Seismic Risk Assessment, Conceptual Seismic Retrofit, Budgetary Structural Construction Estimate and Additional Liquefaction Analysis Report

CONNEL F. SMITH BUILDING/IL-7-4
1101 Ohio St.
Cairo, Illinois

OCTOBER 19, 2022
Prepared for:
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1.0 INTRODUCTION

1.1 Background

Fendrich Engineering, Inc. (FEI) was contracted by Eggemeyer Associates Architects (EAA) in July of 2021 to perform a structural assessment of the Smith Building due to concerns raised over observed continued concrete deterioration of the exposed concrete building structure. During this condition evaluation and after reviewing the original construction documents FEI recommended that a brief, preliminary seismic evaluation be included in the Building Structural Condition Report (August 2021). The following is a portion the preliminary seismic recommendation:

> 'Recommendation – It is highly likely if a thorough seismic analysis of the existing building structure were completed, a significant seismic retrofit upgrade would be required. For this Report, the seismic retrofit would consist of strategically placed reinforced concrete shear walls throughout the building.'

Due to seismic concerns raised in the Building Structural Condition Report, preliminary design began for the seismic retrofit and concrete repairs of Smith Building in March 2022. FEI contacted Hohbach-Lewin, Inc. (H-L) Structural Engineers to perform the building seismic analysis and seismic retrofit design for the project. Preliminary analysis of the building was conducted by H-L in April 2022, and H-L found a number of building deficiencies. H-L also expressed concern regarding site liquefaction due to presence of sand layers indicated in the original borings conducted in 1965.

Due to the site liquefaction concerns EAA contacted Shannon & Wilson, Inc. (S&W) to perform a preliminary liquefaction analysis of the Smith Site (July 2022). The analysis was based on the original borings and an earthquake with a seismic hazard consisting of a 10% probability of exceedance in 50 years (475-year return period earthquake – Freddie Mac earthquake insurance criteria). The analysis showed that the silt and sand layers at the site are susceptible to liquefaction with estimated significant earthquake induced settlements at the southeast corner of the building (original boring B-1) of 15 to 25 inches. Also, the liquefaction report concluded that down-drag forces, induced by the settlement, will effectively eliminate the pile capacity. The liquefaction concerns led to this Report.

1.2 Building Description

The Smith Building was constructed in 1967. The building is eight (8) stories and does not have a basement. The building gravity structural system is a reinforced concrete frame consisting of concrete floor and roof slabs, concrete floor and roof beams and concrete columns. The gravity system is also the lateral load resisting system in transverse and longitudinal building directions with the exterior envelope and interior unreinforced masonry walls contributing. The building is founded on piles. It is unclear from the drawings what type of piles were used as the original drawings show the piles to be 14" diameter concrete filled metal casings with the word 'metal' crossed out and replaced with 'wood' handwritten. Note, the condition of the existing piles is unknown. The concrete columns at the building perimeter are fully exposed to the elements at the east elevation and partially exposed at the west elevation. Additionally, at the east elevation, the transverse and perimeter beams and the floor slabs are exposed to the elements due to a walkway at each floor level. At the north and east ends of the building are attached concrete stair towers that are exposed to the elements.

1.3 Scope of Work

1.31 Seismic

1) Evaluate the existing structure per ASCE 41 – 17 utilizing ETABS and the response spectrum method of the linear dynamic procedure under a seismic hazard with a 10% probability of exceedance in 50 years (475-year return period earthquake).
2) Conduct a building stability assessment. We have assumed this should be interpreted as equivalent to an ASCE 41 collapse prevention performance objective with respect to the seismic hazard noted above.
3) Provide a conceptual seismic retrofit to meet the seismic performance objective typically required by loan providers (e.g. Freddie Mac Earthquake Insurance Requirements), which is anticipated to be a Scenario Expected Loss of less than 20% under the seismic hazard noted above.

1.32 Liquefaction

Provide additional liquefaction analysis for the Smith Building at different earthquake accelerations to indicate at what maximum earthquake acceleration liquefaction is not expected to be triggered.

2.0 SUMMARY OF RESULTS

2.1 Seismic – Hohbach-Lewin, Inc. (See Appendix A)

1) The Smith building is laterally braced by concrete moment frames and unreinforced masonry infill walls. Since the infill masonry walls are significantly stiffer that the concrete frames, the building's initial seismic resistance and resistance to lower levels of lateral force, will be provided by the infill walls. In the event of larger seismic forces these infill walls are expected to fail, and the concrete moment frames will function as the lateral force resisting system of the building.
2) The longitudinal direction concrete moment frames consist of the slab bands and columns oriented in their weak direction. They are relatively flexible and weak and are anticipated to yield at an equivalent seismic force level as the infill walls. Their demand capacity ratio with respect to the scenario seismic event is approximately 5.
3) The transverse direction concrete moment frames have beams and columns and are considered moderately ductile and approximately capable of resisting the scenario seismic event without collapse.
4) The building is supported on 55 feet deep timber piles that partially gain their support from liquefiable layers of soil. The ground motion that may induce liquefaction of these soils and thus loss of foundation support is similar to the ground motion that is expected to cause failure of the building in the longitudinal direction.
5) The model suggests a earthquake spectral response of 0.15g will likely cause major structural damage or collapse of the building. An earthquake ground motion with a 29.2% probability of exceedance in 50 years ground motion was determined to cause the 0.15g spectral response. This is an equivalent return period of 145 years.
6) The analysis results outlined above have also been utilized to gain an understanding of the building's seismic risk in its existing condition. We have estimated that the building has a 0.3% annual risk of collapse. Perhaps a meaningful comparison for this number would be to compare it to a typical existing building that would likely pass the Freddie Mac stability criterion to not collapse in the earthquake ground motion prescribed in the current edition of the International Building Code. We have estimated that this hypothetical building has a 0.03% annual risk of collapse, or

approximately 1% over the estimated life 30-year life of that building. The Smith Building would accumulate an equivalent amount of seismic risk in a little over 3 years

7) The proposed conceptual seismic retrofit comprises the removal of all unreinforced masonry infill walls and the provision of a new lateral system with concrete shear wall buttresses with deep pile foundations, collector elements within the building and concrete grade beams beneath the building to support the building in the event of liquefaction.

2.2 Liquefaction – Shannon & Wilson, Inc. (See Appendix B)

A preliminary soil liquefaction evaluation was performed based on the results provided to us of Borings B-1 and B-4 at the Smith Building site. The evaluations utilized a Site Class E and an adjusted PGA of 0.59g, for a 10% probability of exceedance in 50 years (return period of 475 years). The results indicate that the soil will liquefy resulting in settlements of the ground surface of up to 25 inches, and that the settlement could be differential to nearly an equal amount across the building footprint. An additional liquefaction analysis was performed to find the approximate threshold ground motion that first induces liquefaction at the two borings. The results indicated that an earthquake magnitude 7.5 that produces a PGA of 0.18g and 0.31g for the soil profiles in Borings B-1 and B-4, respectively, has the possibility of inducing liquefaction. These settlements will induce downdrag loads that overstress the pile supporting the building. The following assumptions/limitations apply to this analysis:

1. The analysis was performed with subsurface data collected during the initial construction of the building in the 1960's. Those borings were extended to depths ranging from 50 to 91 feet below the ground surface. The soil borings do not indicate hammer efficiency, drilling method, Atterberg limits or grain size distribution. Assumptions had to be made regarding the hammer efficiency, drilling and sampling equipment and method, and engineering parameters.

2. We assumed a depth to groundwater of 20 to 25 feet below the ground surface.

3. ASCE 7-16 Chapter 20 indicates that soils that are susceptible to liquefaction shall be classified as Site Class F and a site response analysis shall be performed to determine spectral accelerations for seismic design.

4. The analysis of the piles assumes the existing piles are fully intact and that the as-built records for pile tip elevations are accurate.

Changes to these assumptions will change the results presented in our reports. Please refer to our report titled *Preliminary Liquefaction Analysis, Alexander County Housing Authority Building, Cairo, Illinois* dated July 27, 2022 for details about our assumptions.

3.0 PROPOSED CONCEPTUAL SEISMIC RETROFIT

The proposed conceptual seismic retrofit comprises the removal of all unreinforced masonry infill walls and the provision of a new lateral system with concrete shear wall buttresses with deep pile foundations, collector elements within the building and concrete grade beams beneath the building to support the building in the event of liquefaction.

See Appendix C – Conceptual Seismic Retrofit Drawings

S1 – Foundation, Typical Floor and Roof Framing Plans
S2 – Building Sections

4.0 RETROFIT STRUCTURAL BUDGETARY COST ESTIMATE

Structural Budgetary Cost Estimate ____ **$7,700,000**
See Appendix D for estimate breakdown
Discussion – A local contractor was used to determine the unit costs for the major structural items including the piles, seismic grade beam, buttresses, and floor slabs. The cost of the piling is significant and is due to the length of the piles and lack of information due to limited borings.

<u>Note, the costs presented in this report represent ONLY the structural costs and do not include any other costs including architectural, MEP and A/E fees.</u>

5.0 EXECUTIVE SUMMARY OF RESULTS

5.1 Building Analysis

The Smith Building has a high probability of collapse in the event of a significant earthquake. Building analysis indicates that the ground motion that may induce site liquefaction is similar to the ground motion that is expected to cause building failure/collapse in the building's longitudinal direction.

An example earthquake that could cause building failure/collapse is as follows:

Earthquake Magnitude	4.9	Moment Magnitude Scale (USGS)
Site Modified		
Peak Ground Acceleration (PGA)	0.23g	
Distance from Site	9.3	miles

A building seismic retrofit/upgrade is recommended as shown in Appendix C.

The building seismic retrofit/upgrade budgetary cost estimate is $7,700,000.

5.2 Site Liquefaction Analysis

The results indicate that the Smith Building site soil will liquefy at the 475 years return period ground motion resulting in settlements of the ground surface of up to 25 inches, and that the settlement could be differential to nearly an equal amount across the building footprint. These settlements will induce downdrag loads that overstress the piles supporting the building.

The approximate threshold accelerations with a 7.5 earthquake magnitude that first induces liquefaction at the two borings are as follows:
Boring B-1 (southeast building corner) - 0.18g
Boring B-4 (northwest building corner) – 0.31g

APPENDIX A

Hohbach-Lewin, Inc. Report

Seismic Risk Assessment and Conceptual Retrofit Report

Smith Building
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October 19, 2022
Project No: 15973.2

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1.0 INTRODUCTION

1.1 General

The objective of this report is to present the results of a limited approximate structural analysis based on ASCE 41 guidelines with respect to quantifying the seismic risk and developing a potential seismic retrofit scheme for the existing Smith Building. It is an occupied residential building located at 1101 Ohio Street, Cairo, IL 62914. Refer to Figure 1-1 for vicinity map of the site.

The Smith Building is an existing eight-story concrete mid-rise tower originally constructed in 1967. The existing structure measures approximately 198' – 10" by 35' – 2" in horizontal plan dimensions with 8' – 6" typical story heights and a 12' – 0" bottom story height. The gravity system consists of 6" mild reinforced one-way concrete slabs supported by concrete moment frame beams and columns spanning and oriented in the short direction of the building. The gravity system moment frames also participate as the lateral system of the structure. However, the exterior envelope and interior unit demising walls of the building consist of infill brick masonry walls that may also participate as part of the lateral system. The deep foundation system consists of reinforced concrete pile caps supported by timber piles extending to a depth of 55 feet and interconnected with reinforced concrete grade beam foundation ties. The first floor is a 6" reinforced concrete slab on grade.

1.2 Scope of Work

The seismic evaluation of the subject building consisted of the following steps:

1) Reviewed available design drawings and geotechnical information for the original construction and subsequent modifications.

2) Evaluated the existing structure per ASCE 41 – 17 utilizing ETABS and the response spectrum method of the linear dynamic procedure under a seismic hazard with a 10% probability of exceedance in 50 years (475 year return period earthquake).

3) Conducted a building stability assessment. We have assumed this should be interpreted as equivalent to an ASCE 41 collapse prevention performance objective with respect to the seismic hazard noted above.

4) Assisted the geotechnical engineer as requested with respect to their liquefaction analysis.

5) Provided a conceptual seismic retrofit to meet the seismic performance objective typically required by loan providers (e.g. Freddie Mac Earthquake Insurance Requirements), which is anticipated to be a Scenario Expected Loss of less than 20% under the seismic hazard noted above. We have also incorporated information from the geotechnical engineer's liquefaction analysis.

1.3 Review of Documentation

The following documentation has been reviewed:

1) Original Construction Documents – (33) sheets total, by Lee Potter Smith & Associates Architects, dated December 1965.

2) Original Structural Construction Documents – (13) sheets total, by Building Products Company, dated July 1967.

3) "Building Structural Condition Assessment" letter – (13) pages total, by Fendrich Engineering, Inc., dated August 16, 2021.

4) “Preliminary Liquefaction Analysis, Alexander County Housing Authority Building, Cairo, Illinois” Geotechnical Report – (25) pages total, by Shannon & Wilson, Inc., dated July 27, 2022.

5) “Geotechnical Earthquake Engineering Services, Additional Liquefaction Analysis, Alexander County Housing Authority Building, Cairo, Illinois” Geotechnical Letter – (9) pages total, by Shannon & Wilson, Inc., dated September 19, 2022.

1.4 Limitations

Services associated with the preparation of this report were performed by Hohbach-Lewin in a manner consistent with the level of care and skill ordinarily exercised by members of the structural engineering profession currently practicing under similar conditions. No other warranty, expressed or implied, is made. The report is based on a limited review of the building and was prepared solely for the use of Fendrich Engineering, Inc. and its client. The actual structural characteristics of the building could not be fully assessed since limited calculations were performed. In addition, architectural finishes conceal many features of the structure throughout. Information not available under these conditions to Hohbach-Lewin and hidden construction quality conditions could alter the structural characteristics of the building from what is inferred in this report.

Note also that the conceptual seismic retrofit provided is for the purpose of developing an approximate budget for the construction of a seismic retrofit and is far from complete..

1.5 Reference Documents

1) “Seismic Evaluation and Retrofit of Existing Buildings”, American Society of Civil Engineers, 2017 (ASCE 41 – 17 Standard).
2) Freddie Mac Multifamily Seller/Service Guide, Chapter 64, Seismic Risk Assessment Requirements

Figure 1-1: Vicinity Map

2.0 BUILDING DESCRIPTION

2.1 General

The Smith Building is an existing eight-story concrete mid-rise tower originally constructed in 1967. The existing structure measures approximately 198' – 10" by 35' – 2" in horizontal plan dimensions with 8' – 6" typical story heights and a 12' – 0" bottom story height. The gravity system consists of 6" mild reinforced one-way concrete slabs supported by concrete moment frame beams and columns spanning and oriented in the short direction of the building. The gravity system moment frames also participate as the lateral system of the structure. However, the exterior envelope and interior unit demising walls of the building consist of infill brick masonry walls that may also participate as part of the lateral system. The deep foundation system consists of reinforced concrete pile caps supported by timber piles extending to a depth of 55 feet and interconnected with reinforced concrete grade beam foundation ties. The first floor is a 6" reinforced concrete slab on grade. Exterior elevations of the Smith Building are shown in Figures 2 – 1 through 4.

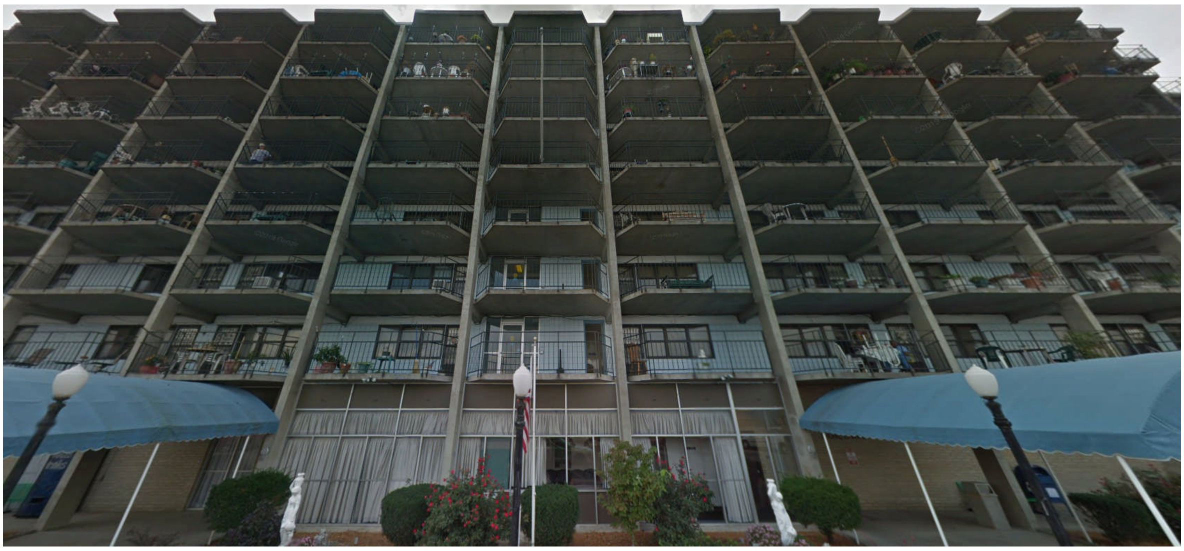

Figure 2 – 1: Smith Building North Elevation

Figure 2 – 2: Smith Building East Elevation

Figure 2 – 3: Smith Building South Elevation

Figure 2 – 4: Smith Building East Elevation

2.2 Site Visit

A site visit, including condition assessment and visual observation, has been performed by Fendrich Engineering, Inc. At the time of this site visit, the overall existing building condition was determined to be in fair to poor condition. Observed existing structural conditions contributing to the overall condition assessment include concrete cracking, spalling, and exposed reinforcement at several column, beam, and slab locations throughout the structure and are further described within the referenced Fendrich Engineering, Inc.'s report.

Based on these visual observations, the existing building configuration is in accordance with the structural documents that were reviewed.

No structural testing was conducted in conjunction with preparation of this report.

There was no evidence the building has been seismically retrofitted.

2.3 Gravity System Description

The gravity system consists of one-way mild reinforced concrete slabs that span between regularly spaced bays of concrete moment frame columns and beams oriented and spanning in the short direction of the building. The concrete moment frame column and beam sections are both rectangular. Exterior columns have unreinforced hollow block masonry and unreinforced brick masonry infill walls that span between them. Interior columns have unreinforced block masonry demising infill walls separating residential units.

2.4 Lateral System Description

The gravity system also acts as the primary lateral force resisting system for this structure. The floor and roof diaphragms consist of mild reinforced concrete slabs. The lateral system in the short direction consists of the regularly spaced concrete moment frames with unreinforced hollow block masonry infill walls. The lateral system in the long direction effectively consists of unreinforced hollow block masonry walls.

3.0 SITE SOILS AND SEISMICITY

3.1 Geotechnical Conditions

The geotechnical report obtained soil information from the original boring logs shown in the drawings and found that the subsurface conditions generally consisted of 9 feet of fill material, 7 to 20 feet of clay and silty clay, 6 to 14 feet of silt loam and sand that extended to the borings termination depths. Groundwater was encountered in the borings between 20 and 25 feet below the ground surface. The geotechnical report also notes that the site has liquefiable soils and should be classified as Site Class F and thus a site-specific geotechnical investigation should be accomplished, however that analysis was not authorized in the scope of services.

3.2 Seismicity

For the purposes of this report, it was determined that for this preliminary analysis, code-based ground motions could be provided by the geotechnical engineer rather than a site-specific geotechnical investigation. The disaggregation for 10% probability of exceedance in 50 years indicates a soft rock/firm soil peak ground acceleration (PGA) of 0.47g for Site Class B/C boundary, which when adjusted for Site Class E results in a PGA of 0.59g. The existing structure is within the New Madrid Seismic Zone that has the potential for high seismicity. Potential earthquake-induced geological hazards, as determined by the geotechnical engineer, include strong ground shaking, liquefaction, lateral spreading.

3.3 Liquefaction

Please see the referenced liquefaction reports for a detailed discussion of this topic. Of note is that the peak ground accelerations that are anticipated to yield the structure are similar to those that anticipated to induce liquefaction and that these accelerations are expected to recur more frequently than the 475 year return period scenario seismic event.

4.0 LATERAL ANALYSIS

4.1 Qualitative Seismic Evaluation

Engineering judgment has been used in lieu of calculations, where appropriate, to determine if potential deficiencies are likely present.

The following list of potential deficiencies were identified.

1. NONDUCTILE CONCRETE MOMENT FRAMES: The existing concrete moment frames do not have modern ductile detailing, however they have been determined to be moderately ductile and capable of deforming after yielding.
2. UNBRACED SLENDER GROUND FLOOR CONCRETE MOMENT FRAME COLUMNS: The existing exposed ground floor concrete moment frame columns are slender due to the tall first story, small cross-sectional dimensions, and lack of minor axis bracing. There is a high potential of minor-axis buckling of these columns in the event of an earthquake.
3. EXTERIOR UNREINFORCED HOLLOW BLOCK AND UNREINFORCED BRICK MASONRY WALLS: The existing exterior unreinforced hollow block and unreinforced brick masonry walls lack adequate strength to span out-of-plane in the event of an earthquake. Furthermore, the existing out-of-plane anchorage of these walls is inadequate. Given these two potential deficiencies, brittle response of these walls in the event of a major earthquake is expected to result in falling debris hazard and the blocking of egress pathways.
4. INTERIOR UNREINFORCED HOLLOW BLOCK MASONRY PARTITION WALLS: The existing unreinforced hollow block masonry partition walls, due to their relatively large stiffness, are expected to function as the lateral force resisting system for the building for modest earthquakes however they lack adequate strength to span out-of-plane in the event of a major earthquake. Furthermore, the existing out-of-plane anchorage of these walls is inadequate. Given these two potential deficiencies, brittle response of these walls in the event of a major earthquake is expected to result in falling debris hazard and the blocking of egress pathways.
5. PILE CAPS: The existing pile caps lack adequate connection to the existing piles. These pile caps also have no top reinforcement. Given these deficiencies, the existing pile caps have near negligible tension uplift capacity in the event of an earthquake.
6. PILES: The geotechnical engineer has determined that, in the event of an earthquake, the downdrag loads anticipated along the entire length of the piles located at the northeastern portion of the building will effectively eliminate their capacity.
7. LIQUEFACTION: The geotechnical report has determined that there is significant risk of liquefaction generating differential settlements in the range of 13.5 to 24.5 inches.
8. LATERAL SPREADING: The geotechnical engineer has determined that there is moderate to high potential for lateral spreading (displacement) of soil in the event of an earthquake.

4.2 ETABS Analysis of Existing Building

4.2.1 General

A three-dimensional computer model of the existing building was created to gain a better understanding of the building's expected performance in the scenario seismic event. The unreinforced masonry walls are treated only as dead load and seismic mass, although they are anticipated to contribute somewhat to the buildings lateral strength and bracing of exterior columns. All the concrete elements of the building were considered to contribute to the seismic force resisting system of the building including horizontal concrete diaphragms, beams, and columns. The response spectrum method of the linear dynamic analysis has been performed utilizing ETABS software. The output forces and displacements shown below are obtained from running the model with seismic hazard level with a 10% probability of exceedance in 50 years (475-year return period earthquake). Site class "E" was utilized as a site-specific geotechnical investigation has not been provided yet for the designated site class "F" per geotechnical report. Below are the seismic design criteria as related to our design response spectrum and structural performance objective:

- S_{XS} = (1.111)g
- S_{X1} = (0.863)g
- T_L = 12 seconds
- "Collapse Prevention" Structural Performance Objective

4.2.2 Analysis Major Assumptions

Site class "E" is assumed. However, site class "F" is appropriate and requires a site-specific geotechnical investigation for more accurate seismic design criteria, as described in the geotechnical report.

The exterior egress concrete stairs at the east and west ends of the existing building, and the concrete stair walls that support them, were assumed to participate in a negligible fashion with respect to the overall lateral system. Thus, their seismic mass is incorporated into the model, but are treated as secondary components.

The unreinforced masonry wall lateral system in the longitudinal direction were found to fail at relatively low force level compared to the forces induced by the scenario seismic event and thus were assumed to participate negligibly with respect to the overall lateral system due to their lack of strength and drift capacity. Thus, their seismic mass and dead load are incorporated into the model, but are treated as secondary components.

ETABS stiffness modifiers were modified as follows per ACI 318-19 Table A.8.4:

- Effective flexural stiffness for columns taken as 70% of nominal values
- Effective shear stiffness for columns taken as 40% of nominal values
- Effective flexural stiffness of beams taken as 30% of nominal values
- Effective shear stiffness of beams taken as 40% of nominal values
- Effective axial, flexural, and shear stiffnesses of slabs taken as 25% of nominal values
- Effective in-plane flexural stiffness of walls (retrofit) taken as 35% of nominal values
- Effective in-plane shear stiffness of walls (retrofit) taken as 20% of nominal values
- Effective out-of-plane flexural stiffness of walls (retrofit) taken as 25% of nominal values
- Effective out-of-plane shear stiffness of walls (retrofit) taken as 40% of nominal values

Existing columns, beams, and slabs were modeled as normal weight concrete with an effective (expected) compressive strength of 6,000 psi. Grade 50 mild reinforcement is utilized as the existing construction documents specify this material throughout. New wall elements were modeled assuming normal weight concrete with a compressive strength of 6,000 psi.

Foundation supports are modelled as pinned base, with anticipated liquefaction-induced differential settlements, per the geotechnical report, ranging linearly from 1 inch at the southwest corner of the building to 20 inches at the northeast corner of the building with a maximum of 25 inches at the northeastern corner column location.

4.2.3 ETABS Input Summary

A three-dimensional view and plan view of the model is shown in Figure 4 – 1 and 4 – 2, respectively.

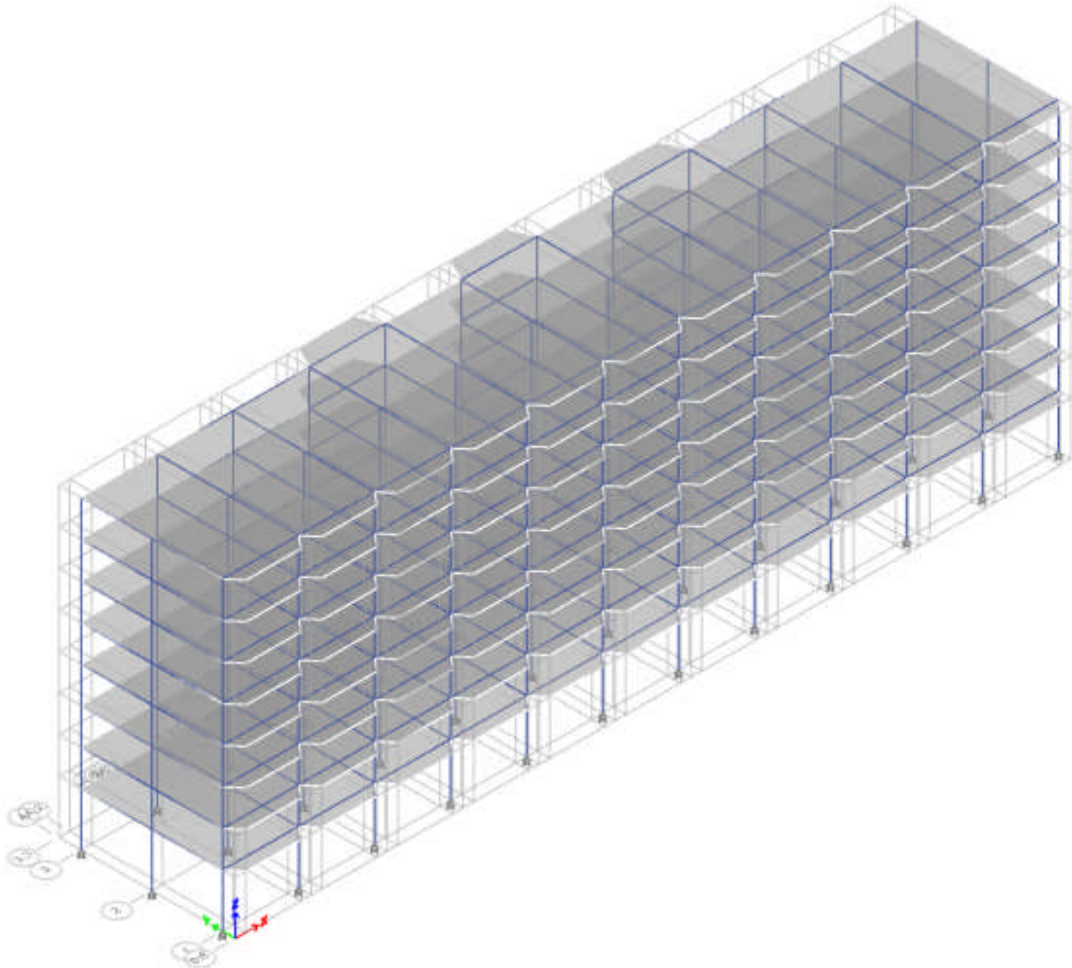

Figure 4 – 1: Three-Dimensional View of the ETABS Model of Existing Building

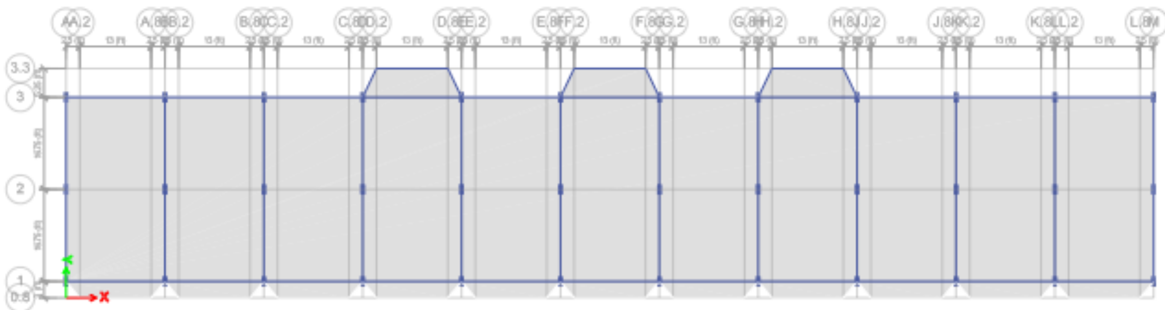

Figure 4 – 2: Typical Plan View of the ETABS Model of Existing Building

4.2.4 Existing Building Fundamental Period and Drifts

Calculated existing building fundamental period and drifts, including the estimated settlement due to liquefaction, are tabulated in Table 4 – 1 below. Both the existing building fundamental period and drift in the long direction are excessively large due to the lack of a competent lateral system in this long direction. There is also significant existing building drift in the short direction.

Existing Building Fundamental Period	T = 3.72 seconds
Existing Building Drift, Long (X) Direction	Delta_X = 42.2 inches (4.9%)
Existing Building Drift, Short (Y) Direction	Delta_Y = 18.0 inches (2.1%)

Table 4 – 1: Existing Building Fundamental Period and Drifts

4.2.5 Deficiency Summary and Preliminary Conclusion

The linear dynamic analysis corroborated the deficiencies that were identified in Section 4.1 and substantiated the concern that the existing building could collapse in the scenario seismic event. The most prominent finding of this analysis is that both the existing building fundamental period and drift in the long direction are excessively large, with the latter being considerably exacerbated by anticipated liquefaction settlement. The corroborated description of existing building seismic deficiencies derived from our structural analysis model are detailed in Table 4 – 2 below.

NONDUCTILE CONCRETE MOMENT FRAMES	A representative existing concrete moment frame column at grid lines “A2” (L1) was evaluated to have a compression uniaxial bending strength DCR of 1.1 and a shear strength DCR of 1.1. A representative existing concrete moment frame beam at grid lines “A12” (L2) was evaluated to have a bending strength DCR of 1.1 and a shear strength DCR of 1.2. A representative existing concrete moment frame beam-column joint at grid lines “A2” (L2) was evaluated to have a joint shear strength DCR of 0.55.
UNBRACED SLENDER GROUND FLOOR CONCRETE MOMENT FRAME COLUMNS	A representative existing concrete moment frame column at grid lines “F1” (L1) was evaluated to be in excess of slenderness limits and does not have adequate compression buckling strength with respect to force-controlled seismic compression loads.
EXTERIOR UNREINFORCED HOLLOW BLOCK AND UNREINFORCED BRICK MASONRY WALLS	It is unlikely that any unreinforced hollow block masonry walls, nor any unreinforced brick masonry walls, can accommodate the anticipated existing building drift of 2% & 5% without failure. Additionally, the force-controlled in-plane shear DCR of the bottom soft story in the long direction of the building is 4.8.
INTERIOR UNREINFORCED HOLLOW BLOCK MASONRY DEMISING WALLS	It is unlikely that any unreinforced hollow block masonry walls, nor any unreinforced brick masonry walls, can accommodate the anticipated existing building drift of 2% & 5% without failure.
PILE CAPS	In addition to the existing concrete pile caps not having any significant uplift capacity, due to the lack of top reinforcement, the existing concrete moment frame column dowels are also straight (no hooks) with a development length of only twenty bar diameters. Typical existing concrete moment frame column dowel tension development lengths into the existing concrete pile caps have a DCR of 1.6. Fortunately, it is anticipated that even with potentially removing

	the self-weight of the existing unreinforced masonry, the net tension uplift DCR would be 0.4.
PILES	In addition to the existing piles' significant capacity reduction due to liquefaction settlement downdrag, their connections to the existing concrete pile caps is unknown. Therefore, the shear and uplift capacities at these critical foundation connections are also unknown.
LIQUEFACTION	The estimated liquefaction settlement would increase the overall building drifts in the scenario seismic event by 15% and 30% in the short and long directions, respectively. Note also that the building is anticipated to yield at a similar intensity ground motion as will induce liquefaction of the site.
LATERAL SPREADING	Lateral spreading of the site towards the river would significantly exacerbate the deficiencies identified above and could affect the connection of the piles to the pile caps

Table 4 – 2: Detailed Description of Existing Building Seismic Deficiencies

4.2.6 Seismic Risk Characterization

The analysis results outlined above have also been utilized to gain an understanding of the building's seismic risk in its existing condition. We have estimated that the building has a 0.3% annual risk of collapse. Perhaps a meaningful comparison for this number would be to compare it to a typical existing building that would likely pass the Freddie Mac stability criterion to not collapse in the earthquake ground motion prescribed in the current edition of the International Building Code. We have estimated that this hypothetical building has a 0.03% annual risk of collapse, or approximately 1% over the estimated life 30 year life of that building. The Smith Building would accumulate an equivalent amount of seismic risk in a little over 3 years.

4.3 Concrete Shear Wall Buttress Retrofit

4.3.1 Introduction

A conceptual retrofit with concrete shear wall buttresses, concrete grade beam supports, and deep foundations has been developed. The design intent of the concrete shear wall buttresses is to provide a new lateral system for the existing building. The design intent of the grade beams is to provide adequate support for the existing pile caps. Both the concrete shear wall buttresses and grade beams would be supported by deep foundations and pile caps. Additional fiber reinforced polymer (FRP) column strengthening, FRP collector trusses, and demolition of the existing unreinforced hollow block masonry and unreinforced brick masonry walls are also included in this conceptual retrofit. The scope of this conceptual retrofit is to mitigate all the seismic deficiencies previously noted for the existing building, including the mitigation of liquefaction and lateral spreading risk. The scope of this conceptual retrofit is shown in Appendix A. The three-dimensional structural analysis model is shown below in Figure 4 – 3.

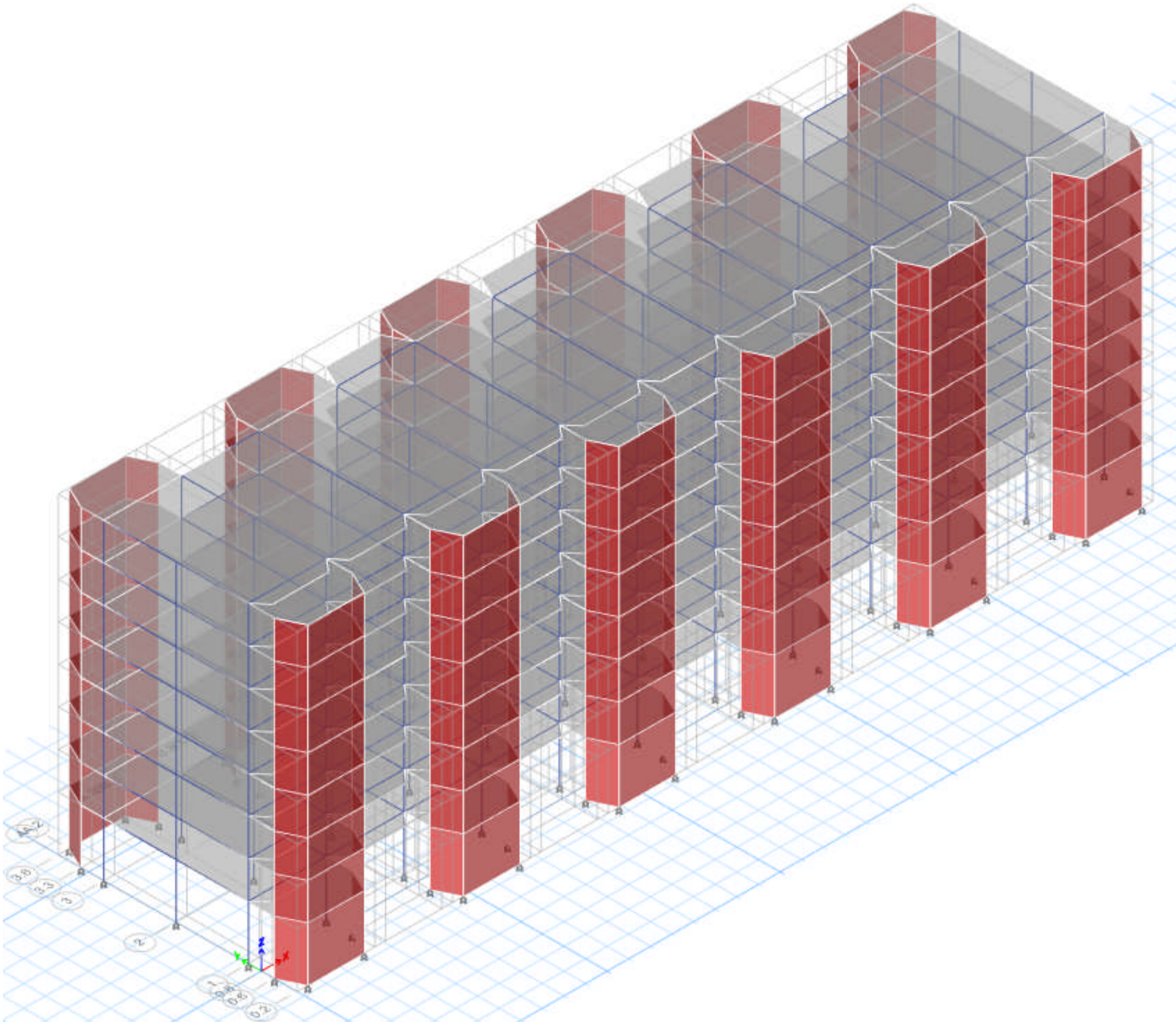

Figure 4 – 3: Conceptual Retrofit Structural Analysis Model

4.3.2 Concrete Shear Wall Buttresses

The proposed concrete shear wall butresses are 18 inches thick as shown in Appendix A. These concrete shear wall buttresses significantly change the dynamic response of the building by reducing the fundamental period which results in the primary mode of vibration being oriented in the short direction of the retrofitted building as opposed to the long direction of the existing building. Fiber reinforced polymer (FRP) collector trusses encased in a concrete slab buildout to match the existing concrete floor/roof slab thicknesses are proposed to transfer the retrofitted building's inertial forces into these concrete shear wall buttresses as shown in Appendix A.

4.3.3 Grade Beams

The proposed grade beams are 4' – 0" deep as shown in Appendix A. The design intent of these grade beams is to support the existing concrete column pile caps to directly mitigate liquefaction differential settlement at these locations. These grade beams span to the proposed concrete shear wall buttress pile caps.

4.3.4 Pile Caps and Deep Foundations

The proposed pile caps are 4’ – 0” deep as shown in Appendix A. The deep foundations consist of pipe 14 standard concrete filled 120’ – 0” deep piles as shown in Appendix A. The embedment length of these piles is an estimate as a site-specific geotechnical investigation has not been provided at this time.

4.3.5 Retrofitted Building Fundamental Period and Drifts

The proposed conceptual retrofit significantly reduces the fundamental period of the building, significantly reduces building drift in the long direction of the structure, and significantly increases the building stiffness, ductility, and strength particularly in the long direction of the building. The summary of the retrofitted building fundamental period and drifts are shown below in Table 4 – 3.

Retrofitted Building Fundamental Period	T = 0.82 seconds	78% Reduction
Retrofitted Building Drift, Long (X) Direction	Delta_X = 4.7 inches (0.6%)	89% Reduction
Retrofitted Building Drift, Short (Y) Direction	Delta_Y = 18.0 inches (2.1%)	Negligible Reduction

Table 4 – 3: Retrofitted Building Fundamental Period and Drifts

4.4 Limitations of Lateral Analysis

The above analysis and findings are based on a simplified three-dimensional linear dynamic analysis and assumed material properties and reinforcing. Since this building is expected to experience a moderate degree of nonlinear performance (yielding and damage) in the scenario seismic event, nonlinear analysis is recommended to validate any proposed retrofit. Additional material testing, with respect to the existing building, will also be required to obtain knowledge of the expected existing building construction material performance. The preliminary linear dynamic analysis was intended to support preliminary design of some of the needed retrofit measures, to support approximate cost analysis, and development of a conceptual design.

5.0 RETROFIT SUMMARY

We recommend that a seismic upgrade of this building be performed as shown in Appendix A. Significant testing and documentation of the existing building structural conditions will need to be performed so that more accurate analysis and design may be prepared with respect to the conceptual retrofit.

APPENDIX B

Shannon & Wilson Report

SHANNON & WILSON

September 19, 2022

Andrew Fendrich
Fendrich Engineering, Inc.
305 E. Monroe St.
Springfield, IL 62701
fendeng@aol.com

RE: GEOTECHNICAL EARTHQUAKE ENGINEERING SERVICES
ADDITIONAL LIQUEFACTION ANALYSIS, ALEXANDER COUNTY HOUSING AUTHORITY BUILDINGS, CAIRO, ILLINOIS

Dear Mr. Fendrich:

This letter includes additional liquefaction analysis in support of your project at the Alexander County Housing Authority buildings, Smith Building and Shuemaker Building, located in Cairo, Illinois. Refer to our report titled *Preliminary Liquefaction Analysis, Alexander County Housing Authority Building, Cairo, Illinois* dated July 27, 2022 and prepared for Eggemeyer Associate Architects for information about the site, project description, soil profile, regional seismicity, and our engineering analysis.

On August 24, 2022, we received a request from you to perform additional liquefaction analysis for the Smith Building and the adjacent Shuemaker Building. The request consisted of determining the threshold acceleration required to initiate liquefaction at the building's location. On August 30, 2022, you indicated via email that there is no geotechnical data for the Shuemaker Building. During a conference call on August 31, 2022, we discussed that in the absence of additional geotechnical data for the Shuemaker Building, we will use the geotechnical information from the Smith Building to perform the liquefaction analysis.

PROJECT UNDERSTANDING

Our preliminary liquefaction analysis was performed following ASCE 41-17 Seismic Evaluation and Retrofit of Existing Structures and a 10% probability of exceedance in 50 years (return period of 475 years). The subsurface information used in our preliminary liquefaction analysis used the geotechnical information collected before construction of the Smith Building and provided to us in Sheet S 3 of the drawing set titled *PHA CRO Low Rent Housing Projects, ILL. 7-4, Alexander County Housing Authority* dated December 13, 1965.

ENGINEERING ANALYSES

Refer to our report titled *Preliminary Liquefaction Analysis, Alexander County Housing Authority Building, Cairo, Illinois* dated July 27, 2022 for details about our assumptions, site soil classification, seismic design, results of our liquefaction analysis for a Site Class E site, and evaluation of other earthquake-induced geologic hazards and existing foundations for the Smith Building.

In our preliminary liquefaction analysis report we indicated that the soils at the site are susceptible to liquefaction and shall be classified as Site Class F. ASCE 41-17 requires that a site response analysis be performed to determine spectral accelerations for seismic design. The liquefaction analysis and information provided below is for informational purposes only and should not be a substitute for a site response analysis. Additional subsurface explorations will need to be completed for the Smith Building and Shuemaker Building to provide adequate subsurface information for use in the site-specific response analysis since the previously completed borings do not extend to soil or rock layers with a high enough shear wave velocity.

Liquefaction Analysis

To evaluate the liquefaction potential at this site we used the empirical methods as described in Youd and others (2001) and Idriss and Boulanger (2014). These empirical methods are based on correlations between SPT resistance, peak ground acceleration (PGA), and earthquake magnitude. We used an earthquake magnitude of 7.5, as determined from the 2014 USGS NSHMP deaggregation. We calculated the factor of safety against liquefaction for a series of accelerations. If the factor of safety against liquefaction is below 1.0, then the soil is going to experience liquefaction under that intensity of shaking.

Our analyses indicates that an acceleration of 0.18g and 0.31g for the soil profiles in Borings B-1 and B-4, respectively, has the possibility of inducing liquefaction. The results of our liquefaction analyses are presented in Figures 1 and 2. We have also calculated the return period and probability of exceedance for the accelerations that will trigger liquefaction. For Boring B-1, the acceleration of 0.18g has a return period of 127 years and a probability of exceedance of 34% in 50 years. For Boring B-4, the acceleration of 0.31g has a return period of 234 years and a probability of exceedance of 19% in 50 years.

ADDITIONAL WORK

We recommend that additional explorations and shear wave velocity measurements be performed at the site to evaluate the transition of subsurface conditions from the northeast to southwest, characterize the earthquake-induced geologic hazards, and perform the site response analysis as

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required to determine the design ground motion in accordance with ASCE 41-17. We are available to perform the site response analysis upon request.

CLOSURE

We have appreciated this opportunity to assist you on this project and look forward to working with you again. Please contact us if you have questions concerning this letter, or if we may be of further service.

Sincerely,

SHANNON & WILSON, INC.
Illinois Professional Design Firm
License No. 184-001377

LICENSED PROFESSIONAL ENGINEER
VONMARIE
MARTINEZ-CHALUISANT
062.068114
STATE OF ILLINOIS

Exp: 11/30/2023
09/19/2022

Vonmarie Martinez, P.E.
Senior Professional I

VMC:PMK:TJA/tad

Enc. Figure 1: Results of Liquefaction Analyses, Boring B-1 (1965), M=7.5, PGA=0.18g
Figure 2: Results of Liquefaction Analyses, Boring B-4 (1965), M=7.5, PGA=0.31g
Important Information About Your Geotechnical Report

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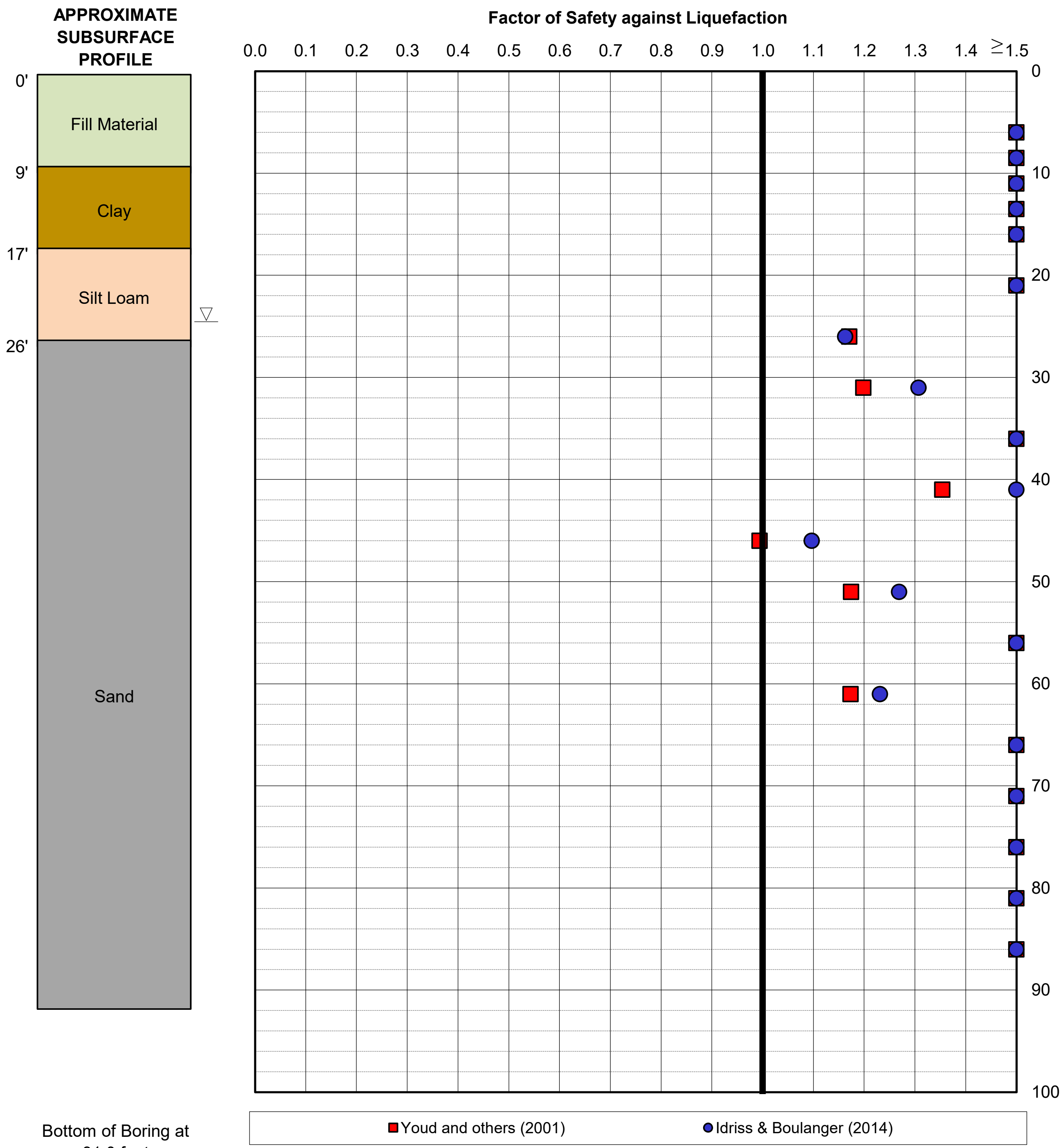
B-1 7.5M refined.xlsm 9/13/2022

VMC



Bottom of Boring at 91.0 feet

NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Liquefaction Analysis
Alexander County Housing Authority Building
Cairo, Illinois

RESULTS OF LIQUEFACTION ANALYSES
BORING B-1 (1965)
M = 7.5, PGA = 0.18g

September 2022 110072-001

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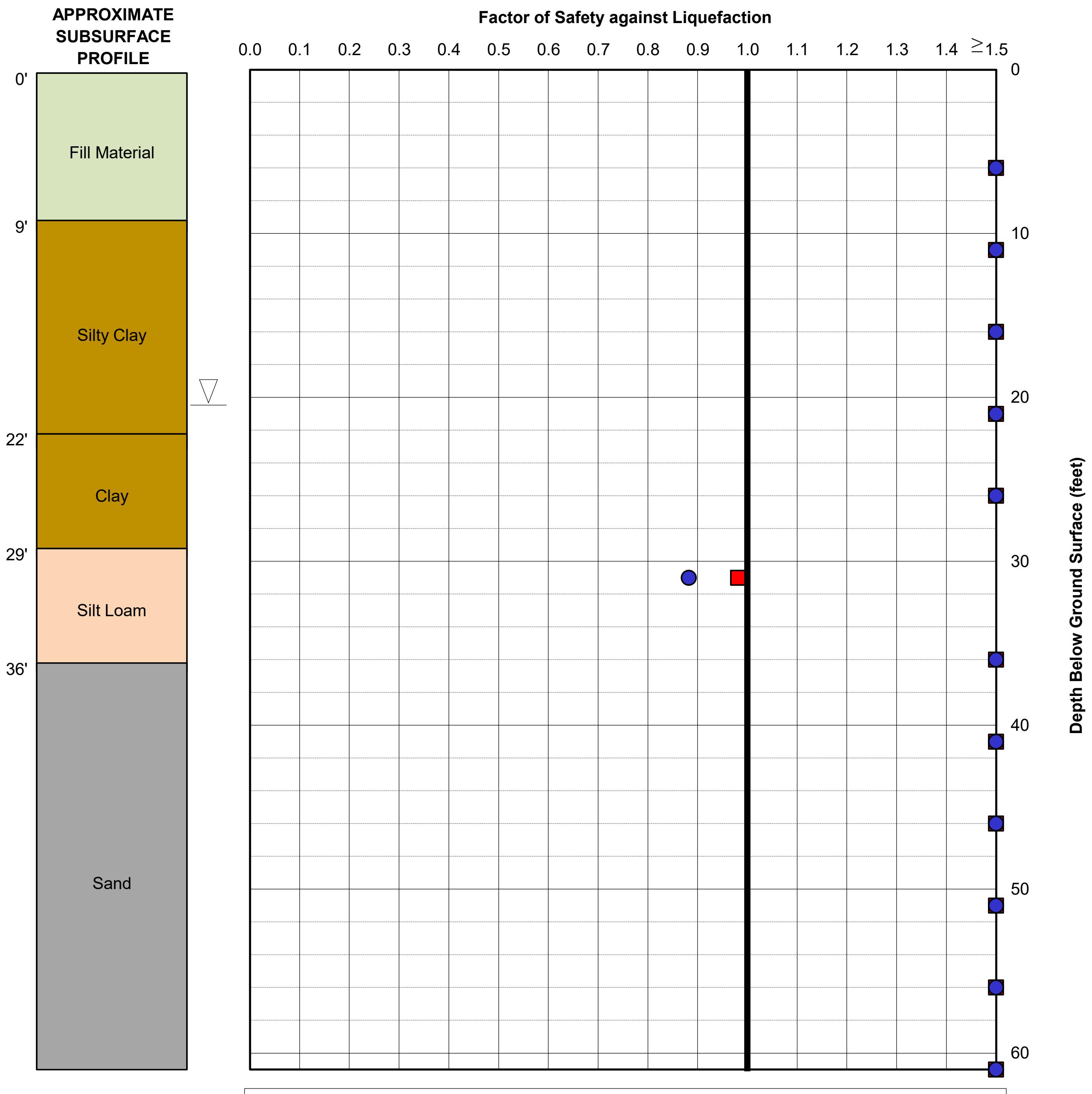
FIG. 1

VMC



Bottom of Boring
at 61.0 feet

NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Liquefaction Analysis
Alexander County Housing Authority Building
Cairo, Illinois

RESULTS OF LIQUEFACTION ANALYSES
BORING B-4 (1965)
M = 7.5, PGA = 0.31g

September 2022 110072-001

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Geotechnical and Environmental Consultants

FIG. 2

IMPORTANT INFORMATION

Important Information

About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied

IMPORTANT INFORMATION

judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

IMPORTANT INFORMATION

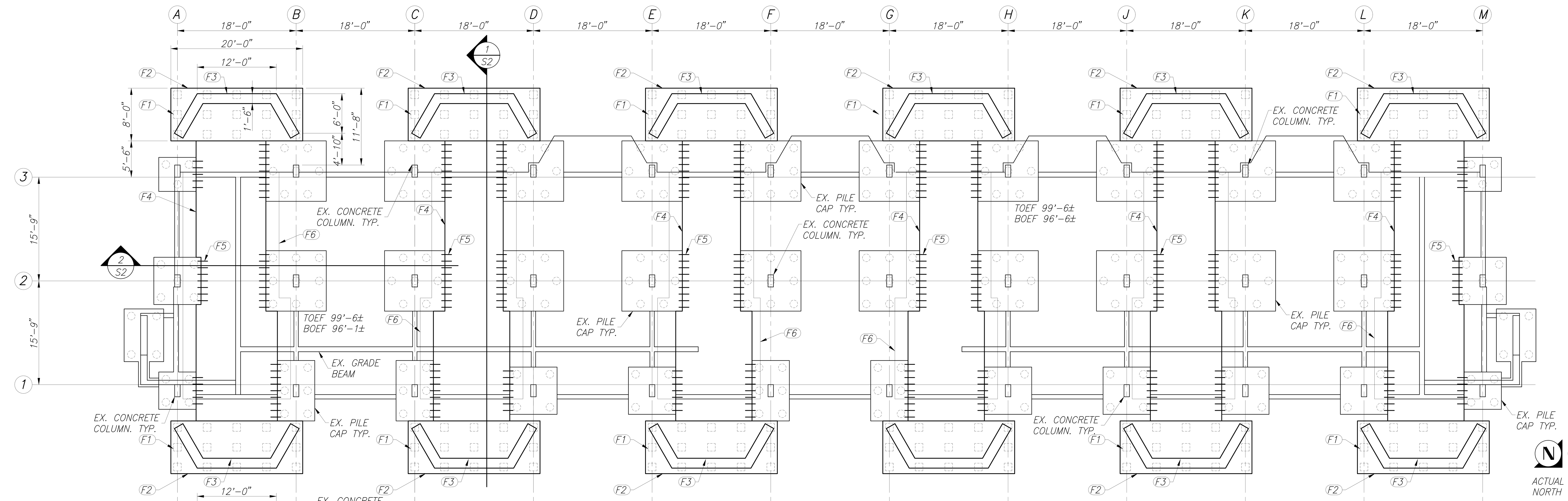
READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

APPENDIX C

Conceptual Seismic Retrofit Drawings



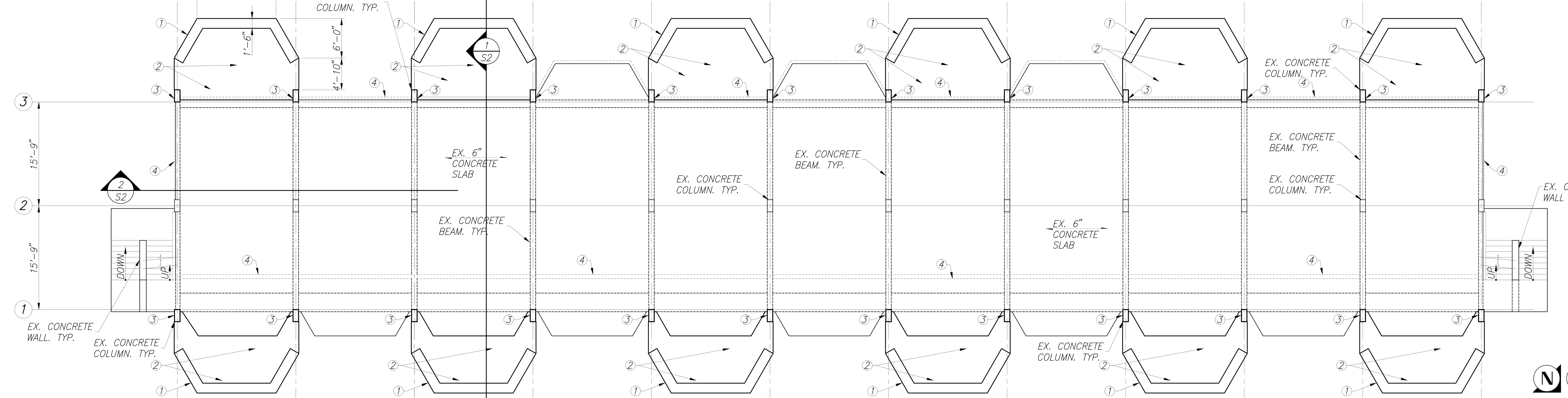
FOUNDATION PLAN 1/S1

ACTUAL NORTH PLAN NORTH

SCALE: 1/8" = 1'-0

FOUNDATION NOTES

- (F1) 14" SQUARE x 120'-0 PRECAST, PRESTRESSED CONCRTE PILE.
- (F2) REINFORCED CONCRETE PILE CAP.
- (F3) 18" THICK, REINFORCED CONCRETE BUTTRESS.
- (F4) REINFORCED CONCRETE GRADE BEAM BETWEEN BUTTRESS PILE CAPS.
- (F5) SHEAR FRICTION DOWELS FROM GRADE BEAM INTO EXISTING CONCRETE PILE CAPS.
- (F6) OUTLINE OF REMOVAL AND REPLACEMENT OF EXISTING 6" CONCRETE SLAB-ON-GRADE.



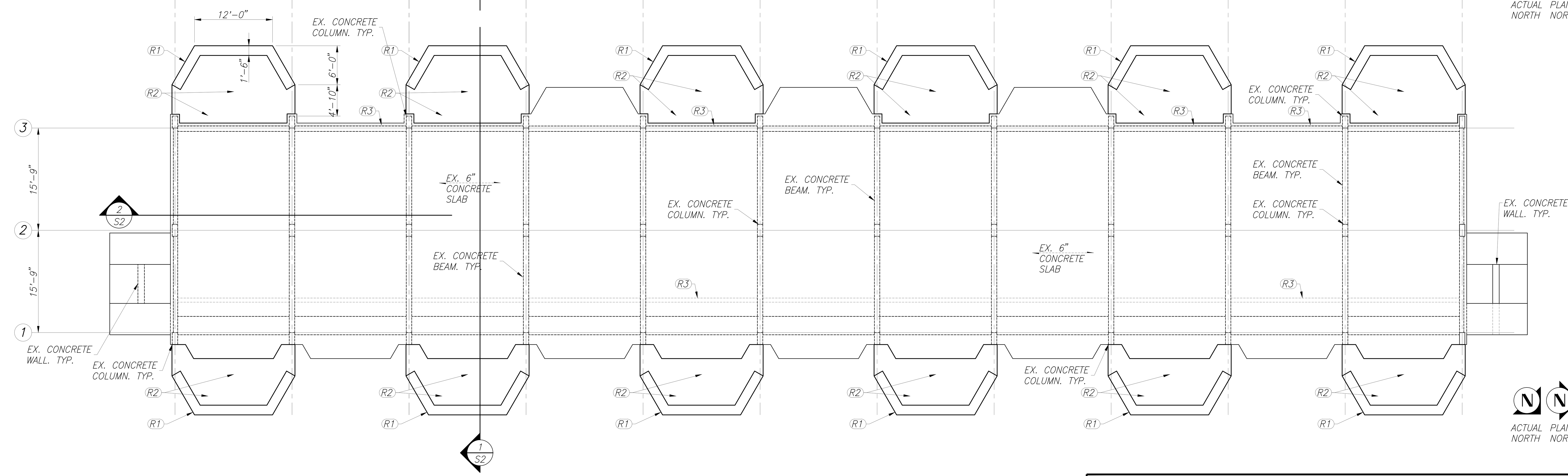
TYPICAL FLOOR FRAMING PLAN 2/S1

ACTUAL NORTH PLAN NORTH

SCALE: 1/8" = 1'-0

TYPICAL FLOOR NOTES

- (1) 18" THICK, REINFORCED CONCRETE BUTTRESS.
- (2) 6" REINFORCED CONCRETE FLOOR SLAB WITH EMBEDDED FIBER REINFORCED POLYMER (FRP) COLLECTOR TRUSS.
- (3) RFP COLUMN STRENGTHENING AT FIRST FLOOR SLENDER COLUMNS.
- (4) ALL UNREINFORCED MASONRY WALLS AND BRICK VENEER AT EACH FLOOR TO BE REMOVED.



ROOF FRAMING PLAN 3/S1

ACTUAL NORTH PLAN NORTH

SCALE: 1/8" = 1'-0

ROOF NOTES

- (R1) 18" THICK, REINFORCED CONCRETE BUTTRESS.
- (R2) 5" REINFORCED CONCRETE ROOF SLAB WITH EMBEDDED FIBER REINFORCED POLYMER (FRP) COLLECTOR TRUSS.
- (R3) ALL UNREINFORCED MASONRY WALLS AND BRICK VENEER AT EACH FLOOR TO BE REMOVED.

SHEET TITLE: **FOUNDATION, TYPICAL FLOOR AND ROOF FRAMING PLANS**			
ENGINEER: FENDRICH ENGINEERING, INC. 305 E. MONROE ST. SPRINGFIELD, IL 62701 PHONE: 217/638-0927	HOHBACH-LEWIN, INC. 909 MONTGOMERY ST., SUITE 260 SAN FRANCISCO, CA 94133 PHONE: 415/318-8520	PROJECT: SEISMIC ASSESSMENT AND CONCEPTUAL RETROFIT REPORT CONNEL F. SMITH BUILDING/IL-7-4/CAIRO LOCATION: 1101 OHIO ST. CAIRO, ILLINOIS	PROJECT NO. 21287 DATE: OCT. 19, 2022 SHEET NUMBER **S1** OF 2 SHEETS

FOUNDATION NOTES

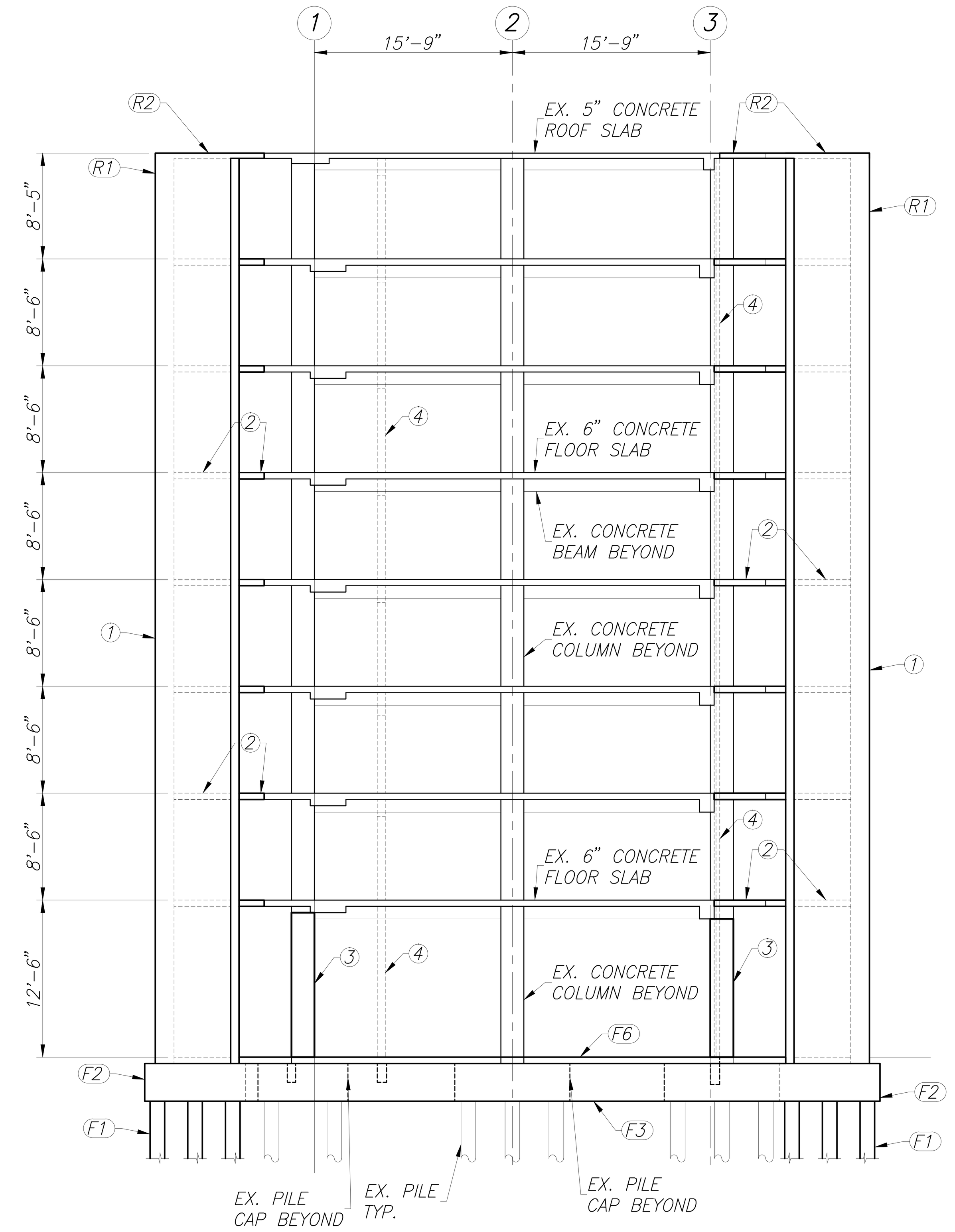
- F1 14" SQUARE x 120'–0 PRECAST, PRESTRESSED CONCRTE PILE.
- F2 REINFORCED CONCRETE PILE CAP.
- F3 18" THICK, REINFORCED CONCRETE BUTTRESS.
- F4 REINFORCED CONCRETE GRADE BEAM BETWEEN BUTTRESS PILE CAPS.
- F5 SHEAR FRICTION DOWELS FROM GRADE BEAM INTO EXISTING CONCRETE PILE CAPS.
- F6 OUTLINE OF REMOVAL AND REPLACEMENT OF EXISTING 6" CONCRETE SLAB–ON–GRADE.

TYPICAL FLOOR NOTES

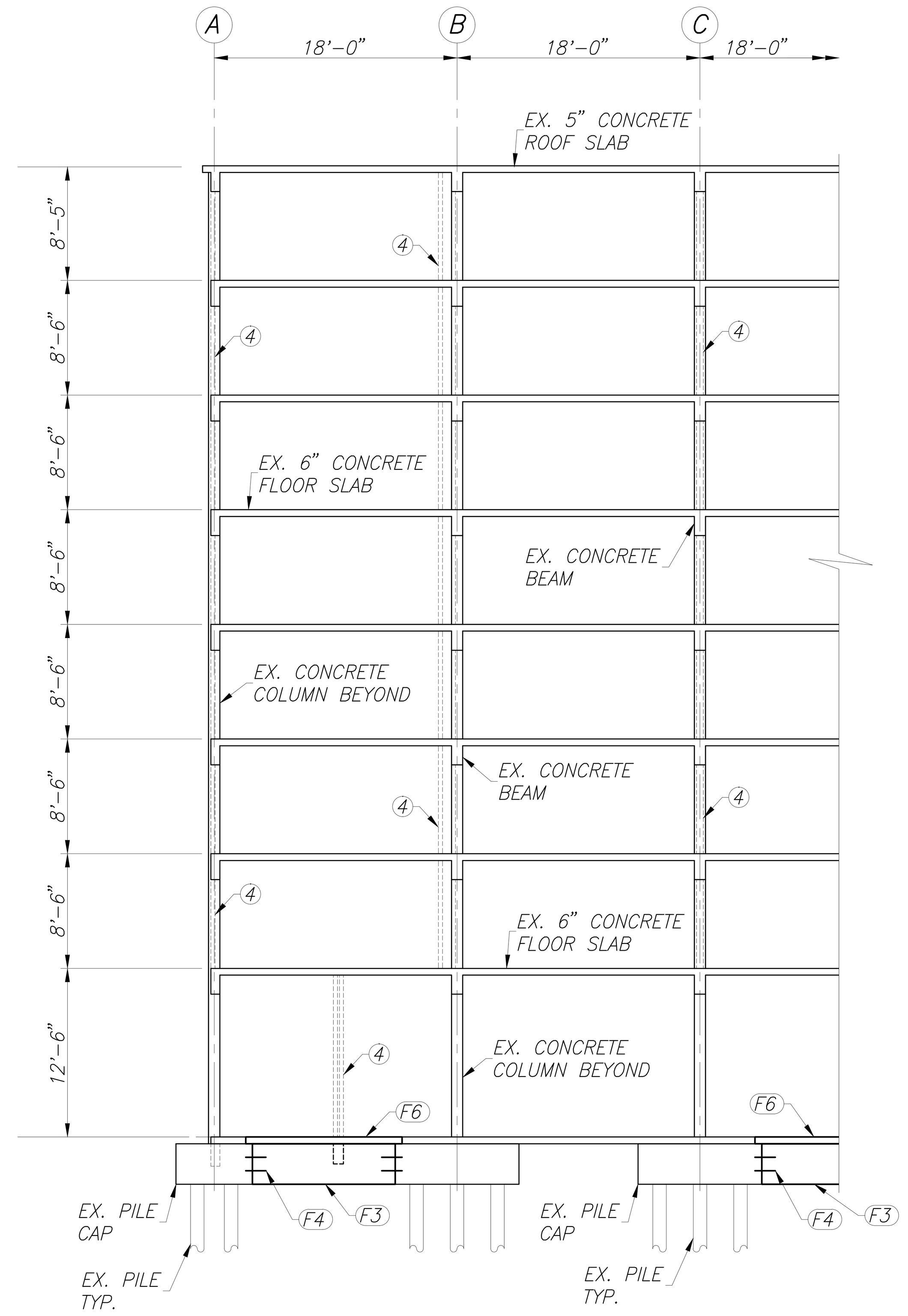
1. 18" THICK, REINFORCED CONCRETE BUTTRESS.
2. 6" REINFORCED CONCRETE FLOOR SLAB WITH EMBEDDED FIBER REINFORCED POLYMER (FRP) COLLECTOR TRUSS.
3. RFP COLUMN STRENGTHENING AT FIRST FLOOR SLENDER COLUMNS.
4. ALL UNREINFORCED MASONRY WALLS AND BRICK VENEER AT EACH FLOOR TO BE REMOVED.

ROOF NOTES

- R1 18" THICK, REINFORCED CONCRETE BUTTRESS.
- R2 5" REINFORCED CONCRETE ROOF SLAB WITH EMBEDDED FIBER REINFORCED POLYMER (FRP) COLLECTOR TRUSS.
- R3 ALL UNREINFORCED MASONRY WALLS AND BRICK VENEER AT EACH FLOOR TO BE REMOVED.



TRANSVERSE BUILDING SECTION 1/S2
SCALE: 1/8"=1'–0
LOOKING SOUTH



PARTIAL LONGITUDINAL BUILDING SECTION 2/S2
SCALE: 1/8"=1'–0
LOOKING WEST

SHEET TITLE: **BUILDING SECTIONS**			
ENGINEER: FENDRICH ENGINEERING, INC. 305 E. MONROE ST. SPRINGFIELD, IL 62701 PHONE: 217/638-0927	HOHBACH-LEWIN, INC. 909 MONTGOMERY ST., SUITE 260 SAN FRANCISCO, CA 94133 PHONE: 415/318-8520	PROJECT: SEISMIC ASSESSMENT AND CONCEPTUAL RETROFIT REPORT CONNEL F. SMITH BUILDING/IL-7-4/CAIRO LOCATION: 1101 OHIO ST. CAIRO, ILLINOIS	PROJECT NO. 21287 DATE: OCT. 19, 2022 SHEET NUMBER **S2** OF 2 SHEETS

APPENDIX D

Structural Budgetary Cost Estimate

BUDGETARY CONSTRUCTION COSTS FOR CONCEPTUAL SEISMIC RETRFIT

PROJECT: Smith Building - Seismic Retrofit
1101 Ohio Street
Cairo, Illinois
FEI # 21287

DIVISION 2 - GENERAL REUIREMENTS - SELECTIVE DEMOLITION (STRUCTURAL)

Demo - Ex. 6" Slab-on-grade	450	ft2 x	6	$/ft2 =	$	2,700	x	6.00		=	$16,200
Demo - Ex. Grade Beam	351	ft x	30	$/ft						=	$10,530
Misc		say								=	$10,000
									SUB-TOTAL	=	$36,730
									SUB-TOTAL DIVISION 2	=	$36,730
									5% General Conditions	=	$1,837
									TOTAL DIVISION 2	=	$38,567

DIVISION 3 - CONCRETE

14" SQ. x 120' - Precast Pile or 14" dia. Shell Pile	15	ea x	20000	$/ea =	$	300,000	x	12		=	$3,600,000
Typical Pile Cap	24	yd3 x	500	$/yd3 =	$	11,852	x	12		=	$142,222
Typical Seismic Grade Beam	74	yd3 x	500	$/yd3 =	$	37,037	x	6		=	$222,222
Shear Friction Dowels	108	ea x	100	$/ea =	$	10,800	x	6		=	$64,800
18" Thick Buttress	97	yd3 x	1700	$/yd3 =	$	165,246	x	12		=	$1,982,956
Concrete Floor/Roof Connector Slabs - W	4	yd3 x	2500	$/yd3 =	$	9,722	x	48		=	$466,667
Concrete Floor/Roof Connector Slabs - E	3	yd3 x	2500	$/yd3 =	$	7,407	x	48		=	$355,556
FRP Collector Truss - Ex. to Buttress	18	ft x	180	$/ft =	$	3,240	x	96		=	$311,040
FRP Column Strengthening - 1st Floor	67	ft2 x	25	$/ft2 =	$	1,675	x	24		=	$40,200
Replacement of Ex. Grade Beams	351	ft x	100	$/ft						=	$35,100
Misc		say								=	$50,000
									SUB-TOTAL	=	$7,270,762
6" slab on grade replacement	450	ft2 x	7.00	$/ft2 =	$	3,150	x	6.00		=	$18,900.00
									SUB-TOTAL	=	$18,900
									SUB-TOTAL CONCRETE	=	$7,289,662
									5% General Conditions	=	$364,483
									TOTAL CONCRETE	=	$7,654,145

TOTAL DIVISION 2	=	$38,567
TOTAL CONCRETE	=	$7,654,145
TOTAL	**=**	**$7,692,712**