San Juan County Courthouse

Seismic Evaluation

350 Court Street
Friday Harbor, WA 98250

REPORT

April 15, 2022

WJE No. 2019.5417

PREPARED FOR:

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BACKGROUND

At the request of the Facilities department of San Juan County (County), Wiss, Janney, Elstner Associates, Inc. (WJE) has performed a seismic evaluation of the San Juan County Courthouse located at 350 Court Street and Annex Building located at 135/145 Rhone Street in Friday Harbor, Washington.

BUILDING DESCRIPTION

The San Juan County Courthouse (Figure 1) is comprised of an original two-story building with two 1980s era expansions and one 1990s era addition. The original Courthouse building (Figure 2 and Figure 3), which has been placed on the National Register of Historic Places, was constructed in 1906. According to information provided by the County, in 1982, a two-story expansion was constructed at the north side of the original Courthouse building that included a lobby, an elevator, and restrooms as well as a one-story structure that provided office space. A few years later, a vertical expansion was constructed above the one-story expansion and consists of offices, a library, and a court room (Figure 4 and Figure 5). In 1996, the single-story Sheriff's Wing addition was constructed on the northwest side of the building (Figure 6 and Figure 7).

The original Courthouse building consists of unreinforced mass masonry wall construction. The floors and roof are reinforced concrete. A partial basement exists under the northeast half of the building. Reportedly, in the 1980s, some structural modifications were made to the original 1906 structure to improve seismic performance. The 1980s courthouse additions include reinforced brick masonry walls and wood-truss floor framing. The second story addition is constructed with wood framed walls and roof with steel columns. The second story exterior walls are sheathed with brick veneer that matches the reinforced brick masonry exterior walls at the first story. The sheriff's wing addition is a light-framed wood structure with brick veneer.

Structurally, these buildings can be considered three independent buildings: the original 1906 courthouse building, the 1980s courthouse additions, and the 1996 Sheriff's wing expansion. The added second story courthouse addition can be seismically evaluated a separate building from the original addition since the structural framing is different and more flexible than the story below.

Located approximately two blocks west of the courthouse, the San Juan County "Annex" (Figure 8 through Figure 11) is a one-story, concrete masonry unit building with a flat roof, believed to have been constructed in the 1940s or 1950s, that is utilized for various county department operations. One or more wood-framed expansions have been added on the west side of the building since original construction.

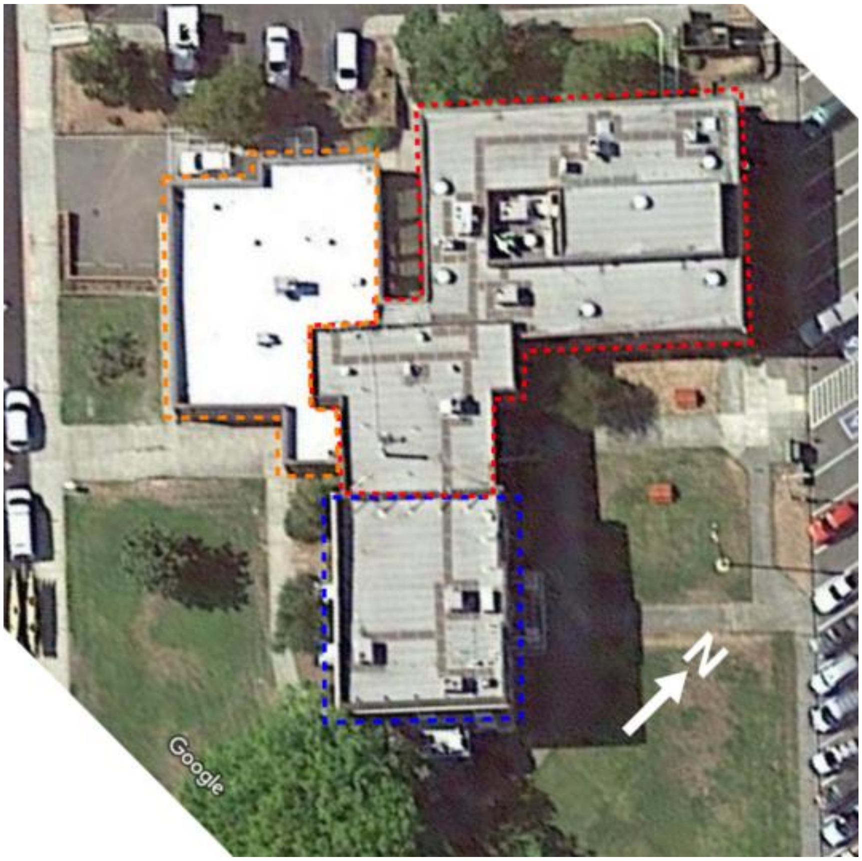

Figure 1. Aerial view of the San Juan County Courthouse © Google. The original 1906 Courthouse is outlined in blue, the 1980s Courthouse Expansion is outlined in red, and the 1996 Sheriff's Wing Addition is outlined in orange.

Figure 2. South elevation of the original 1906 Courthouse.

Figure 3. East elevation of the original 1906 Courthouse. The 1980s two-story addition can be seen on the north (right) side of the original building.

Figure 4. South and east elevations of the 1980s two-story addition. The original 1906 Courthouse building can be seen on the south (left) side of the building.

Figure 5. East elevation of the 1980s two-story Courthouse expansion.

Figure 6. West elevation of the 1996 one-story Sheriff Wing addition.

Figure 7. Looking south towards the two-story Courthouse expansion (left) and Sheriff Wing addition (right).

Figure 8. Aerial view of the San Juan County Courthouse Annex © Google. Taller roof area outlined in red.

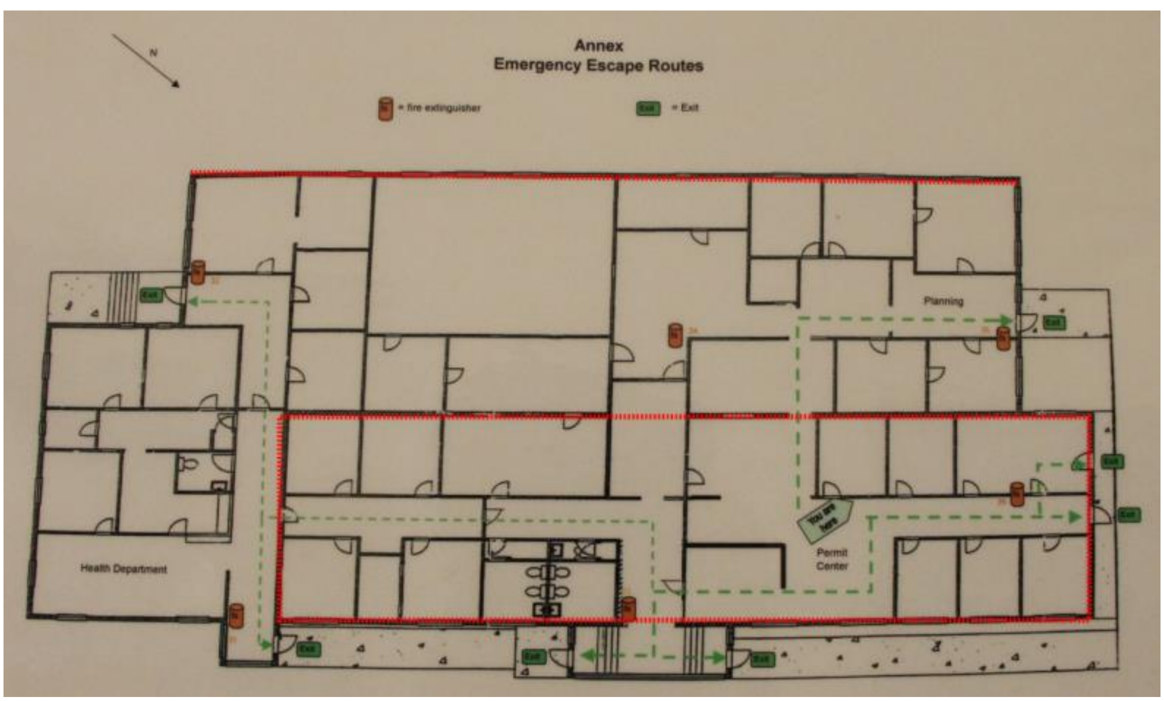

Figure 9. Courthouse Annex First Floor Plan. The red lines are annotated by WJE and indicate the location of CMU walls.

Figure 10. The east (left) and north (right) elevations of the Courthouse Annex building.

Figure 11. The west elevation of the Courthouse Annex building.

DOCUMENT REVIEW

In preparation of the seismic evaluation, WJE reviewed the following documents which were provided to us by the County:

- Courthouse expansion drawings titled *San Juan County Courthouse Remodel and Expansion - 1980* by Don L. McKee Architect, undated.
- Courthouse remodel drawings titled *San Juan Courthouse Phase II | Remodel of Existing Building* by Don L. McKee Architect dated March 5, 1982 with structural sheets by Michael E. Harkin, Consulting Structural Engineer.
- *National Register of Historic Places Inventory - Nomination Form* dated March 12, 1984.
- Courthouse second floor addition drawings titled *San Juan County Courthouse Second Floor Addition* by The Henry Klein Partnership, Architects dated May 23, 1984.
- Sheriff's Wing addition drawings titled *San Juan County Sheriff's Wing Addition* by The Henry Klein Partnership, Architects with structural sheets by Martens Chan Consulting Engineers dated February 21, 1996.

1980 Courthouse Expansion Drawings

The courthouse expansion drawings from 1980 available for our review were architectural sheets only and depict the first expansion to the courthouse. This expansion includes a two-story building with basement north of and adjacent to the original 1906 courthouse structure. The expansion also includes a one-story wing north of the two-story addition (Figure 12). An outline and note for "Future Building Height in Second Phase" is shown above the one-story wing. Exterior walls are shown as reinforced masonry with 7-1/2" wide brick units at the first and second levels. Concrete foundation walls are shown at the basement level and vary in thickness from 7-1/2" to 11-1/2".

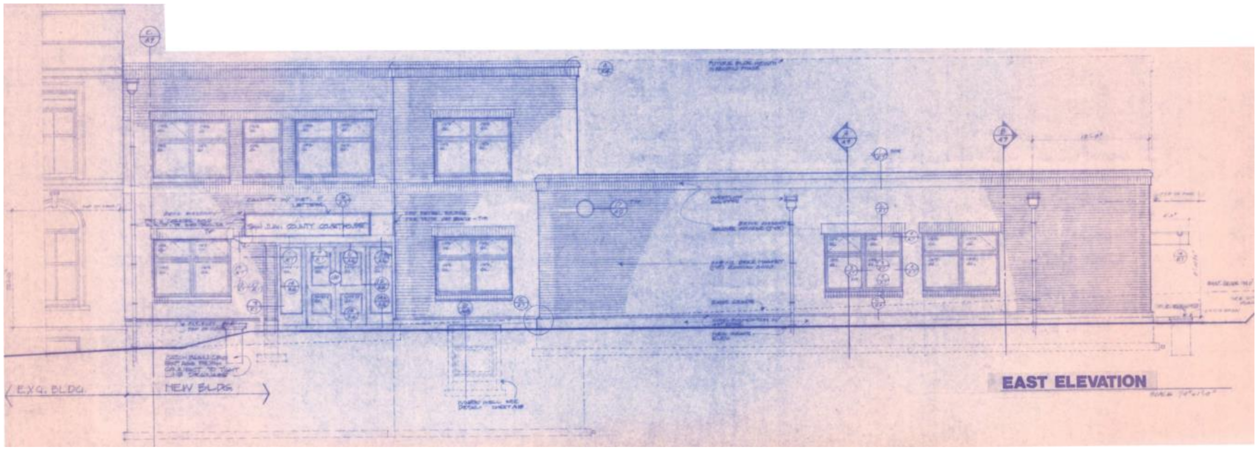

Figure 12. East Elevation of the 1980 courthouse expansion drawings taken from Sheet R-5. The original 1906 era courthouse can be seen on the left (south).

1982 Courthouse Remodel Drawings

The courthouse remodel drawings from 1982 depict renovations to the original 1906 era courthouse building (Figure 13). Structural sheets include documentation of the existing structure prior to the proposed renovations. The courthouse roof is shown as 4-inch-thick reinforced concrete slab spanning between 12 inches x 20 inches concrete beams which run east-west between exterior brick walls and 12-inch-square reinforced concrete columns. The roof slab is noted to be reinforced with wire fabric. The second level is also shown to have a 4-inch-thick slab reinforced with wire fabric. The second level slab spans between 12 inch x19 inch concrete beams which run north-south between exterior brick walls and interior reinforced concrete columns. The first level consists of a 4-inch-thick concrete slab-on-grade on the west side of the building and a 4-inch-thick slab elevated over the partial basement level on the east side of the building.

Exterior walls are shown as unreinforced brick masonry 1′-5-1/2″ thick at level 1 and 1′-0-1/2″ thick at level 2 and 10-inch-thick at typical roof parapets. Foundation walls below level 1 are shown to be "rubble" with thickness varying from 1′-6″ to 1′-11″. A vault space, approximately 19 feet x 22 feet in plan, occurs on the east side of level 1 and also has unreinforced brick masonry walls.

Proposed structural modifications in the 1982 drawings include the addition of steel beams at the roof level, second level, and first level (above the basement), new steel pipe columns with concrete footings at the basement level, and a new 8-inch-thick reinforced concrete wall at the west side of the basement level adjacent to the existing rubble foundation walls. New steel beams are shown attached to existing concrete beams with steel wedge anchor connections and are shown bearing at existing masonry walls at new grouted beam pockets.

The structural notes on Sheet S-6 states that the governing code at the time of the remodel was the 1979 Uniform Building Code and the building is noted to be in Seismic Zone 3.

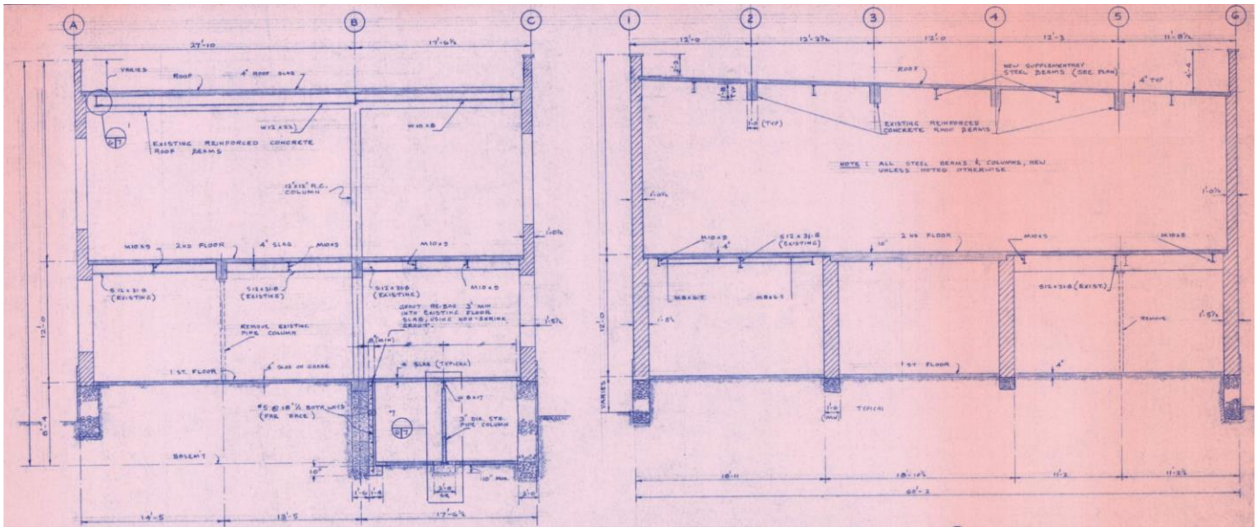

Figure 13. Building section of the original 1906 era courthouse building looking north (left) and looking west (right) from Sheet S-6 from the 1982 remodel drawings.

1984 National Register of Historic Places Inventory Nomination Form

The 1984 National Register of Historic Places Inventory Nomination Form provides a brief history of the building and notes that the original Courthouse building was constructed in 1906. The structure of the building is described as "reinforced concrete beams and floors resting on load-bearing brick walls and piers" with a "high rubble stone foundation which is covered in a cement veneer scored to imitate coursed stone". The report states that the original Courthouse building roof collapsed during construction and "the building was declared unsafe by summer of that first year and measures had to be taken to shore up the slanting floors". No further context is provided for why the building was "declared unsafe".

1984 Second Floor Addition Drawings

The 1984 Second Floor Addition drawings available for our review were architectural sheets only and depict the second floor addition to the 1980 building expansion. The second floor addition is located over the north wing of the 1980 Courthouse expansion. Exterior walls at the second floor are shown as 2x6 at 16-inch wood stud walls with 3-1/2"x3-1/2"x11-1/2" brick veneer with masonry anchors at 16 inches vertical and horizontal spacing. Steel pipe columns are shown at several locations within walls and Note 3 on Sheet 3 states "align interior columns directly above first floor columns". At the Superior Court room, the roof is raised to create a high ceiling (Figure 14). The roof framing, while shown only conceptually in the architectural drawings, consists of wood trusses (at 32-inch spacing per Detail 1 on Sheet 8), that span in the north-south direction between walls and/or glue-laminated wood beams.

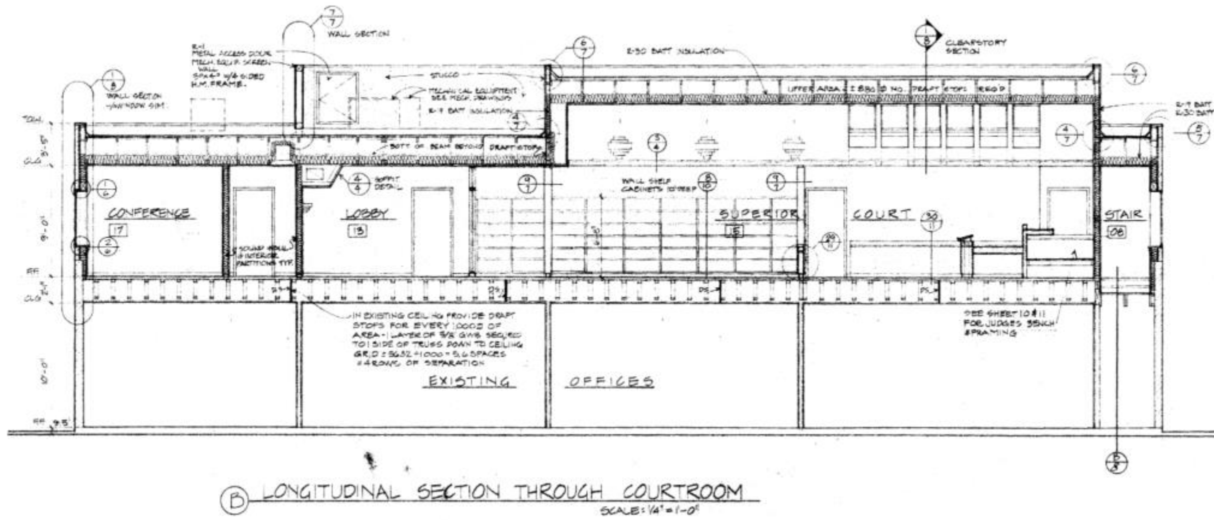

Figure 14. Detail B on Sheet 7 of the 1984 Second Floor Addition drawings depicts a section cut through the building looking North.

1996 Sheriff's Wing Addition Drawings

The 1996 Sheriff's Wing addition drawings show that this single-story addition was designed as an independent building with its own structural system including wood-framed exterior walls and interior steel pipe columns with a wood framed roof. The Sheriff's Wing addition is approximately 58 feet by 85 feet in plan at its widest. The framing members at the roof include wood truss joists, I-joists, and glue laminated beams and steel wide flange beams. The roof diaphragm is shown as 3/4"-thick tongue-and-groove (T&G) plywood. The first floor is shown as a reinforced concrete slab-on-ground. The lateral system of the building is wood shear walls with 1/2"-thick plywood on each side of shear wall. Wood shear walls are located at select locations on the east, west, north, and south exterior walls. Holdowns are shown at the end of each shear wall and 5/8" diameter anchor bolts are shown with spacings varying from 18 inches to 32 inches. A concrete stem wall with a 30-inch-wide concrete continuous strip footing is shown at the perimeter of the building supporting exterior walls and brick veneer. The structural General Notes state that the building was designed in accordance with the 1991 Uniform Building Code and Seismic Zone III with design coefficient C = 2.75.

SITE OBSERVATIONS

Mr. Zachary Stutts, PE, SE, and Mr. Brian Kehoe, PE, SE, of WJE, visited San Juan County Courthouse and Annex on February 3 and 4, 2022 as part of the seismic assessment. The observations made during this visit provided a limited visual assessment of the condition and configuration of the buildings. During the site visit, we reviewed copies of various construction drawings representing various phases of construction of the Courthouse. No construction drawings were provided for the Courthouse Annex. Relevant information obtained from the review of drawings is included in the *Document Review* section of this report.

WJE utilized ground penetrating radar (GPR) and a metal detector at select locations to detect the presence of steel reinforcing bars in concrete and masonry elements.

Original Courthouse Building

The exterior walls of the original courthouse building have experienced minor deterioration (Figure 15). Some areas of the mortar joints have been re-pointed; however, this re-pointing work has not matched the existing mortar (Figure 16).

Parapet bracing has been installed on all four sides of the roof. Diagonal steel angles spaced approximately 8 feet on center span between the parapet coping and the roof structure below (Figure 17). The roof membrane and coping metal obscured the connections.

Steel plate “rosettes” were observed at the second floor and roof levels from three sides of the building exterior (Figure 18) with rosettes also visible at the roof level on the fourth side. Rosette plates are typically used to tie wood floor framing to masonry walls. However, the 1982 remodel drawings show the roof structure to be concrete. Due to the presence of hard ceiling finishes, the construction type of the roof could not be verified.

There is a partial basement below the first level on the east side of the building. Steel framing has been installed in the basement apparently to support the Level 1 concrete slab (Figure 19). The 1982 remodel drawings show the Level 1 concrete slab as four inches thick, however, GPR scanning demonstrated the concrete slab to be approximately 7-1/2 inches thick. The steel framing includes W10 beams spanning north-south between W12 beams, which are supported by steel posts at the perimeter of the basement. At the perimeter of the basement, the steel framing is connected to the concrete beam above the stone masonry exterior foundation walls with bolted connections (Figure 20). The presence of more modern steel framing suggests that the gravity load of the Level 1 slab is intended to be supported by the retrofit steel framing.

There is a 1-1/2 inch wide gap separating the Courthouse Expansion building from the Original Courthouse building (Figure 21). The gap is filled with mortar or similar cementitious material.

As expected, no steel reinforcement was detected by nondestructive techniques within the brick masonry walls.

Figure 15. Minor deterioration of exterior brick below the windows.

Figure 16. Poorly matched repointing of mortar joints.

Figure 17. Steel angles (arrows) have been installed to brace the masonry parapet at all four sides.

Figure 18. Steel rosette plates (arrows) at the west side of the Courthouse at the roof level.

Figure 19. View of the Courthouse basement. Note the steel beams (orange arrows) supporting the floor above and stone walls (blue arrows) on the east and west sides of the basement.

Figure 20. Steel beams (blue arrow) supporting the Level 1 slab are connected to the concrete beam above the stone masonry foundation walls in the basement. The steel beams are also supported by steel pipes (orange arrow) at the perimeter of the basement.

Figure 21. Mortar-filled gap between the original courthouse building and the 1980s expansion.

1980s Courthouse Expansion

Brick masonry at the first level walls was measured to be 7-1/2" thick. At select locations along the first floor exterior brick masonry walls, GPR scanning detected vertical reinforcing bars at 30 inch spacing and horizontal reinforcing at 56 inches and 98 inches above the first level floor. At the concrete basement wall on the south side of the building, adjacent to the 1906 courthouse, GPR scanning detected vertical reinforcing bars at 16 inch spacing and horizontal reinforcing bars at 10 inch spacing. The concrete basement wall was determined to be approximately 6 inches thick based on GPR scanning.

At approximately ten locations through the building, WJE removed drop ceiling finishes to observe as-built conditions of the structure which were otherwise obscured by finishes. At conditions along exterior walls, wood floor or roof framing was typically connected to wood ledgers which were anchored to exterior walls with 7/8-inch diameter anchor bolts that were typically spaced at 12 inches (Figure 22 and Figure 23). At one location at the roof above the Clerk's office; however, anchor bolts were spaced up to 22 inches. At the Superior Court high roof, there is an unsheathed wood pony wall which supports the high roof framing.

As part of a separate investigation, the County had created an investigation opening in the south wall between the first and second stories (Figure 24). The opening confirmed that the wall framing at the first story is reinforced brick masonry (vertical steel reinforcement spaced at 24 inches) and the wall framing for the second story is light wood framing with brick veneer (Figure 25). A wood sill plate for the second story is attached to the reinforced brick masonry wall with expansion anchors.

Figure 22. Typical floor framing-to-wall connection (Arrow indicates top chord bearing on ledger).

Figure 23. Typical floor framing-to-wall connection.

Figure 24. Interior view of the investigation opening on the south wall of the Courthouse Expansion at the interface between the 1980 first level and 1984 second level.

Figure 25. Exterior view of the investigation opening on the south wall of the Courthouse Expansion at the interface between the 1980 first level and 1984 second level.

1996 Sheriff's Wing Addition

The Sheriff's Wing building appears to generally conform with the 1996 construction drawings as described in the *Document Review* section of this report. The exterior brick veneer walls include expansion joints that are open and weep holes at the base of the wall and above the windows (Figure 26). The ceiling is sheathed with gypsum board with limited access to observe the roof framing, which consists of pre-fabricated trusses with wood top and bottom chords and light gauge metal diagonal members (Figure 27).

Figure 26. Sheriff's Wing, expansion joints and weep holes in masonry veneer.

Figure 27. Sheriff's Wing, prefabricated roof truss (Arrows indicate top chord and diagonal members).

1950s Courthouse Annex

At the Courthouse Annex building, WJE observed conditions from the building interior, building exterior, and the south end of the crawl space. Due to the presence of hard wall and ceiling finishes, WJE was unable to view roof framing members and their connections. To be consistent with the directionality of the Courthouse Building, the entrance of the Annex building will be referred to as the east side of the building in this report.

The single-story Courthouse Annex building is generally rectangular shaped and approximately 70 feet by 140 feet in plan. Although the roof framing was not observable due to finishes, WJE identified at least two locations where glue laminated beams support the roof, which suggests that the rest of the roof is likely wood framed. There is a taller interior space on the west side of the building that has a roof several feet taller than the main roof level. This can be seen in the aerial imagery in Figure 8.

The buildings walls are light frame wood and CMU. Exterior wood-framed walls are sheathed with plywood siding. CMU wall locations are shown in Figure 9. The exterior walls and CMU walls are supported by concrete stem walls and strip footings at the foundation. The entire building is elevated above a crawl space. WJE observed 1/2-inch diameter anchor bolts at 48 inch spacing connecting the exterior wall sill plates to the concrete stem walls. Concrete stem walls at the interior of the building are unreinforced (Figure 28). The first floor is wood framed with 2x8 and 2x10 joists spanning between wood beams and/or the concrete stem walls. Wood beams are supported by 6x6 wood posts with concrete footings. The wood beams are typically connected to the posts with nailed plywood gussets (Figure 29), however in some cases, the beam just bears directly on the post with no side plates or steel connectors, or in other cases, a gusset is present on only one side of the connection. There are typically no mechanical connectors attaching the posts to the footings (Figure 29).

The walls below the high roof are supported by concrete foundations with the exception of the wall supporting the north end of the high roof which bears on Level 1 wood framing. We did not observe any structural damage or deterioration of the wood framing or CMU that would affect the structural capacity of the building. The first floor beams that support the framing for the western section of the building are supported by wood posts adjacent to, but not connected to the concrete stem wall (Figure 30). The interior cripple walls supporting the first floor framing are not sheathed and the sill plates are not anchored to the strip footings (Figure 27).

At select locations on the CMU walls, WJE used GPR to scan for reinforcing bars. GPR scans revealed that the CMU walls are unreinforced with the exception of the west exterior wall. The west exterior CMU wall has grouted cells with vertical reinforcing bars spaced at 48 inches and horizontal reinforcing at 48 inches.

The CMU walls provide the primary resistance to lateral (wind and earthquake loads) for this building. However, the wood framed walls supporting the high roof must also resist lateral loads imposed by the high roof diaphragm.

Figure 28. Existing opening previously chipped in the concrete stem wall in the crawl space reveals no reinforcing bars present.

Figure 29. Posts supporting the first floor framing bear on concrete footings with no mechanical connection to the footing. Plywood gussets can be seen connecting the beams to the wood post (arrow).

Figure 30. Beam supporting first floor framing for the west side of the building not attached to the interior concrete stem wall.

Figure 31. Interior wood-framed cripple wall in the crawl space without sheathing and without anchors into the concrete.

PRELIMINARY SEISMIC EVALUATION

Current methodologies for evaluating the seismic performance of buildings consider a variety of performance levels and a variety of earthquake hazards. The earthquake hazards are generally described in terms of expected recurrence intervals for the ground motion. Typical values of recurrence intervals used for the evaluation of existing buildings and design of new buildings range from 250 years to 2475 years. These are also expressed in terms of probability of exceedance during a 50-year time period where a 250-year recurrence represents a 20 percent chance of being exceeded in 50 years and a 2475-year recurrence represents a 2 percent chance of being exceeded in 50 years. The earthquake ground motions associated with these hazards for a site within the United States are available from the U.S. Geological Survey (USGS). The expected structural performance levels range from Collapse Prevention, which is considered a building that does not collapse during the specific earthquake event of interest but is likely to be unusable following the event, to Immediate Occupancy, which represents a building that may experience some minor damage but would be useable after an earthquake.

The seismic evaluations of the Courthouse and Annex are based on the requirements from the standard *ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings*. Seismic demands are based on site-specific data available from the USGS regarding seismic hazard and site conditions. The preliminary seismic evaluation of the building was performed using the Tier 1 procedures of ASCE 41. The Tier 1 evaluation consists of assessing a series of checklist statements to evaluate the building for potential seismic vulnerabilities. The building is classified by ASCE 41 based on the construction materials and the configuration of the lateral force-resisting system.

The analysis assumes that the use and occupancy of the buildings will remain in their current state. To the extent necessary for Tier 1 evaluation, limited evaluation of nonstructural components was also performed. Based on this analysis, we identified structural characteristics or elements of the building that are deficient using the ASCE/SEI 41-17 evaluation criteria.

Criteria

The structural performance objective for the building is Collapse Prevention or Life Safety according to Risk Category IV, as defined by the *Washington State Existing Building Code*. The Collapse Prevention performance level uses an earthquake hazard described as the BSE-2E, as defined by ASCE 41. The BSE-2E earthquake hazard is defined as an earthquake with a 5 percent probability of being exceeded in 50 years, which is also referred to as an earthquake with an average return period of 975 years. Seismic parameters for the BSE-2E earthquake were obtained from data provide by the U.S. Geological Survey (USGS). These seismic parameters are modified to account for the soil conditions at the site. The soil conditions at the site were assumed to be Site Class C, based on our available information for similar sites in Western Washington. ASCE 41 defines four levels of seismicity, Very Low, Low, Moderate, and High. As a result of the ground shaking demands and the soil Site Class, the site of the building is assigned a seismicity of High.

EVALUATION RESULTS

The ASCE 41 Tier 1 procedure requires completion of two or more checklists based on the building type. Based on our document review and site observations, the Courthouse building can be considered as three independent buildings: the 1906 era original Courthouse, the 1980 and 1984 Courthouse expansion, and the 1996 Sheriff Wing addition. For the Tier 1 procedure, the original Courthouse building is defined as a type URMa (Unreinforced masonry with stiff floors) building, the Courthouse expansion is defined as a type RM1 (Reinforced Masonry with a flexible diaphragm) building at the first story and a W2 (Wood frame commercial or industrial) building at the second floor, and the Sheriff Wing addition is defined as a type W2 (Wood frame commercial or industrial) building. The Courthouse Annex building is defined as a type URM (Unreinforced Masonry with flexible diaphragm) building. The applicable ASCE 41 checklists are the Collapse Prevention Basic Configuration Checklist (Table 17-2) for each building and the Collapse Prevention Structural Checklist for Building Type URMa (Table 17-36) for the original Courthouse Building, RM1 (Table 17-34) for the 1980s Courthouse expansion, W2 (Table 17-6) for the 1980s second story expansion for the Courthouse, W2 (Table 17-6) for the 1996 Sheriff's Wing Addition, URM (Table 17-36) for the Courthouse Annex. A copy of the completed checklists are attached in Appendix A.

Table 1. Building Construction Types

Building	Type		ASCE 41-17 Checklists
1906 original Courthouse Building	Unreinforced Masonry Bearing Walls (with Stiff Diaphragms)	URMa	Table 17-2, Table 17-36
1980s Courthouse Expansion	Reinforced Masonry Bearing Walls (with Flexible Diaphragms)	RM1	Table 17-2, Table 17-34
1980s Courthouse Second Story Expansion	Wood Frames, Commercial and Industrial	W2	Table 17-2, Table 17-6
1996 Sheriff's Wing Addition	Wood Frames, Commercial and Industrial	W2	Table 17-2, Table 17-6
1950s Courthouse Annex	Reinforced Masonry Bearing Walls (with Flexible Diaphragms)	URM	Table 17-2, Table 17-36

WJE evaluated the building for the statements in the checklists and we found that all four buildings comply with most of the required checklist statements. The noncompliant checklist statements represent potential seismic deficiencies. These deficiencies may require further evaluation to assess their significance to the seismic performance of the building or may require seismic strengthening. The nonconforming statements identified in our evaluation are listed below with a brief description of the nonconformance.

Common Deficiencies

The original courthouse building, the 1980s expansion, the 1980s second story expansion, and the sheriff's wind addition are all constructed with no apparent seismic joints separating the buildings. As a result, they are not compliant with the requirement for providing a gap of 1.5 percent of the building height between buildings. Because of the difference in construction of these buildings, their responses to seismic shaking will be different and they will have a tendency to pound against each other during an earthquake. This will likely result in damage to the nonstructural features that cross the gaps between the buildings.

Original Courthouse Building

The original courthouse building is constructed with unreinforced masonry walls and concrete floor framing. In addition to the insufficient separation between the adjacent 1980s courthouse building on the north side, several additional deficiencies were noted in the Tier 1 screening.

Wall Proportions

Scaled measurements from the 1982 remodel drawings show that the unreinforced masonry walls are up to 16′-4″ at the second floor feet tall and the thickness of the walls is 12-1/2 inches. The ratio of height to thickness is approximately 16. The maximum height-to-thickness ratio compliance limit for the top story of an unreinforced masonry building is 9 and therefore the proportions of the second level walls are not compliant. First level walls, however, meet proportion compliance criteria. The excessive height-to-thickness ratio could lead to out-of-plane failure of the walls during strong earthquake shaking.

Shear Stresses

Tier 1 structural analysis demonstrated that shear stresses in the unreinforced masonry walls would be expected to be between 60psi and 70psi at the second level walls and 40 to 50psi at first level walls for a BSE-2E level earthquake. The maximum shear stress allowed for unreinforced masonry shear walls is 30psi and therefore the walls are not compliant.

Diaphragm Connections

The construction of the connection between concrete roof and floor diaphragms to exterior shear walls is shown conceptually in the 1982 remodel drawings but details are not explicitly known and could not be verified on site. Based on the age of construction of the building and the proposed extent of structural renovations in the 1982 remodel drawings, it is unlikely that the diaphragms are connected for transfer of seismic forces to the shear walls. It is also unlikely, yet unknown if, the exterior walls are anchored for resisting out-of-plane forces from the exterior walls at diaphragms with anchors that are developed into the diaphragm.

Diaphragm Openings at Shear Walls

The stair opening on the east side of the second level creates an opening in the diaphragm that is 11 feet long in the direction of the wall, exceeding the compliant diaphragm opening length of 8 feet adjacent to a shear wall. The length of the opening could result in out-of-plane damage to the section of wall along the stair opening since it lacks a structural connection along this opening.

Beam Supports

With the exception of floor framing at the basement level, concrete and steel beams supporting floors and roofs at the unreinforced masonry walls do not have independent secondary columns for support of vertical loads. This is not compliant since seismic damage to the masonry walls could result in loss of support for the floor second floor and roof.

1980s Courthouse Expansion

In addition to the insufficient separation between the 1980s courthouse expansion building and the original courthouse and Sheriff's Wing, a summary of additional potential deficiencies based on the Tier 1 screening are described below.

Reinforcing Steel

The two-story addition to the original courthouse building is constructed with reinforced masonry exterior walls. The size of the reinforcing bars is assumed to be consistent with the reinforcing steel exposed at the wall opening. The spacing of the reinforcing was assessed based on testing with GPR. Based on this testing the amount of reinforcing in the walls is not sufficient to meet the minimum reinforcing requirements. If the amount of reinforcing steel does not meet the minimum requirements, the walls would need to be considered as unreinforced.

Diaphragm Openings at Shear Walls

The stairwell and elevator at the southwest corner of the addition creates an interruption in the connection of the second floor to the exterior reinforced masonry shear walls. The stairwell that was added at the east side with the second story addition also interrupts the connection of the second floor to the exterior reinforced masonry shear wall at the east side.

In the portion of the building that was originally constructed as a single-story building, the original roof has long spans. There is insufficient information to assess whether the floor diaphragms were adequately detailed with cross ties. This has been assumed to be deficient.

Torsion

The majority of the exterior masonry shear walls are located near the southwest corner of the building. This creates a torsional irregularity since the estimated center of mass (considering the original building and the second story addition) is more than 20 percent of the building width from the center of stiffness. This has been assumed to be deficient.

Shear Stresses and Narrow Wood Shear Walls

The second story addition is a wood-framed structure with exterior brick veneer. The exterior walls are sheathed with plywood and these walls are considered to provide the primary resistance to lateral forces. The Tier 1 evaluation indicates that the quantity of plywood shear walls is insufficient creating an overstress in the walls for resisting the required seismic loads. Several of these plywood shear walls are also relatively narrow (height-to-width ratio greater than 2). Reliance on these narrow shear walls is also a deficiency.

Load Path Discontinuity

The portion of the roof over the courtroom in the second floor addition is raised relative to the roof height over the remainder of the roof. The details for the connections of this raised portion of the roof to the lower roof could not be verified. Clerestory windows frame between the upper and lower roof levels along one side of the upper roof. This vertical offset creates a discontinuity in the load path for seismic forces from the upper roof. This has been assumed to be deficient

1996 Sheriff's Wing Addition

The Sheriff's Wing addition is a relatively modern building that was reportedly designed to meet the seismic requirements of the 1991 Uniform Building Code (UBC). The ASCE 41 Tier 1 screening procedure allows some modern buildings to be classified as benchmark buildings, indicating that if those buildings were designed and constructed in accordance the minimum building code requirements for the specific building type, the building is considered to meet the ASCE 41 evaluation requirements. For a W2 building, which is the applicable building type for the Sheriff's Wing Addition, the minimum design requirement as a benchmark building is for the design to be in conformance with the 1976 UBC or later building code. Since the Sheriff's Wing Addition was designed using a more recent edition of the UBC, this building would be considered to meet the ASCE 41 performance objective.

It should be noted however, that the common deficiency related to the lack of a separation joint also applies to this building. The drawings show that the building was to be constructed with a 2-inch gap between the Sheriff's Wing and the 1980s Courthouse Addition. This gap is slightly deficient compared with the ASCE 41 criteria, which requires a gap of 1.5 percent. For a 12-foot story height, the required gap is 2.16 inches. An expansion joint appears to exist at the roof level (Figure 32) and along the exterior walls (Figure 33). The width of the gap should be confirmed.

Figure 32. Horizontal expansion joint between the roof of the Sheriff's Wing and the exterior wall of the 1980s Courthouse Addition.

Figure 33. Vertical expansion joint between the exterior wall of the Sheriff's Wing and the exterior wall of the 1980s Courthouse Addition.

1950s Courthouse Annex

We identified several deficiencies in the Tier 1 evaluation. Several other potential issues could not be confirmed because of lack of as-built drawing information and construction that was concealed by finishes. For most of these unknown conditions, we have used judgment to estimate the likely conditions. A summary of the potential deficiencies based on the Tier 1 screening are described below.

The majority of the CMU walls were found to be unreinforced and therefore the building is evaluated as an unreinforced masonry building. Only the west wall was identified as being reinforced. We were not able to identify the size of reinforcing used for the west wall, but considering that the spacing of the vertical and horizontal reinforcing is at 48 inches, it would be unlikely that the amount of reinforcing steel in that wall meets the minimum amount of reinforcing required by the Tier 1 screening procedure for a reinforced masonry building.

Load Path

Wood-framed walls supporting the high roof on the west side of the building must resist lateral loads imposed from the high roof diaphragm. Because the high roof north wall does not continue down to the foundation, the lateral load path is interrupted. Elsewhere in the crawl space, unsheathed cripple walls lack sufficient strength and stiffness to transfer lateral loads to the footings below. It is unclear if the connections and strength of the level 1 floor diaphragm are sufficient to provide an adequate load path for lateral loads to reach the foundation walls and this is an assumed deficiency.

Proportions

The unreinforced CMU walls are about 9-1/2 feet tall and the thickness of the walls is 7-5/8 inch. The ratio of height to thickness is about 15. The maximum height-to-thickness ratio allowed for unreinforced masonry for a single-story building is 13 and therefore the proportions of the walls are not compliant.

Girder-Column Connection

The girders supporting the first floor framing were observed to not have direct positive connections using plates, connection hardware, or straps between the girder and the column support as required by the Tier 1 screening procedure. There is also no positive connection between the bases of the posts to the foundations.

Roof Diaphragm Details

The Tier 1 procedure includes several requirements for the detailing of flexible wood diaphragms and their connections to the masonry walls. The details of the roof framing were not accessible for review and as a result, these conditions have been judged to be unknown. These include the presence of cross ties between diaphragm chords, limitations on the aspect ratio of straight sheathing used for the roof diaphragm, and the spans of the diaphragms. Based on the age of the building and details of construction that are observable, we believe that some of these diaphragm detailing conditions are not compliant.

Nonstructural Building Components

Nonstructural building components, where visible and readily accessible, were evaluated for Life Safety criteria in accordance with ASCE 41-17 Table 17-38.

Suspended ceilings

At the courthouse expansion building, suspended ceilings are not braced to resist lateral seismic forces. At the Sheriff's Wing, ceiling finishes are not suspended. At the original courthouse and at the courthouse annex, ceiling finishes attachment could not be verified but are assumed to not be suspended. Therefore, the suspended ceilings are considered deficient only at the courthouse expansion building.

Masonry Veneer Ties

At the 1980s courthouse expansion second level, it is unknown how well the brick veneer is tied back to the wood framed walls. The Life Safety compliance criteria requires corrosion-resistant ties for masonry veneer every 2.66 square feet, with spacing of ties no greater than 36 inches. Given the age of construction of this building, it is unlikely existing masonry veneer ties meet this criteria and is assumed to be deficient

Parapets

The original courthouse building is the only building with significant parapet height. Steel parapet bracing is installed approximately every eight feet around the perimeter of the original courthouse roof. However, this does not technically meet the compliance criteria of parapet bracing of six feet or less.

RECOMMENDATIONS

Original Courthouse Building

Many of the deficiencies identified in the original courthouse building are the result of the lateral force resisting system being unreinforced masonry walls. Some of these deficiencies can be addressed through installation of vertical reinforcing bars into the existing brick walls, effectively turning the lateral force resisting system into reinforced masonry walls. However, this approach requires extensive drilling and grouting as reinforcing would be required at a regular spacing; other approaches may be more cost-effective. This however, may be less visually obtrusive and may be desirable to minimize the impact on the historic fabric of the building.

The second level walls have height-to-thickness ratios which exceed the recommended limit which suggests potential for out-of-plane deformation of walls due to seismic forces. To address this vulnerability, vertical steel “strongbacks” can be installed between windows to stiffen and strengthen the walls out-of-plane. “Strongback” steel elements should be tied to the wall with steel anchors epoxied into the existing brick as well as anchored to the second level and roof diaphragms. The required spacing of strongbacks will depend on the strength and stiffness of steel elements used.

To address walls which are anticipated to be subjected to shear stresses higher than 30psi, walls can be strengthened with reinforced shotcrete or fiber reinforced polymer (FRP) wraps on one side of the wall. However, given the historic significance of this building, these methods may not be palatable. Alternatively, testing of shear strength of the in-situ bricks may provide justification for allowing shear stresses greater than 30psi. It may also be possible to tie the 1906 courthouse building to the 1980s era two-story building to the north to take advantage of the lateral stiffness and strength of the adjacent building’s reinforced masonry walls, however this approach would require further analytical study.

Because the connection of diaphragms to walls is unknown and likely deficient, diaphragm-to-wall connections should be enhanced. Lateral forces can be transferred from the concrete diaphragms to the brick masonry walls via steel elements, such as an angle, along the perimeter of the building interior. The steel elements should be anchored to both the concrete diaphragms and brick masonry walls with regularly spaced fasteners, the size and spacing of which would be determined by further analysis. Out-of-plane forces can be resisted with new steel straps on the roof anchored and placed perpendicular to the brick masonry walls. The steel straps would be spaced approximately four to six feet on center around the perimeter of the roof and would be attached to the walls and roof with steel fasteners.

The stairwell on the east side of the building creates an opening in the second floor diaphragm adjacent to the east exterior brick wall which is larger than recommended. Large openings in diaphragms adjacent to walls may cause stress concentrations within the diaphragm and prevent the exterior walls from being braced continuously. To address this deficiency a horizontal structural element, such as a steel channel or concrete beam, can be installed along the wall at the stairwell and connected to the diaphragm at the north and south end of the opening. The structural element should be anchored to the diaphragm and to the brick masonry walls with regularly spaced fasteners.

Concrete and steel beams at the roof and second level bear on the unreinforced masonry walls at the perimeter of the building with no independent secondary support which is considered a deficiency. The intent of this provision is to provide alternate load paths for gravity loads should there be localized failure of the wall. To address this deficiency, steel posts can be installed below concrete and steel beams at the perimeter of the building below the roof and second floor framing. Installation of steel posts should continue to the building foundation and may need to be embedded within the first level walls to sufficiently support the floor and roof framing.

The proposed locations of these conceptual retrofit components described above is presented in Figure 34 and Figure 35.

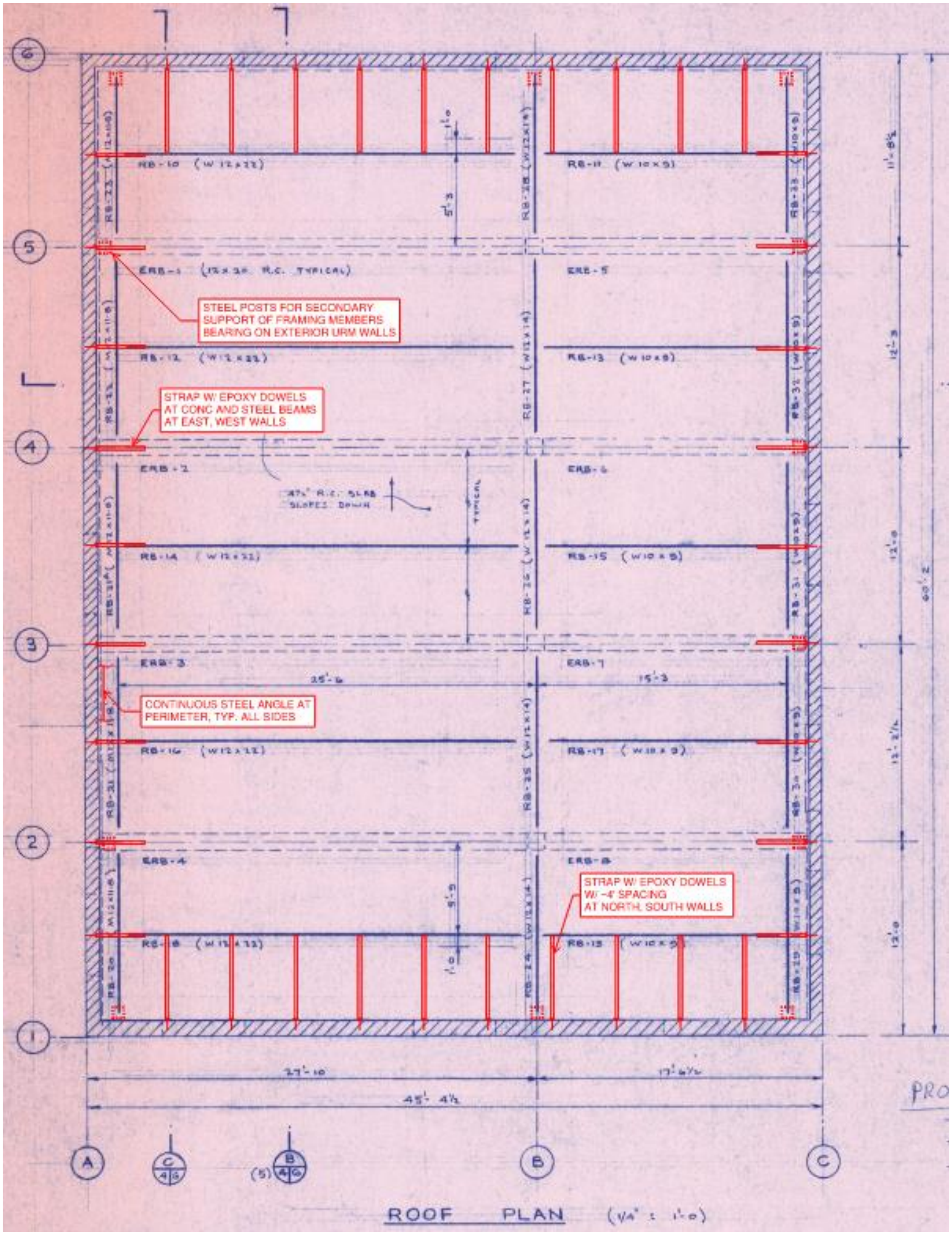

Figure 34. Courthouse Roof Plan from Sheet S-4 of the 1982 courthouse remodel drawings. WJE annotations are shown in red describing proposed conceptual retrofit options to address seismic deficiencies identified.

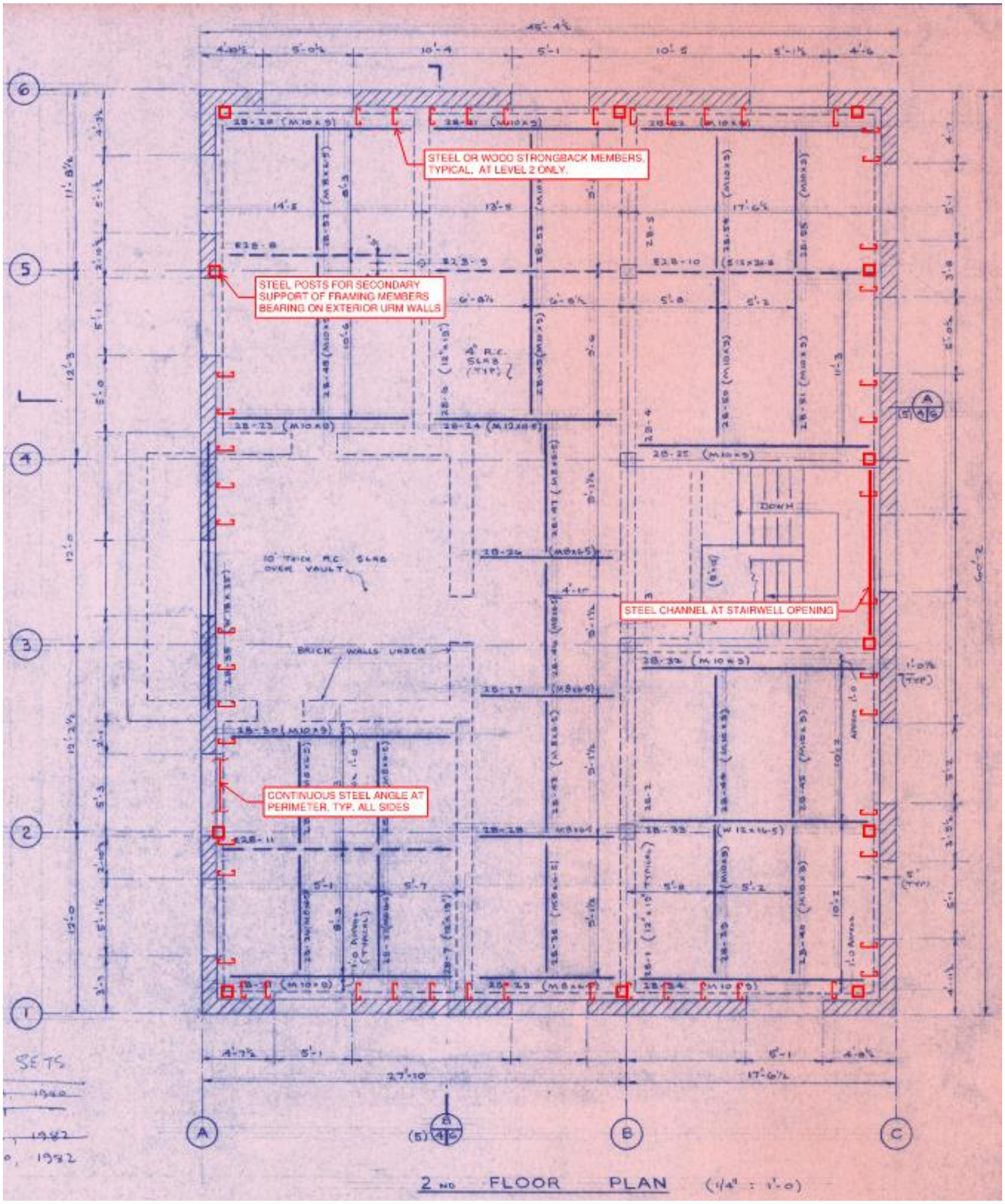

Figure 35. Courthouse Second Floor Plan from Sheet S-4 of the 1982 courthouse remodel drawings. WJE annotations are shown in red describing proposed conceptual retrofit options to address seismic deficiencies identified.

1980s Courthouse Expansion

The deficiency in the strength of the second story addition can be mitigated by the addition of new shear walls in the north-south direction that will extend from the roof to the ground floor. These walls would replace existing walls. The proposed new shear wall configuration is shown conceptually in Figure 36 and Figure 37. In the east-west direction, the interior steel columns and associated steel and glulam beams can be removed and replaced with new steel moment-resisting frames that extend from the roof to the foundation. The new steel columns would be installed where the existing columns are located to minimize disruption of the space, however, the steel columns of the new moment-resisting frames will be larger and will need to be protrude past the existing walls that currently hide the existing columns.

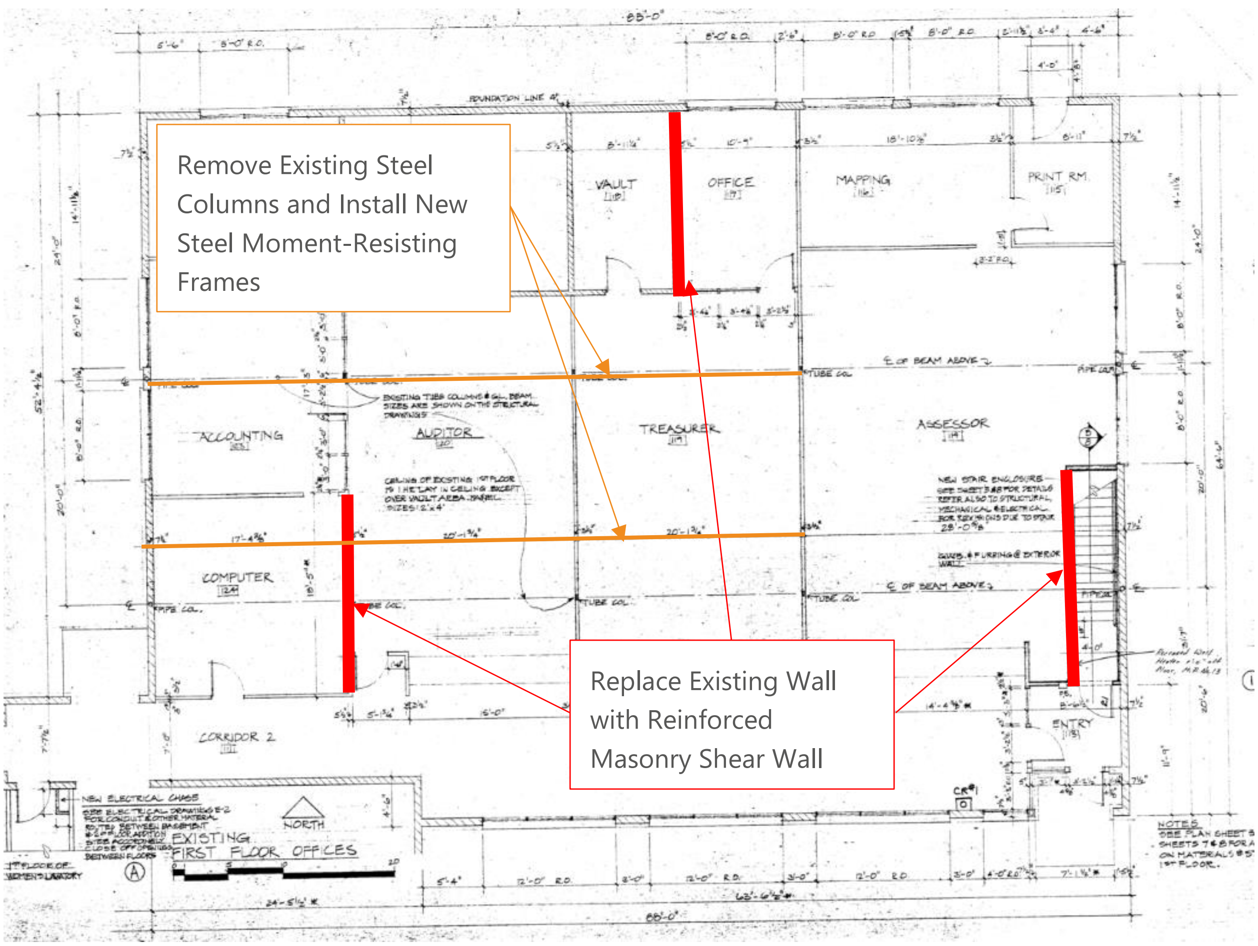

Figure 36. Proposed First Story Strengthening Concept for the 1980s Courthouse Addition

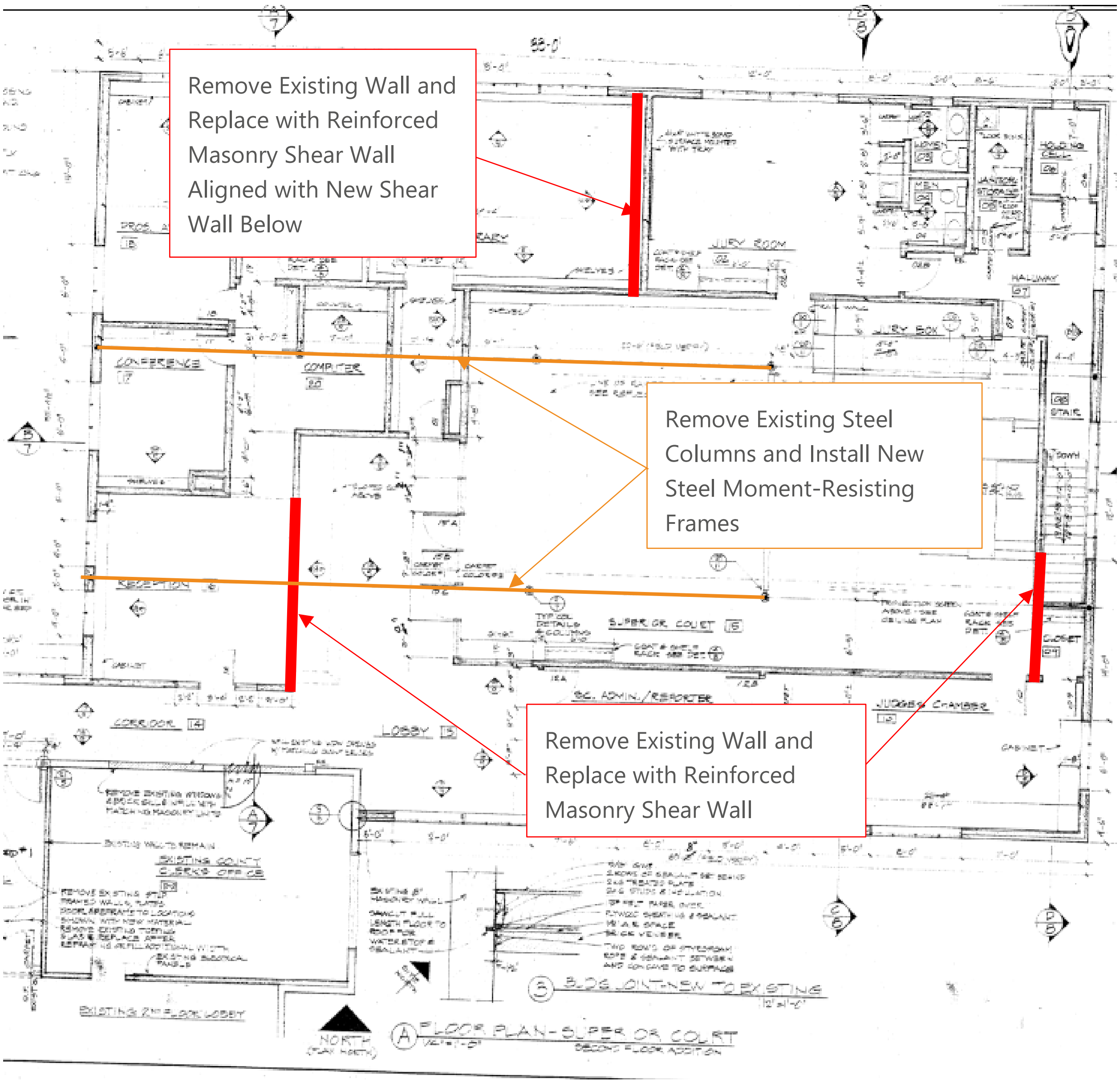

Figure 37. Proposed Second Story Strengthening Concept for the 1980s Courthouse Addition

1996 Sheriff's Wing Addition

The Sheriff's Wing is a relatively modern building that appears to have been designed considering seismic resistance. The building also appears to have been constructed with a seismic separation to allow the building to move independently from the adjacent 1980s Courthouse Expansion. The width of the gap is slightly deficient compared to the requirements of ASCE 41, but the difference is relatively minor and is not expected to adversely affect the seismic behavior of the buildings. As discussed in the Evaluation section of this report, the Sherrif's Wing Addition is considered to meet the ASCE 41 performance objective.

1950s Courthouse Annex

We identified several deficiencies and several unknown conditions that are likely deficient. Within the crawl space, we recommend that all of the wood posts have new steel post bases and post caps installed. A conceptual detail for the post base retrofit is shown in Figure 38 and a conceptual detail for the retrofit post cap connecting the posts to the beams is shown in Figure 39. In addition, new anchor bolts connecting the sill plates in the crawl space to the underlying footings should be installed, and selected portions of the wood-framed cripple walls should be sheathed with plywood. Further analysis may be required to address load path discontinuities presented by the vertically discontinuous high roof north wall, but possible retrofit solutions may include addition of sheathed stud walls, wood blocking, and concrete footing below the high roof north wall.

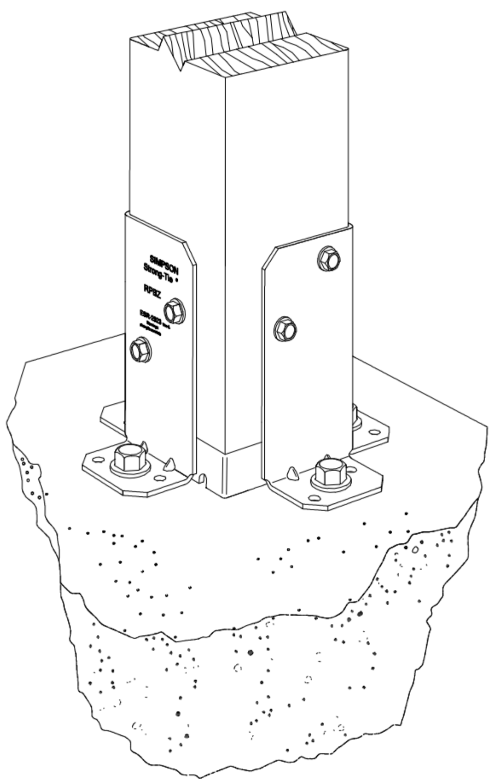

Figure 38. Conceptual retrofit detail for wood posts within the crawl space.

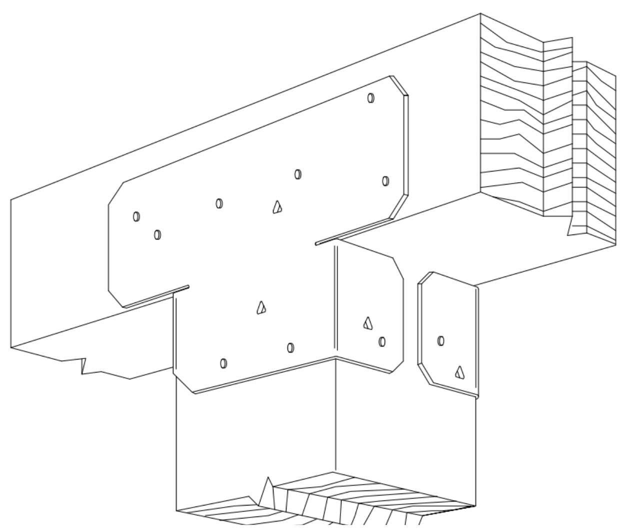

Figure 39. Conceptual retrofit detail for connecting wood posts to the wood floor beams within the crawl space.

Several options are possible for increasing the strength of the existing shear walls as well as addressing the lack of reinforcing steel in most of the CMU walls. The interior CMU walls can be removed and replaced with wood-framed walls with plywood sheathing. The exterior CMU walls can be grouted and reinforcing steel can be added so that the walls have sufficient shear strength, and so the reinforcing meets the minimum reinforcing steel requirements. As an alternative to adding reinforcing within the walls, steel framing could be added along the inside face of the wall to provide bracing.

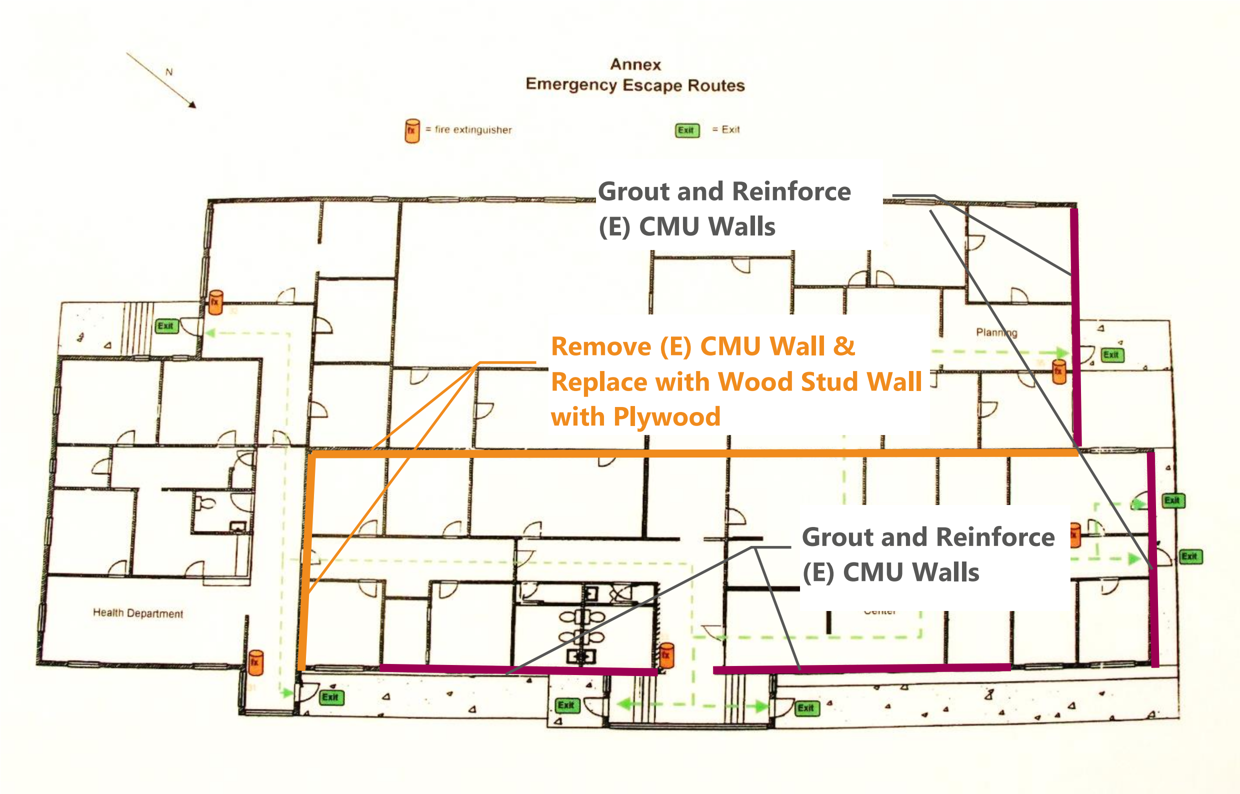

Figure 40. Conceptual Seismic Strengthening of Shear Walls at first story of the Annex Building, annotated in orange, over existing escape route plan developed previously by the County.

Since details for the roof framing are unknown, destructive openings will be needed to identify the configuration of the roof framing and the connections of the roof framing to the shear walls. We anticipate that plywood sheathing will need to be added to existing roof sheathing or existing roof sheathing will need to be replaced with plywood sheathing. Along the sides of the roof, new connections will be needed to anchor the CMU walls to the roof diaphragm to transfer shear forces from the roof to the shear walls and to provide lateral restraint for the tops of the CMU walls.

Nonstructural Building Components

Suspended ceilings

Suspended ceilings should have attachments that resist seismic forces for every 12 square feet of area. This is traditionally accomplished with diagonal tie wires anchored back to the soffit of the structure above.

Masonry Veneer Ties

At the 1980s courthouse expansion second level, supplemental corrosion-resistant ties for masonry veneer can be installed such that there is a tie every 2.66 square feet with spacing of ties no greater than 36 inches. There are several proprietary retrofit masonry tie options on the market, including helical steel ties or steel straps which are fastened to the wood studs and anchored into the interior side of the brick veneer.

Parapets

Although the steel parapet bracing at the roof of the original courthouse building does not technically meet the Tier 1 compliance criteria due to its excessive spacing, assuming the bracing is anchored properly into the masonry parapet and the roof, it is possible that further investigation and analysis may demonstrate the efficacy of the existing bracing. Alternatively, additional diagonal steel angle braces could be installed to reduce the maximum spacing of braces to six feet.

LIMITATIONS

Our observations were limited in extent and were visual-only. No destructive inspection openings were made to observe or document concealed elements or conditions (with the exception of that noted in the Site Observations section of this report), and no destructive testing or finish removal was performed beyond removal of drop ceiling tiles at select locations. Given our limited scope, it was not possible to observe or review each component of the facility in detail. Because of the limited nature of our assessment, we may not have been able to identify hidden or latent defects for which there are no visible indications. Our limited assessment also cannot be construed to warrant or guarantee the subject buildings or building components. The purpose of the assessment was to present potential deficiencies and to provide conceptual strengthening recommendations for further consideration and evaluation by the County.

In-depth analysis and testing of building materials and investigation of latent construction problems are not part of the assessment. Our opinions and comments should not be construed to warrant or guarantee the building structure, its components, or associated land use.

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Appendix A

San Juan County Courthouse

1906 Original Courthouse Building

ASCE 41-17

Tier 1 Checklists

Table 3-1. Common Building Types

Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.
Reinforced Masonry Bearing Walls with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.
Unreinforced Masonry Bearing Walls	
URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

Table 4-6. Checklists Required for a Tier 1 Screening

		Required Checklists[a]					
Level of Seismicity[b]	**Level of Building Performance[c]**	**Very Low Seismicity Checklist (Sec 17.1.1)**	**Basic Configuration Checklist (Sec. 17.1.2)**	**Collapse Prevention Checklist (Sec. 17.2 through 17.17)**	**Immediate Occupancy Checklist (Sec. 17.2 through 17.17)**	**Hazards Reduced or Life Safety Nonstructural Checklist (Sec. 17.19)**	**Position Retention Nonstructural Checklist (Sec. 17.19)**
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-36. Collapse Prevention Structural Checklist for Building Types URM and URMa

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units.	5.5.3.1.1	A.3.2.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following: Top story of multi-story building 9 First story of multi-story building 15 All other conditions 13	5.5.3.1.2	A.3.2.5.2
C NC N/A U	MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids.	5.5.3.4.1	A.3.2.5.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2

continues

Table 17-36 *(Continued)*. Collapse Prevention Structural Checklist for Building Types URM and URMa

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4
C NC N/A U	BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads.	5.7.4.4	A.5.4.5

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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San Juan County Courthouse

1980s Courthouse Expansion

ASCE 41-17
Tier 1 Checklists

Table 3-1. Common Building Types

Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.
Reinforced Masonry Bearing Walls with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.
Unreinforced Masonry Bearing Walls	
URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

Table 4-6. Checklists Required for a Tier 1 Screening

		Required Checklists[a]					
Level of Seismicity[b]	**Level of Building Performance[c]**	**Very Low Seismicity Checklist (Sec 17.1.1)**	**Basic Configuration Checklist (Sec. 17.1.2)**	**Collapse Prevention Checklist (Sec. 17.2 through 17.17)**	**Immediate Occupancy Checklist (Sec. 17.2 through 17.17)**	**Hazards Reduced or Life Safety Nonstructural Checklist (Sec. 17.19)**	**Position Retention Nonstructural Checklist (Sec. 17.19)**
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-34. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1220 mm), and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Stiff Diaphragms			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6

continues

Table 17-34 *(Continued)*. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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San Juan County Courthouse

1980s Second Floor Expansion

ASCE 41-17
Tier 1 Checklists

Table 3-1. Common Building Types

Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.
Reinforced Masonry Bearing Walls with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.
Unreinforced Masonry Bearing Walls	
URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

Table 4-6. Checklists Required for a Tier 1 Screening

		Required Checklists[a]					
Level of Seismicity[b]	**Level of Building Performance[c]**	**Very Low Seismicity Checklist (Sec 17.1.1)**	**Basic Configuration Checklist (Sec. 17.1.2)**	**Collapse Prevention Checklist (Sec. 17.2 through 17.17)**	**Immediate Occupancy Checklist (Sec. 17.2 through 17.17)**	**Hazards Reduced or Life Safety Nonstructural Checklist (Sec. 17.19)**	**Position Retention Nonstructural Checklist (Sec. 17.19)**
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2

continues

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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San Juan County Courthouse

1996 Sheriff's Wing Expansion

ASCE 41-17
Tier 1 Checklists*

**As described in the the Evaluation Results section of this report, the 1996 Sheriff's Wing Expansion was designed to meet the seismic requirements of the 1991 Uniform Building Code (UBC). As a "W2" type wood framed building conforming with the 1991 UBC provisions, it meets the criteria of a benchmark building, indicating that the building is considered to meet the ASCE 41 evaluation requirements.*

Table 3-1. Common Building Types

Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.
Reinforced Masonry Bearing Walls with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.
Unreinforced Masonry Bearing Walls	
URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

Table 4-6. Checklists Required for a Tier 1 Screening

		Required Checklists[a]					
Level of Seismicity[b]	**Level of Building Performance[c]**	**Very Low Seismicity Checklist (Sec 17.1.1)**	**Basic Configuration Checklist (Sec. 17.1.2)**	**Collapse Prevention Checklist (Sec. 17.2 through 17.17)**	**Immediate Occupancy Checklist (Sec. 17.2 through 17.17)**	**Hazards Reduced or Life Safety Nonstructural Checklist (Sec. 17.19)**	**Position Retention Nonstructural Checklist (Sec. 17.19)**
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

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San Juan County Courthouse

1950s Courthouse Annex

ASCE 41-17
Tier 1 Checklists

Table 3-1. Common Building Types

Wood Frames, Commercial and Industrial

W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.

Reinforced Masonry Bearing Walls with Flexible Diaphragms

RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.

Unreinforced Masonry Bearing Walls

URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

Table 4-6. Checklists Required for a Tier 1 Screening

		Required Checklists[a]					
Level of Seismicity[b]	**Level of Building Performance[c]**	**Very Low Seismicity Checklist (Sec 17.1.1)**	**Basic Configuration Checklist (Sec. 17.1.2)**	**Collapse Prevention Checklist (Sec. 17.2 through 17.17)**	**Immediate Occupancy Checklist (Sec. 17.2 through 17.17)**	**Hazards Reduced or Life Safety Nonstructural Checklist (Sec. 17.19)**	**Position Retention Nonstructural Checklist (Sec. 17.19)**
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-36. Collapse Prevention Structural Checklist for Building Types URM and URMa

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units.	5.5.3.1.1	A.3.2.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following: Top story of multi-story building 9 First story of multi-story building 15 All other conditions 13	5.5.3.1.2	A.3.2.5.2
C NC N/A U	MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids.	5.5.3.4.1	A.3.2.5.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2

continues

Table 17-36 *(Continued)*. Collapse Prevention Structural Checklist for Building Types URM and URMa

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4
C NC N/A U	BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads.	5.7.4.4	A.5.4.5

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.