PIMA COUNTY

PROCUREMENT

PIMA COUNTY
PROCUREMENT DEPARTMENT
DESIGN AND CONSTRUCTION DIVISION
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July 13, 2016

ADDENDUM NO. 3

PROJECT: Architecture and Engineering Services
Pima County Old Courthouse Tenant Improvement

SOLICITATION NO.: 221810

TO POTENTIAL RESPONDENTS:

Be advised of the additions, clarifications and/or changes to the above-referenced Solicitation for Qualifications as stated in the following Addendum.

1) Just to be clear, we will have a lead on this job, but will not need a PM, correct? **Pima County will have a PM on the project that will represent Pima County. The Consultant is to provide any/all staff required to manage, lead and represent the Design Team.**

2) Has an historic Structures Report been prepared for this building? **Yes.**

3) Has a structural evaluation of the building been done re. seismic stability? **Yes.**

4) If so, are both available? **Yes, see the attached dated March 2016. The structural improvements recommended in the report will be completed as part of the Old Courthouse exterior renovation project with a construction start date of October 2016.**

5) Will the architectural team be responsible for ensuring the chosen contractor complies with the terms of the contract they are retained under or will this task be carried out by others? **Refer to the solicitation documentation - Exhibit A - Scope of Professional Services - item 7.g for the "CONSULTANT'S" responsibilities during construction.**

6) Are the original construction documents for this building available? **Yes, Pima County Facilities Management (PCFM) does have copies of the original construction documents and can be made available upon request to the Contracts Officer. Due to the file size, the files will be sent via Accellion.**

All other requirements and terms of the Solicitation remain unchanged. Failure to include acknowledgement of all addenda in the introductory letter may be cause for rejection.

This addendum is a total of 14 pages.

If there are any questions, please do not hesitate to contact me via e-mail at matthew.sage@pima.gov or fax (520) 724-4434.

/s/ Matt Sage
Matthew Sage
Contracts Officer

2.3

STRUCTURAL

Era: 1929 and 1954
Integrity: High
Significance: Moderate
Treatment Method: Rehabilitation

SCOPE OF STUDY:

The structural assessment of the existing Courthouse building consisted of the review of architectural and structural drawings for the original 1929 building (Roy Place Architect) and the 1954 south end addition (Blanton & Cole Architects), field observations of existing conditions, and spot check gravity load evaluation of floor framing. A comprehensive gravity and lateral load analysis of the building was not performed and beyond the scope of this study.

It is understood that the intended rehabilitation work on this structure will be permitted using the 2012 International Existing Building Code (2012 IEBC), and the conversion from A-3 (courthouse) to A-3 (museum) would not be considered a change of use. Given the extent and nature of the proposed building rehabilitation, we anticipate the renovation work be categorized as "Level 3 Alteration" and subject to compliance with chapters 7, 8 and 9 of the IEBC. Per sections 807.4 and 807.5, alterations shall not reduce or affect the demands or capacity of existing structural elements. If additional loads or capacity reductions are not introduced in the alteration, the existing structure is not required to comply with current codes or undergo upgrades. Per 807.2, new structural elements introduced in alterations, including connections and anchorage shall comply with the IBC for new con-

Roof deck, steel truss, and steel bar joist

struction. Also, per 907.4.4, any building assigned to Seismic Design Category E, E, or F with concrete or masonry walls and a flexible roof diaphragm, alteration work shall include the installation of wall anchors at the roof line. Although buildings in Tucson generally fall into Seismic Design Category B, adequate anchorage of concrete and masonry walls to wood roof diaphragms is considered a critical life safety condition that should be met. All other structural upgrades listed are recommendations only, for improved life safety and building preservation.

1929 BUILDING STRUCTURE DESCRIPTION (1929)

Roof Framing

The roof structure is steel framed with a combination of open web joists, trusses, and steel beams. The open web joists are spaced at 7 feet ± on center, spanning approximately 20' to trusses or bearing walls. Clear span trusses over the north and south wings bear on concrete walls. 2" wood plank decking, spanning between the open web joists.

Roof top Exercise Deck

The roof level exercise deck is framed with one-way concrete pan floor joists, concrete side walls and is open above with no ceiling structure. The safe live load for this space is 50 psf.

Tower Dome Structure

The dome structure is a 3" thick reinforced concrete shell with a continuous reinforced concrete tension ring around its base. The dome rests on cast-in-place concrete wall piers spaced uniformly at the perimeter. The open deck area beneath the dome (Elev. 149'-6"), has a cast-in-place concrete slab and beam floor system. The safe live load for the space under the dome is 30 psf.

The lower room level of the dome (Elev. 138'-0") has a floor system framed with a combination of one way pan joists slabs and beams. The save live load for the space is 100 psf.

Third Floor Framing

The third floor of the original structure occurs at the west wing only. The floor framing consists of a combination of cast-in-place concrete pan joists, beams and slabs. Safe live load for the third floor area of the west wing is 70 psf.

The original floor opening near the south end of the wing has been infilled with steel framing and concrete topping. Details of member sizes and connections are unknown. The time of when the infill was constructed is also unknown.

Second Floor Framing

The second floor of the north, south and west wings have a cast-in-place concrete pan joists, beam and slab system.

The south wing and north half of the west wing floor area can support a live load of 70 psf.

The north wing and south half of the west wing floor areas have a safe live load capability of 100 psf.

The courtyard corridors over the arcades can safely support a live load of 80 psf. The perimeter columns supporting the arcade structures appear to be hollow precast concrete, presumably grout or concrete filled. It is not known if the center core is reinforced. Connection of the precast columns to the foundation and arcade arches is not known.

First Floor Systems

The first floor of the north and west wings are concrete slab-on-grade construction. Under floor pipe trenches occur along the inside of the perimeter walls with concrete slab covers.

The south wing has a basement level with a concrete pan joist and beam framing system at the first floor. The floor system safe live load is 70 psf.

Foundations

The foundation system consists of reinforced concrete spread footings and continuous wall footings. Foundation depths vary and are approximately 2'-6" below slab at the basement and 6'-6" below first floor slab. The arcade precast columns bear on a continuous deep stem wall and strip footing.

Exterior Walls

The exterior walls of the original structure are 12" thick reinforced concrete. The wall reinforcing is relatively light by today's standards when considering seismic and wind design detailing practice. The walls however, appear to be performing adequately with negligible evidence of distress.

Lateral Load Resisting System

As with most building construction of this era, design and detailing for wind and seismic resistance is marginal or non-existent. This building has been performing reasonably well structurally for the last 85 years however, there are a few areas of concern that should be corrected or investigated further by lateral analysis.

Roof Diaphragm

Adequate diaphragm shear capacity and wall anchorage to the diaphragm is often a problem with wood sheathed roofs constructed in this era. The roof deck consists of 2" thick planking, which is not recognized as a qualified diaphragm. There is inadequate wall anchorage and diaphragm connection along perimeter concrete walls.

It is recommended that the roof diaphragm system be improved by sheathing the entire roof surface with 1/2" plywood, or alternatively, install a horizontal rod bracing system at the truss bottom chord level. Along the north and south walls of the north and south wings, the concrete bearing walls are anchored to the roof at the truss bearings (21 ft. o.c. ±). At least one additional wall anchorage

Floor / ceiling framing at central breezway

Board formed concrete walls, basement

to the diaphragm should be implemented midway between the existing trusses. Other diaphragm improvements necessary for a complete system capable of transferring lateral forces to shear walls include the installation of continuous steel angles along the top edge of perimeter concrete walls, anchored to the wall and connected to the roof diaphragm with continuous blocking.

Tower Dome Lateral Resistance

The tower dome appears laterally stable from its base, elevation 149'-6", and upward. However, below elevation 149'-6", there are no shear walls in the E-W direction located in the west wing. Lateral load resistance for the west wing is dependent on adequate diaphragm shear transfer to the north and south wings where shear walls exist. Large floor openings in the second and third floors create discontinuities in the diaphragm, limiting the lateral load shear transfer adequately to the north and south wings. It is recommended that two concrete shear walls (20 feet long ±) be constructed on the E-W direction in the west wing, one each side of the dome. The shear walls need to extend from the first floor up to elevation 149'-6" concrete slab supporting the dome

Free Standing Arcade Walkway (East Side)

The free standing arcade structure lacks a complete, well defined lateral load resistance system. The precast column supports are pinned, at best, to the cast in place walkway above and below. The lateral stability of the structure is dependent on the upper level concrete deck spanning horizontally to the north and south wing connection points. There is insufficient reinforcing in the edge beams to develop the chord forces to resist wind and seismic forces. It is recommended that a continuous steel angle be installed along the outer bottom edges of the walkway deck surface. This will provide a diaphragm chord that spans the 116 feet.

Floor Diaphragms

The cast in place concrete floor framing system provides a rigid diaphragm to provide bracing for the walls and distribute lateral forces to perimeter concrete shear walls. The only concern regarding diaphragm shear transfers are in the west wing, as discussed previously regarding the dome stability.

Shear Walls

The perimeter concrete walls provide the lateral load resistance for the structure. The walls are lightly reinforced and not specifically detailed as shear walls; however the continuous wall configuration allows the system to perform adequately.

1954 ADDITION

The 1954 building addition is situated along the south end of the existing structure, with two stories and a basement. There is no expansion joint provision along the interface of the addition and original construction. There is no indication that the two structures are structurally connected together at diaphragm levels and appear to have a flush cold joint between the concrete structures.

The addition appears identical to the original building in terms of character and materials however; there are some differences in structural systems. The following is a description of the structural systems.

<u>Roof Framing</u>
The roof structure is framed with long span steel trusses spanning in the N-S direction and spaced at 7'-8" on center. Steel channels spaced at 48" on center span between the trusses provide support for the 2x6 tongue and groove sheathing. A horizontal and diagonal steel bracing system exists between trusses and along the bottom chord of the roof trusses for roof framing stability and enhanced diaphragm capability. The steel roof trusses

are supported at the perimeter by steel beams and columns spaced at 11 feet on center.

The roof penthouse has a cast-in-place concrete slab floor supported by concrete beams, walls and columns below. The penthouse roof is framed with open web steel joists which bear on unreinforced masonry perimeter walls.

Floor Framing (First Second Floor)
The first and second floors are a cast-in-place concrete pan joist and beam system. The floor live load capability is typically 70 psf with the exception of the northern 12 foot bay which can safely support 100 psf.

Foundations
The foundation system consists of reinforced concrete spread column footings and continuous wall footings. The existing foundations along the south edge of the existing building were underpinned for the new columns and basement construction.

Exterior Walls
The perimeter exterior walls of the addition are unreinforced masonry infill between the column and beam framing. The south walls of the arcade are reinforced concrete up to the second floor and transition to masonry infill up to the roof.

Lateral Load System

Roof Diaphragm

The roof sheathing, 2x6 tongue and groove, does have marginal diaphragm capabilities; however the nailing frequency and size are unknown. The horizontal and diagonal bracing located in the plane of the bottom chord of the truss provides a relatively complete lateral load path through the diaphragm to shear walls.

The roof construction of the addition extends across the joint between the existing roofs. Expansion provisions or rigid structural ties have not

Penthouse roof, south addition

been built into the system, so relative movements of the two structures have not been addressed for temperature, wind, and seismic considerations. The only rigid material contact between the two buildings occurs along the penthouse floor. The concrete floor appears to be cast flush against the wall of the original building. Epoxy dowels or through bolts should be installed through the joint to restrain differential movements. Due to different building stiffness's and natural periods of the two structures, hammering can occur during a seismic event.

Floor Diaphragms

The cast-in-place concrete floor system provides a rigid diaphragm which provides bracing for the walls and lateral force transfer to shear walls. The concern at the floors pertains to the joint between the two buildings. The concrete floor beams and joists are cast hard against the original building concrete walls without rigid ties or expansion provisions to account for movement. It is recommended that epoxy dowels or through bolts be installed through the joint to prevent hammering during an earthquake.

Shear walls

Lateral load resistance for the south addition is provided by unreinforced masonry infill wall panels. The infill panels are confined by concrete frame up to the second floor and by steel frame above the second floor to roof level. In addition to the perimeter shear wall system, lateral load resistance is provided by interior reinforced concrete walls located at the elevator and stair core.

RECOMMENDATIONS

The following repairs and improvements are recommended to reduce life safety risks, enhance structural performance, and preserve the useful life of the building.

- Install an adequate roof diaphragm over the 1929 original building roof. This can be accomplished by either installing 1/2" plywood sheathing over the existing 2x6 plank deck or, install a horizontal steel rod bracing system below the roof at the bottom chords of the roof truss system.
- Install wall braces along the north and south exterior walls at the north and south wings of the 1929 original buildings. Braces should occur as a minimum, at midpoints of the existing trusses at approximately 21 feet on center. The braces will connect the wall back to the diaphragm.
- Install continuous steel angles along the top edge of perimeter concrete shear walls, bolted to the wall and continuously connected to the roof diaphragm. This applies to the roof of the 1929 original structure and does not apply to the addition.
- Install two 10" thick by 20' long concrete shear walls each side of the dome E-W direction, full height up to the lower floor of the dome (elevation 149'-6").
- Install two continuous steel angles along the outer edges of the upper walkway deck at the east side arcade.
- Install epoxy dowels or through bolts along the floor lines and penthouse floor between the original building and addition in order to prevent "hammering" during an earthquake.
- As an alternate consider installing QuakeWrap on first floor south wing to increase live load capacity as needed for high intensity assembly uses.
- A complete "as-built" investigation and documenting of the third floor opening infill framing and connections is recommended and evaluated for safe loading limitations.

5. Floor Live Load Diagrams

The floor live load diagrams provided on the following pages are recommended maximums for space planning purposes. Superimposed dead

loads have been assumed to account for flooring material, ceiling, mechanical, electrical and plumbing systems. It is recommended that final structural evaluation and verification be provided when occupancy, space use, corridor layout, lobby areas, finish material selection and infrastructure systems are designed. The superimposed dead loads affect the maximum live load the structure is capable of supporting.

The following live loads for intended occupancy is provided as a reference, and specified in Table 1607.1 of the IBC:

- Office: 50 psf plus 15 psf minimum allowance for partitions
- Lobbies and Assembly areas: 100 psf
- Corridors: 100 psf on first floor, 80 psf above first floor
- Computer and File rooms: 100 psf
- Library: 150 psf
- Stairs and elevator lobbies: 100 psf
- Restaurant and Dining: 100 psf

The live loads listed from Table 1607.1 are the minimum design live loads and the live load key plans included herein are the maximum live loads the spaces can support. The partition allowance of 15 psf is the minimum required by the current IBC. Slab on grade portions of the first floor can support high intensity public use and assembly spaces. Portions of the first floor framed over the partial basement will be limited to lower intensity live load uses such as office or classroom, or spaces for which occupancy can be limited. Alternately, the concrete pan joist and beam first floor framing can be reinforced using surface applied carbon fiber strengthening systems (QuakeWrap) to increase the design live load to 100 psf, and allow for more intensive uses. An alternate price for this work has been included within the cost estimate.

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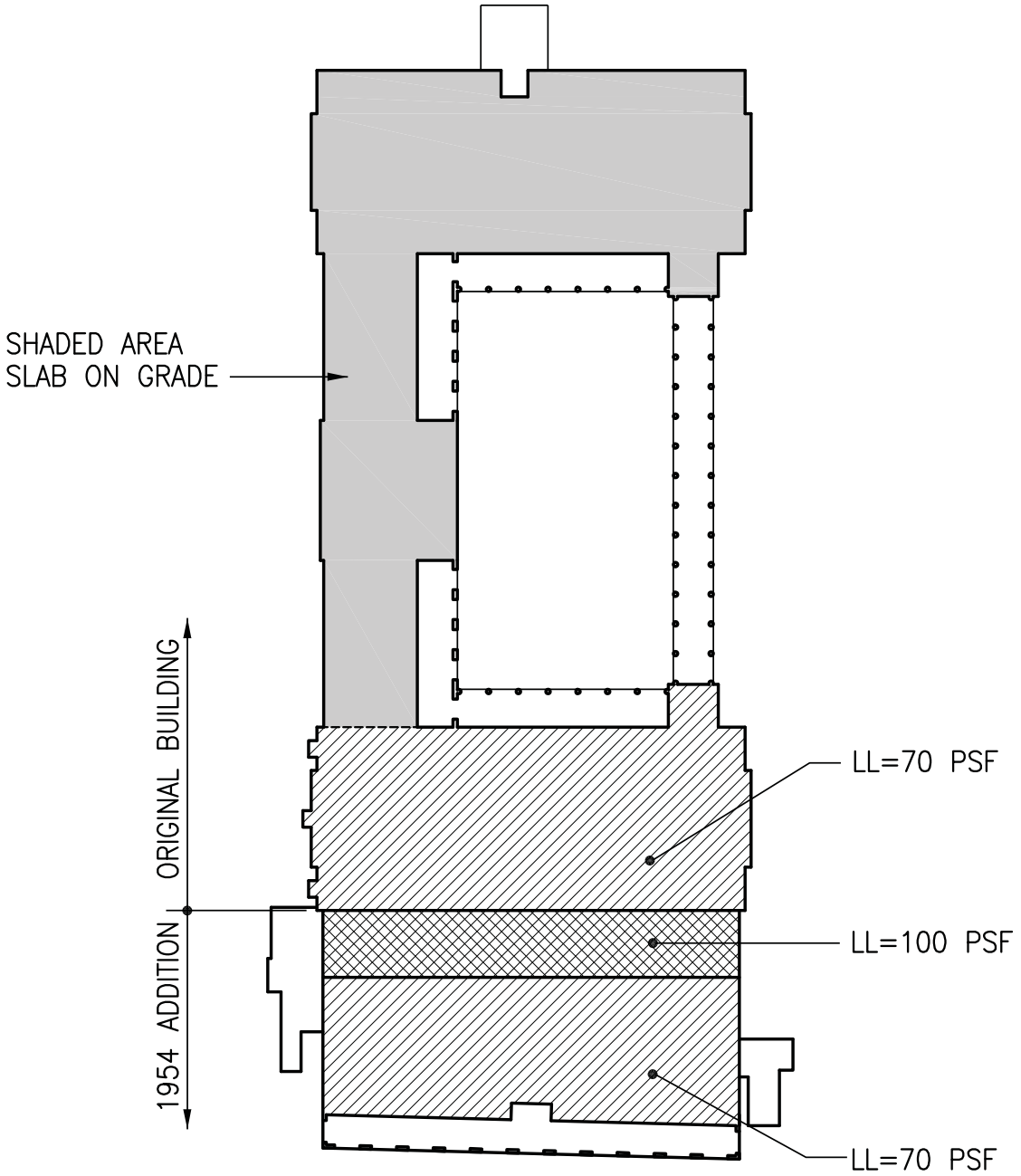
Project PIMA COUNTY COURT HOUSE Sheet 1 of 5

Client POSTER FROST MIRTO Job No. 15045

By WLW

Date 3/2/16



FIRST FLOOR - RECOMMENDED MAXIMUM LIVE LOAD

16' 0 16' 32'

NORTH

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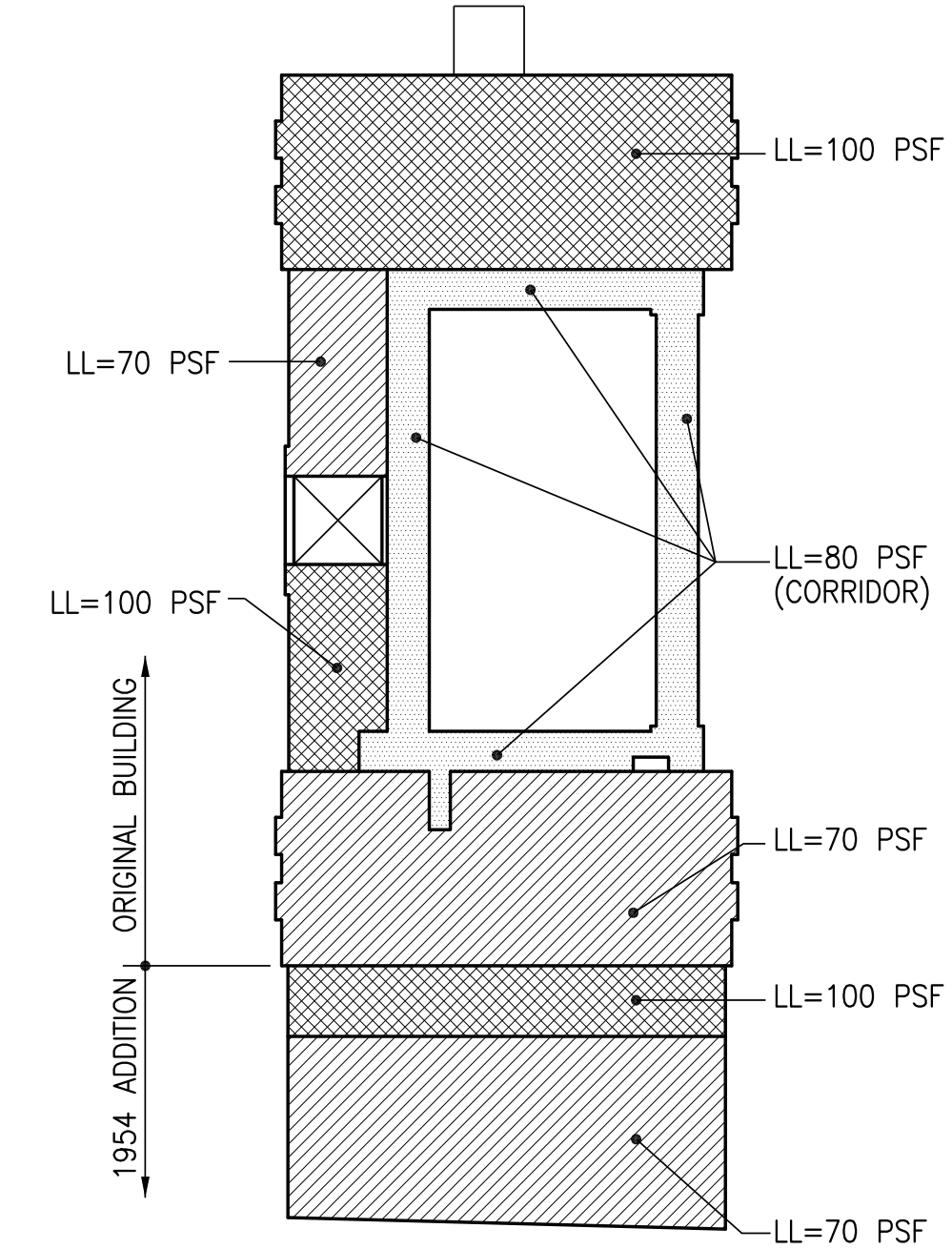
Project PIMA COUNTY COURT HOUSE Sheet 2 of 5

Client POSTER FROST MIRTO Job No. 15045

By WLW

Date 3/2/16



SECOND FLOOR - RECOMMENDED MAXIMUM LIVE LOAD

16' 0 16' 32'

NORTH

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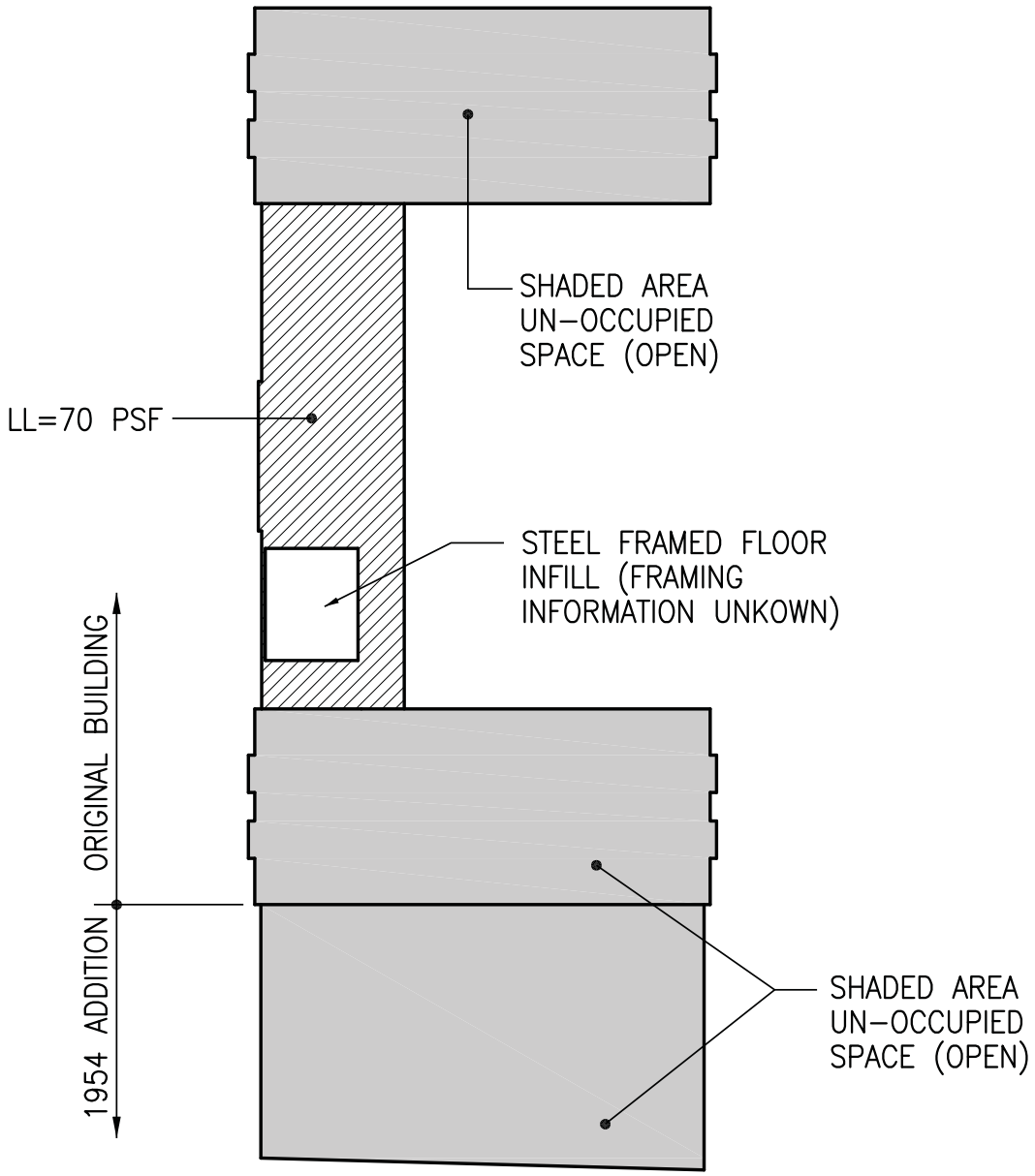
Project PIMA COUNTY COURT HOUSE Sheet 3 of 5

Client POSTER FROST MIRTO Job No. 15045

By WLW

Date 3/2/16



THIRD FLOOR - RECOMMENDED MAXIMUM LIVE LOAD

16' 0 16' 32'

NORTH

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Sheet 5 of 5
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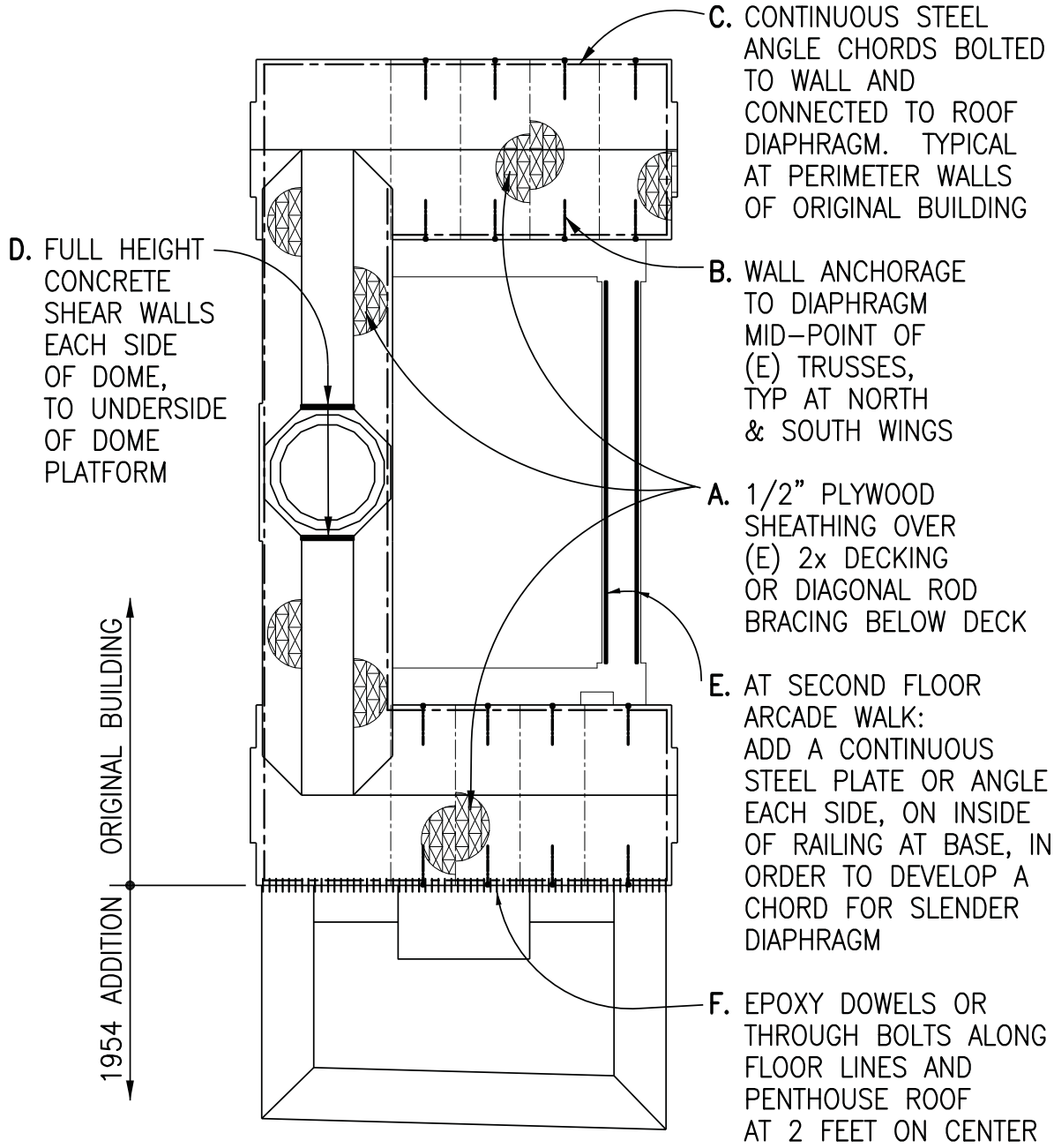

SUGGESTED STRUCTURAL STRENGTHENING / IMPROVEMENTS

16' 0 16' 32'

NORTH

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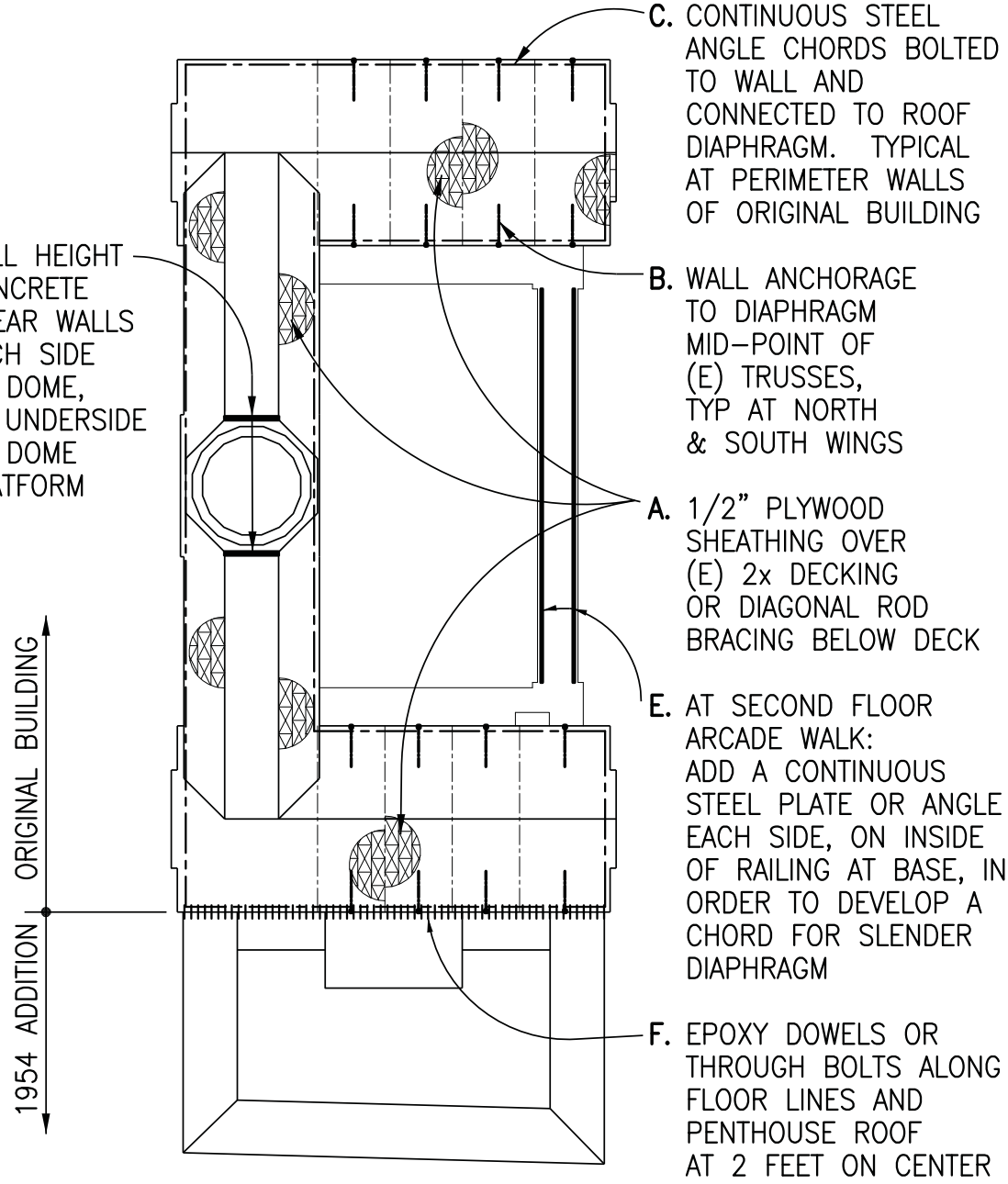
Project PIMA COUNTY COURT HOUSE Sheet 5 of 5

Client POSTER FROST MIRTO Job No. 15045

By WLW

Date 3/2/16



SUGGESTED STRUCTURAL STRENGTHENING / IMPROVEMENTS

16' 0 16' 32'

NORTH