material to collect and divert moisture to the outside through weepholes. Regardless of whether flashing is used, weepholes should be provided in the facing at the level of the lintel to permit the escape of any accumulated moisture. See *Technical Notes* 7A for further information on flashing and weepholes.

Movement Provisions

Because of the diversity of movement characteristics of different materials, it is necessary to provide for differential movement of the materials. This is especially true at locations where a number of different materials come together. *Technical Notes* 18 Series provides additional information on differential movement.

Expansion Joints. Expansion joints in brick masonry are very important in preventing unnecessary and unwanted cracking. There are two types of expansion joints which will need to be carefully detailed when lintels are involved: vertical and horizontal.

Vertical - Vertical expansion joints are provided to permit the horizontal movement of the brick masonry. Where these expansion joints are interrupted by lintels, the expansion joint should go around the end of the lintel and then continue down the wall.

Horizontal - In multi-story walls where the lintels are a continuation of shelf angles supporting masonry panels, horizontal expansion joints to accommodate vertical movement must be provided. Often a simple soft joint below the shelf angle is all that is needed. See *Technical Notes* 18A, 21 Rev, and 28B Rev for typical details.

Installation

The installation of steel lintels in masonry walls is a conventional construction operation, familiar to most members of the building team. The walls are built to the height of the opening, the lintel is placed over the opening, and the masonry work is continued. One item of special construction that must be noted is temporary shoring.

Temporary Shoring. If the steel lintel is being designed assuming in-plane arching of the masonry above, then the lintel must be shored until the masonry has attained sufficient strength to carry its own weight. This shoring period should not be less than 24 hr. This minimum time period should be increased to three days when there are imposed loads to be supported. If the masonry is being built in cold weather construction conditions, the length of cure should be increased. If the lintel is designed for the full uniform load of the masonry and other superimposed loads ignoring any inherent arching action, then no shoring is required.

STRUCTURAL DESIGN

General

The structural design of steel lintels is relatively simple. The computations are the same as for steel beams in a building frame, but because of the low elasticity of the masonry, and the magnitude and eccentricity of the loading, the design should not be taken lightly. A proper design must consider the loads, stresses, and serviceability of the system. If these are not properly taken into account, problems of cracking and spalling could occur.

Loads

The determination of imposed loads is an important factor. Fig. 2 shows an example of a lintel design situation. On the left is an elevation showing an opening in a wall with planks and a beam bearing on the wall. On the right is a graphic illustration of the distribution of the superimposed loads.

Uniform Loads. The triangular wall area (ABC) in Fig. 2b above the opening has sides at 45-deg angles to the base. Arching action of a masonry wall will carry the dead weight of the wall and the superimposed loads outside this triangle, provided that the wall above Point B (the top of the triangle) is sufficient to provide resistance to arching thrusts. For most lintels of ordinary wall thickness, loads and spans, a depth of 8 to 16 in. (200 mm to 400 mm) above the apex is sufficient. If stack bonded masonry is used, horizontal joint reinforcement must be provided to ensure the arching action.

Providing arching action occurs, the dead weight of the masonry wall, carried by the lintel, may be safely assumed as the weight of masonry enclosed within the triangular area (ABC). To the dead load of the wall must be added the uniform live and dead loads of the floor bearing on the wall above the opening and below the apex of the 45-deg triangle. Again, providing arching occurs, such loads above the apex may be neglected. In Fig. 2b, D is greater than L/2, so the floor load may be ignored, but, in order to use this assumed loading, temporary shoring must be provided until the masonry has cured sufficiently to assure the arching action.

If arching action is not assumed and temporary shorting is not to be used, the steel lintel must be designed for the full weight of the masonry and other superimposed live and dead loads above the opening. There could be quite a substantial difference in the final lintel sizes required in each case.

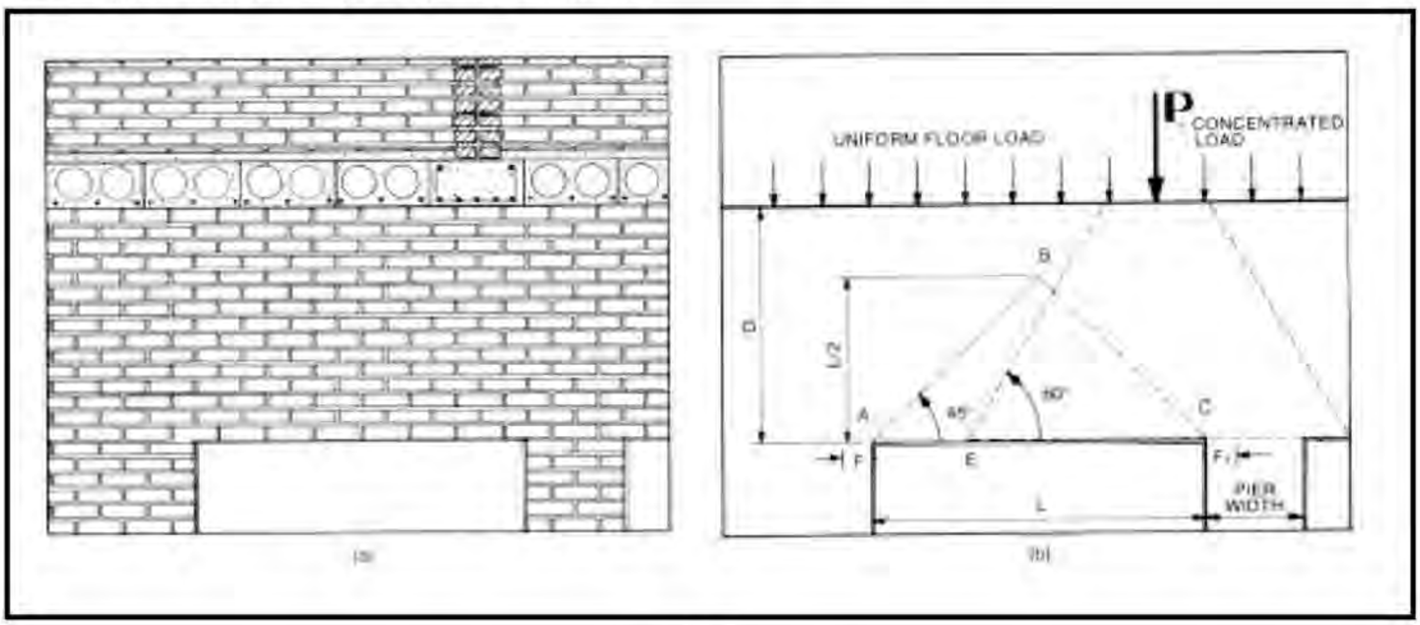


Fig. 2
Lintel Load Determination

Concentrated Loads. Concentrated loads from beams, girders, or trusses, framing into the wall above the opening, must also be taken into consideration. Such loads may be distributed over a wall length equal to the base of the trapezoid and whose summit is at the point of load application and whose sides make an angle of 60 deg with the horizontal. In Fig. 2b, the portion of the concentrated load carried by the lintel would be distributed over the length, EC, and would be considered as a partially distributed uniform load. Arching action of the masonry is not assumed when designing for concentrated loads. Again, if stack bonded masonry is used, horizontal joint reinforcement must be provided to assure this distribution.

Stresses

After the loads have been determined, the next step in the design of the lintel is the design for stresses. Which stresses need to be checked will depend upon the type and detailing of the lintel.

Flexure. In a simply supported member loaded through its shear center, the maximum bending moment due to the triangular wall area (ABC) above the opening can be determined by:

$$M_{max} = \frac{WL}{6}$$

where:

M_{max} = maximum moment (ft---lb)

W = total load on lintel (lb)

L = span of lintel, center to center of end bearing (ft)

As an alternative, the designer may wish to calculate an equivalent uniform load by taking 2/3 of the maximum height of the triangle times the unit weight of the masonry as the uniform load across the entire lintel. If this is done, the maximum bending moment equation becomes:

$$M_{max} = \frac{wL^2}{8}$$

where:

w = equivalent uniformly distributed load per unit of length (lb per ft).

To this bending moment should be added the bending moment caused by the concentrated loading, if any. Where such loads are located far enough above the lintel to be distributed as shown in Fig. 2b, the bending moment formula for a partially distributed uniform load may be used. Such formulae may be found in the " Manual of Steel Construction," by the American Institute of Steel Construction (AISC). Otherwise, concentrated load bending moments should be used.

The next step is the selection of the required section. The angle, or other structural steel shape, should be selected by first determining the required section modulus. This becomes:

$$S = \frac{12M_{max}}{F_b}$$

where:

S = section modulus (in^3)

F_b = allowable stress in bending of steel (psi)

The allowable stress, F_b, for ASTM A 36 structural steel is 22,000 psi (150 MPa) for members laterally supported. Solid brick masonry walls under most conditions provide sufficient lateral stiffness to permit the use of the full 22,000 psi (150 MPa). This is especially true when floors or roofs frame into the wall immediately above the lintel. The design for non-laterally supported lintels should be in accordance with the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*.

Using the design property tables in the AISC Manual, a section having an elastic section modulus equal to, or slightly greater than, the required section modulus is selected. Whenever possible, within the limitations of minimum thickness of steel and the length of outstanding leg required the lightest section having the required section modulus should be chosen.

Combined Flexure and Torsion. In some cases, the design for flexure will need to be modified to include the effects of torsion. This is the case in cavity and veneer walls where the load on the angle is not through the shear center.

In some situations, such as veneers, panel or curtain walls, the lintel may be supporting only the triangular portion of masonry directly over the opening. If this is the case, then the torsional stresses will usually be negligible compared to the flexural stresses, and can be safely ignored.

If, on the other hand, there are imposed uniform loads within the triangle or imposed concentrated loads above the lintel, then a detailed, combined stress analysis will be necessary. The design of a lintel subjected to combined flexure and torsion should be in accordance with the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.*

Shear. Shear is a maximum at the end supports, and for steel lintels it is seldom critical. However, the computation of the unit shear is a simple calculation and should not be neglected. The allowable unit shear value for ASTM A 36 structural steel is 14,500 psi (100 MPa). To calculate the shear:

$$v_{max} = \frac{R_{max}}{A_s}$$

where:

V_{max} = the actual maximum unit shear (psi)

R_{max} = maximum reaction (lb)

A_s = area of steel section resisting shear (sq. in.)

Bearing. In order to determine the overall length of a steel lintel, the required bearing area must be determined. The stress in the masonry supporting each end of the lintel should not exceed the allowable unit stress for the type of masonry used. For allowable bearing stresses, see "Building Code Requirements for Engineered Brick Masonry," BIA; "American Standard Building Code Requirements for Masonry," ANSI A41.1-1953 (R 1970); or the local building code. The reaction at each end of the lintel will be one-half the total uniform load on the lintel, plus a proportion of any concentrated load or partially distributed uniform load. The required area may be found by:

$$A_b = \frac{R_{max}}{f_m}$$

where:

A_b = required bearing area (sq in.)

f_m = allowable compressive stress in masonry (psi)

In addition, any stresses due to rotation from bending or torsion of the angle at its bearing must be taken into account.

Since in selecting the steel section, the width of the section was determined, that width divided into the required bearing area, A_b, will determine the length of bearing required, F and F_1, in Fig. 2b. This length should not be less than 3 in. (75 mm).

If the openings are close together, the piers between these openings must be investigated to determine whether the reactions from the lintels plus the dead and live loads acting on the pier exceed the allowable unit compressive stress of the masonry. This condition will not normally occur where the loads are light, such as in most one and two-story structures.

Serviceability

In addition to the stress analysis for the lintel, a serviceability analysis is also important. Different types of lintels have different problems of deflection and rotation, and each must be analyzed separately to assure its proper performance.

Deflection Limitations. After the lintel has been designed for stresses, it should be checked for deflection. Lintels supporting masonry should be designed so that their deflection does not exceed 1/600 of the clear span nor more than 0.3 in (8 mm) under the combined superimposed live and dead loads.

For uniform loading, the deflection can be found by:

$$\Delta_t = \frac{5wl^4 (1728)}{384 EI}$$

where:

Δ_t = total maximum deflection (in.)

E = modulus of elasticity of steel (psi)

I = moment of inertia of section (in.^{4})

For loadings other than uniform, such as concentrated loads and partially distributed loads, deflection formulae may be found in the AISC Manual.

Torsional Limitations. In cases where torsion is present, the rotation of the lintel can be as important as its deflection. The rotation of the lintel should be limited to 1/16 in. (1.5 mm) maximum under the combined superimposed live and dead loads. As mentioned before, all additional bearing stresses due to angle rotation must be taken into account in the design for bearing.

Design Aids

In order to facilitate the design of steel angle lintels, several design aids are included. These design aids are not all-inclusive, but should give the designer some help in designing lintels for typical applications. Conditions beyond the scope of these tables should be thoroughly investigated.

Table 1 contains tabulated load values to assist the designer in the selection of the proper size angle lintel, governed either by moment or deflection under uniform load. Shear does not govern in any of the listed cases. The deflection limitation in Table 1 is 1/600 of the span, or 0.3 in. (8 mm), whichever is less. Lateral support is assumed in all cases.

Table 2 lists the allowable bearing stresses taken from ANSI A41.1-1953 (R 1970). In all cases, allowable bearing stresses set by local jurisdictions in their building codes will govern.

Table 3 lists end reactions and required length in bearing, which may control for steel angle lintels.

SUMMARY

This *Technical Notes* is concerned primarily with the design of structural steel lintels for use in brick masonry walls. It presents the considerations which must be addressed for the proper application of this type of masonry support system. Other *Technical Notes* address the subjects of reinforced brick masonry lintels and brick masonry arches.

The information and suggestions contained in this *Technical Notes* are based on the available data and the experience of the technical staff of the Brick Institute of America. The information and recommendations contained herein, if followed with the use of good technical judgment, will avoid many of the problems discussed. Final decisions on the use of details and materials as discussed are not within the purview of the Brick Institute of America, and must rest with the project designer, owner, or both.

TABLE 1

Allowable Uniform Superimposed Load (lb per ft) for ASTM A 36 Structural Steel Angle Lintels [1,2,3,4,5,6]

Horizontal Leg (in)	Angle Size (in x in x in)	Weight per ft (lb)	Span in Feet (Center to Center of Required Bearing						Resisting Moment (ft-lb)	Elastic Section Modulus (in^3)	Moment of Inertia (in^4)
			3	4	5	6	7	8			
2 1/2	2 x 2 1/2 x 1/4	3.6	352	146	73				458	0.25	0.372
	2 1/2 x 2 1/2 x 1/4	4.1	631	279	141	80			715	0.39	0.703
	5/16	5.0	777	336	170	96			880	0.48	0.849
	3/8	5.9	923	390	197	112			1045	0.57	0.984
	3 x 2 1/2 x 1/4	4.5	908	467	237	135	83		1027	0.56	1.17
	3 1/2 x 2 1/2 x 1/4	4.9	1233	692	366	210	130	86	1393	0.76	1.80
	5/16	6.1	1509	846	446	255	158	104	1705	0.93	2.19
	3/8	7.2	1769	992	521	298	185	122	1998	1.09	2.56
3 1/2	2 1/2 x 3 1/2 x 1/4	4.9	664	308	155	88			752	0.41	0.777
	3 x 3 1/2 x 1/4	5.4	956	518	263	150	92		1082	0.59	1.30
	3 1/2 x 3 1/2 x 1/4	5.8	1281	718	409	234	145	95	1448	0.79	2.01
	5/16	7.2	1590	891	498	285	177	116	1797	0.98	2.45
	3/8	8.5	1865	1046	583	334	207	136	2108	1.15	2.87
	4 x 3 1/2 x 1/4	6.2	1672	938	594	341	212	140	1888	1.03	2.91
	5/16	7.7	2046	1147	726	417	260	172	2310	1.26	3.56
	5 x 3 1/2 x 5/16	8.7	3153	1770	1130	779	487	324	3557	1.94	6.60
	3/8	10.4	3721	2089	1333	918	574	381	4198	2.29	7.78
	6 x 3 1/2 x 3/8	11.7	5268	2958	1889	1308	958	638	5940	3.24	12.90

[1] Allowable loads to the left of the heavy line are governed by moment, and to the right by deflection.
[2] F_b = 22,000 psi (150 MPa)
[3] Maximum deflection limited to L/600
[4] Lateral support is assumed in all cases.
[5] For angles laterally unsupported, allowable load must be reduced.
[6] For angles subjected to torsion, make special investigation.

TABLE 2

Allowable Compressive Stresses (psi) in Masonry [1]

Type of Wall	Type of Mortar			
	M	S	N	O
Solid walls of brick or solid units of clay when average compressive strength of unit is as follows:				
8000 plus psi	400	350	300	200
4500 to 8000 psi	250	225	200	150
2500 to 4500 psi	175	160	140	110
1500 to 2500 psi	125	115	100	75
Grouted solid masonry of brick and other solid units of clay				
4500 plus psi	350	275	200	-
2500 to 4500 psi	275	215	155	-
1500 to 2500 psi	225	175	125	-
Masonry of hollow units	85	75	70	-

1 Adapted from "American Standard Building Code Requirements for Masonry," National Bureau of Standards, ANSI A41. 1-1953 (R 1970).

TABLE 3
End Reaction[1] and Required Length of Bearing[2] for Structural Angle Lintels

2 1/2" Leg Horizontal				
f_m	**Length of Bearing**			
psi	**3**	**4**	**5**	**6**
400	3000	4000	5000	6000
350	2625	3500	4375	5250
300	2250	3000	3750	4500
275	2063	2750	3438	4125
250	1875	2500	3125	3750
225	1688	2250	2813	3375
215	1613	2150	2688	3225
200	1500	2000	2500	3000
175	1313	1750	2188	2625
160	1200	1600	2000	2400
155	1163	1550	1938	2325
150	1125	1500	1875	2250
140	1050	1400	1750	2100
125	938	1250	1563	1875
115	863	1150	1438	1725
110	825	1100	1375	1650
100	750	1000	1250	1500
85	638	850	1063	1275
75	563	750	938	1125
70	525	700	875	1050

31/2" Leg Horizontal				
f_m	**Length of Bearing**			
psi	**3**	**4**	**5**	**6**
400	4200	5600	7000	8400
350	3675	4900	6125	7350
300	3150	4200	5250	6300
275	2888	3850	4813	5775
250	2625	3500	4375	5250
225	2363	3150	3938	4725
215	2258	3010	3763	4515
200	2100	2800	3500	4200
175	1838	2450	3063	3675
160	1680	2240	2800	3360
155	1628	2170	2713	3255
150	1575	2100	2625	3150
140	1470	1960	2450	2940
125	1313	1750	2188	2625
115	1208	1610	2013	2415
110	1155	1540	1925	2310
100	1050	1400	1750	2100
85	893	1190	1488	1785
75	788	1050	1313	1575
70	735	980	1225	1470

[1] End Reaction in lbs.
[2] Length of Bearing in inches.

REFERENCES

1. *AISC, Manual of Steel Construction*, American Institute of Steel Construction, Inc., New York, New York, Eighth Edition, 1980.
2. AISC, *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, Inc., New York, New York, 1978.
3. ANSI, *American Standard Building Code Requirements for Masonry*, ANSI A41.1-1953 (R 1970), American National Standards Institute, New York, New York.
4. BIA, *Building Code Requirements for Engineered Brick Masonry*, Brick Institute of America, McLean, Virginia, 1969.

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