Gannett Fleming

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January 30, 2017

Mr. Anthony Vigorito
Plant Operations Manager
The Mahoning Valley Sanitary District
1181 Ohltown McDonald Road
Mineral Ridge, Ohio 44440

Subject: Contract No. G-102, Amendment to March 26, 2014 Engineering Services Agreement
Dam & Spillway Improvements Project
PO No. 2016000317: Opinion on Structural Issues Associated with the Dam and Buildings Task - Letter Report Deliverable
GF Project No. 58721

Dear Mr. Vigorito:

Gannett Fleming is pleased to submit this Letter Report providing Mahoning Valley Sanitary District (District) with an opinion on structural issues associated with Mineral Ridge Dam and water treatment-related buildings at the District's Meander Reservoir / Ohltown-McDonald Road campus location. This deliverable has been prepared in accordance with the requirements of the referenced Purchase Order and Gannett Fleming's proposal dated October 18, 2016, and in accordance with discussions during our January 19, 2017 Webex teleconference.

Background

Under this Purchase Order, Gannett Fleming:

- Performed a limited detail review of documents provided by the District prior to conducting a site visit. The documents provided for review included as-built structural and architectural drawings of the filter building addition and related prior engineering condition assessment reports and as-built drawings of concrete and masonry structures at the dam.
- Participated in a one-day site visit on November 21, 2016. Attendees during the site visit included Mr. Vladimir Cecka, P.E., of Gannett Fleming and Mr. Thomas Holloway, Chief Engineer. During the visit, Mr. Cecka observed and documented pertinent conditions related to cracking visible at the buildings and dam and obtained background information and historical perspective related to the cracking. The site visit was conducted on foot, walking inside and outside of the buildings and taking notes and photographs. The names and numbers of buildings and the dam features that are referenced herein are shown on the site plan in Exhibit 1. Detailed inspection, measurements, or survey and mapping of cracks was not performed during the site visit.

Findings and observations from the site visit and a summary assessment of observed conditions are provided below.

Findings/Observations

Building 4 – High Service Pumping Station

- The low roof portion of this building is comprised of masonry bearing walls with steel trusses, and the high roof portion, over the pump room, is comprised of non-bearing walls with steel columns and trusses.
- A vertical crack was observed in the brick exterior at a corner running from the top of the concrete portion to the roof at three locations on this building. See attached Photos 1 & 2.

Building 7 - Headhouse

- This building 7 consists of lime bins and chemical feed areas at the center portion of the building that is high with lower building wings on each side. The high roof portion of the building is a steel framed building encased in concrete. Two low roof portions of the building on the west side have exterior steel columns with steel trusses. Two low roof portions of the building on the east side have exterior masonry bearing walls with interior steel columns and steel trusses.
- There are a few horizontal cracks at the upper level brick causing external displacement of some bricks. It appears this condition is located near the interior floor elevations. See attached Photo 3.
- There are multiple locations with cracks in the brick exterior, both in the lower and upper portions of the building. Many were recently sealed to prevent water intrusion. Typically the cracks extend vertically the entire height. See attached Photos 4, 5, 6 & 7.

Building 11 – Carbon Feed

- This building was built in the 1990s and has a brick exterior.
- No cracks were observed. See attached Photo 8.

Building 15 – Warehouse

- This building has bearing masonry brick walls supporting steel beams and a precast concrete roof panel system.
- A plate at bottom of steel lintel beam above an entrance roll up door is deflecting and separating from the steel beam. See attached Photo 9. This building has minor vertical and diagonal cracking in the brick exterior with the majority of the cracks already repaired. See attached Photo 10.

Building 16 – Boiler House

- This building has bearing brick masonry walls and steel trusses.
- The brick is generally in good condition with very few cracks. See attached Photo 11.

Building 18 – Chemical Building

- This building was built in the 1990s and has a brick exterior.
- No cracks were observed. See attached Photo 12.

Building 19 – Filter Building

- This building has interior steel columns supporting steel roof trusses and metal roof decking. The exterior walls are masonry brick with pilasters to support the trusses. An addition was construction in 1958 on the north side of the existing Filter Building with similar construction as the existing building.
- There are two diagonal cracks on the interior of the brick located on outside corners on the west side wall of the original building and minor interior cracks located throughout. These diagonal cracks on the corners are reflected on the exterior of the brick as well. See attached Photos 13 & 14. There are also many minor horizontal cracks at the windows and where there is a horizontal steel lintel that connects to masonry pilasters. See attached Photos 15, 16 & 17. Generally there are more cracks on the west side of the building. The majority of the cracks were previously repaired with sealant. There were no cracks visible in the interior or exterior of the 1958 Filter Building addition.

Building 19 – Filter Building 2005 Addition

- This structure was constructed in 2005. The exterior masonry bearing walls are supported on two foot wide strip footings and concrete foundation walls. The interior masonry walls are non-bearing and are supported on thickened concrete slab. Roof steel joists with metal decking are supported on exterior concrete masonry unit (CMU) walls. A corridor was constructed connecting the old and new structure. It is typically supported on strip footings, however, adjacent to the existing building there is an underground chamber and the wall bear directly on the concrete top of the chamber.
- Cracks are visible in the concrete foundation walls on each side where the underground chamber stops and the strip footings begin. See attached Photo 19. Cracks from the inside of the hallway are reflected to the exterior, and a vertical shift of 1/8" can be seen in concrete footing wall. See attached Photo 20. This has also caused cracks to develop below the lintel beams on both sides of the windows. See attached Photos 18, 23 & 25. The corridor has a control joint with a sealant in the masonry wall on each side. There are cracks in the paint at this joint which are only superficial, as the sealant below is in good condition. See attached Photo 21.
- In the boiler room area, some sealant is separated at the corner CMU joints, and evidence of differential vertical movement is visible between the exterior and interior wall. The exterior masonry wall is supported on a strip footing, but the interior masonry wall is supported on a thickened slab on grade. See attached Photo 22.
- In the electrical room there are cracks in the corner joints (similar to the boiler room) and no sealant is present to cover the mortar.
- The generator room has a shrinkage crack in the floor. See attached Photo 24. The corner joints have no sealant and the mortar is cracked due to differential settlement between exterior and interior wall as noted in the previous rooms.

Building 21 – Administration Building

- The building has steel beams encased in concrete and steel columns. The floor beams and roof trusses bear on brick masonry walls.
- There are numerous cracks on the south side of the building and at the southeast corner. Many of the cracks were previously repaired. Typically, there were more defects visible near the bottom of the wall than at the top. See attached Photos 26, 27, 28, 29 & 30.

Building 23 – Drain Water Pump Station

- This building was built in the 1990s. The exterior walls are CMU with a brick exterior.
- The exterior brick is in good condition. See attached Photo 31. There is movement at the corner of the building and the retaining wall. The expansion joint between the building and retaining wall has opened up in the corner. The retaining wall is shifting, causing a crack to develop in the concrete portion of the pumping station. This is most likely a result of a dowel that is placed between the retaining wall and the pump station. See attached Photos 32 & 33. There is some interior paint that is peeling away from the concrete walls, which does not appear to be structural-related. See attached Photo 34.

Building 25 – Backwash Pump Station

- This building has a brick exterior and was built in the 1990s. The roof is supported by steel beams and girders which rest on concrete walls.
- There are a few minor interior cracks visible in the concrete walls, which appear to be due to shrinkage and are not signs of structural deficiency. See attached Photo 35. In the lower level of the building, water is leaking in through an existing conduit communication cable into a conduit box. See attached Photo 36.

Building Gate House

- This building has a brick interior and a stone exterior.
- A repointing project was completed in 1995. In general, it appears that the overall condition of the building exterior is not significantly changed from the conditions that were documented in the Field Investigation Summary Report dated October 2014 and prepared by Gannett Fleming. See attached Photos 37 & 38.

Dam Roadway Slab

- This is a concrete slab on grade, which is unreinforced except at the emergency spillway crest locations.
- A large crack typically runs down approximately the center of the slab with other cracks present in many locations. See attached Photos 39 & 40.

Principal Spillway Top of West Training Wall / Inspection Manhole

- Access to the interior of the inspection manhole was not available to inspect for cracking or to observe the horizontal control joints in the concrete that extend from the face of the principal spillway west training wall near the crest to the interior of the inspection manhole. See attached Photos 41 & 42, which are file photos from a 2014 field inspection.

Summary Assessment of Observed Conditions

The existing buildings were constructed in the 1930s and 1990s, and the latest Building 19 Filter Building addition was completed in 2005. The newer buildings have masonry control joints in the exterior walls that help with crack development due to temperature changes. See Exhibit 2 for the Brick Institute of America Technical Notes 18 and 18A on Brick Construction, which provide additional information on masonry control joints.

The Building 19 Filter Building addition has developed a few cracks. These cracks appear to be caused by a differential settlement between new strip footings and the portion of the building supported on top of the existing underground chamber (corridor area). Differential settlement was observed between the exterior walls supported on a strip footing and the interior walls supported on a thickened slab-on-grade. These cracks are aesthetic in nature, and further settlement is expected to be minimal. The current observations support the conclusions contained in ms consultants, Inc.'s report dated June 2012 that the primary cause of the cracks in Building 19 Filter Building addition is settlement, which may have been potentially aggravated by the earthquake activity reported in 2011. A copy of the ms consultant's report is shown in Exhibit 3.

The existing buildings from the 1930s do not have any control or expansion joints in the exterior masonry walls. This is typical for this construction period. In general, it is likely that over many cycles of temperature changes, these older brick buildings have developed cracks in the exterior brick that could have been prevented with the addition of masonry control joints during the original construction. It did not appear that any of the observed cracks in the buildings from the 1930s were due to settlement or seismic activity.

At Building 23 Drain Water Pump Station, corrosion of the dowel bars that are currently holding the cracked and rotated concrete corner of the building in place may fail and result in the collapse of the concrete at some time in the future.

The cracks in the top of dam roadway may have developed due to heavy equipment traveling over the slab with wheel alignment near the edges. Beyond the limits of the twin emergency spillways, the as-built drawings show that the slab at the top of dam is unreinforced concrete, and this detail does not allow for transfer of moment at the center.

Recommendations

1. All buildings:
    a. Perform annual walk-through inspection of all building interiors and exteriors. Identify and document new cracking, other defects and the locations where previously applied sealant has failed and not preventing water intrusion.
    b. Seal open, exterior cracks, where present, to prevent water intrusion. Perform advance routing of cracks and surface preparation as recommended by sealant manufacturer, including removal of existing sealant, where present. Narrow cracks should be routed using a 1/8-inch bit to prepare a slot for sealant installation.
2. Building 7 - Headhouse: Perform a detailed, close-up inspection of the cracking in the exterior, upper level (tower) brick. Assess the cause of the cracking and develop and implement the necessary repairs. A temporary means of temporary access such as a man lift or other suitable equipment will be required to conduct the inspection.
3. Building 23 – Drain Water Pump Station, at the corner where the expansion joint between the building and retaining wall has opened (see attached Photos 32 & 33).
    a. Temporary, short-term repair:
        i. Restrict pedestrian access to ground area in the vicinity of the corner until long-term repairs can be implemented.
        ii. Install new sealant at cracks and open joints that are subject to water intrusion. Perform advance surface preparation as recommended by sealant manufacturer, including removal of existing sealant, where present.
    b. Long-term repair: Investigate as-built construction details and develop repair details to remove and re-build the concrete that is rotated and / or cracked. Repairs should include cutting of the existing dowel bars located between the retaining wall and the pump station corner.
4. Building 21 – Administration Building: Remove mortar from joints along several of the horizontal cracks that have developed between windows and doors and investigate the cause of the cracking. Develop and implement appropriate repairs.
5. Building 25 – Backwash Pump Station, lower level: Investigate the source of the water leaking from the existing conduit into the conduit panel box. Develop and implement repairs needed to stop the leakage into the box, and which will not adversely affect the safety and functionality of the existing conductor.
6. Top of Dam Roadway: Replacement of the top of dam roadway is included as a feature in the upcoming design and construction of dam modifications project.
7. Principal Spillway Top of West Training Wall / Inspection Manhole: The treatment of the horizontal control joints in the west training wall / inspection manhole will be investigated and addressed as needed in the upcoming design and construction of dam modifications project.

Additional Information

For informational purposes, the following United States Geological Survey links provide current information related to earthquake notification service and real time feeds and notifications: https://sslearthquake.usgs.gov/ens/ and http://earthquake.usgs.gov/earthquakes/feed/.

If you have any questions, please feel free to contact me at (717) 763-7212, extension 2122 or vcecka@gfnet.com or Tim Johnston at extension 2398 or tjohnston@gfnet.com.

Sincerely,
GANNETT FLEMING ENGINEERS AND ARCHITECTS, PC.

Vladimir Cecka, P.E.
Project Structural Engineer

Attachments as Noted
Exhibits as Noted

Building 4: Photo 1

Building 4: Photo 2

-

Building 7: Photo 3

Building 7: Photo 4

Building 7: Photo 5

Building 7: Photo 6

Building 7: Photo 7

Building 11: Photo 8

Building 15: Photo 9

Building 15: Photo 10

Building 16: Photo 11

Building 18: Photo 12

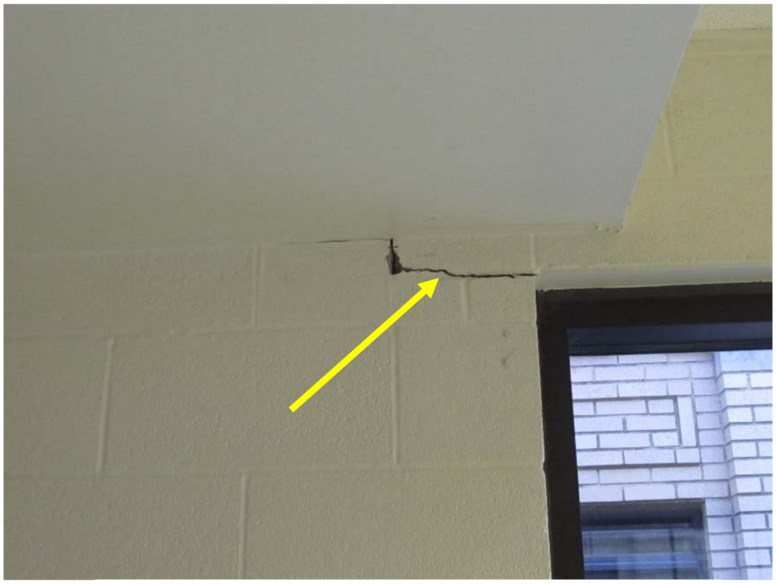
Building 19: Photo 13

Building 19: Photo 14

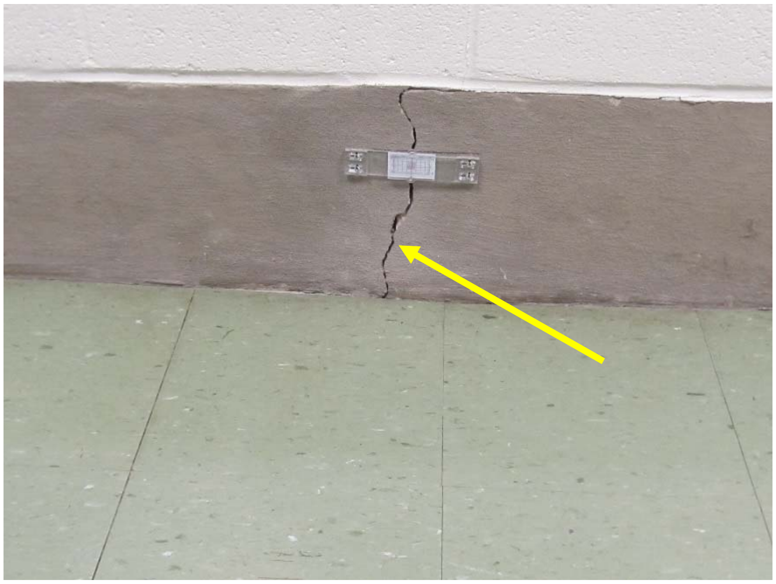
Building 19: Photo 15

Building 19: Photo 16

Building 19: Photo 17



Building 19 2005 Addition: Photo 18



Building 19 2005 Addition: Photo 19

Building 19 2005 Addition: Photo 20

Building 19 2005 Addition: Photo 21

Building 19 2005 Addition: Photo 22

Building 19 2005 Addition: Photo 23

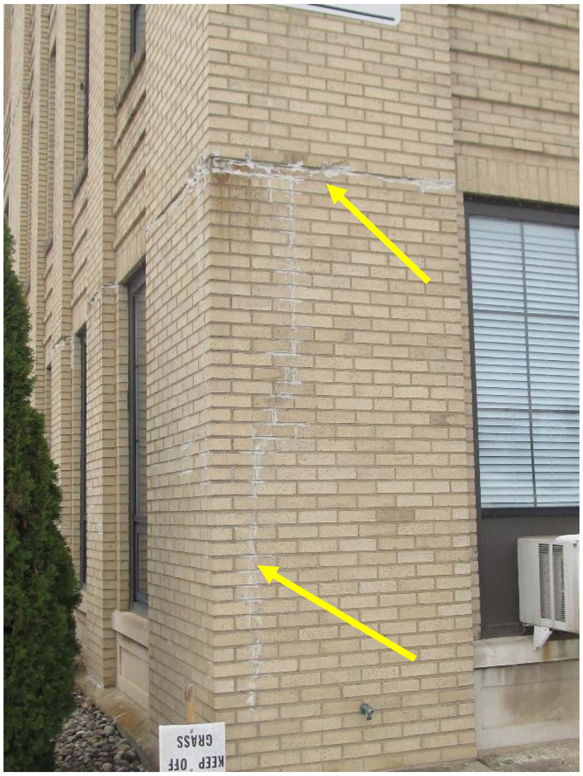
Building 19 2005 Addition: Photo 24

Building 19 2005 Addition: Photo 25

Building 21: Photo 26

Building 21: Photo 27

Building 21: Photo 28



Building 21: Photo 29

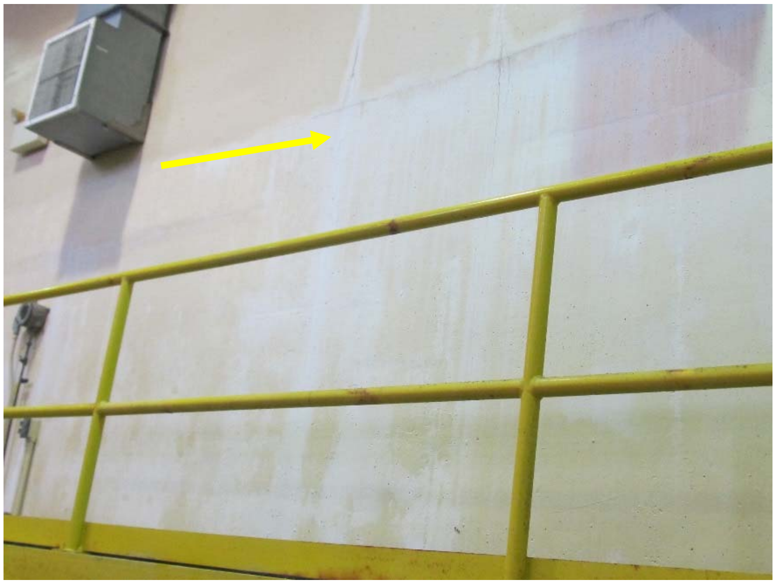
Building 21: Photo 30

Building 23: Photo 31

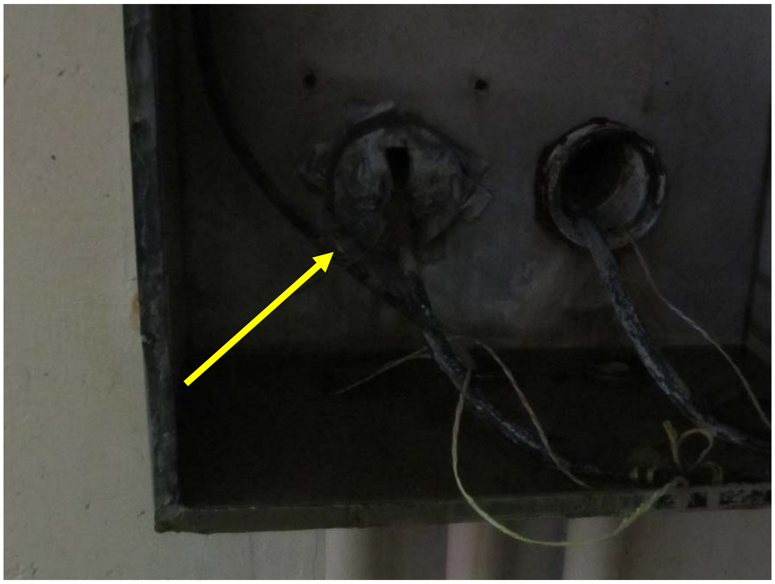
Building 23: Photo 32

Building 23: Photo 33

Building 23: Photo 34



Building 25: Photo 35



Building 25: Photo 36

Building Gate House: Photo 37

Building Gate House: Photo 38

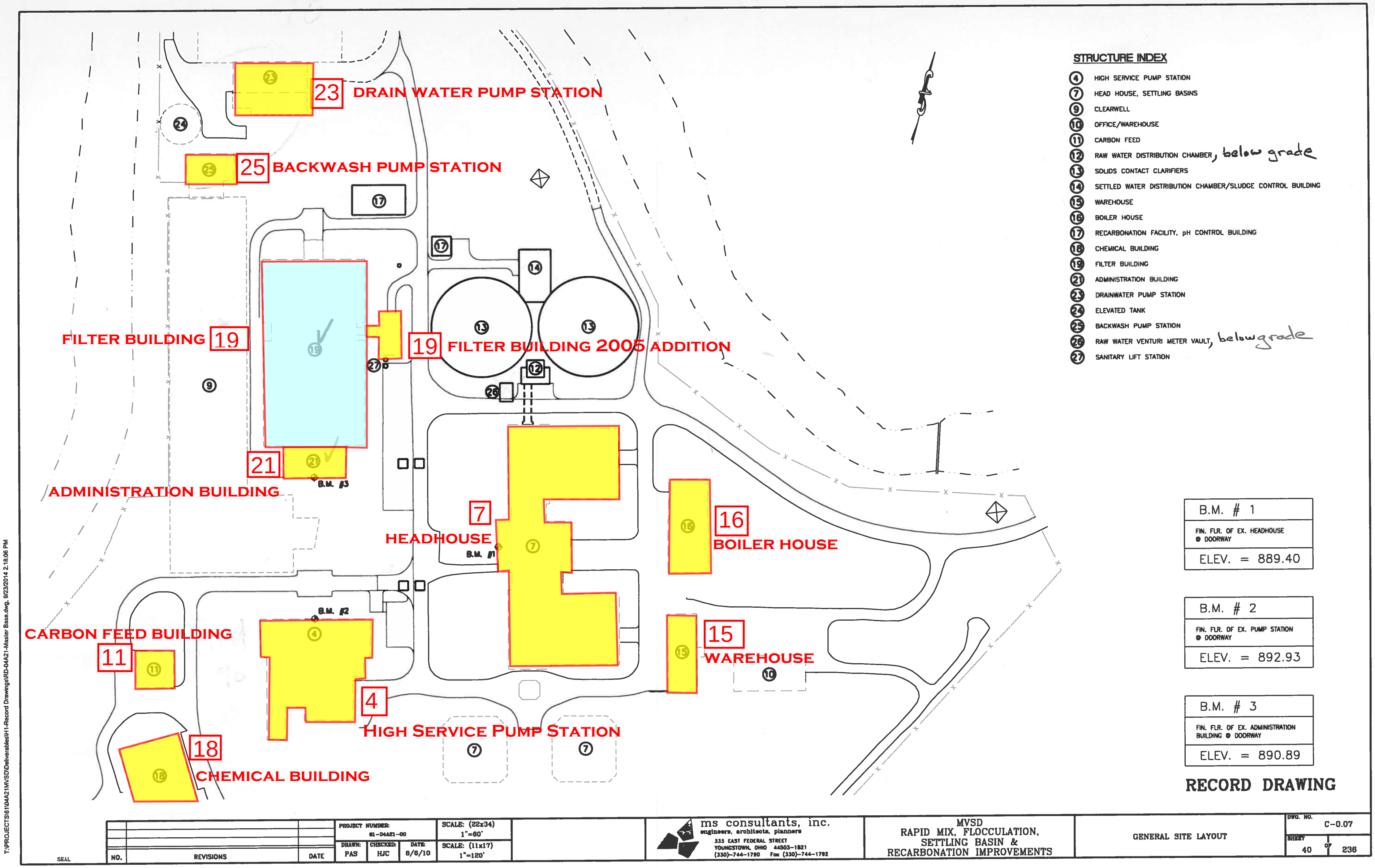
Dam Roadway Slab: Photo 39

Dam Roadway Slab: Photo 40

Dam Spillway West Training Wall: Photo 41

Dam Inspection Manhole: Photo 42

Exhibit 1



WEBEX TELCONFERENCE -MAHONING VALLEY SANITARY DISTRICT

JANUARY 19, 2017

OPINION ON STRUCTURAL ISSUES ASSOCIATED WITH THE DAM AND BUILDINGS

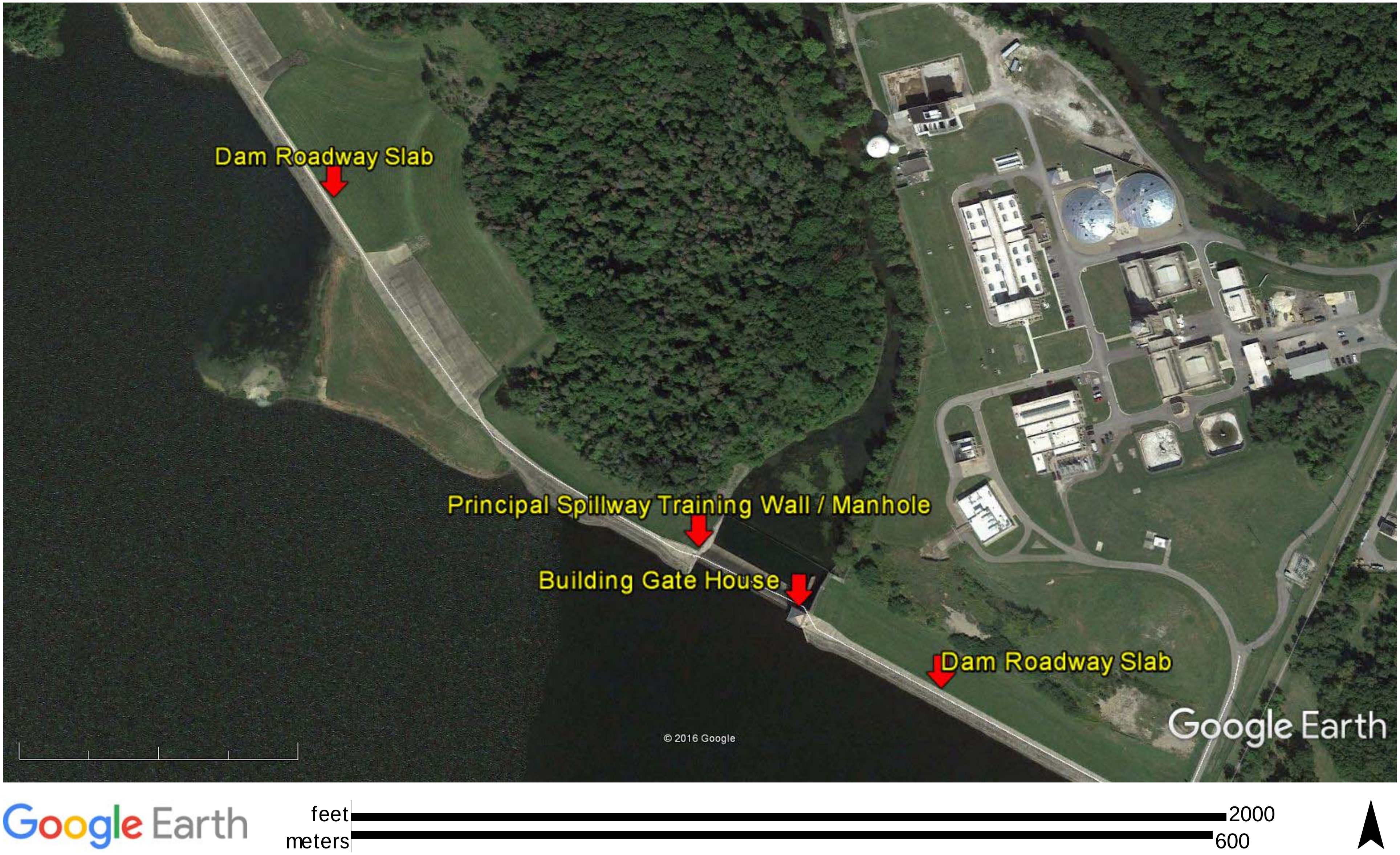

Dam Roadway Slab
Principal Spillway Training Wall / Manhole
Building Gate House
Dam Roadway Slab
© 2016 Google
Google Earth
Google Earth
feet
2000
meters
600

Exhibit 2

BIA Technical Notes on Brick Construction

Brick Institute of America 11490 Commerce Park Drive, Reston, Virginia 22091

18
REVISED

January
1991

MOVEMENT
VOLUME CHANGES AND EFFECT OF MOVEMENT
PART I

Abstract: This *Technical Notes* describes the various movements that occur within buildings. Movement induced by changes in temperature, moisture, elastic deformations, creep, and other factors develop stresses if the brickwork is restrained. Restraint of these movements may result in cracking of the masonry. Typical crack patterns are shown and their causes identified.

Key Words: brick, corrosion, cracks, differential movement, expansion

INTRODUCTION

The various materials and elements that are used to construct a building are in a constant state of motion. All building materials change in volume due to internal or external stimuli. These stimuli may be changes in temperature, moisture, elastic deformations due to loads, creep, or other factors. Restraint of these movements may cause stresses within the building elements which in turn may result in cracks.

To avoid cracks, the design should minimize volume change, prevent movement or accommodate differential movement between materials and assemblies. A system of movement joints can eliminate cracks and the problems they cause. Movement joints can be designed by estimating the magnitude of the several types of movements which may occur in masonry and other building materials.

This *Technical Notes* describes the various volume changes in brick masonry and other building materials. It also describes the effects of volume change when the materials are restrained. Other *Technical Notes* in this series address the design and detailing of movement joints and the types of anchorage which permit movement.

MOVEMENTS OF CONSTRUCTION MATERIALS

The design and construction of most buildings does not allow precise prediction of movements of building elements. Volume changes are dependent on material properties and are highly variable. Age of material and temperature at installation also influence expected movement. When mean values of material properties are used in design, the actual movement may be underestimated or overestimated. The designer should use discretion when selecting the applicable values. The types of movement experienced by various building materials are indicated in Table 1.

Temperature Movements

All building materials expand and contract with variations in temperature. For unrestrained conditions, these movements are theoretically reversible. Table 2 indicates the coefficients of thermal expansion for various building materials.

Unrestrained thermal movement is the product of temperature change, the coefficient of thermal expansion, and the length of the element. The stresses developed by restrained thermal movements are equal to the change in temperature multiplied by the coefficient of thermal expansion and the modulus of elasticity of the material. The temperature change used for estimating thermal movements should be based on mean wall temperatures. For solid walls, temperatures at the center of the wall should be used. In cavity walls and veneers, the temperature at the center of each wythe or component should be used. In discontinuous construction, the wythes will have different temperatures due to the separation of the wythes by an air space.

Surface temperatures of brick walls may be much higher than the ambient air temperature. Wall orientation, wall type and color are governing factors. It is possible for a dark, south facing wall to reach surface temperatures as high as 140°F (60°C), while the ambient air temperature is well below 100°F (37.7°C). The mean wall temperature of a 4 in. (100 mm) thick insulated brick veneer wall is very close to the surface temperature of the brick. A thicker or non-

TABLE 1

Types of Movement of Building Materials

Building Material	Thermal	Reversible Moisture	Irreversible Moisture	Elastic Deformation	Creep
Brick Masonry	X	—	X	X	X
Concrete Masonry	X	X	—	X	X
Concrete	X	X	—	X	X
Steel	X	—	—	X	—
Wood	X	X	—	X	X

TABLE 2

Thermal Expansion

Material	Average Coefficient of Lineal Thermal Expansion, × 10^{-6}/°F
Clay Masonry	
Clay or shale brick	3.6
Fire clay brick or tile	2.5
Clay or shale tile	3.3
Concrete Masonry	
Dense aggregate	5.2
Lightweight aggregate	4.3
Stone	
Granite	4.7
Limestone	4.4
Marble	7.3
Concrete	
Gravel aggregate	6.0
Lightweight, structural	4.5
Metal	
Aluminum	12.8
Bronze	10.1
Stainless steel	9.9
Structural steel	6.5
Wood, Parallel to Fiber	
Fir	2.1
Oak	2.7
Pine	3.0
Wood, Perpendicular to Fiber	
Fir	32.0
Oak	30.0
Pine	19.0
Plaster	
Gypsum aggregate	7.6
Perlite aggregate	5.2
Vermiculite aggregate	5.9

insulated wall may experience a smaller temperature difference between the outside and inside surfaces.

Other materials such as metals or wood will expand and contract at rates different from that of brick masonry. These differences are important in applications such as window frames, railings, or copings which are attached to brick masonry. Distress may occur in either material.

Moisture Movements

With the notable exception of metals, many building materials tend to expand with an increase in moisture content and contract with a loss of water. For some building materials these movements are reversible; while for others they are irreversible or only partially reversible.

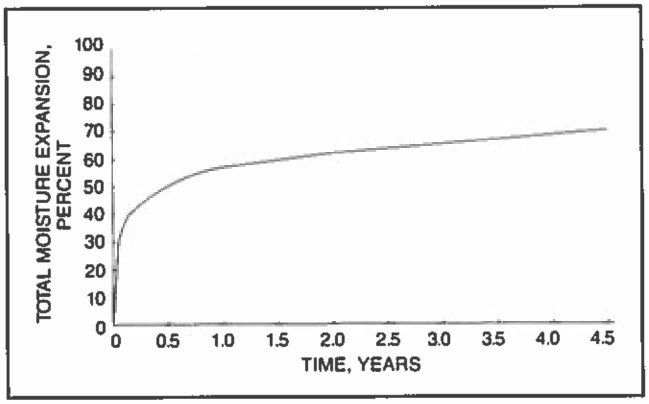
Clay Products. Brick units expand slowly over time upon exposure to water or humid air. This expansion is not reversible by drying at atmospheric temperatures. A brick unit is smallest in size when it cools after coming from the kiln. The unit will increase in size due to moisture expansion from that time. Most of the expansion takes place quickly over the first few weeks, but expansion will continue at a much lower rate for several years (see Figure 1). The moisture expansion behavior of brick depends primarily on the raw materials and secondarily on the firing temperatures. Brick made from the same raw materials that are fired at lower temperatures will expand more than those fired at higher temperatures.

Moisture expansion of individual brick or brick masonry can be measured for a given length of time. Predicting the total moisture expansion of brick is much more difficult. At present there are no standard tests to predict moisture expansion or measure moisture expansion which occurs in service. Based on past research, long term moisture expansion of brick can be estimated at between 0.0002 and 0.0009. A design value of 0.0003 should be used when designing composite masonry walls. A design value of 0.0005 should be used in veneer walls where an upper bound of movement is estimated.

Concrete Masonry. Concrete masonry units experience shrinkage as a result of moisture loss and carbonation. Shrinkage of concrete masonry is affected by method of curing, aggregate type, change in moisture content, cement content, and wetting and drying cycles. Total shrinkage is determined by ASTM C 425 Test Method for Drying Shrinkage of Concrete Block which measures shrinkage from a saturated condition to a 17% moisture condition. Typical total linear shrinkage values range between 0.0002 and 0.0007. Type I concrete masonry units must conform to moisture content requirements found in the material specifications which limits wall shrinkage.

Concrete. Concrete shrinks as it cures and swells as it becomes wet. Shrinkage of concrete is influenced by the water cement ratio, composition of the cement, type of aggregate, size of concrete member, curing conditions, and amount and distribution of reinforcing steel. Values of final shrinkage for ordinary concretes are generally of the order of 0.0002 to 0.0007 depending on the factors listed.

Wood. Wood will shrink during the natural seasoning process as the moisture content drops from the fiber saturation point (28 to 30%) until it reaches equilibrium moisture content with local atmospheric conditions. Shrinkage occurs differently in the tangential, radial and longitudinal dimensions of the member. Table 3 indicates the range of shrinkage values for commonly used woods. Moisture expansion and contraction continues with changes in moisture content.



Projected Moisture Expansion of Fired Brick vs. Time

FIG. 1

TABLE 3

Shrinkage of Wood[1,2]

Species	Radial Shrinkage, %	Tangential Shrinkage, %	Volumetric Shrinkage, %
Douglas Fir	3.8–4.8	6.9– 7.6	10.7–12.4
Red Oak	4.0–5.0	8.6–11.3	13.7–19.0
Southern Pine	4.6–5.4	7.4– 7.7	12.1–12.3
Spruce	3.8–4.3	6.8– 7.8	11.0–11.8

[1] Adapted from Reference 10

[2] Dried from 30% moisture content to 0%

Elastic Deformation

In the structural design of a building, the designer must consider all forces imposed on the structure. These include dead loads, live loads, snow loads, and such external lateral forces as wind, soil, earthquake and blast. All of these forces create stresses in the building materials resulting in deflections of the building elements.

All materials, when subjected to a force, respond to stress with an associated strain. The stress-strain relationship for masonry materials is approximately linear and is defined by the modulus of elasticity. Axial deformation is determined by dividing the stress by the modulus of elasticity and multiplying the quotient by the length under load. The deflections of horizontal elements, lateral deflections of walls and columns, and reductions in lengths of axially loaded structural elements due to design loads must be considered.

Creep

Creep, or plastic flow, is the continuing deformation of materials under load or stress. The magnitude of movement due to creep in masonry and concrete depends on the stress level, material age, duration of stress, material quality, and environmental factors.

Brick. Creep in brick masonry primarily occurs in the mortar joints and is negligible. The ACI 530/ASCE 5 "Building Code Requirements for Masonry Structures" suggests 0.7×10^{-7} in./in. per psi of load.

Concrete Masonry. Concrete masonry exhibits more creep than brick masonry because of the cement content in the units. The ACI 530/ASCE 5 Code suggests a value of 2.5×10^{-7} in./in. per psi of load.

Concrete. Creep is most significant in concrete frame structures. Creep in concrete begins after load is applied and proceeds at a decreasing rate. High-strength concretes show less creep than low-strength concretes. Creep is slightly greater in lightweight aggregate concretes than normal-weight concretes. In high-rise buildings, the total elastic and inelastic shortening of columns and walls due to gravity loads and shrinkage may be as high as 1 in. (25.4 mm) for every 80 ft (24.4 m) of height.

Corrosion of Steel

Corrosion of steel embedded in masonry can cause cracking or spalling of masonry. The volume of rust is greater than that of the steel from which it is formed. This volume increase causes pressure on the surrounding masonry. Metals embedded in grout, such as reinforcing bars, are less susceptible to corrosion than ties and joint reinforcement embedded in mortar joints since they are protected by the grout and not exposed. Other items in masonry susceptible to corrosion are steel lintels, steel shelf angles, joint reinforcement, anchor bolts and other metal fasteners in masonry. To minimize corrosion, do not use additives in mortar, such as calcium chloride, which would accelerate corrosion. See *Technical Notes* 44B for more on corrosion resistance of metal wall ties.

Other Causes of Movement

There are other causes of movement in building elements which may occur under given conditions. These include freezing expansion, carbonation of concrete and mortars, drift of the building frame, deflection of building elements, and the action of unstable soils. It is beyond the scope of this *Technical Notes* to discuss these items in detail. However, the designer should recognize and consider these factors.

Masonry materials exhibit expansion due to freezing when saturated. Freezing expansion has a small effect on total expansion of masonry. Based on limited data, the freezing expansion for brick ranges from 0 to 10.3×10^{-4} in./in. A design value for brick masonry of 2×10^{-4} in./in. is recommended. The expansion occurs when saturated brick are subjected to temperatures at or below 14°F (−10°C).

Carbonation is the chemical combination of hydrated portland cement with carbon dioxide present in air. Although it is known that materials containing portland cement shrink upon carbonation, little is known about the extent of the carbonation or the resulting shrinkage.

The drift or side-sway of a structural frame may cause distress to brick masonry used as in-fill walls or exterior cladding. Wind or earthquake loads will be transferred to the more rigid brickwork if attached rigidly to the frame. The same is true for deflection of floor slabs or spandrel beams. Masonry built up in contact with these elements will be loaded due to the deflection of the member. Masonry intended to be non-loadbearing may become loadbearing.

Foundation movements and differential settlement often cause cracking in masonry walls supported on foundations. Unstable soils or expansive soils are of special concern. Proper foundation design should be performed to ensure a stable support or allow uniform settlement.

EFFECTS OF MOVEMENT

Changes in building design have affected the design and behavior of many building components, including masonry walls. The most significant change for brick masonry is the shift from loadbearing masonry walls to skeleton frame construction. Other factors include the use of thinner walls, composite walls and insulated walls. The increased use of portland cement mortars and the tendency to specify high compressive strength mortars have become common. Although stronger units and mortars increase the compressive strength of the masonry, they do so at the expense of other important properties. Thus, masonry walls are thinner and more brittle than their massive ancestors. These thinner walls are more susceptible to cracking and spalling if provisions for differential movement are not accounted for properly.

Cracking and Spalling

Cracking is probably the distress which occurs most often in masonry walls. Cracks result from many different sources, but there are typical shapes and patterns of cracks. Often the type and magnitude of cracking will indicate the cause.

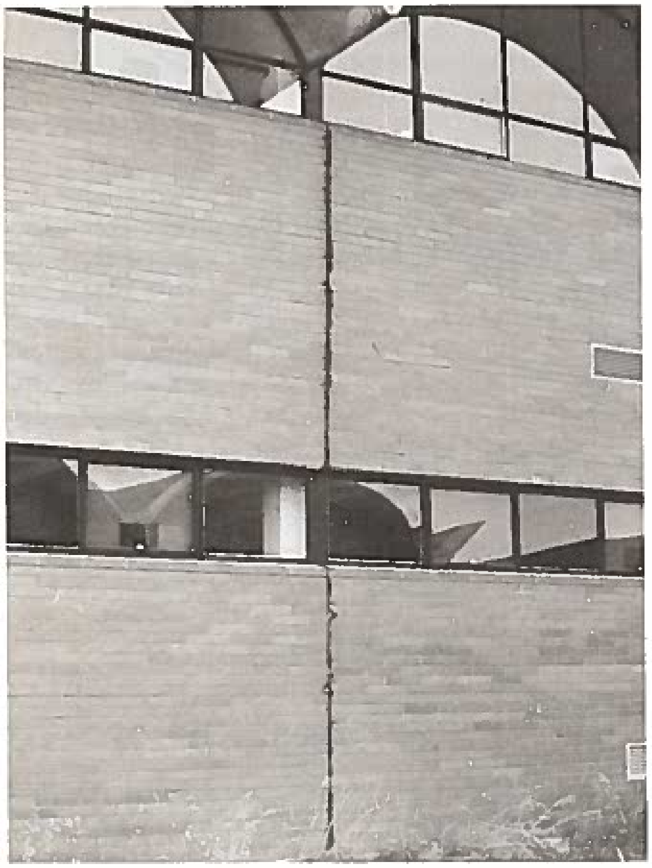
It is more beneficial to show what can happen if movement is not considered in design than to show a properly designed and detailed project. Following are some typical locations where cracks occur in masonry walls and the major cause of each. *Technical Notes* 18A will describe ways to avoid these problems.

Long Walls. Long walls or walls with large distances between expansion joints may cause distress within the wall. The expansion of the brickwork may force sealant material out of the expansion joint or crack the brickwork between expansion joints (see Fig. 2). Diagonal cracks often occur in piers between window or door openings. Such cracks usually extend from the head or sill at the jamb of the opening, depending upon the direction of movement and the path of least resistance.

Corners. An insufficient amount or improper location of expansion joints in walls can lead to cracking at the corners. Perpendicular walls will expand in the direction of the corner causing rotation and cracking near the corner. This typically occurs at the first head joint from either side of the corner (see Fig. 3).

Offsets and Setbacks. Vertical cracks are quite common at wall setbacks or offsets if movement is not accommodated. When parallel walls expand towards the offset, the movement produces rotation of the offset causing vertical cracks (see Figs. 4 and 5).

Shortening of Structural Frames. In frame structures, predominately concrete frame buildings, vertical shortening due to creep or shrinkage of the structural frame may impose high stresses on the masonry. These stresses may develop at window heads, shelf angles, and other points where stresses are concentrated. Fig. 6 shows brick veneer supported by a steel shelf angle on a concrete frame. Over time the concrete frame has shrunk and caused the steel shelf angle to bear on the masonry below. Because a horizontal expansion joint was not provided, stresses became concentrated on the mortar joint directly below the angle causing crushing of the masonry below. This phenomenon can also cause bowing of brickwork between floors, if the brickwork is not adequately attached to the backing, or the backing is not sufficiently rigid.



Expansion of Long Wall

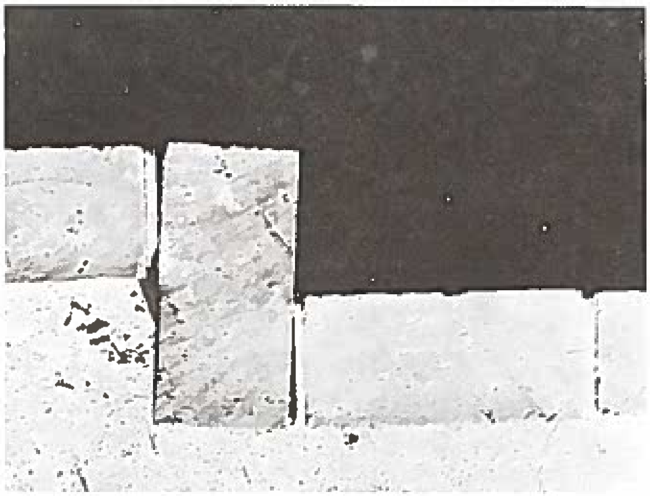
FIG. 2

Crack at Corner

FIG. 3

Crack at Offset

FIG. 4



Rotation at Offset

FIG. 5

Spalling Due to Shortening of Structural Frame

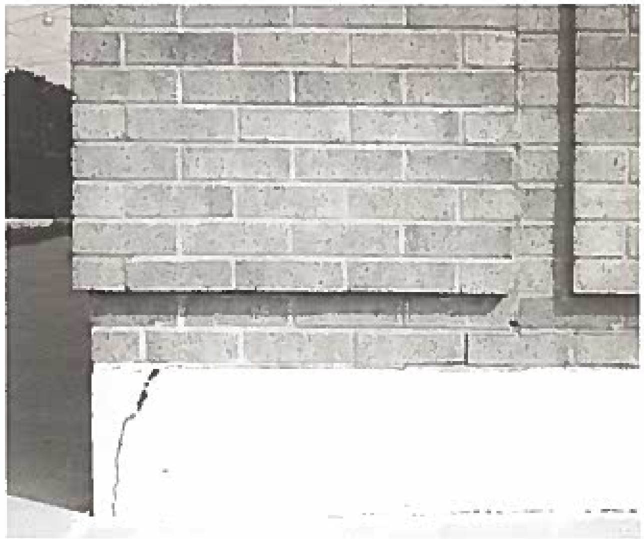
FIG. 6

Parapet Walls. Parapets exposed on three sides are subjected to extremes of moisture and temperature which may be substantially different from those in the wall below. Also, parapets lack the dead load of masonry above to help resist movement. Expansion can cause parapets to bow if restrained at both corners or move away from corners if restrained only at one end (see Fig. 7).

Foundations. Masonry walls above grade built on concrete foundations will expand while the concrete foundation will shrink. This differential movement will cause shear at the foundation interface if bonded together. Movement of the brick away from the corner or cracking of the concrete often results (see Fig. 8).

Bowing of Parapet Due to Expansion

FIG. 7



Crack at Foundation Corner

FIG. 8

Deflection and Settlement. Deflection and settlement cracks are identified by a tapering shape. Fig. 9 shows a deflection crack due to insufficient support of the brickwork on a lintel. The crack is wider at the steel angle and tapers to nothing. *Technical Notes* 31B Structural Steel Lintels details the proper design of steel lintels supporting masonry. Deflection cracks may also occur at steel shelf angles attached to spandrel beams that deflect.

Crack Due to Deflection

FIG. 9

Crack Due to Differential Settlement

FIG. 10

Fig. 10 shows a crack due to differential settlement of the foundation. If all settlement is equal, then little harm is done. Cracking occurs when one portion of a structure settles more than an adjacent part.

Encased Columns. Where structural elements are rigidly encased in masonry, any movement of the column is transferred to the masonry, causing cracks. These movements may be due to drift of the building frame or lateral expansion from creep. These cracks occur on the exterior as well as the interior of the building (see Fig. 11).

Curling of Concrete. If a concrete slab is cool and dry on top and warm and moist on the bottom, the top may become shorter than the bottom causing the slab to curl upward. Cast-in-place concrete slabs also curl up at the corners due to deflection when the forms are removed and loads applied. This curling can lift masonry attached to or laid on the concrete slab (see Fig. 12).

Embedded Items. Items embedded in or attached to masonry may cause spalling or cracking when they move or expand. Joint reinforcement that is continuous across an expansion joint may buckle, pushing out adjacent mortar (see Fig. 13). Corrosion of metal elements within masonry causes volume increases of such a magnitude as to crack or spall the masonry.

SUMMARY

This *Technical Notes* describes the various movements that occur within all building materials and constructions. It also explains the effects of these movements. Cracking in brickwork can be eliminated if all factors are taken into consideration and the anticipated movement is accommodated.

Encased Column

FIG. 11

Crack Due to Curling of Concrete Slab

FIG. 12

Spalling Due to Buckling of Joint Reinforcement

FIG. 13

The information and suggestions contained in this *Technical Notes* are based on the available data and the experience of the engineering staff of the Brick Institute of America. 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 Notes* are not within the purview of the Brick Institute of America and must rest with the project architect, engineer, owner or all.

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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: 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 door frames
  - 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 C 920, Grade NS, Use M
- Class 50 minimum extensibility recommended; Class 25 alternate
- Consult sealant manufacturer's literature for guidance regarding use of primer and backing materials

Bond Breaks:
- Use building paper or flashing to separate brickwork from dissimilar materials, foundations and slabs

Loadbearing 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 within brickwork. Details of expansion joints are provided for loadbearing and nonloadbearing applications. While most examples are for commercial structures, movement joints, although rare, also must 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 that may be needed include control joints, building expansion joints and construction joints. Each of these is designed to perform a specific task, and they should not be used interchangeably.

An *expansion joint* separates brick masonry into segments to prevent cracking 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 that the structural integrity of the brickwork is not compromised.

A *control joint* determines the location of cracks in concrete or concrete masonry construction due to volume changes resulting from shrinkage. It creates a plane of weakness that, in conjunction with reinforcement or joint reinforcement, causes cracks to occur at a predetermined location. A control joint is usually a vertical gap through the concrete or concrete masonry wythe and may be filled with inelastic materials. A control joint will tend to open rather than close. 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 that stresses developed in one section will not affect the integrity of the entire structure. The building expansion joint is a through-the-building joint and is typically wider than an expansion or control joint.

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, the joint also must resist water penetration and air infiltration. 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.

Mortar, ties or wire reinforcement should not extend into or bridge the expansion joint. If this occurs, movement will be restricted and the expansion joint will not perform as intended. Expansion joints should be formed as the wall is built, as shown in Photo 1. However, vertical expansion joints may be cut into existing brickwork as a remedial action.

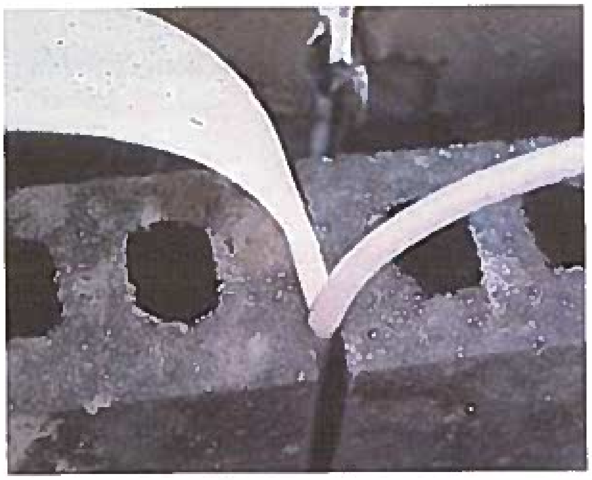


Photo 1
Vertical Expansion Joint Construction

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 should conform to ASTM C 920, *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. Sealant manufacturers should be consulted for the applicability 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. Manufacturers recommend three generic types of elastomeric sealants for use on brickwork: polyurethanes, silicones and polysulfides. Most sealants suitable for use in brickwork expansion joints meet an ASTM C 920 Class 25 or Class 50 rating that requires them to expand and contract by at least 25 percent or 50 percent of the initial joint width, respectively. Sealants meeting Class 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.

Use a circular foam backer rod behind sealants to keep the sealant at a constant depth and provide a surface to tool the sealant against. 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 1/4 in. (6.4 mm).

VERTICAL EXPANSION JOINTS

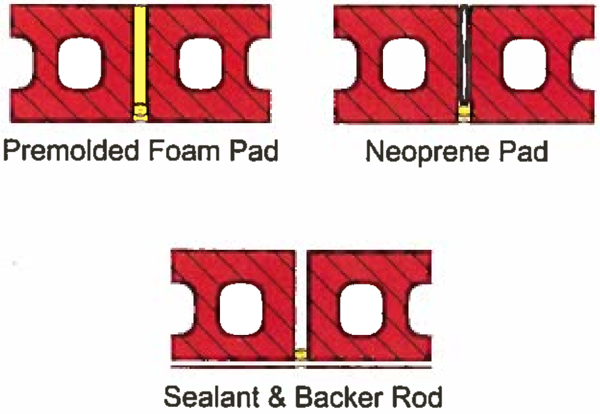


Figure 1

Figure 1 shows typical methods of forming vertical expansion joints with either a premolded foam pad, a neoprene pad or a backer rod.

While generally limited to rain screen walls, a two-stage joint as shown in Figure 2 can increase resistance to water and air infiltration. This type of joint provides a vented or pressure-equalized joint. The space between the sealants must be vented toward the exterior to allow drainage. This is typically achieved by leaving a hole or gap in the exterior sealant joint at the top and bottom of the joint.

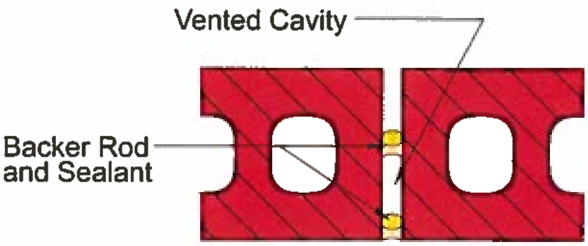


Figure 2
Two-Stage Vertical Expansion Joint

Spacing

No single recommendation on the positioning and spacing of expansion joints can be applicable to all structures. Review each structure for the extent of movements 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 size of the expansion joint and the compressibility of the expansion joint 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 extensibility of expansion joint material

The expansion joint is typically sized to resemble a mortar joint, usually $^3/_8$ in. (10 mm) to $^1/_2$ in. (13 mm). The width of an expansion joint may be limited by the sealant capabilities. Extensibility of sealants in the 25 percent to 50 percent range is typical for brickwork. Compressibility of filler materials may be up to 75 percent.

Example. Consider a typical brick veneer with a desired expansion joint size of $^1/_2$ in. (13 mm) and a sealant with 50 percent extensibility. Eq. 1 gives the following theoretical expansion joint spacing:

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

Therefore, the maximum theoretical spacing between vertical expansion joints in a straight wall would be 23 ft - 2 in. (7.1 m). This 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, but it may be economically desirable to exceed the theoretical maximum spacing as a calculated risk. 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 (7.3 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.

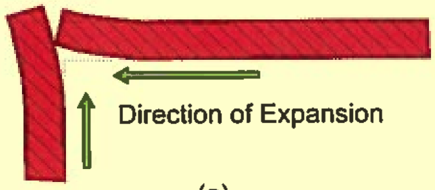
Placement

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 corners, offsets, openings, wall intersections, changes in wall heights and parapets.

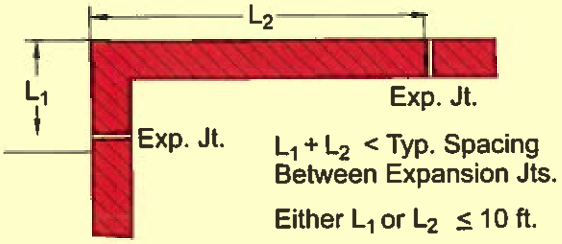
Corners. Walls that intersect will expand toward their juncture, typically causing distress on one or both sides of a corner, as shown in Figure 3a. Place expansion joints near corners to alleviate this stress. The best location is at the first head joint on either side of the corner; however, this may not be aesthetically pleasing. Masons can typically reach about 2 ft (600 mm) around the corner from the face where they are working. An expansion joint should be placed within approximately 10 ft (3 m) of the corner in either wall, but not necessarily both. The sum of the distance from a corner to the adjacent vertical expansion joints should not exceed the spacing of expansion joints in a straight wall, as shown in Figure 3b. 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.

Offsets and Setbacks. Parallel walls will expand toward an offset, rotating the shorter masonry leg, or causing cracks within the offset, as shown in Figure 4a. Place expansion joints at the offset to allow the parallel walls to expand, as Figure 4b illustrates. Expansion joints placed at inside corners are less visible.

Openings. When the spacing between expansion joints is too large, cracks may develop at window and door openings. In structures containing punched windows and door openings, 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 may cause cracks that emanate from the corners of the opening, as in Figure 5. This pattern of cracking does not exist in structures with continuous ribbon windows.

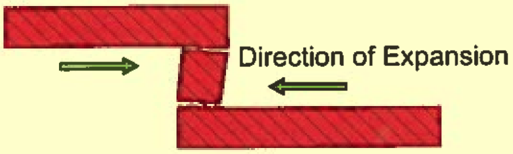


(a)
Movement at Corner Without Expansion Joints

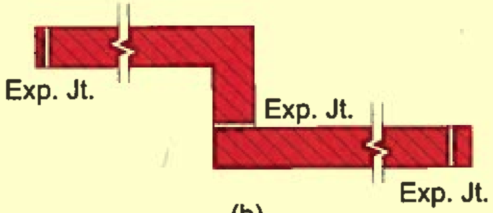


(b)
Proper Expansion Joint Locations at Corner

Figure 3
Vertical Expansion Joints at Corners



(a)
Movement at Offset Without Expansion Joints



(b)
Proper Expansion Joint Locations at Offset

Figure 4
Vertical Expansion Joints at Offsets

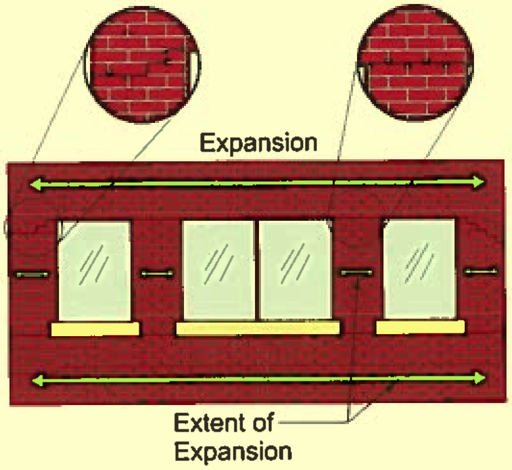


Figure 5
Cracking in Structure with "Punched" Windows, Without Proper Expansion Joints

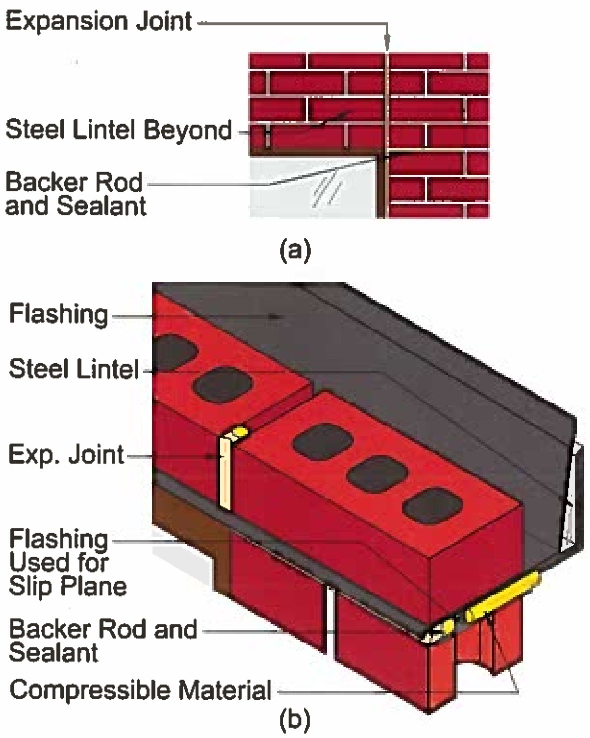
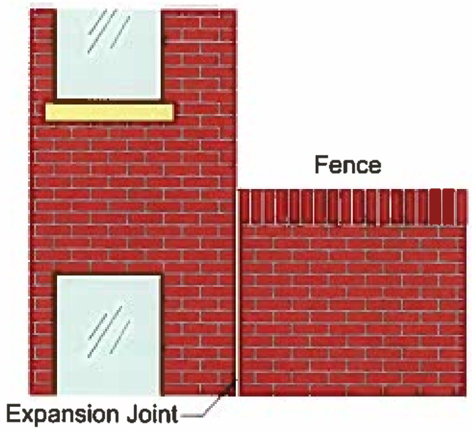
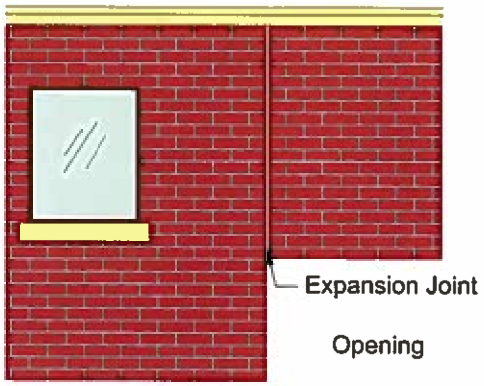


Figure 6
Expansion Joint at Loose Lintel



Different Environmental Exposure
(a)



Different Support Conditions
(b)

Figure 7
Expansion Joints at Junctions

Window and door openings weaken the wall and act as "natural" expansion joints. One alternative is to place expansion joints halfway between the windows. This requires a sufficiently wide section of masonry between the openings, typically 4 ft (1.2 m). It is often desirable to locate vertical expansion joints along the edge or jamb of the opening. In cases where the masonry above an opening is supported by shelf angles attached to the structure, a vertical expansion joint can be placed alongside the opening, continuing through the horizontal support.

If a vertical expansion joint runs alongside an opening spanned by a loose lintel as shown in Figure 6a, the loose steel lintel must be allowed to expand independently of the masonry. 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 6b. Because steel expands more than masonry, a $^1/_8$ to $^1/_4$ 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. Locating the expansion joint adjacent to the window will influence the dead weight of the masonry bearing on the lintel. Instead of the usual triangular loading, the full weight of the masonry above the angle should be assumed to bear on the lintel. See *Technical Note* 31B for more information about steel lintel design. If a vertical expansion joint cannot be built in this manner, do not place it alongside the opening.

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 should be separated with expansion joints since each area will move differently. An exterior wall containing brickwork that extends through glazing into a building's interior should have an expansion joint separating the exterior brickwork from the interior brickwork. You may need to use expansion joints 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 7.

Parapets. Parapets with masonry exposed on the back side are exposed on three sides to extremes of moisture and temperature and may experience substantially different movement from that of the wall below. Parapets also lack the dead load of masonry above to help resist movement. Therefore, extend all vertical expansion joints through parapets. Since parapets are subject to more movement than the wall below, they must be treated differently. When vertical expansion joints are spaced more than 15 ft (4.6 m) apart, the placement and design of expansion joints through parapets need to accommodate this additional movement. In this situation, make expansion joints in the parapet wider or add expansion joints placed halfway between those running full height. These additional expansion joints must continue down to a horizontal expansion joint. As a third alternative, install joint reinforcement at 8 in. (203 mm) on center vertically in the parapet.

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 2.

Photo 2
Accentuated Expansion Joint

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. Toothing of expansion joints to follow the masonry bond pattern is not recommended. It is more difficult to keep debris out of the joint during construction; such debris could interfere with movement. Further, most sealants do not perform well when subjected to both shear and tension.

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.

Other Considerations

Location of vertical expansion joints will be influenced by additional factors. Spandrel sections of brickwork supported by a beam or floor may crack because of deflection of the support. Reduced spacing of expansion joints will permit deflection to occur without cracking the brickwork.

Building Code Requirements for Masonry Structures (ACI 530/ASCE 5/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.

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 space for vertical expansion of the brickwork below and deformation of the shelf angle and the structure to which it is attached. Structures that support the brick wythe on shelf angles, usually done for each floor, must have horizontal expansion joints under each shelf angle. Figure 8 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, any temporary shims that support the angle must be removed after the shelf angle is connected. The joint is formed by a clear space or highly compressible material placed beneath the angle, and a backer rod and

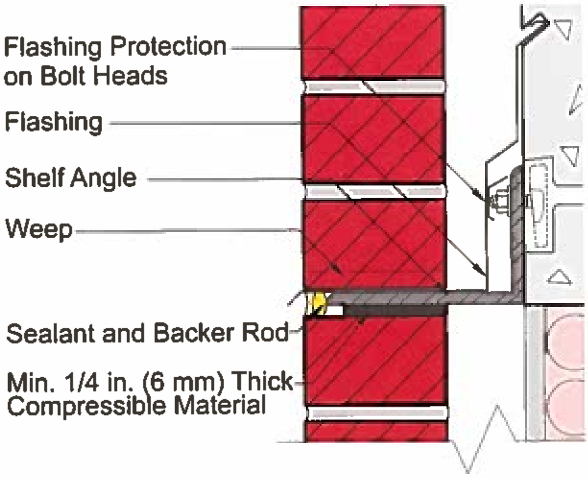


Figure 8
Expansion Joint at Shelf Angle

sealant at the toe of the angle to seal the joint. 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.

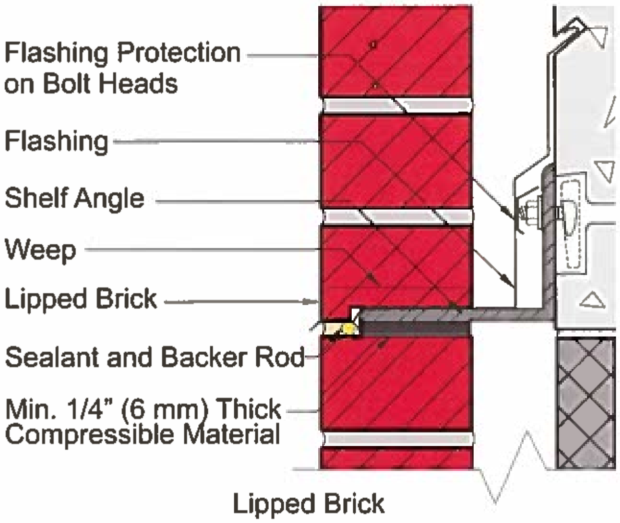
The size of the horizontal expansion joint should take into account movements of the brickwork and movements of the frame. Frame movements include both material and load-induced movements, such as deflections of the shelf angle, rotation of the horizontal leg of the shelf angle, and movement of the support 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). Lipped brick should be made by the brick manufacturer for quality assurance purposes.

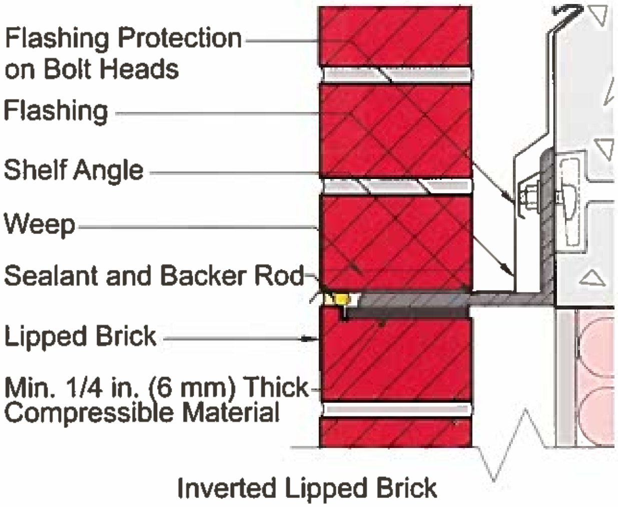
Construction using lipped brick requires careful consideration of the frame movements noted previously. Allowance for adjacent material tolerances including the building frame should also be considered. Adequate space should be provided between the lipped portion of the brick and the shelf angle to ensure 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 9a. Flashing should be placed between the shelf angle and the lipped brick course. Proper installation of flashing is made more difficult because the flashing must conform to the shape of the lip. This shape may be achieved with stiffer flashing materials such as sheet metal. If the specified flashing materials are made of composite, plastic or rubber, 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 movement and moisture entry.

Lipped brick also may be inverted and placed on the last course of brickwork below a shelf angle, as shown in Figure 9b. 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 course to move independent of the shelf angle. Thus, there is an increased possibility of the shelf angle coming in contact with the lipped brick course, resulting in cracking at the lip. It is difficult, if not impossible, to install compressible material below the shelf angle. Further, it is likely that temporary shims may be left in place between the lipped brick and the shelf angle.



Lipped Brick
(a)



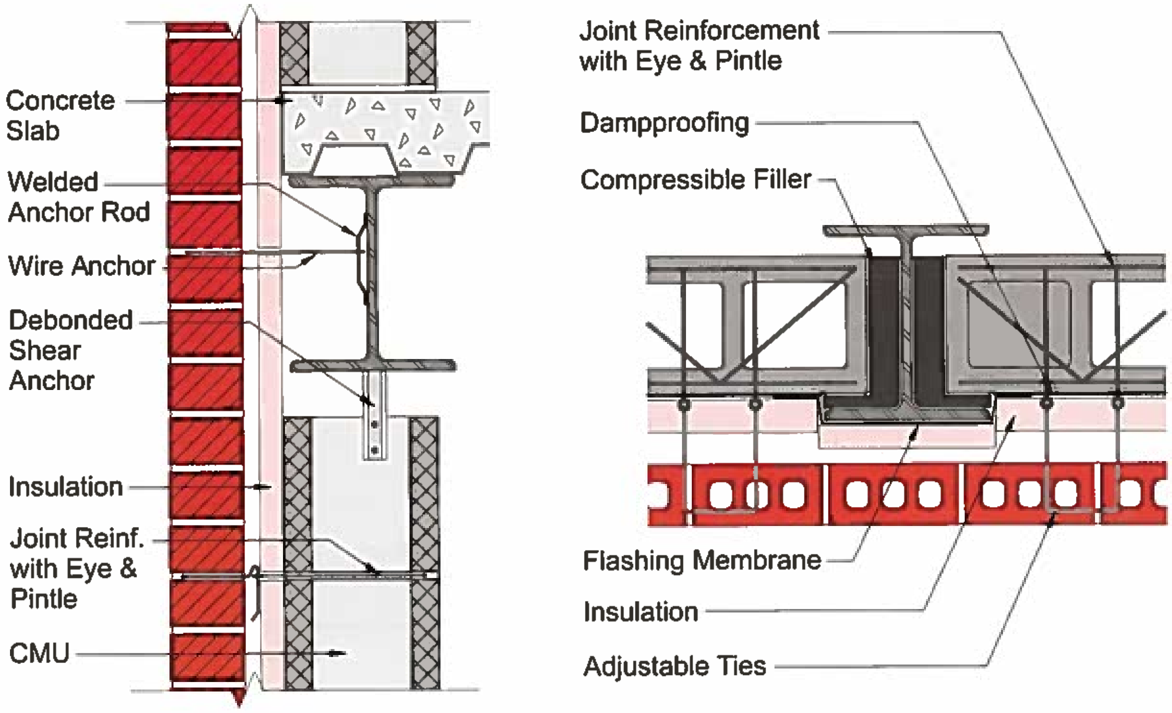
Inverted Lipped Brick
(b)

Figure 9
Alternate Expansion Joint Detail

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.

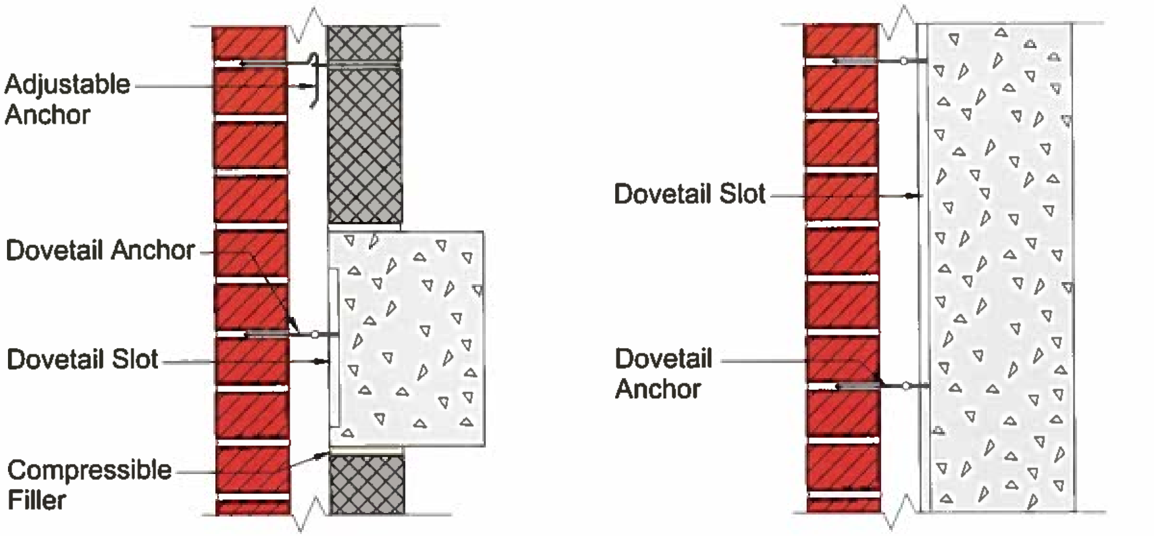
STRUCTURES WITHOUT SHELF ANGLES

Some buildings with brick veneer construction do not support the brickwork on shelf angles. These typically include low-rise buildings constructed with wood and steel stud framing and buildings with shear walls. *Building Code Requirements for Masonry Structures* limits brick veneer with wood or steel stud backing to a height of 30 ft (9 m) to the top plate and 38 ft (12 m) to the top of a gable. Brick veneer with a rigid backing of concrete or concrete masonry has no such limitation in the code. Brick veneer with this rigid backing may be supported by the foundation without intermediate shelf angles to a recommended maximum height of about 50 ft (15 m), provided the building is



Anchorage to Steel Beam (section)
(a)

Anchorage to Steel Column (plan)
(b)



Anchorage to Concrete Beam (section)
(c)

Anchorage to Concrete Column or Wall (section)
(d)

Figure 10
Flexible Anchorage to Beams and Columns

detailed appropriately for the differential movement and the moisture drainage system is designed and constructed properly. In these buildings, differential movement is accommodated by the anchor or tie system, window details, detailing at top of the wall and where other building components pass through the brickwork. These details must provide independent vertical movement between the brickwork and the backing. Building components that extend into or through the brick veneer (e.g., 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 movements between the structure and the brickwork. Figure 10 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

Movement joints must be provided in multi-wythe brick and concrete masonry walls. 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.

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 or flashing. 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 11.

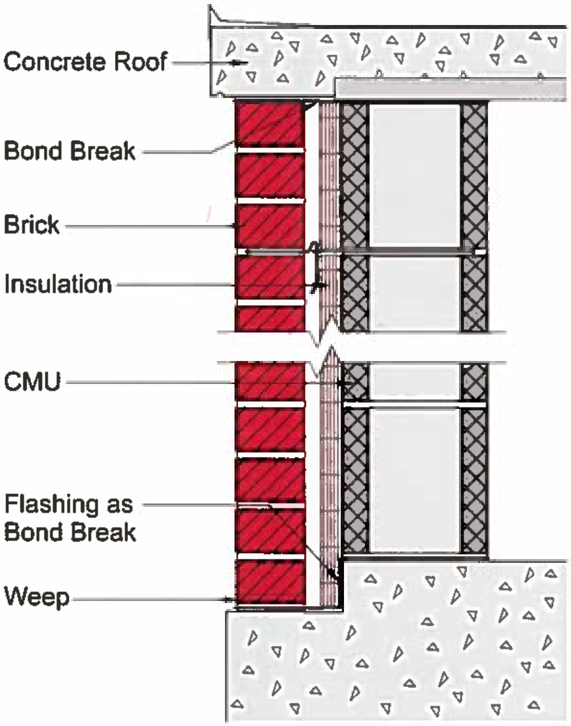


Figure 11
Bond Breaks in Loadbearing Cavity Wall

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.

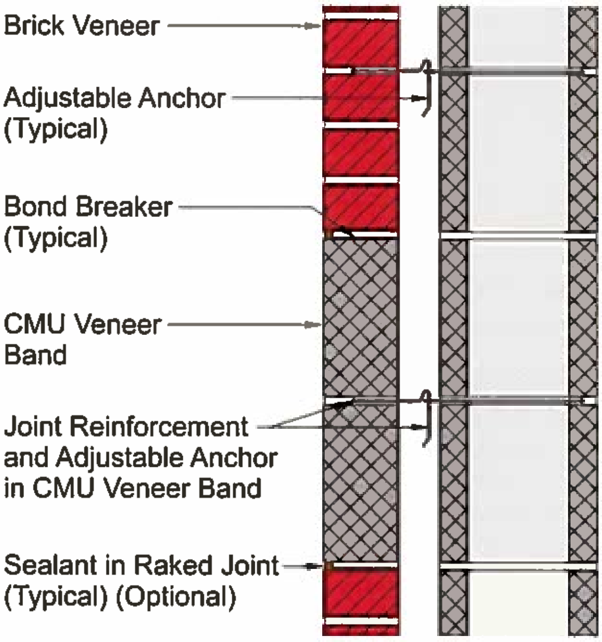
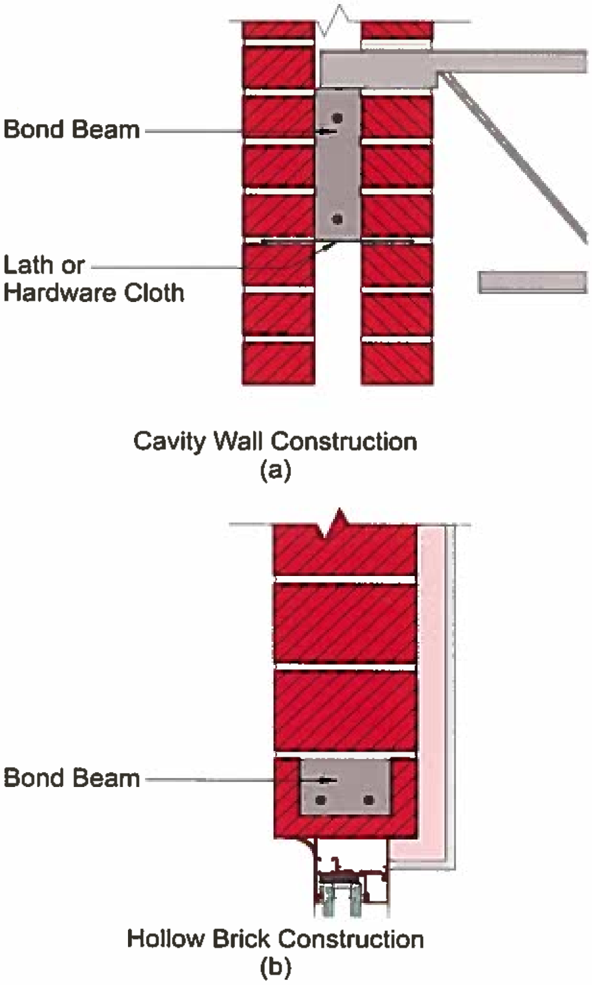


Figure 12
Multi-Course Concrete Masonry Band in Brick Veneer

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, additional control joints are recommended in the band. If the band is a single course, 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 12.

LOADBEARING MASONRY

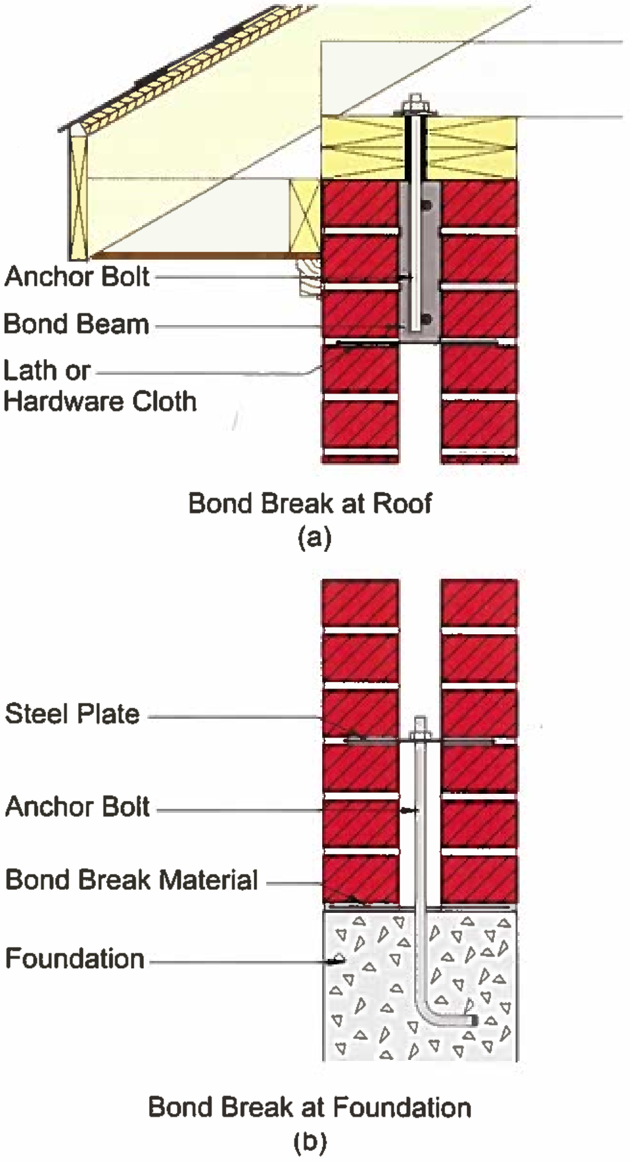
The potential for cracking in loadbearing masonry members is less than in nonloadbearing masonry members because compressive stresses from dead and



Cavity Wall Construction
(a)

Hollow Brick Construction
(b)

Figure 13
Bond Beams



Bond Break at Roof
(a)

Bond Break at Foundation
(b)

Figure 14
Bond Breaks

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 accommodate or distribute high stresses will also help control the effects of movement. Reinforcement may be placed in bed joints or in bond beams, as shown in Figure 13. Historic loadbearing structures were not constructed with expansion joints. However, these walls were made of multi-wythe brick construction, unlike typical structures built today.

When it is necessary to anchor a masonry wall to a foundation or to a roof, it is still possible to detail the walls in a manner that allows some differential movement, as shown in Figure 14a and Figure 14b. Such anchorage is often required for loadbearing walls subjected to high winds or seismic forces.

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 C 920, Standard Guide for Use of Elastomeric Joint Sealants, *Annual Book of Standards*, Vol. 04.07, ASTM International, West Conshohocken, PA, 2006.
2. Beall, C., *Masonry Design and Detailing for Architects, Engineers and Contractors, Fifth Edition*, McGraw Hill, Inc., New York, NY, 2003.
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* (ACI 530-05/ASCE 5-05/TMS 402-05), The Masonry Society, Boulder, CO, 2005.
5. "Building Movements and Joints," Portland Cement Association, Skokie, IL, 1982.

Exhibit 3

MAHONING VALLEY SANITARY DISTRICT
FILTER BUILDING ADDITION

Prepared for:
Mahoning Valley Sanitary District
1181 Ohltown-McDonald Road
Mineral Ridge, Ohio 44440

Prepared by:
ms consultants, Inc.
333 East Federal Street
Youngstown, Ohio 44503

June 2012

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INTRODUCTION

The purpose of this report is to provide recommendations for remediation of structural damage to the filter building addition, which has experienced cracking at the concrete floor slab-on-grade, at interior non-bearing masonry walls, and at exterior load bearing masonry walls. Photographs and a drawing showing locations of existing crack damage are included in Appendix A.

An earthquake of magnitude 4.0 occurred in the locality of the site on December 31, 2011. The cracking was first noticed by MVSD personnel on January 3, 2012. However, it is uncertain as to when the cracks occurred, and it is not known if the cracks existed prior to the earthquake. After the cracks were found, crack monitors were installed to determine if movement of the building addition is ongoing.

EXISTING BUILDING INFORMATION

The existing addition, built in 2005, has an overall plan dimensions of approximately 60 feet (north-south) and 30 feet (east-west). It is comprised of exterior masonry bearing walls founded on wall footings bearing approximately 5 feet below the finished floor. Drawings of the building floor plan, wall sections and structural details are contained in Appendix B.

PREVIOUS GEOTECHNICAL INVESTIGATIONS

Two previous geotechnical investigations have been performed specifically for the building addition: 1) for design of foundations, floor slab-on-grade and related site work; and 2) to determine possible geotechnical reasons that could cause the building to settle and crack.

2004 Geotechnical Investigation by Profession Service Industries, Inc.

In March 2004, two (2) test borings were made by Professional Service Industries, Inc. (PSI) in proximity of the building limits to obtain subsurface information for development of recommendations for design and construction of foundations to support the building addition, floor slab, as well as the general site development. The boring depths extended to 20 feet below existing grade at that time. In general, these borings encountered fill materials to a depth ranging from 12.5 to 15 feet below the ground surface. Below the fill materials, natural soils consisting of silt-clay materials were encountered until termination of the borings at the depth of 20 feet. No free water was encountered in the borings during the drilling operations. A copy of this investigation is contained in Appendix C.

In PSI's report, the recommendations for foundation design of the building addition stated, "that the building structure may be supported on continuous wall footing foundations bearing on compacted structural fill or suitable natural soils." The PSI report recommended that the footings for the new addition be designed for a maximum bearing pressure of 2000 psf; and that the minimum foundation widths for column and wall footings should be 24 inches and 18 inches, respectively, regardless if the design bearing pressure is less than 2000 psf. PSI recommended that exterior perimeter footings be placed at a minimum depth of 42 inches below finished ground to protect against frost action. In the PSI report, it stated, "We estimate maximum total and differential settlements of less than 1 inch and 3/4 inch, respectively."

Additionally in PSI's report, the recommendations for seismic design considerations stated, "Based on the field and laboratory tests and our experience with geology of the area, the average N-value information, and the apparent depth to the bedrock based on geologic references, we recommend that the seismic design be based on the site classification C."

2012 Geotechnical Investigation by S&ME

In April 2012, four (4) borings were made by S&ME about the perimeter of the building addition in an attempt to evaluate possible geotechnical cause(s) of the cracking found in the building's walls and floor slab. As part of that investigation, S&ME made a site reconnaissance, reviewed information related to the building cracks noted, and reviewed previous geotechnical information at the site including PSI's 2004 report. A copy of this investigation is contained in Appendix D.

In general, the borings made by S&ME encountered soils defined as fill, probable fill and possible fill consisting of silt-clay materials with varying amounts of sand, gravel, and/or rock fragments extending to depths of about 13 to 18 feet below the ground surface. Below this, and extending to depths of about 20.5 to 23.5 feet were natural soils comprised of clayey silt with varying amounts of sand and rock fragments. Underlying the natural soils, the borings encountered and were terminated in very soft shale bedrock. Water was encountered at only one of the four borings: first during drilling at a depth of about 16.5 feet, and later at completion of drilling at a depth of about 22.3 feet. All of the boreholes caved in at depths from about 19 to 22.5 feet at completion of drilling after removal of the augers.

In S&ME's report, the geotechnical evaluation for cracking of the building addition stated, "It is highly probable that settlement of the existing fill resulting from the self weight and the weight of new fill and loads of spread foundations is the primary cause of the cracks, and that the earthquake prompted the owner to recently look for and record existence of the cracks."

Additionally in S&ME's report, the geotechnical evaluation stated, "Based on the PSI recommendations, it is likely that the Filter Building addition was designed using a seismic site class C. Our seismic evaluation resulted in a recommendation to use seismic site class D. Therefore, we recommend that a structural engineer determine the associated affects to the structure, if any, resulting in a change of seismic site class from C to D."

CONCLUSIONS AND RECOMMENDATIONS

We agree with S&ME's geotechnical evaluation that the primary cause of the cracks found on the floor slab, and interior and exterior walls of the Filter Building Addition is most likely the result of settlement of the existing fill beneath the building. We also agree with S&ME's statement that the earthquake on December 31, 2011, was reason to look for and record the existence of cracks that may have resulted due to the potential effects of shaking from the earthquake hazard. However, it is uncertain if the cracks existed prior to the earthquake.

We agree with S&ME's seismic site classification based on the average standard penetration resistance value "N" that Site Class D is appropriate, and not C based on PSI's report recommendations. A description of seismic site classification as defined in ASCE 7-02 and ASCE 7-05 is included in Appendix E as presented by Dominic Kelly, "Seismic Site Classification for Structural Engineers", STRUCTURE magazine, December 2006.

The significance of seismic site classifications is that Site Class C will lead to a more economical structural design than Site Class D because Site Classes A, B, and C produce less intense shaking than Site Class D. We agree as stated in S&ME's report, "Therefore, we recommend that a structural engineer determine the associated affects to the structure, if any, resulting in a change of seismic site class from C to D."

The Earthquake Hazard Maps by FEMA (see Appendix F) use the Seismic Design Category (SDC) concept to categorize structures to seismic risk. Please note that SDC Categories A thru E used by FEMA **are not** the soil profile Site Classes A thru F used by ASCE. According to the FEMA Earthquake Hazard Maps, MVSD is located in SDC B, which is described as shaking of moderate intensity and the potential effects of slight damage.

Conclusion

We believe the primary cause of the cracks is probably the result of settlement of the existing fill beneath the building. Although the cracks may have existed prior to the earthquake on December 31, 2011, it is also possible that the 4.0 magnitude earthquake may have aggravated the severity and/or extent of the cracking, and the cracks became more noticeable.

Since the crack monitors that were installed on January 19, 2012, have yet to show any new movement, and the cracks are considered cosmetic damage and not detrimental to the function of the building; we recommend no major repairs be made at this time, and that the monitoring be continued until a change is noticed with regard to crack openings.

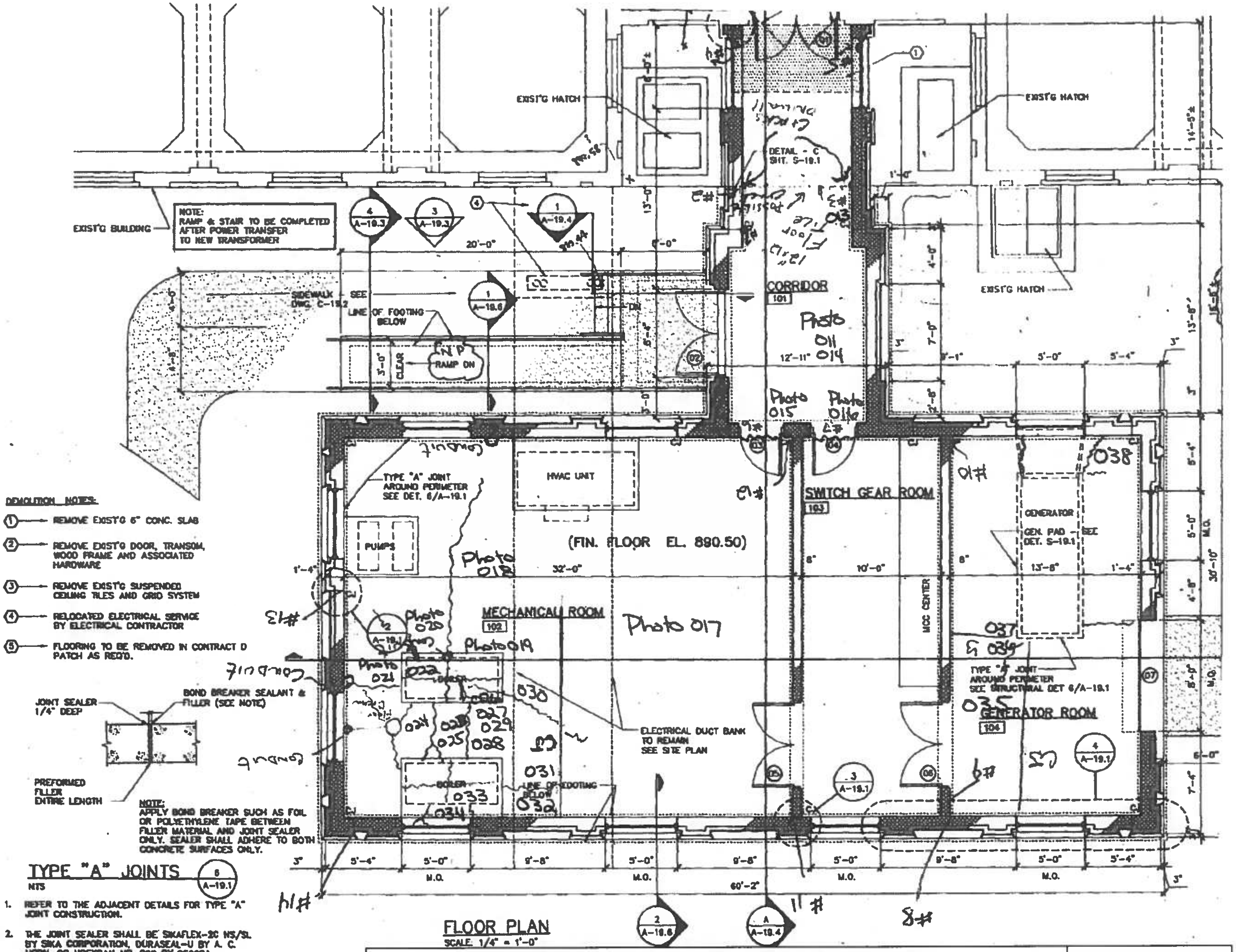
If cracks begin to seperate

Should any additional separation of the cracks occur due to settlement, there are remedies that can be considered to correct the problem; such as helical piers that are drilled into the lower solid soils and then attached to the existing foundation. However, prior to remediation of the structure or the existing crack damage, we recommend a building code evaluation relative to the proper seismic rating of the building addition first be made to determine if renovations to the structure are appropriate for resistance to Seismic Site Class D. Extensive and costly restorations to repair or stabilize possible further cracking and settlement may be of little practical value, if they will be inclusive in extensive rebuilding necessary for seismic upgrading.

of

APPENDIX A

EXISTING CRACK DAMAGE



NOTE:
RAMP & STAIR TO BE COMPLETED AFTER POWER TRANSFER TO NEW TRANSFORMER

DEMOLITION NOTES:

① REMOVE EXIST'G 6" CONC. SLAB

② REMOVE EXIST'G DOOR, TRANSOM, WOOD FRAME AND ASSOCIATED HARDWARE

③ REMOVE EXIST'G SUSPENDED CEILING TILES AND GRID SYSTEM

④ RELOCATED ELECTRICAL SERVICE BY ELECTRICAL CONTRACTOR

⑤ FLOORING TO BE REMOVED IN CONTRACT D PATCH AS REQ'D.

JOINT SEALER 1/4" DEEP

BOND BREAKER SEALANT & FILLER (SEE NOTE)

PREFORMED FILLER ENTIRE LENGTH

NOTE:
APPLY BOND BREAKER SUCH AS FOIL OR POLYETHYLENE TAPE BETWEEN FILLER MATERIAL AND JOINT SEALER ONLY. SEALER SHALL ADHERE TO BOTH CONCRETE SURFACES ONLY.

TYPE "A" JOINTS 6 A-19.1
NTS

1. REFER TO THE ADJACENT DETAILS FOR TYPE "A" JOINT CONSTRUCTION.
2. THE JOINT SEALER SHALL BE SIKAFLEX-2C NS/SL BY SIKA CORPORATION, DURASEAL-U BY A. C. HORN, OR UREXPAN NR-200 BY PECORA CORPORATION.
3. THE NON-EXTRUDING PREFORMED FILLER SHALL CONFORM TO THE REQUIREMENTS OF "STANDARD SPECIFICATIONS FOR PREFORMED SPONGE RUBBER AND CORK EXPANSION JOINT FILLERS FOR CONCRETE

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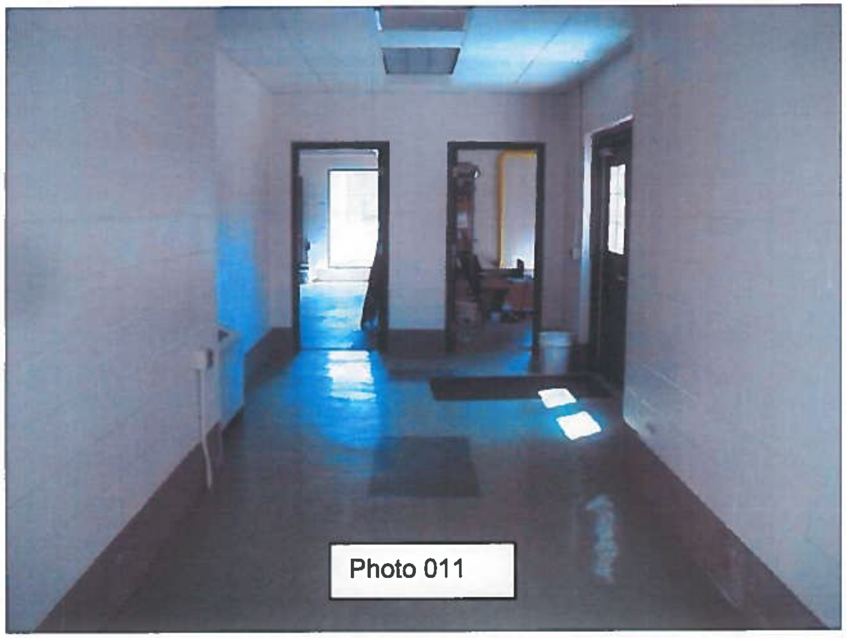
PHOTO AND CRACK MONITOR LOCATION MAP

0XX – INDICATES PHOTO NUMBER AND LOCATION
#XX – INDICATES (UPSIDE DOWN) CRACK MONITOR AND LOCATION

NORTH →



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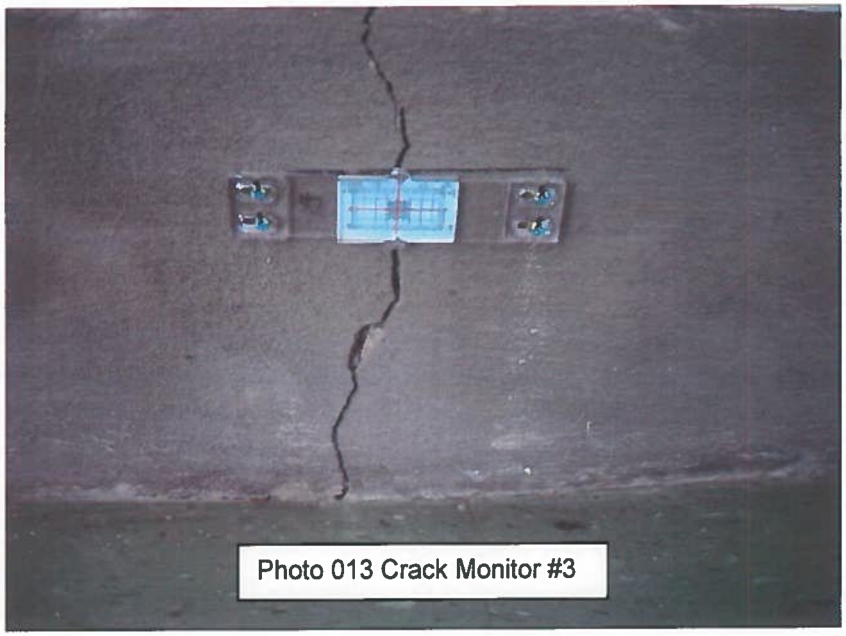

Photo 013 Crack Monitor #3

Photo 014

Photo 015 Crack Monitor #6

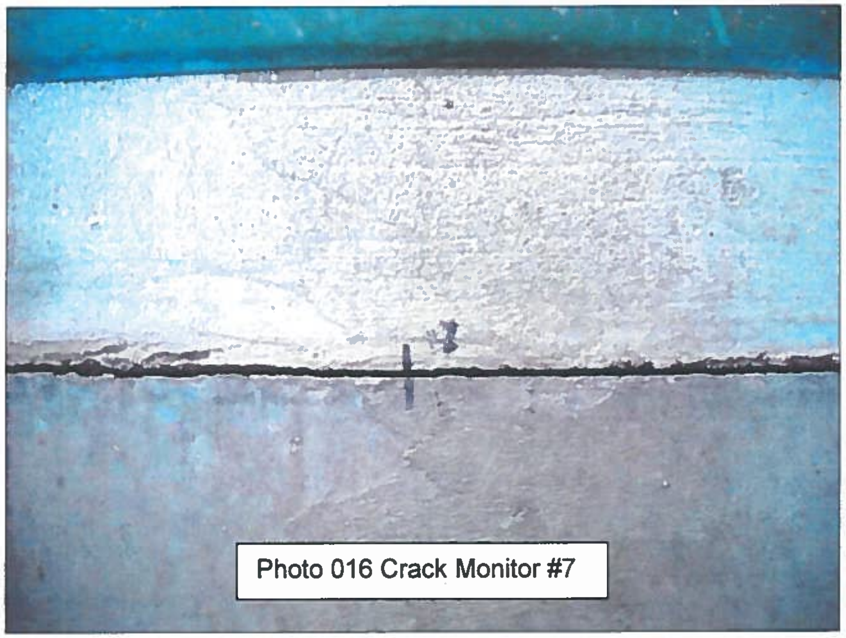

Photo 016 Crack Monitor #7

Photo 017

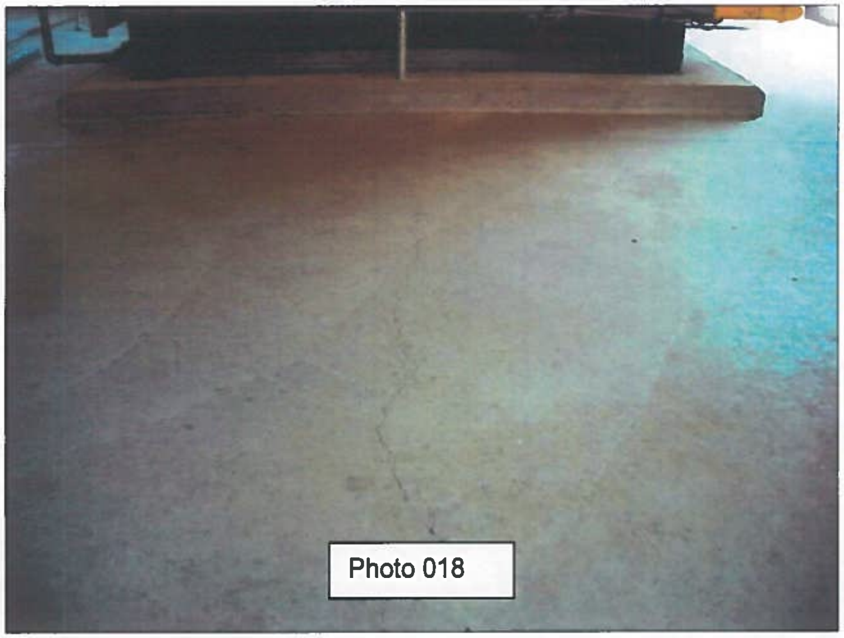

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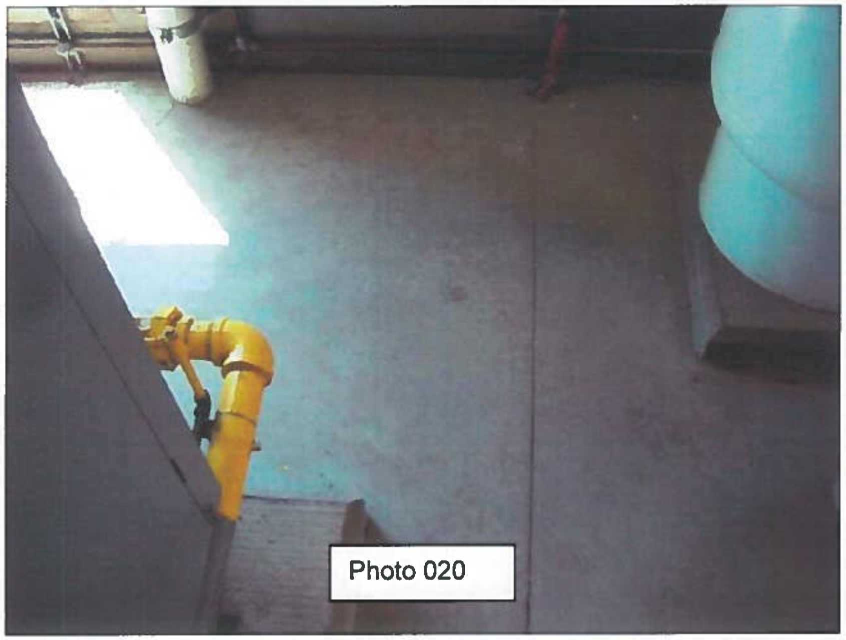
Photo 019



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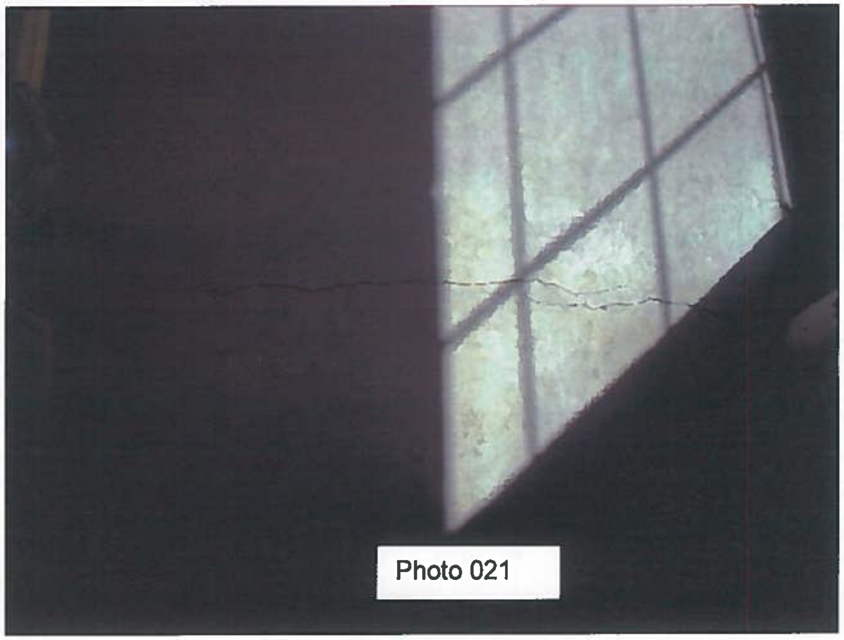

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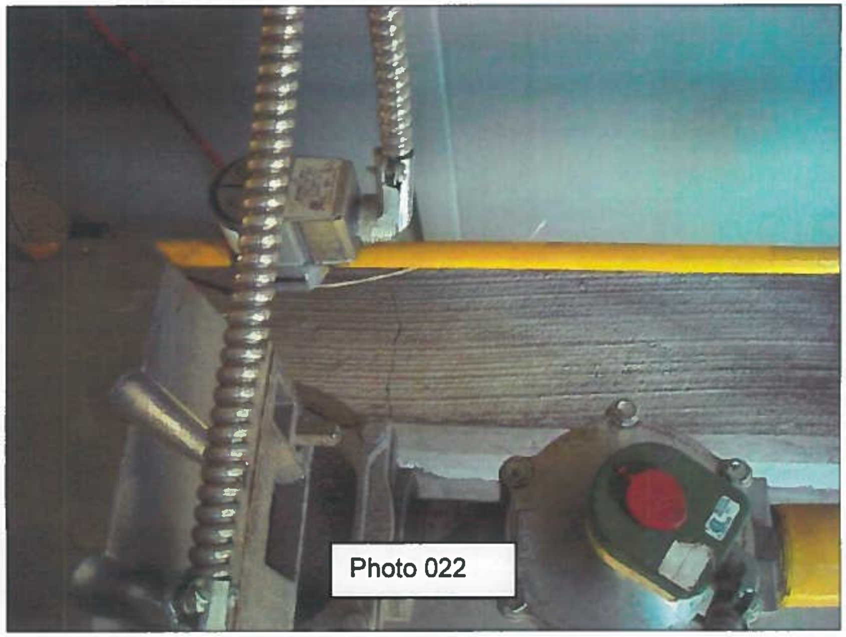

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Photo 023

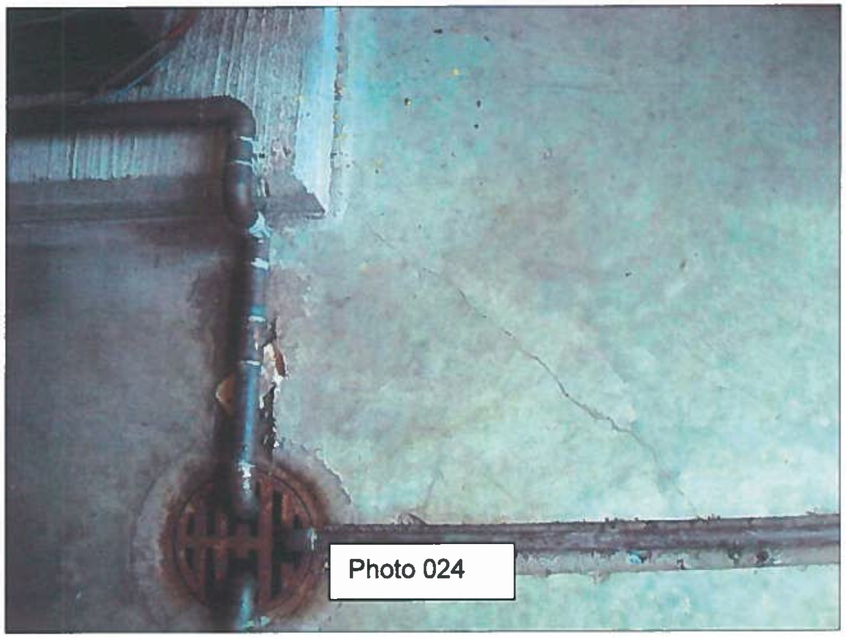

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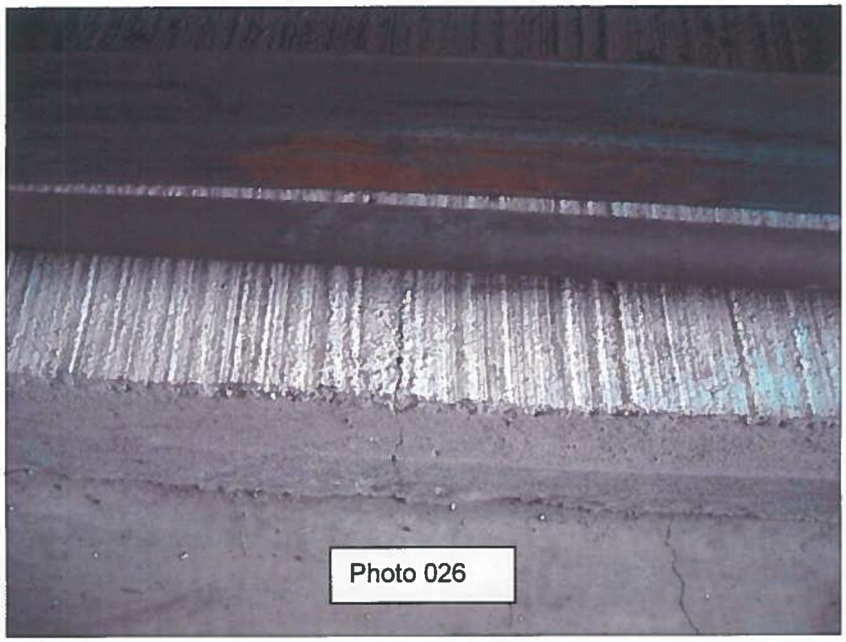
Photo 025



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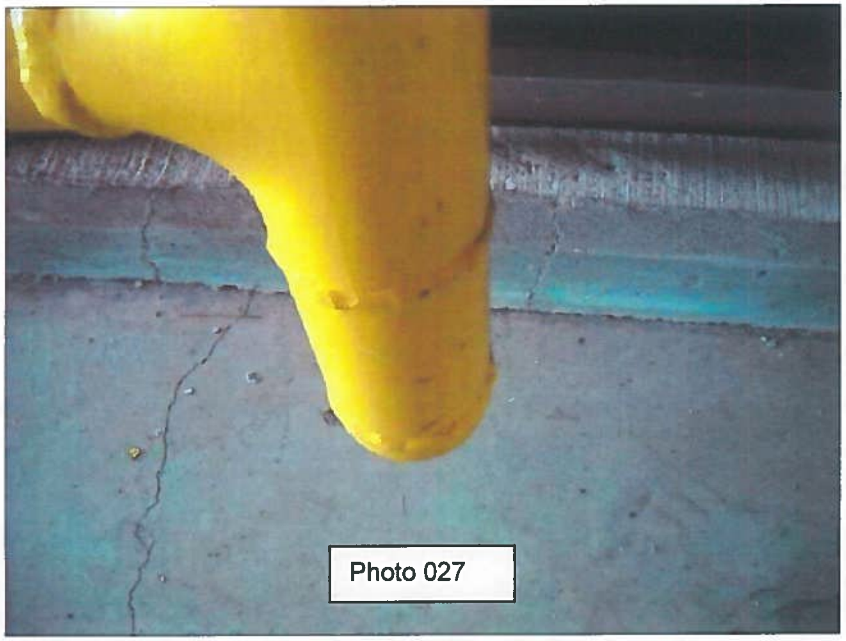

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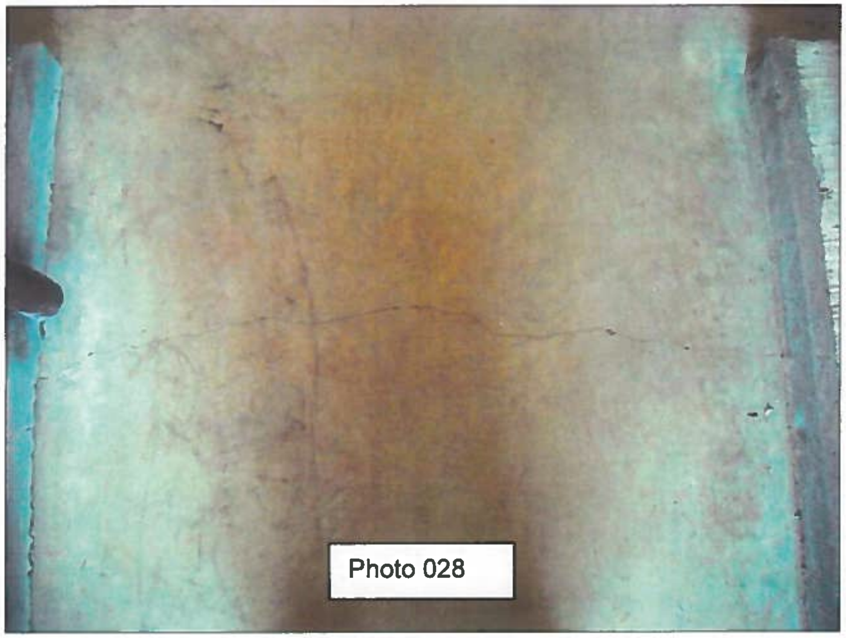

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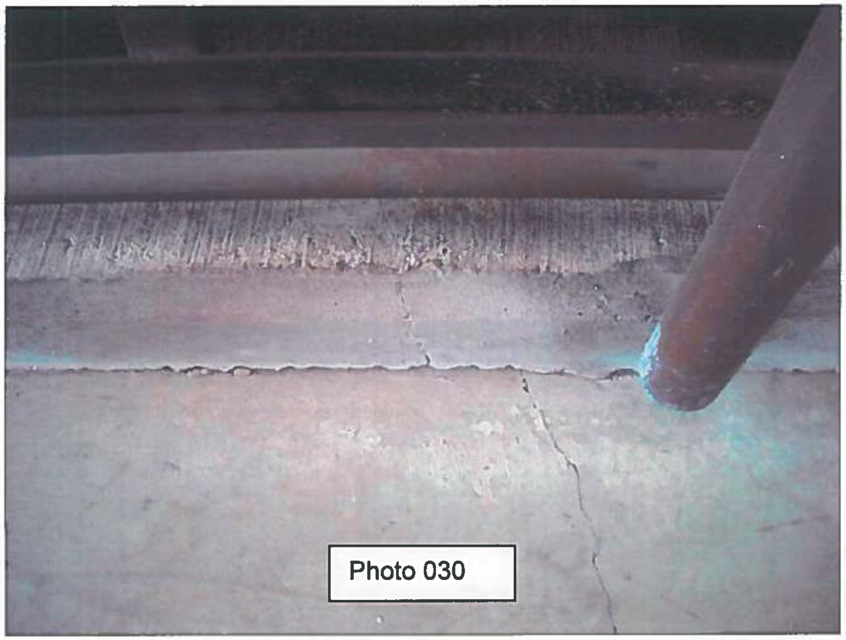
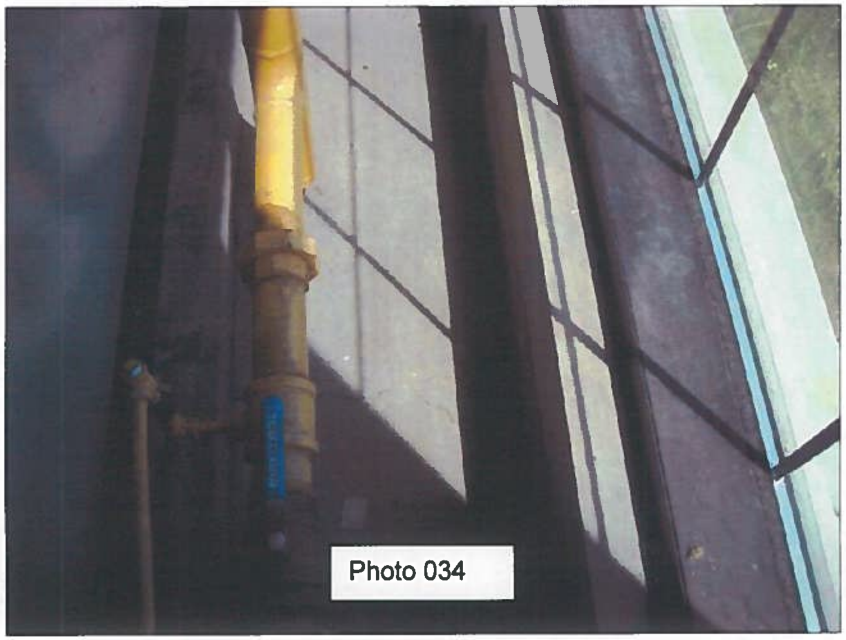


Photo 030

Photo 031

Photo 032

Photo 033



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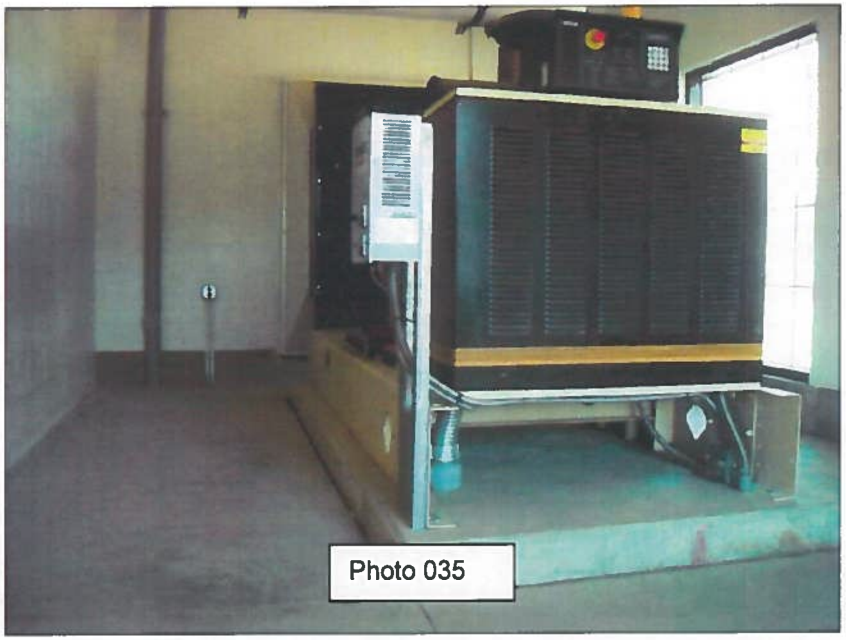


Photo 035

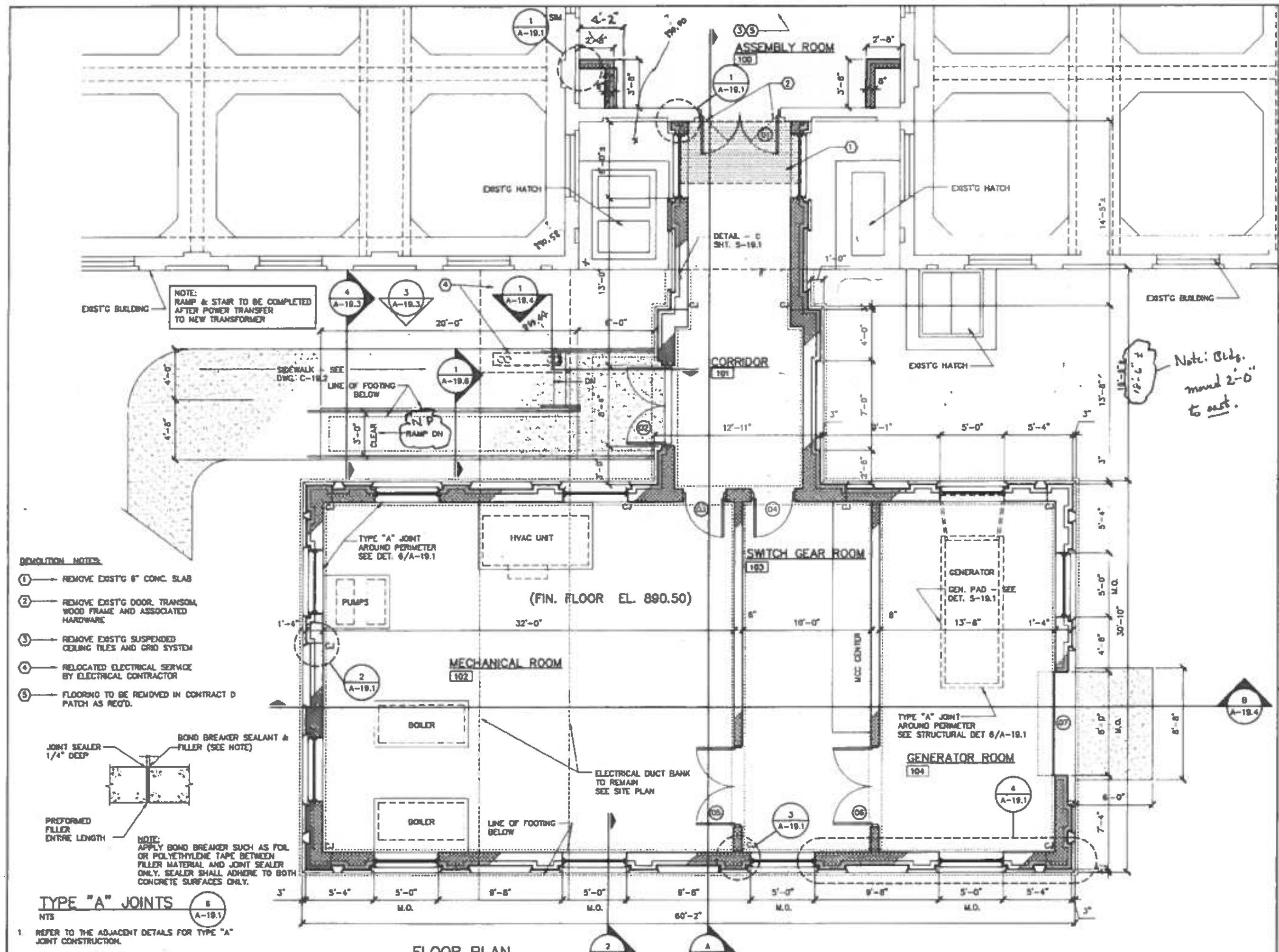
Photo 036

Photo 037

Photo 038

APPENDIX B

EXISTING BUILDING DRAWINGS



FLOOR PLAN

SCALE: 1/4" = 1'-0"

DEMOLITION NOTES:

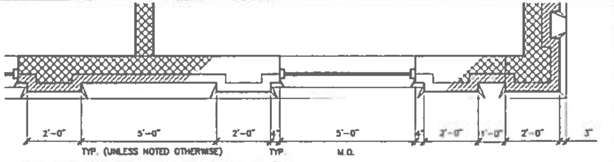
- ① REMOVE EXIST'G 6" CONC. SLAB
- ② REMOVE EXIST'G DOOR, TRANSOM, WOOD FRAME AND ASSOCIATED HARDWARE
- ③ REMOVE EXIST'G SUSPENDED CEILING TILES AND GRID SYSTEM
- ④ RELOCATED ELECTRICAL SERVICE BY ELECTRICAL CONTRACTOR
- ⑤ FLOORING TO BE REMOVED IN CONTRACT D PATCH AS REQ'D.

NOTE:
APPLY BOND BREAKER SUCH AS FOIL OR POLYETHYLENE TAPE BETWEEN FILLER MATERIAL AND JOINT SEALER ONLY. SEALER SHALL ADHERE TO BOTH CONCRETE SURFACES ONLY.

TYPE "A" JOINTS

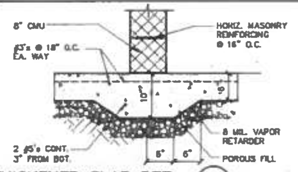
NTS 6 A-19.1

1. REFER TO THE ADJACENT DETAILS FOR TYPE "A" JOINT CONSTRUCTION.
2. THE JOINT SEALER SHALL BE SIKAFLEX-2C NS/SL BY SIKA CORPORATION, DURASEAL-U BY A. C. HORN, OR UREXPAN NR-200 BY PECORA CORPORATION.
3. THE NON-EXTRUDING PREFORMED FILLER SHALL CONFORM TO THE REQUIREMENTS OF "STANDARD SPECIFICATIONS FOR PREFORMED SPONGE RUBBER AND CORK EXPANSION JOINT FILLERS FOR CONCRETE PAVING AND STRUCTURAL CONSTRUCTION" ASTM D 1752, TYPE I, SPONGE RUBBER.
4. JOINT PREPARATION AND SEALANT INSTALLATION SHALL BE IN STRICT COMPLIANCE WITH THE SEALANT MANUFACTURER'S INSTRUCTIONS. ALL JOINT SURFACES MUST BE CLEAN, DRY AND FREE OF LOOSE AGGREGATE AND DUST. APPLY THE MANUFACTURER'S RECOMMENDED PRIMER TO THE TWO SIDES OF THE GROOVE PRIOR TO INSTALLING THE SEALANT. POLYETHYLENE BOND BREAKER TAPE SHALL BE APPLIED TO THE BOTTOM OF THE JOINT TO PREVENT BOND.



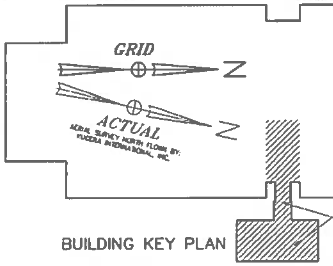
PARTIAL MASONRY LAYOUT

SCALE: 1/2" = 1'-0" 4 A-19.1

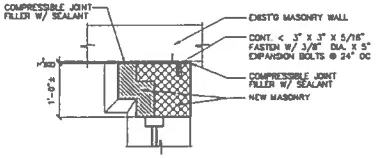


THICKENED SLAB DET.

SCALE: 1" = 1'-0" 5 A-19.1

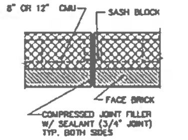


BUILDING KEY PLAN



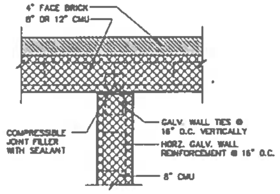
MASONRY DET.

SCALE: 1" = 1'-0" 1 A-19.1



CONTROL JOINT DET.

SCALE: 1" = 1'-0" 2 A-19.1



MASONRY DET.

SCALE: 1" = 1'-0" 3 A-19.1

REVISION	DATE	REMARKS	CHECKED

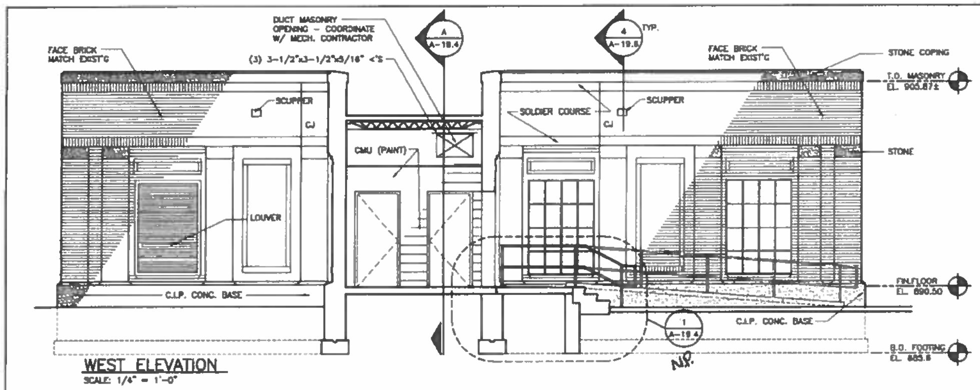
STATE OF OHIO
THE MAHONING VALLEY SANITARY DISTRICT
CONTRACT NO. 161

BUILDING ADDITION
FLOOR PLAN &
MASONRY DETAILS

DRAWN: A.D.T.
CHECKED: A.D.T.
APPROVED: W.G.G.
ISSUE DATE: AUGUST, 2004
SCALE: AS SHOWN
DWG. NO. A-19.1

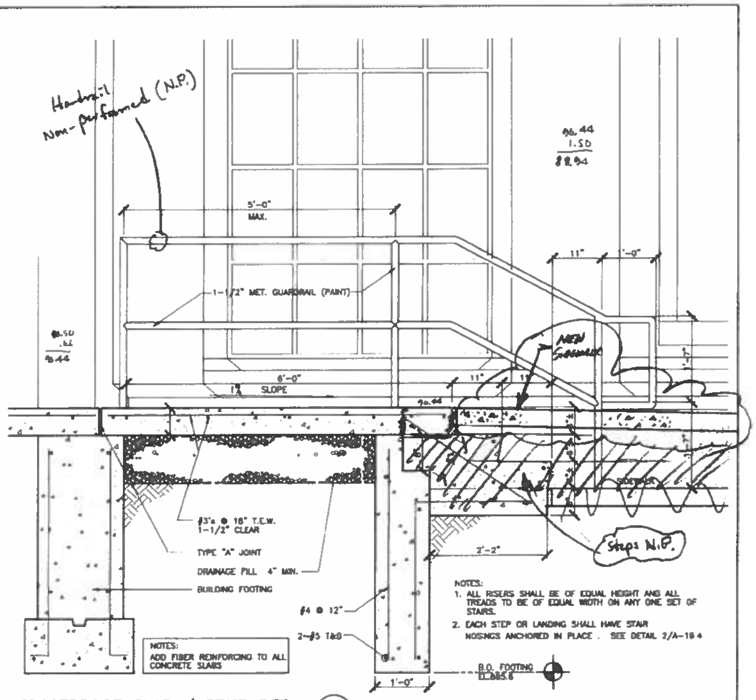
CT Consultants
engineers | architects | planners

SEAL



WEST ELEVATION

SCALE: 1/4" = 1'-0"



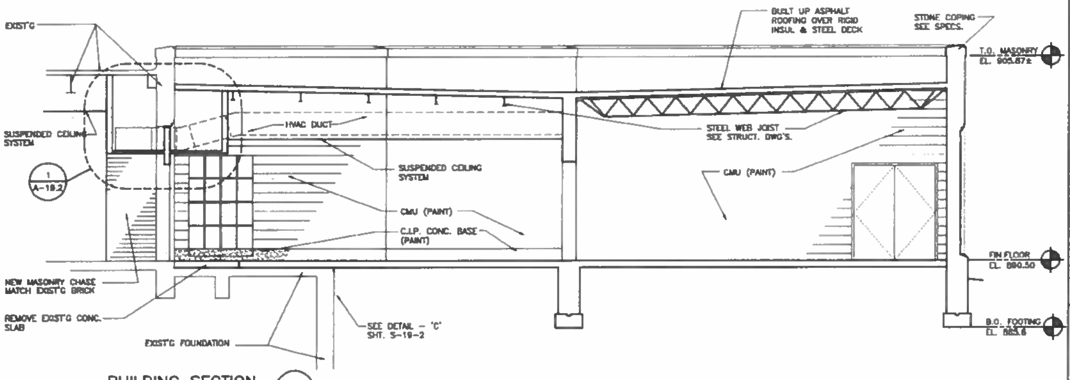
NOTES:
1. ALL RISERS SHALL BE OF EQUAL HEIGHT AND ALL TREADS TO BE OF EQUAL WIDTH ON ANY ONE SET OF STAIRS.
2. EACH STEP OR LANDING SHALL HAVE STAIR NOSINGS ANCHORED IN PLACE. SEE DETAIL 2/A-19.4

NOTES:
ADD FIBER REINFORCING TO ALL CONCRETE SLABS

FROSTPROOF SLAB / STAIR DET.

1
A-19.4

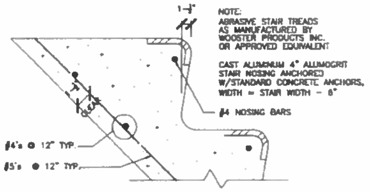
SCALE: 1" = 1'-0"



BUILDING SECTION

A
A-19.4

SCALE: 1/4" = 1'-0"



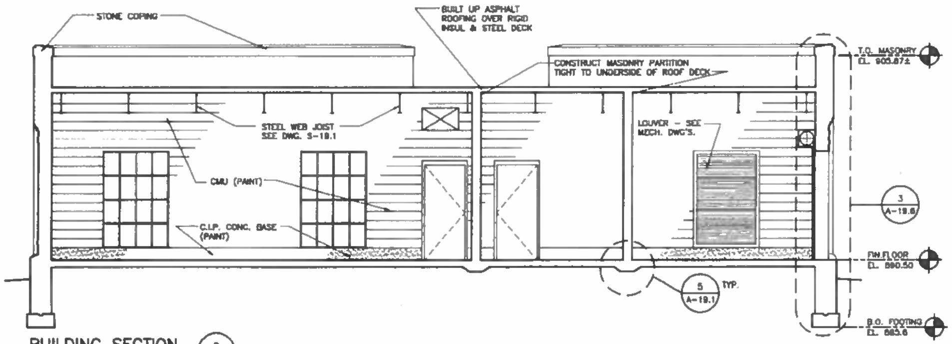
NOTE:
ABRASIVE STAIR TREADS AS MANUFACTURED BY WOOSTER PRODUCTS INC. OR APPROVED EQUIVALENT

CAST ALUMINUM 4" ALUMOGRIT STAIR NOSING ANCHORED W/STANDARD CONCRETE ANCHORS, WIDTH = STAIR WIDTH - 8"

STD. CONC. STAIRS

2
A-19.4

NTS



BUILDING SECTION

B
A-19.4

SCALE: 1/4" = 1'-0"

REVISION	DATE	REMARKS	CHECKED

STATE OF OHIO
THE MAHONING VALLEY SANITARY DISTRICT
CONTRACT NO. 181

BUILDING SECTIONS / ELEVATIONS RAMP & STAIR DETAILS

DRAWN: A.B.T.
CHECKED: A.B.T.
APPROVED: R.G.C.

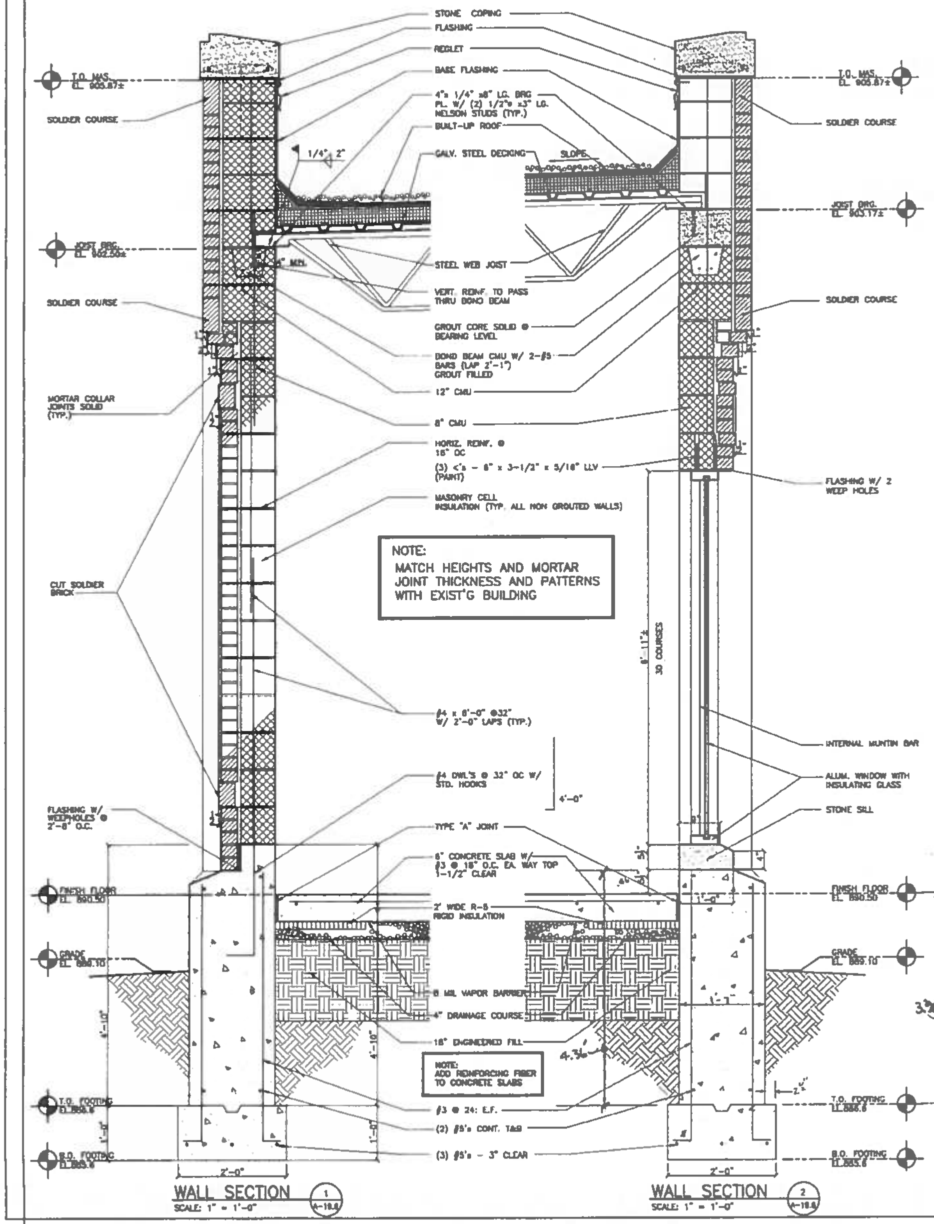
ISSUE DATE: AUGUST, 2004
SCALE: AS SHOWN

DWG. NO. A-19.4

CT Consultants
engineers | architects | planners

SEAL

DATE 07-28-04 10:38 , CT CONSULTANTS, INC. H:\2003\03269\Dwg\03269A02.dwg

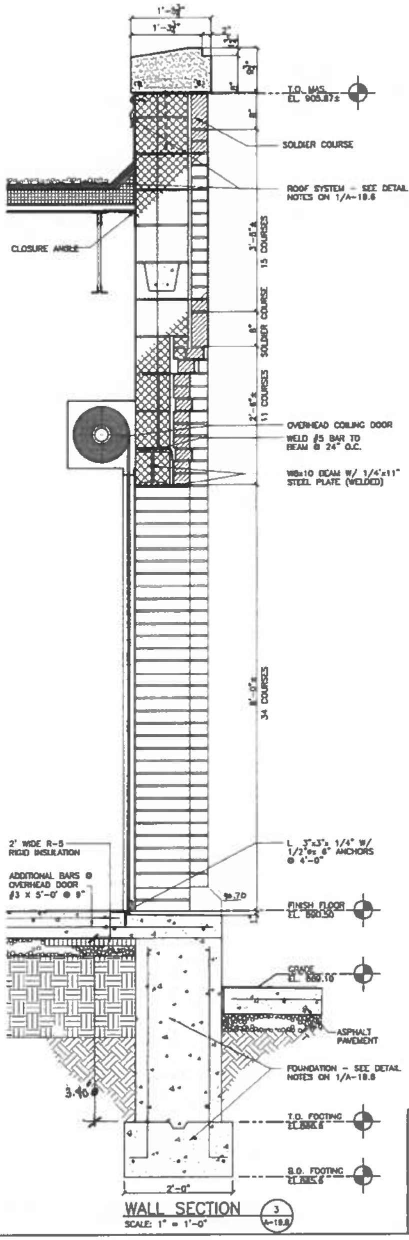


NOTE:
MATCH HEIGHTS AND MORTAR JOINT THICKNESS AND PATTERNS WITH EXIST'G BUILDING

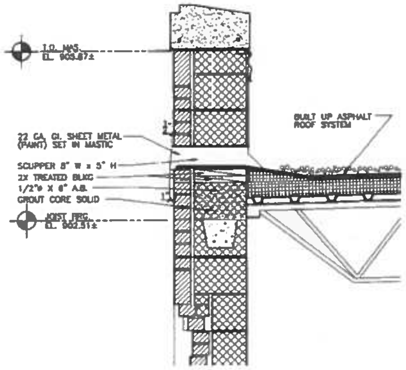
NOTE:
ADD REINFORCING FIBER TO CONCRETE SLABS

WALL SECTION ① A-19.6
SCALE: 1" = 1'-0"

WALL SECTION ② A-19.6
SCALE: 1" = 1'-0"



WALL SECTION ③ A-19.6
SCALE: 1" = 1'-0"



SCUPPER DET. ④ A-19.6
SCALE: 1" = 1'-0"

REVISION	DATE	REMARKS	CHECKED

STATE OF OHIO
THE MAHONING VALLEY SANITARY DISTRICT
CONTRACT NO. 161

WALL SECTIONS
DETAILS

DRAWN: A.D.T.
CHECKED: A.D.T.
APPROVED: W.C.C.

ISSUE DATE: AUGUST, 2004
SCALE: AS SHOWN

DWG. NO. A-19.6

CT Consultants
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SEAL

APPENDIX C

2004 GEOTECHNICAL INVESTIGATION BY PSI

GEOTECHNICAL ENGINEERING SERVICES REPORT

For

PROPOSED BUILDING ADDITION
MAHONING VALLEY SANITARY DISTRICT
Niles, Trumbull County, Ohio

Prepared for

CT CONSULTANTS, INC.
35000 Kaiser Court
Willoughby, Ohio 44094

Prepared by

Professional Service Industries, Inc.
1057-L Trumbull Avenue
Girard, Ohio 44420

Telephone (330) 759-0288

PSI PROJECT NO. 139-45004

March 9, 2004

psi Information To Build On
Engineering • Consulting • Testing

psi Information To Build On
Engineering • Consulting • Testing

March 9, 2004

Mr. Ronald J. Champlin, P.E.
Structural Section Manager
CT Consultants, Inc.
35000 Kaiser Court
Willoughby, Ohio 44094

Geotechnical Engineering Services Report
Proposed Building Addition
Mahoning Valley Sanitary District
Salt Springs Road
Niles, Trumbull County, Ohio
PSI Project No. 139-45004

Dear Mr. Champlin:

Professional Service Industries, Inc. (PSI) is pleased to submit three (3) copies of this Geotechnical Engineering Services Report for the above referenced project. Included in this presentation are the results of the exploration and recommendations concerning the design and construction of the foundations and pavements, as well as general site development.

We appreciate the opportunity to have provided you with our geotechnical engineering services and look forward to participation in the construction phase of this project. If you have any questions concerning this report or if we may be of further service in any manner, please contact our office.

Respectfully submitted,
Professional Service Industries, Inc.

Robert A. Williamson, E.I.T.
Staff Engineer

Alagaiya Veeramani SH
Alagaiya Veeramani, PE
District Manager

psi Information To Build On
Engineering • Consulting • Testing

GEOTECHNICAL ENGINEERING SERVICES REPORT

For

PROPOSED BUILDING ADDITION
MAHONING VALLEY SANITARY DISTRICT
Niles, Trumbull County, Ohio

Prepared for

CT CONSULTANTS, INC.
35000 Kaiser Court
Willoughby, Ohio 44094

Robert A. Williamson, E.I.T.
Staff Engineer

Prepared by

Professional Service Industries, Inc.
1057-L Trumbull Avenue
Girard, Ohio 44420

Telephone (330) 759-0288

Alagaiya Veeramani SH
Alagaiya Veeramani, P.E.
District Manager

PSI PROJECT NO. 139-45004

March 9, 2004

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  Boring Location Plan
  General Notes
  Boring Records (B-1 through B-2)
  Unified Soil Classification

GEOTECHNICAL ENGINEERING SERVICES REPORT

INTRODUCTION

PSI has completed a subsurface exploration and geotechnical engineering evaluation for the proposed building addition at the Mahoning Valley Sanitary District, located off of Salt Springs Road in Niles, Trumbull County, Ohio. PSI's services for this project were performed in accordance with PSI Proposal No. 142-450077, dated February 20, 2004. Authorization to perform this exploration and analysis was in the form of CT Consultants, Inc. purchase order no. 6640-04, dated February 23, 2004.

The purpose of this study was to explore the subsurface conditions at the site to enable an evaluation of possible foundation systems for the proposed building addition. This report briefly outlines the testing procedures, describes the site and subsurface conditions, and discusses the foundation recommendations.

The scope of services did not include an environmental assessment for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, ground water or air, on, or below or around this site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of the client. Prior to development of this site, an environmental assessment is advisable.

Under the terms of our proposal No. 142-450047, dated February 20, 2004, PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. As stated in the above referenced proposal, mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Site conditions are outside of PSI's control and mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or reoccurrence of mold amplification.

PROPOSED CONSTRUCTION

We understand that the proposed addition to the Mahoning Valley Sanitary District building will consist of a single-story, masonry, slab-on-grade structure measuring approximately 1,800 square feet in plan area, with no basement. No structural load information for the structure was provided at the time of this report. Column spacing as well as grade changes were not supplied at the time of this report. It has been estimated that design floor loads will be 150 psf.

The recommendations provided in this report are based on the provided plan and our presented assumptions. If any of the above information should change significantly or be in error, it should be brought to our attention so that we may review the recommendations made in this report.

TESTING PROCEDURES

Field Operations

Two (2) soil test borings were performed at the site at the approximate locations shown on the Boring Location Plan presented in the Appendix. The boring depths extended to approximately twenty (20) feet below existing grade. The boring depths and locations were selected by PSI. All test borings were field located by the representatives of PSI by measuring from readily identifiable site features shown on available project plans. Additionally, a representative of the Mahoning Valley Sanitary District was present during boring location selection to ensure no obstructions would be encountered during drilling operations.

The borings were advanced into the ground using hollow stem augers. At regular intervals throughout the boring depths, soil samples were obtained with a split spoon sampler. The split spoon sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot, where possible, with blows of a 140 pound hammer falling 30 inches. The number of hammer blows required to drive the sampler each six inch increment is recorded in the field. The penetration resistance "N-value" is designated as the number of hammer blows required to drive the sampler the final foot and, when properly evaluated, is an index to cohesion for clays and relative density for sands. The split spoon sampling procedures used during this exploration are in basic accordance with ASTM Designation D-1586.

Laboratory Testing

The soil samples obtained during the field exploration were transported to the laboratory and visually examined. The soil samples obtained from the drilling operation were classified in general accordance with ASTM D-2488 (Visual-Manual Procedure for Description of Soils). Soil classifications include the use of the Unified Soil Classification System described in ASTM D-2487 (Classification of Soils for Engineering Purposes). Water content determinations (ASTM D-2211) were also conducted. Descriptions of the soils encountered in the test borings are provided on the Boring Logs included in the Appendix. Groundwater conditions, standard penetration resistances, and other pertinent information are also included. The soil samples will be retained at our office for 60 days from the date of this report and then discarded.

SITE AND SUBSURFACE CONDITIONS

Site Location and Description

The overall site for the proposed building addition to the Mahoning Valley Sanitary District building, upon which this soils exploration has been performed, is located along the southern side of Salt Springs Road, Niles, Trumbull County, Ohio. Specifically, the proposed site area is located approximately 900 feet west of the intersection with State Route 46.

The site consists of multiple building complexes utilized for the purification and distribution of water within the Mahoning Valley. The proposed building addition is to be located immediately adjacent to the administration building along the western side. More specifically, the proposed building addition is to be located immediately west of the filter building located within the administration building at an approximate surface elevation of 922 ft above mean sea level.

The site slopes is relatively flat. There are areas within the site that contain depressions. More specifically, there is one (1) depression located approximately 150-feet west of the proposed building addition. Overhead and/or underground utilities were observed to be running into the site, and therefore, we recommend that all utilities be marked prior to construction activities.

Subsurface Conditions

The following subsurface description is of a generalized nature, provided to highlight the major soil strata encountered. The Boring Logs should be reviewed for specific information as to individual boring locations. The stratifications shown on the Boring Logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual.

At the test boring locations B-1 and B-2, a surficial layer of asphalt, gravel and/or slag material was encountered having a thickness of approximately 12 to 18 inches.

Beneath the overlying asphalt, gravel and/or slag, the test boring locations B-1 and B-2 consisted of clayey gravel fill material to a depth ranging of approximately 12.5 to 15-feet below surface grade. Below the fill materials, natural soils consisting of silt and silty clay were encountered until the termination depths. The cohesive natural soils have moisture contents ranging from approximately 9 to 24 percent, and exhibit soft to hard consistency based on the Standard Penetration tests.

Groundwater Conditions

During the field drilling operations at the test boring locations B-1 and B-2, no free water was encountered. However, due to the short time the boreholes remained open, the water level observations in the boreholes may not be representative of actual groundwater levels. For safety purposes, all test borings were backfilled at the time of drilling completion. Please note that groundwater levels will fluctuate and may occur at higher elevations at some time in the future. We recommend that the contractor determine the actual groundwater level at the time of construction to determine the impact, if any, on the construction procedures.

SITEWORK RECOMMENDATIONS

It is recommended that PSI be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. PSI cannot accept responsibility for any conditions which deviate from those described in this report, nor for the performance of the foundation system if not engaged to also provide construction observation and testing for this project.

construction traffic following placement, and must be protected against such disturbance by limiting traffic and/or placing aggregate.

Drainage and Groundwater Considerations

During the field drilling operations at the test boring locations B-1 and B-2, no free water was encountered. However, water seepage and/or free water may be encountered within the upperlying fill soils during cut excavations to remove objectionable materials and also during foundation excavation and installation. If water is encountered during excavation operations, a gravity drainage system, sump pump or other conventional dewatering procedure, as deemed appropriate by the field conditions, should be employed during construction. Every effort should be made to keep the excavations dry if water is encountered.

Water should not be allowed to collect near the foundation or over the floor slab areas of the building structure either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater or surface runoff. Positive site drainage should be provided at all times during construction to reduce infiltration of surface water around the perimeter of the building and beneath the floor slab. Overall site area drainage is to be arranged in a manner such that the possibility of water impounding below slab-on-grade areas, pavement sectors and over the structural fill is prevented.

Floor Slab Preparation

Proofrolling should be performed as outlined in the Site Preparation section of this report. We recommend that the floor slab subgrade be evaluated by a representative of the Geotechnical Engineer immediately prior to placing stone and beginning floor slab construction. If low consistency soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted fill material placed in accordance with the *Structural Fill* section of this report or with well-compacted crushed stone materials.

Federal Excavation Safety Regulations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better insure the safety of workers entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred. If the excavations are left open and exposed to the elements for a significant length of time, desiccation of the clays may create minute shrinkage cracks which could allow large pieces of clay to collapse or slide into the excavation. Materials removed from the excavation should not be stockpiled immediately adjacent to the excavation, inasmuch as this load may cause a sudden collapse of the embankment.

Seismic Design Considerations

In the 2002 Ohio Building Code (OBC), the State of Ohio has adopted the provisions of the 2000 International Building Code (IBC). Under the current code provisions, the effect of soil amplification on earthquake ground motions is taken into account by adjusting the earthquake spectral response accelerations for the soil or rock conditions at the site. The code groups soil or rock conditions into five Site Classes, as defined in Table 1615.1.1, with the site coefficients Fa and Fv increasing from Site Class A through F. The Site Class is based on a weighted average of known or estimated soil properties for the uppermost 100 feet of the subsurface profile.

Soil borings in the structure area at the project site extended to depths of approximately twenty (20) feet below existing grades. Based on our review of the available data, PSI evaluated the Site Class using the weighted average of the Standard Penetration Test (SPT) N-values of the soil samples. Based on the field and laboratory tests and our experience with the geology of the area, the average N-value information, and the apparent depth to the bedrock based on geologic references, we recommend that the seismic design be based on the site classification C.

FOUNDATION AND FLOOR SLAB RECOMMENDATIONS

Foundation Design

The results of the test borings and our evaluation indicate that the building structure may be supported on conventional spread or continuous wall footing foundations bearing on compacted structural fill or suitable natural soils.

Spread footings for building columns and continuous footings for bearing walls should be designed for a maximum soil bearing pressure of 2,000 psf based on dead load plus design live load. Minimum foundation widths for column and strip footings should be 24 inches and 18 inches, respectively, even if the bearing pressure is less than the recommended values. All perimeter footings and canopy foundations must be placed at a minimum depth of 42 inches below the finished grade in order to protect against frost action. Interior foundations not subject to frost action or in heated areas may be placed at a minimum depth of 18 inches below the floor slab, provided they will be bearing on acceptable soils.

The recommended soil bearing pressure includes a factor of safety of at least 3.0 against shear failure. We estimate maximum total and differential settlements of less than 1-inch and ¾-inch, respectively.

Foundation bearing surface evaluations should be performed in the shallow foundation excavations to identify isolated poor quality soils and to enable the development of remedial measures, if needed. Foundation bearing surface evaluations should be performed in each excavation prior to placement of reinforcing steel by a representative of PSI. Soft or loose soil zones encountered at the foundation subgrades should be remediated as directed by the Geotechnical Engineer.

After opening, footings should be evaluated and concrete placed as quickly as possible to avoid exposure of the footing bottoms to disturbance. If it is required that footing excavations be left open for more than a few hours, they should be protected against disturbance with a lean concrete mud mat.

Floor Slab Design

An on-grade floor slab supported on suitable proofrolled existing fill, compacted engineered fill, or natural soils may be used for this structure. We recommend that a subgrade modulus (k) of 100 pci be used in floor slab design calculations.

We recommend that a minimum 6-inch thick, free-draining granular material, such as AASHTO No. 57 stone, be placed beneath the floor slab to enhance drainage. The floor slab should be jointed in accordance with ACI specifications to reduce cracking resulting from any differential movement and shrinkage. We also suggest that, where practical, the floor slabs not be rigidly connected to columns, walls, or foundations.

Impermeable vapor barriers under concrete slabs will be required for this structure. The final decision to use a vapor barrier is left to the owner and designers. If used, however, we recommend that a 10-mil thick polyethylene sheeting as recommended by ACI's *Guide for Concrete and Floor Slab Construction*, be utilized as a vapor barrier, and be placed between the crushed stone materials and the concrete slab.

We recommend that the floor slab subgrades be evaluated by a PSI representative immediately prior to placing stone and beginning floor slab construction. If low consistency soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted soil or crushed stone material placed in accordance with the *Structural Fill* section of this report.

GENERAL COMMENTS

The recommendations submitted are based on the available soil information obtained by PSI and preliminary design details furnished by CT Consultants, Inc. for the proposed building addition. If there are any revisions to the plans for the proposed development, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be retained to determine if changes in the foundation recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the performance of the structure.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made after being prepared in accordance with generally accepted professional engineering practices in the local areas. No other warranties are implied or expressed.

After the plans and specifications are more complete, it is recommended that the geotechnical engineer be provided the opportunity to review the final design and specifications to determine if the engineering recommendations have been properly interpreted and implemented. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of CT Consultants, Inc. for the specific application to the proposed building addition at the Mahoning Valley Sanitary District Facility, Niles, Trumbull County, Ohio.

APPENDIX

Site Vicinity Map

Boring Location Plan

General Notes

Boring Records (B-1 – B-2)

Unified Soil Classification

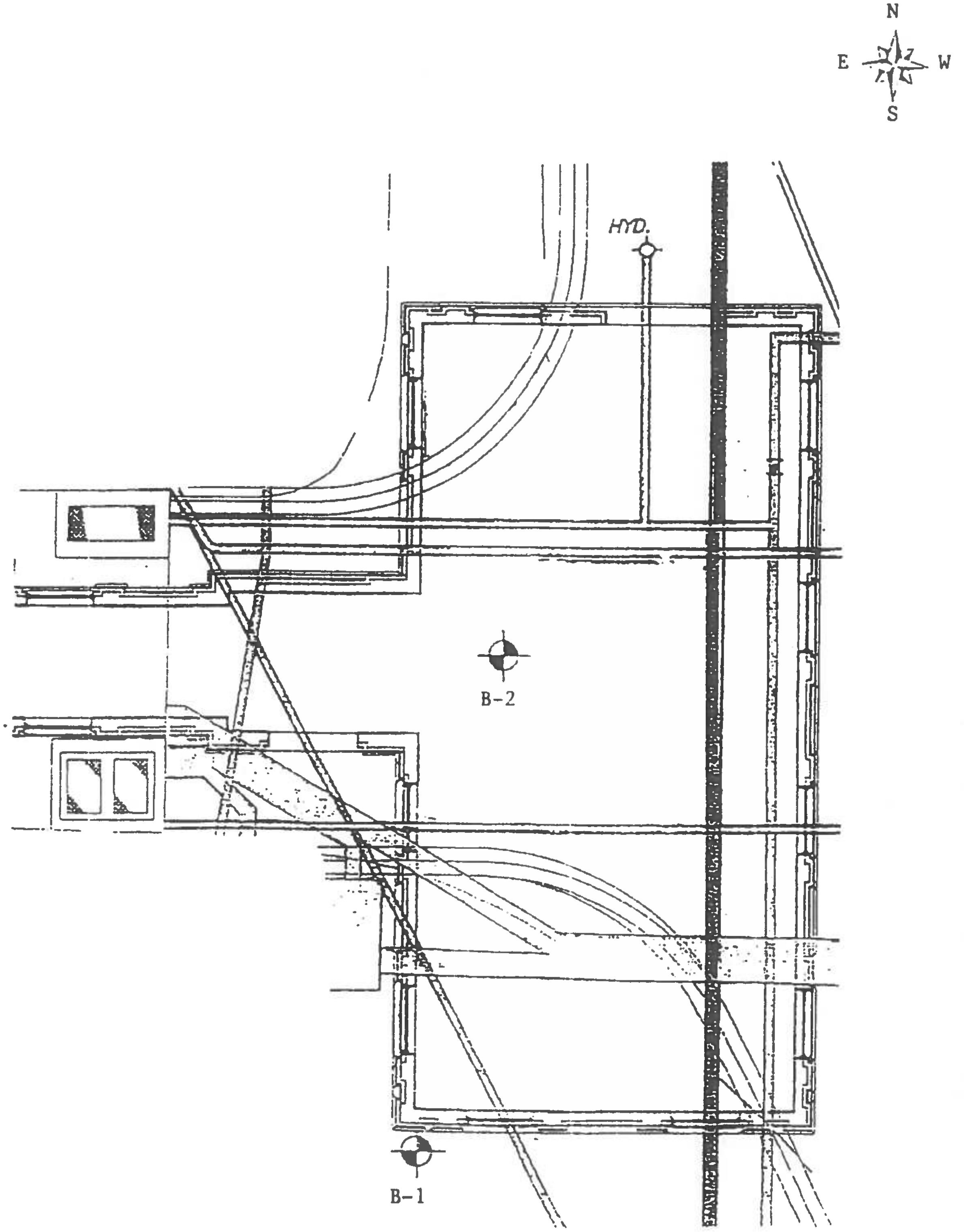
Project Name:
Mahoning Valley Sanitary District
Salt Springs Road
Niles, Trumbull County, Ohio

Map Provided by Topozone.com
Scale: NOT TO SCALE

Project No.:
139-45004

Date:
March 9, 2004

Professional Service Industries



Plan Not To Scale

PROJECT NAME
Mahoning Valley Sanitary District
Salt Spring Road
Niles, Trumbull County, Ohio

Boring Location Plan
Plan Provided by CT Consultants, Inc.

PROJECT NO
139-45004

DATE
March 9, 2004

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System is used to identify the soil unless otherwise noted.

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2 inch O.D. split-spoon.

Qu: Unconfined compressive strength, TSF

Qp: Penetrometer value, unconfined compressive strength, TSF

Mc: Water content, %

LL: Liquid limit, %

PI: Plasticity Index, %

δd: Natural dry density, PCF

▼: Apparent groundwater level at time noted after completion.

DRILLING AND SAMPLING SYMBOLS

SS	Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST	Shelby Tube - 3" O.D., except where noted.
AU	Auger Sample.
DB	Diamond Bit.
CB	Carbide Bit.
WS	Washed Sample.

RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

TERM (NON-COHESIVE SOILS)	STANDARD PENETRATION RESISTANCE
Very Loose	0 - 2
Loose	2 - 4
Slightly Compact	4 - 8
Medium Dense	8 - 16
Dense	16 - 26
Very Dense	Over 26

TERM (COHESIVE SOILS)	Qu - (TSF)
Very Soft	0 - 0.25
Soft	0.25 - 0.50
Firm (Medium)	0.50 - 1.00
Stiff	1.00 - 2.00
Very Stiff	2.00 - 4.00
Hard	4.00+

PARTICLE SIZE

Boulders	8 in. +	Coarse Sand	5mm-0.6mm	Silt	0.074mm-0.005mm
Cobbles	8 in.-3 in.	Medium Sand	0.6mm-0.2mm	Clay	-0.005mm
Gravel	3 in.-5mm	Fine Sand	0.2mm-0.074mm		

RECORD OF SUBSURFACE EXPLORATION

Boring: B-1

Project Name: Proposed Building Structure
Site: Mahoning Valley Sanitary District, Niles, Ohio

Date of Boring: 03/01/04
Project No.: 139-45004

Description	Depth(ft)	Sample	N	Qu	Qp	Mc	Remarks
Surface							
Loose,moist,gray,Cinder,Slag and Gravel (FILL)		SS-1	20	--	--	12	
Loose to Medium Dense,moist,brown, Clayey Gravel with Sand and Slag (FILL)		SS-2	8	--	--	12	
	5						
		SS-3	14	--	--	11	
		SS-4	15	--	--	12	No Free Water
	10						
Soft to Firm,moist,gray,Silt and Sand (ML)		AU-5	--	--	--	19	
Hard,moist,brown,Silt and Clay,trace Gravel (ML-CL)		SS-6	49	--	4.0	9	
	15						
		SS-7	52	--	4.0	12	
End of Boring - 20.0'							

RECORD OF SUBSURFACE EXPLORATION

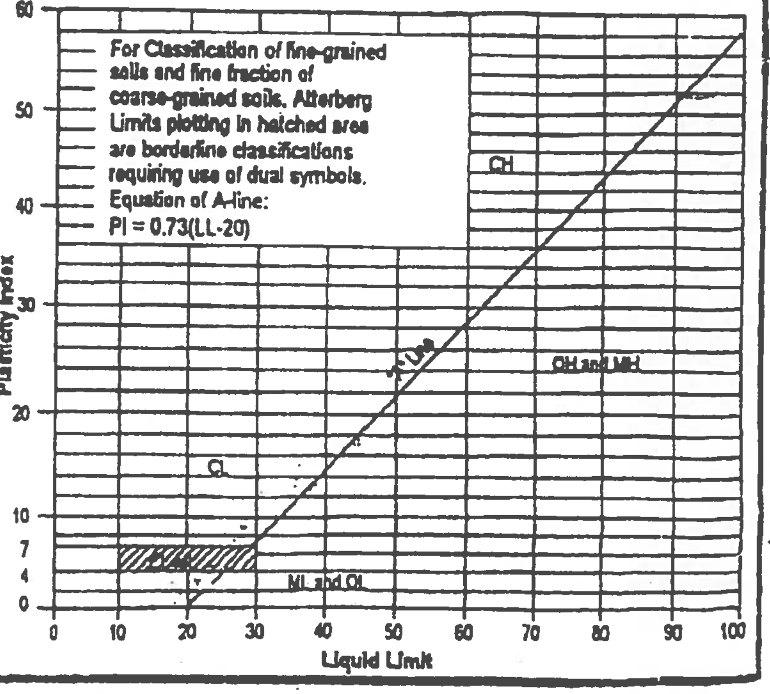
Boring: B-2

Project Name: Proposed Building Structure
Site: Mahoning Valley Sanitary District, Niles, Ohio

Date of Boring: 03/01/04
Project No.: 139-45004

Description	Depth(ft)	Sample	N	Qu	Qp	Mc	Remarks
Surface							
*							
**		AU-1	--	--	--	14	*1" Asphlat
Loose to Medium Dense,moist,brown, Clayey Gravel (FILL)		SS-2	8	--	--	11	
	5						
		SS-3	12	--	--	11	
		SS-4	11	--	--	7	No Free Water
	10						
Stiff to Very Stiff,moist to wet,brown,Sandy Silty Clay, some Gravel (ML-CL)		SS-5	13	--	1.5	24	
	15						
		SS-6	34	--	4.0	20	
End of Boring - 20.0' ** Loose,moist,black and gray,Cinder,Slag and Gravel (FILL)							

nified Soil Classification

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
(More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean Gravels (Little or no fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines	Determine percentage of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse grained soils are classified as follows: Less than 5%..........GW, GP, SW, SP More than 12%..........GM, GC, SM, SC 5 to 12%..........Borderline cases requiring dual symbols	$Cu = \frac{D_{60}}{D_{10}}$ greater than 4; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW.	
		Gravels with fines (Appreciable amount of fines)	GM	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with PI greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean Sands (Little or no fines)	SW		Well graded sands, gravelly sands, little or no fines		$Cu = \frac{D_{60}}{D_{10}}$ greater than 6; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW.	
		Sands with fines (Appreciable amount of fines)	SM	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or PI less than 4	Limits plotting in hatched zone with P1 between 4 and 7 are borderline cases requiring use of dual symbols.
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with PI greater than 7	
(More than half of material is smaller than No. 200 sieve size)	Silts and Clays (liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	PLASTICITY CHART 		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and Clays (liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt		Peat and other highly organic soils			

APPENDIX D

2012 GEOTECHNICAL INVESTIGATION BY S&ME

S&ME

SUBSURFACE INVESTIGATION
MVSD FILTER BUILDING ADDITION SETTLEMENT EVALUATION
Mineral Ridge, Ohio

S&ME Project No. 1179-12-015A
April 23, 2012

April 23, 2012

ms consultants, inc.
333 East Federal Street
Youngstown, Ohio 44503-1821

Attention: Mr. John P. Pierko, P.E.

Reference: **Subsurface Investigation**
MVSD Filter Building Addition Settlement Evaluation
Mineral Ridge, Ohio
S&ME Project No. 1179-12-015A

Dear Mr. Pierko:

In accordance with our revised proposal dated February 27, 2012, S&ME, Inc. (S&ME) has completed a geotechnical subsurface investigation in an attempt to define and determine possible geotechnical cause(s) for the cracking of the existing MVSD Filter Building addition at 1181 Ohltown McDonald Road in Mineral Ridge, Ohio. This report does not address any potential structural reasons for the noted building cracks. The project site is located as shown on the Vicinity Map presented as Plate 1 in the Appendix of this report.

PROJECT INFORMATION

Based on our conversations and our site reconnaissance with ms consultants, inc. (MS) personnel as well as our review of a recent newspaper article, S&ME understands that the existing Filter Building Addition at the MVSD Facility, which was built in 2005, has experienced cracking in its walls and floor slab. The cracking was first noticed on January 3, 2012 by MVSD personnel; however, it is uncertain as to when the cracks actually appeared. It has been noted that a magnitude 4.0 earthquake occurred in the project area on December 31, 2011. It is not known whether the cracks were already present at the time of the earthquake. At the present time, the cracks appear to be cosmetic and do not appear to be detrimental to the functionality of the structure. Plate 9 in the Appendix includes some representative photographs of cracking in the structure.

After the cracks were found, MS installed crack monitors on January 19, 2012 at several of the crack locations in an effort to determine whether the movement is ongoing. According to information provided to S&ME on April 18, 2012 by MS, the crack monitor readings have yet to show any new movement since their installation.

The existing addition experiencing cracking is reportedly supported on spread footing foundations bearing in undocumented earth fill of previously undocumented composition, density, and thickness. The addition has exterior brick walls and has overall plan dimensions of approximately 60 feet (north-south) by 30 feet (east-west). It is located 37 to 39 feet to the west of a large, shored excavation for construction of new clarifier tanks that are part of an ongoing plant upgrade. The main building, to which the addition is attached, is reportedly supported on drilled shafts bearing on bedrock and has not experienced any documented cracking/distress.

Topographic drawings prepared by MS in 2010 for the area surrounding the existing addition indicate existing site grades at the time of our investigation were generally between Elevation 889 and 890. The finished floor of the existing addition was constructed near Elevation 890.

PREVIOUS GEOTECHNICAL INFORMATION

MS supplied S&ME a log of Boring B-1 advanced by Ohio TestBor, Inc. (OTB) in 1990 for a previous project at the site. An unlabeled and undated plan of borings included with the OTB log suggests that B-1 was located to the east of the northeast corner of the addition. Boring B-1 encountered approximately 11.3 feet of silty sand and gravel with some topsoil (fill) overlying about 8 inches of buried topsoil. These were underlain by silty sand and gravel from a depth of about 12 feet to about 16.5 feet and then sandy clayey silt with gravel and rock fragments from about 16.5 to 21.5 feet. Shale bedrock extended from 21.5 feet to the 30-foot boring termination depth. Water was encountered neat a depth of 14 feet in the silty sand and gravel layer. The existing ground surface was shown to be at Elevation 887.5+/- on the OTB log.

MS also supplied S&ME a geotechnical report for the existing addition that was prepared by Professional Service Industries, Inc. (PSI) for CT Consultants, Inc. and dated March 9, 2004. A Boring Location Plan included in the PSI report suggests that they drilled their Boring B-1 near the southwest corner of the addition and their Boring B-2 near the center of the addition. Both borings extended to a depth of 20 feet. In general, PSI Boring B-1 encountered approximately 12.5 feet of loose to medium-dense clayey gravel with sand and slag (fill) overlying approximately one (1) foot of soft to firm silt and sand. Hard silt and clay was encountered from approximately 12.5 to the 20 foot boring termination depth. PSI Boring B-2 encountered loose to medium-dense clayey gravel identified as fill to a depth of about 14 feet. Stiff to very-stiff sandy silty clay was encountered from 14 feet to the 20 foot boring termination depth. Neither B-1 nor B-2 encountered free water. Existing ground surface elevations were not shown on the PSI logs. The PSI report suggests that the site grades in the area of the Filter Building addition were near Elevation 922 which is inconsistent with the elevation on the OTB log as well as our current topographic information. In the PSI report it was recommended that the footings for the new addition be proportioned for a maximum soil pressure of 2,000 psf and that a seismic site classification of C be used for design. It was also recommended that the footings bear on compacted structural fill or on suitable natural soils without further guidance as to what would constitute as suitable structural fill or suitable natural soil.

S&ME FIELD WORK

S&ME boring locations were selected and field marked at locations mutually agreed upon by S&ME, MS, and MVSD personnel. The ground surface elevations at the boring locations were estimated to the nearest approximately one (1) foot from topographic information on the 2010 MS drawings. The Ohio Utilities Protection Service (OUPS), Oil & Gas Producers Underground Protection Service (OGPUPS), and

MVSD personnel were contacted at least 48 hours prior to initiation of drilling.

On March 14, 2012, four (4) borings (denoted as B-1 through B-4) were advanced to depths ranging from 22.6 to 27.1 feet around the perimeter of the Filter Building addition footprint. General locations of the borings are shown on the Plan of Borings submitted as Plate 2 in the Appendix. The borings were advanced with a truck-mounted drilling rig using 2-1/4" I.D. hollow-stem augers. At 2-1/2 foot intervals, disturbed (but representative) soil and bedrock samples were obtained by lowering a 2-inch O.D. split-barrel sampler to the sampling depth where it was driven 18 inches in the strata encountered by blows from a 140-pound hammer freely falling 30 inches (ASTM D1586, Standard Penetration Test – SPT). The number of blows for every 6 inches of split barrel sampler advancement was recorded at each sampling interval. Split-Barrel samples were examined immediately after recovery and representative samples were preserved in air tight containers. At the completion of drilling, ground water measurements taken inside the augers and after the auger removal (when applicable) were recorded along with the depths to cave at each location.

Upon completion of SPT sampling in Boring B-3 an offset hole was performed and relatively undisturbed thin-walled (Shelby) tube samples were obtained from 8.5 to 10.5 and from 10.5 to 12.5 feet depth in a layer that was identified as fill and contained varying amounts of organics from the SPT samples.

In the field, experienced personnel provided supervision of the drilling and sampling procedures and performed the following specific duties: assumed responsibility for handling and preserving all samples after they were recovered; prepared a field log of each boring; made seepage and groundwater observations during and after the completion of drilling; backfilled each boring and repaired the surrounding ground surface to the best of our abilities; and provided close liaison with our Project Engineer so that the program of exploration could be effectively modified, if required, due to unanticipated conditions.

S&ME LABORATORY TESTING

In the laboratory, under the direction of a Professional Engineer, all samples obtained by S&ME were visually identified, and select samples were tested for their moisture content (ASTM D 2216) and/or liquid and plastic (Atterberg) limits. Two Shelby tube samples were extruded, their soils were visually identified, and moisture content tests were performed on selected portions of the tubes. Five (5) loss-on-ignition (LOI) tests were performed on selected SPT and Shelby tube samples.

Based on the results of the laboratory identification and testing process, soil and bedrock descriptions on the field logs were modified, where necessary, and copies of the laboratory-corrected logs of the borings have been submitted as Plates 4 through 7 in the Appendix. Shown on these logs are: measured thicknesses of the topsoil or concrete surrounding the addition, descriptions of the soil and bedrock stratigraphy encountered; depths from which samples were preserved; sampling efforts (blow-counts) required to obtain the specimens in the borings; and, seepage and groundwater observations.

Soils described in this report have been classified in general accordance with the Unified Soil Classification System, augmented by the use of adjectives to designate the approximate percentages of minor soil components. Definitions of these adjectives and an explanation of the notes and symbols used on the boring logs are presented on Plate 3 in the Appendix. Also, logs of the Shelby tube samples have been submitted as Plate 8 in the Appendix.

GENERAL SUBSURFACE CONDITIONS

Approximately 2 inches of topsoil or topsoil and rootmat were encountered at the ground surface in S&ME Borings B-1 and B-4. Concrete pavement approximately 6 inches thick was encountered beginning at the ground surface in Boring B-2.

For the purposes of this report, soil is labeled as fill if there is clear evidence of man-made or man-placed materials. It is labeled as probable fill if there is some evidence of man-made or man-placed materials encountered. Soil is called possible fill if the coloring, texture, or bedding does not appear natural. In general, soils identified as fill, probable fill, or possible fill and consisting of stiff to hard silty clay or clayey silt with varying amounts of sand, gravel, and/or rock fragments were encountered in each of the borings beginning beneath the topsoil/pavement (or beginning at the ground surface in Boring B-3) and continuing to depths ranging from about 13 to 18 feet below the ground surface in all four borings. Exceptions were noted in Borings B-2 through B-4 were portions of the subsurface profile ranging in thickness from about 2.5 feet to 10 feet had consistencies varied from very-soft to medium-stiff and were moist to wet. In addition, soils identified as possible buried topsoil or soils that were described as being slightly organic were encountered in Borings B-2 through B-4 as well.

Underlying the fill, probable fill, and possible fill, and extending to depths ranging from about the 20.5 to 23.5 feet were natural soils comprised of hard brown and/or gray clayey silt with varying percentages of sand and rock fragments. The clayey silt was underlain by very-soft to soft shale to the boring termination depths.

Groundwater and seepage observations were made during drilling as each boring was advanced and upon completion of drilling. Neither seepage nor groundwater was encountered in Borings B-1, B-2, or B-4. In B-3 water was encountered at a depth of about 16.5 feet during drilling and was at a depth of about 22.3 feet upon completion of drilling. All of the boreholes caved at depths ranging from about 19 to 22.5 feet upon completion of drilling and sampling and removal of the augers from the ground.

Please refer to the individual boring logs for a summary of the soil, bedrock and groundwater/seepage conditions encountered at each boring location. Inferences should not be made to the subsurface conditions in the areas between or away from the borings without field verification. Groundwater levels can fluctuate from those encountered on the boring logs with seasonal changes in precipitation.

DISCUSSION AND CONCLUSIONS

Geotechnical Evaluation

Using the information found in this subsurface investigation and the existing topographic and other project information provided by MS, S&ME believes that the possible geotechnical reasons which could cause the building to settle and crack could include the following mechanisms:

a. Settlement of the uncontrolled fill beneath the addition,
b. Excavation of the new clarifier tank adjacent to the building,
c. Drawdown of groundwater due to the construction of the nearby clarifiers,
d. Heave due to expansive soils.
e. Settlement of the fill soils due to the recent earthquake.

Uncontrolled fill was encountered to depths ranging from 13.5 to 18.0 feet below grade, and consisted of uncontrolled cohesive soils containing organic soils and some sand and gravel. These conditions typically would not be considered suitable for the support of a masonry building using shallow foundations due to the potential risk of total and differential settlements which could cause cracking in masonry walls and floor slab. The conditions found in this investigation varied considerably in soil type and exhibited lower blowcounts than was previously determined in the PSI report. It is highly probable that settlement of the existing fill resulting from the self weight and the weight of new fill and loads of spread foundations is the primary cause of the cracks, and that the earthquake prompted the owner to recently look for and record the existence of the cracks.

The distance between the shored excavation for the new clarifier tanks and the existing building addition is about 37 feet. The depth to competent soils consisting of hard clayey silt and shale bedrock in Boring B-3 is approximately 13 feet below the anticipated bearing elevation of the existing footings. The influence of foundation loads typically remain in a zone beneath the footing which extends downward at an angle of 2:1 vertical to horizontal from the bottom edge of spread footing. Therefore the distance from the clarifier excavation to the Filter Building addition make it unlikely that the excavation caused movement which would cause slab and building cracking.

The borings indicated that a limited volume of groundwater exists which is not consistent enough to establish that the fill is saturated with a defined groundwater level. The limited groundwater found in the borings appears to have been confined to within relatively thin granular soil layers interbedded within the natural clayey soil stratum. Based upon the relatively small layer thickness and the medium-dense relative density of the granular layers encountered, it is unlikely that draw-down of the water table from the construction of the nearby clarifiers has had any effect on settlement of the building addition.

An indicator of the heave potential of soil is predicted when site soils have a high plastic limit value as determined by a laboratory test. Atterberg (liquid and plastic) limits and water content tests performed on selected representative samples of the fill encountered suggest that there is a low probability that the on-site silty clay and clayey silt soils are expansive in nature. Therefore, we do not believe that heave of the fill soils is the cause of the cracking.

It was reported to S&ME that the existing building addition is supported on conventional spread footings bearing near a depth of about 4 feet below existing site grades surrounding the addition. Based on the subsurface stratigraphy encountered from the four (4) recent S&ME borings, the known depth to bedrock, and the SPT blow-counts encountered in Boring B-3 which was the "worst case" boring, it is our engineering opinion that seismic site class D would be applicable to the area of the building addition as characterized by the 2007 Ohio Building Code. Note that a seismic site class of C was provided for design in the March 9, 2004 geotechnical report prepared by PSI.

It is possible that the magnitude 4.0 earthquake could have resulted in settlement of the existing supporting fill soils, however, not probable as cohesive fill soils would be less apt to settle than loose granular fill soils with a lower level earthquake.

Based on the PSI recommendations, it is likely that the Filter Building addition was designed using a seismic site class C. Our seismic evaluation resulted in a recommendation to use a seismic site class D. Therefore, we recommend that a structural engineer determine the associated affects to the structure, if any, resulting in a change of seismic site class from C to D.

FINAL CONSIDERATIONS

The analyses and conclusions discussed in this report are based on conditions as they exist at the time of our field investigation and further on the consideration that the exploratory borings are representative of subsurface conditions throughout the area investigated. Actual subsurface conditions between and away from the borings might differ from those encountered at the boring locations. If subsurface conditions are observed that vary from those discussed in this report, S&ME should be notified immediately so that we may evaluate the effects, if any, on our engineering evaluation and conclusions.

S&ME appreciates the opportunity to be of service on this project. If there are any questions concerning the findings or conclusions included within this report, please contact us at your convenience.

Respectfully submitted,
S&ME, Inc. – Cleveland, Ohio

ERIC A. ANGYAL
61252
4/23/12

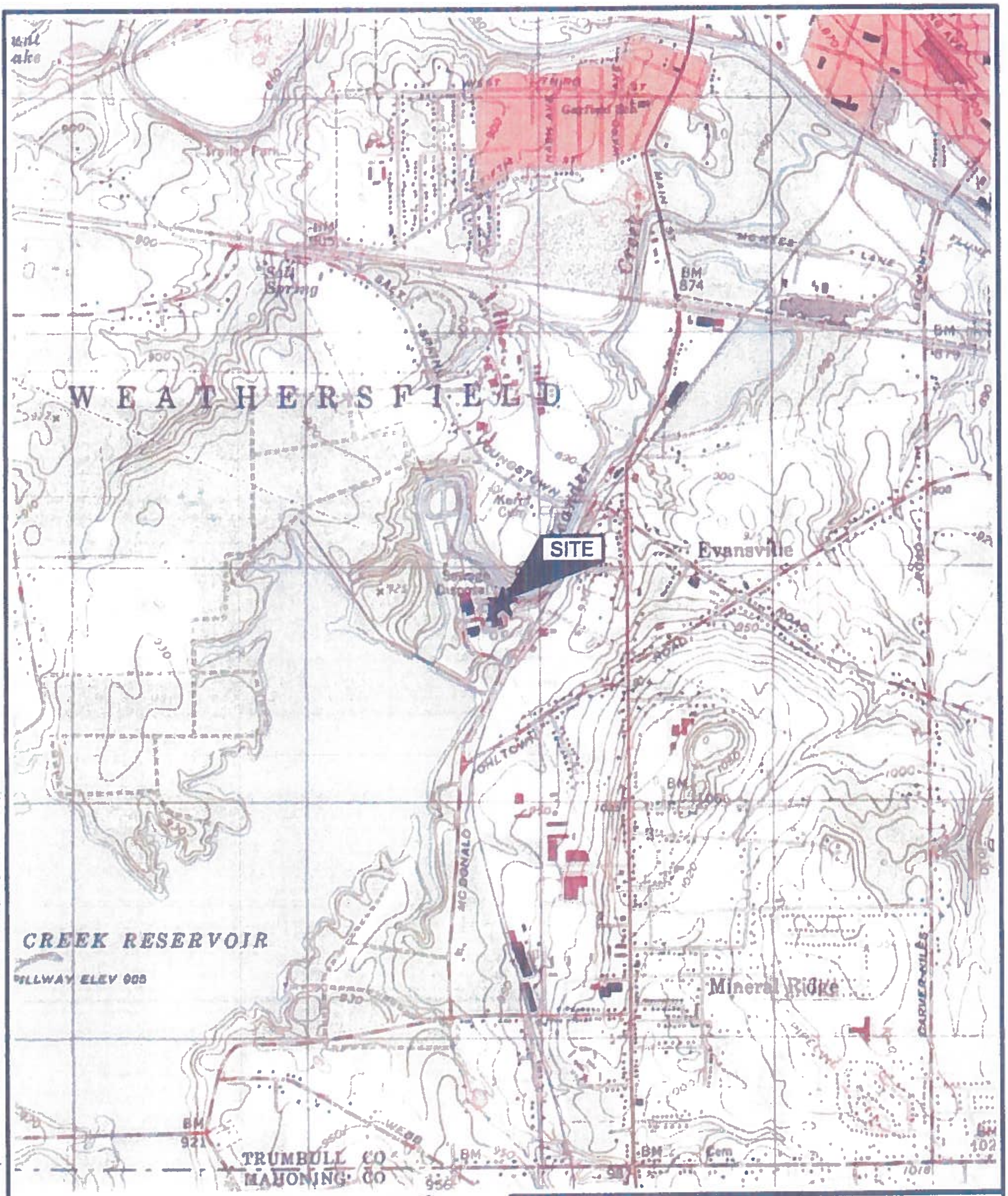
Eric A. Angyal, P.E.
Senior Engineer

for
Stephen C. Pasternack, P.E.
Senior Reviewer

Attachments: Appendix (Plates 1 through 9)

Submitted: One copy via email in PDF format
Three (3) copies via U.S. Mail

Appendix



USGS Mapping.
Warren, Ohio

0 2000 4000
SCALE IN FEET

VICINITY MAP

MVSD FILTER BUILDING
MINERAL RIDGE, TRUMBULL COUNTY, OHIO

Project: 1179-12-015A
Drawing Date: 3-19-2012
Last Updated: 3-19-2012
Drawn By: KJD
Approved By: EAA
Scale: 1:2000 1:1

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ENGINEERING LICENSE NO: F-0176

PLATE 1

Images: - TOPO-map.JPG
Xrefs:
File Last Updated: Mar 19, 2012 @ 11:14am By: KDohlen
Plot Info: 3-19-2012 @ 11:14am By: KDohlen
S&ME Filename: I:\DEPTS\cadd\Drawings\BST Projects\1179-Cleveland\1179-12-015A\PLANS\VMap.dwg Layout: V-Map

LEGEND

B-1 BORING NUMBER AND LOCATION

Plan of Borings

MVSD FILTER BUILDING
MINERAL RIDGE, TRUMBULL COUNTY, OHIO

Project:	1179-12-15A
Drawing Date:	3-19-2012
Last Updated:	4-18-2012
Drawn By:	KJD
Approved By:	EAA
Scale: 1:50	1:1

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ENGINEERING LICENSE NO: F-0176

PLATE 2

Images: ~Aerial Photo.jpg
Xrefs:
File Last Updated: Apr 18, 2012
Plot Info: 4-18-2012 @ 9:03am By: KDohlen
S&ME Filename: I:\DEPTS\cadd\Drawings\BST Projects\1179-Cleveland\1179-12-015\PLANS\POB.dwg Layout: 8.5x11P

Page 1 of 1

LOG OF BORING NO. B-2
MVSD FILTER BUILDING
MINERAL RIDGE, OHIO

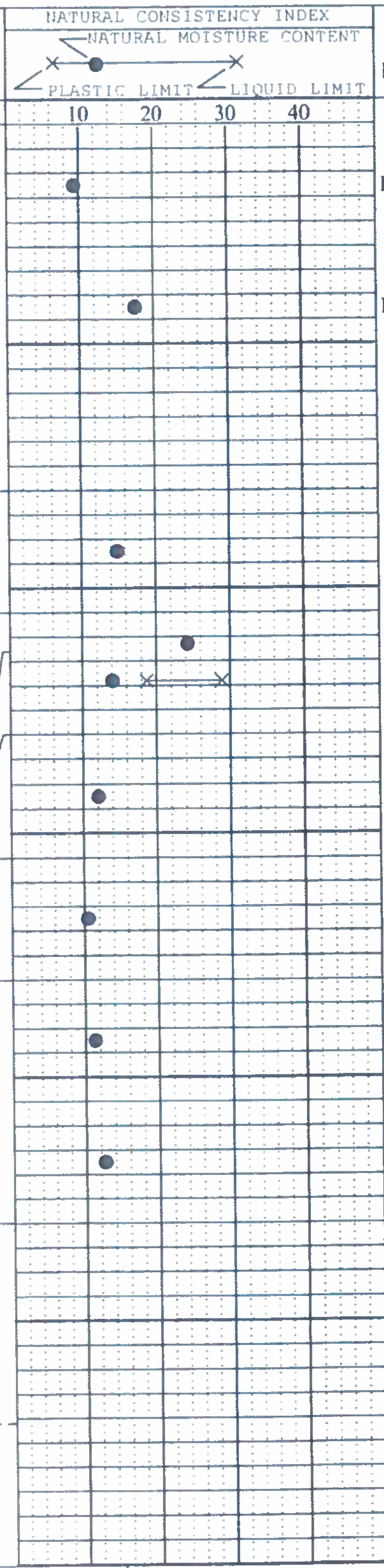
S&ME

LOCATION: **As Shown on Plan of Borings** ELEVATION: **890** DATE: **3/14/12**

DRILLING METHOD: **2-1/4" I.D. Hollow-stem Auger** COMPLETION DEPTH: **27.1'**

SAMPLER(S): **2" O.D. Split-barrel Sampler**

ELEV.	DEPTH, FEET	SAMPLE NUMBER	SAMPLE EFFORT	N₆₀	SAMPLE REC-%	DESCRIPTION	TEST RESULTS
889.5	0					CONCRETE - 6 INCHES	
		1	3/2/5	9	33	FILL: Stiff brown clayey silt, some fine to coarse sand, trace fine to coarse gravel; trace slag and cinders, damp to moist.	H=1.0
		2	4/3/2	7	60		H=1.0-1.5
	5		16/13/6	26	0		
882.0		3	4/5/4	12	67	FILL: Very-stiff brown and gray clayey silt, some fine to coarse sand, some fine to coarse gravel, damp.	H=2.0-3.5
879.5	10	4A	2/3/3	8	100	FILL: Very-soft brown silty clay; contains many organics (possible buried topsoil), moist. - LOI = 6.32% at 11.2'.	H=0.0
878.7		4B			67		H=0.0
877.0						FILL: Very-soft brown clayey silt, some fine to coarse gravel, some fine to coarse sand, moist.	
		5	8/6/7	18	53	POSSIBLE FILL: Stiff brown clayey silt, some fine to coarse sand, some fine to coarse gravel, moist.	H=1.5
874.5	15	6	12/17/16	45	67	POSSIBLE FILL: Hard brown clayey silt, some fine to coarse sand, contains many rock fragments, damp.	H=4.0
872.0		7	7/12/19	42	80	Hard brown becoming gray clayey silt, some fine to coarse sand, trace fine to coarse gravel; contains few rock fragments and many shale fragments at 21', damp.	H=4.0-4.5
	20	8	15/22/26	65	80		H=4.5+
867.0		9	12/50-4"R		100	Very-soft severely weathered gray shale, dry.	H=4.5+
	25						
862.9		10	50-1"R		100		
	30						



NOTES:
- No seepage encountered during drilling.

WATER LEVEL: ▽ "Dry" ▼
WATER NOTE: Caved at 19.0'
DATE: 3/14/12

SYMBOLS USED TO INDICATE TEST RESULTS

G - Gradation
Q - Uncon Comp
T - Triax Comp
C - Consol.
See Separate Curves

H - Penetrometer (tsf)
W - Unit Dry Wt (pcf)
D - Relative Dens (%)

Drill Rod Energy Ratio : 0.81
Last Calibration Date : 08/28/10
Drill Rig Number : RW ATV 550

2010 NEW DEFAULT BORING LOG-W/ N60 1179-12-015.GPJ BBCM.GDT 4/23/12

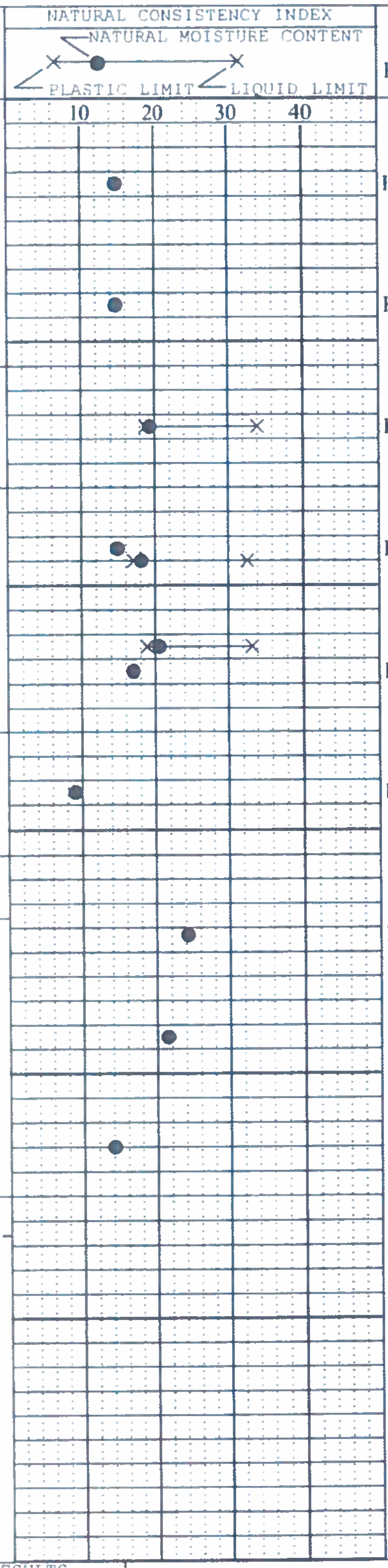
JOB: 1179-12-015A

PLATE 5

Page 1 of 1

LOG OF BORING NO. B-3
MVSD FILTER BUILDING
MINERAL RIDGE, OHIO

LOCATION: **As Shown on Plan of Borings**
ELEVATION: **889**
DATE: **3/14/12**
DRILLING METHOD: **2-1/4" I.D. Hollow-stem Auger**
COMPLETION DEPTH: **23.3'**
SAMPLER(S): **2" O.D. Split-barrel Sampler**



ELEV.	DEPTH, FEET	SAMPLE NUMBER	SAMPLE EFFORT	N_{60}	SAMPLE REC-%	DESCRIPTION	TEST RESULTS
	0					FILL: Very-stiff brown clayey silt, trace fine to coarse sand, trace fine to coarse gravel, damp.	
		1	7/6/5	15	33		H=3.5
		2	5/4/4	11	33		H=3.5
883.5	5						
		3	SH/2/1	4	33	PROBABLE FILL: Stiff dark brown silty clay, trace fine to coarse gravel, trace fine to coarse sand, moist.	H=1.0
881.0							
		4	1/2/1	4	100	PROBABLE FILL: Very-soft to medium-stiff brown, dark gray and gray silty clay, some fine to coarse sand, trace fine rounded sand; contains few rock fragments; possible buried topsoil inclusions from 9' to 12', moist to wet. LOI = 2.35% at 9'. LOI = 3.1% at 11'.	H=0.5-0.7
	10	11	P		65		
		12	P		65		
		5	1/2/2	5	100		H=0.0-0.2
876.0							
		6	8/5/11	22	53	Hard brown clayey silt, some fine to coarse sand, some fine to coarse gravel; contains many rock fragments, dry to damp.	H=4.0
873.5	15						
		▽7A	10/12/7	26	100	Medium-dense brown fine to coarse sand, trace fine to coarse gravel, wet.	
872.2							
		7B			71	Hard brown mottled with gray becoming brown clayey silt, trace fine to coarse sand, damp to moist.	H=4.5
		8	8/10/15	34	100		H=4.5
	20						
		9	15/50-6"R		100		H=4.5+
866.5		▼					
		10	50-3"R		100	Very-soft gray shale, highly weathered.	
865.7							
	25						
	30						

NOTES:
- Encountered water at 16.5' during drilling.
- Samples 11 and 12 were obtained from an offset borehole 4' southeast of B-3.

	▽	▼
WATER LEVEL:	16.5	22.3
WATER NOTE:	Encountered	Caved at 22.5'
DATE:	3/14/12	3/14/12

SYMBOLS USED TO INDICATE TEST RESULTS
G - Gradation
Q - Uncon Comp
T - Triax Comp
C - Consol.
See Separate Curves
H - Penetrometer (tsf)
W - Unit Dry Wt (pcf)
D - Relative Dens (%)

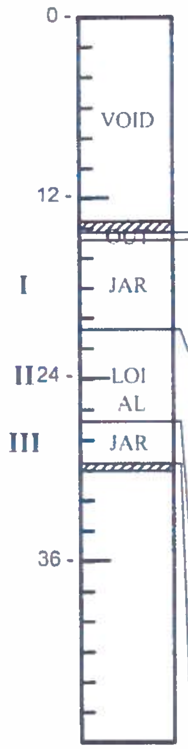
Drill Rod Energy Ratio : 0.81
Last Calibration Date : 08/28/10
Drill Rig Number : RW ATV 550

JOB NUMBER : **1179-12-015A**
PROJECT : **MVSD FILTER BUILDING**
LOCATION : **MINERAL RIDGE, OHIO**

S&ME

LABORATORY LOG OF SHELBY TUBES

Boring : **B-3A** Sample : **S-11**
Depth : **8.5' to 9.8'** Recovery : **16"**



FILL: Soft to stiff brown intermixed with gray silty clay, little fine to coarse sand, trace fine gravel.
H=0.4-1.2

FILL: Soft to stiff brown intermixed with gray silty clay, trace fine to coarse sand, trace fine gravel, contains few pockets of topsoil.
- LOI = 2.35%
H=0.4-1.2

FILL: Soft to medium-stiff brown intermixed with gray silty clay, trace fine to coarse sand, trace fine gravel, contains few pockets of topsoil, contains few slag fragments.
H=0.3-0.8

36" tube

Boring : **B-3A** Sample : **S-12**
Depth : **10.5' to 11.8'** Recovery : **15"**

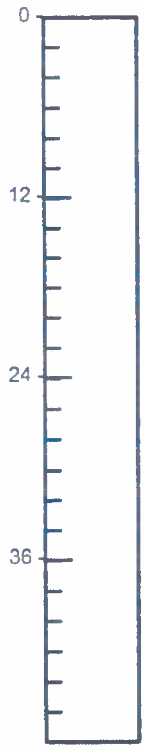
FILL: Soft to medium-stiff brown intermixed with gray silty clay, little fine to coarse sand, trace fine gravel, contains few black organic pockets.
H=0.3-0.8

FILL: Soft to medium-stiff brown intermixed with gray silty clay, little fine to coarse sand, trace fine gravel, contains few black organic pockets.
- LOI = 3.1%
H=0.4-0.6

FILL: Fine to coarse limestone gravel intermixed with brown silty clay.

36" tube

Boring : Sample :
Depth : Recovery :



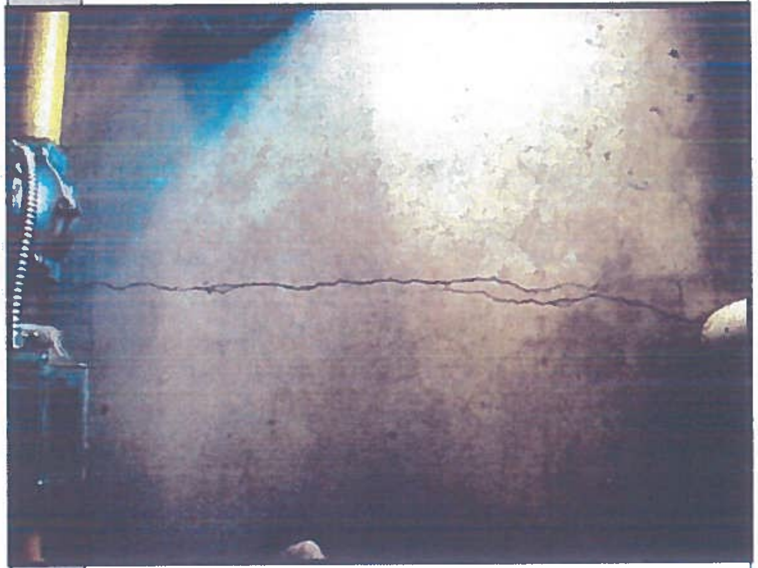
LEGEND

■ - Consolidation, Incremental	☐ Swelling, Test	▨ - Wax	H - Hand Penetrometer (tsf)	TV - Torvane (tsf)
C R S - Consolidation, C R S	◿ - Unconfined Compression Test	☒ - Triaxial Compression Test	Ds - Direct Shear	POR - Porosity
P P - Permeability, Vertical / Horizontal			LOI - Loss on Ignition	UDW - Unit Dry Weight
			AL - Atterberg Limits	MC - Moisture Content
			MA - Sieve/Hydrometer	D_R - Relative Density
			SG - Specific Gravity	S - Sieve

1 Interior Wall Cracking

2 Interior Wall Cracking

3 Exterior Wall Cracking



4 Floor Slab Cracking

S&ME

Mahoning Valley Sewer District
Mineral Ridge, Mahoning County, Ohio

S&ME Project #1179-12-015A

Taken by: EAA

Date Taken: 1/6/2012

PLATE 9

APPENDIX E

"SEISMIC SITE CLASSIFICATION FOR STRUCTURE ENGINEERS"

Seismic Site Classification for Structural Engineers

by Dominic Kelly

Many states and municipalities have adopted the International Building Code (IBC) and, by reference, the seismic provisions in *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02 and ASCE 7-05). As engineers use these documents, they are beginning to realize how dependent the magnitude of the design earthquake force is on the site class. In seismic provisions of previous model building codes other than the 1997 UBC, the soil type impacted the force level for mid-rise and high-rise buildings, but generally did not affect the seismic design force for low-rise buildings. The site classes in the IBC, ASCE 7-02, and ASCE 7-05 directly impact the seismic design force for all buildings, whether a low-rise or high-rise building. In regions of low or moderate seismicity, a difference in site class may change the seismic design category (SDC), resulting in a difference in design and detailing requirements.

Substantial differences in seismic design force and detailing requirements based on the site class are consistent with observed earthquake damage. Typically, buildings on soft or loose soils sustain substantially more damage than comparable buildings on stiff soil or rock sites.

Although geotechnical engineers typically classify sites, understanding how sites are classified is valuable to a structural engineer's practice. A structural engineer can check a geotechnical engineer's classification, counsel clients when additional work is advisable to classify a site less conservatively, and classify a site for additions when adequate information is available from existing borings.

Basis for Site Classification

The source document for the site classifications defined in the IBC, ASCE 7-02, and ASCE 7-05 is *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 450). Information regarding the basis for the site classifications is provided in its commentary. The commentary describes how soil deposits amplify the level of ground shaking relative to the level of shaking of bedrock. The amount of ground-motion amplification depends on wave-propagation characteristics of the soils, which can be estimated from the measurements of the shear-wave velocity. Soft soils with slower shear-wave velocities generally produce greater amplification than stiff soils with faster shear-wave velocities. The site classes of the IBC, ASCE 7-02, and ASCE 7-05 are defined in terms of shear wave velocity.

Table 1 – Site Class Definitions from ASCE 7-02 and ASCE 7-05

Site Class	Site Profile Name	Soil Shear Wave Velocity, v_s(ft/sec)	Standard Penetration Resistance, $\bar{N}$ or Nch	Undrained Shear Strength, S_u (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	NA	NA
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	NA	NA
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	> 50	> 2,000 psf
D	Stiff soil	$600 < \bar{v}_s \le 1{,}200$	15 to 20	1,000 to 2,000 psf
E	Soft clay soil	$\bar{v}_s \le 600$	<15	<1,000psf
		Any profile with more than 10 ft of soil having the following characteristics: • Plasticity index PI > 20 • Moisture content $w \ge 40\%$, and • Undrained shear strength $S_u < 500$ psf		
F	Soil requires site response analysis	Liquefiable soils, peat, high plasticity clay		

Site Class Definitions

The IBC and ASCE 7-02, and ASCE 7-05 define six site classes, Site Class A to Site Class F, based on the upper 100 feet of soil and rock from the base of a building. Base is defined as "the level at which the horizontal seismic ground motions are considered to be imparted to the structure." In almost all cases, the base is the ground level of the building. The commentary to FEMA 450 states: "Conversely, for structures with basements supported on firm soils or rock below soft soils, it is reasonable to classify the site on the basis of the soils for rock below the mat, if it can be justified that the soft soils contribute very little to the response of the structure." In taking advantage of this, consideration needs to be given as to whether the ground level should be treated as an elevated level, which would increase the seismic force on the building.

Descriptions of the site classes defined in ASCE 7-02 and ASCE 7-05 are provided in *Table 1*, along with the definition in terms of shear wave velocity.

Classifying Rock Sites

Site Classes A and B are rock sites, and Site Class C is sometimes assigned to rock sites. Site Class A sites, hard rock, are generally east of the Rocky Mountains. Measured shear wave velocities at the site, or at a site of the same rock type with similar or less severe weathering and fractures, are required to assign Site Class A. Competent rock sites in the West Coast are typically Site Class B. Site Class B may be assigned to any competent rock site with moderate fracturing and weathering, based on either measured or estimated shear wave velocities. Soft rock and highly fractured and weathered rock must be assigned Site Class C, unless measured shear wave velocities demonstrate that Site Class B is applicable. If there is more than 10 feet of soil between the rock surface and the bottom of the spread footing or mat foundation, Site Classes A and B shall not be assigned to the site.

continued on next page

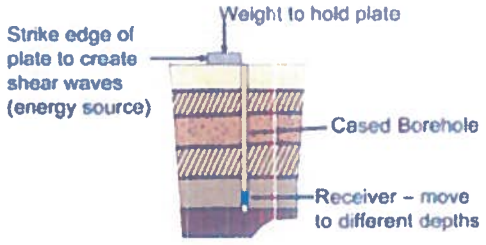


Figure 1 Seismic Down-Hole Test

Classifying Soil Sites

Although the site class descriptions are for a single type of soil or rock type, most sites consist of multiple layers of soil and rock. In classifying a site, all soil and rock layers in the upper 100 feet of the site profile are considered. Sites consisting predominately of very dense glacial tills, sands, gravels, and soil sites with very shallow rock often qualify as Site Class C. When shallow foundations are allowed for a building on a soil site, Site Classes C and D are generally applicable, with Site Class D being more common. When deep foundations are required, the applicable site class is generally Site Class E, though some sites with relatively shallow deep foundation elements, on the order of 30 feet or less, will sometimes qualify as Site Class D. When a site has soils susceptible to collapse during an earthquake, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils, Site Class F is applicable. Site Class F requires a site specific response spectrum analysis to assess the ground motion amplification of the site, except when the fundamental periods of the building are less than 0.5 seconds and the presence of liquefiable soils is the reason for the assigning Site Class F. For a default site class, ASCE 7-02 and ASCE 7-05 state: "Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site."

When competent rock is encountered before reaching the bottom of the upper 100 feet of site profile, it is usually acceptable to treat the remainder of the profile the same as the first encountered competent rock. A rare exception to this is for sites with geologically recent volcanic rock that is over soil. Such sites exist in Hawaii.

ASCE 7-02 and ASCE 7-05 include three procedures to assign Site Classes C, D, and E based on the following:

- Average shear wave velocity
- Average field standard penetration resistance
- Average undrained shear strength

Average Shear Wave Velocity Procedure

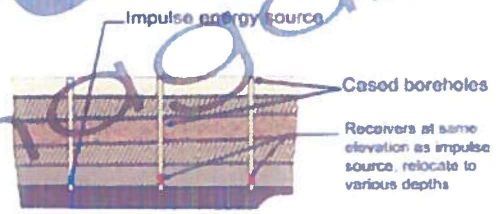
The most accurate site classification is obtained using the average shear wave velocity procedure, because the site classes are defined based on measured shear wave velocity. A weighted average shear wave velocity is used to account for the greater site amplification that occurs in relatively softer or looser soils with slower shear wave velocities. The weighted average shear wave velocity is obtained using the following:

$$\frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

d_i is the thickness of any layer between 0 and 100 feet.

v_{si} is the shear wave velocity in feet/second

$\sum_{i=1}^{n} d_i$ is equal to 100 feet.



Dropping the weight on the rod that drives the split spoon sampler can serve as an energy source that creates shear waves

Figure 2 Seismic Cross-Hole Test

Measuring shear wave velocity adds cost to a geotechnical investigation, so most sites are classified using one of the other two procedures. However, the greater accuracy of the average shear wave velocity procedure over that of the average field standard penetration resistance, or average undrained shear strength procedure, can be worth the added cost if its use results in a different site classification. The more favorable site classification will lead to lower design forces, and perhaps even a less severe seismic design category. The resulting savings in cost of a building's structure, and perhaps savings in the anchorage and bracing of architectural and mechanical components, can far exceed the costs required to measure the shear wave velocity.

Four tests are available for measuring shear wave velocity for the purposes of classifying a site:

- Seismic Down-Hole Test
- Seismic Cone Test
- Seismic Cross-Hole Test
- Surface Wave Tests

The seismic down-hole test is the most common test for measuring shear wave velocity for the purposes of classifying a site. It requires a single cased borehole, an impulse energy source at the surface, and a movable receiver or a string of receivers as represented in *Figure 1*. The travel time of shear waves is measured to various depths, and a travel time versus depth curve is generated. Interpretation of the speed is more difficult when ground water is present, but it does not preclude the use of the down-hole test.

The seismic cone test is similar to the seismic down-hole test. A receiver is placed above the friction sleeve of a conventional cone penetrometer. At various depths, penetration is momentarily stopped and an impulse is generated at the surface. The travel time of shear waves is measured to various depths, and a travel time versus depth curve is generated. Cone penetrometers cannot be advanced through very stiff and very dense soil layers, or through gravels and boulders, without damaging the equipment. In areas that have a prevalence of these soil types, such as the Northeast, the seismic cone test is not used extensively.

The seismic cross-hole test is the second most commonly used test for measuring shear wave velocity. It requires a minimum of two and preferably three or more boreholes, an impulse energy source within a borehole, and receivers at the other boreholes as represented in *Figure 2*. Because multiple boreholes are required, a seismic cross-hole test is more expensive than a seismic down-hole test. Seismic cross-hole tests are generally used to measure shear wave velocities for sites with critical facilities, such as nuclear power plants or large dams; they are rarely used for typical building sites. The impulse energy source and receivers are set at the same elevation, and the shear wave velocity is measured at that elevation. By measuring the shear wave velocity at multiple elevations, a shear wave velocity profile can be generated. Care must be taken to avoid over-estimating the shear-wave velocity of a soft or loose layer adjacent to a stiff layer. In this case, the shear-waves can travel from the soft layer to the stiffer layer and back into the soft layer, arriving at the receiver faster than the shear waves that travel directly through the soft layer.

Several surface wave tests are also available to measure shear wave velocity. The most common surface wave test is the spectral analysis of surface waves (SASW) test represented in *Figure 3*. These tests require an impulse

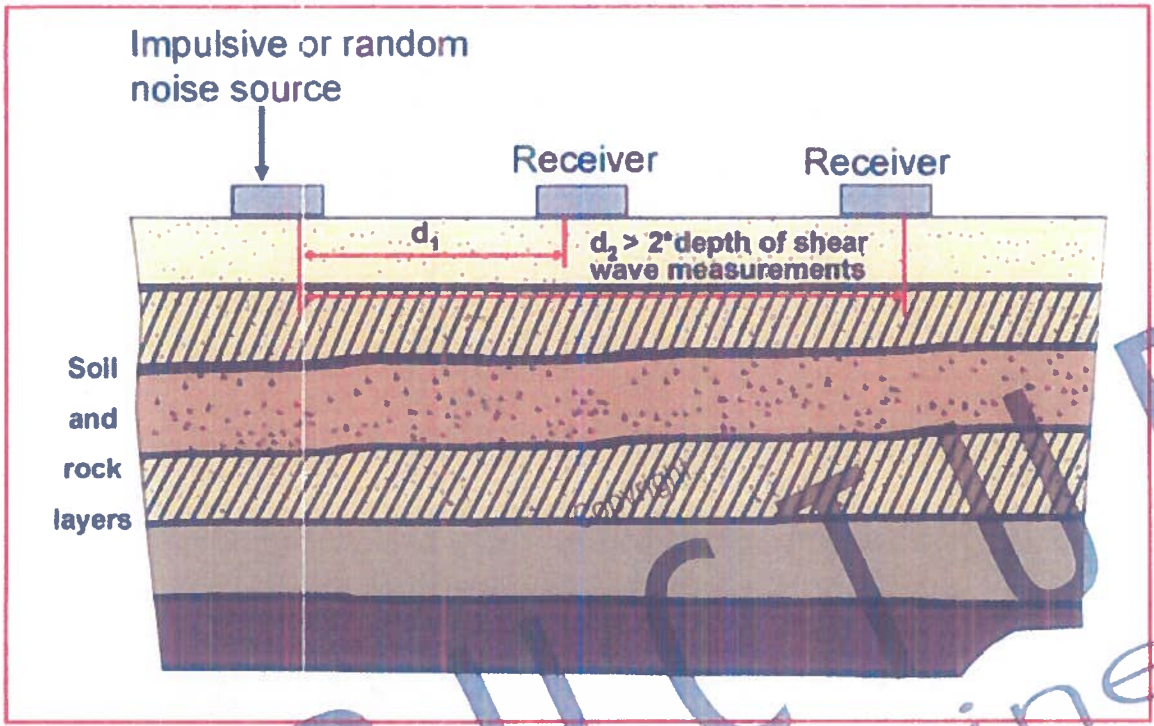


Figure 3 Spectral Analysis of Surface Waves Test (SASW)

source and receivers on the surface. These tests are good for locations where boreholes cannot be drilled. There are some limitations as to the sites where these tests can be performed. The tests are difficult to perform in an urban environment, although it may be possible in a large basement of a building. To obtain accurate shear wave velocities, the equipment should only be used by operators with expertise in the test method. At this time, there is limited commercial availability of these tests, and they have generally been used at critical facilities. However, surface wave tests are inexpensive and hold great promise for classifying sites in the future.

Average Standard Penetration Resistance Procedure

The most commonly used procedure for classifying a site is the standard penetration resistance procedure. This procedure requires little or no additional field investigation than geotechnical engineers typically provided in the past. Generally, only one boring is extended to a depth of 100 feet and the other borings are extended to depths as required to make foundation support recommendations.

This procedure is, by design, conservative because the correlation between site amplification and standard penetration resistance is more uncertain than the correlation between site amplification and shear wave velocity. It is most conservative for sites with substantial layers of cohesive soils.

A weighted average standard penetration resistance is used to account for the greater site amplification that occurs in softer or loose soils. The weighted average standard penetration resistance is obtained using the following:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

d_i is the thickness of any layer between 0 and 100 feet.

N_i is the standard penetration resistance blows/foot for the layer.

$\sum_{i=1}^{n} d_i$ is equal to 100 feet.

The standard penetration resistance procedure is presented in ASCE 7-02 in a manner that has led some engineers to conservatively exclude rock layers within the upper 100 feet of the site profile. Excluding the rock can lead to a less favorable and unrealistic site classification. ASCE 7-05 is clear that all layers within the upper 100 feet are included. When rock or very stiff soil layers with N_i greater than 100 blows/foot are encountered, N_i for those layers is to be taken as 100 blows/foot.

ASCE 7-05 states the standard penetration resistance is to be as directly measured in the field without corrections. Based on comments made by Lawrence F. Johnsen of Heller and Johnsen, Francis Leathers of GEI Consultants, and Kenneth Stokoe of the University of Texas at Austin, the author believes the intent is to not correct field measured standard penetration resistance for soil overburden pressures, but to normalize standard penetration resistance to a hammer energy ratio of 60%. The rope and cathead safety hammers have energy ratios of 55 to 65%. Energy ratios for the old fashioned donut hammers are 35 to 45%, and the energy ratio for newer automatic safety hammers varies by manufacturer with typical ratios being 75 to 85%. (*Refer to sidebar, "Standard Penetration Test Energy Measurements" by Lawrence F. Johnsen for a description on how Standard Penetration test energies are measured.*) The author understands that ASCE 7 Seismic Task Committee will address correcting field measured standard penetration resistances for hammer energy as new business for the next version of ASCE 7.

Average Undrained Shear Strength

This procedure is, by design, conservative because the correlation between site amplification and undrained shear strength is more uncertain than the correlation between site amplification and shear wave velocity. However, for sites with substantial deposits of cohesive soils, this procedure is generally less conservative than the average standard penetration resistance method.

In using the average undrained shear strength procedure, the cohesive and cohesionless soils must be treated separately. Thus two formulae must be used, and two site classes must be determined.

For the cohesive soil layers, the weighted average undrained shear strength is obtained using the following:

$$\bar{S}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{S_{ui}}}$$

$d_c = \sum_{i=1}^{k} d_i$, where k is the number of cohesive soil layers and d_c is the total thickness of cohesive soil layers.

S_{ui} is the undrained shear strength in psf of a cohesive layer, not to exceed 5,000 psf.

The weighted average undrained shear strength of the cohesive soil layers is used with the criteria in *Table 1* to assign a site classification.

For the cohesionless soil layers, the weighted average standard penetration resistance is obtained using the following:

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

$d_s = \sum_{i=1}^{m} d_i$, where m is the number of cohesionless soil layers and d_s is the total thickness of cohesionless soil layers.

N_i is the standard penetration resistance blows per foot for the cohesionless soil layers.

The weighted average standard penetration resistance is compared with the criteria in *Table 1* to assign a site classification for the cohesionless soil layers.

The site classifications for the cohesive and cohesionless soil layers are compared and the classification with the lower shear wave velocity is assigned to the site. This procedure can be very conservative when substantial rock is in the 100 foot profile, the cohesive soil layers are soft and not very thick, or when the cohesionless layers are loose and not very thick.

Summary

With an understanding of how site classes are assigned, structural engineers will be in a position to make valuable recommendations to their clients. Because the structural engineer determines seismic design forces and establishes the seismic design category, he or she is in a better position than the geotechnical engineer to identify how significant an impact the site classification makes on the design and cost of the building. If the structural engineer recognizes that a change in site class to the next class with a higher shear wave velocity will change the seismic design category or significantly lower the design force, he or she can advise their client to undertake additional geotechnical investigations, such as classifying the site using shear wave velocities instead of using the standard penetration resistances.▪

Dominic Kelly is a Staff Consultant with Simpson Gumpertz & Heger Inc. in Waltham, MA and a member of ASCE 7 Seismic Task Committee for the 2002 and 2005 versions of ASCE 7. He has extensive experience with the seismic design and evaluation of buildings in both the eastern and western U.S.

References

Detailed descriptions of shear wave velocity tests and references for them are in *Geotechnical Earthquake Engineering* (Kramer 1996).

Kramer, Steven L., *Geotechnical Earthquake Engineering*, Prentice Hall, Upper Saddle River, New Jersey 07458,1966.

Federal Emergency Management Agency (FEMA) *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Part 1: Provisions (FEMA 450-2), 2003 Edition, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C., 2004.

American Society of Civil Engineers and Structural Engineering Institute, *Minimum Design Loads for Buildings and Other Structures*, Including Supplement NO. 1 (ASCE 7-05), 2005.

Standard Penetration Test Energy Measurements

By Lawrence F. Johnsen, P.E.

Today, equipment is readily available to measure the energy transfer from a Standard Penetration Test (SPT) to the drill string. The method most commonly used is the Force Velocity method in which force and velocity are integrated over time.

Equipment typically consists of a pile driving analyzer, which is connected to an instrumented drill rod that contains two strain gages and two accelerometers. The instrumented drill rod section (*shown in the photo below*) is placed at the top of the drill string.

Energy transfer measurements are made for every hammer drop during an individual SPT test. Tests are performed on several but not all SPT tests. The attachment of the instrumented rod section adds about 15 minutes to the driller's time for each test. A typical report includes a tabulation of energy measurements for each hammer drop, along with the average energy transfer and coefficient of variation for each SPT test.▪

Lawrence Johnsen, P.E. is a principal of the geotechnical engineering firm Heller and Johnsen in Stratford, Connecticut.

Attaching SPT to Instrumented Rod Section

APPENDIX F

FEMA EARTHQUAKE HAZARD MAPS

FEMA

Earthquake Hazard Maps

- How to Read the Maps
- Maps
- Data for Building Design Professionals

How to Read the Maps

The maps displayed below show how earthquake hazards vary across the United States. Hazards are measured as the likelihood of experiencing earthquake shaking of various intensities.

The colors in the maps denote "seismic design categories" (SDCs), which reflect the likelihood of experiencing earthquake shaking of various intensities. (Building design and construction professionals use SDCs specified in building codes to determine the level of seismic resistance required for new buildings.)

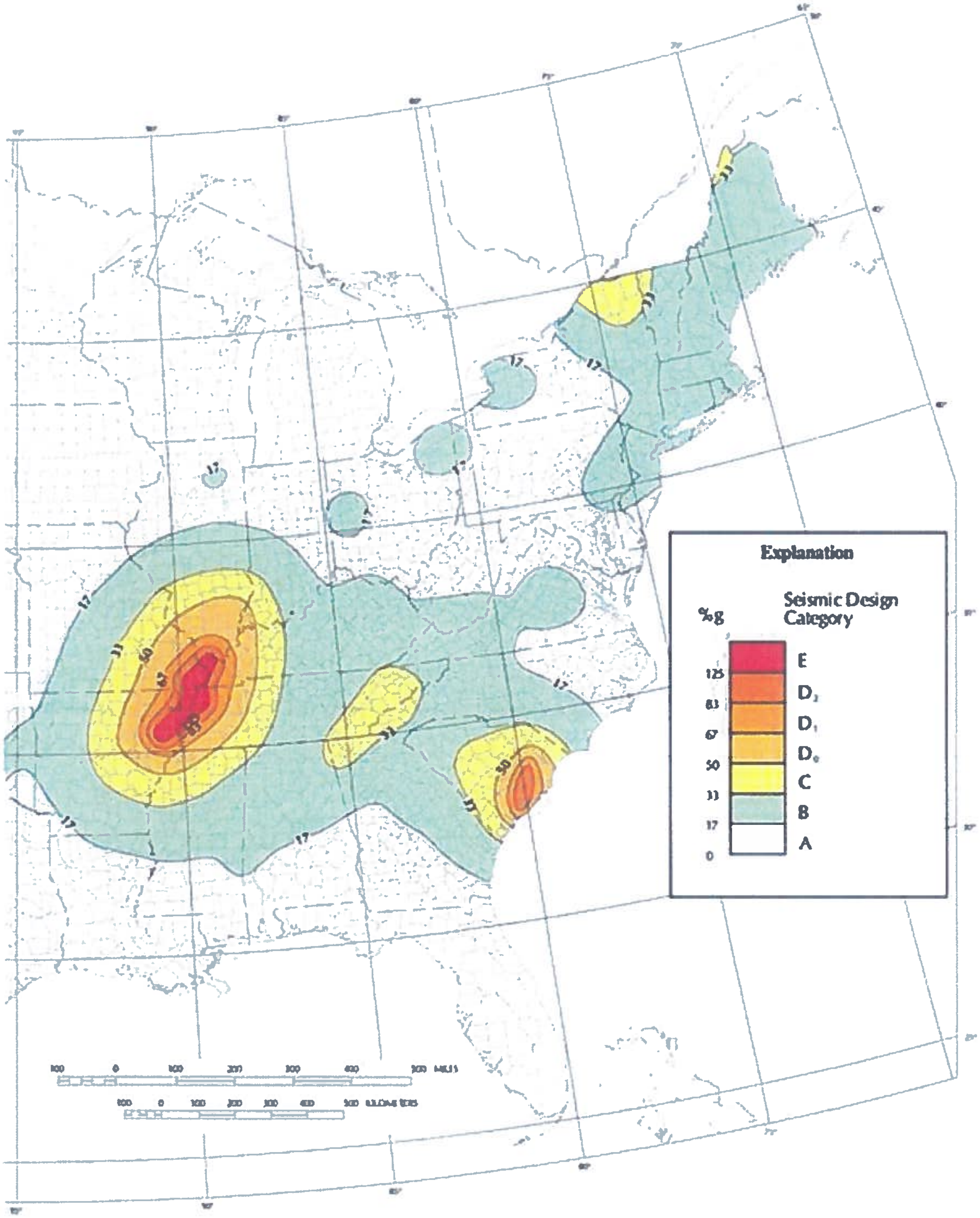
The following table describes the hazard level associated with each SDC, and the associated levels of shaking. Although stronger shaking is possible in each SDC, it is less probable than the shaking described.

SDC	Map Color	Earthquake Hazard	Potential Effects of Shaking*
A	White	Very small probability of experiencing damaging earthquake effects.	
B	Gray	Could experience shaking of moderate intensity.	Moderate shaking—Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
C	Yellow	Could experience strong shaking.	Strong shaking—Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built structures.
D_0 D_1 D_2	Light brown Darker brown Darkest brown	Could experience very strong shaking (the darker the color, the stronger the shaking).	Very strong shaking—Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures.
E	Red	Near major active faults capable of producing the most intense shaking.	Strongest shaking—Damage considerable in specially designed structures; frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Shaking intense enough to completely destroy buildings.
* Abbreviated descriptions from The Modified Mercalli Intensity Scale.			

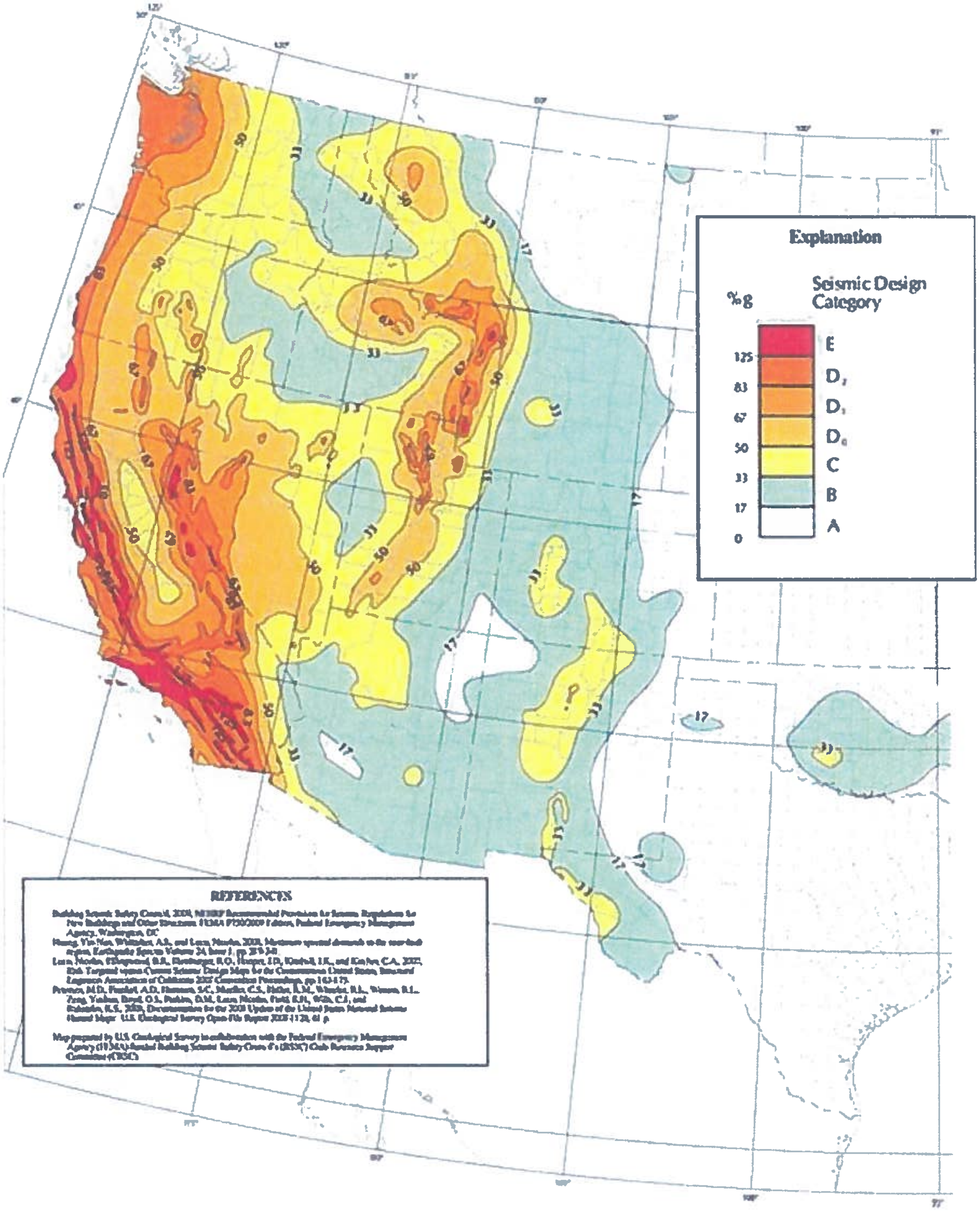
SDCs take into account the type of soil at the site, as poor soils can significantly increase earthquake shaking. These maps have simplified this by assuming normal Site Class "D" soils, which are the most commonly found.

When viewing the maps, it is important to remember that areas with high earthquake hazards do not necessarily face high seismic risks. Defined as the losses that are likely to result from exposure to earthquake hazards, seismic risks are determined not only by hazard levels but also by the amount of people and property that are exposed to the hazards, and by how vulnerable people and property are to the hazards. This is explained in more detail in Your Earthquake Risk.

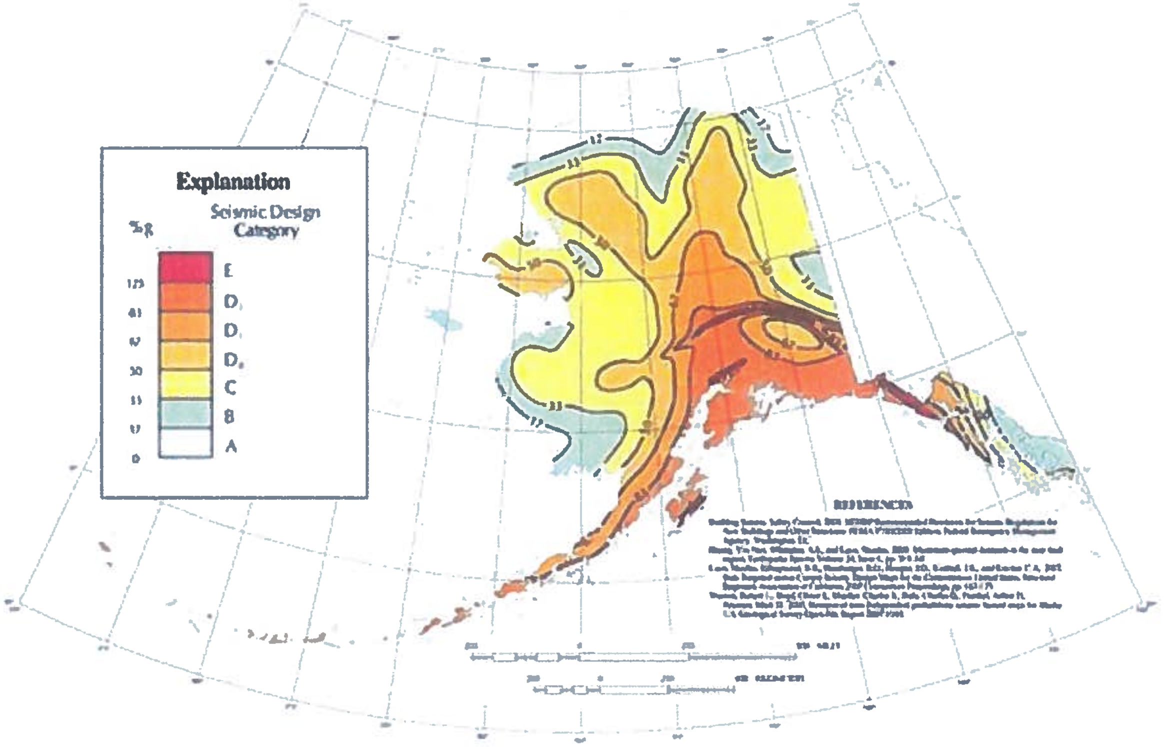
Maps



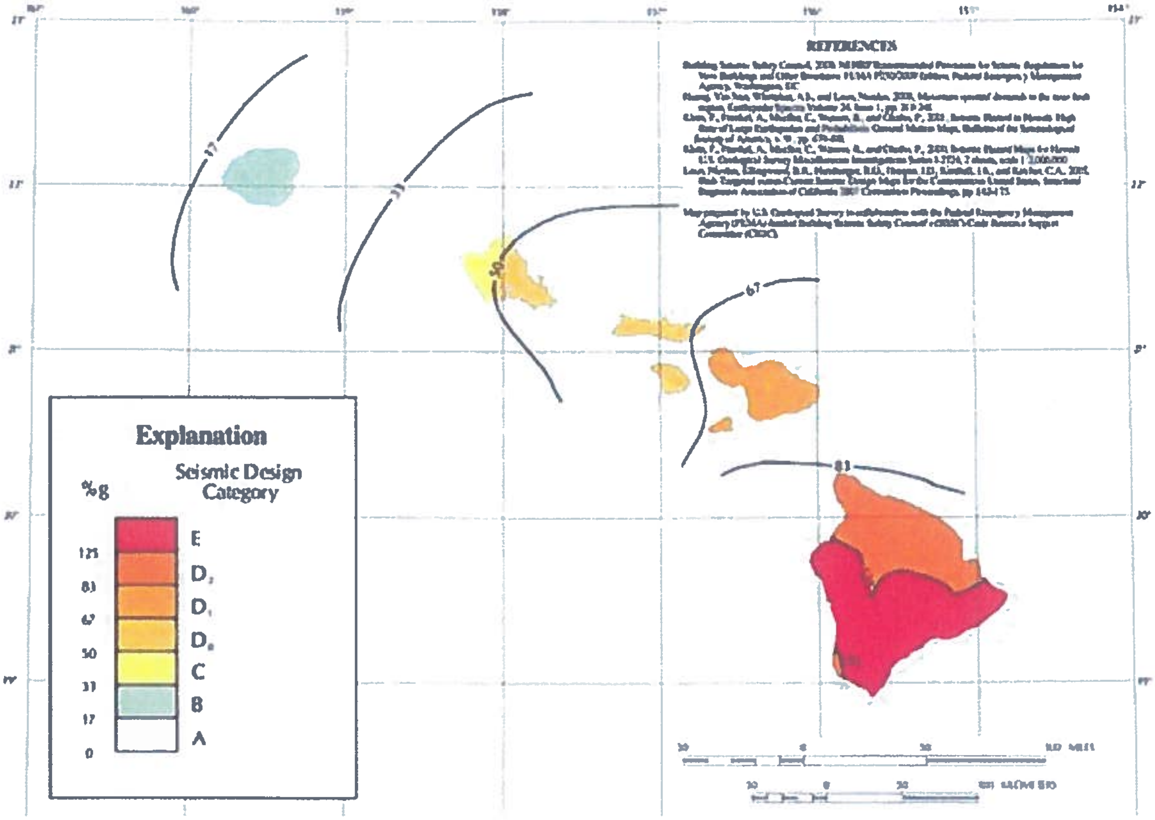
SDC map of the Eastern United States for low-rise Occupancy Category I and II structures located on sites with average alluvial soil conditions.



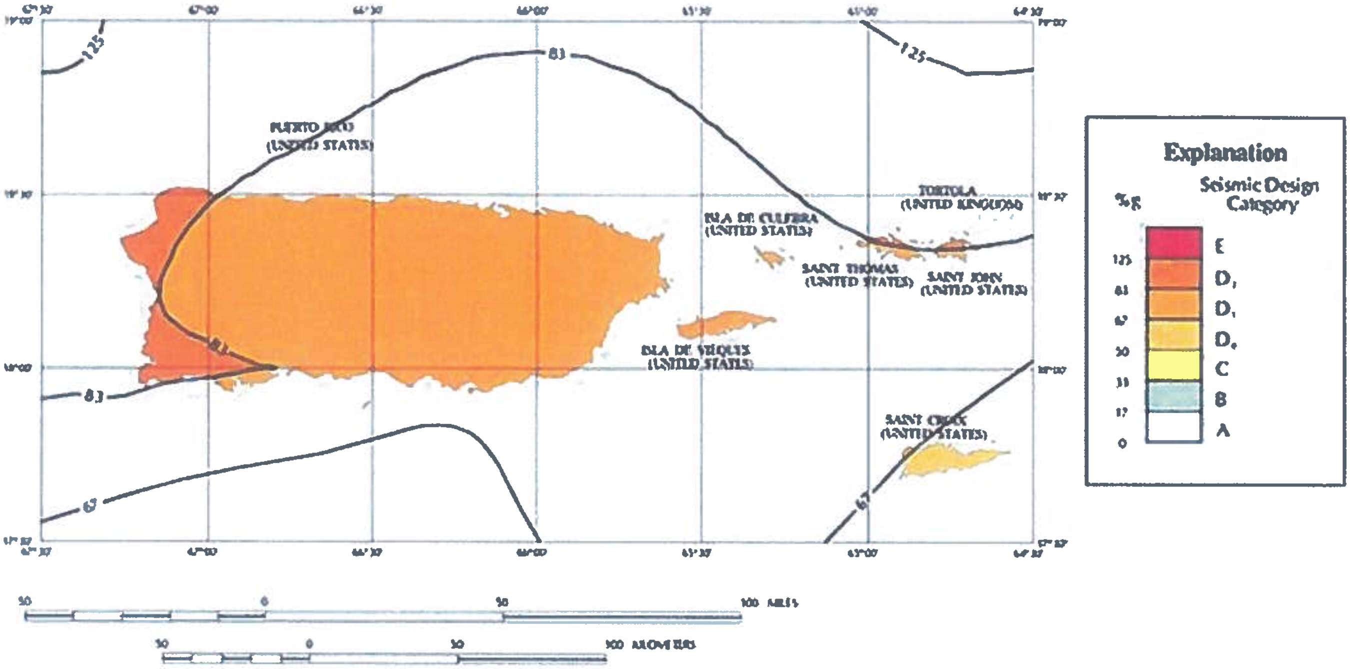
SDC map of the Western United States for low-rise Occupancy Category I and II structures located on sites with average alluvial soil conditions.



SDC map of Alaska for low-rise Occupancy Category I and II structures located on sites with average alluvial soil conditions.



SDC map of Hawaii for low-rise Occupancy Category I and II structures located on sites with average alluvial soil conditions.



SDC map of Puerto Rico, the United States Virgin Islands, and Tortola for low-rise Occupancy Category I and II structures located on sites with average alluvial soil conditions.

Data for Building Design Professionals

The U.S. Geological Survey, in cooperation with FEMA and the Building Seismic Safety Council, has developed a web-based seismic design application for building designers. This program can be used to obtain the earthquake ground motion parameters needed to design structures for specific geographic locations in accordance with the latest building code reference documents. To access this application, as well as the seismic design maps on which it is based, go to the U.S. Seismic "DesignMaps" Web Application.