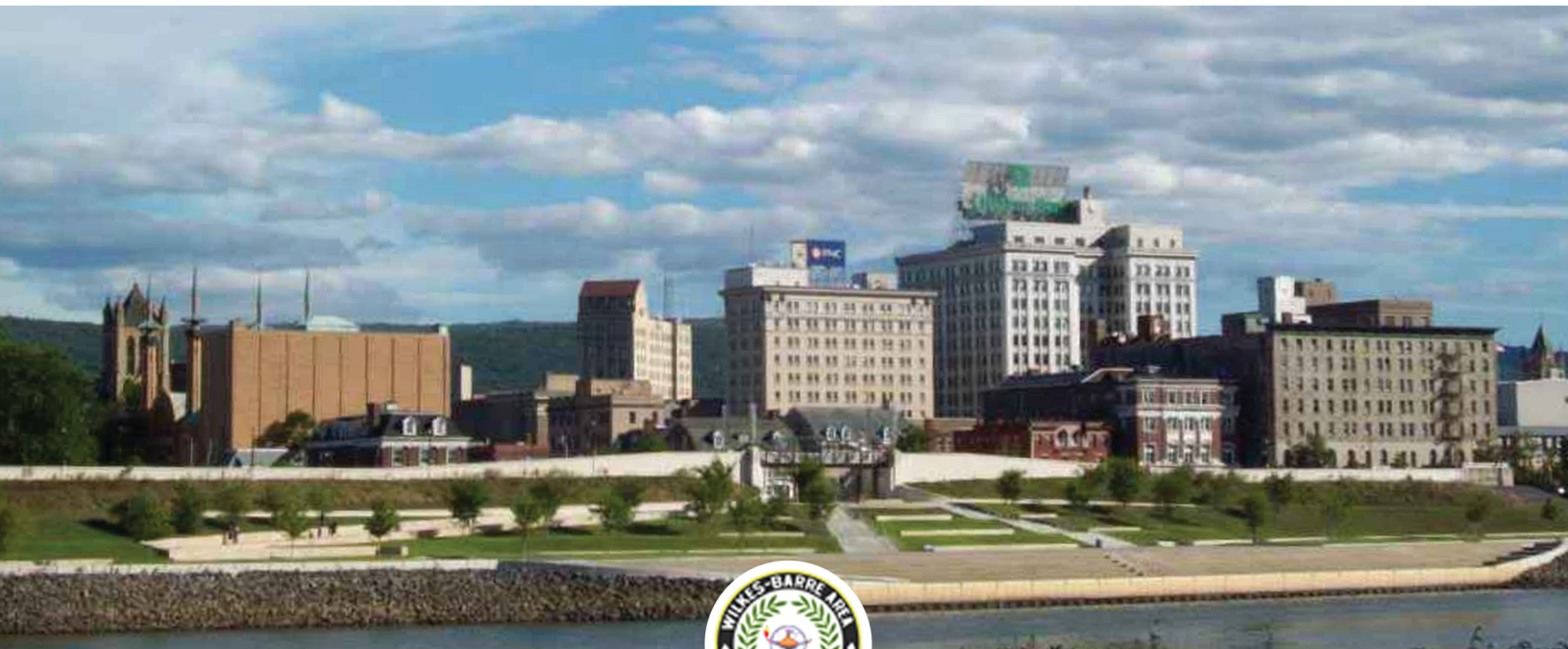




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## WILKES-BARRE AREA SCHOOL DISTRICT STRUCTURAL EVALUATION OF ELMER L. MEYERS HIGH SCHOOL

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December 8, 2014

Wilkes-Barre Area School District  
730 South Main Street  
Wilkes-Barre, PA 18702

**Re: Structural Evaluation of Meyers High School**

Please find enclosed our detailed structural evaluation of Meyers High School. The study represents our findings after many hours of on-site inspection along with review of the original structural design drawings. Using the previously undiscovered original drawings, we were able to analyze the superstructure with conclusions differing greatly from previous studies. We also performed a detailed façade evaluation in order to offer recommendations to keep the students and general public safe in perimeter areas of the school.

In general, the building is currently safe for continued occupancy with some minor work recommended. However it is our opinion that a full 20-year renovation is not feasible.

The enclosed report explains the current conditions of the 84 year-old original school and why the structure cannot be feasibly renovated to meet current codes. We have also included representative calculations and framing plans. Full-size framing plans depicting all recovered shop drawings are available upon request.

Please call me with any questions.

Sincerely,

A handwritten signature in black ink, appearing to read 'T. G. Leonard', is written over a horizontal line.

Thomas G. Leonard, P.E.  
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251 Mundy Street  
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# Meyers High School Structural Evaluation

Wilkes-Barre Area School District  
Wilkes-Barre, Pennsylvania



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Models and Calculations

Appendix E Geotechnical Report of 2007

Appendix F Floor Elevation Survey Comparisons

## Meyers High School Structural Evaluation

Wilkes-Barre Area School District  
Wilkes-Barre, Pennsylvania



### Overview

Meyers High School was designed in the late 1920's and construction of the main building was completed in 1930. For this evaluation, we searched the District archives and were able to locate and review portions of the original structural design drawings along with the associated steel and terra cotta shop drawings. Based on this review and our inspections, it was determined that the majority of the building remains in its original configuration today. The only notable changes in the structure since its construction include the removal of skylights over the original pools (time unknown), conversion of the boys pool to a weight room (time unknown), the demolition of the attached stadium bleachers (2010) and removal of the high parapets (2008).

Per the designations used in the original drawings, the school structure is composed of six modules:

1. The V-shaped "Main Building" classroom portion of the school. This module runs along the Carey Ave. and Hanover Street sides and contains three stories and a basement.
2. The Auditorium, which ties into the northern end of the Main Building.
3. The Gymnasium which is above a basement level that currently houses the wrestling rooms.
4. The Cafeteria which is above a basement level containing classrooms.
5. The "Shops Building" module is the three-story original wing facing the stadium. This module ties into the southwest end of the Main Building on Hanover Street as well as the Cafeteria on the northwest corner of the school. This module is based at the playing field level (main school basement level) and contains ground level locker rooms, a first floor library & maintenance room and second floor classrooms. The maintenance room on the first floor is grade accessible with a ramp off of Hanover Street.
6. The "Science Building" which is located inside the area encompassed by the other five modules. This is a four-story structure with the "basement" level located at finished grade. It is accessible internally from the corridor running along the Gymnasium and Auditorium. This module contains single story sides which were originally the boys' and girls' pools. The boys' pool has been converted into a weightlifting room while the girls pool is currently abandoned. This building is also accessible externally from the main courtyard described below.

The interior portions of the Main Building, Science Building and Shops Building are also connected by central corridors at the first and second floor levels.

The above modules enclose a pair of interior courtyards. A large one used for parking is accessed via a tunnel which runs between the basement levels of the Gymnasium and Cafeteria. A smaller one is formed between the Science Building and the Main Building, adjacent to the abandoned girls' pool.

There is also a detached multistory heating plant located on the north side of the Auditorium and Gymnasium. The heating plant is separated from the main school above grade by a sidewalk that continues to serve as an elevated access walkway to the Gymnasium, Cafeteria service doors and the dining area. Originally this walkway continued to the stadium bleachers which were demolished in 2010.

See the plan view indicating the original design modules and school overall layout in the Appendix A.

### **Building Structural Components**

The school was constructed with a steel frame supporting concrete floor slabs and roof decking similar to tectum. The exterior façade consists of face brick with terra cotta accents at the entrances and above door and window openings.

The steel superstructure is supported by a conventional shallow foundation system. The foundation walls on the Carey Avenue and Hanover Street sides are formed with mass concrete retaining walls. The walls return on both ends, running along a portion of the boiler house on the north side and also as the face of the Hanover Street driveway that faces the stadium. The retaining wall also extends beyond the building and is visible where it becomes the stadium retaining wall running along Hanover Street.

### **Building Conditional Assessment**

In order to evaluate the structural condition of Meyers High School, the following steps were taken by Leonard Engineering:

- The school structure was inspected for signs of floor cracking, excessive movements, steel corrosion, etc. to determine if any unsafe conditions existed.
- Surveys of the floor elevations were performed in order to compare present-day structural slab conditions with those measured during the 2007 study.
- Access planking was installed in the Main Building attic spaces to allow inspection of the roof framing. The attics, especially areas known to be exposed to long-term roof leaks, were checked for corrosion. The framing members were also measured and verified to match the original drawings in size and locations.
- Using the discovered original building plans/shop drawings and spot checking the sizes of exposed steel members, we performed a structural analysis of the school framing. Our analysis included full models of the Main Building, Auditorium and Gymnasium. Typical framing bents of the Science Building, Shops Buildings and the Cafeteria were analyzed to confirm that the framing capacity in those areas was similar to the areas fully modeled.
- A study of the exterior façade was performed by our historic masonry consultant, Masonry Preservation Services, Inc. A copy of their report is attached as an appendix to this study.

### **Existing Frame Analysis**

The existing steel framing was analyzed to determine allowable floor and roof load capacities based on today's steel code. The steel manufactured in the late 1920's was researched (see Appendix D) and determined to have an allowable working stress of 18,000 pounds per square inch (psi) or 18 kips per square inch (ksi) and a yield stress of 28 to 33 ksi. For comparison, today's structural steel is typically has a yield strength 36 or 50 ksi and working stresses of 24 and 30 ksi respectively.

Based upon the allowable stresses for the older steel, our analysis determined the floor live load capacity to be approximately 150 pounds per square foot (psf) throughout the structure. The 2009 IBC code requires current schools to be designed for the following live loads:

- |                         |         |
|-------------------------|---------|
| • Classrooms:           | 40 psf  |
| • 1st Floor Corridor    | 100 psf |
| • Upper floor corridors | 80 psf  |

Therefore, the existing framing is quite sufficient, with a minimum reserve capacity of 50% in corridors and over 300% in classroom areas. The existing roofs were also checked for snow load capacity. Our analysis determined the total capacity of the roof framing is approximately 70 psf. When the dead loads of the roof panels (8 psf), suspended ceilings (12 psf), piping, lights, etc (5 psf). are deducted the snow load capacity is conservatively estimated to be approximately 45 psf. The current design roof snow load in Wilkes-Barre is 35 psf and therefore the capacity exceeds current code requirements by more than 25%.

The reinforced concrete floor slabs are shown to be 6" thick on the original drawings and have performed well for over eighty years. Because no material notes indicating the reinforcing type or concrete strength were discovered, we can only estimate the capacity of the floor slab system. Based upon a 2500 psi concrete mix and minimal reinforcing, the 6" thick slab was likely designed to support a similar live load as calculated for the steel framing - 150 psf. Although the concrete strength and reinforcing can be approximated with destructive testing, the more exact analysis of the steel framing can be used to reasonably conclude the design loads used by the original engineers.

Although the original structural design complies with current code requirements for gravity loads, it was not designed to resist today's wind or seismic requirements. The beam to column connections are simple double angles or stiffened seats with no moment resisting capacity. The structure was designed to resist wind loads with the brick interior and exterior walls. The older mortar, especially the deteriorated exterior walls, are not capable of transferring the required seismic lateral loads to the foundations. In order to bring the structure into compliance as required for a long-term renovation, the existing framing will require significant reinforcement including modified beam to column connections and the installation of chevron or X-bracing.

Based upon a preliminary seismic analysis of the main classroom wing, the seismic forces that must be resisted are approximately 7 tons at the base of each typical frame line (3500 lbs at each column base), spaced at only ten feet on center. If alternate frames are reinforced, these forces are multiplied by the number of frames skipped. With the amount of lateral force applied, the affected column footings will also likely require reinforcing or enlarging.

If reinforcing of the frames to accommodate these forces is done at each line, the work involved in removal and replacement of finishes to allow reinforcements will be tremendous. The reinforcing will also be time consuming as there are over 400 connections in these wings alone. The central stair area between the classroom wings, which has over 12" of uneven settlement, will be more difficult since the frames to not form straight lines and forms of intrusive bracing, chevron or X-braces, will be required.

Other areas of the school such as the Gymnasium and Auditorium will provide much more difficult lateral bracing challenges due to the heights of the columns and clear space required.

Design of a code-compliant lateral bracing for the complicated framing of Meyers is beyond the scope of this study. However preliminary discussions with steel contractors revealed the costs could potentially exceed \$25 million, excluding the potential costs of footing reinforcement.



**Previous Study**

A previous investigation of Meyers High School was performed in 2007 after it was discovered that settlements of up to twelve inches (one foot) had occurred in the floors. At that time a geotechnical analysis was performed by Geo-Science Engineering Co., Inc. and is included in Appendix E.

The 2007 geotechnical report concluded that the excessive settlements had likely occurred during flood events (1936 and 1972) and on other occasions where the river levels exceeded the ground elevation at Meyers. The report suggested that the excessive settlements may be cause for structural concern, although it was noted that the structural engineer for that study, Highland Associates, inspected beam-to-column connections and found no signs of overstress. The report also stated that additional soil testing and water level monitoring would be necessary to determine the exact cause of the settlements and to predict future movements.

**Existing Interior Structural Conditions**

The school was inspected with non-destructive methods and the existing framing was spot checked to verify the recovered steel shop drawings had been followed. Interior and exterior walls, accessible steel framing and concrete floor slabs were checked for signs of damage, deterioration and unsafe conditions. No areas of immediate concern were discovered.

After verifying the existing steel sizes matched the shop drawings, the framing was analyzed to determine if it was sufficiently designed to carry current building code-required school live loads and roof snow loads. The main concern with the existing superstructure is the excessive differential settlements discovered in the area of the Main Building entrance. This portion of the building ties together the two classroom wings which run parallel with the Hanover St. and Carey Avenue. The entrance area contains a large staircase and also connects the entrance to the Science Building located behind the main stair.

The 2007 survey indicates, the floor in the Main Building adjacent to the north side of the main staircase slopes twelve inches (12") from the Carey Ave. wall to the interior courtyard wall. The settlement differential between the edges of the classroom floors on the opposite end of this corridor, adjacent to the Auditorium, is six inches (6") from front to rear. Therefore the floor along the courtyard side slopes 6" from the Auditorium to the main staircase while the floor elevations along the Carey Ave. wall remain relatively level. The measured slopes are consistent on all framed floor levels.

The floor slabs on the Hanover Street side of the staircase vary in elevation to a much lesser degree - less than 2" over the entire length of the corridor. This differential settlement would be considered normal given the age of the structure and would also fit within the predicted long-term settlements of the 2007 geotechnical report.

For this evaluation, portions of the floor surveys performed in 2007 were replicated to determine if the structural settlements had continued over the past 7 years. There were several high water events during that period including September 2011, when the artificial turf surface at Meyers stadium bubbled up due to hydrostatic pressure. In addition to the numerous high water events, the structure withstood a minor earthquake in 2011. The results of the 2014 survey indicate that little or no additional settlements took place in the past seven years.

The 2007 survey was also replicated in 2012 by Borton-Lawson Engineering to verify if any 5-year movements had occurred. That survey report, included in Appendix F, also indicated the structure had been stable since 2007. As noted in the Borton-Lawson report, the minor differences in some of the shot elevations can be attributed to the difference in equipment used for each survey. This is true for the 2014 survey as well.

The 2014 survey also included shots in the mechanical (4th) floor above the main entry and the results were similar to the floors below, with the mechanical slab sloping approximately 12" from front to rear. No cracking was noted in the unfinished mechanical room concrete floor slab.

During the 2014 survey the brick mortar joints were surveyed along the rear wall of the Carey Ave. side of the Main Building. The survey indicated the brick joints slope 6" from the Auditorium to the staircase, matching the floor slope along that wall.

Drawings indicating representative 2007 and 2014 surveyed floor elevations, along with the Borton-Lawson 2012 report, are included in Appendix F of this evaluation.

The previous geotechnical study was performed in 2007 without the benefit of the original drawings or previous floor elevation surveys. It was concluded in that report that the footings had likely settled excessively due to high water levels at various times throughout the school history. It was noted that this type of settlement had also occurred in nearby structures constructed during the same time frame and the combination of high water and the existing soil characteristics formed a sound basis for this conclusion. However, the conclusions and predictions of future settlements were not definitive with the report noting that additional testing was recommended.

The 2007 report also stated that the soils should be capable of supporting 2,000 to 3,000 pounds per square foot (psf). At that time the soil pressures were estimated based on unknown footing sizes and the assumption the column lines were spaced at 15 to 20 feet on center. With the discovered framing plans, we have verified that typical column line spacing is 10 feet and that the design soil pressure was 3,000 psf using the footing sizes shown on the original plans.

One of the main observations made during our inspections of the exterior is that there is no major cracking in the brick which would be expected with the large differential settlements, particularly in the main staircase area. The corner joints on the exterior of the rounded rear stair tower walls are in very good condition and do not appear to have been modified recently. Had the 12" differential settlements occurred after these walls were constructed, large cracks would be expected throughout the exterior, especially where the staircase walls meets the narrow corridor to the Science Building and in the interior courtyard wall corners. However there are no notable cracks in any of these areas to indicate the large or even minor differential settlements.

There are also no minor or major cracks in the interior plaster finishes covering the beams where they tie into the walls. Cracking in these areas would certainly be expected with the movements, especially if the settlements are ongoing.

Given the extreme amount of differential settlement between building elements, there can only be two explanations for the lack of cracking in the exterior walls and interior finishes:

1. The walls and interior plaster finishes have been repaired since the most recent settlement occurred.
2. The settlements occurred before the brick and interior finishes were installed.

As noted previously, there do not appear to be signs of recent brick repairs in the areas where large-scale cracking would be expected. Cracking in the building tie-in corners would have been present at all floor levels and misaligned patterns in the mortar and slightly mis-colored brick would be clearly indicative of repair. Repairs of this type are clearly visible in several courtyard areas where windows and exterior doors were cut-in or in-filled during the life of the structure. If the settlements were caused by high water events, which have occurred as recently as 2006 and 2011, it is highly likely that cracks would have developed in the tie-in corners. It is certain no major brick repairs have been performed in that time span.

It is our theory that the majority of the settlements occurred on non-compacted soil during the school construction. It is unknown if a geotechnical study was performed prior to designing of the foundation system or if proper backfilling and compaction methods were employed. It is possible the settlements occurred as the steel frame was erected and the concrete floor slabs were installed. Once the soil compacted under the frame loads to a stable condition it was capable of supporting the pressures indicated in the 2007 geotechnical report. Any cracking occurring in the slabs due to uneven settlements would have been repaired at that time and would not translate into floor finishes applied after the soils stabilized.

It is also possible that the large-scale differential settlements occurred as an aftermath of the 1936 or 1972 floods noted in the geotechnical report. However, no records of major structural repairs or façade replacement have been discovered for either time period.

Regardless of the time-frame of the settlements it is apparent that the structure has been stable for at least the past 7 years. This opinion is based upon the current condition of the brick façade joints, the lack of excessive cracking in the floors or interior finishes and the close comparisons of the 2007-2012-2014 floor surveys.

During our investigation we uncovered an unnamed drawing of the Main Building entrance area showing existing fourth floor framing to be removed and a new lower roof over the mechanical room. The revision drawing is dated November of 1928 and indicated the adjacent roofs were "present". The drawing also called out the original upper floor as "new roof". The discovered steel shop drawings dated 1927 indicate the reference elevations for this area are above the "5th floor" which was not completed. The shop drawings we recovered were not a complete set and the sheet expected to contain the upper roof framing was not found. All of the framing shown in this area, including what changed from the 5th floor to the "new" mechanical room roof was field verified as existing.

We also discovered a 1926 architectural elevation of the front entrance which indicated a large dome structure above the main entrance. However, no reference to the dome was discovered on the later steel

shop drawings and it may have been revised to the referenced fifth floor. No other references to this upper area were discovered.

The steel columns and beams shown to be removed did not cover the entire mechanical room, indicating the decision to eliminate the upper floor was made during the erection of this steel. It is unknown if the deletion of the dome and/or the additional floor was a result of uneven building settlements or simply a late architectural revision. Unfortunately we did not discover architectural floor plans showing the original design of this area. The exterior brick walls are also shown as "present" indicating they were in place at the time the decision to remove the upper framing. If the rotation of the framing had already occurred, it is possible this floor was eliminated because the additional height of the framing in an out-of-plumb condition may have caused the architects or engineers to be concerned with the frame stability with the increasing amount of eccentric loads. It's also possible the amount of rotation in the first few columns erected above the much lower side roofs may have been noticeably visible.

In addition to the framed floor deflections, the 2007 basement slab on grade elevations were compared to the 2014 survey. These surveys indicate localized settlements of up to 2" have occurred, which indicates the soils may have been affected by recent water line and pool system leaks. The floor slabs in the original boys' pool area (current wrestling room) are noticeably warped. Based upon the original drawings, the footings were based on soil while the areas above the footing bases, including slabs, were shown to be supported by "ash fill". The fills were determined to be wet and loose to medium dense in the 2007 geotechnical report (pg. 9) and are likely to move differently than the native soils.

While these basement slab movements are not structurally significant from a overall building stability standpoint, it does reinforce the fact that the main structure footings are stable and have not been affected by the under floor leaks, high water table or seismic events in recent years.

### **Existing Exterior Structural Conditions**

The exposed structural elements along the school perimeter were inspected for deterioration and damages. Areas checked include the elevated walkway along the Gymnasium and Cafeteria, exposed columns adjacent to the previously removed stadium bleachers and the retaining wall along the Hanover Street maintenance access ramp.

The steel framing supporting the walkway is showing signs of deterioration. It contains a sprayed-on fireproofing coating which is apparently trapping moisture and contributing to the corrosion. For a short-term (5 year) continued occupancy, we recommend having the fireproofing removed to allow closer inspection of the steel and to allow any repairs or reinforcing required by that inspection.

Also noted on the walkway were numerous corroded guardrail connections and one broken one which should be repaired .

The columns along the west wall beneath the removed stadium walkway appear to be in stable condition. However the base of the ramp leading from Hanover Street to the Maintenance shop is in poor condition with a potential of stones falling to the lower level. While this is a fairly remote location with regards to parking, etc., we recommend having the area cordoned off.



A small addition constructed in the Courtyard adjacent to the Girls Pool has extensive wall damage at the base. The wall has apparently been subject to extensive water infiltration and the masonry foundation is crumbling and repairs are recommended.

**Exterior Façade Conditions**

The exterior skin of the school is comprised of brick walls with terra cotta lintels and stone cornices. Originally the Main Building contained high parapets on the Carey Ave and Hanover St. elevations. A large portion of these parapets were removed in 2007 when they were investigated and found to be in very poor condition. The parapets had been subject to years of storm water infiltration and which deteriorated the upper portion significantly. This results of that study are included in the Masonry Preservation Services (MPS) 2014 evaluation contained in Appendix C of this evaluation. The current evaluation of the façade explains the amount of deterioration and current areas of concern.

A 2014 follow-up study of the Main Building entrance architraves by MPS (as recommended in the 2007 study) classified the area "as safe with repairs within a year". Therefore that area will require a restoration program or have protective netting installed prior to the commencement of the 2015-2016 school year. Based on that same study, protective netting was installed above the Auditorium entrance and also on the Heating Plant wall to protect the access road leading to the interior courtyard and stadium. In both of these areas loose pieces of the terra cotta architrave joint grout and the loose Heating Plant brick were about to fall.

As this evaluation is being completed in December 2014, additional protective measures have been recommended and are currently being designed for installation in the interior courtyard area, the Cafeteria walkway exit doors and along the north side of the auditorium.

While no other areas of immediate concern were noted in our recent inspection with MPS, it is recommended the façade conditions are monitored on a semi-annual basis in order to quickly address future potentially dangerous conditions as they develop. Inspections are recommended in early spring and late summer. These monitoring periods will reveal damage due to recent extreme temperatures.

Because of the potential for small pieces to break free and fall throughout the Courtyard area, it is recommended that all personnel and students parking in the courtyard are required to sign damage waivers. The area should also be posted "Park at your own risk" signage.

Refer to the façade report prepared by Masonry Preservation Services, Inc. in Appendix C for additional information regarding the façade condition and details regarding long term restoration costs.

**Heating Plant**

The Heating Plant building is located on the north side of the school, adjacent to the Auditorium. The multistory building and is a steel framed structure with a brick and terra cotta façade which matches the school. No original steel design drawings were discovered for this building.

A 2014 inspection of the façade revealed excessive mortar deterioration along the northern and western walls of the building. Because of the loose bricks encountered, a netting system was installed to protect

the adjacent heavily traveled driveway. Per the current MPS façade evaluation in Appendix C, this façade cannot be feasibly restored for a long-term restoration due to the extent of mortar loss.

Internally, the reinforced concrete floors are exhibiting areas of extensive deterioration, with reinforcing steel corroded and exposed on the underside. This is likely due to water leaking from the heating system from the equipment located directly above the slab and supported on the steel frame.

Based upon the façade inspection and obsolescence of the heating system it is our opinion that long-term restoration is not feasible. Therefore no detailed framing evaluation was performed on this structure. Short term occupancy of the school will require monitoring the façade for additional deterioration.

### **Site Structural Conditions**

In addition to the school and Heating Plant, we have recently inspected the retaining walls surrounding the stadium. In the summer of 2014 the deteriorated top portion of the north endzone wall was removed and the overall height was reduced. A deteriorated and cracked portion of the above-grade brick wall adjacent to the Old River Road bleacher entrance was also removed and replaced with chain link fencing. At this time the remainder of the concrete foundation walls and brick extensions appear to be in stable condition. The areas of brick wall along the Hanover Street sidewalk contain minor cracks must be monitored if short-term occupancy is anticipated. Long-term renovations will likely include partial or total replacement of the upper brick portions and repairs to the lower mass concrete retaining walls.

It was also noted that the concrete retaining wall and chain link fence bordering the driveway which runs along the Heating Plant are in very poor condition.

### **Recommendations:**

Following are recommendations for actions/repairs necessary for immediate, short-term and long-term school occupancy. (Estimated costs in parentheses)

#### *Immediate Actions:*

1. Install protection structures in the areas identified in the MPS façade report. (\$30K)
2. Cordon off the area below the Maintenance access ramp off of Hanover Street (< \$1K)
3. Cordon off area directly behind the Heating Plant (< \$1K)
4. Install "Park At Your Own Risk" signage and obtain damage waivers from all drivers accessing the interior Courtyard (< \$1K)

#### *Short-Term Occupancy (5 years)*

1. Perform semi-annual façade evaluations, estimated cost \$10K per evaluation.
2. Install netting or re-point the main entrance architrave.
3. Monitor the existing façade for movements and/or additional fallen objects. (\$5K/year)
4. Install protective netting or sheds as required by above evaluations. (\$10K to \$50K)
5. Remove fireproofing from walkway steel to allow inspection and repairs (\$10K to \$500K)
6. Repair broken and damaged walkway guardrails and brick posts. (\$2K to \$15K)
7. Perform annual stadium retaining wall inspections and necessary repairs (\$2K to \$250K)
8. Repair base of wall at small addition adjacent to Girls Pool. (\$4K to \$15K)

*Long-Term Occupancy (25 Years)*

1. Perform façade restoration as detailed in MPS evaluation (\$11 to 15 Million)
2. Install lateral bracing system (\$10 to \$30 Million)
3. Demolish existing Heating Plant structure. (\$500K)
4. Repair or replace stadium retaining walls and brick extensions (\$1 to 2 Million)

**Conclusions**

Meyers High School has been operating for 84 years during which the exterior façade has been damaged by periods of excessive water infiltration. The Main Classroom building exists in a warped condition due to excessive settlements which likely occurred during construction. Despite the settlements, the steel and concrete framed superstructure appears to be in sound overall condition and capable of supporting the current code-required live and snow loads.

The façade has been exhibiting deterioration for many years, requiring large portions of parapets to be removed in 2008 for safety reasons. There are current concerns of façade pieces falling throughout the exterior and actions are currently being taken to protect the most vulnerable areas from this danger. Unfortunately this condition will continue to expand and the façade will have to be monitored in all areas where students and the general public have access as long as the school remains in operation.

The costs estimated for long-term structural costs in the previous section do not include the long-term renovation costs required for architectural, mechanical, electrical and plumbing systems. Because of those costs and the extensive work required on the school façade and lateral bracing, it is our opinion that long-term restoration of the school will not be feasible.

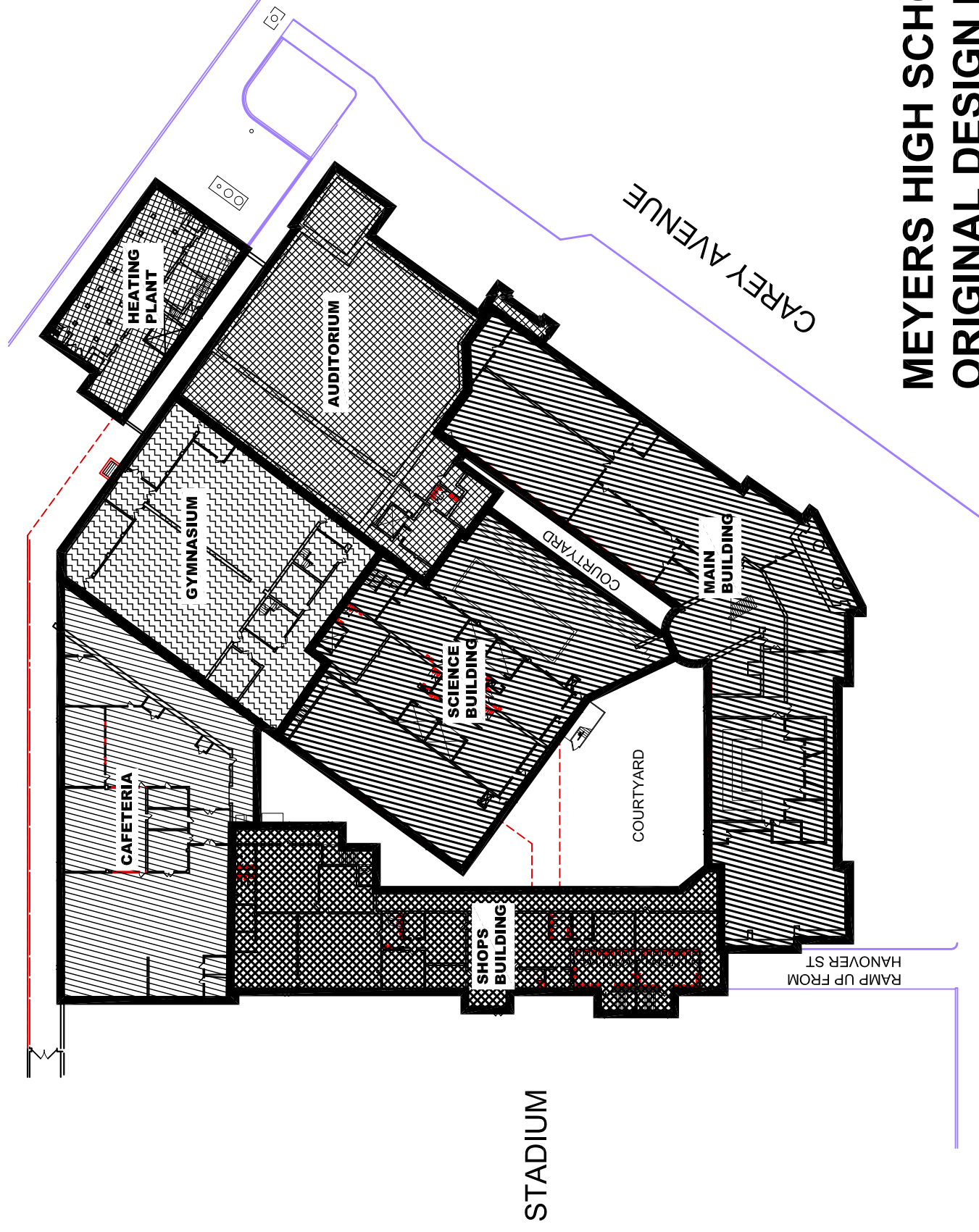
The school may continue to operate safely providing the recommended short-term actions/repairs are performed in a timely manner.

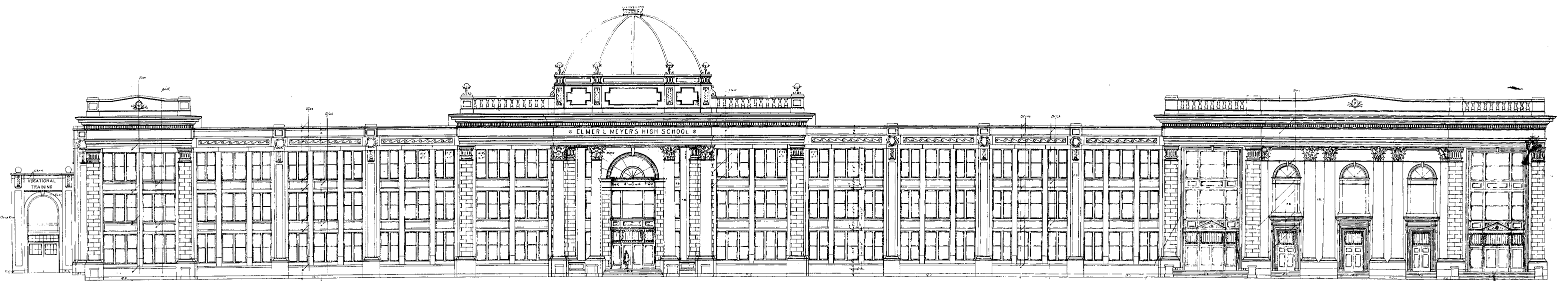
## APPENDIX A – FLOOR PLAN

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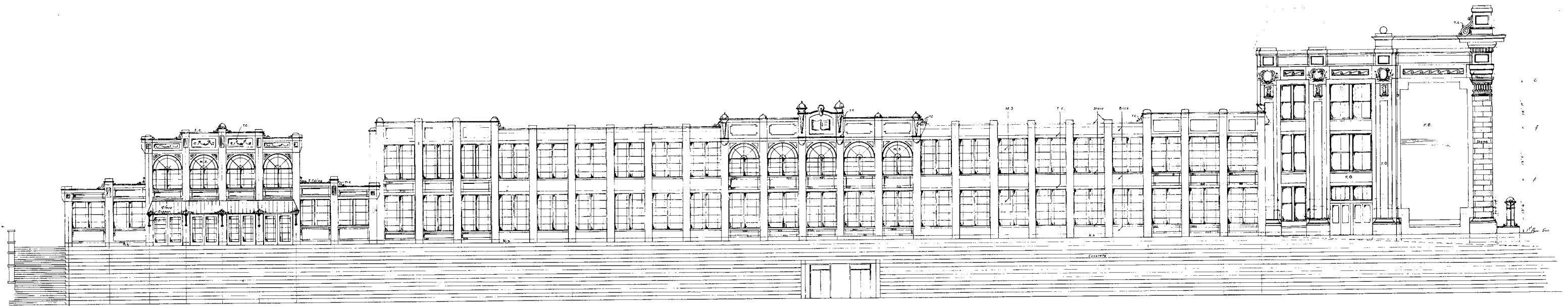
# MEYERS HIGH SCHOOL ORIGINAL DESIGN MODULES



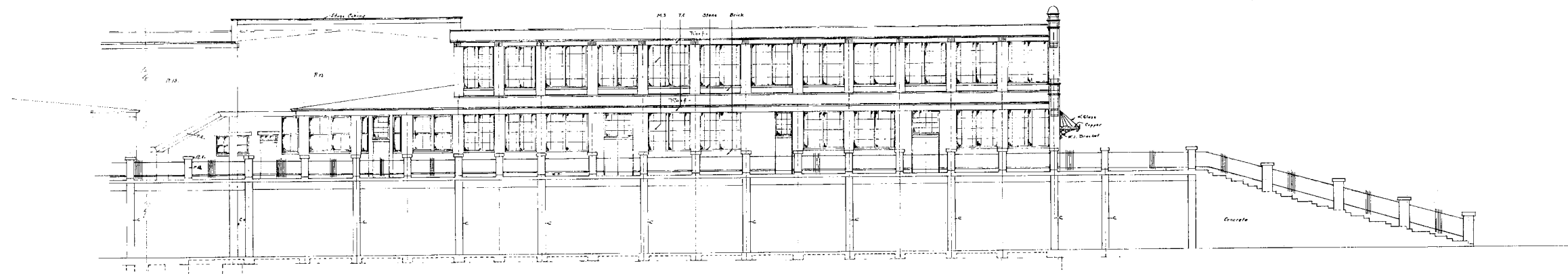


FRONT ELEVATION

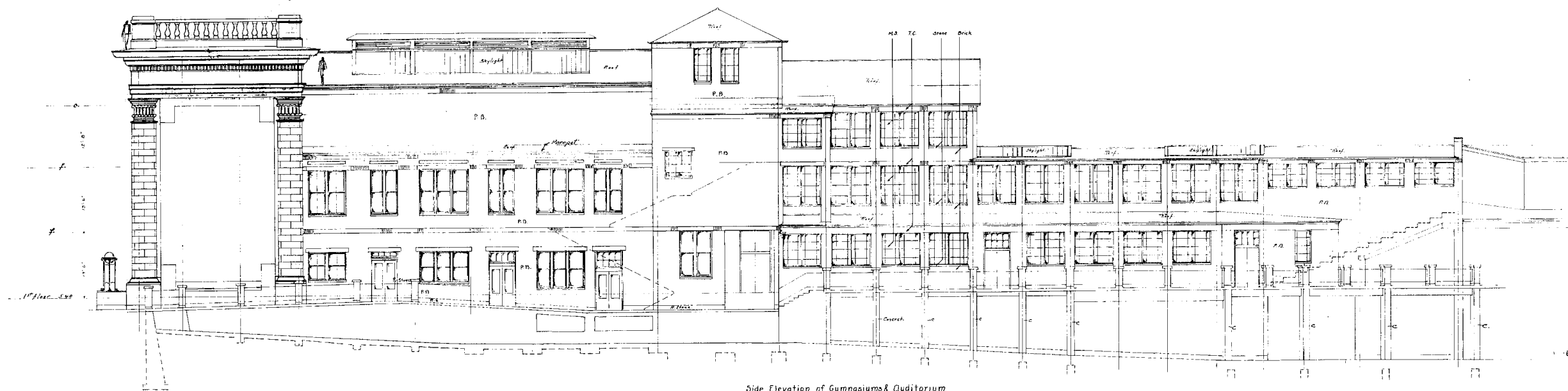
ELMER L. MEYERS HIGH SCHOOL  
 WILKES-BARRE CITY SCHOOL DISTRICT  
 WILKES-BARRE, PA.  
 DA. 1947 GRA. 16-170



Elevation of Shops, Cafeteria & Stadium



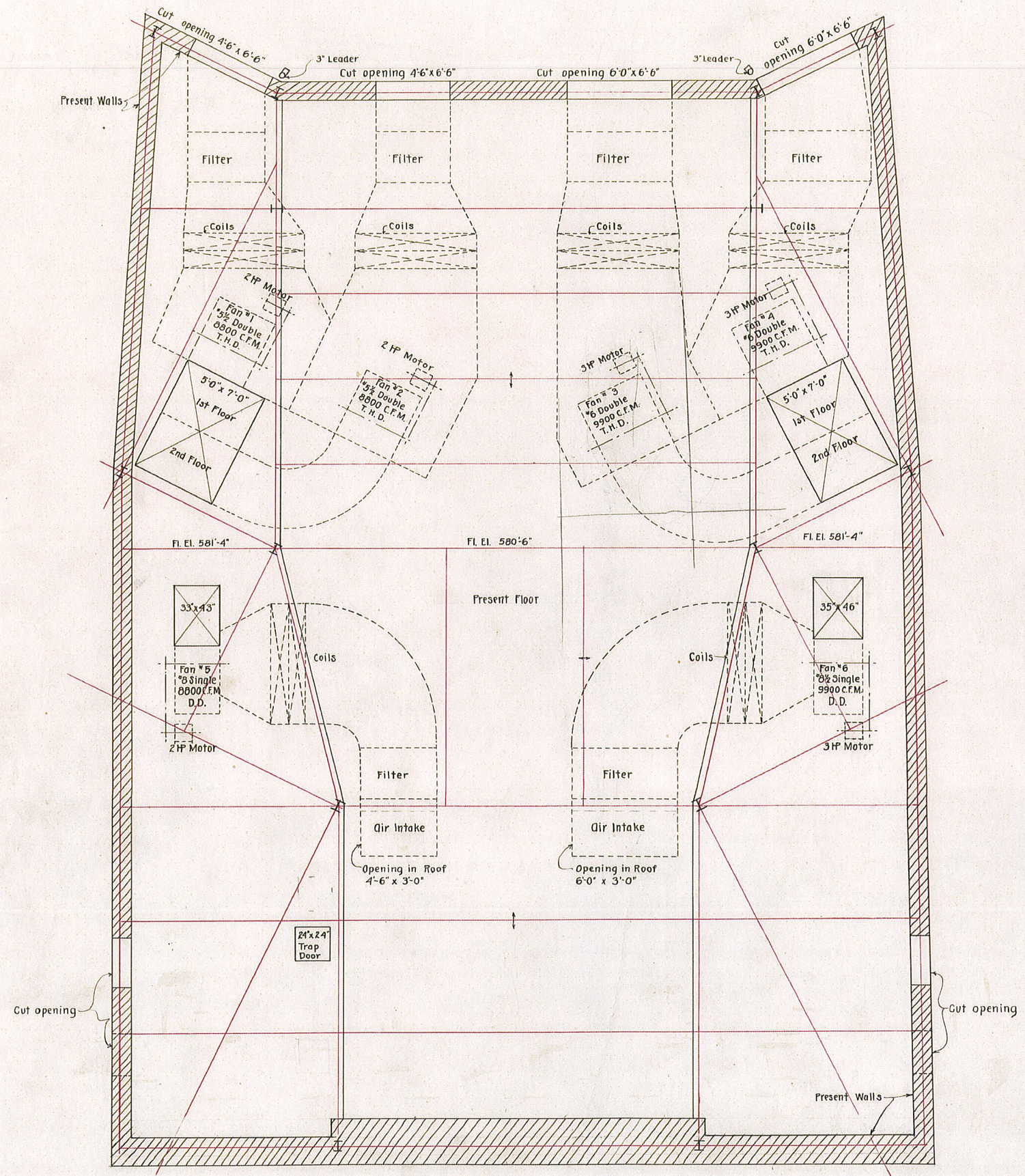
Side Elevation of Cafeteria & Stadium



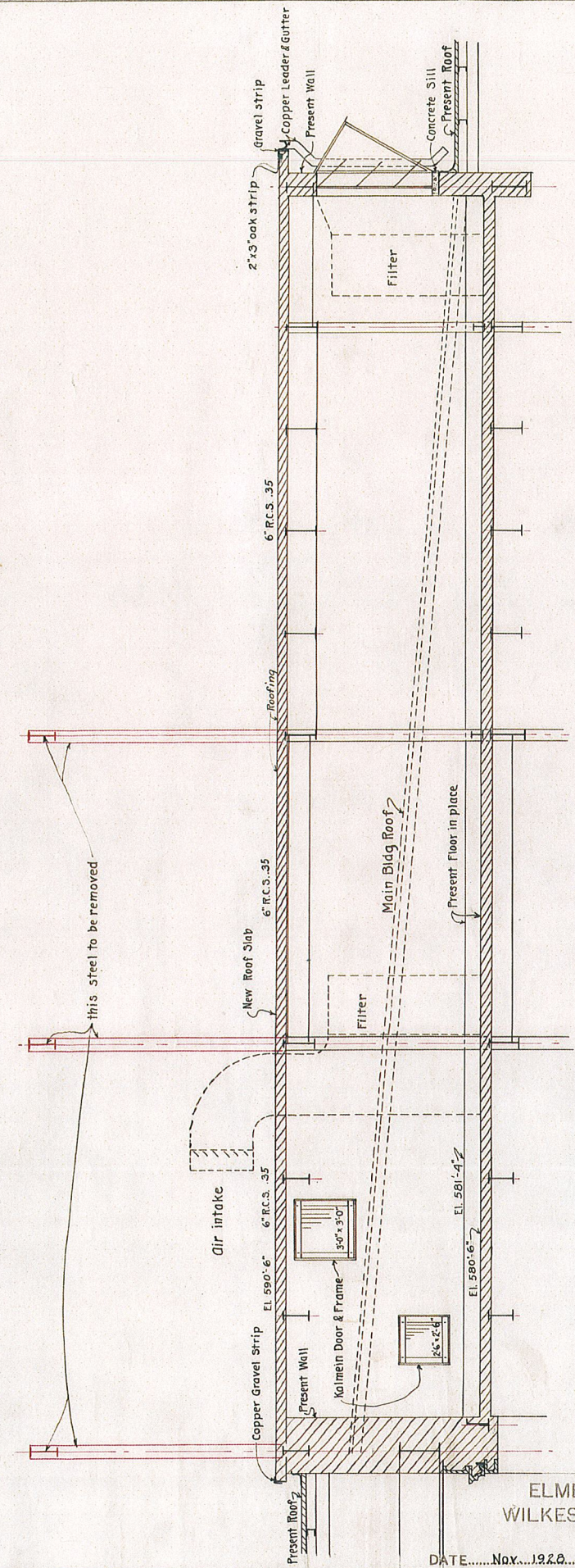
Side Elevation of Gymnasium & Auditorium

ELMER L. MEYERS HIGH SCHOOL  
WILKES-BARRE CITY SCHOOL DISTRICT  
WILKES BARRE, PA.  
JAN. 1937.  
SCALE 3/16"





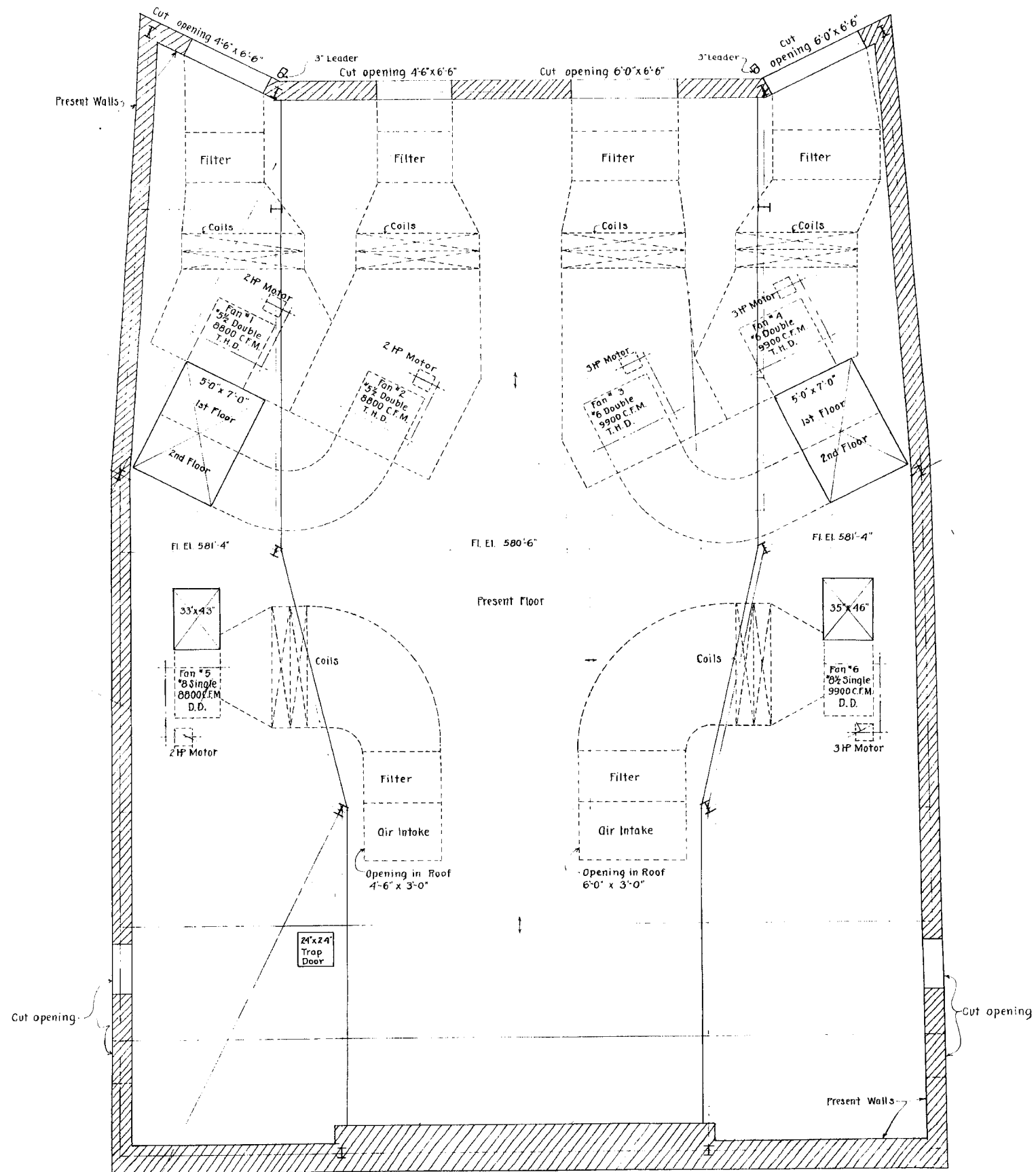
Fourth Floor of Main Building



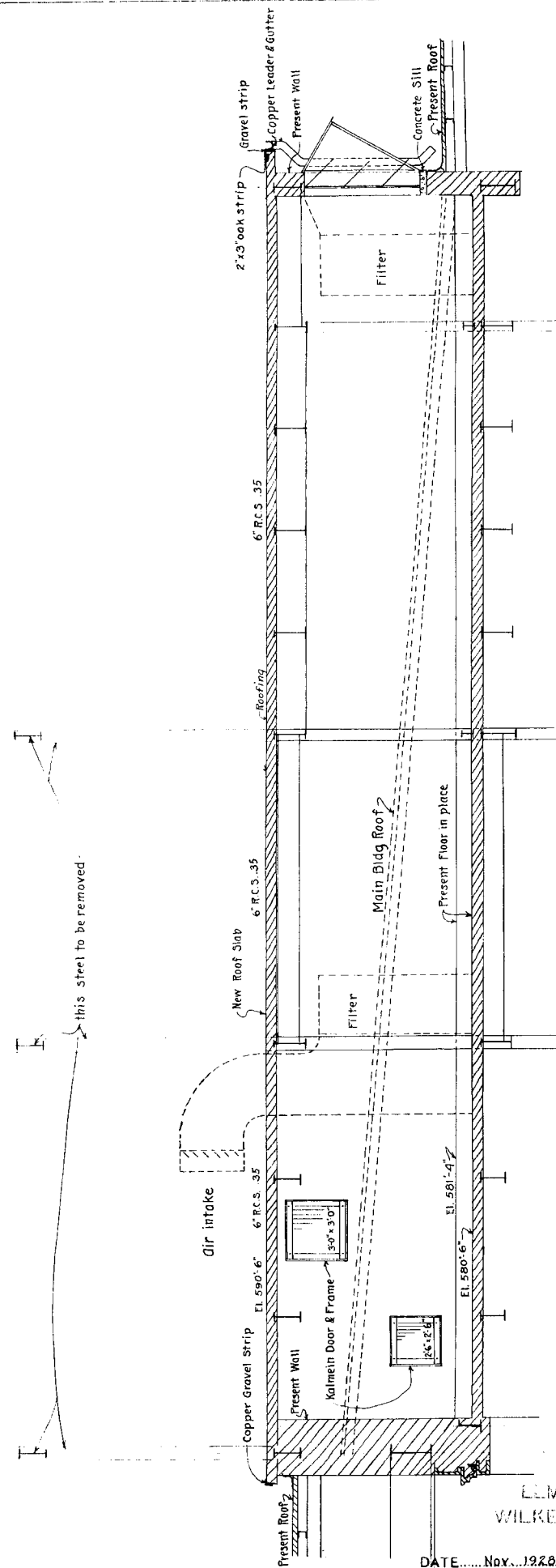
ELMER L. MEYERS HIGH SCHOOL  
WILKES-BARRE CITY SCHOOL DISTRICT  
WILKES-BARRE, PA.

DATE Nov. 1920 SCALE 1/4" = 1'-0"



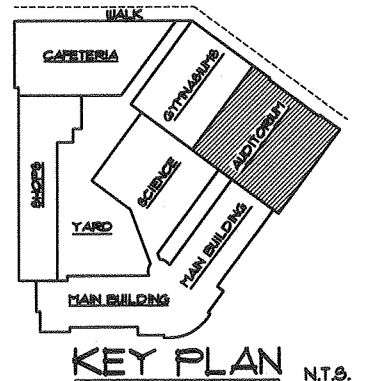
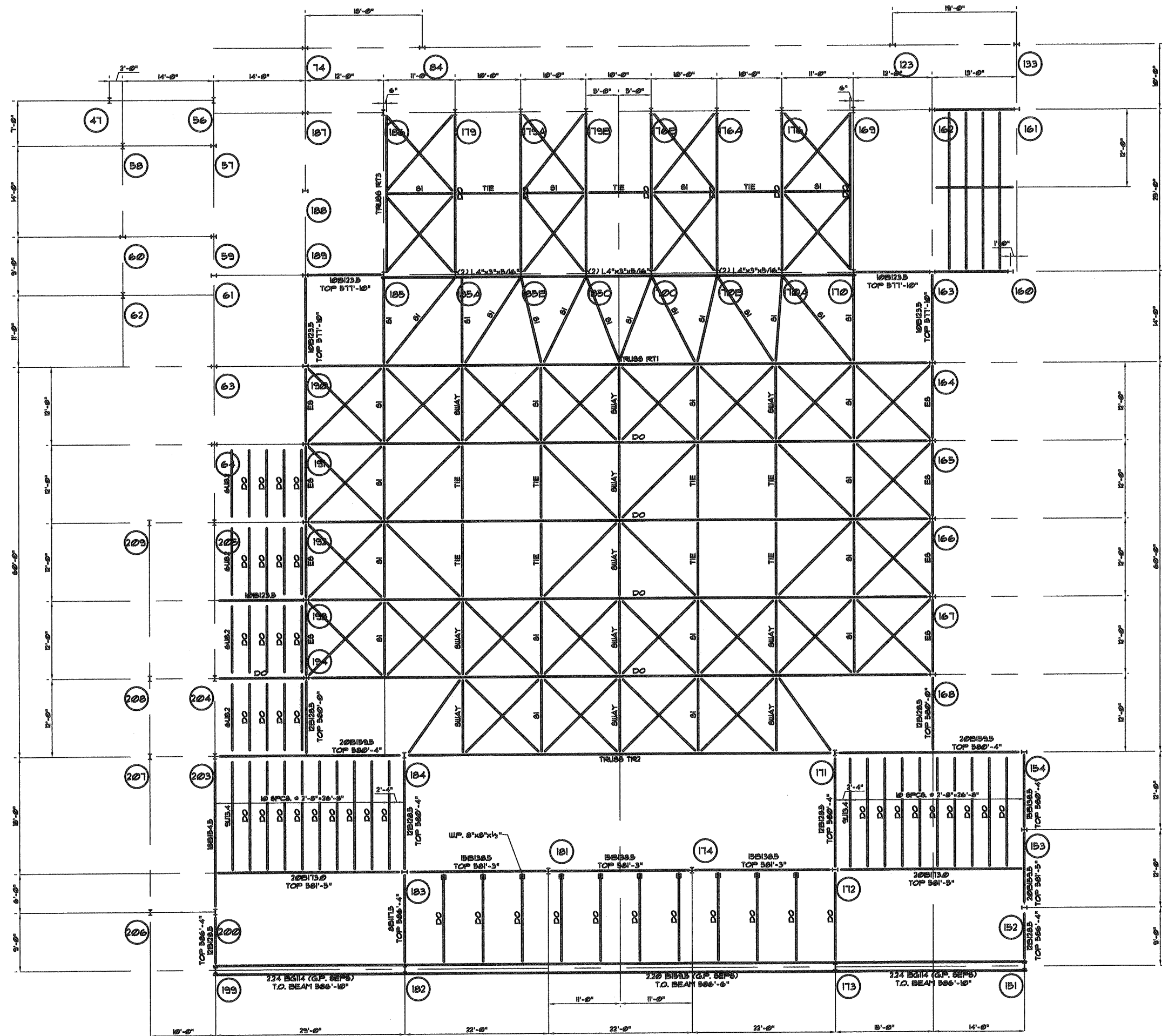


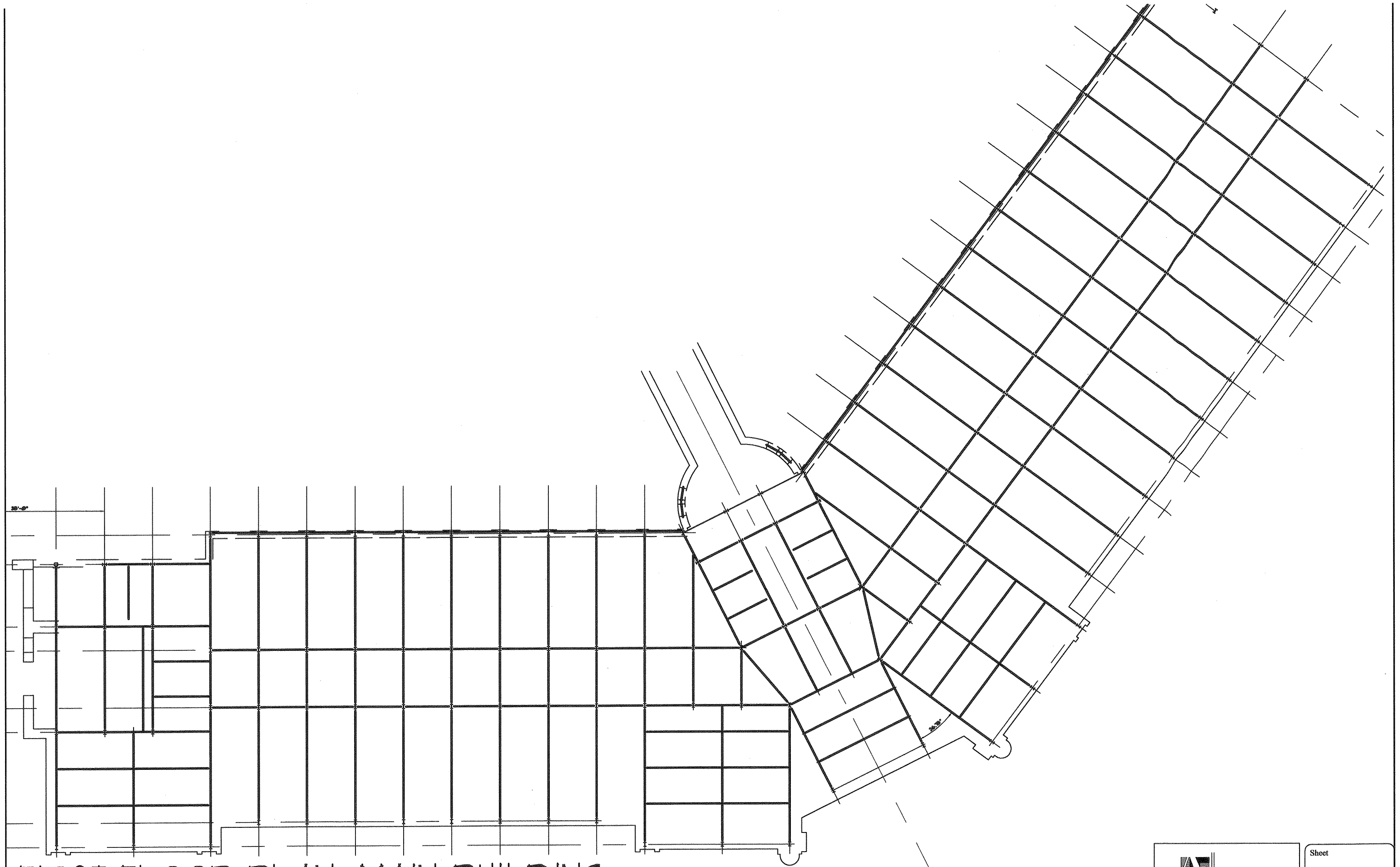
Fourth Floor of Main Building



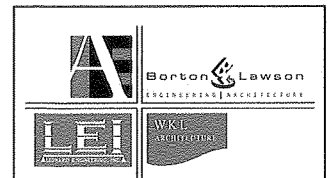
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WILKES-BARRE, PA.

DATE Nov. 1928 SCALE 1/4" = 1'-0"



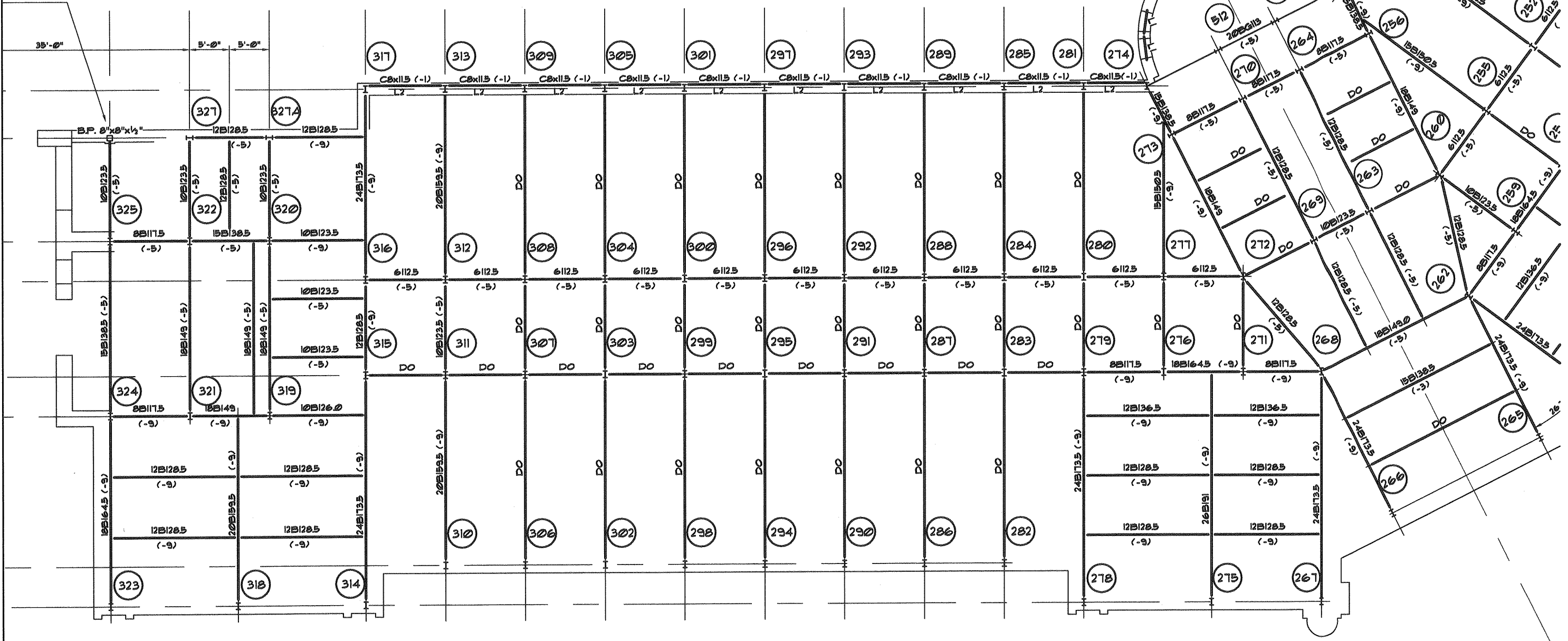


FIRST FLOOR PLAN: MAIN BUILDING



Sheet

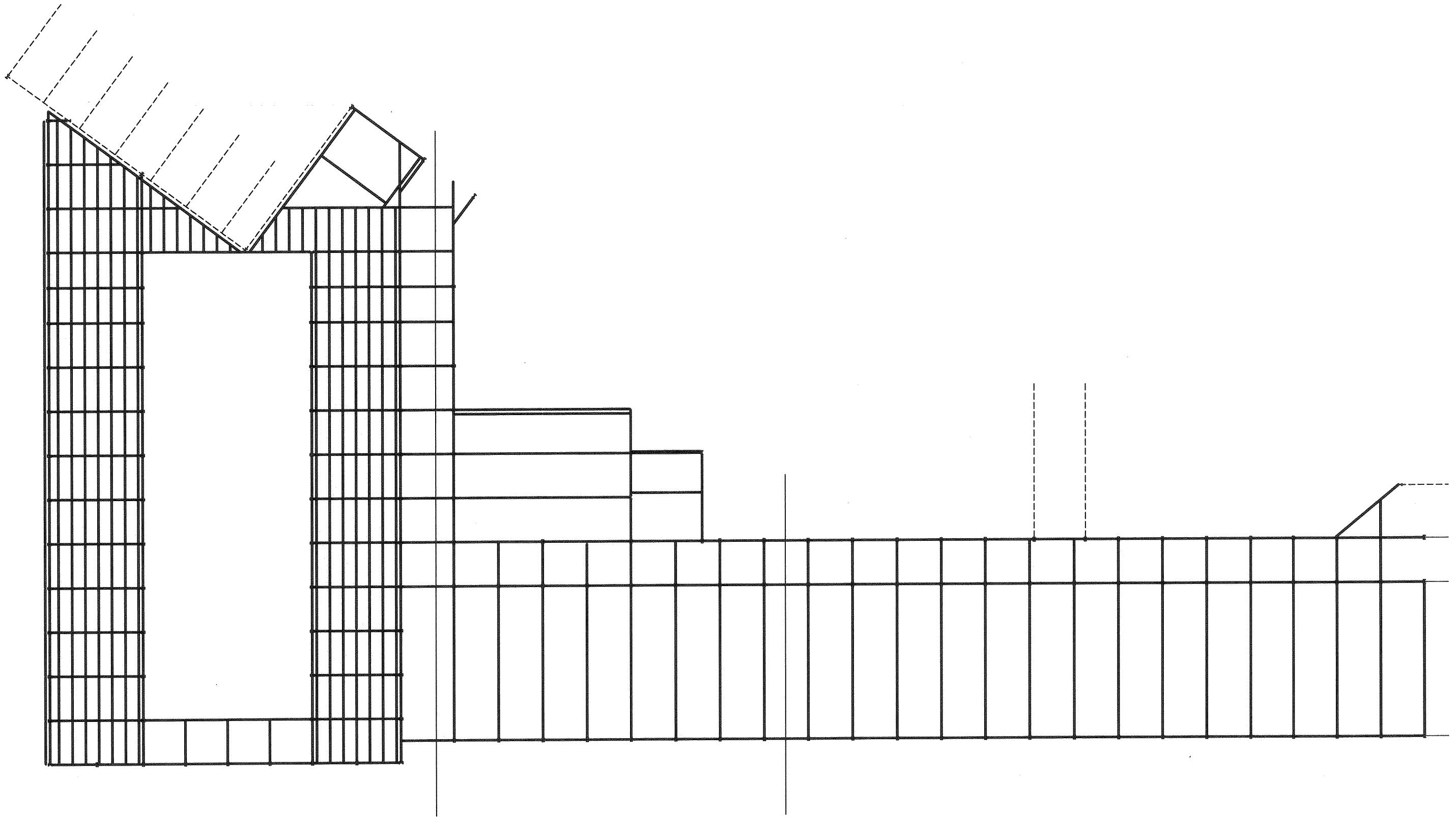
L1 = LOOSE LINTEL  
 (1) - C8"x11.5"x3'8"  
 (1) - L3 1/2"x3"x3/8"-3'8"  
 L2 = (1) - L3 1/2"x3"x3/8"

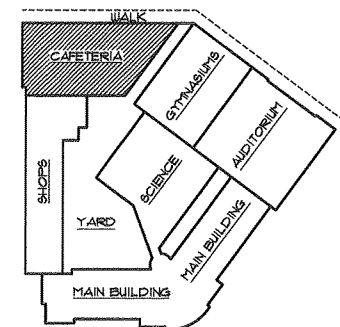
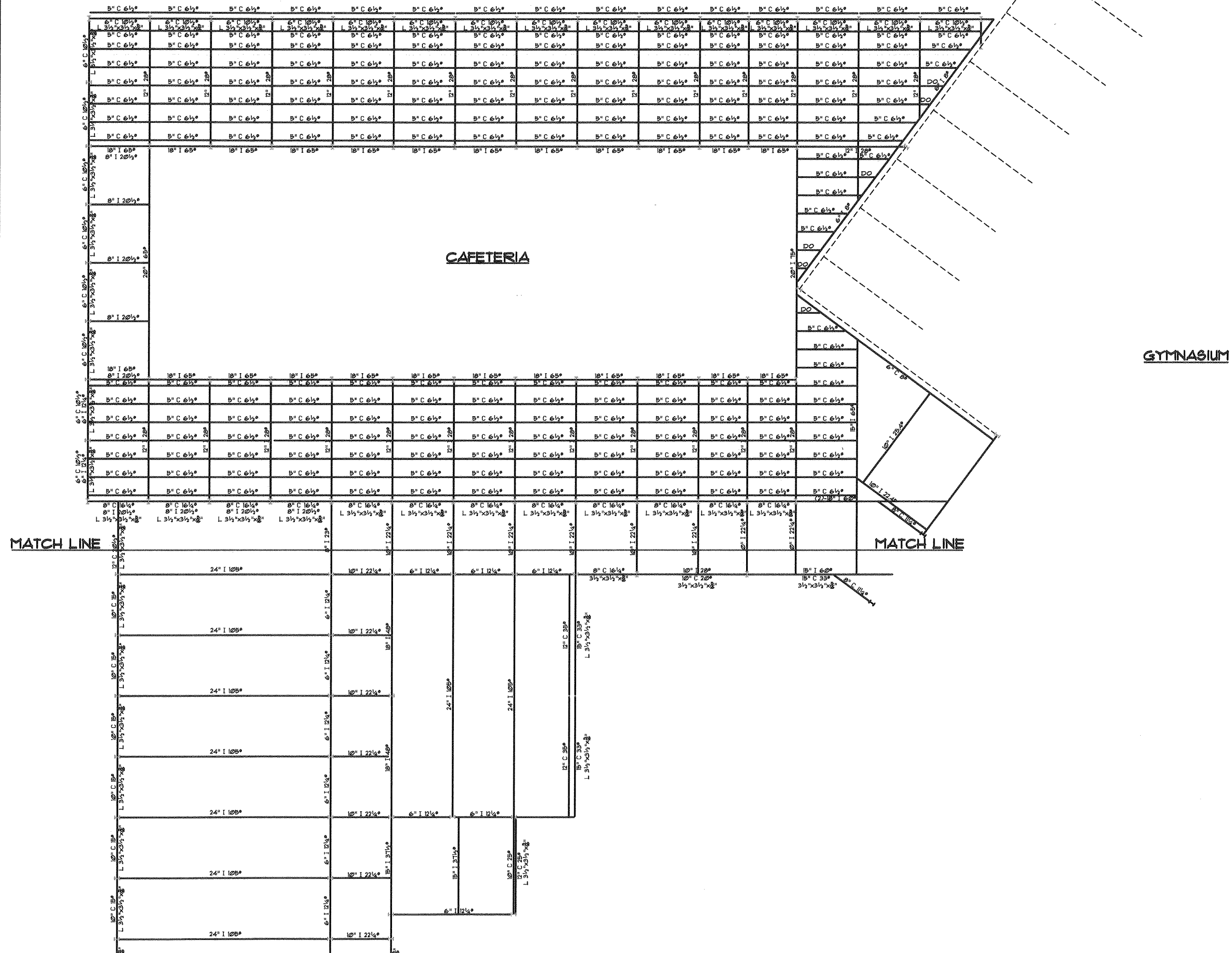


# FIRST FLOOR PLAN: MAIN BUILDING



# FIRST FLOOR PLAN: MAIN BUILDING





**KEY PLAN** N.T.S.

**Barton & Lawson**

ARCHITECTS

Sheet

**LEI**

ARCHITECTS

**WRL**

ARCHITECTS



MATCH LINE

MATCH LINE

SHOPS

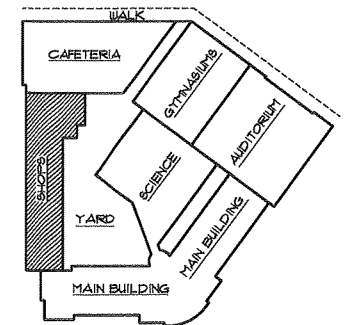
MAIN BUILDING

CAFETERIA

MATCH LINE

MATCH LINE

SHOPS



KEY PLAN N.T.S.

Sheet



## **APPENDIX B - PHOTOGRAPHS**

**See Appendix C for additional façade/exterior photos**



Main Building roof framing visible in attic above Hanover Street



Main Building roof framing visible in attic above Hanover Street side.



Tie-in of Hanover Street classroom wing to main entrance



Main Building attic above entrance and central staircase. Note steel beams and column exhibit little to no corrosion.





Main Building mechanical room over front entrance. Hole in brick wall allows access to ceiling in Architrave above entrance landing.



Ceiling and roof in architrave above main entrance



Old mechanical equipment in Mechanical room above Main Building staircase. Floor slopes down 12" to brick wall beyond.



Steel roof beam to girder connection in mechanical room above Main Building staircase





Door leading into basement level of Main Building staircase. Note the slope of the brick above door.



First floor interior corner at entrance to Main Building staircase from corridor running parallel with Carey Avenue. The Floor slopes 12" from front to rear in this area.

Note lack of cracking in corner finishes.



Mass concrete retaining wall  
below base of exterior brick  
wall.





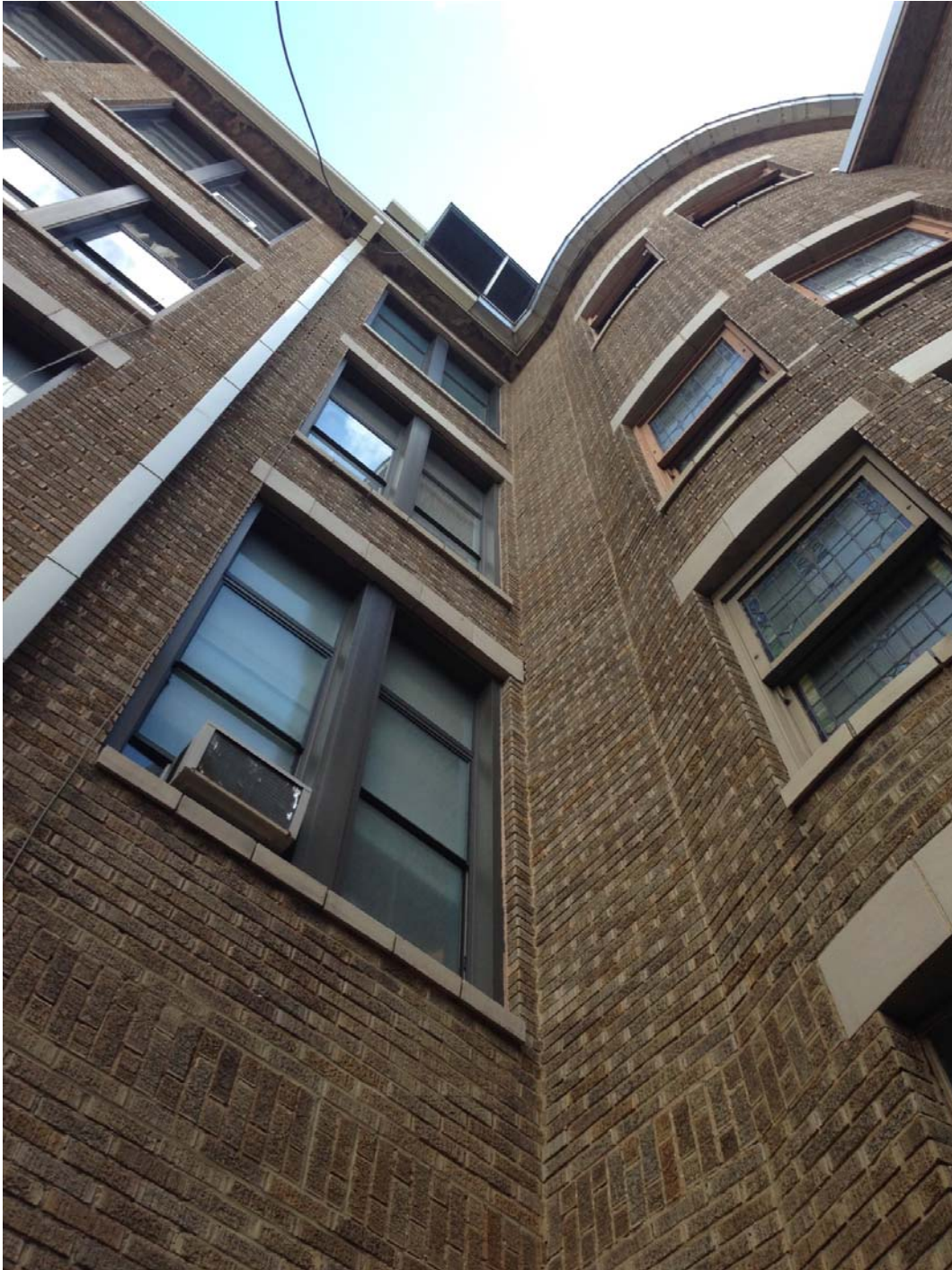
Cracking in basement archway near large pipe penetrations.





View from interior courtyard between girls' pool and Main Classroom Building. Circular wall is rear portion of main entrance staircase wall. Note lack of cracking in vertical corner joints.

Wall on left slopes 6" from Auditorium to stair wall. Corner at stair is low point in area where floors have settled 12" from front to rear.



Corner is low point where floors have settled 12" from front to rear. Auditorium is to left in this view.

Survey of brick mortar joints along classroom wall shows 6" drop in joints from Auditorium to stair wall.





Base of exterior wall of small addition adjacent to Girls Pool. The concrete masonry unit (CMU) backup wall and different color brick indicate this structure was not built as part of the original construction. The exterior face of the backup foundation wall has been severely damaged by apparent water infiltration.



Interior courtyard view where Main Building classroom wing ties into auditorium area. Classroom wing wall drops 6" from this corner to tie in with Main Building staircase.





Damaged walkway guardrail. Bottom connection is broken and moves easily. Many other rails are corroded and additional connections may fail in the near future.



Corroded steel beam under the north end walkway. Fireproofing on this beam has failed and fallen from beam. Remnants of the fireproofing remain on the beam web with moss growth, indication high moisture content.





Damage to ceiling above Courtyard tunnel entrance under the north end walkway.



Walkway at Cafeteria exit. Note pieces of fallen terra cotta on abandoned portion beyond fence. Protective cover is currently being installed above the doors.





Hanover Street entrance ramp to maintenance room. It is recommended this area is cordoned off due to loose brick above concrete retaining wall.



Columns under abandoned walkway at former stadium bleachers. Steel is in stable condition. Loose brick has been in place for extended time and it is recommended it is stabilized or the area under the walkway cordoned off.





Entrance door in interior Courtyard contain has deteriorated terra cotta above. Protective structure is currently being built over ramp.



Heating Plant north wall above driveway. Note loose brick in center. This an several others were easily removed prior to the installation of a protective netting system.





Heating Plant protective netting over Courtyard and Stadium access driveway.



Retaining wall at driveway adjacent to Heating Plant





Deteriorated section of brick wall adjacent to Old River Road stadium entrance prior to removal in Summer of 2014



Deteriorated brick wall removal near Old River Road stadium entrance. The section was replaced with chain link fencing.

**APPENDIX C -  
MASONRY PRESERVATIONS SERVICES, INC.  
FAÇADE EVALUATION**



7255 Old Berwick Road  
Bloomsburg, PA 17815

(570) 752-3607 ♦ FAX: (570) 752-7413  
masonrypreservation.com

December 3, 2014

Leonard Engineering  
251 Mundy Street, Suite C  
Wilkes-Barre, PA 18703

**ATTENTION: Thomas Leonard**

**SUBJECT: Cursory Evaluation**  
Elmer L. Meyers Junior / Senior High School  
*Wilkes-Barre, PA*  
MPS Project No.: 201440

Masonry Preservation Services, Inc. (MPS) has completed the cursory evaluation as outlined in our Proposal dated September 8, 2014. The intent of this work was to review the condition of the exterior masonry and determine areas of potential causes of masonry deterioration.

Based on our observations, there were multiple deficiencies leading to extensive recommended repairs. The observed deteriorated masonry wall components, coupled with exposed conditions, allowed the deterioration to accelerate. The constant presence of water trapped within the wall caused by a lack of flashing and failed mortar joints lead to the corrosion of the embedded steel members. The corroded steel, and the associated volume expansion caused by the exfoliating rust (leafing), in turn caused cracking and spalling of the terra cotta pieces.

We have recommended several repairs to address the observed deficiencies. We recommend the following budget costs to complete the repairs:

Mobilization / Access / General Conditions	\$1,800,000
100% Repointing of the Stone and Brick Mortar Joints	\$2,600,000
Replacement / Cleaning and Coating Corroded Steel Columns	\$700,000
Terra Cotta Window Header Replacement	\$4,800,000
New Coping Flashing	\$200,000
Selective Terra Cotta Replacement	\$500,000
Limestone Patching	\$20,000
Replacement of Building Sealants	\$280,000
Restoration Cleaning	\$220,000

**TOTAL: \$11,120,000**

The mechanical / boiler building needs replacement. This was not included in our budget. We also did not include any costs for replacement of the windows.

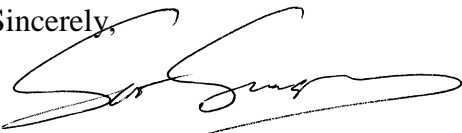
If repairs are not completed, a detailed evaluation should be completed in accordance with ASTM E-2270, "*Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions*" and ASTM E-2841, "*Standard Guide for Conducting Inspections of Building Facades for Unsafe Conditions*". This would provide a plan for keeping the school in service. Recommended repairs and associated pricing would be included as part of the evaluation.

The deterioration of the building has caused numerous terra cotta cracks and spalls to develop, creating a safety concern. The cracking and spalling on the terra cotta can be expected to increase exponentially as the corrosion of the supporting steel members accelerates. We observed several locations where spalled pieces of terra cotta have come loose from the building and fallen to the ground. The potential for this to occur exists at every elevation on the building, and will continue to increase with time.

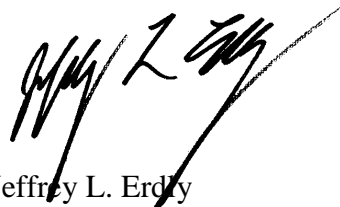
We recommend that the school district install containment netting, construct overhead protection or cordon off, at a minimum, the areas identified in this cursory evaluation. A continued semi-annual review of the façade should be completed as long as the structure remains in use.

We look forward to your review of this report.

Sincerely,



Scott Siegfried  
Project Manager



Jeffrey L. Erdly  
CEO







# Elmer L. Meyers Junior / Senior High School

## Cursory Evaluation

MPS Job No.: 201440

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### APPENDICES

- APPENDIX A: Photographs
- APPENDIX B: Drawings
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## 1.0 GENERAL INTRODUCTION

The Elmer L. Meyers Junior Senior High School is a mass masonry building located in Wilkes-Barre, PA. Built circa 1930, the building has a brick veneer with decorative terra cotta elements and large punched window openings. MPS was hired by Leonard Engineering to complete a cursory masonry evaluation of the building façade. The intent of the cursory evaluation is to provide recommendations for what would be required for masonry restoration of the exterior walls including an order of magnitude budget estimate. MPS provided a revised proposal dated September 8, 2014 which was approved, forming the basis of this review. This was a cursory evaluation, without any exploratory probes, and was not in accordance with ASTM E-2270, “*Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions*”, ASTM E-2841, “*Standard Guide for Conducting Inspections of Building Facades for Unsafe Conditions*” and ASTM E-2128, “*Standard Guide for Evaluating Water Leakage of Building Walls*”, the industry standards used to complete detailed evaluations of masonry facades. Our review did not include interior investigation with regard to corrosion of structural members which support the building’s floor and roof loads.

During the fall of 2006, MPS assisted the Apollo Group and subsequently teamed with Sutton Kennerly Associates regarding the masonry parapets and terra cotta cornice. This work identified extreme structural problems with these parapet assemblies, which were eventually removed from the structure as part of a comprehensive roof replacement project completed circa 2008. During this effort the building’s main masonry chimneys were rebuilt due to extensive deterioration identified.

## 2.0 OBSERVATIONS

Observations were completed from grade, from several of the roofs of the building and from an 80-foot boom lift. The attached photographs provide additional information regarding our observations on-site. Following is a summary of our observations:

### 2.1 Brick

The veneer of the exterior walls is comprised of multi-colored wire-cut brick with predominately tan shades. The brick was installed in a running bond pattern. The brick appeared to be in fair condition; however, we did note several areas of cracked brick. Many of the cracks appeared at the brick piers between the window bays. The brick mortar joints were in poor condition. There were multiple locations of cracked, debonded, missing and weathered mortar joints on each elevation of the building. The condition of the mortar was noticeably worse at the higher elevations. There were several generations of mortar on the building. It appears that at one time a majority of the upper most brick mortar joints were repointed / grouted and throughout the history of the building there were multiple spot pointing projects completed. Upon close examination, we found that many of mortar joints had been repointed with a very thin mortar joint. We observed several locations where the repointed mortar joint was less than 1/8-inch thick. A thin joint does not promote long-term performance of the mortar and will allow a higher rate of water infiltration into the exterior wall.



Mass masonry walls use the girth of the wall to absorb water during periods of rain and then dry out through evaporation between periods of rain. As the exterior masonry continues to deteriorate, the wall absorbs more water during each period of rain. The additional water infiltration increases the rate of deterioration and the cycle continues. This typically leads to interior leaks and significant deterioration of the masonry.

Properly functioning mortar joints are vital in mass masonry walls. The mortar joints reduce the amount of water that infiltrates the wall which allows the wall to dry between periods of rain. If the wall cannot dry out and remains constantly saturated the rate of corrosion of embedded steel increases and the assembly becomes susceptible to freeze-thaw and sub-efflorescence damage. Sub-efflorescence damage occurs when masonry wall assemblies retain liquid water in sufficient quantities and duration to allow for the entrapped moisture to solubilize salts. When the liquid water eventually evaporates, the solubilized salts recrystallize at or near the masonry surface. This process causes an expansion of the salts that can cause wide spread spalling of individual units. We noted efflorescence staining on many sections of brick near the top of the three and four story walls sections.

According to the BIA Technical Notes 18 and 18A, brick is at its smallest when it first comes out of the kiln and then grows over time due to irreversible moisture expansion. Brick will draw moisture from its environment which causes it to expand (initial irreversible growth). The majority of the expansion occurs in the first few months, but brick can continue to expand for several years. Brick also expands and contracts due to temperature changes. Vertical and horizontal expansion joints are used to accommodate these brick movements. Vertical expansion joints consist of full-depth joints in the brick veneer filled with sealant to prevent water infiltration. There were no vertical expansion joints installed in the brick veneer. While this is typical for mass masonry buildings of this age, that does not mean they may not be warranted. The stress caused by the movement of the brick appears to be contributing to some of the observed cracking.

Another contributing factor to the cracking appears to be the structural steel embedded in the masonry. A large percentage of the cracks appear at the piers between the window bays. There are steel columns locations behind each pier. As the exterior walls continue to degrade they allow a greater amount of water infiltration. This exposes the steel columns to an increased amount of moisture which accelerates the rate of corrosion of the steel. Corrosion threatens any ferrous metal (iron, steel, etc.) component, particularly where embedded and in direct contact with adjacent material (brick, mortar, etc.). In the presence of water, steel will corrode (rust), and expand with significant pressure created by the exfoliating rust (the corroding steel combined with exfoliating rust will occupy a larger volume of space than the original steel alone). The pressure generated produces stress in adjacent materials, oxide jacking, and in many building assemblies, can cause severe damage. Many of the cracks appear to be caused by the corrosion of the steel columns.



## 2.2 Terra Cotta

There are numerous terra cotta elements on the building, including copings, cornices, architraves, column capitals, window headers, window sills and decorative panels. The front elevation of the building originally had a large terra cotta cornice with a tall parapet. Due to the amount of water that was entering the exterior walls through the parapet and cornice, they were removed and the top of the exterior walls were roofed over. We noted that there were several spalls on the terra cotta pieces just below the metal drip edge of the new roof system. Many of the spalls were held in place by the metal drip edge.

The architraves and column capitals have previously been evaluated by MPS, see the attached report regarding the architraves in *Appendix B: MPS Architrave Evaluation*.

The condition of the remaining terra cotta elements varied greatly around the building. The copings, which were still present on the rear elevations of the building, were in fair condition; however, we noted there was no through-wall flashing below them. Without flashing, large quantities of moisture pass through the copings and enter the exterior wall. We noted that under the copings the deterioration of the mortar was more advanced and the presence of efflorescence was more visible. The joints between the copings had been sealed multiple times, but there were still numerous failures in the sealant.

The window headers were cracked at many locations. Most of the cracks were horizontal across the face of the headers and often times they extended through two or more pieces. Some of the cracks had grown large enough that small pieces of the terra cotta had come loose and fallen to the ground. The header pieces were typically supported by the steel beam above, either directly or hung from steel wires. At some locations we found pins between the pieces. The terra cotta pieces were all grouted. It appears that the corrosion of the steel members supporting the terra cotta is causing the cracking. We also noted several pieces had surface spalls along the vertical mortar joint; often times the spalling occurred on both sides of the joint. This appeared to be from thermal movement of the pieces. When the pieces expand during high temperatures they are generating stress at the edges of the terra cotta units, leading to the surface spalls we observed. This stress is usually accommodated by the lower strength mortar joints, but that does not appear to be happening. It could be that the mortar joints are too narrow or the repointing mortar used may have too high a strength.

The brick sills were in fair condition. We noted one sill with a large spall, and some other sills with some cracking, but the majority of the sill pieces did not show any signs of significant deterioration. The mortar joints between the sill pieces had failed at most of the sills we observed. We did not observe any evidence of flashing below the coping pieces. The failure of the mortar joints and lack of flashing allows excessive water infiltration into the masonry below.

The decorative panels were in good condition, with only a few minor cracks noted.





The cracking and spalling on the terra cotta can be expected to increase exponentially as the corrosion of the supporting steel members accelerates. We observed several locations where spalled pieces of terra cotta have come loose from the building and fallen to the ground. The potential for this to occur exists at every elevation on the building, and will continue to increase with time. We have provided the drawing SK-1 in *Appendix B: Drawings*, to highlight three locations where significant spalls exist above locations that are regularly traveled by pedestrians. To be clear, every elevation on the building has several pieces of terra cotta that are cracked or spalled, and a piece of terra cotta could come loose from anyone of these locations at any time and fall to the ground. The locations we highlighted are areas with a higher potential for a piece to fall which are near pedestrian traffic.

### 2.3 Limestone

The base of the building on the front elevation consists of limestone, and below both of the architraves there are large limestone columns. The limestone appears to be in fair condition. We did not note any deterioration on the columns, but the base stone was cracked at several locations. The cracking appeared to be from thermal movement or the weight of the masonry above. We also noted some locations of surface spalling caused by the use of snow melting salts, which get absorbed into the stone.

### 2.4 Sealants

Sealant was installed at the window and door perimeters. The existing sealants were cracked, hardened, split open, de-bonded, squeezed-out, and generally deteriorated at several areas. Many openings were wide enough to easily allow water to pass.

### 2.5 Additional Items

The building that houses the mechanical equipment / boiler was severely deteriorated. The brick and mortar was so deteriorated the brick were coming loose from the wall, requiring the installation of an extensive netting system. It is our opinion the deterioration is so extensive the building is beyond repair and requires replacement.

The original windows on the building were steel framed. At many of the windows, particularly on the front elevation, new retrofit windows were installed. The new retrofit windows are aluminum and were installed over the steel frames of the original windows. During construction of the building it was common practice to lay the brick exterior wall around the steel frame of the windows and then use the frame as temporary support for the terra cotta until the grout in the terra cotta can cure. It appears this procedure may have been used at this building. When the windows are replaced, extensive masonry work will be required to properly remove the original frames.



### 3.0 RECOMMENDATIONS

The Elmer L. Meyers Junior / Senior High School has functioned for over 80 years with little maintenance to the exterior façade, but the lack of maintenance has allowed advanced deterioration to develop which will require substantial repairs to address. To restore the exterior walls to function for the next 25 years, we would recommend the following repairs.

#### 3.1 Access & Site

In order to safely and effectively access the different work areas, heavy-duty frame scaffolding will need to be utilized. The scaffolding will need to be integrated with overhead protection at all of the many entrances to the building. Significant coordination and scheduling efforts will be required due to the use of the building and the noise and dust that will be generated during the project. Due to the extent of rebuilding required it may not be possible to use the school during rebuilding.

#### 3.2 Masonry Repointing

100% repointing of the masonry mortar joints is required.

- Repointing should be completed in accordance with applicable portions of the Brick Industry Association guidelines.
- Cut out areas of deteriorated mortar consistently and comprehensively. Ensure that all existing mortar is removed from the edges of the masonry units.
- Install new mortar applied in multiple layers/lifts as each previous layer becomes “thumbprint” hard, tool the last layer to match the original mortar joint profile.
- Complete a restoration cleaning of the masonry surfaces to remove atmospheric pollution and improve appearance.

#### 3.3 Structural Steel Column Replacement / Cleaning and Coating

The corroded structural steel columns at the front elevation of the building should be exposed and either replaced or cleaned and coated.

- Remove brick as necessary to expose full height of the structural steel columns at each brick pier.
- Provide engineering evaluation of the exposed steel columns to determine if the steel needs to be replaced or can be cleaned and coated with a rust preventative coating system.
- Install new brick to match the profile of the original building.
- Complete a restoration cleaning of the masonry surfaces to remove atmospheric pollution and improve appearance.



### 3.4 Terra Cotta Window Header Replacement

The window headers will continue to crack and spall as the support steel corrodes. The terra cotta should be removed and replaced with a material that can provide a similar look, like precast concrete.

- Remove all existing terra cotta window headers and dispose of off-site. Install shoring to support the brick veneer above.
- Provide engineering evaluation of the exposed steel elements to determine if the steel needs to be replaced or can be cleaned and coated with a rust preventative coating system.
- Design and install new through-wall flashing, where appropriate, to protect the steel and properly manage future water infiltration into the wall assembly.
- Install new replacement material to match the original profile of the façade. This will require the design of a new stainless steel anchoring system. The mortar used to relay the pieces should match the characteristics of the original building mortar.

### 3.5 Coping Flashing Installation

The lack of flashing below the copings is directing water into the wall assembly, causing deterioration of the brick below. The copings should be removed, new flashing should be installed, and the copings should be reset.

- Remove all existing terra cotta copings and store on-site.
- Prepare the top of the walls for the installation of new flashing, including rebuilding brick sound masonry, extents will vary. We anticipate that large areas of rebuilding below the copings will be required.
- Install new stainless steel anchors.
- Install new through-wall flashing, incorporating thimbles (metal flashing pieces installed over the stainless steel anchors), at the top of the wall. Install the flashing with a proper drip edge on the outboard side of the parapet wall and a receiver connected to metal counter-flashing on the inboard side of the parapet wall.
- Re-install the existing terra cotta copings with recessed mortar head joints. Install silicone sealant in the recessed mortar head joints and finish the sealant flush with the outside face of the copings. Finish the bed joints with mortar to allow the assembly to breathe.

### 3.6 Selective Terra Cotta Replacement

Select cracked or spalled terra cotta units will need to be replaced.

- All deteriorated terra cotta pieces should be removed and disposed of off-site. Any masonry above a removed unit should be properly shored.
- Reinstall new terra cotta piece to match original.





### 3.7 Limestone Patching

The cracked and spalled limestone pieces should be patched.

- Saw-cut spalled areas square with a minimum ¼" edge at all sides to avoid feathered edges.
- Apply a proprietary repair mortar in accordance with the manufacturer's recommendations. Mock-up repairs should be completed to determine an acceptable color and finish for the patches.

### 3.8 Sealant Replacement

The sealants on the building are beyond their useful service life and have failed in many locations. All of the sealant joints on the building should be removed and replaced.

- Remove all existing sealant at windows and doors.
- Clean substrates and prepare surfaces in accordance with manufacturer's recommendations.
- Install backer rod or bond breaker tape and a new high quality silicone sealant.

We understand that the school may only be required to remain in service for a select period of time. If the school was to function for the next 5 years, the focus of the repairs would then shift to managing the safety of the pedestrians around the building and addressing leaks. We recommend that the school district install containment netting, construct overhead protection or cordon off, at a minimum, the areas identified in this cursory evaluation.

A building of this age and size should be regularly evaluated to ensure catastrophic failures of the façade are avoided. We recommend completion of a masonry façade evaluation in accordance with ASTM E-2270, "*Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions*" and ASTM E-2841, "*Standard Guide for Conducting Inspections of Building Facades for Unsafe Conditions*". Follow-up evaluation should then be completed on a semi-annual basis until the school is taken out of service. Repairs will be determined based on the findings in the report.



## 4.0 SUMMARY

MPS completed the cursory evaluation of the masonry façade at the Elmer L. Meyers Junior / Senior High School. The purpose of our evaluation was to perform an assessment of the masonry condition, identify the cause/causes of deterioration and provide recommendations for repairs to extend the structure's service life.

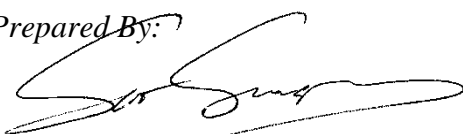
Multiple deficiencies were observed leading to extensive recommended repairs. The observed deteriorated masonry wall components, coupled with exposed conditions, allowed the deterioration to accelerate. The constant presence of water trapped within the wall caused by a lack of flashing and failed mortar joints lead to the corrosion of the embedded steel members. The corroded steel in turn caused cracking and spalling of the terra cotta pieces, particularly at the head of the windows.

Based on the observed deficiencies, extensive repairs will be required to address the deterioration and to implement industry recommended enhancements to help address some of the causal effects impacting the wall assembly. Proper detailing with quality materials in accordance with the Brick Industry Association and other standards are critical to the long-term effectiveness of the repairs. The following recommended repairs are intended to address deficient detailing and limit future deterioration:

- 100% repointing of the masonry mortar joints
- Replacement or cleaning and coating the corroded structural steel columns
- Repairs at the terra cotta window headers
- Installing new flashing under the terra cotta copings
- Selective replacement of deteriorated terra cotta pieces
- Patching of the spalled limestone
- Replacement of building sealants
- Restoration cleaning

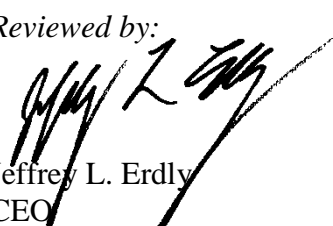
As requested, MPS performed this cursory evaluation of the exterior masonry façade, documented deficiencies, recommended repairs and a course of action, and provided this summary report for guidance. This report has been prepared based on our site observations, information presented to us, interviews with on-site personnel, and our experience with similar projects. If any information becomes available which is not consistent with the observations or conclusions presented in this report, please present it to us for our evaluation. ©2014 Masonry Preservation Services, Inc. (MPS). All rights reserved. The reproduction, distribution, publication, display, or other use of this report without the written consent of MPS is prohibited. The contents of this report are intended to convey information compiled by MPS as relevant to the project outlined within and for the agreed-upon intent, and for no other purposes.

*Prepared By:*



Scott Siegfried  
Project Manager

*Reviewed by:*

  
Jeffrey L. Erdly  
CEO

# **APPENDIX A:**

# **Photographs**





## Appendix A – Photographs

Elmer L. Meyers Junior / Senior High School  
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**Photograph 1:**  
Overview of the  
east side of the  
front elevation of  
the building.



**Photograph 2:**  
Overview of the  
south side of the  
front elevation of  
the building.



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**Photograph 3:**  
Overview of the  
architrave at the  
north end of the  
east elevation.



**Photograph 4:**  
Overview of the  
rear elevation that  
faces the stadium.





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**Photograph 5:**  
Typical courtyard  
elevation that  
comes to grade.



**Photograph 6:**  
Typical courtyard  
elevation that  
interfaces with a  
lower roof.







**Photograph 7:**  
Typical punched  
window opening  
with terra cotta  
header and sill  
pieces.

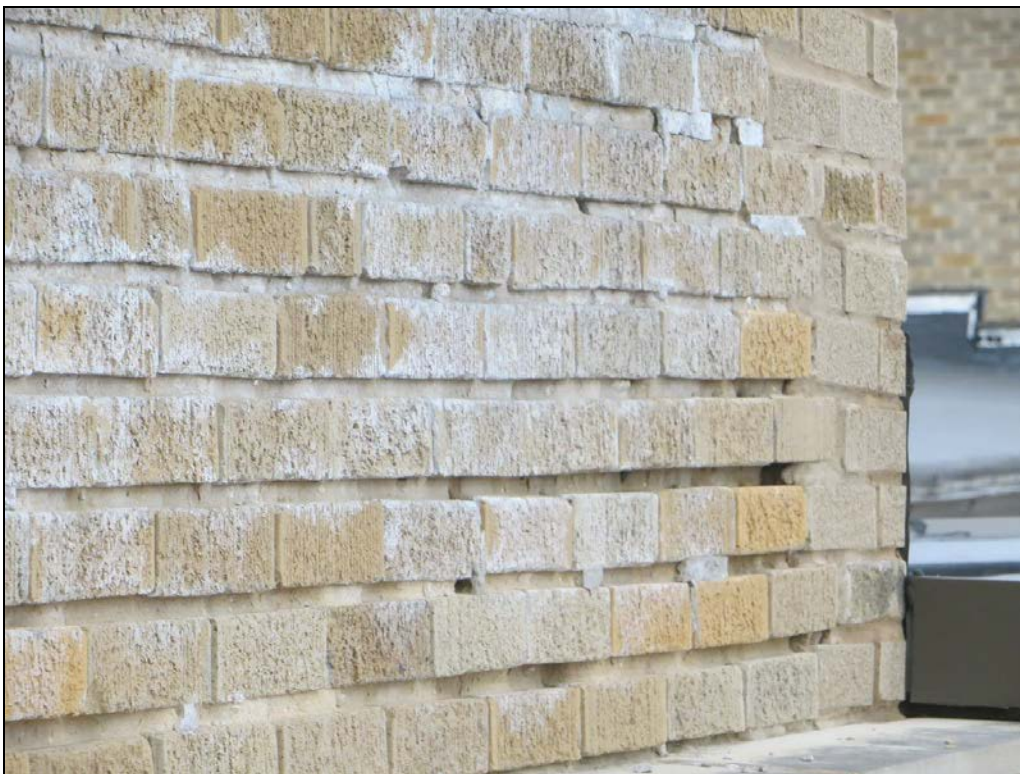


**Photograph 8:**  
View of a section  
of the removed  
parapet and cornice  
and the new  
roofing detail.



**Photograph 9:**

Failed and missing mortar joints at an area that was repointed.



**Photograph 10:**

Missing and weathered mortar joints at an area with heavy efflorescence.





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**Photograph 11:**

The majority of the repointed / grouted mortar is missing at this location.



**Photograph 12:**

Significant loss of mortar at the header joints at this brick section between windows.







**Photograph 13:**  
Open mortar joints  
at the top of an  
elevation.



**Photograph 14:**  
The original mortar  
at this location is  
substantially  
weathered with  
numerous failures.





**Photograph 15:**

At the locations where the mortar was repointed / grouted, the new mortar was installed with a thin profile, which provides little resistance to water infiltration.



**Photograph 16:**

Spot pointing was completed at random locations around the building.







**Photograph 17:**  
Cracked mortar at  
a transition in the  
brick wall which  
appears to have  
been caused by  
thermal movement  
in the brick.



**Photograph 18:**  
Cracked brick  
where the brick  
sets back at a  
window. This also  
appears to have  
been caused by  
thermal movement  
of the brick.







**Photograph 19:**  
Significant cracking at a brick pier may have been caused by corrosion of the embedded structural steel.



**Photograph 20:**  
The crack in the brick continues into the terra cotta.





**Photograph 21:**  
Spalled terra cotta piece at the top of the elevation. The piece has been poorly repaired and it appears the metal drip edge is the only thing preventing the spall from falling to the ground.



**Photograph 22:**  
This insipient spall could come loose at any time.





**Photograph 23:**  
This spall is loose  
and being held in  
place by the metal  
drip edge.



**Photograph 24:**  
A piece of terra  
cotta was found on  
the smaller terra  
cotta cornice  
below.





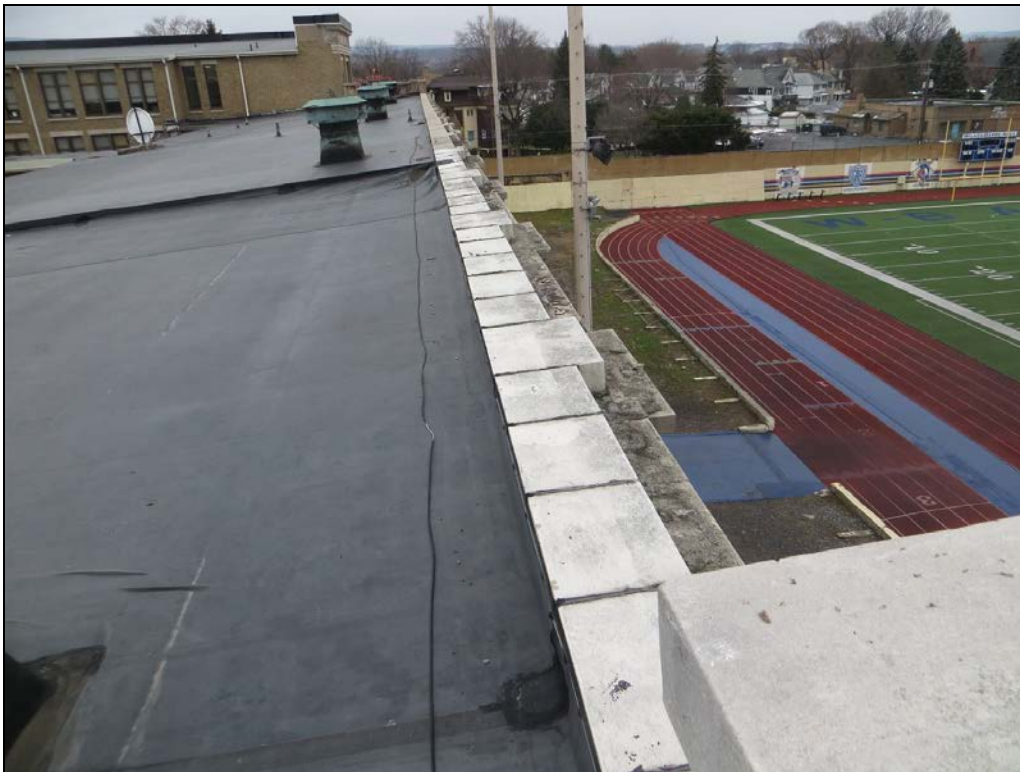
**Photograph 25:**  
Crack through the  
band of terra cotta  
near the top of the  
building.



**Photograph 26:**  
Cracked dentil  
piece that will  
eventually break  
loose and fall to  
the ground.



**Photograph 27:**  
Crack in a terra  
cotta panel.



**Photograph 28:**  
Typical terra cotta  
copings on the rear  
elevation of the  
building.





**Photograph 29:**  
There was no  
evidence of  
flashing under the  
copings.



**Photograph 30:**  
The joints between  
the coping pieces  
had been sealed  
multiple times, yet  
we still found  
multiple failures in  
the sealant.





**Photograph 31:**  
Cracked terra cotta  
window header  
pieces which have  
been sealed.



**Photograph 32:**  
Cracked window  
header piece where  
it is supported by  
brick.



**Photograph 33:**  
Crack on the  
underside of two  
terra cotta window  
header pieces.



**Photograph 34:**  
Multiple cracks in  
one window header  
piece.





**Photograph 35:**  
Crack across all  
five window  
header pieces.



**Photograph 36:**  
Spalled terra cotta  
window header  
piece.



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**Photograph 37:**  
Spall at corroded  
pin between pieces.



**Photograph 38:**  
Large spall and  
crack in the  
window header  
pieces. Additional  
terra cotta pieces  
will eventually fall  
to the ground.





**Photograph 39:**  
Spalls on both  
sides of a mortar  
joint.



**Photograph 40:**  
The spalling is an  
indication the terra  
cotta is  
overstressed,  
typically caused by  
thermal expansion.





**Photograph 41:**  
The joints between the window sill pieces were in poor condition.



**Photograph 42:**  
The failures of the window sill joints are allowing excessive amounts of water in the wall assembly.





**Photograph 43:**  
A cracked window  
sill caused by a  
corroded window  
frame.



**Photograph 44:**  
A terra cotta  
window sill with  
significant damage.



**Photograph 45:**  
Cracked limestone  
piece at the base of  
the building.



**Photograph 46:**  
Spalled limestone  
piece caused by the  
use of deicing  
salts.





**Photograph 47:**  
Failed sealant at the perimeter of a window. There were multiple generations of sealant present.



**Photograph 48:**  
This sealant joint had both adhesive and cohesive failures.





**Photograph 49:**  
All of the window sealant appears to have reached the end of its useful life.



**Photograph 50:**  
The failed sealants are allowing large amounts of bulk rain water to enter the exterior wall assembly.

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**Photograph 51:**  
Overview of the  
front elevation of  
the building that  
houses the  
mechanical  
equipment.



**Photograph 52:**  
Overview of the  
rear elevation of  
the mechanical  
building.







**Photograph 53:**  
Large spall in a  
terra cotta piece at  
the cornice on the  
building.



**Photograph 54:**  
The deterioration  
to the mechanical  
building was so  
severe extensive  
netting was  
required on the  
north elevation of  
the building.





**Photograph 55:**  
The original windows on the building are steel framed. These windows are still present at many of the rear elevations.



**Photograph 56:**  
New retrofit windows were installed over the original steel frames at many locations.



**Photograph 57:**  
Several of the remaining steel framed windows are heavily corroded.



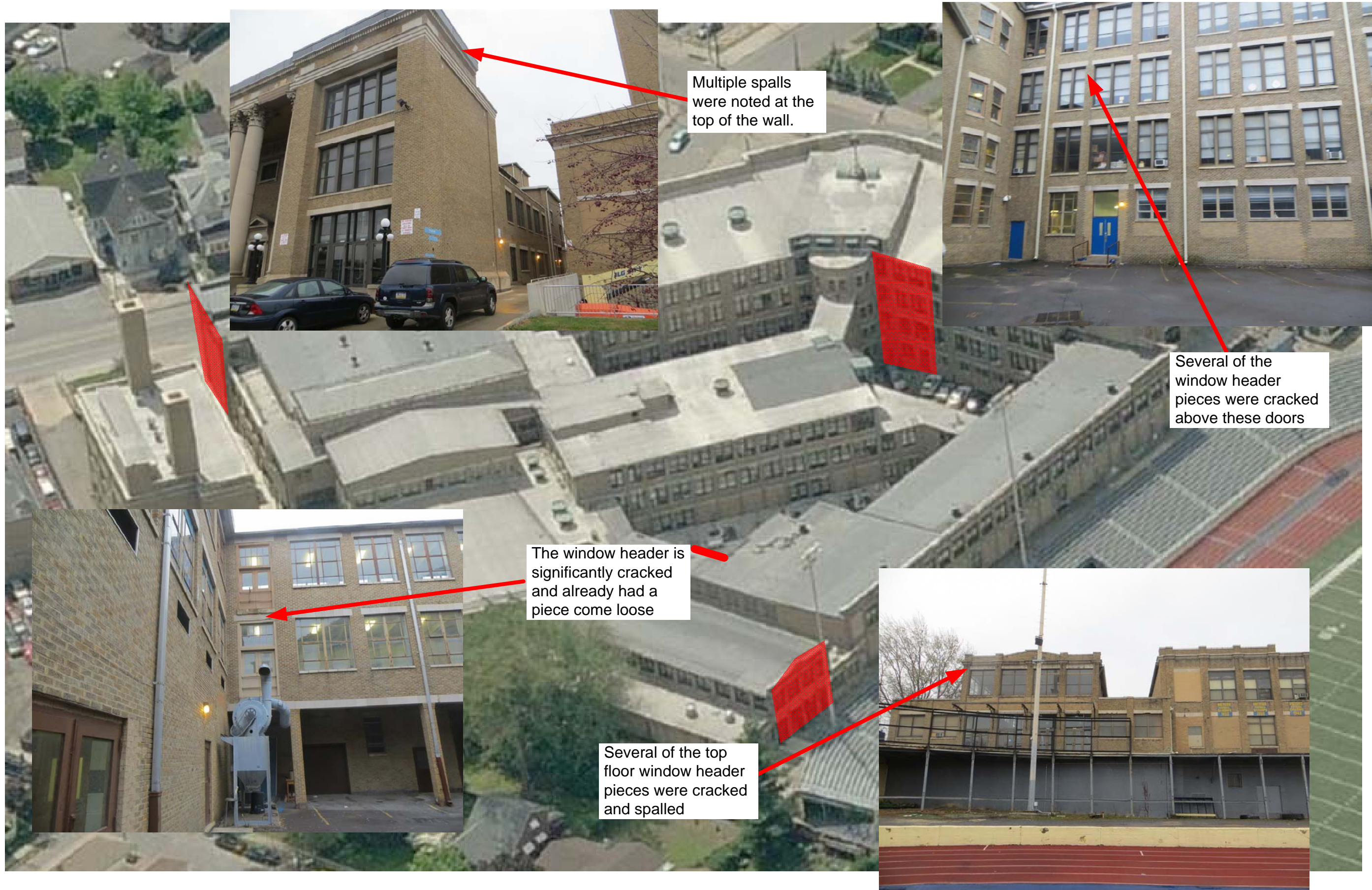
**Photograph 58:**  
The steel frames of the original windows are built into the surrounding masonry and may have been used to temporarily support the terra cotta above. Extensive masonry work will be required when the steel frames are removed.

# **APPENDIX B:**

## **Drawings**







Every elevation on the building has several pieces of terra cotta that are cracked or spalled, and a piece of terra cotta could come loose from anyone of these locations at any time and fall to the ground. The four locations highlighted above are areas with a higher potential for a piece to fall which are near pedestrian traffic.

# **APPENDIX C:**

## **MPS Architrave Evaluation**





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[masonrypreservation.com](http://masonrypreservation.com)

June 25, 2014

Leonard Engineering  
251 Mundy Street  
Suite C  
Wilkes-Barre, PA 18703

**ATTENTION:**        **Thomas Leonard**

**SUBJECT:**         **Masonry Evaluation**  
Elmer L. Meyers Junior Senior High School  
*Wilkes-Barre, PA*  
MPS Project 201423

Masonry Preservation Services, Inc. (MPS) was contracted to complete a masonry evaluation at the architraves and window soffits on the south elevation of the building. MPS utilized a 60 foot high reach to access the locations. The visual evaluation of the wall was completed on June 24, 2014.

### **GENERAL INTRODUCTION**

The Elmer L. Meyers Junior Senior High School is a mass masonry building located in Wilkes-Barre, PA. Built circa 1930, the building has a brick veneer with decorative terra cotta elements. In 2006, Michael J. Pasonik, Jr. and Associates was contracted to design a new roof for the school. During the design process, they raised concerns over the condition of the large masonry parapet and terra cotta water table.

A consulting engineer with experience evaluating masonry structures, Sutton Kennerly & Associates (SKA), was hired to review the condition of the parapet and water table. SKA provided a report dated January 22, 2007 that outlined the extensive deterioration the parapet and water table had undergone and recommended the full removal and rebuilding of the parapet and water table. Due to budgetary concerns, The Wilkes-Barre Area School District elected to permanently remove the water table and reduce the height of the parapet. See attached drawing SK-2 highlighting the areas of the parapet and cornice that were removed.



During the evaluation of the parapet and water table, SKA recommended additional evaluation of the terra cotta architraves over the main entrance and auditorium since it was apparent the architraves were being exposed to excessive moisture infiltration. SKA was contracted to evaluate the architraves and provided a report dated June 18, 2007. MPS provided SKA with craftsmen support during the evaluation. In the report SKA provided several recommendations to extend the service life of the architraves. To-date, none of the recommendations had been implemented. Included in the recommendations was reevaluation of the architraves every 3 to 5 years. MPS was requested to provide a proposal to evaluate the architraves and update the SKA report from 2007. The proposal was accepted, forming the basis of this evaluation.

### **OBSERVATIONS**

MPS completed this evaluation as a supplement to the SKA report. We have attached a copy of their report which can be found in *Appendix A: SKA Report*. The report contains additional information on the configuration of the masonry assemblies. We also included drawings to document our observations during our evaluation, which can be found in *Appendix B: Drawings*.

During their review, SKA noted that there were no issues that required immediate repair

### **New Parapet and Roof Flashing Termination**

The original parapet and cornice were allowing an excessive amount of water into the exterior wall. This was particularly troubling at the architraves and window soffits where steel is used to support the terra cotta. Corrosion of the steel decreases the strength of the steel while increasing the potential for cracking and spalling of the terra cotta. Corrosion threatens any ferrous metal (iron, steel, etc.) component, particularly where embedded and in direct contact with adjacent material (brick, mortar, etc.). In the presence of water, steel will corrode (rust), and expand with significant pressure created by the exfoliating rust (the corroding steel combined with exfoliating rust will occupy a larger volume of space than the original steel alone). The pressure generated produces stress in adjacent materials, oxide jacking, and in many building assemblies, can cause severe damage. SKA noted that reducing the amount of corrosion of the steel supports was vital to extending the service life of the terra cotta assemblies.

When the original parapet and cornice were removed, a much smaller parapet was constructed. The new parapet has a single-ply roof membrane that extends up the inboard side, over the top and down the outboard side. The roof membrane is terminated on a stainless steel drip edge with a termination bar. The use of a termination bar provided good securement of the roof membrane; however, the sealants installed at the termination bar were of low quality and were installed poorly, therefore we observed many locations of failed sealant. The roof membrane appears to be in good condition and has greatly reduced the amount of moisture the masonry below is exposed to. It was noted in 2007 that the masonry assemblies below the cornice and parapet were heavily saturated. During our review we noted that the walls had dried and were no longer saturated.



## **Brick**

Above the critical terra cotta elements of the architraves and the window soffits, there were 11 courses of brick. The brick was laid out to create a two course frame around a herringbone center panel. The majority of the brick appeared to be in good condition. We observed only minor cracking of the brick with little to no spalling. We did note that the mortar joints were in poor condition. All of the mortar was heavily weathered. There were numerous areas of missing, cracking and debonded mortar. There is an increased amount of water infiltration at this location due to the failed mortar. Due to the critical nature of the steel that supports the terra cotta below, it is imperative to minimize the moisture infiltration into the wall assembly.

There was a substantial amount of white staining on the exterior face of the brick. Masonry components with efflorescence are vulnerable to spalling caused by a phenomenon known as sub-efflorescence. This condition occurs when masonry wall assemblies retain liquid water in sufficient quantities and duration to allow the entrapped moisture to solubilize salts, which was occurring until the original parapet was removed. Eventually the liquid water evaporates, which is occurring at a high rate now that the new parapet and roof membrane have been installed. During the evaporation process the solubilized salts recrystallize at or near the masonry surface. This process causes an expansion of the salts that can cause wide spread deterioration of the mortar, and in many cases, of individual masonry units.

## **Terra Cotta**

The terra cotta all appeared to be in good condition. We sounded each piece of terra cotta at the four main assemblies and did not find any that were deteriorated enough to require repair. There were a few pieces with some minor cracks, and there was some spalling of the glazing, but nothing that would impact the stability of any of the pieces. On the west architrave, there were several pieces of terra cotta that were patched, which we believe was done at the exploratory openings that were made during the SKA evaluation. MPS made the openings but did not patch the terra cotta as we were instructed to leave the probes open, so the work must have been completed at a later date. The patches appear to be well bonded to the surrounding substrates.

The original exploratory openings were still present and open on the east architrave. We reviewed each of the openings and found that while the supporting steel has some significant surface corrosion, the steel is largely intact and has had minimal loss of cross sectional area. The installation of the new parapet appears to have reduced the rate of corrosion of the embedded steel supports.

The one area of concern that we noted was on the underside of the east architrave. Many of the mortar joints between the terra cotta pieces were missing or were loose. We removed several pieces of mortar by hand with little force required. The potential for additional pieces of mortar to come loose and fall to the ground is very high in this location.



## **RECOMMENDATIONS**

The conditions we found were very similar to what SKA found in their review. The only area of immediate concern that we observed was the loose mortar on the underside of the east architrave. We recommend the installation of containment netting at this location to prevent the mortar pieces from reaching the ground.

Per ASTM Standard E2270-13 - Standard Practice for the Periodic Inspection of Building Facades for Unsafe Conditions, Section 3.2.13.2, we would rate the remaining areas "Requires Repair / Stabilization". "Requires Repair / Stabilization" is a condition identified at the time of inspection that shall be repaired or stabilized in order to prevent progression into an "unsafe condition" prior to the next scheduled inspection. The repairs we recommend are 100% repointing of the brick and terra cotta along with the repairs recommended in the SKA report. Continued deterioration of the mortar will eventually cause brick to come loose from the exterior wall. Quality repointing should be completed in accordance with applicable portions of the Brick Industry Association guidelines and the Secretary of Interior Standards. The procedure includes deteriorated mortar removal to a depth of approximately  $\frac{3}{4}$  inch (or twice the joint width), new mortar installation applied in multiple (2-3) thin ( $\frac{1}{4}$  inch) layers as each previous layer becomes "thumbprint" hard, and tooling of the last layer to match the original mortar joint profile.

As per the SKA report, we agree that review of the architraves and window soffits should be completed at 3 to 5 year intervals. This level of review is warranted for assemblies like these that span over areas of high pedestrian traffic.

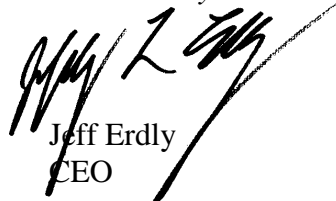
We appreciate the opportunity to provide our services for this project.

*Prepared By:*



Scott E. Siegfried  
Project Manager

*Reviewed By:*



Jeff Erdly  
CEO





# **APPENDIX A:**

## **SKA Report**

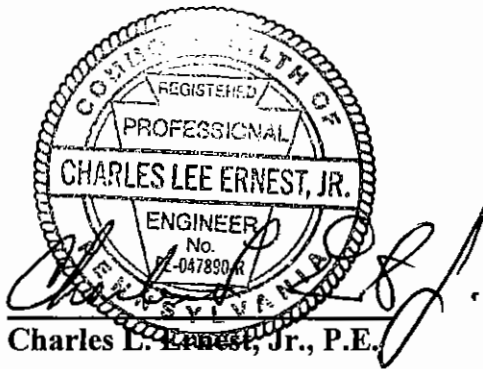


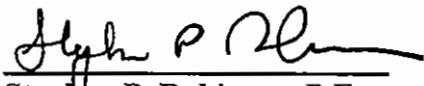
**SUPPLEMENTAL CONDITION SURVEY**

**Architraves at the Auditorium and Main Entry  
Elmer L. Meyers Junior Senior High School  
Wilkes-Barre, Pennsylvania**

**SKA Job No. 060515.0**

**June 18, 2007**



  
Stephen P. Robinson, P.E.

**SUTTON-KENNERLY & ASSOCIATES, INC.**  
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*www.suttonkennerly.com*

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Architraves at the Auditorium and Main Entry  
Elmer L. Meyers Junior Senior High School  
Wilkes-Barre, Pennsylvania**

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**I. Introduction, Objectives, and Scope of Survey**

Sutton-Kennerly & Associates, Inc. (SKA) performed a condition survey of the parapet walls, terra cotta and masonry facades, and chimneys of Elmer L. Meyers Junior Senior High School in November of 2006. SKA submitted a report of our findings on January 22, 2007. In the report, SKA recommended additional exploratory openings at the architraves in order to further assess the potential corrosion of the steel beams and to further evaluate the existing conditions.

As authorized by the Wilkes-Barre Area School District, Sutton-Kennerly & Associates, Inc. (SKA) has performed a supplemental condition survey of the terra cotta architraves at Elmer L. Meyers Junior Senior High School (Meyers HS). SKA conducted the condition survey in May of 2007 with the assistance of Masonry Preservation Services, Inc. The objectives of this condition survey were to:

- Identify and assess the structural components of the architraves located at the auditorium and the main entry.
- Develop recommendations for addressing any items of concern.
- Develop an estimate of probable construction costs for repair recommendations.

In order to achieve these objectives, SKA performed the following scope of service:

- SKA retained Masonry Preservation Services, Inc. (MPS) of Berwick, PA, to assist by providing and operating access equipment and performing excavations in the existing architraves at selected locations. Stephen P. Robinson, P.E. of SKA was on site May 15 -16 to observe the primary exploratory excavations.
- MPS excavated a deep exploratory opening in the brick masonry (just above the terra cotta soffit) of the architrave above the auditorium entry within bay number 2. This opening extended through the entire thickness of the brick masonry wall. The spans between the columns at the auditorium are referenced in this report as

bay numbers 1 through 3 with bay number 1 being the west bay, bay number 2 being the center bay, and bay number 3 being the east bay.

- MPS excavated a shallow exploratory opening in the brick masonry (just above the terra cotta soffit) to expose the structural steel beams within bay number 3 of the architrave above the auditorium entry.
- MPS removed portions of the soffit terra cotta at bay number 2 to reveal the extent of the concrete infill and allow observation of the steel components that support the terra cotta soffit.
- MPS excavated an exploratory opening in the brick masonry (just above the terra cotta soffit) at the main entry. MPS also removed portions of the soffit terra cotta at the main entry to reveal the extent of the concrete infill and allow observation of the steel components that support the terra cotta soffit.
- MPS loosened the bird netting, etc. at existing light wells in the plaster ceiling at the auditorium entry and the main entry. SKA used the openings to observe the roof framing and ceiling framing at these areas.
- SKA documented the existing conditions at the exploratory openings and prepared this supplemental report that includes the results of our condition survey (with photos), our conclusions, our recommendations, and a recommended budget for repairs.
- It is our understanding that no original drawings are available for review. Accordingly, no review of existing drawings was performed.
- This assessment was performed in accordance with SKA's Proposal for Engineering Services (SKA I&D Proposal No. 033-07) of March 29, 2007.

## **II. Executive Summary**

Sutton-Kennerly & Associates, Inc. (SKA) has performed a supplemental condition survey of the architraves at Elmer L. Meyers Junior Senior High School (Meyers HS). The objectives of the survey were to: identify and assess the structural components of the architraves located at the auditorium and the main entry, develop recommendations for addressing any items of concern, and develop an estimate of probable construction costs for repair recommendations.

The existing conditions at the architraves at the auditorium and the main entry were recorded during on site exploratory work. Sketches of a cross section of the auditorium architrave and a cross section of the main entry architrave have been included in the appendix of this report.

The condition survey revealed that an excessive amount of water is penetrating the terra cotta architrave at the auditorium. The excessive water penetration is causing the steel beams and hangers within the auditorium architrave to corrode at areas adjacent to a void that extends along the full length of the front of the architrave. The corrosion is localized to areas adjacent to the void; thus, the location of the corrosion is favorable for repairs that can be executed without removing large portions of the terra cotta and masonry.

The radial architrave at the main entry is filled with concrete and contains no longitudinal steel beams. The architrave requires no repairs other than the general repairs to the terra cotta as recommended in SKA's report of January 22, 2007.

In order to obtain an additional 25 or more years of service from this facility (as a school), SKA recommends the following general repair procedures at the architraves:

- Remove a horizontal strip of the existing brick masonry along the top of the terra cotta soffit at the auditorium and remove the corrosion on the



steel beams and hangers. Fill the void (behind the masonry) with concrete as the brick masonry is reinstalled.

- Localized defects (loose pieces, etc.) should be repaired as described in SKA's report of January 22 at the architraves at the main entry and at the auditorium.
- It may be advantageous to treat the mortar joints of the terra cotta that contain hangers and dowels (within the auditorium and main entry architraves) with a migrating corrosion inhibitor to reduce the rate of corrosion on the hangers and dowels. This option should be researched further before design documents are prepared.
- The architraves at the auditorium and the main entry should be inspected at 3 to 5 year intervals to insure that any deterioration of the terra cotta or brick masonry is corrected before becoming a problem. This recommendation is a precautionary measure that SKA deems essential to the long term safety of the facility when one considers the age and location of the terra cotta cladding at the architraves.
- The repairs to the architraves should have the same priority as the parapet walls over the auditorium and should not be delayed.

The estimated probable construction costs for these repairs (excluding migrating corrosion inhibitors and future inspections) is approximately \$33,680.

### III. Results of Condition Survey

During the field investigation performed on May 15-16 of 2007, SKA noted the following:

#### **Auditorium Architrave**

- 1) Exploratory openings were made in the auditorium architrave as described previously in this report. The existing configuration of the steel framing, terra cotta, and brick masonry within the architrave was documented and sketched. A sketch of the cross section of the auditorium architrave is included in the appendix of this report.
- 2) Two steel beams were observed that run east/west from column to column and support the architrave. The beams are 20" deep and are spaced approximately 16" on center. The beams support the architrave and all masonry above the beams that includes the parapet walls.
- 3) Both beams are encased in concrete (up to the top flange of each beam) except for a void located at the upper exterior portion of the exterior beam. The void measures approximately 8.5" tall by 4.5" deep and runs continuously along the exterior beam. The areas above the steel beams are completely filled with solid brick masonry.
- 4) Corrosion was observed on the web of the exterior steel beam (at the open cavity) within approximately 2" to 4" of the top of the concrete fill. The corrosion appears to have caused a minimal loss of steel cross section at the beam web.

- 5) The terra cotta soffit is filled with concrete and supported by vertical steel hanger rods at each mortar joint. Corrosion was observed on the ½" diameter vertical hanger rods that run through the open cavity. The corrosion appears to have caused a moderate loss of steel on the rods.
- 6) When the top flanges of the beams were exposed (during the exploratory excavations), only minimal corrosion was observed on the flanges. The original (orange) paint was intact at a large portion of the steel flange. Some of the paint may have been compromised during the process of excavating the exploratory opening.
- 7) MPS reported that free water was present on the exterior steel beam (at the void) at the time of the excavations. The outer wythes of masonry were damp within the exploratory openings above the steel beams.
- 8) The terra cotta was excavated at the mortar joints to expose the dowels between the terra cotta units. Corrosion was observed on the dowels and corrosion appears to have caused a minor loss of steel on the dowels.
- 9) A ¾" diameter dowel with an 11/16" x 2.5" x 12.25" plate was observed in the exploratory openings in the brick masonry. The purpose of this assembly was unclear. This assembly aligns with the joints in the dental mold (the terra cotta directly below the water table) and does not align with the joints in the terra cotta soffit.
- 10) The structural steel framing for the roof and the plaster ceiling were visible during observations into the attic over the auditorium portico. The roof is supported by 8" deep steel channels spaced at 32" on center. The roof deck is composed of precast concrete decking. Deck panels are approximately 24-25" wide and at least



2" thick. The roof channels run north/south such that the channels bear on the masonry supported by the steel beams in the architrave.

- 11) The distance from the plaster portico ceiling to the bottom of the roof deck varies because the roof of the auditorium is a gabled roof. The maximum distance at the peak could not be measured, but is estimated to be approximately 86" (based on measurements at the area of the attic that was accessed). The roof slope appears to be 1.5:12 or 1:8 (based on the bearing heights of the steel channels).
- 12) The plaster ceiling is supported by a grid of cold rolled channels and metal lath. The rolled channels are supported by wires connected to 8" deep steel beams (with 5.5" flanges) spaced at approximately 6' on center. The ceiling beams run north/south such that the beams bear on the masonry supported by the steel beams in the architrave. The distance (from the top of the plaster ceiling) to the bottom of the ceiling beams is approximately 8"-9".
- 13) The ceiling and roof beams span approximately 12'-4" across the auditorium portico.

#### **Main Entry Architrave (Radial)**

- 14) Exploratory openings were made in the main entry architrave as described previously in this report. The existing configuration of the steel framing, terra cotta, and brick masonry within the architrave was documented and sketched. A sketch of the cross section of the main entry architrave is included in the appendix of this report.
- 15) Unlike the auditorium architrave, there are no steel beams within the radial portion of the architrave. Excavations and observations revealed that the radial

portion of the architrave is filled with concrete (reinforced) and supported from cantilevered steel beams that extend across the main entry portico from the main building.

- 16) The terra cotta was excavated at the mortar joints in the terra cotta soffit to expose the dowels and hangers between the terra cotta units. The terra cotta soffit is filled with concrete and supported by a system of steel wires and dowels at each mortar joint. The dowels are 1/2" in diameter and the wires are 1/4" in diameter. Light corrosion was observed on the dowels and wires; the corrosion has caused little or no loss of steel.
- 17) Two smooth 1" diameter bars (running along the length of the radial architrave) were observed within the terra cotta soffit. It is unclear if the bars are continuous; however, it seems very unlikely that 1" diameter bars would be used as dowels to support terra cotta units. It seems likely that the 1" bars are continuous reinforcement for the concrete fill and that the hangers from the cantilevered beams likely connect to the 1" diameter bars. Corrosion was observed on the bars and appears to have caused little or no loss of steel.
- 18) The structural steel framing for the roof, architrave, and the plaster ceiling were visible during observations into the attic over the main entry portico. The roof is supported by 4" deep steel channels spaced at 32" on center. The roof channels bear on the main building wall and the masonry supported by the architrave.
- 19) The roof deck is the same as the roof deck at the auditorium portico: precast concrete deck panels measuring approximately 24-25" wide and at least 2" thick.

- 20) The architrave and the masonry above the architrave (including the parapet wall) are supported by 15" deep cantilevered steel beams spaced at 36" on center (within the main radial span). The cantilevered steel beams bear on the main building wall and cantilever up to 100" past the main building wall. Hanger rods are visible at the (exterior) ends of the 15" beams. The end of one of the cantilevered beams was visible after one wythe of masonry was removed from the exterior face of the masonry.
  
- 21) The plaster ceiling is supported by a grid of cold rolled channels and metal lath. The rolled channels are supported by wires connected to 8" deep steel beams (with 5.5" flanges). The ceiling beams bear on the main building wall and the masonry supported by the architrave.
  
- 22) The distance from the upper surface of the plaster portico ceiling to the bottom of the roof deck is approximately 74". The distances (from the top of the plaster ceiling) to the bottom of the cantilevered steel architrave beams and to the bottom of the ceiling beams are approximately 26" and 18-20" (respectively).



#### **IV. Conclusions and Discussion**

Sutton-Kennerly & Associates, Inc. (SKA) has considered all information collected throughout this condition survey and developed the following conclusions:

##### **Auditorium Architrave**

The primary concern at the auditorium architrave is the corrosion on the outer steel beam and the steel hanger rods within the void that runs the length of the architrave. At this time, the corrosion has not damaged the steel components enough to create an imminent threat to the stability of the architrave; however, if this corrosion is allowed to continue, the integrity of the outer steel beam (and thus the architrave) will be compromised.

Corrosion requires the presence of water and oxygen in order to occur. A significant quantity of water is intruding into the void (along the length of the architrave) from above, and the corrosion is concentrated within the void. The water intrusion will be greatly reduced if the recommendations from our January 22, 2007, report are implemented.

Generally, the steel beams (other than a portion of the outer steel beam adjacent to the void) were observed to have little or no corrosion. SKA attributes this fact to the presence of the concrete that encases the uncorroded components. Steel components embedded in concrete are protected from corrosion because the concrete causes a passivating film to form on the surface of the embedded steel.

Corrosion on steel members is typically addressed by abrasively removing the corrosion and coating the steel surfaces to minimize future corrosion. In this case, the corroded members can be accessed by removing a strip of brick masonry just above the terra cotta soffit (without removing any of the terra cotta cladding). The void (along the length of

the architrave) could be filled with concrete to prevent water from accumulating in the void and to provide supplemental protection to the steel members.

A secondary concern is corrosion on the steel hangers and dowels (outside of the void along the length of the architrave). The corrosion observed on the hangers and dowels (outside of the void along the length of the architrave) was generally surface corrosion with little or no loss of steel area. Continued corrosion of these steel hangers and rods could cause cracking and possible displacement of small fragments of the terra cotta, but does not pose a threat to the overall integrity of the architrave.

It may be feasible to address this concern by applying a migrating corrosion inhibitor to the mortar joints in the terra cotta. Migrating corrosion inhibitors are designed to migrate through cementitious materials (such as concrete and mortar) and form a protective film on the steel. Migrating corrosion inhibitors reduce the rate of corrosion dramatically if used properly. This option should be researched further before inclusion into the design documents for these repairs.

A prudent precautionary measure would be to have the architrave inspected by a qualified Professional Engineer at 3 to 5 year intervals to identify and promptly address any localized problems that may arise within the terra cotta in the architraves. This precaution is essential to ensure the long term safety of any facility that contains a significant quantity of aged terra cotta over areas of high traffic.

**Main Entry Architrave (Radial)**

The main entry architrave is supported by cantilevered steel beams that extend outward from the main building. No internal steel beams were observed within the terra cotta. The architrave appears to be completely filled with concrete that is reinforced with two smooth bars (1" diameter). As described previously, the concrete protects the embedded steel from corrosion and minimizes the impact of intruding water.

The only significant threat to the integrity of the main entry architrave is the cracking and distress that might occur if the hangers and dowels continue to corrode. As described above, this concern might be addressed by applying a migrating corrosion inhibitor to the mortar joints in the terra cotta (pending further research of this idea prior to preparation of the design documents). As was the case at the auditorium, it is prudent to have this area inspected at 3 to 5 year intervals to identify and promptly address any localized problems within the terra cotta in the architraves.

Other than the potential corrosion of the dowels and hangers, the only concerns at the main entry architrave are the general terra cotta problems (such as a few cracked units, etc.) as identified in SKA's report of January 22.



**V. Recommendations**

Sutton-Kennerly & Associates, Inc. (SKA) has developed the following repair recommendations based on the premise that the Wilkes-Barre Area School District intends to continue using the Meyers HS campus for the next 25 or more years. The repairs to the architraves should have the same priority as the parapet walls over the auditorium and should not be delayed.

**Auditorium Architrave**

- 1) Remove brick masonry (along the top of the terra cotta soffit) to access the web of the exterior steel beam and the steel hanger rods along the exterior face of the architrave. Temporary supports for the brick masonry to remain will be required.
- 2) Clean and coat the exposed portions of the exterior steel beam and the vertical steel hanger rods.
- 3) Repair all steel hanger rods that are severely damaged by corrosion. Supplementary anchors may be required at some locations.
- 4) Fill the cavity with concrete and reinstall the brick masonry.
- 5) Repair the exploratory openings in the masonry and terra cotta. Perform general terra cotta repairs as described in SKA's report of January 22.
- 6) Have a qualified professional engineer perform an inspection of the terra cotta and brick masonry within the architraves at intervals not to exceed 5 years. The inspections should be performed in accordance with ASTM E2270-05, "Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions".

**Main Entry Architrave (Radial)**

- 7) Repair the exploratory openings in the masonry and terra cotta.
- 8) Perform general terra cotta repairs as described in SKA's report of January 22.
- 9) Have a qualified professional engineer perform an inspection of the terra cotta and brick masonry within the architraves at intervals not to exceed 5 years. The inspections should be performed in accordance with ASTM E2270-05, "Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions".

**Recommendations Regarding the Removal of the Parapet Walls**

SKA's report of January 22, 2007, recommended the removal and replacement of the masonry parapet walls at the east and south elevations of the auditorium, the south main elevation, the radial main entry, the southwest elevation, and the northwest elevation. Caution must be exercised during the removal of the parapet walls at the auditorium and the main entry. In no case can masonry be removed at an elevation below the roof deck because the parapet wall is a load bearing wall for the roof systems at the auditorium and the main entry.

**VI. Estimate of Probable Construction Costs:**

Sutton-Kennerly & Associates, Inc. (SKA) has estimated the cost for the repairs recommended in this report based on our experience with similar projects with similar repairs. The estimates (below) are based on the premise that the work recommended in this report will be performed in conjunction with the work recommended in our report of January 22, 2007. If the work recommended in this report is performed as an independent project, costs for general conditions, mobilization, etc., must be added to these estimates.

The estimates below do not include costs for the general terra cotta repairs recommended in SKA's report of January 22 or future inspections of the terra cotta.

We emphasize that the Estimate of Probable Construction Costs is a budget estimate based upon our limited knowledge concerning the proposed project at this time. The supplemental information gathered during the development of the contract documents during the design phase of this project may alter the scope of work such that the actual cost of the work increases or decreases.

**Estimate of Probable Construction Costs**

<b>Auditorium Architrave</b>		
1. Remove brick masonry, remove corrosion, fill void with concrete, and reinstall brick masonry	\$21,780	
2. Repair exploratory openings	\$ 6,000	
<b>Subtotal</b>		<b>\$27,680</b>
<b>Main Entry Architrave</b>		
Repair exploratory openings	\$ 6,000	
<b>Subtotal</b>		<b>\$ 6,000</b>
<b>Total for Architraves</b>		<b>\$33,680</b>



**Summary of Probable Total Project Costs**  
**(includes recommendations from the January 22, 2007 report)**

<b>Demolition &amp; Roof Edge Termination</b>		<b>\$ 674,904</b>
<b>Water Table Reconstruction</b>		<b>\$ 871,242</b>
<b>Parapet Reconstruction</b>		<b>\$1,798,871</b>
<b>Estimate of Engineering Design and Construction Observation Services (3% to 6%)</b>		<b>\$ 200,000</b>
<b>Total for Architraves (from previous page)</b>		<b>\$ 33,680</b>
<b>Project Total</b>		<b>\$3,578,697</b>

**VII. Limitations of Liability**

Sutton-Kennerly & Associates, Inc. (SKA) has performed a supplemental condition survey of the architraves at the auditorium and the main entry. Other portions and components of the main building and boiler building have not been evaluated by SKA during this supplemental condition survey. SKA specifically disclaims any responsibility for the future performance of any portion of either building not specifically addressed in this report (or SKA's report of January 22, 2007).

It is important to note that conditions that were not visible or were not obvious during the performance of this survey could exist that would alter SKA's opinions and recommendations. SKA specifically disclaims any responsibility for conditions that were not visible or obvious during this condition survey.

It is also important to note that deterioration of building components is an on-going process. SKA specifically disclaims all responsibility for damage, loss, injury or loss of life that occurs due to failure to implement the recommendations of this report.

The Estimate of Probable Construction Costs is not a quote or established price for the work. The estimates were developed with the assistance of qualified professionals based on a general scope of work to be conducted under typical construction conditions. The discovery of additional information that leads to a change in scope, job site restrictions, time limitations, and other factors could cause actual bids to be greater or less than the values contained in this report.

This report is not intended to be a contract document for construction activity at this facility. The scopes of work, schematic details, and other information in this report were not developed for this purpose. SKA specifically disclaims all responsibility for losses incurred if this report is used as a contract document for construction at this facility.

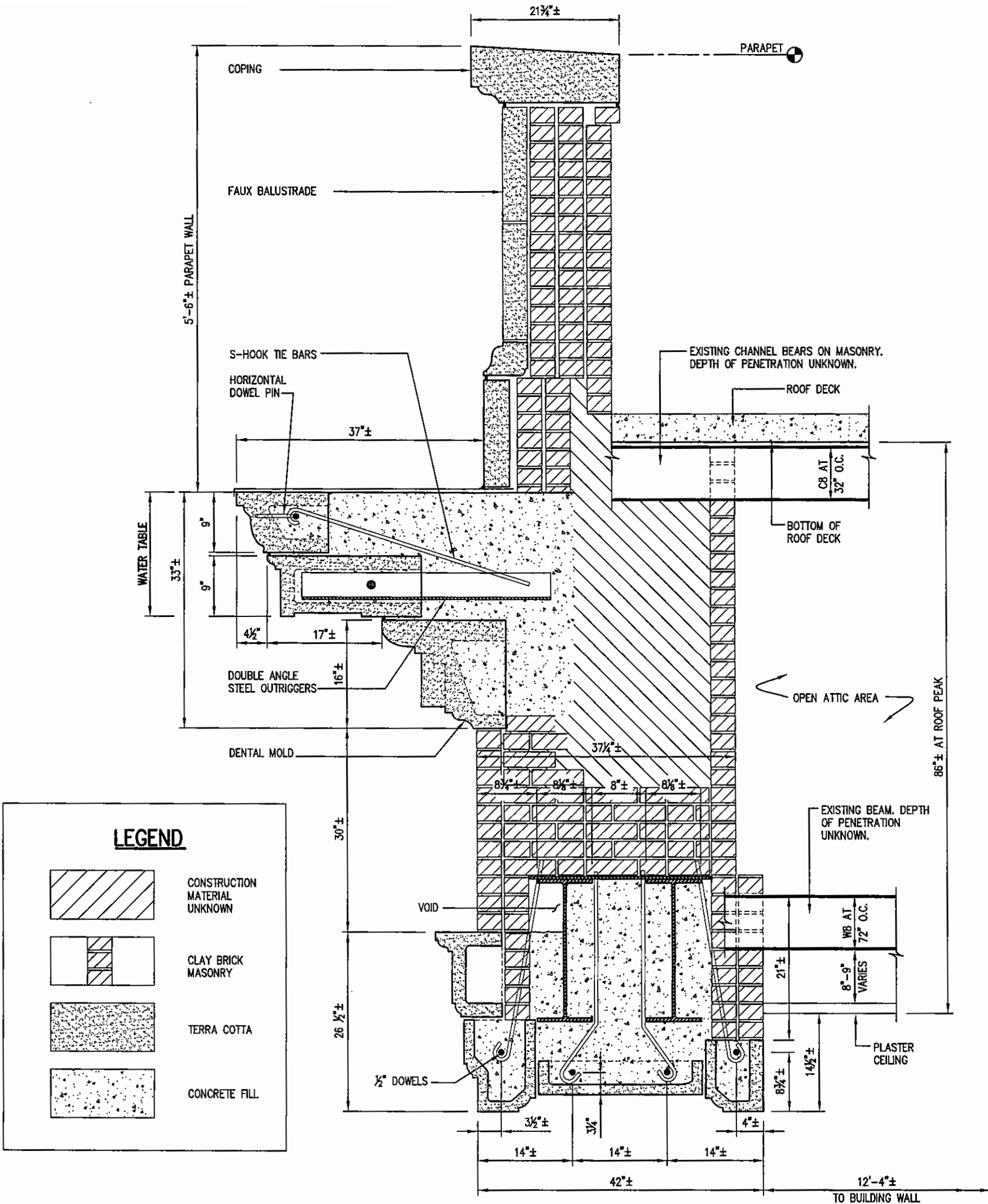
**APPENDICES:**

Sections: Existing Conditions

Photographic Log

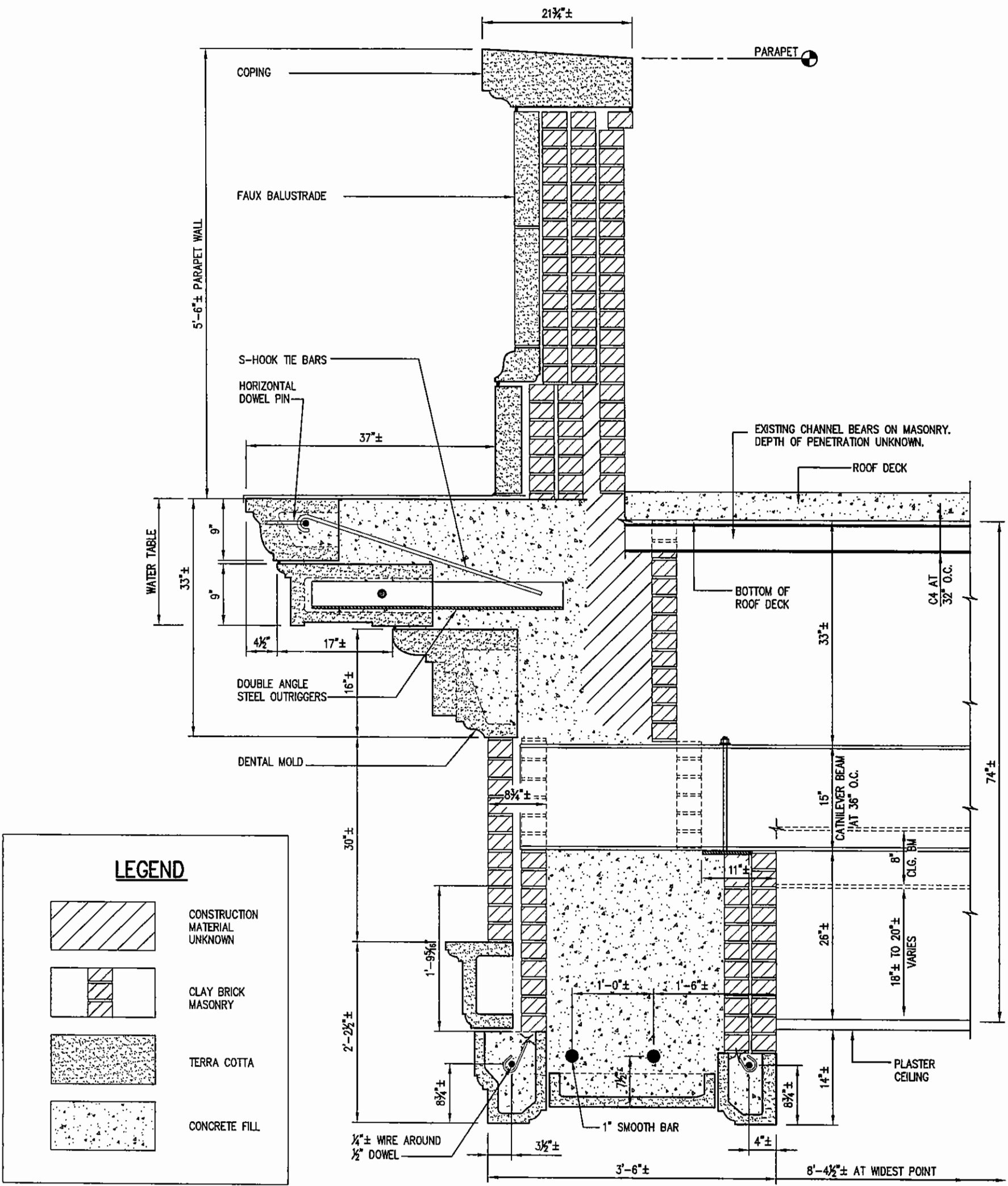


**Sections: Existing Conditions**



**AUDITORIUM  
SECTION**

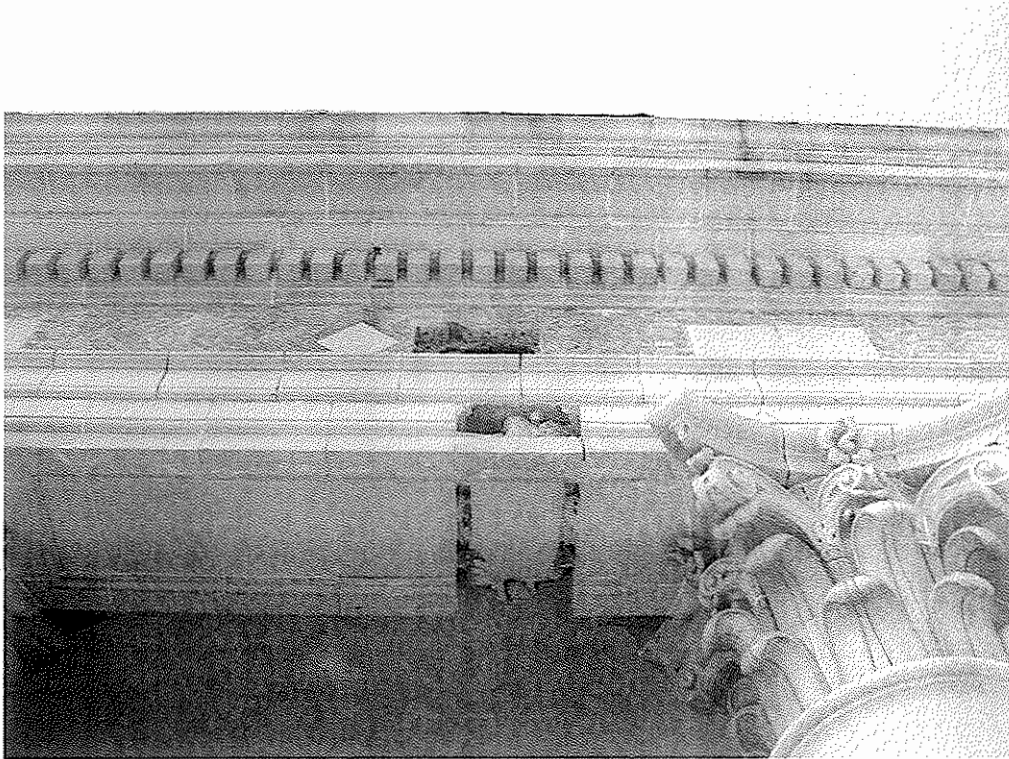
N.T.S.  
Dwg. # 060515-S5600.DWG



**MAIN ENTRY  
SECTION**  
N.T.S.  
Dwg. # 060515-S6601.DWG



**Photographic Log**



Auditorium Architrave



Auditorium - Upper Test Cut at Bay #2

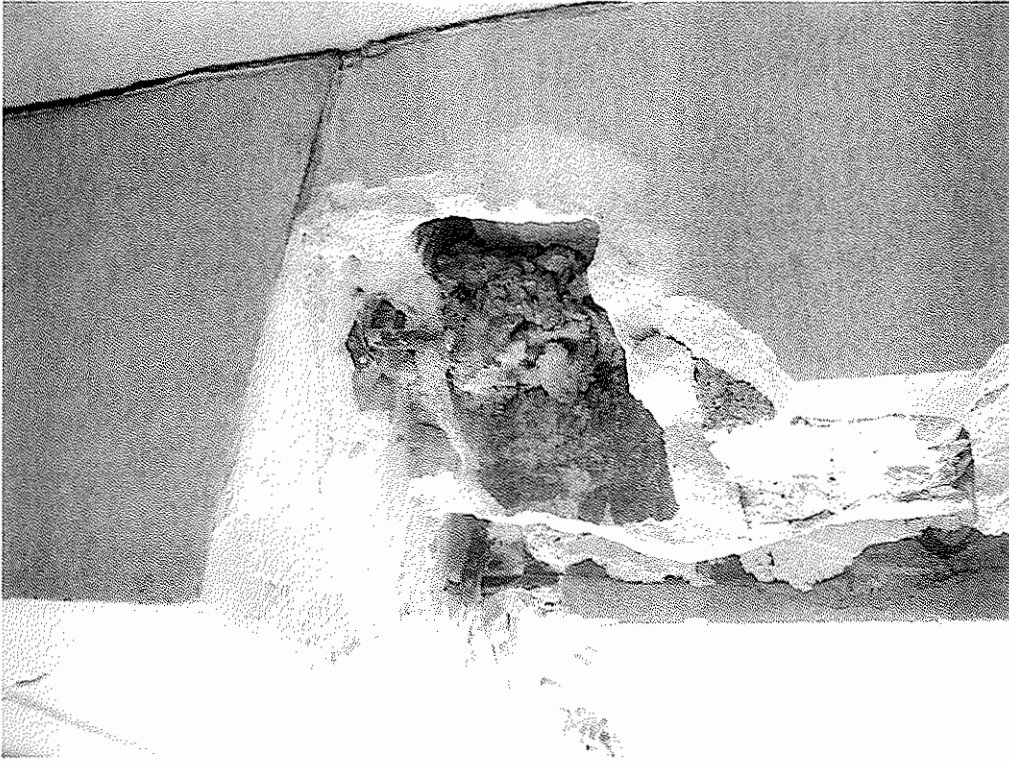


Auditorium - Upper Test Cut at Bay #2

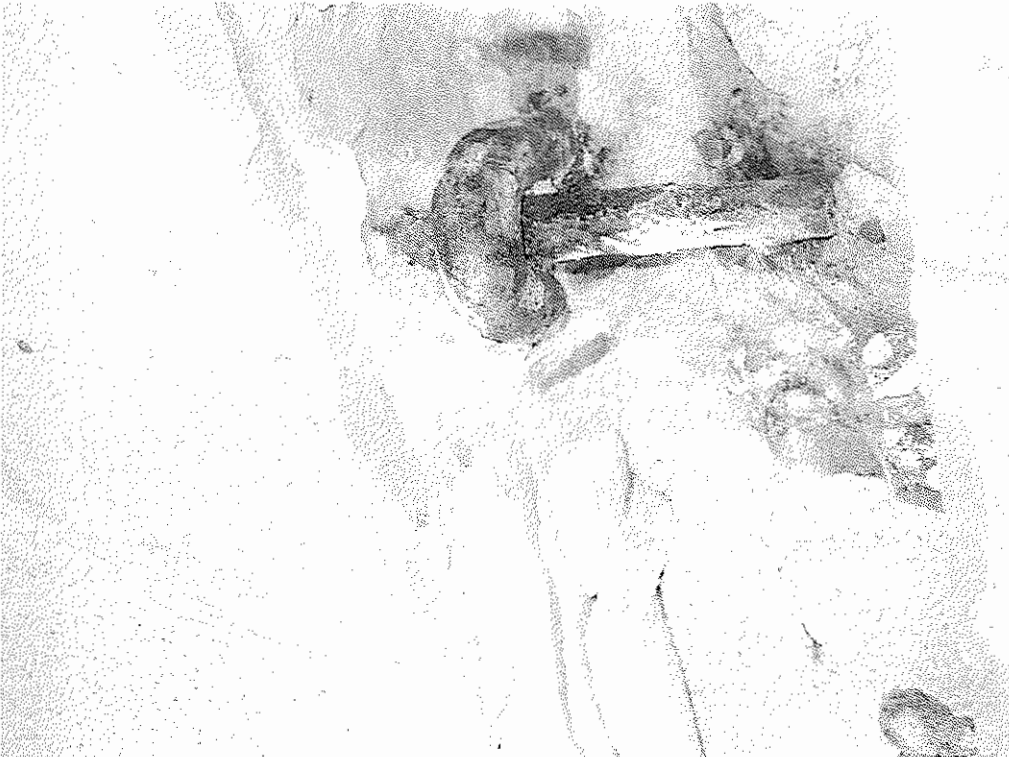


Auditorium - Lower Test Cut at Bay #2





Auditorium - Lower Test Cut at Bay #2



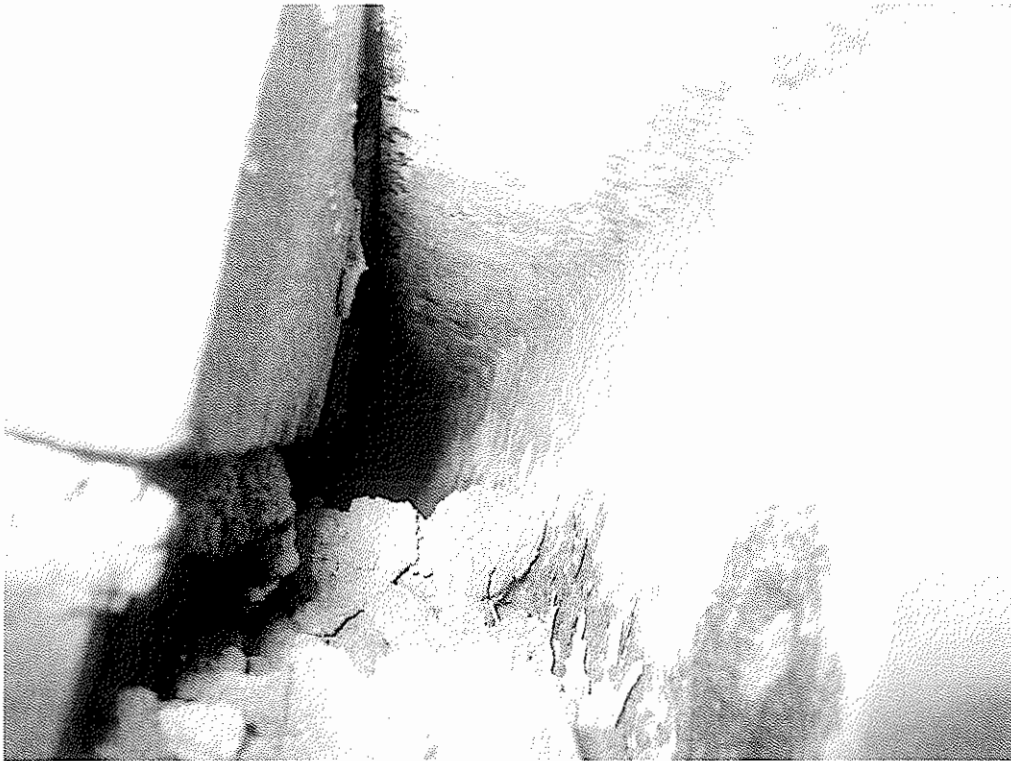
Auditorium - Lower Test Cut at Bay #2



Auditorium - Test Cut at Bay #3



Auditorium - Test Cut at Bay #3



Auditorium - Test Cut at Bay #3 - Void along Beam

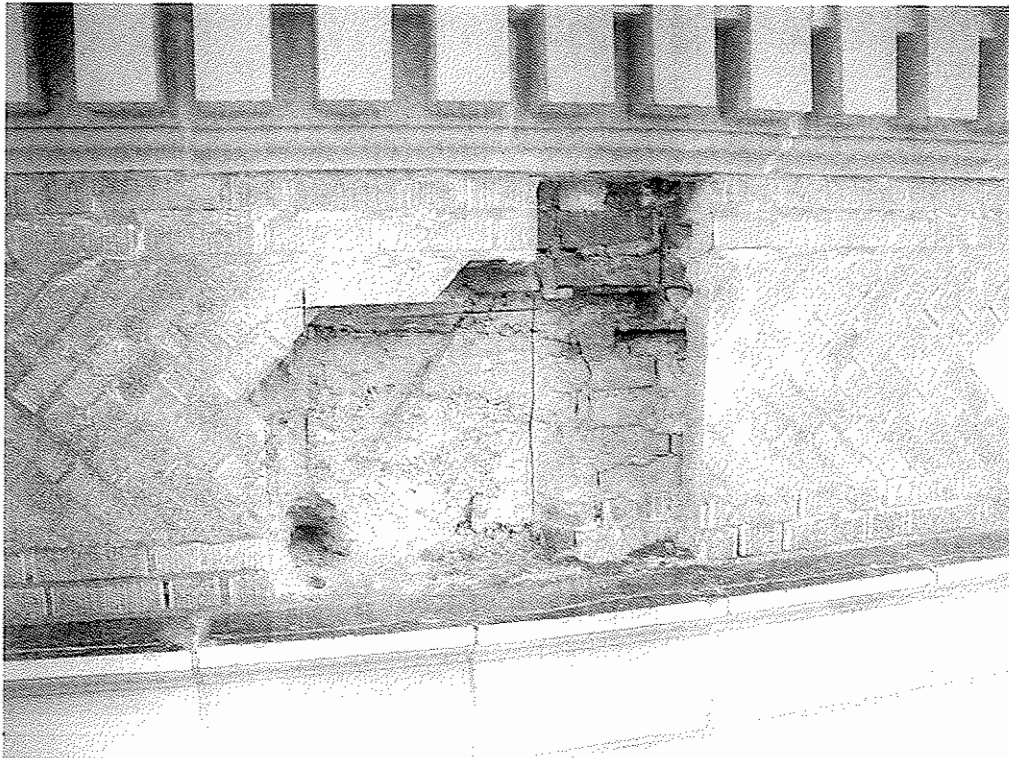


Auditorium - Roof and Ceiling Framing at Portico





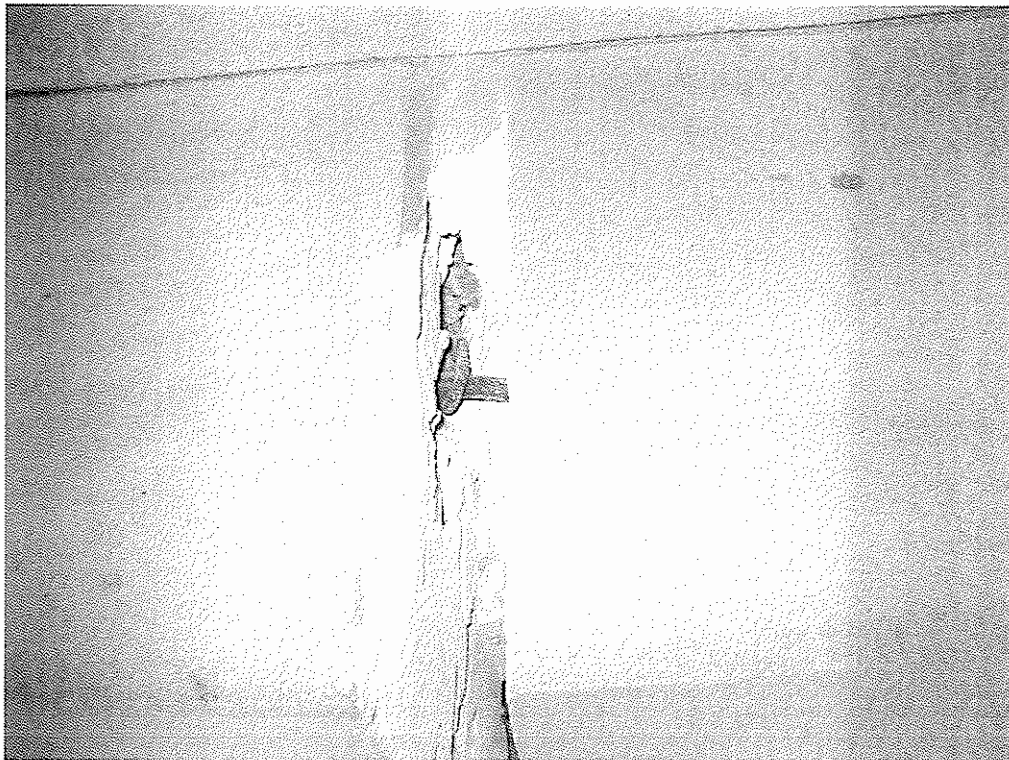
Main Entry Architrave (Radial)



Main Entry - Upper Test Cut



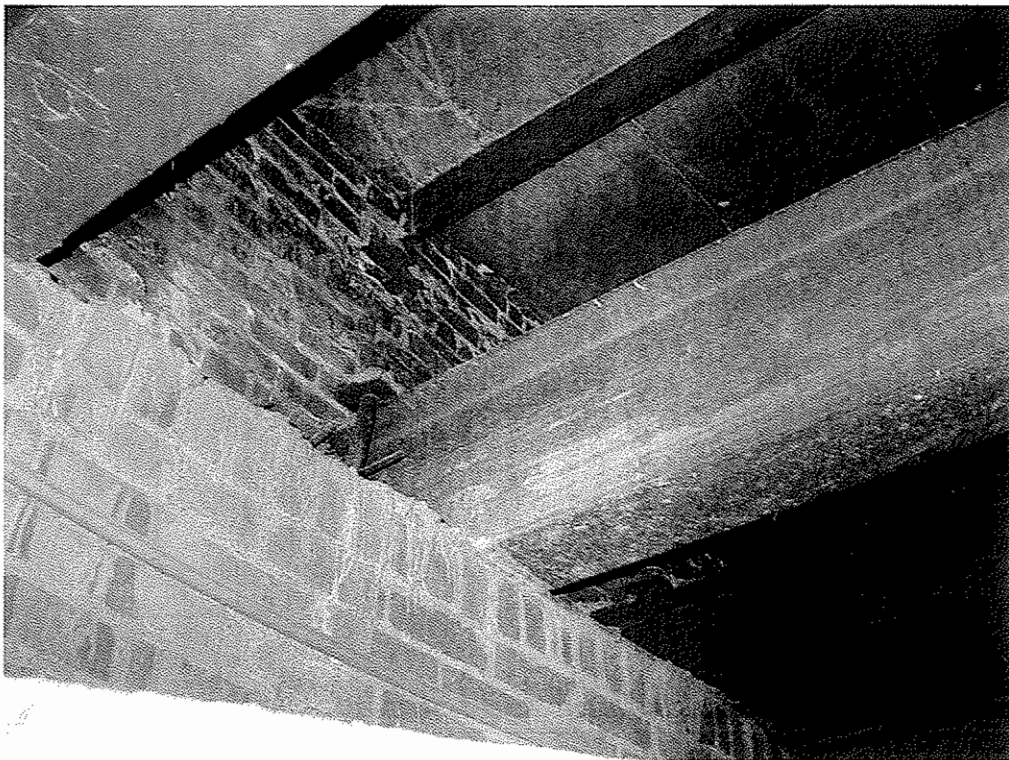
Main Entry - Lower Test Cut



Main Entry - Lower Test Cut



Main Entry - Lower Test Cut



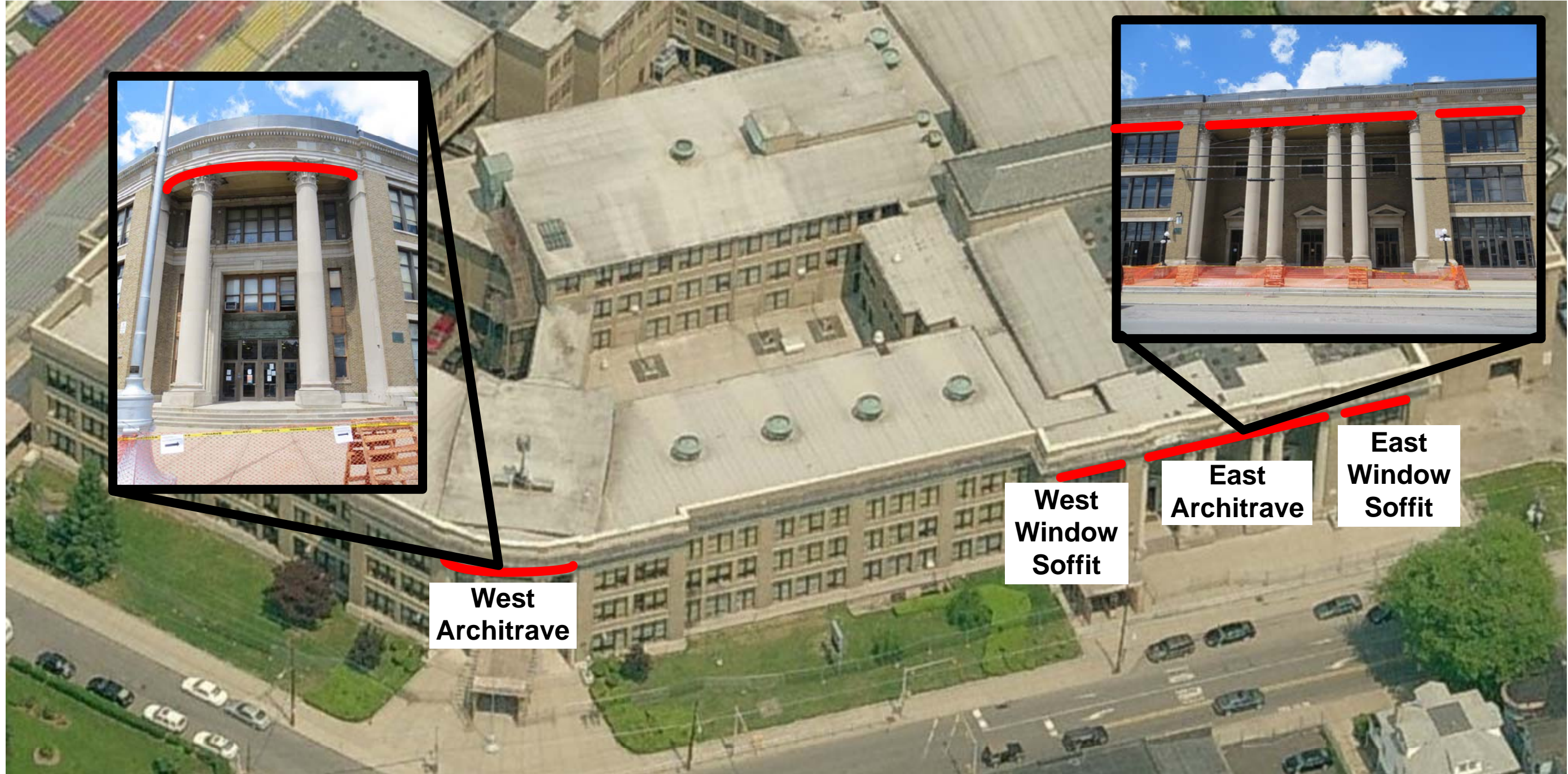
Main Entry - Roof and Architrave Framing



# **APPENDIX B:**

## **Drawings**





Locations of architectural details evaluated

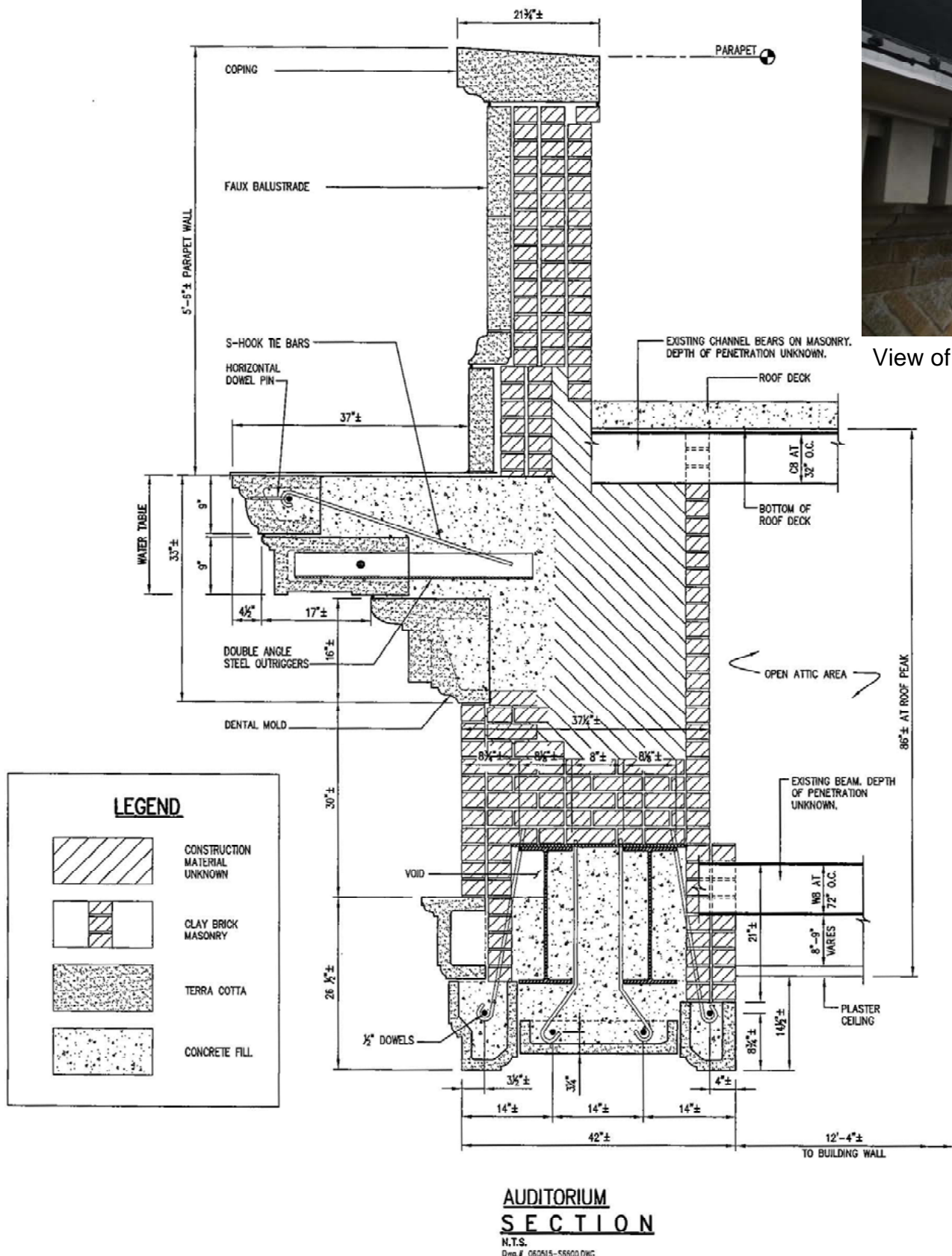


Areas Evaluated

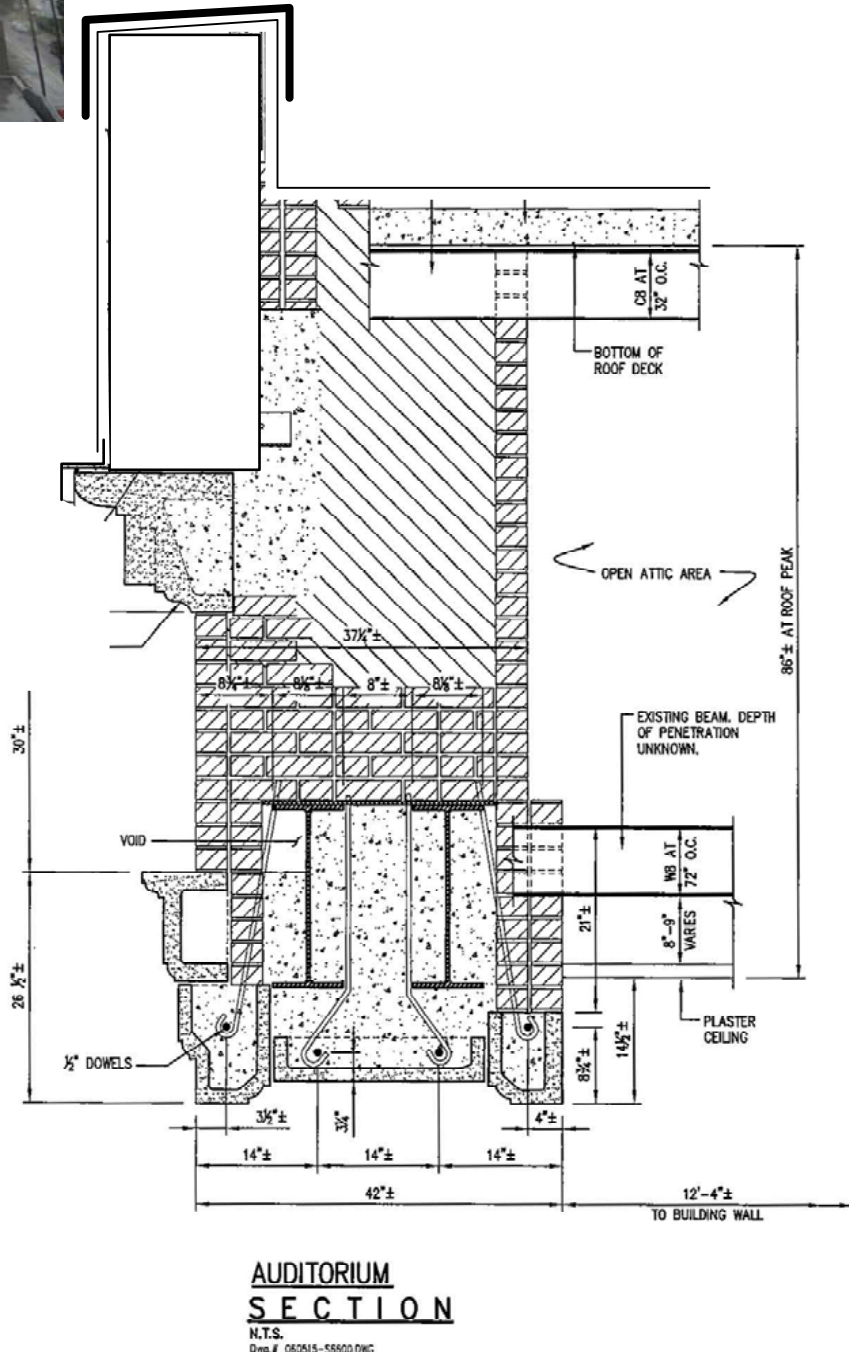
Elmer L. Meyers Junior Senior  
High School  
Wilkes-Barre, PA  
Project No.: 201423

DRAWING NO.:  
SK - 1





View of the new parapet



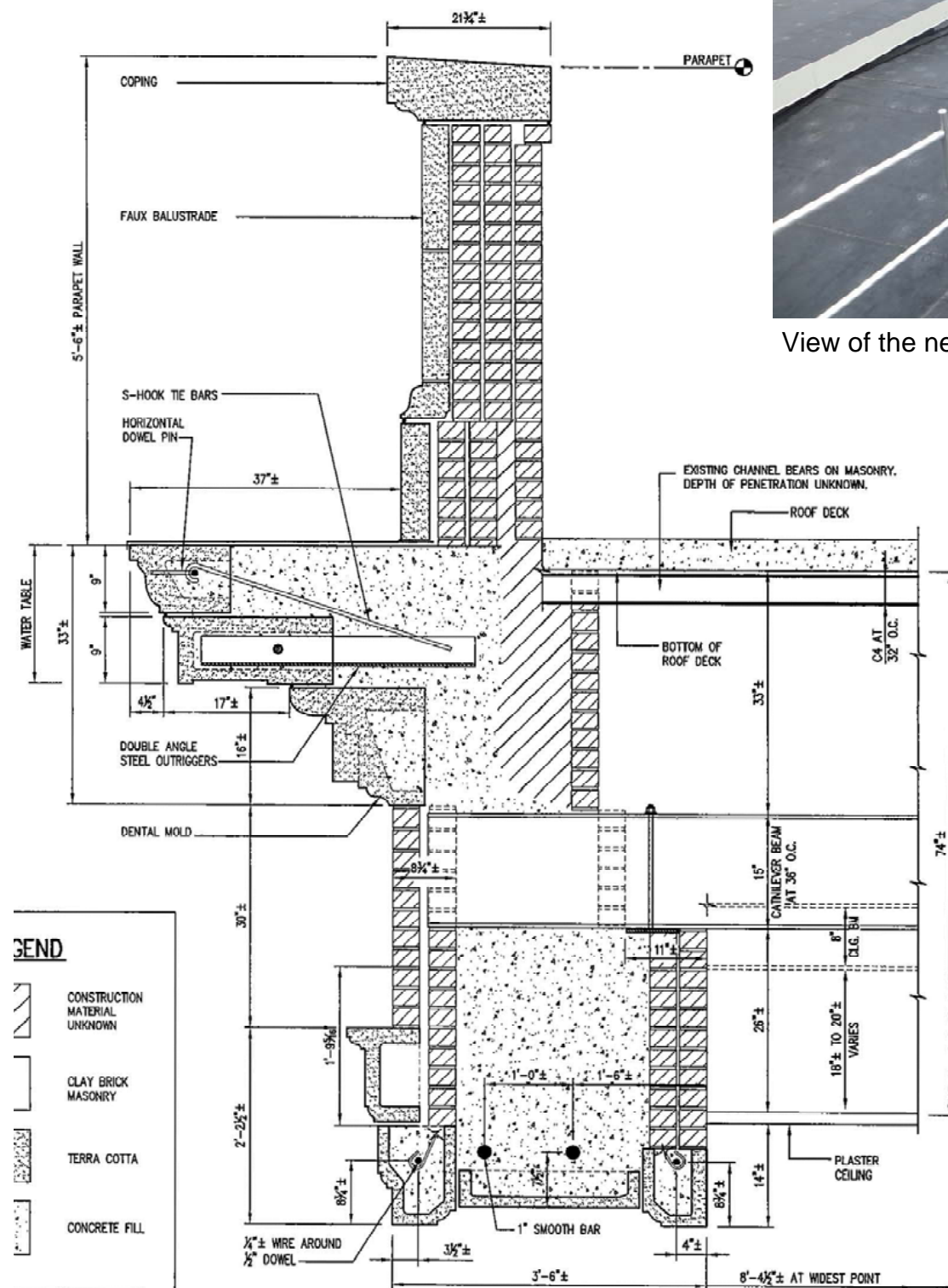
Original Detail

New Parapet

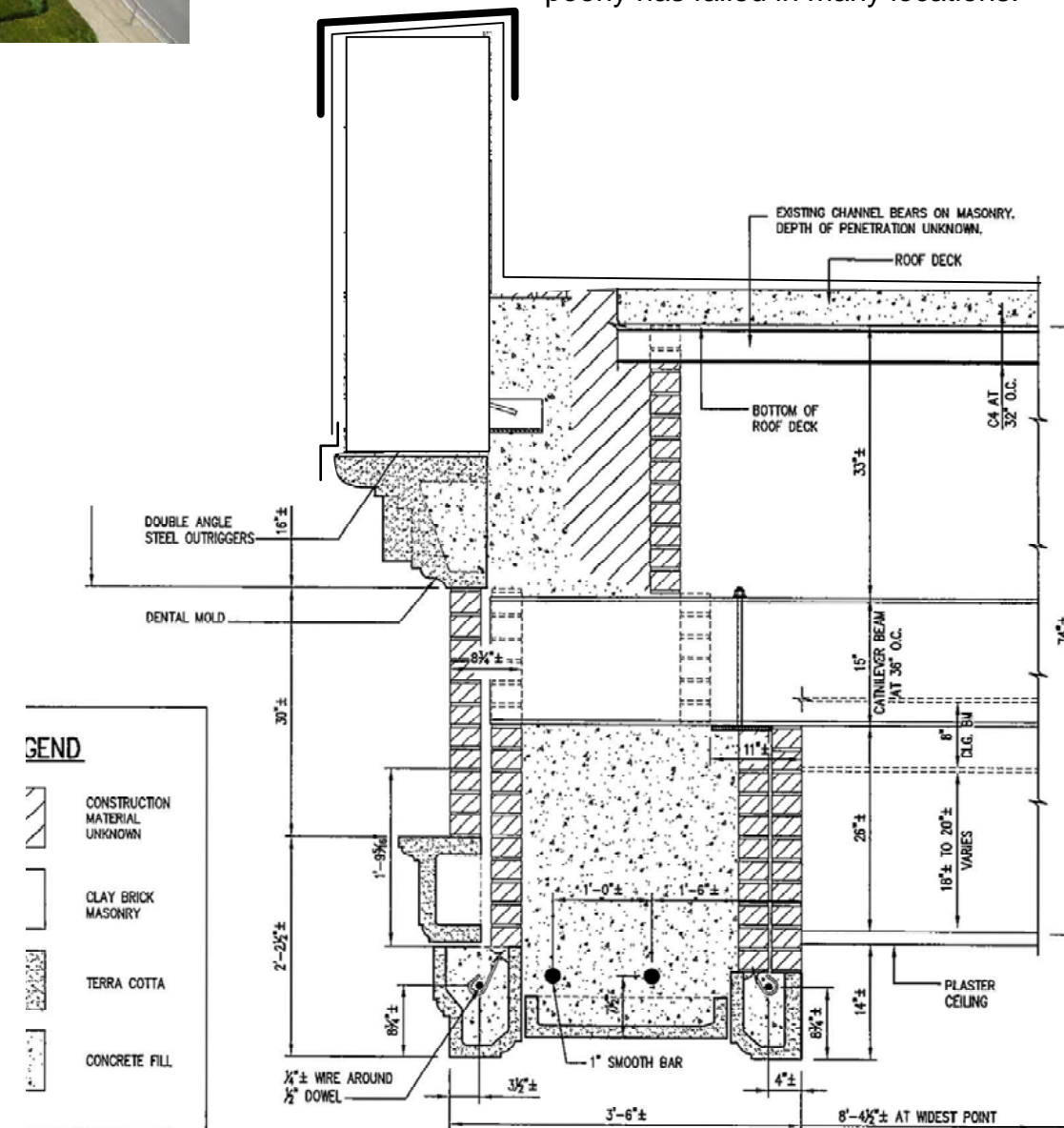
Changes to Original Detail  
 Architrave East

Elmer L. Meyers Junior Senior  
 High School  
 Wilkes-Barre, PA  
 Project No.: 201423





### Original Detail



## New Parapet





Continued failure of the mortar will eventually compromise the stability of the brick.



All of the terra cotta pieces were sounded. We did not find any pieces that were unstable or significantly displaced



A minor crack in the face of the terra cotta panel.



The mortar in the brick above the terra cotta was in poor condition. Previous attempts to repoint the mortar were unsuccessful and have already failed.



East Window Soffit



Sealant was installed over the majority of the terra cotta mortar joints. The sealant and mortar had failed at most locations.





Spalling of the glaze was present at many locations.



Some heavy surface corrosion was noted on the steel supports, but little loss of cross sectional area was observed.



Many of the brick mortar joints had failed, allowing excessive water infiltration.



Beam had some surface corrosion but was in good condition.



A piece of mortar that was removed by hand with little force used.



## East Architrave





Efflorescence (white staining) was present at several sections of brick, a sign that the masonry is drying out.



There were numerous failed brick mortar joints.



Typical detailing under the terra cotta soffit.



Spalling of the glazing was present at a few locations in this area.



West Window Soffit



Sealant was installed over the majority of the terra cotta mortar joints. The sealant and mortar had failed at most locations.





Terra cotta repair mortar appears to be well-bonded to the surrounding substrates.



Several attempts to repoint the mortar were improperly completed and have failed.



Failed mortar joint adjacent to patched terra cotta piece.



Failed mortar joints directly below the terra cotta dentil.



Some small spalls were noted on the dentil above the architrave.



West Architrave

**APPENDIX C:**  
**ASTM E 2270-13**







# Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions<sup>1</sup>

This standard is issued under the fixed designation E2270; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

## 1. Scope

1.1 This standard practice covers methods and procedures for inspection, evaluation, and reporting for periodic inspection of building facades for unsafe conditions. In the context of this practice, unsafe conditions are hazards caused by or resulting from loss of facade material.

1.2 This standard practice does not purport to address the nature of deterioration of various building facade materials nor the performance of their assemblies. It is the responsibility of the owner to retain a qualified professional who can demonstrate expertise in the evaluation of various facade materials and their assemblies.

1.3 Investigative techniques discussed may be intrusive, disruptive, or destructive. It is the responsibility of the qualified professional to anticipate, advise on the nature of procedures, and to plan for implementing repair as necessary.

1.4 It is the responsibility of the specifying authority to establish the usage of this standard practice and to supplement this practice with additional requirements suitable to its local jurisdiction. It is also the responsibility of the specifying authority to determine compliance with local licensing regulations and customary practices.

1.5 The values stated in inch-pound units are to be regarded as standard. The values given in parentheses are mathematical conversions to SI units that are provided for information only and are not considered standard.

1.6 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use. Awareness of safety and familiarity with safe procedures are particularly important for above-ground operations on facades*

*and destructive investigative procedures, which typically are associated with the work described.*

## 2. Referenced Documents

2.1 *ASTM Standards:*<sup>2</sup>

**E631 Terminology of Building Constructions**

## 3. Terminology

3.1 Refer to Terminology **E631**.

3.2 *Definitions:*

3.2.1 *categories of facade conditions:*

3.2.1.1 *ordinary maintenance*—a condition identified at the time of inspection that is not characterized as an “unsafe condition” or “requires repair/stabilization,” but requires maintenance.

3.2.1.2 *requires repair/stabilization*—a condition identified at the time of inspection that shall be repaired or stabilized in order to prevent progression into an “unsafe condition” prior to the next scheduled inspection.

NOTE 1—The immediacy of actions to address conditions requiring repair other than unsafe conditions is highly variable and should be determined by the owner. Such determination may require more detailed investigation than addressed herein to assess the urgency of such action.

3.2.1.3 *unsafe condition*—a condition identified at the time of inspection of a component or system that presents an imminent threat of harm, injury, damage, or loss to persons or property.

3.2.2 *facade*—all areas on the exterior of the building, except for horizontal roof areas. The facade includes all exterior walls, windows, balconies, cornices, parapets, and appurtenances. The facade also includes walls supported at roof level, such as penthouse walls, chimneys, and so forth.

3.2.3 *facade age*—number of years since the original Certificate of Occupancy for building was issued, or since entire facade replacement.

<sup>1</sup> This practice is under the jurisdiction of ASTM Committee **E06** on Performance of Buildings and is the direct responsibility of Subcommittee **E06.55** on Performance of Building Enclosures.

Current edition approved Nov. 1, 2013. Published November 2013. Originally approved in 2003. Last previous edition approved in 2005 as E2270 – 05. DOI: 10.1520/E2270-13.

<sup>2</sup> For referenced ASTM standards, visit the ASTM website, [www.astm.org](http://www.astm.org), or contact ASTM Customer Service at [service@astm.org](mailto:service@astm.org). For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

**3.2.4 facade inspection category**—category assigned to building facade based on the facade material, construction, age, and inspection level/extent required, as outlined in **Annex A1** of this document.

**3.2.5 facade inspection report**—a detailed documentation of qualified professional's findings, observations, discussions, conclusions, and recommendations about the subject building facades.

**3.2.6 levels of facade inspections:**

**3.2.6.1 detailed inspection**—visual observation from less than 6 ft (1.8 m) and tactile evaluation of facade components, including probing and non-destructive testing to observe concealed conditions of wall construction.

**3.2.6.2 general inspection**—visual observation of facade components from distances equal to or greater than 6 ft (1.8 m) with or without magnification or remote optical devices.

**3.2.7 maintenance personnel**—personnel who have been involved in maintenance of the subject building facades.

**3.2.8 non-destructive testing (NDT)**—a test that causes no significant structural damage to building components.

**3.2.9 owner**—the owner, agent, manager, or person in charge, of possession, operation, or management of the building, or any combination thereof.

**3.2.10 probe**—disassembly/removal of selective portions of a facade to observe concealed conditions of wall construction.

**3.2.11 public access area**—any sidewalk, street, alley, park, plaza, playground, schoolyard or other area that is open and accessible to the public, regardless of whether it is publicly or privately owned.

**3.2.12 qualified inspector**—a qualified professional or a person working under the direct supervision of a qualified professional.

**3.2.13 qualified professional**—an architect or civil/structural engineer duly licensed. The qualified professional must be knowledgeable of the design, construction, and inspection of building facades, stability, and deterioration mechanisms relating to the specific materials and assemblies particular to the facade being inspected.

**3.2.14 specifying authority**—party requiring inspection of a building facade.

NOTE 2—The specifying authority may be a governmental body.

**3.2.15 watertight integrity**—the means a facade employs to prevent water intrusion to areas or materials where water intrusion is not intended.

## 4. Significance and Use

**4.1 Intent**—This standard practice is intended to establish the minimum requirements for conducting periodic inspections of building facades to identify unsafe conditions that could cause harm to persons and property. It addresses the required content of the facade inspection to convey to the specifying authority the condition of the facade and allow comparisons of facade conditions at other times. Facade Inspection reports conducted and prepared as outlined in this standard practice will provide specifying authorities the information necessary to

mitigate the threat of harm, injury, damage, or loss to persons or property from unsafe conditions on subject facades.

**4.2 Need for Periodic Facade Inspections**—Due to age, lack of maintenance, design or construction errors, or a combination of these factors, building facades deteriorate. Based on the knowledge gained about the performance of building facades through investigation and research, governing authorities, owners, and qualified professionals are becoming more aware of potential unsafe conditions on building facades that if unaddressed, can jeopardize public safety and surrounding properties.

**4.3 Facade Service History**—Facades require periodic maintenance and repairs to extend their useful life and to minimize and/or correct problems. As a part of any facade inspection, facade service history shall be reviewed because: (1) it may indicate patterns of leakage or other performance problems leading to concealed damage and an unsafe condition; (2) it may show a poorly conceived or improperly implemented maintenance or repair procedure that can contribute and aggravate unsafe conditions; and (3) it is necessary to distinguish between original construction and subsequent repairs or modifications during the inspection process and help identify the source of potential problems.

**4.4 Who Shall Perform the Inspection**—Facade inspection shall be performed by a qualified inspector familiar with the available service history and the available design documents relevant to the building facade. The qualified inspector shall be capable of assessing both the watertight integrity and exterior conditions of the building facade to evaluate and identify potential unsafe conditions. The qualified professional who seals and signs the report shall also oversee all work of the qualified inspector and the inspection process.

**4.5 Facades Requiring Inspection**—Those facades as determined by the specifying authority that pose a potential threat of harm, injury, damage or loss to persons or property.

**4.6 Frequency, extent, and the required level of facade inspections** are dependent on facade age, material, and construction.

**4.7 Observed facade deficiencies** shall be categorized and documented in Facade Inspection Report as “unsafe condition,” “requires repair/stabilization,” or “ordinary maintenance.”

**4.8 Limitations**—Due to the construction techniques and physical properties of the many materials used in facade construction, and the inherent limitations on detecting concealed facade distress based on limited observation and probes, conducting a facade inspection does not assure that all unsafe conditions will be identified.

## FACADE INSPECTION PROCEDURE

### 5. Overview

**5.1** The following sequence of activities is intended to lead to an accumulation of information in a rational and efficient manner, so that each step enhances and supplements the

information gathered in the preceding step. Each activity is discussed in sections below:

	Section
Review of Project Documents	6
Preparation of Inspection Drawings	7
Determination of Service History	8
Assessment of Watertight Integrity	9
Facade Inspection	10
Reporting Procedures for Unsafe Conditions	11
Standard Reporting Procedures	12
Maintenance of Reports	13
Frequency, Extent, and the Required Level of Periodic Inspection of Building Facades for Unsafe Conditions	Annex A1
Detailed Assessment of Water Tightness Integrity of Exterior Facades	Appendix X1

## 6. Review of Project Documents

6.1 Review available project documents provided by the Owner, including original architectural, structural, and shop drawings.

6.2 The qualified inspector shall verify that such documents pertain to the subject building facades.

6.3 Building facades, especially historic and older buildings, may have been detailed in accordance with common practices of the time. Such information may be available in building construction and design reference books dating back to the original construction of the facade.

## 7. Preparation of Inspection Drawings

7.1 Prior to beginning the facade inspection, the qualified inspector shall oversee the acquisition or development of sufficient drawings for documentation of the inspection findings.

7.2 As a minimum, the following information shall be included:

- 7.2.1 Plot plan showing relationship to adjacent properties and publicly accessible areas,
- 7.2.2 Ground level floor plan,
- 7.2.3 Supplemental floor plans if the footprint changes between the ground and roof,
- 7.2.4 Elevation drawings of the facades to be inspected,
- 7.2.5 Penthouse level/main roof plan, and
- 7.2.6 Typical wall details.

7.3 Drawing development using digital photography, perspective corrected photography, or other photographic methods, or any combination thereof, are acceptable as long as the drawings provide a clear depiction of the facade.

7.4 In the case of a facade inspection that is precipitated by discovery of an unsafe condition, the inspection shall not be delayed to acquire or prepare drawings.

## 8. Determination of Service History

8.1 The service history of a facade includes previous maintenance, repairs, modifications and performance issues information. Gathering documentation of this history as part of a facade inspection program serves the following purposes: (a) review and confirmation of previous findings; (b) identification of wall areas or facade details that may have been repaired beyond the scope of normal maintenance, may indicate an underlying problem; (c) understanding of past and present

water infiltration activity, which can focus attention on facade areas where concealed damage is likely; and (d) prioritization of inspection areas.

8.2 As a minimum, the owner shall provide where feasible, and the inspector shall review, the following information about the facade obtained from maintenance records and interviews with the building owner, maintenance personnel or maintenance contractors, or any combination thereof, and engineers/architects involved in past inspections/repairs:

- 8.2.1 Performance problems, such as leaks, rust stains, efflorescence, cracking, spalling, bowing, and so forth,
- 8.2.2 Prior repairs, noted repeated repairs, and
- 8.2.3 Previous facade inspection reports.

## 9. Assessment of Watertight Integrity

9.1 Qualified inspector shall perform a cursory interior leak survey of the exterior facades. The information obtained from the leak survey and from the review of the service history of the facade is useful in selecting locations for detailed inspection and probes. If the specifying authorities require a more thorough assessment, refer to the guidelines in [Appendix X1](#).

## 10. Facade Inspection

10.1 Facade inspections are categorized by two levels: general inspection, and detailed inspection as defined in [Section 3](#) and noted below. A combination of general and detailed inspection is required for a facade inspection. Selection of facade inspection level and frequency is dependent upon the facade age, materials, construction, and service history of the facade. Unless otherwise determined by the specifying authority, use [Annex A1](#) to determine scope of inspection. Detailed inspection shall be on areas with greatest exposure and risk to persons or property.

10.2 *Documentation*—Regardless of the inspection level selected, document overall appearance of the facade and all significant categorized (unsafe conditions, requires repair/stabilization, and ordinary maintenance) observations on the prepared inspection drawings and by photographs.

10.3 *General Inspections*—General inspection is visual observation of facade components from distances equal to or greater than 6 ft (1.8 m) with or without magnification or remote optical devices. The qualified inspector shall methodically scan facade areas and check for out-of-plane displacement of facade elements while scanning the facade horizontally and vertically.

10.4 *Detailed Inspection*—Based upon the findings of the general inspection, the review of project documents, and the service history, the qualified inspector shall choose the representative areas to receive detailed inspection. Detailed inspection is visual observation and tactile evaluation of facade components, including probing and NDT to observe concealed conditions of wall construction. This level of inspection requires tactile contact with facade elements. The qualified inspector shall use, at a minimum, the following techniques in performing the inspection:

- 10.4.1 Viewing horizontal surfaces that can pond water (such as sills, ledges, cornices, water tables, and other such horizontal bands) from above wherever possible,



10.4.2 Checking for out-of-plane displacement of facade elements while scanning the facade horizontally and vertically,

10.4.3 Checking for signs of staining, spalling, water or moisture damage, weathering or distress of facade components,

10.4.4 Sounding of the facade surface with a “non-metallic” hammer if material delamination of facade components is possible,

10.4.5 Pushing against or pulling on facade elements, or both,

10.4.6 Pull test on adhesively attached components at building corners and in the field of the wall,

10.4.7 Evaluating sealant adhesion by NDT,

10.4.8 Probing (exterior or interior, or both) and NDT to observe concealed facade components such as anchors, inserts or support of facade components,

10.4.9 Removing loose or fractured components to reveal cause of distress, where safe to do so, and

10.4.10 Sampling of material obtained from probes for visual examination and laboratory testing as required.

## 11. Reporting Procedures for Unsafe Conditions

11.1 Report unsafe conditions to the specifying authority immediately.

11.2 Written notification shall follow promptly, including potential repair and remedial options to address the unsafe condition.

11.3 Notify the owner of the need to take immediate action to protect the public by appropriate means and that such protection shall not be removed until the unsafe condition has been remedied.

11.4 The qualified professional’s responsibility to report unsafe conditions is limited to 11.1, 11.2, and 11.3.

## 12. Standard Reporting Procedures

12.1 *Intent*—The primary intent of the report is to convey to a layperson clearly and succinctly any threat to persons or property. The secondary purposes of the report are to:

12.1.1 Convey to the specifying authority information obtained about the service history of the facade and the inspection,

12.1.2 Discuss the implications of these findings,

12.1.3 Inform the specifying authority about the condition of the facade,

12.1.4 Make general maintenance and repair recommendations, and

12.1.5 Document conditions of the facade so that it may be compared with past or future observations to establish a rate of deterioration.

12.2 *Report Content*—As a minimum, information in the report shall include:

12.2.1 Identification of all information sources,

12.2.2 Documentation and assessment of facade service history,

12.2.3 Description of observation methods and extent of inspection,

12.2.4 Documentation of relevant conditions of the facade,

12.2.5 Statement on the watertight integrity of the facades,

12.2.6 Identification of detailed inspection method and facade category,

12.2.7 Classification of conditions as “unsafe condition,” “requires repair/stabilization,” and “ordinary maintenance,”

12.2.8 Facade elevations showing relevant findings (drawings or photographs),

12.2.9 Photographic documentation of each unsafe condition,

12.2.10 Representative photographic documentation of “requires repair/stabilization” or “ordinary maintenance” conditions,

12.2.11 Discussion of the significance of findings and description of remedial recommendations and options, and

12.2.12 Signature and seal of the qualified professional, and date of inspection and report.

## 13. Maintenance of Reports

13.1 The specifying authority and the owner shall maintain a readily available copy of the facade inspection report for future reference.

# ANNEX

## (Mandatory Information)

### A1. FREQUENCY, EXTENT, AND THE REQUIRED LEVEL OF PERIODIC INSPECTION OF BUILDING FACADES FOR UNSAFE CONDITIONS

A1.1 Scope of facade inspections is dependent on facade age, material, and construction as outlined in Table A1.1, unless otherwise determined by the specifying authority.

A1.2 *Frequency of Facade Inspection*—Unless otherwise required by the specifying authorities, inspections should be performed at least once every 5 years.

**TABLE A1.1 Facade Inspections**

Facade Material and Construction	Facade Age	Facade Inspection Level and Extent	Facade Inspection Category
Brick Stone Concrete/Cast Stone Terracotta Glass Block GFRC Stucco Barrier EIFS Wall panels with adhesive attachment	More than 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade(s). 3 Probes per facade per subject building. Conduct pull-tests on adhesively attached components, minimum of 3 tests per elevation.	A
	5 to 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade. 3 Probes per facade for Barrier EIFS and other facade materials where movement or rusting is apparent.	B
Wall panels with mechanical attachment	More than 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade(s). 3 Probes per facade per subject building.	C
	5 to 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade(s) Close-up when no movement or rust staining is apparent. Otherwise, Inspect 25 % of each subject facade(s) and, 3 Probes per facade per subject building when movement or rust staining is apparent.	D
All other materials	More than 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade(s). 3 Probes per facade per subject building when movement or rust staining is apparent.	E
	5 to 20 years	General inspection: 100 % of subject facade(s). Detailed inspection: Inspect 25 % of each subject facade(s).	F

## APPENDIX

### (Nonmandatory Information)

#### X1. DETAILED ASSESSMENT OF WATER TIGHTNESS INTEGRITY OF EXTERIOR FACADES

X1.1 As a part of periodic inspection for unsafe conditions, qualified inspector shall perform a cursory interior leak survey of the exterior facades. The information obtained from the leak survey and from the review of the service history of the facade is useful in selecting locations for detailed inspection and probes.

X1.2 If the specifying authorities require a more thorough assessment, as a minimum, qualified inspector should assess the watertight integrity of the following facade components:

X1.2.1 Rain water conductors on the exterior facade,

X1.2.2 Gutters, cornice collectors and drainage details which could impact facade components,

X1.2.3 Balconies and their rain water collection systems,

X1.2.4 Scupper assemblies,

X1.2.5 Mortar and sealant joints,

X1.2.6 Exposed flashing;

X1.2.7 Membrane or sheet metal coverings, or both,

X1.2.8 Coping systems, and

X1.2.9 Any horizontal surface, component or assembly, for insufficient or improper slope.

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**APPENDIX D -  
REFERENCES MATERIAL  
COMPUTER RENDERED  
FRAME MODELS AND CALCULATIONS**

# STRUCTURAL DESIGN

**TABLE 1607.1**  
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS,  $L_e$  AND**  
**MINIMUM CONCENTRATED LIVE LOADS<sup>9</sup>**

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	-	-
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	-
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Follow spot, projections and control rooms	50	-
Lobbies	100	
Movable seats	100	
Stages and platforms	125	
Other assembly areas	100	
5. Balconies (exterior) and decks <sup>h</sup>	Same as occupancy served	-
6. Bowling alleys	75	-
7. Catwalks	40	300
8. Cornices	60	-
9. Corridors, except as otherwise indicated	100	-
10. Dance halls and ballrooms	100	-
11. Dining rooms and restaurants	100	-
12. Dwellings (see residential)	-	-
13. Elevator machine room grating (on area of 4 in <sup>2</sup> )	-	300
14. Finish light floor plate construction (on area of 1 in <sup>2</sup> )	-	200
15. Fire escapes	100	
On single-family dwellings only	40	-
16. Garages (passenger vehicles only)	40	Note a
Trucks and buses	See Section 1607.6	
17. Grandstands (see stadium and arena bleachers)	-	-
18. Gymnasiums, main floors and balconies	100	-
19. Handrails, guards and grab bars	See Section 1607.7	
20. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
21. Hotels (see residential)	-	-
22. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150 <sup>b</sup>	1,000

continued

**TABLE 1607.1-continued**  
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS,  $L_o$ , AND**  
**MINIMUM CONCENTRATED LIVE LOADS<sup>9</sup>**

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
23. Manufacturing		
Heavy	250	3,000
Light	125	2,000
24. Marquees	75	-
25. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	-	-
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000
26. Penal institutions		
Cell blocks	40	-
Corridors	100	
27. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage <sup>i</sup>	10	
Uninhabitable attics with limited storage <sup>i, j, k</sup>	20	
Habitable attics and sleeping areas	30	-
All other areas	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	100	
28. Reviewing stands, grandstands and bleachers	Note c	
29. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 nonreducible	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:		
Over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note 1	Note 1
Roofs used for promenade purposes	60	
Roofs used for roof gardens or assembly purposes	100	
30. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
31. Scuttles, skylight ribs and accessible ceilings	-	200
32. Sidewalks, vehicular driveways and yards, subject to trucking	250 <sup>d</sup>	8,000 <sup>e</sup>
33. Skating rinks	100	-

continued

Historical Record  
Dimensions and Properties

# **ROLLED SHAPES**

Steel and Wrought Iron

## **BEAMS & COLUMNS**

As Rolled in U.S.A., Period 1873 to 1952  
with Sources as Noted

Compiled and Edited by  
Herbert W. Ferris



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# EXPLANATORY NOTES

## GENERAL NOTES:

The aim of this Historical Record is to bring together into one convenient reference book a comprehensive tabulation of all rolled iron and steel beam and column shapes, many of which are not listed in current mill catalogs. The weights, areas, dimensions and properties of all rolled sections appearing in this tabulation were obtained directly from rolling mill catalogs and handbooks issued from 1873 to 1952.

Some of the shapes shown were first produced in the early 1930's and are being rolled today without change. Properties and dimensions of such sections may be found in current catalogs.

The early catalogs did not always furnish all of the information required. In those cases the missing data have been computed from such information as was given. All figures not taken from catalogs, are marked with asterisks.

Inaccuracies, and some minor errors, are apparent in the properties given in some of the earlier catalogs. Since designs may have been based on the information presented in these catalogs, original data were used in this tabulation.

The typical cross section used herein for dimension nomenclature, has sloping flanges. This was chosen to accentuate dimensions  $m$  and  $n$ . Many shapes have parallel flanges; they can be readily identified when  $m$  equals  $n$  and the percentage of slope of inside flange equals zero.

This presentation is in the form given in the catalogs of mills now rolling structural shapes.

## AMERICAN STANDARD BEAMS

Under this classification are included beams similar in weight and dimensions to the AMERICAN STANDARD BEAMS adopted by the Association of American Steel Manufacturers on January 17, 1896. These were rolled both before and after that date. (There were rollings of so-called "Standard Beams" after the above date with weights and depths the same as American Standard Beams, but with other dimensions differing. Other rollings held the dimensions but changed the weights by a small percentage.)

## BEAMS

Under this category are included shapes used more frequently as beams than as columns. If data is required for a section heavier than those listed under BEAMS, such data will be found under COLUMNS.

## COLUMNS

The tabulation of columns, includes shapes designated in catalogs as Columns, and

shapes used as columns as indicated by tabular information given in various mill catalogs and in the A.I.S.C. Steel Construction Manual.

## SECTION NUMBER

When a shape was rolled over a period of years by a mill, and the number or name of the shape varied in the catalogs issued by that mill during this period, the latest designation is shown in Column 1. When a shape has more than one number or name, all of them appear under "References" on each page.

No shape or section numbers, or names, are given in this tabulation for American Standard Beams or Wrought Iron Beams.

## WEIGHT PER FOOT

Prior to 1896, most mills gave the weight of shapes in pounds per yard. These weights have been converted to pounds per foot for consistent tabulation.

## DIMENSIONS

Dimensions  $d$ ,  $b$ ,  $t$ ,  $m$ ,  $n$ ,  $R$ ,  $R'$  for currently produced shapes are given in inches and decimals of an inch, just as they appear in the mill catalogs. In the case of obsolete shapes, where these dimensions are given in fractions of an inch in the catalogs, these fractions have been converted to their decimal equivalent.

## PROPERTIES OF SECTIONS, AXIS 1-1 and AXIS 2-2

In the earlier catalogs Section Modulus  $S$  or  $\frac{I}{C}$ , and Radius of Gyration  $r$ , about one or both axes, were omitted; these have been computed and included herein.

In these early catalogs, the range of weights and the dimensions  $m$  and  $n$  were sometimes given for sections in a series, but dimensions  $b$  and  $t$ , and section properties, were given for the lowest weight only. The increment of the web thickness was given for each additional pound of weight for the section. This increase in the web thickness of shapes heavier than the lowest weight section of a series, has been used for computing dimensions  $b$  and  $t$ , the area, and  $I$  about axis 1-1, from the data given for the lightest section, even though small errors were known to exist occasionally in the original data. This method was implied in the catalogs.

## EXAMPLE:

Given 15" I 80 lbs. and increment of web thickness, for each pound increase in weight equal to 0.02"

	$b$	$t$	$A$	$I$ (Axis 1-1)
From Catalog 15" I 80 lbs.	6.41	.77	23.5	785.9
For 15" I 100 lbs.				
Add $(20 \times .02) =$	<u>.40</u>	<u>.40</u>	$(.40 \times 15) =$	$\frac{6.0 (.40 \times 15^3)}{12} = 112.5$
For 15" I 100 lbs.	6.81	1.17	29.5	898.4



Formulas used to determine I, S and r about Axis 1-1 and Axis 2-2 where not given in catalogs:

Axis 1 - 1

$$I = \frac{bd^3 - \frac{1}{2}(b-t) \left[ (d-2n)^4 - (d-2m)^4 \right]}{12}$$

$$S = \frac{I}{1/2d}$$

$$r = \sqrt{\frac{I}{\text{Area}}}$$

Axis 2 - 2

$$I = \frac{2nb^3 + (d-2m)t^3 + 2 \frac{m-n}{(b-t)} [b^4 - t^4]}{12}$$

$$S = \frac{I}{1/2b}$$

$$r = \sqrt{\frac{I}{\text{Area}}}$$

#### COMPUTED AREAS

When all dimensions were given but the area was not, it has been computed from the dimensions. The area of the fillets was not included in the section area in early catalogs. Later, some mills included the fillet areas in the section area. Where areas have been computed herein for sections rolled by these mills, the fillet areas of course are included.

#### PERCENTAGE OF FLANGE SLOPE

With few exceptions this information was not given in the early catalogs. When sufficient dimensions were given the percentage of flange slope has been computed.

## REFERENCES

"References" appearing at the top of each page, and occasionally at the bottom of the page also, identify the rolling mills and dates of mill catalogs from which the data were taken.

The letters preceding the date designate the company that issued the catalog, as follows:

B	Bethlehem Steel Company 1907
C	The Carnegie Steel Company, Limited 1893 to 1896
C	Carnegie Steel Company 1900 to 1934
C A	Cambria Steel Company
CAM	Cambria Steel Company
C B	Carnegie Brothers & Co., Limited
CIL	Carnegie-Illinois Steel Corporation
C K	Carnegie, Kloman & Co., Union Iron Mills
C P	Carnegie, Phipps & Co., Limited
I L	Illinois Steel Company
I N	Inland Steel Company
J & L	Jones & Laughlins Limited 1893 to 1902
J & L	Jones & Laughlin Steel Company, Beginning 1903
J & L	Jones & Laughlin Steel Corporation, Beginning 1926
K	Kaiser Steel Corporation
L A	Lackawanna Steel Company
N J	New Jersey Steel & Iron Co.
P A	The Passaic Rolling Mill Co.
P E	A. & P. Roberts Company (Pencoyd Iron Works)
P H	The Phoenix Iron Company
P O	Pottsville Iron & Steel Co.
S	Bethlehem Steel Company, Beginning 1909
U S	United States Steel Company

**EARLY UNIT STRESSES USED IN TABLES OF ALLOWABLE LOADS AS  
PUBLISHED IN CATALOGS OF THE FOLLOWING MILLS**

**FOR WROUGHT IRON**

Year	Rolling Mill	Unit Stress
1873	Carnegie Kloman & Co. ("Factor of Safety 3") .....	14000 psi
1874	New Jersey Steel & Iron Co. ....	12000 psi
1881-1884	Carnegie Brothers & Co., Ltd. ....	{12000 psi 10000 psi
1884	The Passaic Rolling Mill Co. ....	{12000 psi 10000 psi
1885	The Phoenix Iron Company .....	12000 psi
1885-1887	Pottsville Iron & Steel Co. ....	12000 psi
1889	Carnegie Phipps & Co., Ltd. ....	{12000 psi 10000 psi

**FOR STEEL**

1887	Pottsville Iron & Steel Co. ....	15600 psi
1889-1893	Carnegie Phipps & Co., Ltd. (Bldgs.) .....	16000 psi
	(Bridges) .....	12500 psi
1893-1908	Jones & Laughlins Ltd. } .....	{16000 psi 12500 psi
	Jones & Laughlin Steel Co. }	
1896	Carnegie Steel Co., Ltd. (Bldgs.) .....	16000 psi
	(Bridges) .....	12500 psi
1897-1903	The Passaic Rolling Mills Co. ....	{16000 psi 12000 psi
1898-1919	Cambria Steel Co. ....	{16000 psi 12500 psi
1900-1903	Carnegie Steel Company (Bldgs.) .....	16000 psi
	(Bridges) .....	12500 psi
1907-1911	Bethlehem Steel Co. (Bldgs.) .....	16000 psi
	(Moving loads) .....	12500 psi
1915	Lackawanna Steel Co. ....	{16000 psi 12500 psi



# HISTORY OF A.S.T.M. AND A.I.S.C. STRUCTURAL STEEL SPECIFICATION STRESSES

Date	Specification	Remarks	ASTM Requirement		
			Tensile Strength psi	Minimum Yield Point psi	
1900	ASTM, A7 Bridges	Rivet Steel	50,000 to 60,000	30,000	
		Soft Steel	52,000 to 62,000	32,000	
		Medium Steel	60,000 to 70,000	35,000	
	ASTM, A9 Buildings	Rivet Steel	50,000 to 60,000	30,000	
		Medium Steel	60,000 to 70,000	35,000	
1901-1904	ASTM, A7 Bridges	Rivet Steel	50,000 to 60,000	1/2 T.S.	
		Soft Steel	52,000 to 62,000	1/2 T.S.	
		Medium Steel	60,000 to 70,000	1/2 T.S.	
	ASTM, A9 Buildings	Rivet Steel	50,000 to 60,000	1/2 T.S.	
		Medium Steel	60,000 to 70,000	1/2 T.S.	
1905-1908	ASTM, A7 Bridges	Structural Steel	Desired 60,000	--- (1)	
		Rivet Steel	Desired 50,000	--- (1)	
		Steel Castings not less than	65,000	--- (1)	
	ASTM, A9 Buildings	Rivet Steel	50,000 to 60,000	1/2 T.S.	
		Medium Steel	60,000 to 70,000	1/2 T.S.	
1909-1912	ASTM, A7 Bridges	Structural Steel	Desired 60,000	--- (1)	
		Rivet Steel	Desired 50,000	--- (1)	
		Steel Castings not less than	65,000	--- (1)	
	ASTM, A9 Buildings	Structural Steel	55,000 to 65,000	1/2 T.S.	
		Rivet Steel	48,000 to 58,000	1/2 T.S.	
1913	ASTM, A7 Bridges	Structural Steel	Desired 60,000	--- (1)	
		Rivet Steel	Desired 50,000	--- (1)	
		Steel Castings were deleted from A7			
	ASTM, A9 Buildings	Structural Steel	55,000 to 65,000	1/2 T.S.	
		Rivet Steel	48,000 to 58,000	1/2 T.S.	
1914-1923	ASTM, A7 Bridges	Structural Steel	55,000 to 65,000	1/2 T.S.	
		Rivet Steel	46,000 to 56,000	1/2 T.S.	
	ASTM, A9 Buildings	Structural Steel	55,000 to 65,000	1/2 T.S.	
		Rivet Steel	46,000 to 56,000	1/2 T.S.	
1923	AISC	Allowable basic working stress 18,000 psi			

(1) No definite requirements for yield point other than it be recorded in test reports.

Date	Specification	Remarks	ASTM Requirement	
			Tensile Strength psi	Minimum Yield Point psi
1933 (Cont.)	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised	Structural Steel	55,000 to 65,000	1/2 T.S. or not less than 30,000
	Oct. 30, 1933 ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings)	Structural Steel	60,000 to 72,000	1/2 T.S. or not less than 33,000
	ASTM, A141-32T adopted as a standard 1933 <b>AISC</b>	Rivet Steel	52,000 to 62,000	1/2 T.S. or not less than 28,000
			<b>Allowable basic working stress same as 1923 (18,000 psi)</b>	
1934-1938	ASTM, A7-34 (Bridges) adopted as a standard	Plates, Shapes, Eyebars	60,000 to 72,000	1/2 T.S. or not less than 33,000
		Eyebars Flats un-annealed	67,000 to 82,000	1/2 T.S. or not less than 36,000
	ASTM, A9-34 (Bldgs.) adopted as a standard	Structural Steel	60,000 to 72,000	1/2 T.S. or not less than 33,000
	ASTM, A141-33	Rivet Steel	52,000 to 62,000	1/2 T.S. or not less than 28,000
1936	AISC	Revised allowable basic working stress to 20,000 psi.		
1939-1948	ASTM, A7-34 and ASTM, A9-34 consolidated into one specification (in 1939) for bridges and bldgs. to be known as ASTM, A7-39	Structural Steel	60,000 to 72,000	1/2 T.S. or not less than 33,000

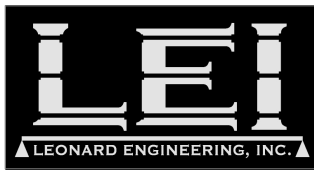
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<u>Date</u>	<u>Specification</u>	<u>Remarks</u>	<u>ASTM Requirement</u>	
			<u>Tensile Strength</u> psi	<u>Minimum Yield Point</u> psi
1939-1948 (Cont.)	ASTM, A141-36 <sup>(2)</sup>	Rivet Steel	52,000 to 62,000	1/2 T.S. or not less than 28,000
	ASTM, A141-39 <sup>(3)</sup>	Rivet Steel	52,000 to 62,000	1/2 T.S. or not less than 28,000
	AISC	Allowable basic working stress same as 1936 (20,000psi)		
1949	ASTM, A6-49T issued as a tentative revision to ASTM, A7-49T covering requirements for delivery			
	ASTM, A7-49T	Structural Steel	60,000 to 72,000	1/2 T.S. or not less than 33,000
	ASTM, A141-49T	Rivet Steel	52,000 to 62,000	min. 28,000
	AISC	Allowable basic working stress 20,000 psi		

(2) This specification is in effect a revision of and replaces rivet steel formerly in A7 and A9.

(3) Prior to the adoption of these specifications as standards, they were published as tentative from 1932-1933.





Leonard Engineering  
251 Mundy Street Suite C

Project Title:  
Engineer:  
Project Descr:

Project ID:

Printed: 6 DEC 2014, 12:58PM

File = c:\Users\TOMLEO~1\DOCUME~1\ENERCA~1\MEYERS~2\EC6

ENERCALC, INC. 1983-2014, Build:6.14.8.25, Ver:6.14.8.31

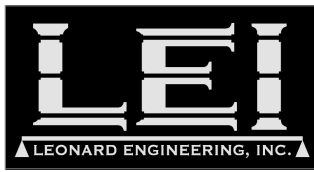
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## ASCE Seismic Base Shear

Lic. # : KW-06008661

### Meyers High School - Seismic Forces in Classroom Wings Adjacent to Main Staircase

Risk Category			Calculations per ASCE 7-10		
Risk Category of Building or Other Structure :			"III" : Buildings and other structures that represent a substantial hazard to human life in the event of a failure.		
ASCE 7-10, Page 2, Table 1.5-1					
Seismic Importance Factor = 1.25			ASCE 7-10, Page 5, Table 1.5-2		
Gridded Ss & S1values ASCE-7-10 Standard			ASCE 7-10 11.4.1		
Max. Ground Motions, 5% Damping :					
Latitude = 41.243 deg North					
Longitude = 75.884 deg West					
Location : Wilkes Barre, PA 18701					
Site Class, Site Coeff. and Design Category					
Site Classification "E" : Shear Wave Velocity must be less than 600 ft/sec			ASCE 7-10 Table 20.3-1		
Site Coefficients Fa & Fv			ASCE 7-10 Table 11.4-1 & 11.4-2		
(using straight-line interpolation from table values)					
Maximum Considered Earthquake Acceleration					
S <sub>MS</sub> = Fa * Ss			ASCE 7-10 Eq. 11.4-1		
S <sub>M1</sub> = Fv * S1			ASCE 7-10 Eq. 11.4-2		
Design Spectral Acceleration					
S <sub>DS</sub> = S <sub>MS</sub> <sup>2/3</sup>			ASCE 7-10 Eq. 11.4-3		
S <sub>D1</sub> = S <sub>M1</sub> <sup>2/3</sup>			ASCE 7-10 Eq. 11.4-4		
Seismic Design Category			ASCE 7-10 Table 11.6-1 & -2		
Resisting System			ASCE 7-10 Table 12.2-1		
Basic Seismic Force Resisting System . . .			Moment Resisting Frame Systems		
			Ordinary steel moment frames		
Response Modification Coefficient " R "			Building height Limits :		
System Overstrength Factor " Wo "			Category "A & B" Limit:		
Deflection Amplification Factor " Cd "			Category "C" Limit:		
			Category "D" Limit:		
			Category "E" Limit:		
			Category "F" Limit:		
NOTE! See ASCE 7-10 for all applicable footnotes.					
Redundancy Factor			ASCE 7-10 Section 12.3.4		
Seismic Design Category of A, B, or C therefore Redundancy Factor " p " = 1.0					
Lateral Force Procedure			ASCE 7-10 Section 12.8.2		
Equivalent Lateral Force Procedure					
The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8					
Determine Building Period			Use ASCE 12.8-7		
Structure Type for Building Period Calculation :			All Other Structural Systems		
" Ct " value = 0.020			" hn " : Height from base to highest level = 45.0 ft		
" x " value = 0.75					
" Ta " Approximate fundamental period using Eq. 12.8-7 :			Ta = Ct * (hn ^ x) = 0.347 sec		
"TL" : Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16			8.000 sec		
Building Period " Ta " Calculated from Approximate Method selected			= 0.347 sec		
" Cs " Response Coefficient			ASCE 7-10 Section 12.8.1.1		
S <sub>DS</sub> : Short Period Design Spectral Response			= 0.251		
" R " : Response Modification Factor			= 3.50		
" I " : Seismic Importance Factor			= 1.25		
Cs : Seismic Response Coefficient =			= 0.0896		



Leonard Engineering  
251 Mundy Street Suite C

Project Title:  
Engineer:  
Project Descr:

Project ID:

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ENERCALC, INC. 1983-2014, Build:6.14.8.25, Ver:6.14.8.31

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## ASCE Seismic Base Shear

Lic. # : KW-06008661

### Seismic Base Shear

ASCE 7-10 Section 12.8.1

Cs = 0.0896 from 12.8.1.1

W ( see Sum Wi below ) = 155.00 k

Seismic Base Shear V = Cs \* W = 13.89 k

### Vertical Distribution of Seismic Forces

ASCE 7-10 Section 12.8.3

"k" : hx exponent based on Ta = 1.00

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi) ^k	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment
4	20.00	50.00	1,000.00	0.2163	3.00	3.00	0.00
3	45.00	40.00	1,800.00	0.3894	5.41	8.41	30.04
2	45.00	27.00	1,215.00	0.2628	3.65	12.06	139.38
1	45.00	13.50	607.50	0.1314	1.82	13.89	302.20

Sum Wi = 155.00 k

Sum Wi \* Hi = 4,622.50 k-ft

Total Base Shear = 13.89 k

Base Moment = 489.7 k-ft

### Diaphragm Forces : Seismic Design Category "B" to "F"

ASCE 7-10 12.10.1.1

Level #	Wi	Fi	Sum Fi	Sum Wi	Fpx
4	20.00	3.00	3.00	20.00	2.51
3	45.00	5.41	8.41	65.00	5.64
2	45.00	3.65	12.06	110.00	4.93
1	45.00	1.82	13.89	155.00	4.03

Wpx ..... Weight at level of diaphragm and other structure elements attached to it.

Fi ..... Design Lateral Force applied at the level.

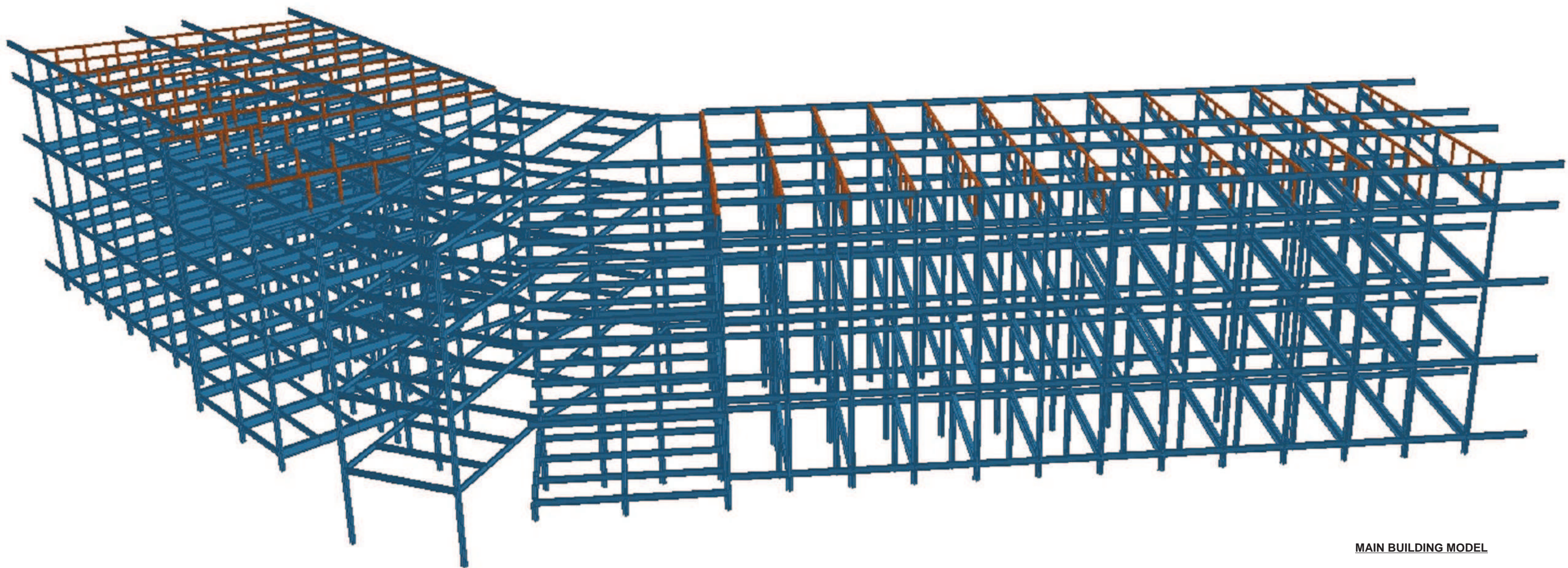
Sum Fi ..... Sum of "Lat. Force" of current level plus all levels above

MIN Req'd Force @ Level .....  $0.20 * S_{DS} * I * W_{px}$

MAX Req'd Force @ Level .....  $0.40 * S_{DS} * I * W_{px}$

Fpx : Design Force @ Level .....  $W_{px} * \text{SUM}(x \rightarrow n) Fi / \text{SUM}(x \rightarrow n) wi$ , x = Current level, n = Top Level

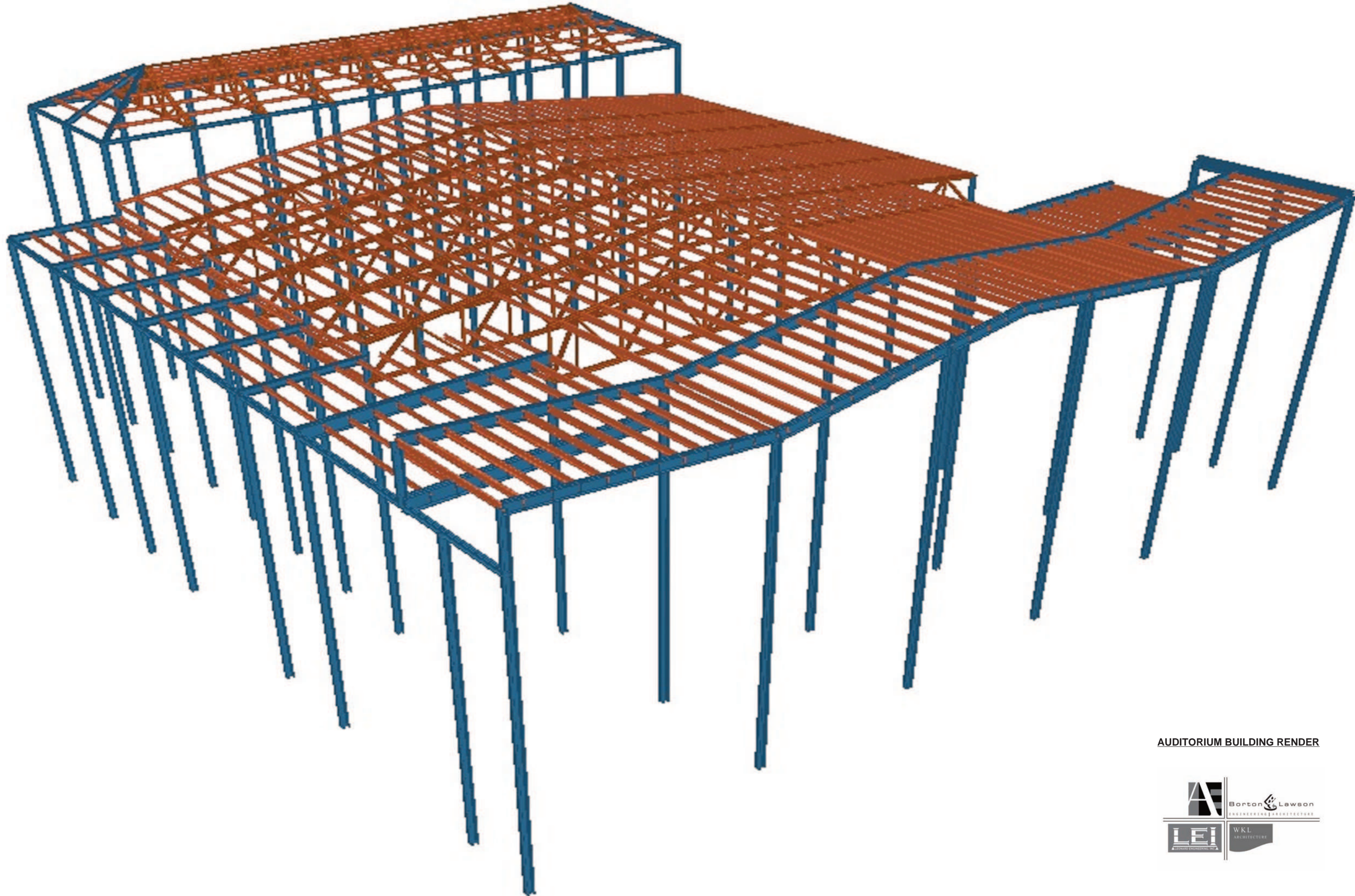




MAIN BUILDING MODEL

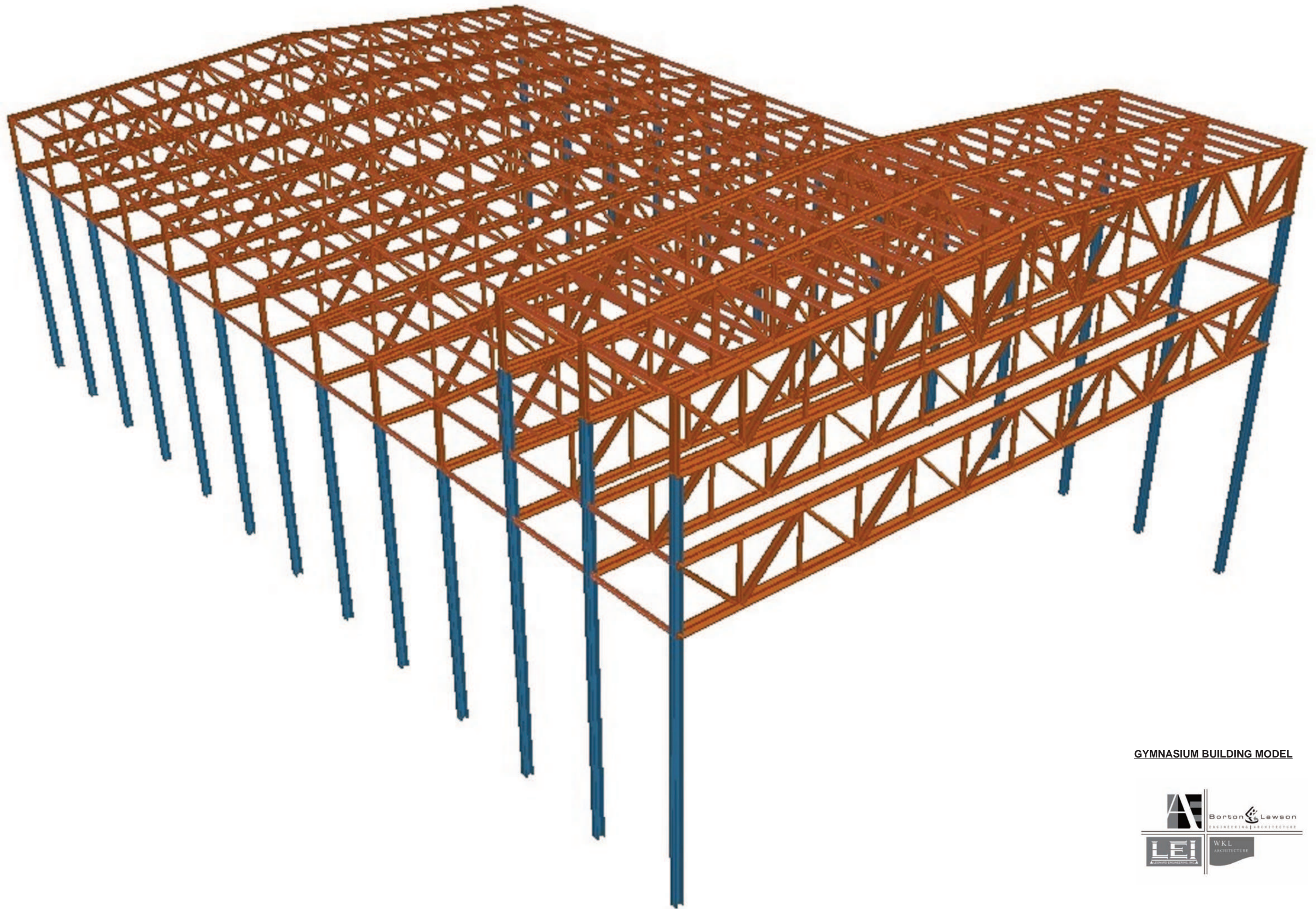






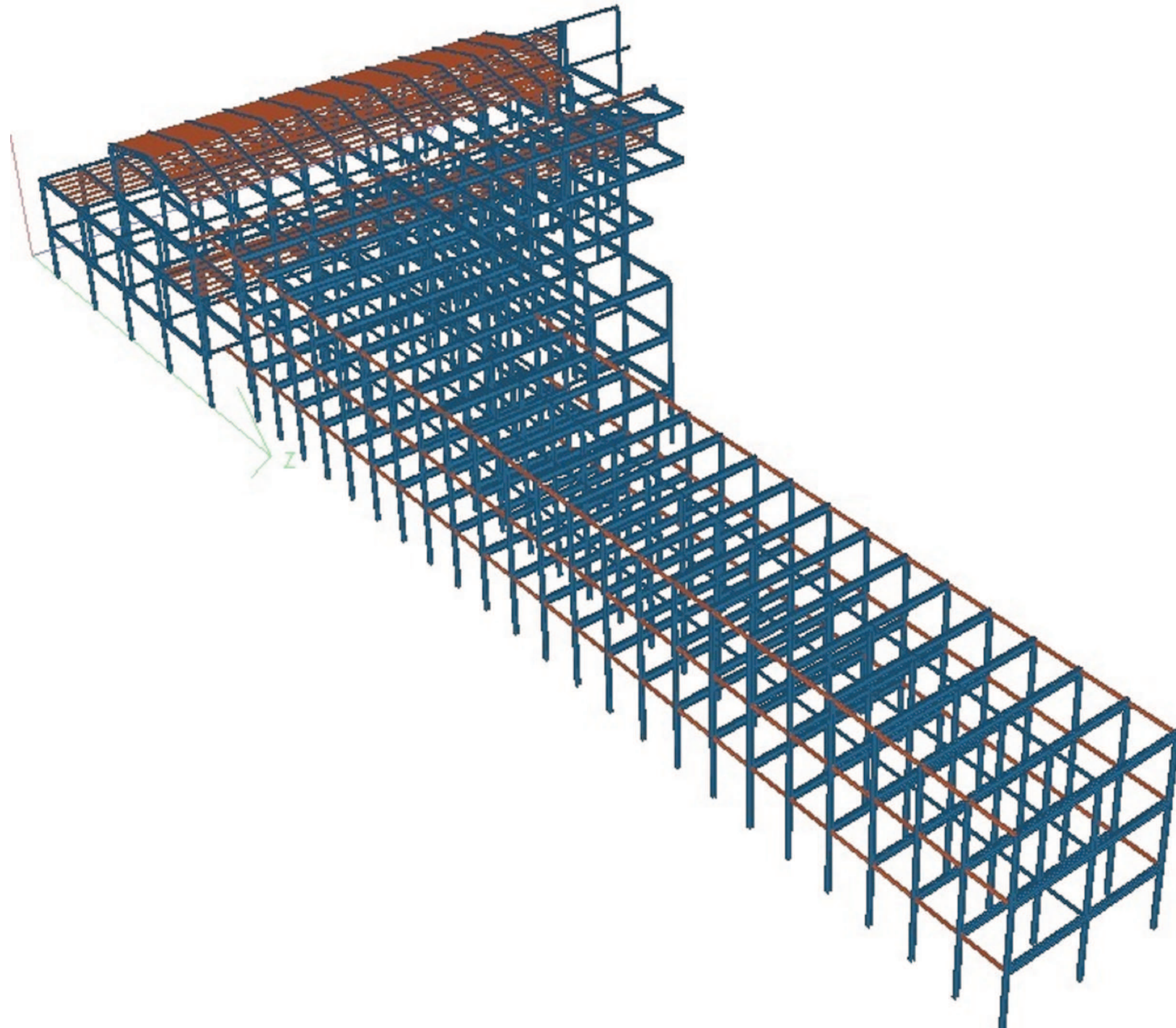
AUDITORIUM BUILDING RENDER





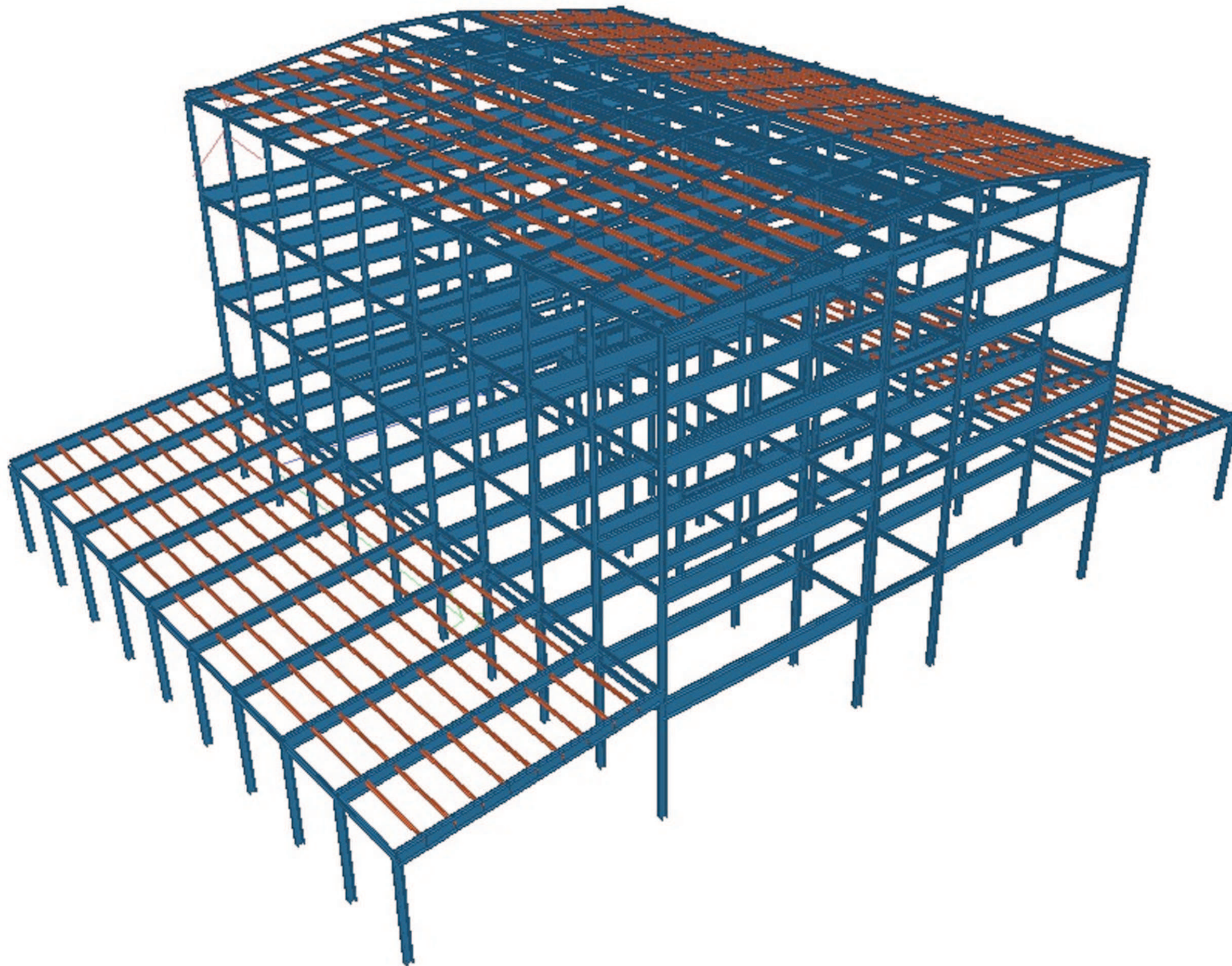
GYMNASIUM BUILDING MODEL





CAFETERIA & SHOPS BUILDINGS  
MODEL





SCIENCE BUILDING MODEL





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Job Title

Ref

By

Date 29-Oct-14

Chd

Client

File 14-124 Typical Bent Fram

Date/Time 04-Nov-2014 11:58

## Job Information

	Engineer	Checked	Approved
Name:			
Date:	29-Oct-14		

Structure Type	PLANE FRAME
----------------	-------------

Number of Nodes	42	Highest Node	59
Number of Elements	61	Highest Beam	67

Number of Basic Load Cases	3
Number of Combination Load Cases	6

Included in this printout are data for:

All	The Whole Structure
-----	---------------------

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOAD + SELFWEIGHT
Primary	2	LIVE LOAD
Primary	3	SNOW LOAD
Combination	4	GENERATED AISC GENERAL 1
Combination	5	GENERATED AISC GENERAL 2
Combination	6	GENERATED AISC GENERAL 3
Combination	7	GENERATED AISC GENERAL 4
Combination	8	GENERATED AISC GENERAL 5
Combination	9	GENERATED AISC GENERAL 6



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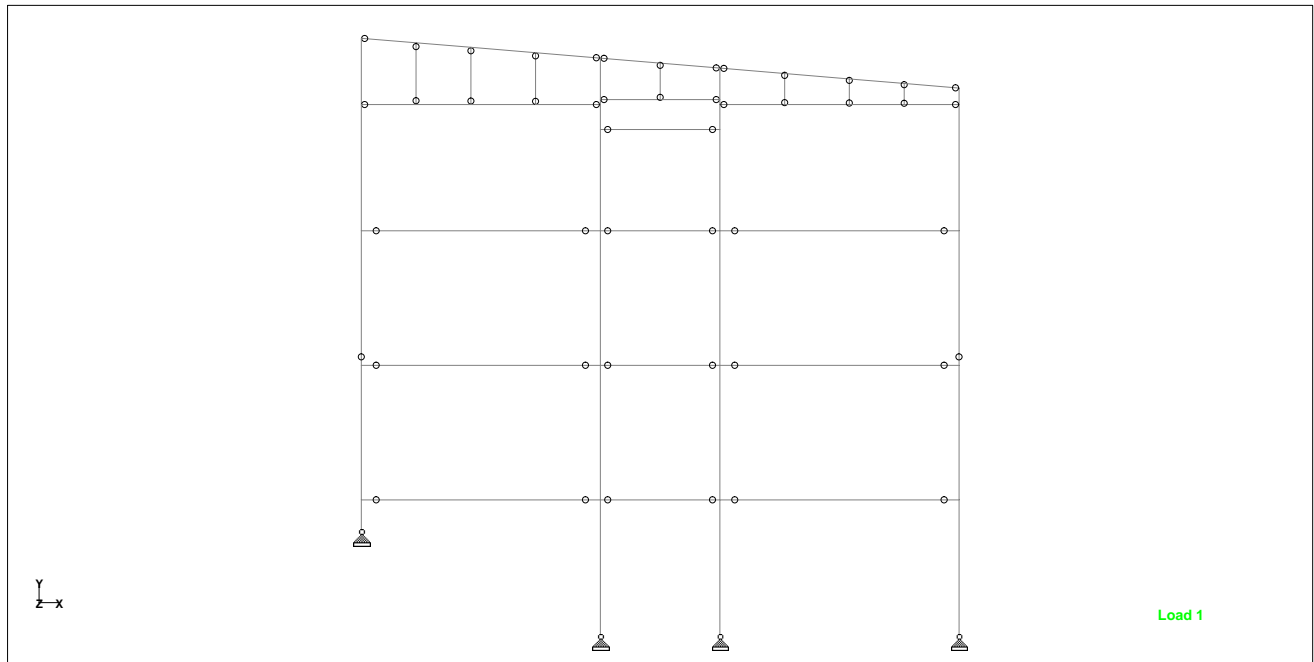
Date 29-Oct-14

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Client

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Date/Time 04-Nov-2014 11:58



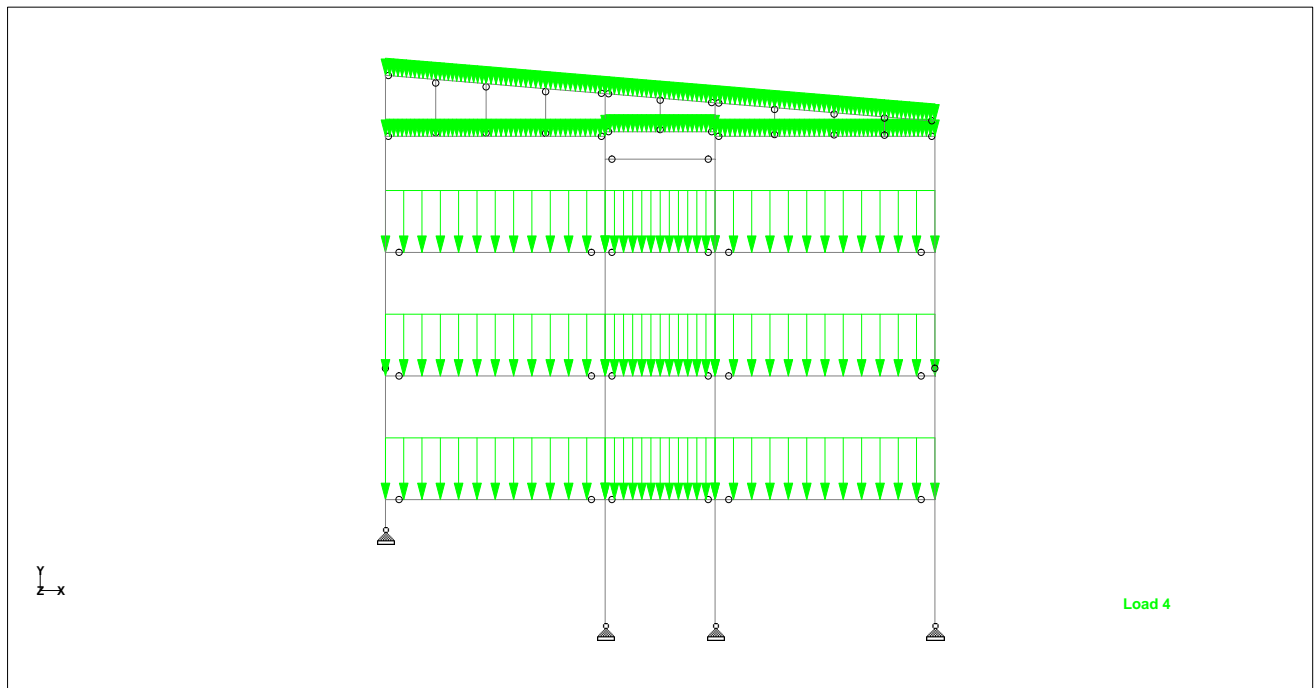
Whole Structure





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Part	Ref	
By	Date29-Oct-14	Chd
Client	File 14-124 Typical Bent Fram	Date/Time 04-Nov-2014 11:58

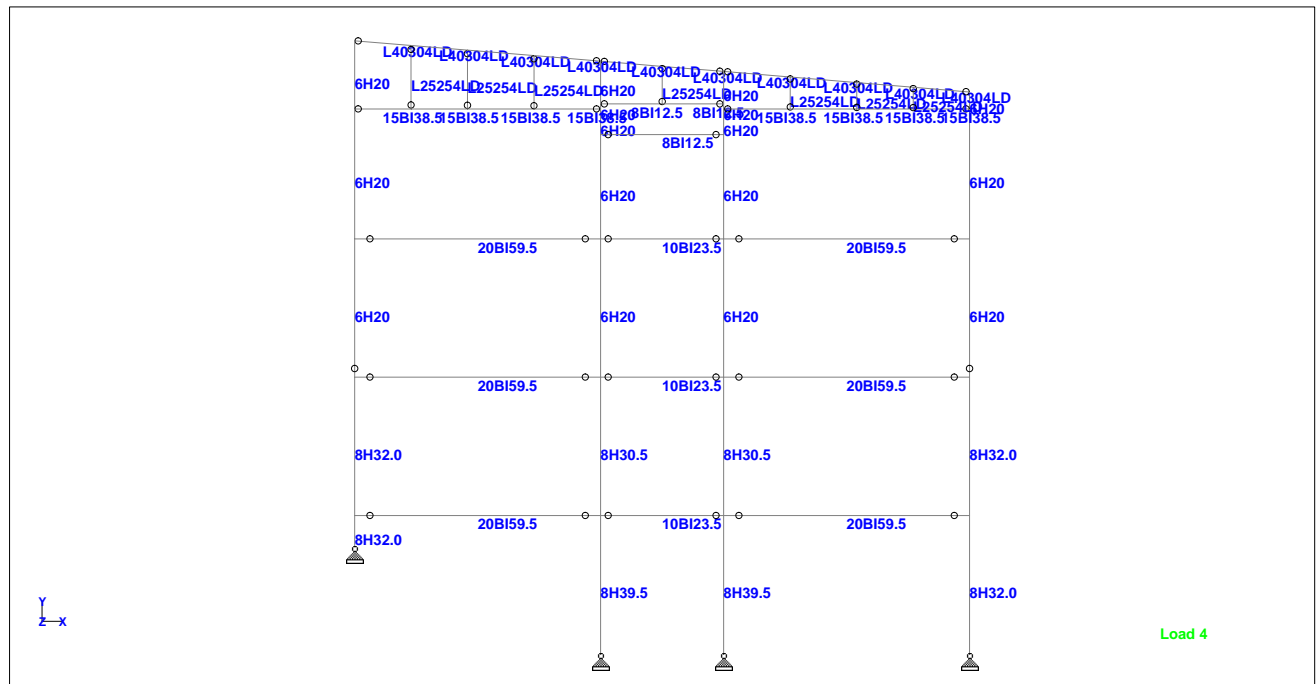


Frame Loads

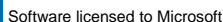


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Part		
Job Title	Ref	
	By	Date 29-Oct-14 Chd
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Frame Sizes



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Date 29-Oct-14

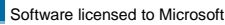
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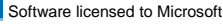
Date 29-Oct-14

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Date 29-Oct-14

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						Y	PROPERTIES IN INCH UNIT	
							== ==	-----
MEMBER	3	*	UPT				--Z	AX = 11.50
DESIGN CODE	*	UP 8H39.5					AY = 2.84	
AISC-1989	*	=====				== ==	AZ = 9.05	
							SX = 12.13	
							SZ = 34.37	
							RX = 2.06	
							RZ = 3.48	
0.0 (KIP-FEET)								
PARAMETER						L8	STRESSES	
IN KIP	INCH					L8 L8	IN KIP INCH	
KL/R-Y=	78.66				L8		FA = 13.53	
KL/R-Z=	46.49						fa = 7.96	
UNL =	162.00			L8 L8			FCZ = 18.00	
CB =	1.00						FTZ = 18.00	
CMY =	0.85			L8			FCY = 22.50	
CMZ =	0.85		L8 L8				FTY = 22.50	
FYLD =	30.00	L0					fbz = 0.00	
NSF =	1.00	+--+---+---+---+---+---+---+---					fby = 0.00	
DFF =	0.00	0.0					Fey = 24.13	
dff=	0.00						Fez = 69.09	
ABSOLUTE MZ ENVELOPE							FV = 12.00	
(WITH LOAD NO.)							fv = 0.00	
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)								
-----								
	AXIAL		SHEAR-Y		SHEAR-Z	MOMENT-Y	MOMENT-Z	
VALUE	0.0		0.0		0.0	0.0	0.0	
LOCATION	0.0		0.0		0.0	0.0	0.0	
LOADING	0		0		0	0	0	
*****								
*							*	
*							*	
DESIGN SUMMARY (KIP-FEET)							*	
-----							*	
							*	
RESULT/		CRITICAL COND/		RATIO/		LOADING/	*	
FX		MY		MZ		LOCATION		
=====		=====		=====		=====		
PASS		AISC- H1-1		5.880E-01		5		
91.52 C		0.00		0.00		0.00		
							*	
*****								



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Job Title	
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Ref
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By \_\_\_\_\_

Date 29-Oct-14

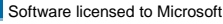
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Client

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*****						Y		PROPERTIES	
								IN INCH UNIT	
MEMBER     5     *						=====		-----	
UPT								AX = 17.33	
DESIGN CODE    *								AY = 7.50	
AISC-1989      *						--Z		AZ = 10.32	
*						=====		SY = 13.78	
*								SZ = 117.15	
*						== ===		RY = 1.78	
*****								RZ = 8.22	
PARAMETER						97.8 (KIP-FEET)			
IN KIP    INCH						L5   L5   L5		STRESSES	
-----						+-----		IN KIP    INCH	
KL/R-Y= 161.45						L5                  L5		FA = 5.73	
KL/R-Z= 35.03						L5                  L5		fa = 0.00	
UNL    = 12.00								FCZ = 19.80	
CB     = 1.00						L5                  L5		FTZ = 19.80	
CMY    = 0.85								FCY = 22.50	
CMZ    = 0.85								FTY = 22.50	
FYLD   = 30.00						L0		fbz = 10.02	
NSF     = 1.00						+---+---+---+---+---+---+---+---+---		fby = 0.00	
DFF     = 0.00						-5.4		Fey = 5.73	
dff=     0.00						ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)		Fez = 121.71	
								FV = 12.00	
								fv = 0.00	
						MAX FORCE/ MOMENT SUMMARY   (KIP-FEET)			
						-----			
						AXIAL		SHEAR-Y	
						SHEAR-Z		MOMENT-Y	
						MOMENT-Z			
VALUE						0.0		0.0	
LOCATION						0.0		0.0	
LOADING						0		0	
*****									
*									
*						DESIGN SUMMARY   (KIP-FEET)			
*						-----			
*									
RESULT/						CRITICAL COND/		LOADING/	
FX						MY		LOCATION	
=====						=====		=====	
PASS						AISC- H1-3		5	
0.00 C						0.00		12.00	
*****									
*									



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**Steel Design (Track 2) Beam 6 Check 1**

*****					Y	PROPERTIES	
MEMBER 6					---	IN	UNIT
DESIGN CODE *					---	---	---
AISC-1989					---	---	---
*****					---	---	---
<---LENGTH (FT)= 12.00 --->					---	---	---
*****					---	---	---
31.0 (KIP-FEET)					---	---	---
PARAMETER					---	---	---
IN KIP INCH					---	---	---
KL/R-Y= 106.22					---	---	---
KL/R-Z= 34.21					---	---	---
UNL = 12.00					---	---	---
CB = 1.00					---	---	---
CMY = 0.85					---	---	---
CMZ = 0.85					---	---	---
FYLD = 30.00					---	---	---
NSF = 1.00					---	---	---
DFE = 0.00					---	---	---
dff= 0.00					---	---	---
ABSOLUTE MZ ENVELOPE					---	---	---
(WITH LOAD NO.)					---	---	---
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					---	---	---
-----					---	---	---
AXIAL					---	---	---
SHEAR-Y					---	---	---
SHEAR-Z					---	---	---
MOMENT-Y					---	---	---
MOMENT-Z					---	---	---
VALUE					---	---	---
LOCATION					---	---	---
LOADING					---	---	---
*****					---	---	---
* DESIGN SUMMARY (KIP-FEET)					---	---	---
* -----					---	---	---
* RESULT/ CRITICAL COND/ RATIO/ LOADING/					---	---	---
* FX MY MZ LOCATION					---	---	---
* =====					---	---	---
* PASS AISC- H1-3 7.697E-01 5					---	---	---
* 0.00 C 0.00 -31.02 6.00					---	---	---
* -----					---	---	---
*****					---	---	---



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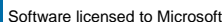
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## Steel Design (Track 2) Beam 7 Check 1

*****					Y	PROPERTIES	
						IN	INCH UNIT
MEMBER	7	*	=====	=====	---	---	
		*	UPT				
DESIGN CODE	*		UP 20BI59.5		--Z		
AISC-1989	*	=====	=====	=====	---	---	
	*						
<---LENGTH (FT)= 24.00 --->							
*****							
97.8 (KIP-FEET)							
PARAMETER			L5	L5	L5	STRESSES	
IN KIP						IN	INCH
INCH							
KL/R-Y=	161.45	+	L5		L5	FA	= 5.73
KL/R-Z=	35.03	+	L5		L5	fa	= 0.00
UNL	= 12.00					FCZ	= 19.80
CB	= 1.00	+	L5		L5	FTZ	= 19.80
CMY	= 0.85					FCY	= 22.50
CMZ	= 0.85	+				FTY	= 22.50
FYLD	= 30.00	L0			L0	fbz	= 10.02
NSF	= 1.00	+	-----	-----	-----	fbv	= 0.00
DFE	= 0.00	-5.4				Fey	= 5.73
dff=	0.00					Fez	= 121.71
ABSOLUTE MZ ENVELOPE						FV	= 12.00
(WITH LOAD NO.)						fv	= 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)							
-----							
	AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z	
VALUE	0.0		0.0	0.0	0.0	0.0	
LOCATION	0.0		0.0	0.0	0.0	0.0	
LOADING	0		0	0	0	0	
*****							
* DESIGN SUMMARY (KIP-FEET) *							
* ----- *							
* RESULT/ CRITICAL COND/ RATIO/ LOADING/ *							
* FX MY MZ LOCATION *							
=====							
PASS	AISC- H1-3	5.062E-01	5				
0.00 C	0.00	-97.84	12.00				
*****							
*****							





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*****		Y	PROPERTIES
MEMBER 9		===	IN INCH UNIT
UP 8H30.5			AX = 8.82
DESIGN CODE			AY = 2.48
AISC-1989		--Z	AZ = 6.65
			SY = 7.28
			SZ = 25.58
			RY = 1.65
			RZ = 3.41
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*****			

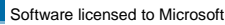
PARAMETER		0.0 (KIP-FEET)		L8		STRESSES	
IN KIP	INCH					IN KIP	INCH
KL/R-Y=	98.45					FA	= 11.85
KL/R-Z=	47.55					fa	= 7.30
UNL	= 162.00					FCZ	= 18.00
CB	= 1.00					FTZ	= 18.00
CMY	= 0.85					FCY	= 22.50
CMZ	= 0.85					FTY	= 22.50
FYLD	= 30.00					fbz	= 0.00
NSF	= 1.00					fby	= 0.00
DFF	= 0.00					Fey	= 15.41
dff=	0.00					Fez	= 66.04
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)						FV	= 12.00
						fv	= 0.00

## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	AISC- H1-1	6.166E-01	5
64.40 C	0.00	-0.01	0.00



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### Steel Design (Track 2) Beam 10 Check 1

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## Steel Design (Track 2) Beam 11 Check 1

*****					Y	PROPERTIES	
						IN	INCH UNIT
MEMBER	11	*	=====	UPT	---	---	
		*	UP	8H32.0		--Z	
DESIGN CODE	*		=====		---	---	
AISC-1989	*		=====		---	---	
	*						
<---LENGTH (FT)= 13.50 --->							
*****							
PARAMETER	0.0 (KIP-FEET)					STRESSES	
IN KIP	INCH		L8			IN KIP	INCH
KL/R-Y=	80.26			L8		FA	= 13.41
KL/R-Z=	47.69					fa	= 4.48
UNL	= 162.00			L8		FCZ	= 18.00
CB	= 1.00					FTZ	= 18.00
CMY	= 0.85			L8		FCY	= 22.50
CMZ	= 0.85				L8 L8	FTY	= 22.50
FYLD	= 30.00					fbz	= 0.01
NSF	= 1.00					fbz	= 0.00
DFE	= 0.00	0.0				Fey	= 23.18
dfv	= 0.00					Fez	= 65.65
ABSOLUTE MZ ENVELOPE						FV	= 12.00
(WITH LOAD NO.)						fv	= 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)							
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z		
VALUE	0.0	0.0	0.0	0.0	0.0		
LOCATION	0.0	0.0	0.0	0.0	0.0		
LOADING	0	0	0	0	0		
*****							
* DESIGN SUMMARY (KIP-FEET) *							
* ----- *							
* RESULT/ CRITICAL COND/ RATIO/ LOADING/ *							
* FX MY MZ LOCATION *							
=====							
PASS AISC- H1-1 3.344E-01 8							
41.07 C 0.00 -0.01 0.00							
*****							



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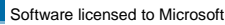
Client

File 14-124 Typical Bent Fram

Date/Time 04-Nov-2014 11:58

## Steel Design (Track 2) Beam 12 Check 1

*****					Y	PROPERTIES	
						IN	INCH UNIT
MEMBER	12	*	=====		---	---	
		*	UPT				
DESIGN CODE	*		UP 20BI59.5			--Z	
AISC-1989	*	=====		=====	---	---	
	*						
<---LENGTH (FT)= 24.00 --->							
*****							
97.8 (KIP-FEET)							
PARAMETER			L5	L5	L5	STRESSES	
IN KIP						IN	INCH
INCH							
KL/R-Y=	161.45	+	L5		L5	FA	= 5.73
KL/R-Z=	35.03	+	L5		L5	fa	= 0.00
UNL	= 12.00					FCZ	= 19.80
CB	= 1.00	+	L5		L5	FTZ	= 19.80
CMY	= 0.85					FCY	= 22.50
CMZ	= 0.85	+				FTY	= 22.50
FYLD	= 30.00	L0			L0	fbz	= 10.02
NSF	= 1.00	+	+-----				



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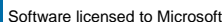
Chd

File 14-124 Typical Bent Fram

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*****					
				Y	PROPERTIES
					IN INCH UNIT
MEMBER	13	*	=====		=== ===
			UPT		
DESIGN CODE	*		UP 10BI23.5		--Z
AISC-1989	*		=====		=== ===
					SZ = 24.43
			<---LENGTH (FT)= 12.00 --->		RZ = 4.21
*****					
		31.0 (KIP-FEET)			
PARAMETER			L5 L5 L5	STRESSES	
IN KIP INCH				IN KIP INCH	
-----+		L5 L5		-----	
KL/R-Y=	106.22			FA =	18.00
KL/R-Z=	34.21			fa =	0.00
UNL	= 12.00			FCZ =	19.80
CB	= 1.00	L5		FTZ =	19.80
CMY	= 0.85			FCY =	22.50
CMZ	= 0.85			FTY =	22.50
FYLD	= 30.00	L0		fzbz =	15.24
NSF	= 1.00	+--+--+--+--+--+--+--+--+---		fbz =	0.00
DFF	= 0.00	-1.7		Fey =	13.24
dff=	0.00	ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)		Fez =	127.59
				FV =	12.00
				fv =	0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET) -----					
AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/ FX	CRITICAL COND/ MY		RATIO/ MZ		LOADING/ LOCATION
=====					
PASS	AISC-H2-1	7.697E-01		5	
0.00 T	0.00	-31.02		6.00	
*****					





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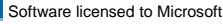
Date 29-Oct-14

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Date/Time 04-Nov-2014 11:58



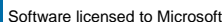
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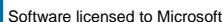
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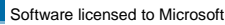
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### Steel Design (Track 2) Beam 19 Check 1

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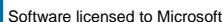
Date 29-Oct-14

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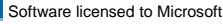
Date 29-Oct-14

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File 14-124 Typical Bent Fram

Date/Time 04-Nov-2014 11:58



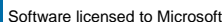
Rev

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						Y	PROPERTIES IN INCH UNIT	
							== ==	----
MEMBER 22 *  =====							--Z	AX = 5.82
UP 6H20 UPT								AY = 1.50
DESIGN CODE *  =====								AZ = 4.51
AISC-1989 * =====								SY = 4.47
*								SZ = 12.89
*  <---LENGTH (FT)= 12.67 --->								RX = 1.51
*****								RZ = 2.58
0.1 (KIP-FEET)								
PARAMETER							L8	STRESSES
IN KIP	INCH						L8	IN KIP INCH
-----+							-----	
KL/R-Y=	100.69						L8	FA = 11.65
KL/R-Z=	58.97						L8	fa = 1.94
UNL =	152.00							FCZ = 18.00
CB =	1.00	+L8						FTZ = 18.00
CMY =	0.85	L8						FCY = 22.50
CMZ =	0.85	+ L8						FTY = 22.50
FYLD =	30.00	L8						fbz = 0.02
NSF =	1.00	+++++-----						fby = 0.00
DFF =	0.00	0.0						Fey = 14.73
dff=	0.00							Fez = 42.95
ABSOLUTE MZ ENVELOPE								FV = 12.00
(WITH LOAD NO.)								fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)								
-----								
AXIAL		SHEAR-Y		SHEAR-Z		MOMENT-Y		MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0	0	0	0
*****								
*								
* DESIGN SUMMARY (KIP-FEET)								
* -----								
*								
*								
* RESULT/ CRITICAL COND/ RATIO/ LOADING/								
FX MY MZ LOCATION								
=====								
PASS	AISC- H1-1	1.673E-01	6					
11.28 C	0.00	-0.02	0.00					
*****								
*								





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*****		Y	PROPERTIES
MEMBER 23		===	IN INCH UNIT
DESIGN CODE			AX = 5.82
AISC-1989			AY = 1.50
			AZ = 4.51
			SY = 4.47
			SZ = 12.89
			RY = 1.51
			RZ = 2.58

PARAMETER		0.1 (KIP-FEET)		L8		STRESSES		
IN KIP	INCH					IN KIP	INCH	
KL/R-Y=	80.82					FA	= 13.36	
KL/R-Z=	47.33					fa	= 2.85	
UNL	= 122.00					FCZ	= 18.00	
CB	= 1.00					FTZ	= 18.00	
CMY	= 0.85					FCY	= 22.50	
CMZ	= 0.85					FTY	= 22.50	
FYLD	= 30.00					fbz	= 0.01	
NSF	= 1.00					fby	= 0.00	
DFF	= 0.00					Fey	= 22.86	
dff=	0.00					Fez	= 66.67	
				FV				= 12.00
				fv				= 0.00

## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	AISC- H1-1	2.135E-01	6
16.58 C	0.00	0.01	0.00



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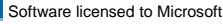
Date 29-Oct-14

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						Y	PROPERTIES IN INCH UNIT	
*****								
MEMBER	25	*	=====				== ==	AX = 5.82
		*	UPT					AY = 1.50
DESIGN CODE	*		UP 6H20				--Z	AZ = 4.51
AISC-1989	*	=====				== ==	SY = 4.47	
		*						SZ = 12.89
		*	<---LENGTH (FT)= 12.67 --->					RZ = 1.51
*****								RZ = 2.58
			0.0 (KIP-FEET)					
PARAMETER							L8	STRESSES
IN KIP	INCH						L8	IN KIP INCH
		+					L8	
KL/R-Y=	100.69							fA = 11.65
KL/R-Z=	58.97	+					L8	fa = 1.91
UNL =	152.00		L8				L8	FCZ = 18.00
CB =	1.00	+						FTZ = 18.00
CMY =	0.85		L8				L8	FCY = 22.50
CMZ =	0.85	+	L8	L8				FTY = 22.50
FYLD =	30.00						L8	fbz = 0.01
NSF =	1.00	+						fbz = 0.00
DFF =	0.00	0.0						Fey = 14.73
dff=	0.00							Fez = 42.95
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)								FV = 12.00
								fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)								
-----								
		AXIAL	SHEAR-Y		SHEAR-Z	MOMENT-Y	MOMENT-Z	
VALUE	0.0	0.0	0.0		0.0	0.0	0.0	
LOCATION	0.0	0.0	0.0		0.0	0.0	0.0	
LOADING	0	0	0		0	0	0	
*****								
*								*
*								*
DESIGN SUMMARY (KIP-FEET)								*
-----								*
*								*
RESULT/ FX	CRITICAL COND/ MY		RATIO/ MZ		LOADING/ LOCATION			*
=====								
PASS	AISC- H1-1	1.649E-01		6				
11.14 C	0.00	-0.01		0.00				
*								*
*****								



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*****		Y	PROPERTIES
MEMBER 26		===	IN INCH UNIT
UP 15BI38.5			AX = 11.26
DESIGN CODE		--Z	AY = 4.35
AISC-1989			AZ = 7.22
		===	SY = 8.13
			SZ = 59.06
<---LENGTH (FT)= 5.50 --->			RY = 1.56
*****			RZ = 6.27

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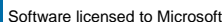
## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	AISC- H1-3	4.880E-01	6
0.01 C	0.00	-47.55	5.50





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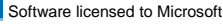
Date 29-Oct-14

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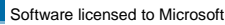
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```

*****
MEMBER      29          *
DESIGN CODE  *
AISC-1989    *
              *
              * |<---LENGTH (FT)=   6.67 --->|
*****

                                Y
PROPERTIES
IN INCH UNIT
-----
AX =   5.82
AY =   1.50
AZ =   4.51
SY =   4.47
SZ =  12.89
RY =   1.51
RZ =   2.58

                                0.1 (KIP-Feet)
PARAMETER     L8
IN KIP  INCH  L8  L8
-----+-----
KL/R-Y=  53.00 +
KL/R-Z=  31.04 +
UNL   =  80.00 |
CB    =   1.00 +
CMY   =   0.85 |
CMZ   =   0.85 +
FYLD  =  30.00 |
NSF   =   1.00 |
DFF   =   0.00 +
dff=   0.00    +
                +-----+-----+-----+-----+-----+-----+-----+-----+-----+-----+
                ABSOLUTE MZ ENVELOPE
                (WITH LOAD NO.)

STRESSES
IN KIP  INCH
-----+-----
FA   =  15.41
fa   =   0.27
FCZ  =  19.80
FTZ  =  19.80
FCY  =  22.50
FTY  =  22.50
fbz  =   0.05
fby  =   0.00
Fey  =  53.17
Fez  = 155.04
FV   =  12.00
fv   =   0.01

MAX FORCE/ MOMENT SUMMARY   (KIP-Feet)
-----

AXIAL           SHEAR-Y       SHEAR-Z       MOMENT-Y       MOMENT-Z

VALUE           0.0            0.0            0.0            0.0            0.0
LOCATION          0.0            0.0            0.0            0.0            0.0
LOADING         0             0             0             0             0

*****
*
*                               DESIGN SUMMARY   (KIP-Feet)
*                               -----
*
* RESULT/          CRITICAL COND/        RATIO/        LOADING/
FX              MY              MZ              LOCATION
=====
PASS           AISC- H1-3          2.031E-02         6
1.59 C         0.00                 0.05              0.00
*
*****

```

### Steel Design (Track 2) Beam 30 Check 1

										Y	PROPERTIES IN INCH UNIT	
*****												
MEMBER	30	*	=====						===	==	AX = 5.82	
		*	UPT								AY = 1.50	
DESIGN CODE	*		UP 6H20							--Z	AZ = 4.51	
AISC-1989	*		=====						===	==	SY = 4.47	
	*										SZ = 12.89	
	*		<---LENGTH (FT)= 0.50 --->								RX = 1.51	
*****												RZ = 2.58
		0.0 (KIP-FeET)										
PARAMETER		L8							STRESSES			
IN KIP	INCH	L8	L8					IN KIP	INCH			
KL/R-Y=	3.97	+		L8					FA =	17.88		
KL/R-Z=	2.33	+			L8					fa =	1.21	
UNL	= 6.00					L8					FCZ =	19.80
CB	= 1.00	+								FTZ =	19.80	
CMY	= 0.85						L8	L8		FCY =	22.50	
CMZ	= 0.85	+							L8	FTY =	22.50	
FYLD	= 30.00								L8	fbz =	0.03	
NSF	= 1.00	+-----+-----+-----+-----+-----+-----+-----+-----							fbx =	0.00		
DFF	= 0.00	0.0									Fey =	9453.42
dff=	0.00							ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)		Fez=		0.2757E+5
										FV =		12.00
										fv =		0.01
MAX FORCE/ MOMENT SUMMARY (KIP-FeET)												
-----												
	AXIAL	SHEAR-Y		SHEAR-Z		MOMENT-Y		MOMENT-Z				
VALUE	0.0	0.0		0.0		0.0		0.0				
LOCATION	0.0	0.0		0.0		0.0		0.0				
LOADING	0	0		0		0		0				
*****												
*											*	
*	DESIGN SUMMARY (KIP-FeET)										*	
*	-----										*	
*											*	
*	RESULT/ FX	CRITICAL COND/ MY		RATIO/ MZ		LOADING/ LOCATION				*		
	=====	=====		=====		=====						
	PASS	AISC- H1-3		6.910E-02		6						
	7.04 C	0.00		-0.03		0.00						
*											*	
*****												





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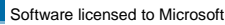
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## Steel Design (Track 2) Beam 31 Check 1

*****					Y	PROPERTIES	
MEMBER 31					---	IN	UNIT
DESIGN CODE *					---	---	
AISC-1989					---	---	
*****					---	---	
<---LENGTH (FT)= 0.50 --->					---	---	
*****					---	---	
0.1 (KIP-FEET)					---	---	
PARAMETER					---	---	
IN KIP INCH					---	---	
KL/R-Y= 3.97					---	---	
KL/R-Z= 2.33					---	---	
UNL = 6.00					---	---	
CB = 1.00					---	---	
CMY = 0.85					---	---	
CMZ = 0.85					---	---	
FYLD = 30.00					---	---	
NSF = 1.00					---	---	
DFE = 0.00					---	---	
dff= 0.00					---	---	
ABSOLUTE MZ ENVELOPE					---	---	
(WITH LOAD NO.)					---	---	
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					---	---	
AXIAL					---	---	
SHEAR-Y					---	---	
SHEAR-Z					---	---	
MOMENT-Y					---	---	
MOMENT-Z					---	---	
VALUE					---	---	
LOCATION					---	---	
LOADING					---	---	
*****					---	---	
DESIGN SUMMARY (KIP-FEET)					---	---	
RESULT/					---	---	
FX					---	---	
CRITICAL COND/					---	---	
MY					---	---	
RATIO/					---	---	
MZ					---	---	
LOADING/					---	---	
LOCATION					---	---	
PASS					---	---	
7.01 C					---	---	
AISC- H1-3					---	---	
0.00					---	---	
7.079E-02					---	---	
-0.07					---	---	
6					---	---	
0.00					---	---	
*****					---	---	

### Steel Design (Track 2) Beam 32 Check 1

				Y		PROPERTIES	
*****				==		IN INCH UNIT	
MEMBER 32		UP 6H20		UP		AX = 5.82	
DESIGN CODE		AISC-1989		--Z		AY = 1.50	
						AZ = 4.51	
						SY = 4.47	
						SZ = 12.89	
						RY = 1.51	
						RZ = 2.58	
*****							
		0.0 (KIP-FEET)					
PARAMETER		L8				STRESSES	
IN KIP INCH		L8				IN KIP INCH	
-----		+ L8 L8				-----	
KL/R-Y= 13.25		+ L8				FA = 17.54	
KL/R-Z= 7.76		+ L8				fa = 0.26	
UNL = 20.00		+ L8 L8				FCZ = 19.80	
CB = 1.00		+ L8 L8				FTZ = 19.80	
CMY = 0.85		+ L8 L8				FCY = 22.50	
CMZ = 0.85		+ L8				FTY = 22.50	
FYLD = 30.00		+ L0				fbz = 0.03	
NSF = 1.00		+-----				fby = 0.00	
DFF = 0.00		0.0				Fey = 850.47	
dff= 0.00		ABSOLUTE MZ ENVELOPE				Fcz = 2479.89	
		(WITH LOAD NO.)				FV = 12.00	
						fv = 0.01	
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
		-----					
		AXIAL		SHEAR-Y		SHEAR-Z	
		MOMENT-Y		MOMENT-Z			
VALUE		0.0		0.0		0.0	
LOCATION		0.0		0.0		0.0	
LOADING		0		0		0	
*****							
*						*	
*		DESIGN SUMMARY (KIP-FEET)				*	
*		-----				*	
*						*	
*						*	
RESULT/		CRITICAL COND/		RATIO/		LOADING/	
FX		MY		MZ		LOCATION	
=====		=====		=====		=====	
PASS		AISC- H1-3		1.605E-02		6	
1.49 C		0.00		0.03		0.00	
*						*	
*****							



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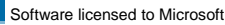
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*****										Y		PROPERTIES	
*****												IN INCH UNIT	
MEMBER 33 *										=====		===	
UP 6H20												--Z	
DESIGN CODE *													
AISC-1989 *										=====		===	
*****													
<---LENGTH (FT)= 4.17 --->													
*****												AX = 5.82	
												AY = 1.50	
												AZ = 4.51	
												SY = 4.47	
												SZ = 12.89	
												RY = 1.51	
												RZ = 2.58	
PARAMETER										0.0 (KIP-FEET)			
IN KIP INCH										L8		STRESSES	
										L8 L8		IN KIP INCH	
KL/R-Y= 33.12										L8			
KL/R-Z= 19.40										L8			
UNL = 50.00										L8			
CB = 1.00										L8			
CMY = 0.85										L8 L8			
CMZ = 0.85										L8			
FYLD = 30.00										L0			
NSF = 1.00										+-----+			
DFF = 0.00										0.0			
dff= 0.00										ABSOLUTE MZ ENVELOPE			
										(WITH LOAD NO.)			
										MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
										-----			
										AXIAL		SHEAR-Y	
										SHEAR-Z		MOMENT-Y	
										MOMENT-Z			
VALUE										0.0		0.0	
LOCATION										0.0		0.0	
LOADING										0		0	
*****													
* * * * *													
DESIGN SUMMARY (KIP-FEET)													
-----													
RESULT/										CRITICAL COND/		LOADING/	
FX										MY		MZ	
=====										=====		=====	
PASS										AISC- H1-3		6	
3.56 C										0.00		0.00	
* * * * *													



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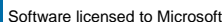
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*****		Y	PROPERTIES	
*****		==	IN INCH UNIT	
MEMBER	36			AX = 5.82
			--Z	AY = 1.50
DESIGN CODE				AZ = 4.51
AISC-1989		==		SY = 4.47
				SZ = 12.89
				RY = 1.51
				RZ = 2.58
*****				
<---LENGTH (FT)= 2.50 --->				

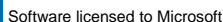
PARAMETER		0.1 (KIP-FEET)			L8		STRESSES	
IN KIP	INCH						IN KIP	INCH
KL/R-Y=	19.87						FA	= 17.26
KL/R-Z=	11.64						fa	= 2.78
UNL	= 30.00						FCZ	= 19.80
CB	= 1.00						FTZ	= 19.80
CMY	= 0.85						FCY	= 22.50
CMZ	= 0.85						FTY	= 22.50
FYLD	= 30.00						fbz	= 0.07
NSF	= 1.00						fby	= 0.00
DFF	= 0.00						Fey	= 378.14
dff=	0.00						Fez	=1102.61
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)							FV	= 12.00
							fv	= 0.01

## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	AISC- H1-1	1.641E-01	6
16.20 C	0.00	-0.07	2.50



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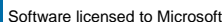
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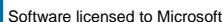
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*****		Y	PROPERTIES
MEMBER 47		==	IN INCH UNIT
UP 15BI38.5		--Z	AX = 11.26
DESIGN CODE			AY = 4.35
AISC-1989			AZ = 7.22
			SY = 8.13
			SZ = 59.06
			RY = 1.56
<---LENGTH (FT)= 5.50 --->			RZ = 6.27
*****			

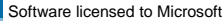
PARAMETER		65.8 (KIP-FEET)			L6		STRESSES	
IN KIP	INCH						IN KIP	INCH
KL/R-Y=	42.28				L6		FA	= 16.08
KL/R-Z=	10.52						fa	= 0.00
UNL	= 12.00				L6		FCZ	= 19.80
CB	= 1.00						FTZ	= 19.80
CMY	= 0.85						FCY	= 22.50
CMZ	= 0.85						FTY	= 22.50
FYLD	= 30.00						fbz	= 13.37
NSF	= 1.00						fby	= 0.00
DFF	= 0.00						Fey	= 83.53
dff=	0.00						Fez	=1349.16
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)							FV	= 12.00
							fv	= 0.58

## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	AISC- H1-3	6.755E-01	6
0.01 C	0.00	-65.82	5.50



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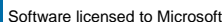
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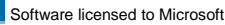
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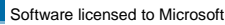
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*****										Y		PROPERTIES					
												IN INCH UNIT					
MEMBER 50 *										==		--Z					
AISC SECTIONS												AX = 2.38					
LD L25254												AY = 0.61					
DESIGN CODE *												AZ = 1.43					
AISC-1989 *												SY = 1.32					
*												SZ = 0.79					
*												RY = 1.24					
<---LENGTH (FT)= 5.75 --->												RZ = 0.77					
*****																	
0.0 (KIP-Feet)																	
PARAMETER												STRESSES					
IN KIP INCH												IN KIP INCH					
-----												-----					
KL/R-Y= 65.12												FA = 12.62					
KL/R-Z= 89.69												fa = 1.45					
UNL = 69.00												FCZ = 18.00					
CB = 1.00												FTZ = 18.00					
CMY = 0.85												FCY = 18.00					
CMZ = 0.85												FTY = 18.00					
FYLD = 30.00												fbz = 0.00					
NSF = 1.00												fby = 0.00					
DFF = 0.00												Fey = 35.22					
dff= 0.00												Fez = 18.56					
												Fv = 12.00					
												fv = 0.00					
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)																	
MAX FORCE/ MOMENT SUMMARY (KIP-Feet)																	
-----																	
AXIAL										SHEAR-Y		SHEAR-Z		MOMENT-Y		MOMENT-Z	
VALUE										0.0		0.0		0.0		0.0	
LOCATION										0.0		0.0		0.0		0.0	
LOADING										0		0		0		0	
*****																	
*																	
*																	
*																	
*																	
DESIGN SUMMARY (KIP-Feet)																	
-----																	
RESULT/										CRITICAL COND/		RATIO/		LOADING/			
FX										MY		MZ		LOCATION			
=====										=====		=====		=====			
PASS										AISC- H1-3		1.148E-01		6			
3.44 C										0.00		0.00		0.00			
*****																	
*																	



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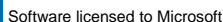
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## Steel Design (Track 2) Beam 53 Check 1

*****					Y	PROPERTIES	
MEMBER 53					---	IN	UNIT
DESIGN CODE					---	---	---
AISC-1989					---	---	---
<---LENGTH (FT)= 6.00 --->					---	---	---
*****					---	---	---
16.1 (KIP-FEET)					---	---	---
PARAMETER					---	---	---
IN KIP INCH					---	---	---
KL/R-Y= 84.66					---	---	---
KL/R-Z= 22.47					---	---	---
UNL = 12.00					---	---	---
CB = 1.00					---	---	---
CMY = 0.85					---	---	---
CMZ = 0.85					---	---	---
FYLD = 30.00					---	---	---
NSF = 1.00					---	---	---
DFE = 0.00					---	---	---
dff= 0.00					---	---	---
ABSOLUTE MZ ENVELOPE					---	---	---
(WITH LOAD NO.)					---	---	---
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					---	---	---
AXIAL					---	---	---
SHEAR-Y					---	---	---
SHEAR-Z					---	---	---
MOMENT-Y					---	---	---
MOMENT-Z					---	---	---
VALUE					---	---	---
LOCATION					---	---	---
LOADING					---	---	---
*****					---	---	---
DESIGN SUMMARY (KIP-FEET)					---	---	---
RESULT/					---	---	---
FX					---	---	---
CRITICAL COND/					---	---	---
MY					---	---	---
RATIO/					---	---	---
MZ					---	---	---
LOADING/					---	---	---
LOCATION					---	---	---
FAIL					---	---	---
0.01 C					---	---	---
AISC- H1-3					---	---	---
1.011E+00					---	---	---
6					---	---	---
-16.07					---	---	---
0.00					---	---	---
*****					---	---	---





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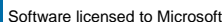
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*****										Y		PROPERTIES	
										==		IN INCH UNIT	
MEMBER 57										==		==	
UP 15BI38.5												AX = 11.26	
DESIGN CODE												AY = 4.35	
AISC-1989										==		AZ = 7.22	
												SY = 8.13	
												SZ = 59.06	
<---LENGTH (FT)= 6.50 --->												RY = 1.56	
*****												RZ = 6.27	
65.5 (KIP-FEET)													
PARAMETER												STRESSES	
IN KIP INCH												IN KIP INCH	
KL/R-Y= 49.97										L6		FA = 18.00	
KL/R-Z= 12.43										L6		fa = 0.00	
UNL = 12.00										L6		FCZ = 19.80	
CB = 1.00										L6		FTZ = 19.80	
CMY = 0.85										L6		FCY = 22.50	
CMZ = 0.85										L6		FTY = 22.50	
FYLD = 30.00										L6		fbz = 13.31	
NSF = 1.00										L6		fby = 0.00	
DFF = 0.00										L6		Fey = 59.80	
dff= 0.00										L6		Fcz = 965.97	
ABSOLUTE MZ ENVELOPE												FV = 12.00	
(WITH LOAD NO.)												fv = 0.21	
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)													
-----													
AXIAL										SHEAR-Y		SHEAR-Z	
MOMENT-Y										MOMENT-Z			
VALUE										0.0		0.0	
LOCATION										0.0		0.0	
LOADING										0		0	
*****													
* DESIGN SUMMARY (KIP-FEET)													
* -----													
* RESULT/										CRITICAL COND/		RATIO/	
* FX										MY		MZ	
* PASS										AISC- H2-1		6.724E-01	
* 0.02 T										0.00		-65.52	
* 6.50													
*****													



### Steel Design (Track 2) Beam 59 Check 1

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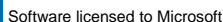
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## Steel Design (Track 2) Beam 62 Check 1

*****				Y	PROPERTIES	
				==	IN INCH UNIT	
MEMBER	62	*	AISC SECTIONS		--Z	AX = 3.38
		*	LD L40304			AY = 0.97
DESIGN CODE	*					AZ = 1.58
AISC-1989	*					SY = 1.84
	*					SZ = 2.00
	*	<---LENGTH (FT)= 6.52 --->				RY = 1.33
*****						RZ = 1.28
		2.1 (KIP-FEET)				
PARAMETER				L6	STRESSES	
IN KIP	INCH		L6	L6	IN KIP	INCH
KL/R-Y=	84.54	+		L6	FA	12.55
KL/R-Z=	61.11	+	L6		fa	0.05
UNL	= 78.27				FCZ	17.07
CB	= 1.00	+L6	L6		FTZ	18.00
CMY	= 0.85			L6	FCY	17.07
CMZ	= 0.85	+			FTY	17.07
FYLD	= 30.00		L5	L3	fbz	12.54
NSF	= 1.00	++-----+			fby	0.00
DFF	= 0.00	0.1			Fey	20.90
dff=	0.00		ABSOLUTE MZ ENVELOPE			Fez = 39.99
			(WITH LOAD NO.)			FV = 12.00
					fv	= 2.26
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)						
-----						
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z	
VALUE	0.0	0.0	0.0	0.0	0.0	
LOCATION	0.0	0.0	0.0	0.0	0.0	
LOADING	0	0	0	0	0	
*****						
* * * * *						
DESIGN SUMMARY (KIP-FEET)						
-----						
RESULT/	CRITICAL COND/	RATIO/	LOADING/			
FX	MY	MZ	LOCATION			
=====						
PASS	AISC- H1-3	7.389E-01	6			
0.17 C	0.00	2.09	6.52			
*****						
* * * * *						



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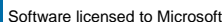
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## Steel Design (Track 2) Beam 64 Check 1

*****				Y	PROPERTIES	
				==	IN INCH UNIT	
MEMBER	64	*	=====		--Z	AX = 3.38
		*	AISC SECTIONS			AY = 0.97
DESIGN CODE	*					AZ = 1.58
AISC-1989	*	-----				SY = 1.84
	*					SZ = 4.48
	*	<---LENGTH (FT)= 5.52 --->				RY = 1.33
*****						RZ = 1.28
				1.7 (KIP-FEET)		
PARAMETER			L6 L6		STRESSES	
IN KIP INCH			L6		IN KIP INCH	
				+L6		
KL/R-Y=	81.42			L6	FA =	18.00
KL/R-Z=	51.71				fa =	0.00
UNL =	66.23		L6		FCZ =	17.07
CB =	1.00				FTZ =	18.00
CMY =	0.85			L6	FCY =	17.07
CMZ =	0.85		L6		FTY =	17.07
FYLD =	30.00				fbz =	10.09
NSF =	1.00				fby =	0.00
DFP =	0.00	-0.1			Fey =	22.52
dfp=	0.00				Fez =	55.85
				ABSOLUTE MZ ENVELOPE		
				(WITH LOAD NO.)		
				FV = 12.00		
				fv = 0.24		
				MAX FORCE/ MOMENT SUMMARY (KIP-FEET)		
				-----		
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z	
VALUE	0.0	0.0	0.0	0.0	0.0	
LOCATION	0.0	0.0	0.0	0.0	0.0	
LOADING	0	0	0	0	0	
*****						
* DESIGN SUMMARY (KIP-FEET) *						
*****						
RESULT/	CRITICAL COND/	RATIO/	LOADING/			
FX	MY	MZ	LOCATION			
=====						
PASS	AISC- H2-1	5.605E-01	6			
0.00 T	0.00	-1.68	2.76			
*****						
*****						





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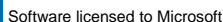
File 14-124 Typical Bent Fram

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						Y	PROPERTIES			
							IN INCH UNIT			
*****						== ==	-----			
MEMBER	65	*	=====				AX =	3.38		
		*	AISC SECTIONS				AY =	0.97		
DESIGN CODE	*	*	LD L40304			--Z	AZ =	1.58		
AISC-1989	*	*	-----				SY =	1.84		
	*	*					SZ =	2.00		
	*	*	<---LENGTH (FT)= 5.52 --->				RX =	1.33		
*****							RZ =	1.28		
						1.3 (KIP-FEET)				
PARAMETER			L6	L6		L6	STRESSES			
IN KIP	INCH				L6		IN KIP	INCH		
-----		+					-----			
KL/R-Y=	81.42						FA =	12.79		
KL/R-Z=	51.71	+L6		L6			fa =	0.05		
UNL =	66.23						FCZ =	17.07		
CB =	1.00	+			L6	L6	FTZ =	18.00		
CMY =	0.85			L6			FCY =	17.07		
CMZ =	0.85	+					FTY =	17.07		
FYLD =	30.00		L3			L5	fzb =	7.62		
NSF =	1.00	+--+--+--+--+--+--+--+--+---					fby =	0.00		
DFF =	0.00	0.1					Pey =	22.52		
dff=	0.00		ABSOLUTE MZ ENVELOPE				Fez =	55.85		
						(WITH LOAD NO.)	FV =	12.00		
							fv =	1.83		
						MAX FORCE/ MOMENT SUMMARY (KIP-FOOT)				
						-----				
						AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
						VALUE	0.0	0.0	0.0	0.0
						LOCATION	0.0	0.0	0.0	0.0
						LOADING	0	0	0	0
*****										
*										*
*	DESIGN SUMMARY (KIP-FOOT)									*
*	-----									*
*										*
*										*
RESULT/	CRITICAL COND/		RATIO/		LOADING/					*
FX	MY		MZ		LOCATION					*
=====										*
PASS	AISC- H1-3		4.500E-01		6					*
0.17 C	0.00		1.27		5.52					*
*****										*
										*

### Steel Design (Track 2) Beam 66 Check 1

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## Statics Check Results

L/C		FX (kip)	FY (kip)	FZ (kip)	MX (kip'in)	MY (kip'in)	MZ (kip'in)
1:DEAD LOAD +	Loads	-0.000	-210.050	0.000	0.000	0.000	-75.7E+3
1:DEAD LOAD +	Reactions	0.000	210.050	0.000	0.000	0.000	75.7E+3
	Difference	0.000	0.000	0.000	0.000	0.000	0.000
2:LIVE LOAD	Loads	0.000	-87.000	0.000	0.000	0.000	-31.3E+3
2:LIVE LOAD	Reactions	0.000	87.000	0.000	0.000	0.000	31.3E+3
	Difference	0.000	0.000	0.000	0.000	0.000	0.000
3:SNOW LOAD	Loads	0.000	-21.073	0.000	0.000	0.000	-7.59E+3
3:SNOW LOAD	Reactions	-0.000	21.073	0.000	0.000	0.000	7.59E+3
	Difference	0.000	0.000	0.000	0.000	0.000	0.000

## Reaction Summary

	Node	L/C	Horizontal	Vertical	Horizontal	Moment		
			FX (kip)	FY (kip)	FZ (kip)	MX (kip'in)	MY (kip'in)	MZ (kip'in)
Max FX	1	8:GENERATEC	<b>0.002</b>	56.416	0.000	0.000	0.000	0.000
Min FX	7	8:GENERATEC	<b>-0.001</b>	56.602	0.000	0.000	0.000	0.000
Max FY	3	5:GENERATEC	-0.001	<b>91.574</b>	0.000	0.000	0.000	0.000
Min FY	1	3:SNOW LOAC	0.000	<b>4.215</b>	0.000	0.000	0.000	0.000
Max FZ	1	1:DEAD LOAD	0.001	42.365	<b>0.000</b>	0.000	0.000	0.000
Min FZ	1	1:DEAD LOAD	0.001	42.365	<b>0.000</b>	0.000	0.000	0.000
Max MX	1	1:DEAD LOAD	0.001	42.365	0.000	<b>0.000</b>	0.000	0.000
Min MX	1	1:DEAD LOAD	0.001	42.365	0.000	<b>0.000</b>	0.000	0.000
Max MY	1	1:DEAD LOAD	0.001	42.365	0.000	0.000	<b>0.000</b>	0.000
Min MY	1	1:DEAD LOAD	0.001	42.365	0.000	0.000	<b>0.000</b>	0.000
Max MZ	1	1:DEAD LOAD	0.001	42.365	0.000	0.000	0.000	<b>0.000</b>
Min MZ	1	1:DEAD LOAD	0.001	42.365	0.000	0.000	0.000	<b>0.000</b>





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File 14-124 Auditorium Truss.s

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## Job Information

	Engineer	Checked	Approved
Name:			
Date:	31-Oct-14		

Structure Type	TRUSS
----------------	-------

Number of Nodes	37	Highest Node	500
Number of Elements	70	Highest Beam	805

Number of Basic Load Cases	2
Number of Combination Load Cases	4

Included in this printout are data for:

All	The Whole Structure
-----	---------------------

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	LOAD CASE 1
Primary	2	LOAD CASE 2
Combination	3	GENERATED AISC GENERAL 1
Combination	4	GENERATED AISC GENERAL 2
Combination	5	GENERATED AISC GENERAL 3
Combination	6	GENERATED AISC GENERAL 4



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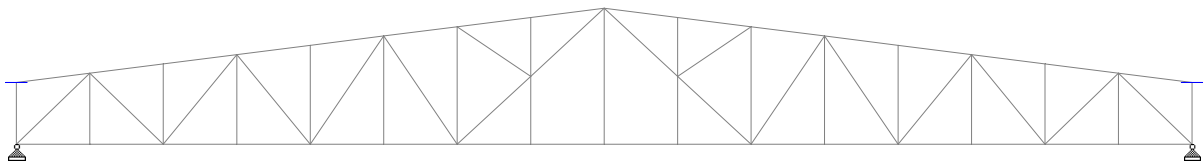
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Y  
Z-x

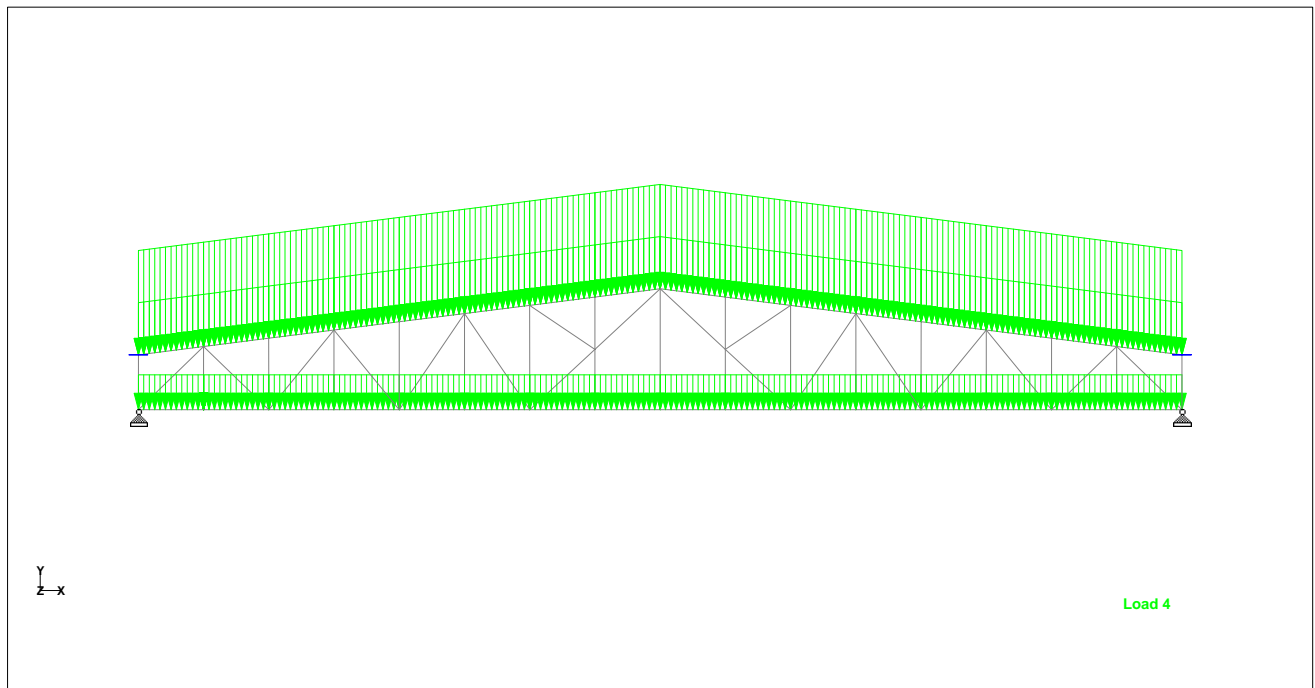
Load 1

Whole Structure

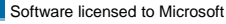


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Truss Loads



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### Load 4

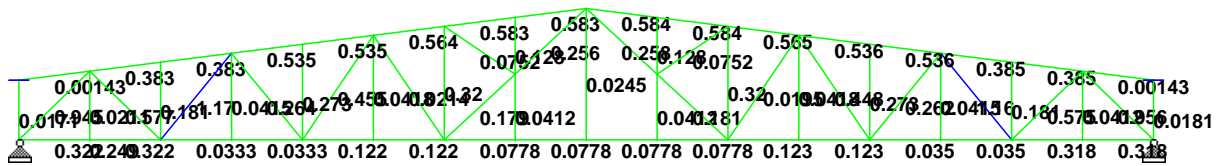
### Truss Members



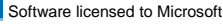


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Truss Stress Results



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							Y	PROPERTIES IN INCH UNIT	
*****									
MEMBER 28		*	=====				==	==	AX = 5.82
		*	UPT						AY = 1.50
DESIGN CODE *		*	UP 6H20				--Z		AZ = 4.51
AISC-1989 *		*	=====				==	==	SY = 4.47
		*							SZ = 12.89
*		*	<---LENGTH (FT)= 5.00 --->						RZ = 1.51
*****									RZ = 2.58
			0.0 (KIP-FEET)						
PARAMETER			L0				L0	STRESSES	
IN KIP INCH			L0				L0	IN KIP INCH	
-----			+L0				L0	-----	
KL/R-Y=	39.74		L0				L0	FA = 16.23	
KL/R-Z=	23.28		+L0				L0	fa = 0.28	
UNL =	60.00		L0				L0	FCZ = 0.00	
CB =	0.00		+L0				L0	FTZ = 0.00	
CMY =	0.00		L0				L0	FCY = 0.00	
CMZ =	0.00		+L0				L0	FTY = 0.00	
FYLD =	30.00		L0				L0	fbz = 0.00	
NSF =	1.00		+--+--+--+--+--+--+--+--+--+					fby = 0.00	
DFF =	0.00		0.0					Fey = 94.53	
dff=	0.00		ABSOLUTE MZ ENVELOPE					Fez = 275.65	
			(WITH LOAD NO.)					FV = 0.00	
								fv = 0.00	
			MAX FORCE/ MOMENT SUMMARY (KIP-FEET)						
			-----						
	AXIAL		SHEAR-Y		SHEAR-Z		MOMENT-Y	MOMENT-Z	
VALUE	0.0		0.0		0.0		0.0	0.0	
LOCATION	0.0		0.0		0.0		0.0	0.0	
LOADING	0		0		0		0	0	
*****									
		*						*	
		*						*	
		*	DESIGN SUMMARY (KIP-FEET)					*	
		*	-----					*	
		*						*	
RESULT/ FX			CRITICAL COND/ MY		RATIO/ MZ		LOADING/ LOCATION		
=====			=====			=====			
PASS			AISC- H1-3		1.709E-02		4		
1.61 C			0.00		0.00		0.00		
		*						*	
*****									



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```

*****
MEMBER 53 *
DESIGN CODE *
AISC-1989 *
*
* |<---LENGTH (FT)= 5.00 --->|
*****

0.0 (KIP-FEET)
PARAMETER |L0
IN KIP INCH |L0
-----+L0
KL/R-Y= 39.74 |L0
KL/R-Z= 23.28 +L0
UNL = 60.00 |L0
CB = 0.00 +L0
CMY = 0.00 |L0
CMZ = 0.00 +L0
FYLD = 30.00 |L0
NSF = 1.00 +-----+-----+-----+-----+-----+-----+
DFF = 0.00 0.0
dff= 0.00

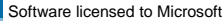
ABSOLUTE MZ ENVELOPE
(WITH LOAD NO.)

MAX FORCE/ MOMENT SUMMARY (KIP-FEET)
-----
AXIAL SHEAR-Y SHEAR-Z MOMENT-Y MOMENT-Z

VALUE 0.0 0.0 0.0 0.0 0.0
LOCATION 0.0 0.0 0.0 0.0 0.0
LOADING 0 0 0 0 0

*****
*
* DESIGN SUMMARY (KIP-FEET)
* -----
*
* RESULT/ CRITICAL COND/ RATIO/ LOADING/
* FX MY MZ LOCATION
* =====
* PASS AISC- H1-3 1.814E-02 4
* 1.71 C 0.00 0.00 0.00
*
*****

```



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*****							
						Y   ==	PROPERTIES  
						 =	IN INCH UNIT -----
MEMBER 738	*	=====					AX = 7.22
	*	AISC SECTIONS					AZ = 1.57
DESIGN CODE	*	SD L60406				--Z	AY = 3.89
AISC-1989	*	-----					AZ = 3.89
	*						SY = 9.85
	*						SZ = 3.21
	*	<---LENGTH (FT)= 5.93 --->					RY = 2.92
*****							RZ = 1.17
 0.0 (KIP-FEET)							
PARAMETER	L0					L0 STRESSES	
IN KIP INCH	L0					L0 IN KIP INCH	
+L0 -----						L0 -----	
KL/R-Y= 75.91	+L0					L0 FA = 13.19	
KL/R-Z= 61.08	+L0					L0 fa = 4.25	
UNL = 71.21	+L0					L0 FCZ = 0.00	
CB = 0.00	+L0					L0 FTZ = 0.00	
CMY = 0.00	+L0					L0 FCY = 0.00	
CMZ = 0.00	+L0					L0 FTY = 0.00	
FYLD = 30.00	+L0					L0 fbz = 0.00	
NSF = 1.00	+--+--+--+--+--+--+--+--+--+--+--+---					fby = 0.00	
DFF = 0.00	0.0					Fey = 25.92	
dff= 0.00	ABSOLUTE MZ ENVELOPE					Fez = 40.02	
	(WITH LOAD NO.)					FV = 0.00	
						fv = 0.00	
 MAX FORCE/ MOMENT SUMMARY (KIP-FEET) -----							
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z		
VALUE	0.0	0.0	0.0	0.0	0.0		
LOCATION	0.0	0.0	0.0	0.0	0.0		
LOADING	0	0	0	0	0		
*****							
*							
*							
DESIGN SUMMARY (KIP-FEET)							
-----							
* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ		LOADING/ LOCATION			
=====							
PASS	AISC- H1-1	3.221E-01		4			
30.66 C	0.00	0.00		0.00			
*****							





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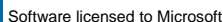
							Y	PROPERTIES		
*****								IN	INCH	UNIT
MEMBER 739		*	=====				==		==	
		*	AISC SECTIONS							
		*	SD L60406					--Z		
DESIGN CODE		*								
AISC-1989		*	-----							
		*								
		*	<---LENGTH (FT)= 5.94 --->							
*****										
			0.0 (KIP-FEET)							
PARAMETER			L0				L0	STRESSES		
IN KIP INCH			L0				L0	IN KIP INCH		
-----			+L0				L0	-----		
KL/R-Y= 75.91			L0				L0	FA = 18.00		
KL/R-Z= 61.12			+L0				L0	fa = 1.40		
UNL = 71.25			L0				L0	FCZ = 0.00		
CB = 0.00			+L0				L0	FTZ = 0.00		
CMY = 0.00			L0				L0	FCY = 0.00		
CMZ = 0.00			+L0				L0	FTY = 0.00		
FYLD = 30.00			L0				L0	fbz = 0.00		
NSF = 1.00			+--+--+--+--+--+--+--+--+---					fby = 0.00		
DFF = 0.00			0.0					Fey = 25.92		
dff= 0.00			ABSOLUTE MZ ENVELOPE					Fez = 39.98		
			(WITH LOAD NO.)					FV = 0.00		
								fv = 0.00		
			MAX FORCE/ MOMENT SUMMARY (KIP-FOOT)							
			-----							
			AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z			
VALUE			0.0	0.0	0.0	0.0	0.0			
LOCATION			0.0	0.0	0.0	0.0	0.0			
LOADING			0	0	0	0	0			
*****										
*										
*										
DESIGN SUMMARY (KIP-FOOT)										
-----										
RESULT/		CRITICAL COND/		RATIO/		LOADING/				
FX		MY		MZ		LOCATION				
=====										
PASS		TENSION		7.779E-02		4				
10.11 T		0.00		0.00		0.00				
*										
*****										

### Steel Design (Track 2) Beam 740 Check 1

*****										Y	PROPERTIES	
												IN INCH UNIT
MEMBER 740										==	==	
AISC SECTIONS												AX = 2.13
SD L25204											--Z	AY = 0.50
DESIGN CODE												AZ = 1.04
AISC-1989												SY = 1.05
												SZ = 0.51
												RY = 1.11
												RZ = 0.59
*****												
0.0 (KIP-FEET)												
PARAMETER		L0						L0		STRESSES		
IN KIP	INCH	L0						L0		IN KIP	INCH	
-----		+L0						L0		-----		
KL/R-Y=	119.88	L0						L0		FA =	18.00	
KL/R-Z=	223.06	+L0						L0		fa =	0.44	
UNL =	132.00	L0						L0		FCZ =	0.00	
CB =	0.00	+L0						L0		FTZ =	0.00	
CMY =	0.00	L0						L0		FCY =	0.00	
CMZ =	0.00	+L0						L0		FTY =	0.00	
FYLD =	30.00	L0						L0		fbz =	0.00	
NSF =	1.00	+++++-----								fby =	0.00	
DFF =	0.00	0.0								Fey =	10.39	
dff=	0.00									Fez =	3.00	
												FV = 0.00
												fv = 0.00

### Steel Design (Track 2) Beam 741 Check 1

*****										Y	PROPERTIES	
												IN INCH UNIT
MEMBER 741										==	==	AX = 10.13
AISC SECTIONS												AY = 2.65
SD L60607											--Z	AZ = 4.68
DESIGN CODE												SY = 11.59
AISC-1989												SZ = 8.16
												RY = 2.67
												RZ = 1.87
*****												



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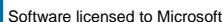
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### Steel Design (Track 2) Beam 743 Check 1

*****					
		Y		PROPERTIES	
				IN INCH UNIT	
		==  ==		-----	
MEMBER 743		AISC SECTIONS		AX = 7.22	
		SD L60406		AY = 1.57	
DESIGN CODE				AZ = 3.89	
AISC-1989				SY = 9.85	
				SZ = 3.21	
				RY = 2.92	
				RZ = 1.17	
*****					
		<---LENGTH (FT)= 5.94 --->			
*****					
		0.0 (KIP-FEET)			
PARAMETER		L0		L0	STRESSES
IN KIP INCH		L0		L0	IN KIP INCH
-----		+L0		L0	-----
KL/R-Y= 75.91		L0		L0	FA = 13.19
KL/R-Z= 61.12		+L0		L0	fa = 4.25
UNL = 71.25		L0		L0	FCZ = 0.00
CB = 0.00		+L0		L0	FTZ = 0.00
CMY = 0.00		L0		L0	FCY = 0.00
CMZ = 0.00		+L0		L0	FTY = 0.00
FYLD = 30.00		L0		L0	fby = 0.00
NSF = 1.00		+--+--+--+--+--+--+--+--+--+--+--+---			fby = 0.00
DFF = 0.00		0.0			Fey = 25.92
dff= 0.00					Fez = 39.98
		ABSOLUTE MZ ENVELOPE			FV = 0.00
		(WITH LOAD NO.)			fv = 0.00
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
		-----			
		AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y
					MOMENT-Z
VALUE		0.0	0.0	0.0	0.0
LOCATION		0.0	0.0	0.0	0.0
LOADING		0	0	0	0
*****					
*					
*		DESIGN SUMMARY (KIP-FEET)			
*		-----			
*					
*					
RESULT/		CRITICAL COND/	RATIO/	LOADING/	
FX		MY	MZ	LOCATION	
=====				=====	
PASS		AISC- H1-1	3.221E-01	4	
30.66 C		0.00	0.00	0.00	
*****					



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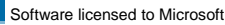
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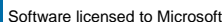
Chd

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							Y	PROPERTIES IN INCH UNIT	
*****									
MEMBER 746						AISC SECTIONS	==	==	AX = 7.22
						SD L60406		--Z	AY = 1.57
DESIGN CODE									AZ = 3.89
AISC-1989									SY = 9.85
									SZ = 3.21
*						<---LENGTH (FT)=	5.94	---	RX = 2.92
*****									RZ = 1.17
						0.0 (KIP-FEET)			
PARAMETER		L0				L0	STRESSES		
IN KIP	INCH	L0				L0	IN KIP	INCH	
-----		+L0				L0	-----	-----	
KL/R-Y=	75.91	L0				L0	FA =	18.00	
KL/R-Z=	61.12	+L0				L0	fa =	2.20	
UNL =	71.25	L0				L0	FCZ =	0.00	
CB =	0.00	+L0				L0	FTZ =	0.00	
CMY =	0.00	L0				L0	FCY =	0.00	
CMZ =	0.00	+L0				L0	FTY =	0.00	
FYLD =	30.00	L0				L0	fbz =	0.00	
NSF =	1.00	+--+--+--+--+--+--+--+--+--+--+				fbz =	0.00		
DFF =	0.00	0.0					Fey =	25.92	
dff=	0.00	ABSOLUTE MZ ENVELOPE					Fez =	39.98	
						(WITH LOAD NO.)		FV = 0.00	
								fz = 0.00	
						MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
						-----			
		AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z			
VALUE	0.0	0.0	0.0	0.0	0.0	0.0			
LOCATION	0.0	0.0	0.0	0.0	0.0	0.0			
LOADING	0	0	0	0	0	0			
*****						*****			
*						*			
*						*			
*						*			
*						*			
*						*			
DESIGN SUMMARY (KIP-FEET)									
-----									
RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ		LOADING/ LOCATION					
=====									
PASS	TENSION	1.221E-01		4					
15.87 T	0.00	0.00		0.00					
*****						*****			





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*****		Y	PROPERTIES	
MEMBER 747		==	==	IN INCH UNIT
AISC SECTIONS				AX = 7.22
SD L60406			--Z	AY = 1.57
DESIGN CODE				AZ = 3.89
AISC-1989				SY = 9.85
				SZ = 3.21
				RY = 2.92
<---LENGTH (FT)= 5.94 --->				RZ = 1.17
*****				

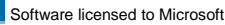
[illegible]

## MAX FORCE/ MOMENT SUMMARY (KIP-FEET)

	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0

## DESIGN SUMMARY (KIP-FEET)

* RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
PASS	TENSION	1.221E-01	4
15.87 T	0.00	0.00	0.00



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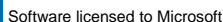
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*****					
		=====		Y == ==	PROPERTIES IN INCH UNIT
MEMBER 748	*	AISC SECTIONS			AX = 7.22
	*	SD L60406		--Z	AY = 1.57
DESIGN CODE	*				AZ = 3.89
AISC-1989	*	-----			SY = 9.85
	*				SZ = 3.21
	*	<---LENGTH (FT)= 5.94 --->			RY = 2.92
*****					RZ = 1.17
0.0 (KIP-FEET)					
PARAMETER	L0			L0	STRESSES
IN KIP INCH	L0			L0	IN KIP INCH
-----	+L0			L0	-----
KL/R-Y= 75.91	L0			L0	FA = 18.00
KL/R-Z= 61.12	+L0			L0	fa = 1.40
UNL = 71.25	L0			L0	FCZ = 0.00
CB = 0.00	+L0			L0	FTZ = 0.00
CMY = 0.00	L0			L0	FCY = 0.00
CMZ = 0.00	+L0			L0	FTY = 0.00
FYLD = 30.00	L0			L0	fzbz = 0.00
NSF = 1.00	+-----+				fby = 0.00
DFF = 0.00	0.0				Fey = 25.92
dff= 0.00					Fez = 39.98
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)					FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET) -----					
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
* DESIGN SUMMARY (KIP-FEET)					
* -----					
*					
* RESULT/ CRITICAL COND/ RATIO/ LOADING/ FX MY MZ LOCATION					
=====					
PASS TENSION 7.779E-02 4					
10.11 T 0.00 0.00 0.00					
*					
*****					



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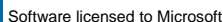
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### Steel Design (Track 2) Beam 752 Check 1

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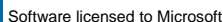
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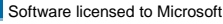
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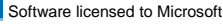
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*****						Y	PROPERTIES	
							IN INCH UNIT	
MEMBER 756						==  ==		
AISC SECTIONS							AX = 7.22	
SD L60406							AY = 1.57	
DESIGN CODE							AZ = 3.89	
AISC-1989							SY = 9.85	
							SZ = 3.21	
<---LENGTH (FT)= 0.00 --->							RY = 2.92	
*****							RZ = 1.17	
0.0 (KIP-FEET)								
PARAMETER		L0		L0		STRESSES		
IN KIP	INCH	L0		L0		IN KIP	INCH	
		+L0		L0		-----		
KL/R-Y=	0.03	L0		L0	FA =	17.07		
KL/R-Z=	0.09	+L0		L0	fa =	4.25		
UNL =	0.10	L0		L0	FCZ =	0.00		
CB =	0.00	+L0		L0	FTZ =	0.00		
CMY =	0.00	L0		L0	FCY =	0.00		
CMZ =	0.00	+L0		L0	FTY =	0.00		
FYLD =	30.00	L0		L0	fbz =	0.00		
NSF =	1.00	+		fby =		0.00		
DFF =	0.00	0.0		Fey=0.1274E+9				
dff=	0.00			Fez=0.2029E+8				
ABSOLUTE MZ ENVELOPE						FV = 0.00		
(WITH LOAD NO.)						fv = 0.00		
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)								
-----								
		AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z		
VALUE	0.0	0.0	0.0	0.0	0.0	0.0		
LOCATION	0.0	0.0	0.0	0.0	0.0	0.0		
LOADING	0	0	0	0	0	0		
*****								
*							*	
*	DESIGN SUMMARY (KIP-FEET)						*	
*	-----						*	
*							*	
*	RESULT/	CRITICAL COND/	RATIO/	LOADING/		*		
	FX	MY	MZ	LOCATION				
	=====	=====	=====	=====				
	PASS	AISC- H1-1	2.489E-01	4				
	30.66 C	0.00	0.00	0.00				
*							*	
*****								



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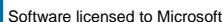
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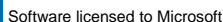
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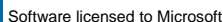
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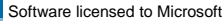
Chd

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*****					
		Y		PROPERTIES	
		== ==		IN INCH UNIT	
MEMBER 763		AISC SECTIONS		AX = 10.13	
		SD L60607		AY = 2.65	
DESIGN CODE				AZ = 4.68	
AISC-1989				SY = 11.59	
				SZ = 8.16	
		<---LENGTH (FT)= 5.98 --->		RY = 2.67	
*****				RZ = 1.87	
PARAMETER		0.0 (KIP-FEET)			
IN KIP INCH		L0		L0 STRESSES	
-----		+L0		L0 IN KIP INCH	
KL/R-Y= 72.40		L0		L0 -----	
KL/R-Z= 38.41		+L0		L0 FA = 14.02	
UNL = 71.82		L0		L0 fa = 5.37	
CB = 0.00		+L0		L0 FCZ = 0.00	
CMY = 0.00		L0		L0 FTZ = 0.00	
CMZ = 0.00		+L0		L0 FCY = 0.00	
FYLD = 30.00		L0		L0 FTY = 0.00	
NSF = 1.00		+++++		L0 fbz = 0.00	
DFF = 0.00		0.0		fbz = 0.00	
dff= 0.00				Fey = 28.49	
		ABSOLUTE MZ ENVELOPE		Fez = 101.20	
		(WITH LOAD NO.)		FV = 0.00	
				fv = 0.00	
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/	CRITICAL COND/	RATIO/	LOADING/		
FX	MY	MZ	LOCATION		
=====	=====	=====	=====		
PASS	AISC- H1-1	3.832E-01	4		
54.44 C	0.00	0.00	0.00		
*****					
*					



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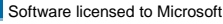
Chd

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*****					
		Y		PROPERTIES	
		==  ==		IN INCH UNIT	
MEMBER 764 *		=====		-----	
DESIGN CODE *		AISC SECTIONS		AX = 10.13	
AISC-1989 *		SD L60607		--Z AY = 2.65	
*		-----		AZ = 4.68	
*				SY = 11.59	
*				SZ = 8.16	
<---LENGTH (FT)= 5.99 --->				RY = 2.67	
*****				RZ = 1.87	
		0.0 (KIP-FEET)			
PARAMETER		L0		L0	STRESSES
IN KIP INCH		L0		L0	IN KIP INCH
-----		+L0		L0	-----
KL/R-Y= 72.40		L0		L0	FA = 14.02
KL/R-Z= 38.42		+L0		L0	fa = 5.37
UNL = 71.84		L0		L0	FCZ = 0.00
CB = 0.00		+L0		L0	FTZ = 0.00
CMY = 0.00		L0		L0	FCY = 0.00
CMZ = 0.00		+L0		L0	FTY = 0.00
FYLD = 30.00		L0		L0	fbz = 0.00
NSF = 1.00		+--+--+--+--+--+--+--+--+--+--+--+---			fby = 0.00
DFF = 0.00		0.0			Fey = 28.49
dff= 0.00					Fez = 101.15
		ABSOLUTE MZ ENVELOPE			FV = 0.00
		(WITH LOAD NO.)			fv = 0.00
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
		-----			
		AXIAL		SHEAR-Y	SHEAR-Z
		MOMENT-Y		MOMENT-Z	
VALUE		0.0		0.0	0.0
LOCATION		0.0		0.0	0.0
LOADING		0		0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
*					
RESULT/ CRITICAL COND/ RATIO/ LOADING/					
FX MY MZ LOCATION					
=====					
PASS AISC- H1-1 3.832E-01 4					
54.44 C 0.00 0.00 0.00					
*****					





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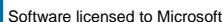
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*****		Y		PROPERTIES	
MEMBER 765		==  ==		IN INCH UNIT	
DESIGN CODE		--Z		AX = 2.63	
AISC-1989				AY = 0.63	
				AZ = 1.20	
				SY = 1.51	
				SZ = 0.81	
				RY = 1.31	
				RZ = 0.75	
*****					
0.0 (KIP-FEET)					
PARAMETER		L0		L0 STRESSES	
IN KIP INCH		L0		IN KIP INCH	
-----		+L0		-----	
KL/R-Y= 78.30		L0		L0 FA = 18.00	
KL/R-Z= 129.07		+L0		L0 fa = 3.23	
UNL = 97.12		L0		L0 FCZ = 0.00	
CB = 0.00		+L0		L0 FTZ = 0.00	
CMY = 0.00		L0		L0 FCY = 0.00	
CMZ = 0.00		+L0		L0 FTY = 0.00	
FYLD = 30.00		L0		L0 fbz = 0.00	
NSF = 1.00		+-----		L0 fby = 0.00	
DFF = 0.00		0.0		Fey = 24.36	
dff= 0.00				Fez = 8.96	
		ABSOLUTE MZ ENVELOPE		FV = 0.00	
		(WITH LOAD NO.)		fv = 0.00	
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
		-----			
		AXIAL		SHEAR-Y	
		SHEAR-Z		MOMENT-Y	
		MOMENT-Z			
VALUE		0.0		0.0	
LOCATION		0.0		0.0	
LOADING		0		0	
*****					
*					
*		DESIGN SUMMARY (KIP-FEET)			
*		-----			
*					
RESULT/		CRITICAL COND/		LOADING/	
FX		MY		LOCATION	
=====		=====		=====	
PASS		TENSION		4	
8.48 T		0.00		8.09	
*					
*****					



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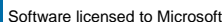
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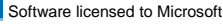
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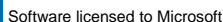
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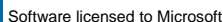
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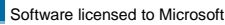
Date/Time 06-Nov-2014 14:28

									Y	PROPERTIES IN INCH UNIT			
*****													
MEMBER 773	*	=====						==		=			
	*	AISC SECTIONS								--Z	AX = 2.63		
	*	SD L30254									AY = 0.63		
DESIGN CODE	*	-----									AZ = 1.20		
AISC-1989	*	-----									SY = 1.51		
	*										SZ = 0.81		
	*	<---LENGTH (FT)= 8.09 --->									RX = 1.31		
*****											RZ = 0.75		
		0.0 (KIP-Feet)											
PARAMETER	L0					L0	STRESSES						
IN KIP INCH	L0					L0	IN KIP INCH						
-----		+L0	-----				L0	-----					
KL/R-Y= 78.30	L0					L0	FA =	18.00					
KL/R-Z= 129.07	+L0					L0	fa =	4.61					
UNL = 97.12	L0					L0	FCZ =	0.00					
CB = 0.00	+L0					L0	FTZ =	0.00					
CMY = 0.00	L0					L0	FCY =	0.00					
CMZ = 0.00	+L0					L0	FTY =	0.00					
FYLD = 30.00	L0					L0	fby =	0.00					
NSF = 1.00	+--+ +--+ +--+ +--+ +--+ +--+ +---						fey =	24.36					
DFF = 0.00	0.0						fez =	8.96					
dff= 0.00		ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)					FV =	0.00					
							fv =	0.00					
MAX FORCE/ MOMENT SUMMARY (KIP-FEET) -----													
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z								
VALUE	0.0	0.0	0.0	0.0	0.0								
LOCATION	0.0	0.0	0.0	0.0	0.0								
LOADING	0	0	0	0	0								
*****													
*													
*													
DESIGN SUMMARY (KIP-FEET)													
-----													
*													
*													
RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION										
=====													
PASS	TENSION	2.560E-01	4										
12.10 T	0.00	0.00	8.09										
*													
*****													

## Steel Design (Track 2) Beam 774 Check 1

*****					
			Y	PROPERTIES	
				IN INCH UNIT	
MEMBER 774			== ==		
AISC SECTIONS				AX = 2.38	
SD L25254			--Z	AY = 0.61	
DESIGN CODE				AZ = 1.04	
AISC-1989				SY = 1.05	
				SZ = 0.79	
				RY = 1.05	
<---LENGTH (FT)= 7.16 --->				RZ = 0.77	
*****					
0.0 (KIP-FEET)					
PARAMETER		L0	L0 STRESSES		
IN KIP	INCH	L0	L0 IN KIP INCH		
-----		+L0	L0 -----		
KL/R-Y=	85.33	L0	L0 FA = 18.00		
KL/R-Z=	111.68	+L0	L0 fa = 1.35		
UNL =	85.92	L0	L0 FCZ = 0.00		
CB =	0.00	+L0	L0 FTZ = 0.00		
CMY =	0.00	L0	L0 FCY = 0.00		
CMZ =	0.00	+L0	L0 FTY = 0.00		
FYLD =	30.00	L0	L0 fbz = 0.00		
NSF =	1.00	+--+--+--+--+--+--+--+--+--+	fbz = 0.00		
DFF =	0.00	0.0	Fey = 20.51		
dff=	0.00	ABSOLUTE MZ ENVELOPE			Fez = 11.97
			(WITH LOAD NO.)		
			FV = 0.00		
			fv = 0.00		
MAX FORCE/ MOMENT SUMMARY (KIP-FOOT)					
-----					
AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FOOT)					
-----					
*					
*					
RESULT/	CRITICAL COND/	RATIO/	LOADING/		
FX	MY	MZ	LOCATION		
=====					
PASS	TENSION	7.523E-02	4		
3.22 T	0.00	0.00	7.16		
*					
*****					





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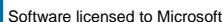
Date 31-Oct-14

Chd

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g Date/Time 06-Nov-2014 14:28

*****					
		=====		Y == ==	PROPERTIES IN INCH UNIT
MEMBER 775	*	AISC SECTIONS			AX = 7.22
	*	SD L60406		--Z	AY = 1.57
DESIGN CODE	*				AZ = 3.89
AISC-1989	*	-----			SY = 9.85
	*				SZ = 3.21
	*	<---LENGTH (FT)= 5.94 --->			RY = 2.92
*****					RZ = 1.17
0.0 (KIP-FEET)					
PARAMETER	L0			L0	STRESSES
IN KIP INCH	L0			L0	IN KIP INCH
-----	+L0			L0	-----
KL/R-Y= 75.91	L0			L0	FA = 13.19
KL/R-Z= 61.16	+L0			L0	fa = 4.19
UNL = 71.29	L0			L0	FCZ = 0.00
CB = 0.00	+L0			L0	FTZ = 0.00
CMY = 0.00	L0			L0	FCY = 0.00
CMZ = 0.00	+L0			L0	FTY = 0.00
FYLD = 30.00	L0			L0	fbbz = 0.00
NSF = 1.00	+-----+				fby = 0.00
DFF = 0.00	0.0				Fey = 25.92
dff= 0.00					Fez = 39.93
ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)					FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION		
=====	=====	=====	=====		
PASS	AISC- H1-1	3.180E-01	4		
30.28 C	0.00	0.00	0.00		
*					
*****					



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Date: 31-Oct-14

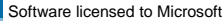
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Date/Time 06-Nov-2014 14:28

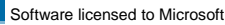
*****		Y		PROPERTIES	
MEMBER 777		AISC SECTIONS		IN INCH UNIT	
SD L60406		--Z		AX = 7.22	
DESIGN CODE				AY = 1.57	
AISC-1989				AZ = 3.89	
				SY = 9.85	
				SZ = 3.21	
				RY = 2.92	
				RZ = 1.17	
*****		<---LENGTH (FT)= 5.94 --->			
0.0 (KIP-FEET)					
PARAMETER		L0		STRESSES	
IN KIP INCH		L0		IN KIP INCH	
-----		+L0		-----	
KL/R-Y= 75.91		L0		FA = 18.00	
KL/R-Z= 61.12		+L0		fa = 0.63	
UNL = 71.25		L0		FCZ = 0.00	
CB = 0.00		+L0		FTZ = 0.00	
CMY = 0.00		L0		FCY = 0.00	
CMZ = 0.00		+L0		FTY = 0.00	
FYLD = 30.00		L0		fbz = 0.00	
NSF = 1.00		+-----+		fby = 0.00	
DFF = 0.00		0.0		Fey = 25.92	
dff= 0.00		ABSOLUTE MZ ENVELOPE		Fez = 39.98	
		(WITH LOAD NO.)		FV = 0.00	
				fv = 0.00	
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
		-----			
		AXIAL		SHEAR-Y	
		SHEAR-Z		MOMENT-Y	
		MOMENT-Z			
VALUE		0.0		0.0	
LOCATION		0.0		0.0	
LOADING		0		0	
*****					
* DESIGN SUMMARY (KIP-FEET)					
* -----					
* RESULT/		CRITICAL COND/		LOADING/	
* FX		MY		LOCATION	
* =====		=====		=====	
* PASS		TENSION		4	
* 4.55 T		0.00		0.00	
* 3.502E-02		0.00			
*****					



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*****					
		=====		Y ==  ==	PROPERTIES IN INCH UNIT
MEMBER 778	*	AISC SECTIONS			AX = 7.22
	*	SD L60406		--Z	AY = 1.57
DESIGN CODE	*				AZ = 3.89
AISC-1989	*	-----			SY = 9.85
	*				SZ = 3.21
	*	<---LENGTH (FT)= 5.94 --->			RY = 2.92
*****					RZ = 1.17
		0.0 (KIP-FEET)			
PARAMETER		L0		L0	STRESSES
IN KIP	INCH	L0		L0	IN KIP INCH
-----		+L0		L0	-----
KL/R-Y=	75.91	L0		L0	FA = 18.00
KL/R-Z=	61.12	+L0		L0	fa = 2.21
UNL =	71.25	L0		L0	FCZ = 0.00
CB =	0.00	+L0		L0	FTZ = 0.00
CMY =	0.00	L0		L0	FCY = 0.00
CMZ =	0.00	+L0		L0	FTY = 0.00
FYLD =	30.00	L0		L0	fbz = 0.00
NSF =	1.00	+--+ +--+ +--+ +--+ +--+ +--+ +--+ +--+ +---			fby = 0.00
DFF =	0.00	0.0			Fey = 25.92
dff=	0.00	ABSOLUTE MZ ENVELOPE (WITH LOAD NO.)			Fez = 39.98
					FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ		LOADING/ LOCATION	
=====					
PASS	TENSION	1.230E-01		4	
15.98 T	0.00	0.00		0.00	
*****					



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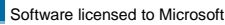
Chd

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*****					
		Y		PROPERTIES	
		== ==		IN INCH UNIT	
MEMBER 779 *		AISC SECTIONS		AX = 7.22	
DESIGN CODE *		SD L60406		AY = 1.57	
AISC-1989 *				AZ = 3.89	
*				SY = 9.85	
*				SZ = 3.21	
*		<---LENGTH (FT)= 5.94 --->		RY = 2.92	
*****				RZ = 1.17	
0.0 (KIP-FEET)					
PARAMETER		L0	L0		STRESSES
IN KIP INCH		L0	L0		IN KIP INCH
-----		+L0	L0		-----
KL/R-Y= 75.91		L0	L0		FA = 18.00
KL/R-Z= 61.12		+L0	L0		fa = 2.21
UNL = 71.25		L0	L0		FCZ = 0.00
CB = 0.00		+L0	L0		FTZ = 0.00
CMY = 0.00		L0	L0		FCY = 0.00
CMZ = 0.00		+L0	L0		FTY = 0.00
FYLD = 30.00		L0	L0		fzbz = 0.00
NSF = 1.00		+-----+			fby = 0.00
DFF = 0.00		0.0	Fey = 25.92		
dff= 0.00		ABSOLUTE MZ ENVELOPE			Fez = 39.98
		(WITH LOAD NO.)			FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/FX		CRITICAL COND/MY	RATIO/MZ	LOADING/LOCATION	
=====					
PASS	TENSION	1.230E-01	4		
15.98 T	0.00	0.00	0.00		
*****					





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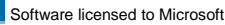
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*****					
		Y		PROPERTIES	
		== ==		IN INCH UNIT	
MEMBER 780 *		AISC SECTIONS		AX = 7.22	
*		SD L60406		AY = 1.57	
DESIGN CODE *				AZ = 3.89	
AISC-1989 *				SY = 9.85	
*				SZ = 3.21	
*		<---LENGTH (FT)= 5.94 --->		RY = 2.92	
*****				RZ = 1.17	
		0.0 (KIP-FEET)			
PARAMETER		L0		L0 STRESSES	
IN KIP INCH		L0		L0 IN KIP INCH	
-----		+L0		L0 -----	
KL/R-Y= 75.91		L0		L0 FA = 18.00	
KL/R-Z= 61.12		+L0		L0 fa = 1.40	
UNL = 71.25		L0		L0 FCZ = 0.00	
CB = 0.00		+L0		L0 FTZ = 0.00	
CMY = 0.00		L0		L0 FCY = 0.00	
CMZ = 0.00		+L0		L0 FTY = 0.00	
FYLD = 30.00		L0		L0 fbz = 0.00	
NSF = 1.00		+-----+		fbz = 0.00	
DFF = 0.00		0.0		fby = 0.00	
dff= 0.00				Fey = 25.92	
		ABSOLUTE MZ ENVELOPE		Fez = 39.98	
		(WITH LOAD NO.)		FV = 0.00	
				fv = 0.00	
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
		AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y
					MOMENT-Z
VALUE		0.0	0.0	0.0	0.0
LOCATION		0.0	0.0	0.0	0.0
LOADING		0	0	0	0
*****					
*					
*					
* DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/		CRITICAL COND/	RATIO/	LOADING/	
FX		MY	MZ	LOCATION	
=====		=====		=====	
PASS		TENSION	7.779E-02	4	
10.11 T		0.00	0.00	0.00	
*****					
*****					



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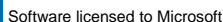
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*****					
		Y		PROPERTIES	
		== ==		IN INCH UNIT	
MEMBER 781		AISC SECTIONS		AX = 7.22	
		SD L60406		AY = 1.57	
DESIGN CODE				AZ = 3.89	
AISC-1989				SY = 9.85	
				SZ = 3.21	
		<---LENGTH (FT)= 5.93 --->		RY = 2.92	
*****				RZ = 1.17	
0.0 (KIP-FEET)					
PARAMETER		L0	L0		STRESSES
IN KIP INCH		L0	L0		IN KIP INCH
-----		+L0	L0		-----
KL/R-Y= 75.91		L0	L0		FA = 13.19
KL/R-Z= 61.08		+L0	L0		fa = 4.19
UNL = 71.21		L0	L0		FCZ = 0.00
CB = 0.00		+L0	L0		FTZ = 0.00
CMY = 0.00		L0	L0		FCY = 0.00
CMZ = 0.00		+L0	L0		FTY = 0.00
FYLD = 30.00		L0	L0		fzbz = 0.00
NSF = 1.00		+-----+			fby = 0.00
DFF = 0.00		0.0	Fey = 25.92		
dff= 0.00		ABSOLUTE MZ ENVELOPE			Fez = 40.02
		(WITH LOAD NO.)			FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
AXIAL		SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
RESULT/ FX		CRITICAL COND/ MY		LOADING/ LOCATION	
=====		=====		=====	
PASS	AISC- H1-1	3.180E-01		4	
30.28 C	0.00	0.00		0.00	
*****					



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## Steel Design (Track 2) Beam 783 Check 1

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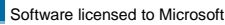
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*****					
		Y		PROPERTIES	
		== ==		IN INCH UNIT	
MEMBER 785		AISC SECTIONS		AX = 10.13	
		SD L60607		AY = 2.65	
DESIGN CODE				AZ = 4.68	
AISC-1989				SY = 11.59	
				SZ = 8.16	
		<---LENGTH (FT)= 5.98 --->		RY = 2.67	
*****				RZ = 1.87	
		0.0 (KIP-FEET)			
PARAMETER		L0		L0	STRESSES
IN KIP INCH		L0		L0	IN KIP INCH
-----		+L0		L0	-----
KL/R-Y=	72.40	L0		L0	FA = 14.02
KL/R-Z=	38.41	+L0		L0	fa = 7.52
UNL =	71.82	L0		L0	FCZ = 0.00
CB =	0.00	+L0		L0	FTZ = 0.00
CMY =	0.00	L0		L0	FCY = 0.00
CMZ =	0.00	+L0		L0	FTY = 0.00
FYLD =	30.00	L0		L0	fzbz = 0.00
NSF =	1.00	+-----+		L0	fby = 0.00
DFF =	0.00	0.0			Fey = 28.49
dff=	0.00	ABSOLUTE MZ ENVELOPE			Fez = 101.19
		(WITH LOAD NO.)			FV = 0.00
					fv = 0.00
MAX FORCE/ MOMENT SUMMARY (KIP-FEET)					
-----					
	AXIAL	SHEAR-Y	SHEAR-Z	MOMENT-Y	MOMENT-Z
VALUE	0.0	0.0	0.0	0.0	0.0
LOCATION	0.0	0.0	0.0	0.0	0.0
LOADING	0	0	0	0	0
*****					
*					
*					
DESIGN SUMMARY (KIP-FEET)					
-----					
*					
RESULT/ CRITICAL COND/ RATIO/ LOADING/					
FX MY MZ LOCATION					
=====					
PASS	AISC- H1-1	5.364E-01	4		
76.19 C	0.00	0.00	0.00		
*****					



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Date 31-Oct-14

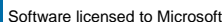
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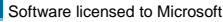
Date 31-Oct-14

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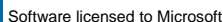
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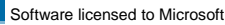
Date 31-Oct-14

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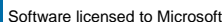
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*****					
		Y		PROPERTIES	
				IN INCH UNIT	
MEMBER 794		==  ==			
*					
AISC SECTIONS				AX = 2.13	
SD L25204				AY = 0.50	
*				--Z AZ = 1.04	
DESIGN CODE				SY = 1.05	
AISC-1989				SZ = 0.51	
*				RY = 1.11	
<---LENGTH (FT)= 6.50 --->				RZ = 0.59	
*****					
0.0 (KIP-FEET)					
PARAMETER		L0		STRESSES	
IN KIP INCH		L0		IN KIP INCH	
-----		+L0		-----	
KL/R-Y= 72.56		L0		FA = 8.53	
KL/R-Z= 131.81		+L0		fa = 1.54	
UNL = 78.00		L0		FCZ = 0.00	
CB = 0.00		+L0		FTZ = 0.00	
CMY = 0.00		L0		FCY = 0.00	
CMZ = 0.00		+L0		FTY = 0.00	
FYLD = 30.00		L0		fbz = 0.00	
NSF = 1.00		+-----+		fby = 0.00	
DFF = 0.00		0.0		Fey = 28.36	
dff= 0.00		ABSOLUTE MZ ENVELOPE		Fez = 8.60	
		(WITH LOAD NO.)		FV = 0.00	
				fv = 0.00	
		MAX FORCE/ MOMENT SUMMARY (KIP-FEET)			
		-----			
AXIAL		SHEAR-Y		SHEAR-Z	
				MOMENT-Y	
				MOMENT-Z	
VALUE		0.0		0.0	
LOCATION		0.0		0.0	
LOADING		0		0	
*****					
*					
*		DESIGN SUMMARY (KIP-FEET)			
*		-----			
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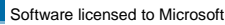
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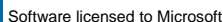
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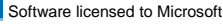
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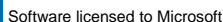
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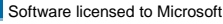
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## Statics Check Results

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**APPENDIX E -  
GEOTECHNICAL REPORT  
JULY 24, 2007**



**GEO-SCIENCE ENGINEERING Co., Inc.**  
CONSULTING GEOTECHNICAL ENGINEERS

July 24, 2007

Highland Associates  
102 Highland Avenue  
Clarks Summit, Pennsylvania 18411

Attention: Mr. William Flynn, R.A.

Reference: Existing Subsurface Conditions  
Elmer L. Meyers High School  
341 Carey Avenue  
Wilkes-Barre, Pennsylvania  
GSE Project No. 07036

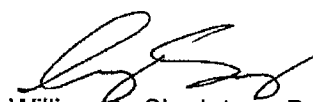
Dear Mr. Flynn:


Enclosed is our report for the above referenced project. The report was based upon our discussions and your subsequent authorization. The study concludes there are several issues with the site.

Please review the findings noted within the report at your convenience. Upon completion of the review, we will be available for a meeting to discuss the report and answer any questions that may have developed.

We are pleased to have been of service to you on this project. Should you have any questions please do not hesitate to contact us.

Sincerely,

  
William A. Singletary, P.E.  
Senior Geotechnical Engineer

  
P. Richard Scheller, P.E.  
President

PRS/lmm  
07036:MEYERSGEOTECHRPT  
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File (1)

BRANCH OFFICE

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☒ Jessup, PA  
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Fax: (570) 489-8714

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☐ Urbana, Illinois  
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Analysis and Discussion	3.0	11
Conclusions	4.0	17
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Appendix A	Plates
Appendix B	Field and Laboratory Test Data
Appendix C	Drillers Boring Logs
Appendix D	Important Information About Your Geotechnical Report



**INTRODUCTION****1.0****1.1 GENERAL INTRODUCTION**

The geotechnical investigation reported herein was performed at the request of Mr. William Flynn, R.A., Highland Associates, in connection with an existing condition review of the Elmer L. Meyers High School, Wilkes-Barre Area School District. The site is located at the intersection of Carey Avenue and Hanover Street in Wilkes-Barre, Pennsylvania. A "Regional Location Plan" showing the general location of the project site is presented as Plate 1 in Appendix A. More specific site location is shown on the test boring plan, Plate 3, Appendix A.

The purpose of this investigation was to examine the subsurface conditions at the existing school through the use of test borings. This investigation was determined to be necessary by Highland Associates due to apparent signs of settlement within the structure. The test borings were scheduled to develop a geotechnical understanding of the subground. The results of this study would assist the Architect in the general analysis and recommendations to renovate the school.

The scope of work included review and planning of the investigation with Highland Associates, obtaining an independent test boring contractor, laboratory testing of representative samples, engineering analysis of the available geotechnical data, and the subsequent preparation of the enclosed report.

A description of the subsurface exploration program and logs of the test borings are included in Appendix B, together with the laboratory test program as may have been required to analyze the data.

Authorization to perform this study was issued by Mr. William Flynn, R.A, Highland Associates.

**1.2 PROJECT DESCRIPTION**

Based upon the general data description provided by Highland Associates, the existing building is a multi story school structure with a basement. The structure has a footprint of approximately  $102,000 \pm \text{ft}^2$ . The basement elevation is approximately  $527 \pm$  feet. The sidewalk at the main entrance is approximately  $539 \pm$  feet. This would suggest the first floor elevation is approximately  $542 \pm$  feet. The noted elevations were estimated from local references and should not be identified as the actual elevations. The structure appears to be a structural steel skeleton with masonry concrete exterior and interior walls with a

brick veneer.

The actual column spacing has not been determined. From a drawing prepared by Highland Associates, at least in one direction the spacings will be in the range of 15-20 feet on center for the building. Highland Associate's preliminary estimate of column loads is in the range of 120 to 150 kips design loads for the building. Wall loads were estimated to be in the range of 5 to 10 KLF.

The footprint is trapezoidal in shape. The gross area is approximately 230,000ft<sup>2</sup>.

### 1.3 SITE CONDITIONS

The site is located on the north side of Carey Avenue at the intersection with Hanover Street. The Elmer L. Meyer is currently servicing as a 7-12 grade high school. The school complex sits on approximately 7.5 acres. The center core of the school has an open to the outside basement level access area. Two to three-story structures are located along the football field side. The field has bleachers and other typical structures found at stadium complexes.

In order to access the physical conditions from a geotechnical standpoint, both a walk through and walk around the structure was conducted. These visits were accompanied with members of the Highland Associates team. The exterior of the structure had numerous cracks and separations. There was a number of locations where masonry joint materials had spalled from the joint. Our initial assessment suggested there was some observed settlement cracks.

Our walk through of the building did not yield any severe cracking or distortion. The exception was floors. In some cases, the floors were out of level. A golf ball was set in several of the corners of classrooms and the golf ball immediately rolled to the opposite side. During one of the site visits, it was raining. A number of upper level ceiling leaks were observed especially on the upper most floor. The basement also showed signs of movement. Water leaks from water and steam(?) piping ran across areas of the floors and connected in low spots. In several locations water was observations beneath the floor covering. Site observations also revealed damage to down spouts from the rain gutters which were discharging water immediately adjacent to the foundation.

From our visual observation, the grading changes direction in many locations to drain to inlets. The overall drainage is from the southwest toward the northeast.

No subsidences, pothole depressions or other limiting conditions were expressed during the walk over. Grasses and vegetation indicate the site is manicured with ornamental trees

and no undergrowth. In the limited landscaped zone there is a topsoil layer.

#### 1.4 HISTORICAL ISSUES

The current Elmer L. Meyers High School was constructed in approximately 1930. There were no records available to determine the type of foundation used to support the school. It was suspected the structure may have been supported by a pile foundation system due to the alluvial soils and as this was a school. Traditionally, schools and other structures of this classification (government offices, hospitals) are usually designed to a higher standard. Most of the local multi-story structures were constructed on a spread footing type foundation. Wilkes-Barre and in particular South Wilkes-Barre is an alluvial flood plain for the Susquehanna River. The river is located approximately 2,000± feet from the school. During various epochs the Susquehanna River carved and flooded the surrounding area. Deposits from these sequences of flooding formed the area beneath the school. As the flood deposits continued to build-up, shallow river banks were forming.

In 1810 historical records were begun. Based upon these records, the Susquehanna River basin is one of the most flood prone areas in the nation. Since that time fourteen (14) major flooding events have occurred with the worst disaster occurring in 1972 with Tropical Storm Agnes. The river containment system was constructed as a result of the flood that struck on St. Patrick's Day in 1936. On that day, the river crested at 33 feet and flooded a significant portion of Wilkes-Barre, Kingston and the surrounding area. As a result of the event, the levees were constructed to protect from a flooding event to a height of 36 feet. The flood of 1902 crested at 31 feet. In 1865, another flood crested at 33 feet. While the area was principally farming and the citizenry build-up was relatively sparse.

After the construction of levee, the levee was nearly overtopped on several occasions. In 1940 and 1946, the river reached 32 feet, but the levees contained the river and flooding was avoided. The 1972, the levee system was breached by tropical Storm Agnes. During this event, the river level rose to beyond 40.9 feet, approximately 5 feet above the levee. The consequences of that event are well known. It was reported that water was observed to have completely flooded the basement and extended approximately 8 feet above the first level of the Elmer L. Meyers High School. The school was also reported flooded during the St. Patrick's Day flood of 1936. After the Agnes Flood, the levee system was again raised to protect from the Agnes disaster. The levee raising was not completed until January 2003. Since 1972, the river has crested above 34 feet four (4) additional times. The maximum height was 35.1 feet in 1975, a near disaster for Wilkes-Barre again.

River levels have historically exceeded the ground level of Elmer L. Meyers High School at least nine (9) times since construction. Based upon historical records, it appears that

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Elmer L. Meyers High School has been flooded at least two (2) times since it's construction.

**2.0**

**FIELD EXPLORATIONS**

**2.1 FIELD RECONNAISSANCE**

A field reconnaissance was conducted at the project site at various times during May/June/July 2007. Existing site conditions in the areas of the improvement reflect altered topography. These conditions were created during the construction of the school and surrounding structures.

**2.2 SUBSURFACE INVESTIGATIONS**

A total of six (6) core borings were drilled at the site. The test borings were completed by Eichelbergers, Inc., a drilling company under contract with GSE, Inc. Drilling inspection was provided by Geo-Science Engineering Company, Inc. Field boring locations were placed by Highland Associates/Geo-Science Engineering Co., Inc. The approximate location of each boring drilled is presented on the Core Boring Location Plan. (Appendix A and attached on the following sheet)

A total of 386± lineal feet of drilling was completed during the subsurface exploration program. Rock was encountered at a depth of 129 elevation (397.6) feet.

Each boring drilled was initially advanced through the soil overburden using 6" hollow stem augers or by driven 4" casing until the design depth, top of boulders, or bedrock was encountered. Standard Penetration Tests (SPT's) were generally conducted at periodic intervals until refusal occurred due to the presence of cobbles and/or boulders as designated by 50 blows over six (6) inches of penetration or less. Upon successful penetration through cobbles and/or boulders, SPT's were resumed until bottom of the boring was achieved. At the conclusion of each boring, after the final water level reading was recorded, each core boring was backfilled with cuttings.

Drilling was generally conducted according to accepted protocols and standards. The SPT's were generally performed in accordance with ASTM Designation D 1586, "Standard Method for Penetration Test and Split-Barrel Sampling of Soils."



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Detailed boring logs for each of the core borings were prepared by Geo-Science Engineering Company, Inc., and are presented in Appendix B.

Logs as prepared by the drilling contractor are presented in Appendix C.

Each boring log prepared includes the following items:

- The approximate depth, and description of materials encountered;
- The type and location samples;
- The Standard Penetration Resistance obtained from SPT's conducted at the location of split barrel samples along with sample recovery measurements;
- The percent recovery and rock quality designation (RQD) for each run of rock core and the RQD for each rock stratum encountered;
- The Unified Soil Classification System soil classification of samples by visual determination; and
- The water levels in the boring at the time transpired after drilling completion

The following is a tabulation of the approximate test boring elevations. These elevations were estimated from regional topography.

**Tabulation of Test Boring Elevations**  
**Table 2.2.1**

BORE HOLE #	SURFACE ELEVATION RELATIVE (FT)	DEPTH OF HOLE (FEET)
B-1	539	50
B-2	527	50
B-3	539	50
B-4	527	136
B-5	527±	50
B-6	527±	50

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In addition to the test borings, a series of cores were obtained in the floor. Table 2.2.2 details the results of these cores.

**Table 2.2.2**  
**Summary of Core Results**

<u>CORE NUMBER</u>	<u>LOCATION</u>	<u>CORE THICKNESS (INCLUDES TILE)</u>	<u>VOID BENEATH SLAB THICKNESS</u>	<u>SUBGRADE</u>
1	Football Locker Room	8.5"	1/4"	Moist, Silty Sand
2	Batting Cages	5.25"-11.25"	None	Moist, Silty Sand
3	Basement Beneath Stairs	5.5"	None	Moist, Silty Sand
4	Hall Near pool	5.75"	3/4"	Moist, Silty Sand
5	Pool Area	5.25"	1 1/4"	Moist, Silty Sand
6	Pool Area	5.50"	2 1/4"	Moist, Silty Sand

### 2.3 LABORATORY INVESTIGATIONS

Upon completion of the field explorations, the recovered soil samples were examined by our staff geotechnical engineer and a laboratory program was conducted to ascertain engineering characteristics of the subsurface materials encountered. All phases of the laboratory testing were conducted in general accordance with applicable ASTM specifications. Gradation analyses, atterberg limits test, visual referrals, and natural moisture content were also performed. The results of these tests are noted in Appendix B and the table below. Descriptions of the soils follow general outlined procedures established by the Unified Soil Classification System and ASTM. The soils are more adequately described in the following sections.

**Table 2.3.1**  
**Summary of Laboratory Tests**

<u>Test Boring</u>	<u>Sample</u>	<u>Depth</u>	<u>Classification</u>	<u>-200</u>	<u>LL</u>	<u>PL</u>	<u>Density</u>	<u>Natural Moisture</u>
B-1	S-4	8-10	ML	72.4	NP	NP	-	24.2
B-1	S-9	33-35	ML	99.8	NP	NP	-	-
B-6	S-7	23-25	SP	2.8	NP	NP	-	-

### 2.3.1 Surface Covering

The site surface is covered by asphalt, minor concrete and exposed soil and grass materials. The covering surfaces vary up to 12 inches in thickness. The thickness could easily vary between test boring locations. The surface covering in the asphalt material includes the subbase which is 4 inches to 6 inches in thickness.

### 2.3.2 Fills

Fills were encountered on this site in the test borings. The probable depth of the fills is noted on the following chart. Other locations will be adjacent to building foundations, around and over utility locations/manholes, etc.

**Table 2.3.2.1**  
**Summary of Probable Fill Depths**

<u>BORING NUMBER</u>	<u>SURFACE ELEVATION (FT.)</u>	<u>DEPTH OF FILL (FT.)</u>	<u>BOTTOM OF FILL ELEVATION (FT.)</u>
1	539	8 (28)	531 (511)
2	527	8	519
3	539	18	521
4	527	4	523
5	527	10	517
6	527	6	521

Based upon the test borings, the fills contained some construction debris, coal fragments and other indicators that would suggest the material was not sediment

from the river. We suspect the material was probably placed to raise the surrounding grade. The fills are rather uniform in that they generally range from 517 to 523. The exception is B-1, which has approximately 8 feet of fill. The base of the fill is ash, but there are some soil materials that would appear to be fill to a depth of 28 feet (elevation 511). The fills range from moist to wet and are very loose to medium dense.

### 2.3.3 Native Soils

The native soil types of the area are alluvial sediments, which contain varying percentages of clay, silt, sand and gravel in distinct beds. The samples tested had approximately 9 to 99% finer than the #200 sieve which is typical for the river alluvial deposits at this general location. Moisture contents for this strata vary between 8%± and 30%±. The samples indicate moist to wet conditions. The "N" values determined from the Standard Penetration Tests vary from very loose (very soft) to medium dense (stiff).

Some varving of the silts and clays and sands and clays have been observed. From studies performed in the surrounding area, the soils have experienced a pre-consolidation pressure of approximately 1 TSF.

### 2.3.4 Groundwater

Groundwater was reported in the test borings. The water depth below the surface varied from 28 feet to 18 feet. The approximate elevation is from 500 feet to 511 feet. The water levels generally are within a gravelly sand, silty sand with gravel layer that is sandwiched between two (2) silt layers. The water is usually at the underside of the silt layers.

Based upon past experience in the immediate area, the average river level is 515± feet. It is suspected the groundwater levels will parallel river levels with a time lag. Discussions with Apollo Group indicated in 2006 when they excavated for some construction dewatering of the excavation was necessary.

### 2.3.5 General Geology

The project site is above the coal bearing Llewellyn Formation which is of Pennsylvania Geologic Age. The Llewellyn Formation is the principal, near surface stratigraphic unit present over the site. This formation is composed of interbedded

sandstone, siltstone, and conglomerate rock units. The sandstones are medium to coarse grained and light gray to black. The black shales have a high carbon content and are referred to as carboniferous. The Llewellyn Formation also contains several major anthracite coal beds. The formation strikes generally northeast/southwest and ranges in dip from flat to nearly vertical depending on where the point of reference is obtained such as the many oblique angle folds in the base of the synclinalorium or near the outcrops. Bedding throughout the unit is moderately well developed. Coal and shale bedding is generally thin, while the sandstone, siltstone, and conglomerate bedding may be thin to massive. Joints are moderately developed and moderately abundant in blocky patterns and moderately spaced in regular sequence. The joints can be open and steep depending on the lithology. In many cases there is abundant fracturing in the area due to the removal of the anthracite mineral and the resultant lithostatic adjustment of stress.

#### **2.3.6 Bedrock**

Bedrock was encountered in test boring B-4 at the site. The bedrock was encountered at a depth of 129± feet below the ground surface. All of the other borings were terminated in the unconsolidated alluvium.



## 3.0

## ANALYSIS AND DISCUSSION

As a result of our on-site visual review, it appeared the existing structure has undergone settlement. It was discussed with Highland Associates structural department, that with the floor slabs tilted, the structure must have moved. It was our initial thoughts the school may be supported by piles. However, if the school was supported by piles it would be most likely the piles were driven to the underlying bedrock (approximately 130± feet below the ground surface). If the building settled, we suspected that the issue would be mine related. Test boring B-4 was drilled to confirm the top of bedrock. According to the mine data, the top of bedrock is approximately elevation 425-430 feet. The test boring data suggests the top of bedrock to be at elevation 398 feet, approximately 30 feet lower than the mine map would suggest. Apparently the top of rock was identified with drill logs S1010 and S1239 performed by the mining companies, in addition to others. In many cases there is coarse sand and gravel over the bedrock which when drilled with churn drills or air advanced percussion equipment may appear to be bedrock. At the location of test boring B-4, the dense bottom layer did not exist. Our surface elevations are consistent with USGS data for the site. This elevation difference must be related to discrepancies in drill hole information. The depth of drilling for this project was monitored by this office. The depth at which rock was encountered is correct. The elevation difference between rock information may be off by several feet due to differences in ground surface elevations, but not 30 feet. A review of the mine data was conducted to determine the potential for a mine related collapse.

The first mined measure beneath the site is the Abbott Bed. The coal measure is approximately at elevation 80± feet or 320 to 340 feet below the bedrock surface depending upon the actual rock surface elevation. There is a barrier pillar that separates the two (2) colliers approximately 1/3 of the way into the school in a east-west direction. There was a small heading driven into the center of the school and then stopped. Mining was apparently stopped in this area in 1944±. In the South Wilkes-Barre Colliery mining stopped approximately 50 to 100 feet from the school. The geology at this location appears to be a small oblique fold within a faulted area. The faulting changes the attitude of all the coal measures in the general vicinity of the fault.

The next lowest coal bed, the Kidney Bed was the first mined level completely mined to the barrier pillar on both sides of the barrier pillar. The coal measure is approximately at elevation -30 to +30 feet MSL. The bed was first mined with a 40% extraction ratio. Most pillars are 30 feet in width. The coal measure was approximately 5.0 feet thick.

The third vein is the Hillman Bed which is approximately 8 feet in mineable thickness. The

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coal bed is found at -8 to -100± feet below MSL. The coal bed is about 40 to 70 feet below the Kidney Bed. Again, this coal bed is about 450 feet below the top of bedrock. The extraction ratio is ±40% with pillars approximately 35' wide. No remnant mining was recorded.

The fourth bed in sequence is the Stanton Bed. The coal was approximately 5 feet in thickness. The extraction ratio is approximately 40% with pillars exceeding 30 feet in width. First mining has been recorded. There is not evidence of remnant mining performed within this coal bed.

Below the Stanton Bed on the south side of the barrier pillar there is a 250 feet thickness of unmined zone due to the upheaval at the faulted zone. The next level of mining is the Baltimore Bed. On the north side of the barrier there is fragmented mining below the Hillman Bed. The Five Foot Bed reoccurs in addition to the Lance Bed which are above the Baltimore Bed.

Analysis of the remaining coal pillars documented indicate that only first mining was conducted. No robbing or removal of pillars is reported. There is some evidence of pillar splitting within the differing coal measure. It is our opinion the pillars are safe and will remain with sufficient factors of safety for at least the next 50+ years. In order to have a large scale collapse in room and pillar mines at least 4-5 adjacent pillars must collapse. Our review of the data suggests that this is not reality unless the coal pillars have been removed and the roof is artificially supported. The coal pillars are inundated which helps reduce oxidation and spalling. Therefore, a large area collapse while not impossible is highly unlikely to occur, if a collapse in any bed above the Baltimore Bed were to occur, the beds above would be impacted. This in turn would cause ground lowering at the surface. If all the coal beds were to fail from the Stanton Bed to the Abbott Bed, the resultant ground lowering could be as much as 7 feet. Of only the Stanton Bed were to collapse, the potential maximum ground lowering would be ±1.5 feet. However, it is highly unlikely this event has occurred. If the event did occur, we would see more damage to the existing structure.

Our examination of the mine condition indicates there is a very low risk of a mine related movement at this site. If some movement has occurred, it may be the result of lithostatic creep. Our analysis indicates this movement would be limited to much less than a few inches at most. This would indicate to us the building is probably not supported by a deep foundation.

Upon conclusion of the mine analysis, a series of six (6) test borings were conducted at the

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site. The borings were drilled to a depth of approximately 50 feet. Boring B-4 was drilled to approximately 100 feet. In addition, a survey of the building was conducted by Michael Pasonick Jr. Engineering. This survey indicated the building has moved considerably. From the survey, we plotted approximate contours. In reviewing the elevation in the eastern most part of the main structure there is a greater than 12 inches of difference. In other parts of the school, the elevation differences range from 1-2 inches to greater than 7 inches.

In order to access the settlement value, we had to confirm if the structure was supported on piles or a conventional foundation system. We requested that a foundation be exposed so that we could examine the construction. Apollo Construction was contacted and a foundation location was selected. The foundation was found to bear approximately 7 feet below the basement floor slab (approximate elevation  $520 \pm$  feet). The footing was determined to be approximately 7' x 6'. A sketch prepared by Michael J. Pasonick Jr. Inc. is included in Appendix A. The footing was probed to determine if there was pile support. Based upon a review of the footing thickness and the results of probing, it was concluded the foundation was a conventional footing supported by the ground at -7 feet (elevation  $520 \pm$  feet) below the basement elevation. From an analysis of probable maximum loads it was determined the footing was being loaded to approximately 3,000 psf. In order to examine the shearing resistance for this soil, we examined the "N" values within 3 times the footing width (18 feet) below the base of the footing.

Boring B-2 through B-6 were similar. Boring B-1 appeared to be more dense. One possible explanation of the difference is the density difference may be related to a former river bank which contained slightly denser materials. The area to the north was part of the former river channel which has been filled with sediment.

The borings revealed the following "N" values.

<u>BORING NUMBER</u>	<u>AVERAGE "N"</u>
B-1	13
B-2	4
B-3	7
B-4	7
B-5	7
B-6	7

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The average "N" value from all borings was 7 blows/foot. "N" values at this level could result in punching shear issues for foundation performance.

Based upon current water levels, the recorded ground water elevation are far enough below the foundation levels so that ground water would not impact the bearing capacity analysis. However, as we know there has been several floods and high river levels that temporary rises in groundwater could occur. We included a water table at the subground surface. Our analysis suggests the allowable bearing capacity for the conditions noted would range from 2,000 psf to 3,000 psf. This would be consistent with the footing sizes and the approximate calculated column load provided by Highland Associates. Generally speaking for shallow foundation design, settlement limitations govern bearing stresses. In this case, the bearing stress applied is 2,000-3,000 psf. Back calculating estimated settlements reveals values ranging from 1 to 1.75 inches depending upon the correction factors used and the methods employed. Schmertman, Hough and others indicate that even with granular soils continued movement does occur due to reorientation of grains and creep movement. It is suggested that the computed settlements need to be adjusted upwards. At about 50 years, the anticipated increase is 50%. Therefore the predicted settlements would be in the 1.5 to 2.5± inches under normal application of the foundation loads.

At this level of movement, we would have anticipated some minor cracking and distortion, but in general a reasonable building performance. From discussions with maintenance personnel, a number of door frames have been repaired/adjusted to free from sticking. In order to access the data provided by Pasonick, we drew lines of equal contours through the survey data. Those contour lines indicated the maximum deviation in the structure is approximately 1.1 feet or 13.2 inches.

The greatest amount of movement has occurred in the western area of the complex, where nearly the entire western wing has moved approximately 1 foot. The greatest gradient appears at the front entrance to the face of the stair tower. This gradient is noted to be 0.8± feet of movement. The eastern portion of the school, the gradient is relatively uniform front to back at 0.6± feet (7.21 inches) total apparent movement. This movement is in excess of the movement predicted from purely building loads and subsurface performance.

In order to search for additional explanations of the movement, we examined rain leader discharge, broken water lines, truck traffic vibrations, etc. We observed several locations along the interior courtyard where rain leaders were disconnected and discharging directly to the ground surface. We have conducted numerous forensic investigations where the continued flow of water eventually caused settlement of foundations. However, these conditions were spot specific and easily identifiable.

Similarly, there was some reports of broken water lines and a leaking pool. The continued leakage would create settlement conditions that would be location specific and the effects of the movement would be discernable.

Additionally, repeated truck traffic induces low frequency vibrations into the soil. Generally, the vibrations are attenuated within short distances and do not significantly impact structural performance. With the fill zone and the silty sand layer these vibrations over time could create grain realignment such that some settlement may be recorded. While this value is not great, it will add to the total overall settlement.

In order to examine the distortion, Highland Associates structural department requested and received to have wall covering and fireproofing removed to expose several of the connections. Based upon Highland Associates review it was their opinion the connections do not show signs of significant distress.

To further review existing site conditions, we had some discussions with some knowledgeable people and a further review of our files for any local information. This search indicated after the Agnes Flood in 1972, there were areas where sand boils were observed along both the eastern and western sides of the levee. One area is not vary far from the school location. There was some asphalt roadways that had settled or voids were encountered beneath the pavements during reconstruction. Upon construction of an addition to a multi-story structure (within proximity to this structure), an out of alignment condition was discovered. This difference in elevation amounted to 12-18 inches, much similar to the condition reported for the existing school structure.

Based upon the additional information, we began to examine the potential impact of the Susquehanna River, the consequences of flooding and the rise and fall of river levels within the confines of the dike. There are no piezometers in the area that record ground water elevation change due to rise and fall of the river. During the time of this investigation, the river gauge was approximately at normal river flow conditions. Average river levels are approximately  $\pm 5$  feet of the gauge elevation. The recording station at Wilkes-Barre, Pennsylvania has a gauge datum of 510.86 feet. From the test borings, the ground water



elevation is approximately 505 feet. We were unable to find the river gauge elevations during May/June to compare. But, based upon discharge, the river was below normal and there may be a definitive correlation of river level and ground water. A conversation with a member of the Apollo Group indicated that in 2006 they had opened an excavation for construction. The excavation immediately filled with water and dewatering was required. It was offered by the construction personnel, it was believed the river level was up!!! Obviously, this is a bias, but does provide some in-sight that the ground water levels and river levels are connected. We attempted to contact the U.S. Corps of Engineers to determine if any hydraulic studies had performed in the vicinity of the school to examine velocities and gradients of the moving ground water. Unfortunately, this information was not available. Historically, upriver from the school around Gordon Avenue and the pump station there has been a number of sand boils that develop upon the river reaching certain elevations. In 2006 when the river reached a crest of  $34\pm$  feet, the sand boils were encountered in both sides of the river in Kingston and Gordon Avenue. In order to accurately examine fluctuating river levels and its impact on the soils beneath the school, additional studies would be necessary. A series of piezometer would need to be installed and monitored. Insitu vertical and horizontal permeability tests of the soils between the levee and the school would need to be established. The soils profiles between the levee and the school will need to be understood. Upon completion of this effort, a detailed analysis can be constructed. This analysis could take a year to complete due to the necessity of establishing the baseline information, constructing appropriate flow nets and performing the analysis.

It is known that the downward movement of water through the soil mass can effectively place the soil into a more close packet arrangement. The common practice of jetting was performed to consolidate utility trenches. Even today, after utility excavations are completed and a rainfall occurs, the trench surface settles.

4.0

CONCLUSIONS

From the results of this geotechnical study, the following information has been determined.

1. The existing school has been constructed over an alluvial flood plain deposit.
2. The school has been flooded at least two (2) times in its history, 1936 and 1972.
3. The existing school is supported by a conventional shallow spread footing foundation system.
4. The school is underlain by a series of abandoned anthracite mines. The closest bed is approximately 400± feet beneath the ground surface, approximately 320 feet below the rock surface.
5. Data provided by Pasonick Engineering indicates there is a deviation within survey readings taken in support of this project. The deviations exceed 12 inches in some locations.
6. Uncovered structure connections do not visually appear to be overstressed (torn, stretched, ripped, etc).
7. Estimates of column loads and footings size indicates an applied bearing pressure of 2,000 to 3,000 psf. Based upon the test borings, an allowable capacity of 2,000-3,000 psf is computed using ground water levels which vary from the basement subgrade to elevation 505 feet, its current level.
8. Settlement computations indicate up to 2.5 inches of movement can be estimated based upon existing subgrade conditions. It is possible some additional movement can be attributed to other factors such as deep mining, rain leaders, traffic vibrations, etc.
9. Although a site specific hydraulic study was not performed to determine critical gradients, velocities, sand boils and uplift potential, it is our opinion the remaining movement can be attributed to both Floods of 1936 and 1972 and the ground water connection with the rise and fall of the river levels.
10. Voids were encountered beneath the slabs in the hall near the pool. These

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voids most probably developed due to the movement of water through the site, but more probably due to leaks in the shower room adjacent to the area. This void should be grouted to reestablish contact with the floor slab.

## 5.0

## OPINION

Based upon the results of this study, several questions were raised to future predictions. Based upon the computations, the predicted movement since construction would yield values of 1.5 to 2.5 inches. This level of movement would be well tolerated by the structure. Although we would have anticipated some maintenance work would be necessary to repair the minor cosmetic damage. If we look further into the future and based upon the existing soil conditions, pushing out another 50 years, the relationship for predicting movement becomes rather sketchy at best. It would be our estimate that another 1 to 2 inches of movement may occur. At this settlement we would be reaching the probable tolerable limits for this type of structure. If this settlement were to occur, we would have predicted some structural repairs might be necessary. However, when we examine the survey data settlement greater than 12 inches has been reported. At this level, we would have surely anticipated some severe structural problems. While not structural engineers, this level of movement is troublesome. We believe the movement most likely occurred during elevated river events including the floods where receding water caused settlement in the subground soils. Much of this movement needed to be uniform so not to produce some substantial cracking within the structure.

If we were to look into the future, the only guidance we would rely upon if that is a correct term, would be to believe the conditions which developed the current building state would occur again.

If desired, the existing site conditions could be modified to accept the potential of these conditions developing again. It would take the necessary steps to develop a hydraulic model to better understand the hydraulic connectivity with the site soil conditions and the nearby river so that an in-depth engineering judgement can be obtained. If hydraulic issues are determined, relief wells can be constructed to reduce pressure head differentials. It is possible to develop support from the bedrock surface beneath the structure or floating the entire structure on a mat slab. In addition, an in-depth seismic analysis would be required as the site class falls into a F-E classification. Any significant modification to the foundation system would require the foundation to achieve seismic resistance defined by the new building codes.

It is our opinion should the School District desire to maintain the school at this location, geotechnical/structural upgrades could be completed to reduce the potential for future movement. It is also our opinion this effort will be costly. Since we are not contractors, a cost estimate should be provided by the District's Construction Manager. Similarly, if the School District desires to build a new school at this location, sufficient geotechnical/structural components would be added to resist the potential impact of what

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previously occurred from occurring again.

ELMER L. MEYERS HIGH SCHOOL  
WILKES-BARRE, PENNSYLVANIA

JULY 2007



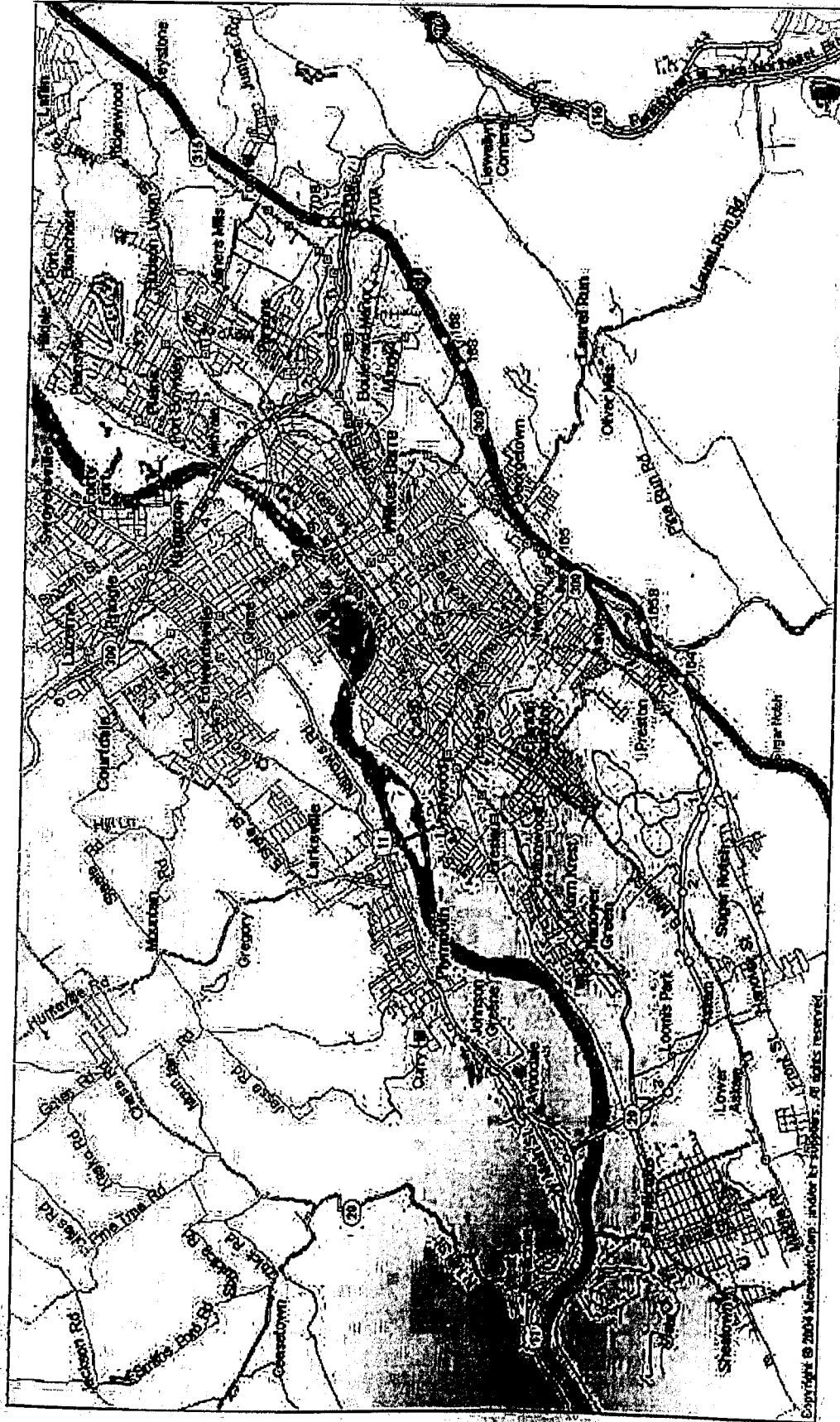
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2014 Note:

Illegible Plates and Pictures in the copy of the original report received for this study not shown

**APPENDIX A**

**Plates**



**MEYERS HIGH SCHOOL  
WILKES-BARRE, PENNSYLVANIA**



**GEO-SCIENCE ENGINEERING CO., INC.**

1252 Mid Valley Drive • Jessup, Pennsylvania 18434

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SHEET NO.

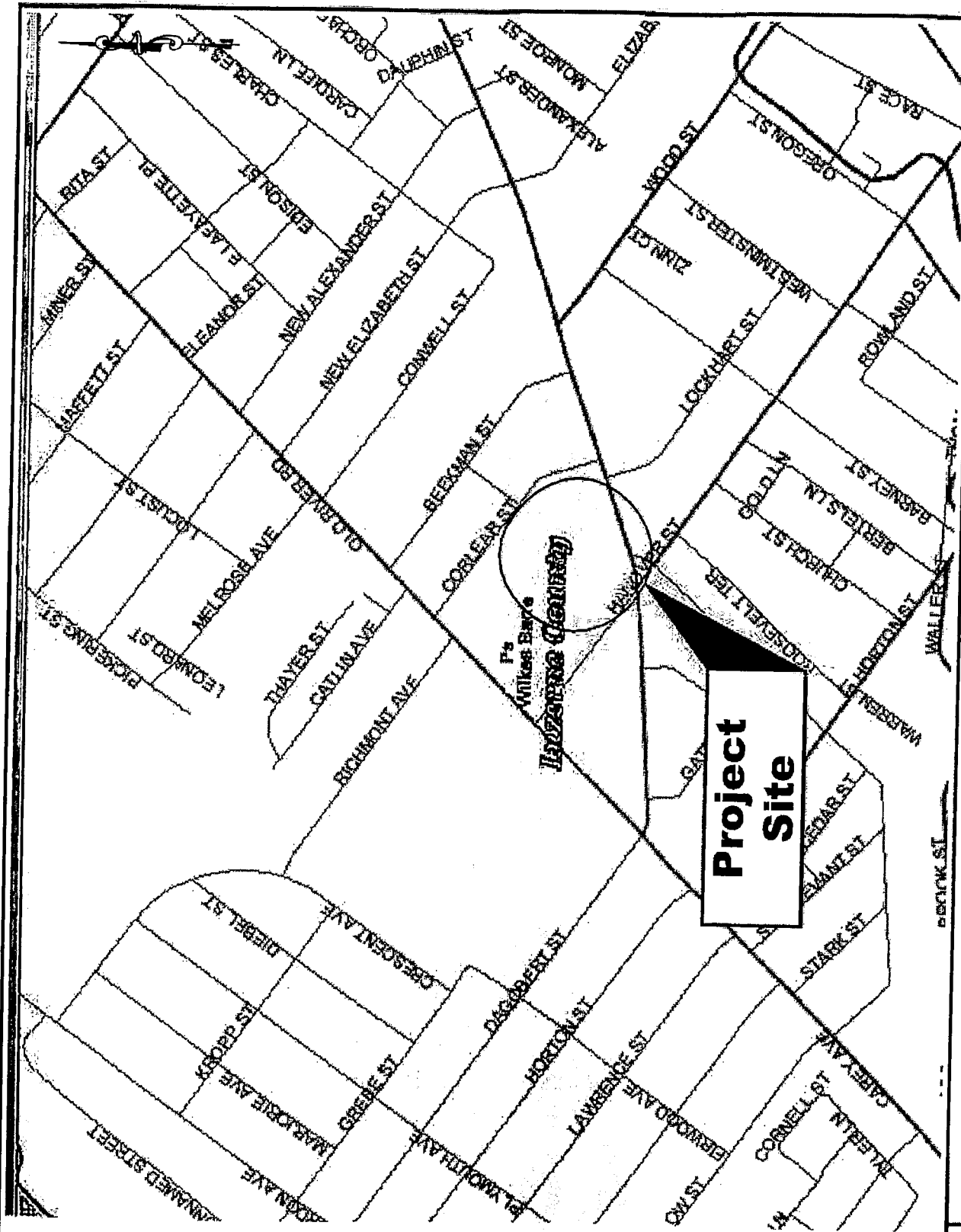
A-1

Soil Descriptions:

Ps - POPE SOILS

Uf - URBAN LAND,  
RARELY FLOODED

Ag - ALLUVIAL LAND



Soils Map

Scale: NTS

Wilkes-Barre Area School District  
341 Carey Avenue  
City of Wilkes-Barre, Luzerne County, Pennsylvania



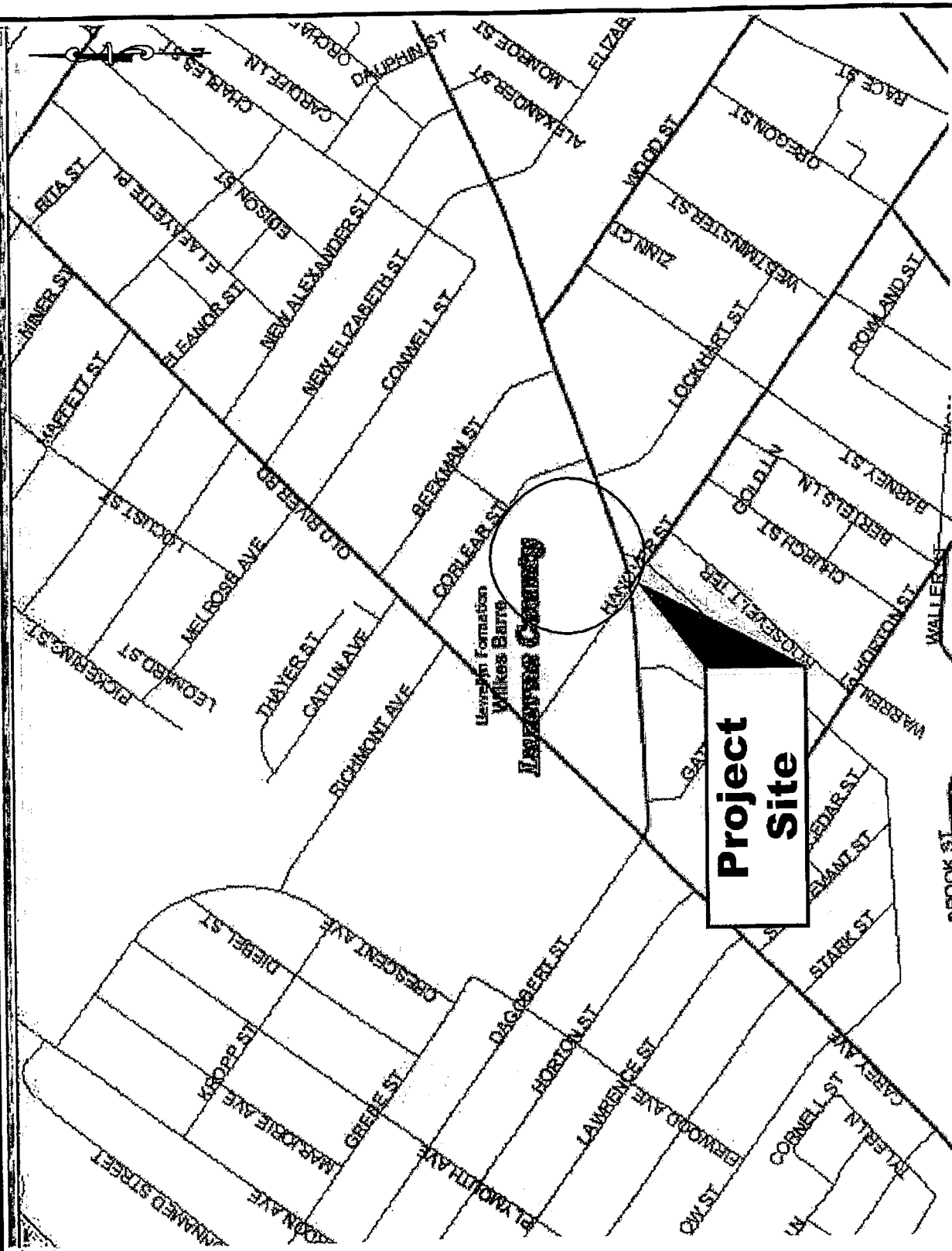
Geo-Science Engineering Co., Inc.  
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Phone (570) 489-8717 Fax (570) 489-8714

Sheet No.

**A-4**

**Geologic Descriptions:**

**Llewellyn Formation:**  
 Pennsylvanian Sandstone,  
 Siltstone, shale,  
 conglomerate, and coal



Sheet No.  
**A-5**

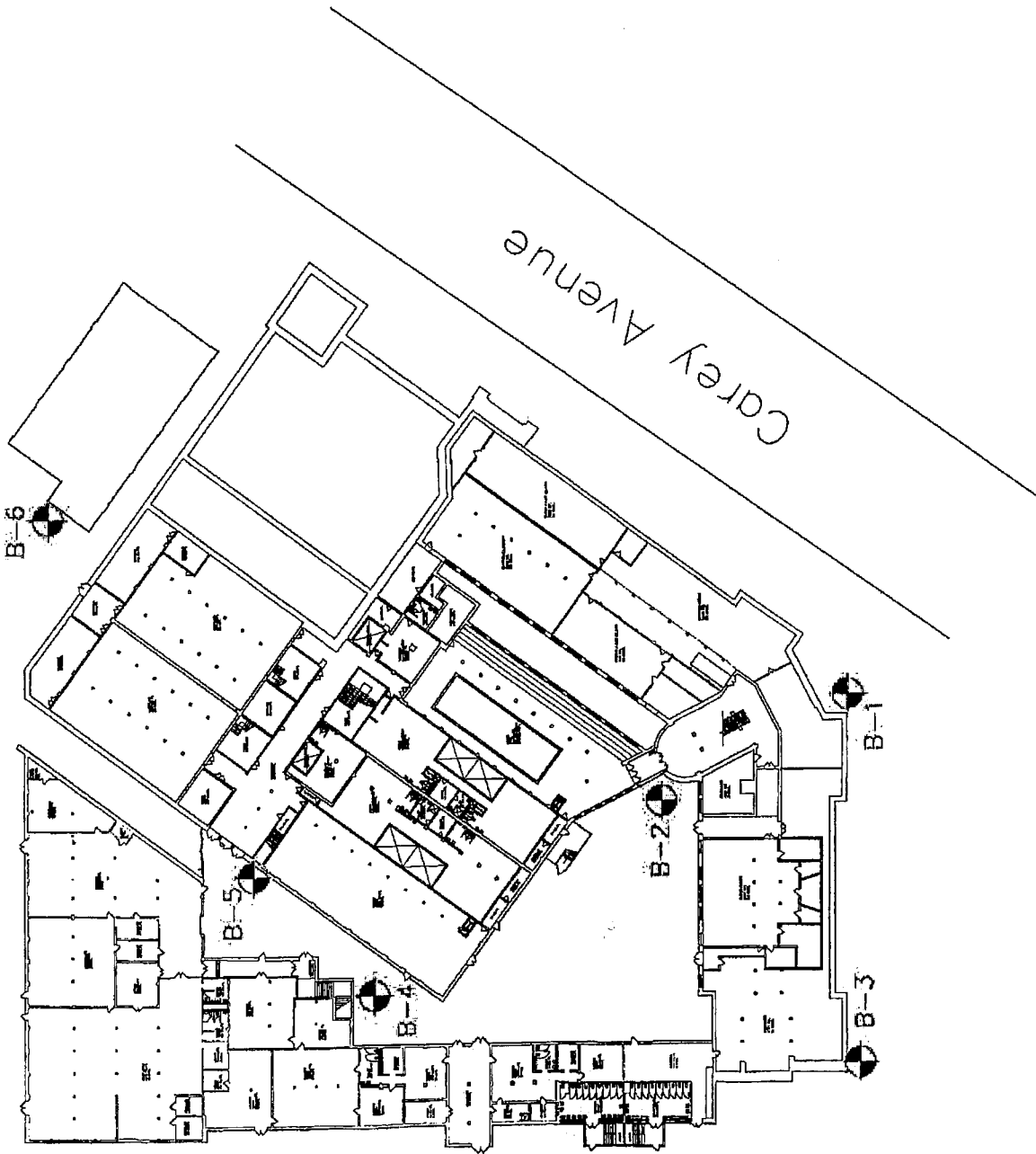
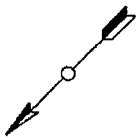
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 Jessup, PA 18434  
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Wilkes-Barre Area School District  
 341 Carey Avenue  
 City of Wilkes-Barre, Luzerne County, Pennsylvania

Geologic Map

Scale: NTS



Test Boring Location  
Plan

Scale: NTS

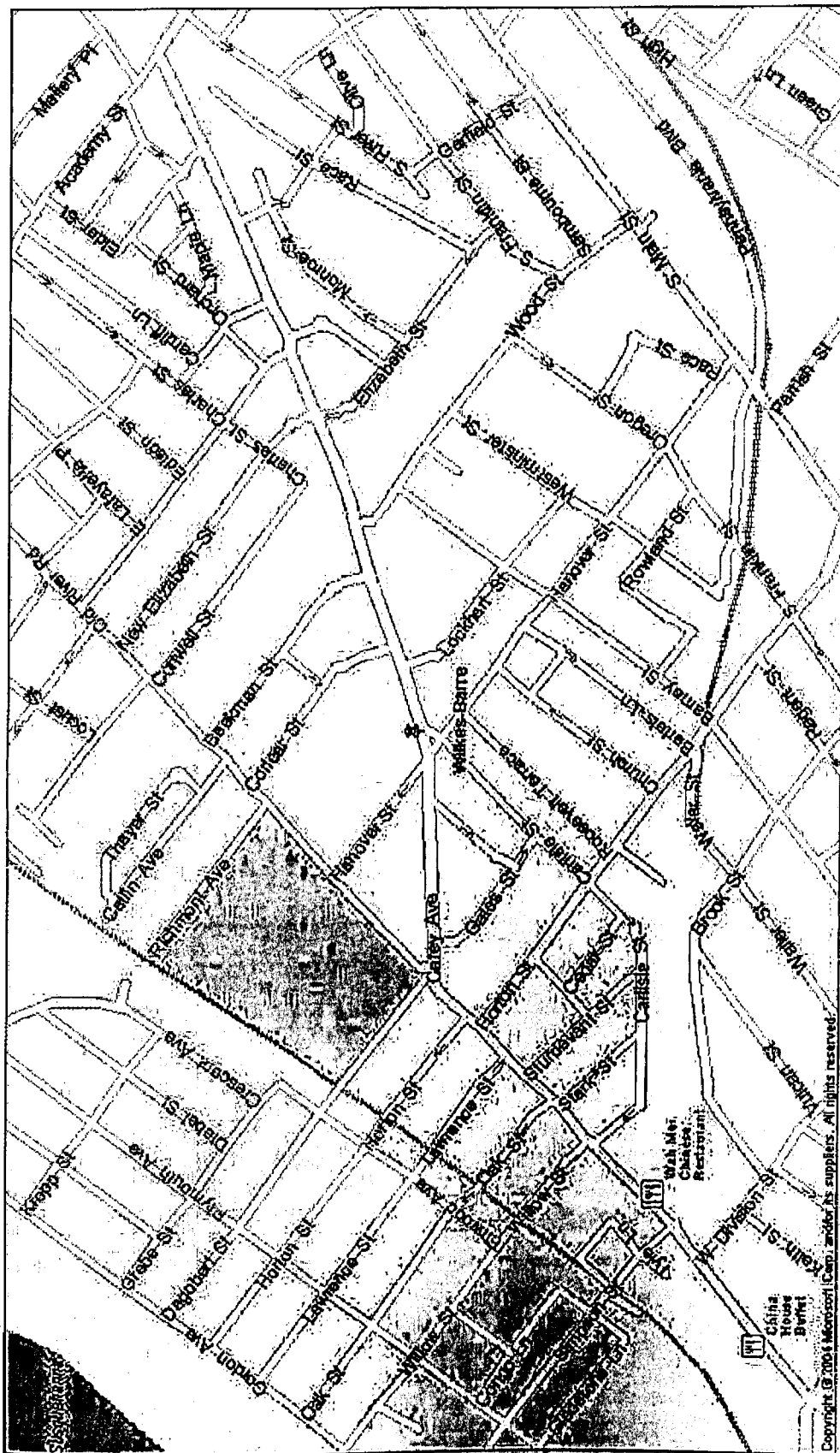
Wilkes-Barre Area School District  
341 Carey Avenue  
City of Wilkes-Barre, Luzerne County, Pennsylvania



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Sheet No.  
**A-6**





**MEYERS HIGH SCHOOL  
WILKES-BARRE, PENNSYLVANIA**

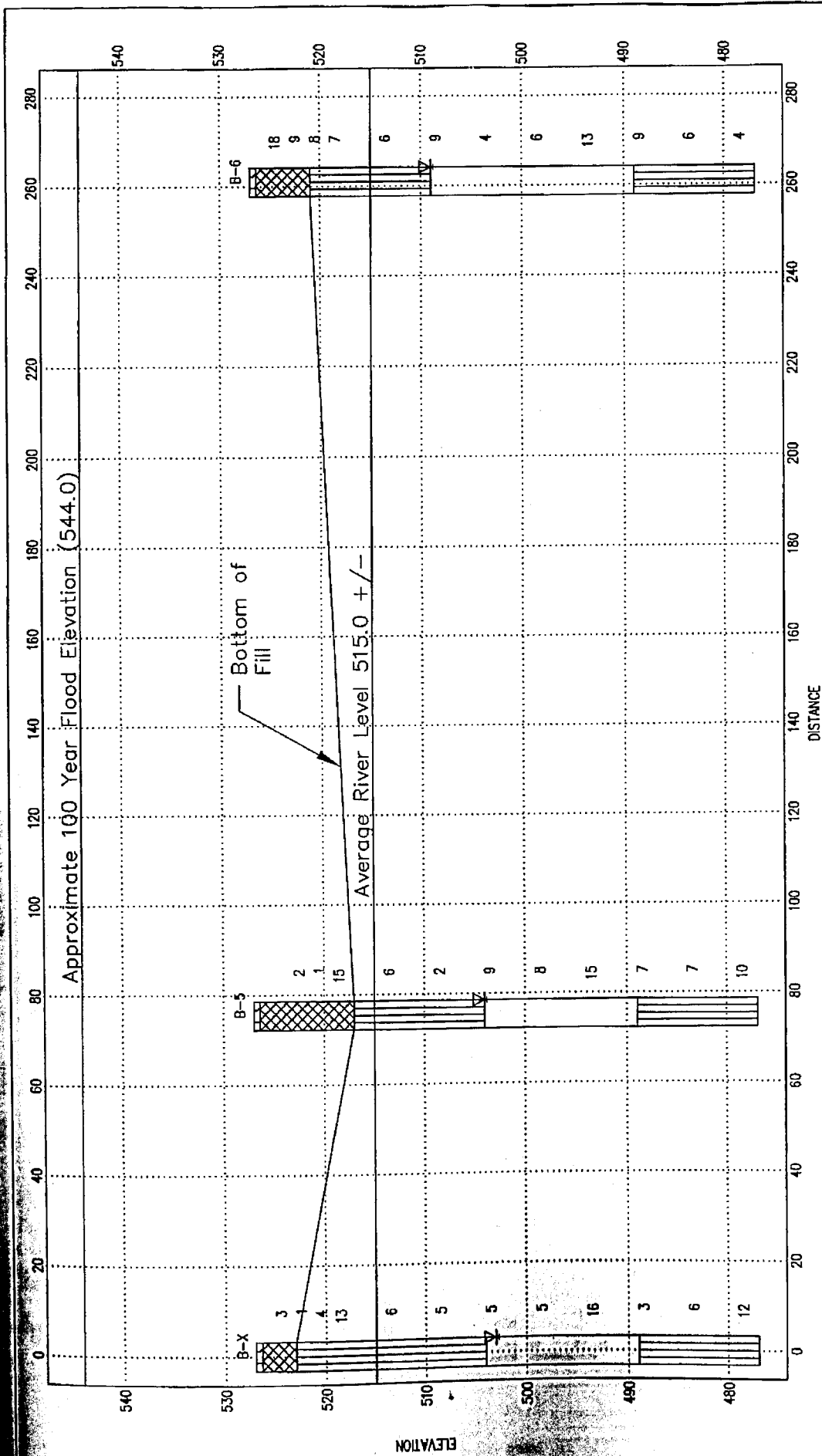


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**SHEET NO.**

**A-2**




**Graphics Legend**

- Asphalt
- Silt (ML)
- Fill (made ground)
- Silt with Sand (ML)
- Poorly-graded Sand with Gravel (SP)

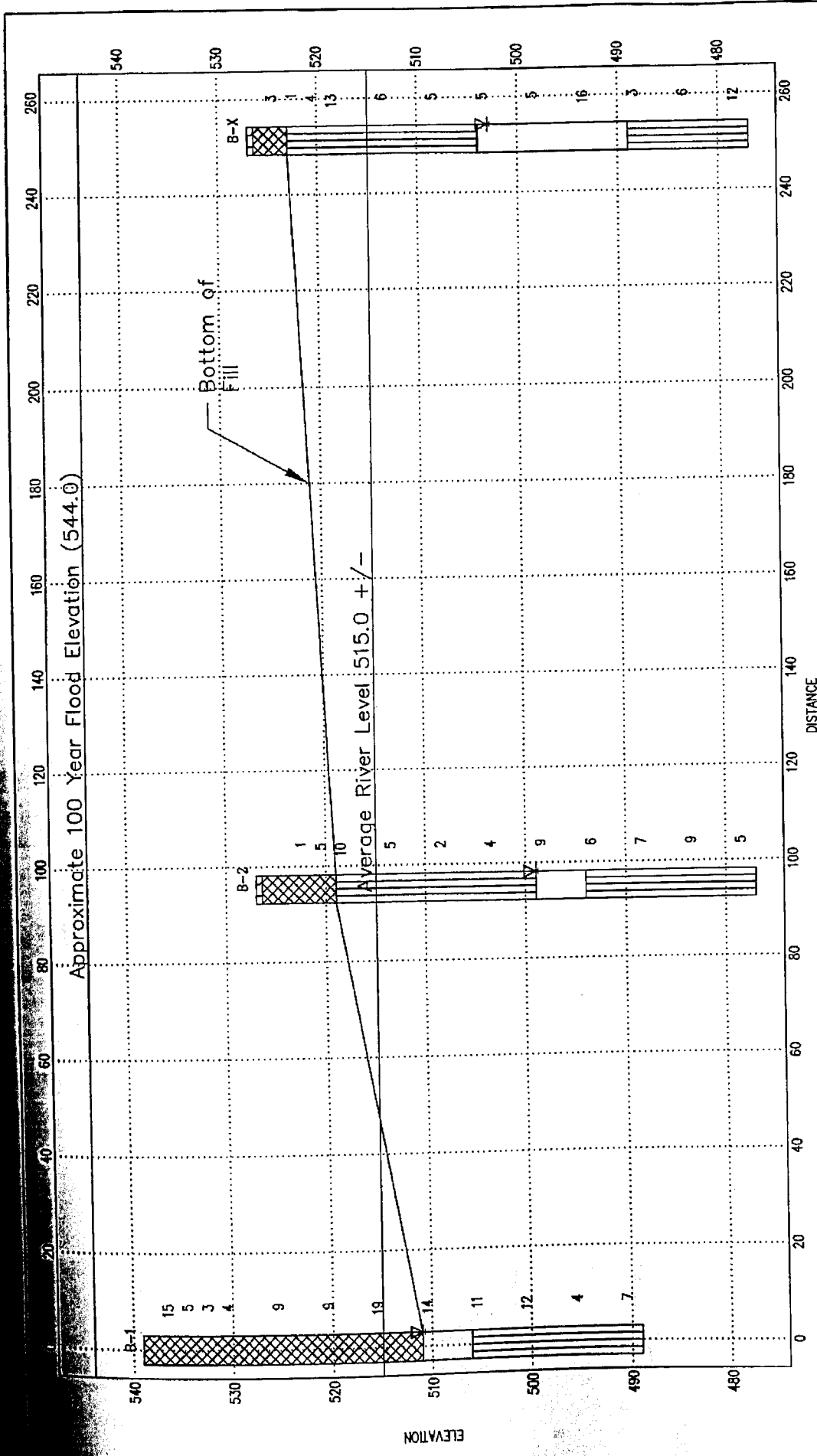
Profiles are intended as best guess representations of conditions between test boring locations and are intended to assist the Engineer in his/her interpretation of the subsurface conditions. The contractor should not rely on these profiles for estimating quantities.

SUBSURFACE PROFILE  
SECTION: a

Wilkes-Barre Area School District - 341 Carey Ave.  
City of Wilkes-Barre, Luzerne County, Pennsylvania

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Consulting Geotechnical Engineers  
1252 Mid-Valley Drive  
Jessup, PA 18434 (570) 489-8717

**SHEET NO. A-7**



**Graphics Legend**

- Fill (made ground)
- Poorly-graded Sand with Gravel (SP)
- Silty Sand (ML)
- Silty with Sand (ML)
- Asphalt

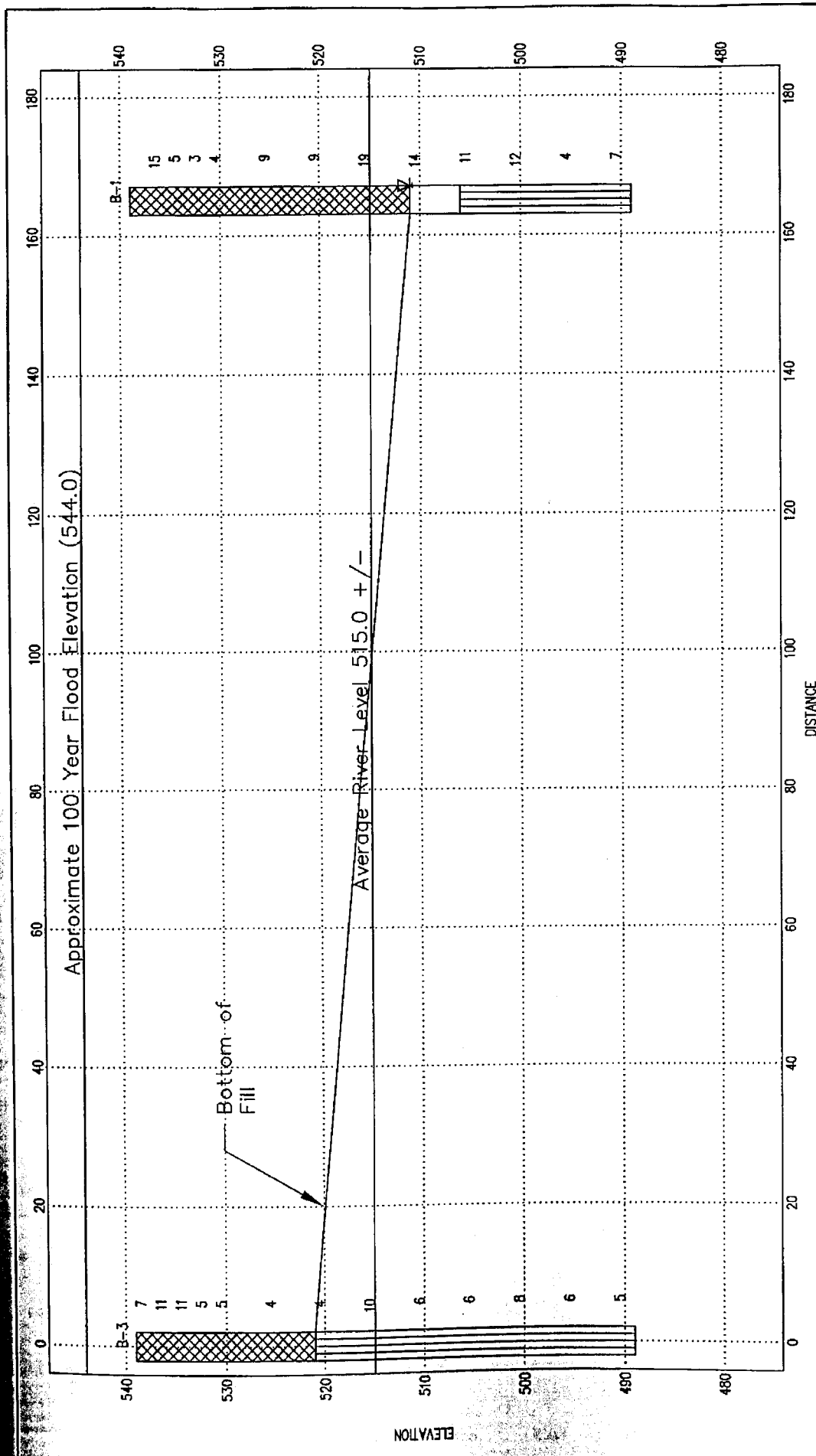
**SUBSURFACE PROFILE**  
SECTION: b

Wilkes-Barre Area School District - 341 Carey Ave.  
City of Wilkes-Barre, Luzerne County, Pennsylvania

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SHEET NO. **A-8**

Profiles are intended as best guess representations of conditions between test boring locations and are intended to assist the Engineer in his/her interpretation of the subsurface conditions. The contractor should not rely on these profiles for estimating quantities.



**Graphics Legend**

- Fill (made ground)
- Poorly-graded Sand with Gravel (SP)
- Silt (ML)
- Silt with Sand (ML)

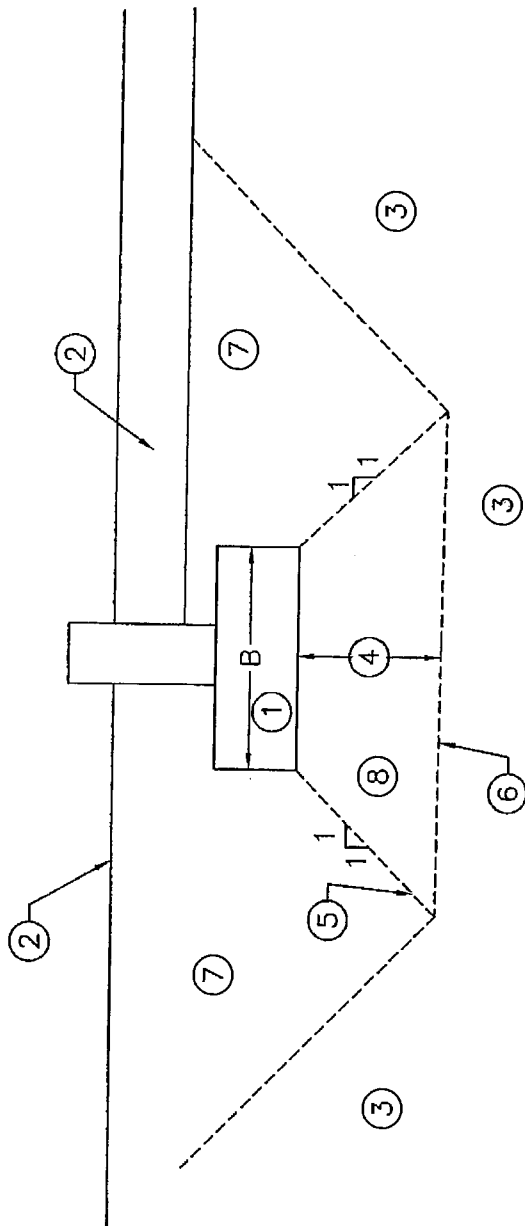
**SUBSURFACE PROFILE**  
SECTION: c

Wilkes-Barre Area School District - 341 Carey Ave.  
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Consulting Geotechnical Engineers  
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SHEET NO. **A-9**

Profiles are intended as best guess representations of conditions between test boring locations and are intended to assist the Engineer in his/her interpretation of the subsurface conditions. The contractor should not rely on these profiles for estimating quantities.



- |   |   |
|---|---|
| ① Footing Width                         | ⑤ Proportioning Line for Undercutting Limits, Not A Construction Line |
| ② Floor Slab And Base or Exterior Grade | ⑥ Base or Undercut Compact Existing Grade (95% ASTM D1557)            |
| ③ Existing Site Materials               | ⑦ Load Bearing Fill Compacted to 95% ASTM D1557                       |
| ④ Undercut Depth                        | ⑧ Select DOT Structural Subbase Fill. Compacted to 97% ASTM D 1557    |

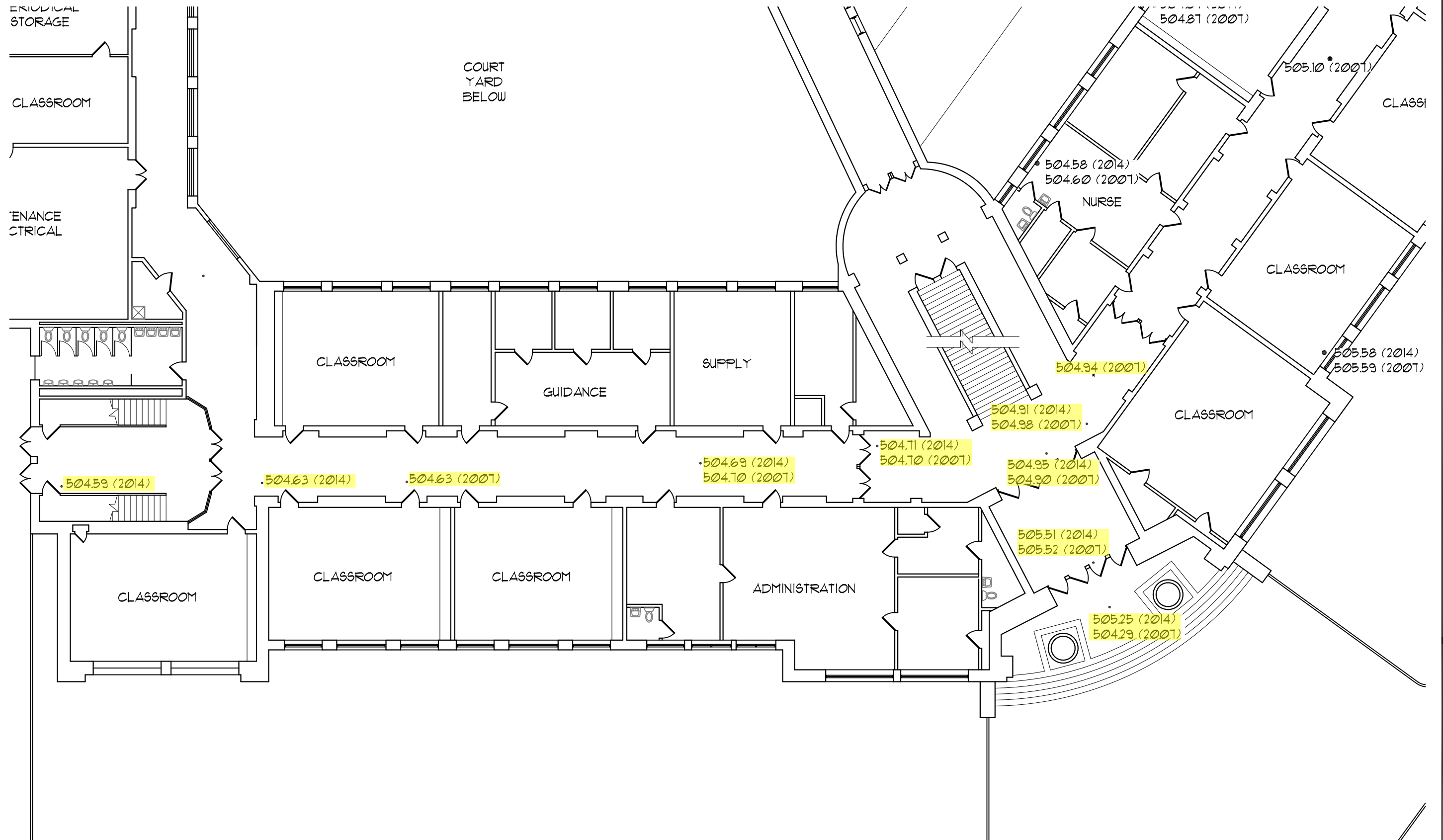
## Foundation Preparation



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**APPENDIX F -  
MAIN BUILDING  
FLOOR ELEVATION SURVEY  
COMPARISON**



Main Building - West Wing: First Floor Elevation Survey



# **MEYERS HIGH SCHOOL SURVEY ELEVATION REPORT**

**PREPARED BY**

**BORTON-LAWSON**

Borton-Lawson Survey was contracted to take elevations readings on each floor of Meyers High School and compare them with elevations taken by Pasonick Engineering in 2007. The purpose was to determine if there was any noticeable settlement of the structure within five (5) years. Borton Lawson was provided a copy of the Pasonick survey that showed the location of their elevations. The Pasonick Survey also showed the location of various “control” marks that they set on each floor of the school. These marks were small tacks or an “X” made on the floor with an indelible marker. Approximately ninety (90) of these marks were recovered. Borton-Lawson also checked elevations referred to as “spots”. These locations were reestablished by taking measurements from the walls in various rooms to relocate the spots that were taken by Pasonick. Elevations were taken at approximately 300 of the Pasonick “spot” locations from the basement to the third floor.

A review of the Pasonick survey that was provided to Borton-Lawson indicates that the elevations were most likely taken using a total station. This type of instrument uses electronic means to capture the data. The total station measures a horizontal distance and vertical angle to a reflector rod. An internal processor then applies a trigonometric function to the measurements and calculates the elevation at a specific location. This method will produce results with an accuracy of approximately 0.02 to 0.03 of a foot (1/4” to 3/8”) depending upon the calibration of the equipment and the accuracy to which the operator can measure both the height of the instrument and the reflector rod.

Borton-Lawson used differential leveling to record our elevations. This method requires the use of an automatic level and rod. This method requires the surveyor to take direct elevation readings on the level rod through the automatic level. This method will produce accuracy to 0.01 of a foot (1/8”) depending upon the calibration of the equipment. Borton-Lawson calibrated our equipment prior to beginning this project. To eliminate operator error in rod readings, two readings were taken at each location and the elevations compared for accuracy.

The Pasonick and Borton-Lawson elevations were both entered into an Excel spreadsheet and compared for accuracy. The differences in elevations found between the two surveys were averaged for each floor surveyed. The following results are for each floor surveyed and represent the average difference in elevation that Borton-Lawson found from the Pasonick Survey.

## **AVERAGE DIFFERENCE ELEVATION**

Control Marks (Basement) - 0.02' or 1/4"

Control Marks (First Floor) - 0.02' or 1/4"

Control Marks (Second Floor) - 0.02' or 1/4"

Control Marks (Third Floor) – 0.05' or 5/8"

Spots & Control Marks (Basement) – 0.02' or 1/4"

Spots & Control Marks (First Floor) – 0.02' or 1/4"

Spots & Control Marks (Second Floor) – 0.02' or 1/4"

Spots & Control Marks (Third Floor) – 0.06' or 3/4"

## **CONCLUSION**

After reviewing all the data Borton-Lawson saw no significant difference in elevation on the basement, first or second floor as shown on the 2007 Pasonick Survey. The differences that we saw in the data can be contributed to the two methods used to acquire the elevations. The greater difference that was found on the third floor may be contributed to difference that Borton-Lawson found in the Pasonick Control Marks on that floor. Borton-Lawson does not know the exact procedure that Pasonick used to transfer elevations from floor to floor but does realize that inaccuracies can occur when transferring elevations to different floors. Borton-Lawson transferred our elevations from floor to floor using our leveling procedure. Each level run to the individual floors was looped back to the Pasonick Benchmark located in the sidewalk along Carey Avenue. The level loop allows us to mathematically check our elevations by closing on the Benchmark. Each level loop ran to the individual floors resulted in errors less than 0.01' or 1/8". Borton-Lawson did not consult with Pasonick regarding their survey and only used the data that was provided in the original 2007 report.