STRUCTURAL EVALUATION OF

PINOLE VALLEY HIGH SCHOOL

WEST CONTRA COSTA UNIFIED SCHOOL DISTRICT (WCCUSD)

For

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DASSE Design Project No. 01B300x2

TABLE OF CONTENTS

Appendix A - Figures

Appendix B - Drawings

10.1 Introduction

The purpose of this report is to perform a seismic assessment of the Pinole Valley High School in Pinole, CA. The structural assessment includes a site walk through and a limited study of available architectural and structural drawings. The purpose of the structural assessment is to identify decay or weakening of existing structural materials (when visible), to identify seismic deficiencies based on our experience with school buildings, and to identify eminent structural life-safety hazards.

The school campus has had a walk-through site evaluation and a limited study of available architectural and structural drawings. The general structural condition of the buildings and any seismic deficiencies that are apparent during our site visit and review of existing drawings are documented in this report. This report includes a qualitative and quantitative evaluation of the buildings. A limited lateral (seismic) numerical analysis was performed to identify deficient lateral elements that could pose life safety hazards.

The site visits did not include any removal of finishes. Therefore, identification of structural conditions hidden by architectural finishes or existing grade was not performed.

10.2 Description of School

The school was built in 1968. The original buildings are wood-framed structures with some concrete pre-cast frames at the library and administration, performing arts, cafetorium, and industrial arts buildings. The gymnasium has both pre-cast concrete walls and frames. There are eight main buildings (permanent structures), forty portable buildings, and numerous steel- and wood- framed covered walkway structures (see figure 1). There are twenty-eight portables built between 1965 and 1969, two 1985 portables, two 1989 portables, one 1998 portable, and four 1999 portables. The total square footage of the permanent structures is about 160,619 square feet.

10.3 Site Seismicity

The classroom, performing arts, and industrial arts buildings have an educational occupancy (Group E, Division 1 and 2 buildings) and the library and administration, cafetorium, and gymnasium buildings have an assembly occupancy (Group A, Division 3). Both of these occupancies have an importance factor in the 2001 CBC of 1.15. The campus is located at a distance of 4.3 kilometers from the Hayward fault. The classroom buildings have plywood shear walls. The lateral force resisting system for the library and administration, performing arts, cafetorium, and industrial arts buildings is a combination of plywood shear walls and intermediate concrete moment frames. The gymnasium has pre-cast concrete non-bearing walls and intermediate concrete moment frames in addition to some plywood shear walls. All of these systems have a response modification factor $R = 5.5$. The 2001 CBC utilizes a code level earthquake, which approximates an earthquake with a 10% chance of exceedance in a 50-year period or an earthquake having a 475-year recurrence period.

The seismic design coefficient in the 2001 CBC is:

$$V = \frac{2.5CaIW}{R} = \frac{2.5(0.44x1.27x1.15)W}{5.5} = 0.292W$$

The site seismicity is used to provide a benchmark basis for the visual identification of deficient elements in the lateral force resisting systems of campus buildings. The calculated base shear was used to perform a limited lateral analysis of the school buildings as described in section 10.7.

10.4 List of Documents

1. Pinole Valley High; Barbachano-Ivanitsky & Associates, Inc., Architect, and Rutherford & Chekene, Structural Engineers; sheets AA-AB, A1-A38, S1-S27, AR1-AR3*, RS4*, P1-P7, M1-M8; May 10, 1965 (May 5, 1966 for Addendum No. 2, noted as *); DSA #26159.
2. "Measure D" - WCCUSD Middle and High Schools– UBC revised parameters by Jensen- Van Lienden Associates, Inc., Berkeley, California.

10.5 Site Visit

DASSE visited the site on August 14, 2002 and October 18, 2002. The purpose of the site visits was to evaluate the physical condition of the structure and in particular focus on the lateral force resisting elements of the building. Following items were evaluated during the site visit:

1. Type and Material of Construction
2. Type of Sheathing at Roof, Floor and Walls
3. Type of Finishes
4. Type of Roof
5. Covered Walkways
6. Presence of Clerestory Windows
7. Presence of Window Walls or High Windows in exterior and interior walls
8. Visible cracks in superstructure, slab on grade and foundation

All of the permanent buildings are one-story structures with similar finishes. The exterior walls are coated with a stucco finish and generally have few window and door openings. An architectural feature, triangular-shaped in section, sticks out from the exterior walls of the buildings just below the roof level and looks like a sloped roof and eave. Some buildings have pre-cast concrete frames that are taller than the perimeter walls, which form popped-up high roof areas (see figures 3 and 6).

The library and administration building (Building A/B, see figure 1) has a combination of wood bearing walls and pre-cast concrete frames. At the high roof area supported by the pre-cast concrete frames, there are clerestory windows along the north side, whereas the other three sides have solid wall (see figure 5). The typical ceiling is acoustical tile attached to the underside of ceiling joists.

The three classroom buildings (Buildings C, D, and DR, see figure 1) are somewhat similar in plan and in construction (see figure 7). They all have central corridors that run longitudinally down the middle of the buildings with classrooms on either side. At buildings C and D, corridors run in the transverse direction at each end of the building. Skylights are located in the ceiling over the central corridors of all of the classroom buildings. The corridors have acoustical tile ceilings attached to ceiling joists and the classrooms have suspended acoustical tile ceilings (see figure 8).

The Performing Arts Building (Building E) is unusual for this campus because its roof area is almost entirely high roof supported by pre-cast concrete frames, with very small low roof areas at the east and west sides of the building (see figure 8). The floor slab is stepped in some locations with concrete risers to create stadium-style seating areas. In other areas, there are raised platform for seating. There are no clerestory windows at the high roof area. The majority of the ceiling has suspended acoustical tile.

The Cafetorium (Building F) has a high roof area supported by pre-cast concrete frames at the main dining and seating space with low roof areas in the kitchen and behind the stage. There are concrete stairs at the rear of the building (see figures 10 and 11). The ceilings are made up of acoustical tiles.

The Gymnasium (Building G) has a large high-ceiling central gymnasium space with low roof areas on all four sides (see figures 13 and 14). The central gymnasium area is significantly taller than the other buildings on the campus and has concrete walls around the perimeter. The low areas house the boys' and girls' locker rooms, a lobby, and additional athletic space. The low roof areas on all four sides make the building roughly cruciform in plan. The ceilings are either gypsum board or plaster ceilings throughout, except in the entry lobby where there is acoustical tile (see figure 15).

The industrial arts building (Building H) is C-shaped in plan. The central area has a high roof supported by pre-cast concrete frames and the two wings have low roofs. At the front of the side wings, there are no windows along the exterior of the building, which would cause the roof diaphragm to have to cantilever out horizontally to resist lateral loads (see figure 16). Also, there are clerestory windows along the north side of the concrete frame (see figures 16 and 17). The building has gypsum board ceilings in most of the area, with suspended acoustical tile ceiling in the northern ends of the low roof wings. There is also a partial mezzanine level at the north side of the building in room 503 (see figure 1). Some moisture damage was noted at the rear wall of the building (see figures 19 and 20), which may have led to some deterioration of the structural members. This potential damage should be investigated and repaired if necessary, and measures should be taken to prevent the entry of water in the future.

Numerous covered walkways pass throughout the campus. The walkways are supported by tube steel moment frames that run in the transverse direction (see figure 21). The covered walkways are not connected to the buildings and have explicit seismic separations between them. The seismic separations are flashed to protect against rainwater.

At the portable classrooms, there is electrical conduit running between the classroom units near the roof level (see figure 22). There are no flexible connections to allow for relative movement of the units. During an earthquake these conduits may be damaged and pose a threat to life safety due to falling electrical lines.

10.6 Review of Existing Drawings

There are several structural features that all of the buildings have in common. The typical slab-on-grade is 4" thick and is reinforced with welded wire fabric. The bearing walls are supported on 12" wide by 18" deep grade beams that span between 18" diameter cast-in-place drilled piers. These piers are typically about 10 ft. long and are located at the intersections of bearing walls, under pre-cast frame columns, and at other locations spaced approximately 17 to 20 feet apart. The exterior stud walls have 3/8" plywood sheathing nailed with 8d nails at 6" spacings. The typical top plate splice is 8-16d nails, with 12-16d nails at long diaphragm spans. The sill plates are attached to the grade beams with 5/8" diameter bolts spaced at 4 ft. centers and the slender wall panels have holdown anchors at each end. Where the exterior faux sloped roof occurs, the wall sheathing is continuous behind the architectural feature and the faux roof is nailed onto the outside face of the wall. In general, the low roof areas have gravel on the roof and the high roof areas have single membrane roofing without any gravel.

The pre-cast concrete frames have some typical features. All the frame beams and columns are 14" wide, except for the Gymnasium building, which has 24" wide frames members. The columns are rectangular in section, but vary in depth along their height. In general, their depth is constant from the slab level up to the low roof level, but then the column becomes deeper and begins to incline inward between the low roof and high roof levels. The beams also vary in depth along their length, generally being deepest at the exterior of the span and shallowest at the center. At the changes in slope along the bottom of the frame members, the bottom reinforcing bars intersect and continue past the intersection with similar detailing as a re-entrant corner. The stirrup spacing is fairly tight at the beam-column intersection and along the height of the column, but is significantly increased near the center of the span. There is a concrete tie beam between the pier caps of each frame to resist the shear forces at the column base due to gravity loads.

The connections of the columns to the foundation consist of grouted pockets in the top of the pier caps, which does not provide significant flexural strength or stiffness. The "pin" connection at the base of the columns reduces the amount of shear that can be developed in the columns and should keep them from failing in a brittle manner during an earthquake. Although the frames at the Gymnasium have a positive steel connection between the column and the pier cap that resists seismic shear and tension, the frames in the other buildings have no direct connection to the foundation resisting uplift. It is possible that a column might "jump" out of the grout pocket during an earthquake, causing damage to the building and a threat to life safety.

In the direction perpendicular to the frames, there are pre-cast concrete girders that span between the frames at the low roof level. They create moment frames in the transverse direction at the low roof level. At most of the buildings, this moment frame is infilled with plywood-sheathed shear walls. In some locations, clerestory windows occur between the frames at the high roof level. At these locations, the pre-cast frames act out of plane as cantilevers spanning up above the low roof level moment frames to resist seismic loads from the high roof.

Detailing of the reinforcement in the pre-cast concrete moment frames does not meet the current requirements for special moment resisting frames. The detailing of the beams and columns appears to generally conform to the requirements of intermediate moment frames. The notable exceptions are the stirrup spacing in the beams near the midspan and the large differences between the positive and negative moment capacities of the beams and columns. Intermediate moment frames are no longer allowed in new construction under the 2001 CBC.

It appears that the behavior of the frame will be governed by flexural hinging at the top of the columns, and that the beams and columns should not yield in a brittle shear failure mode. Additional ties have been added in the zones that could be subjected to plastic hinging during an earthquake so that the shear capacity of section discounting the concrete contribution is adequate to withstand the shear forces due to flexural yielding. However, the spacing of the ties is not as close as would be required by the current building code. Furthermore, the beam and column ties are closed hoops and the bends at the ends of the hoops are only 90 degrees instead of the code-required 135 degrees. These two detailing issues greatly reduce the amount of anticipated confinement of the concrete in the plastic hinge zones, thereby reducing the ductility capacity of the frame. Although the detailing of the frames does not strictly conform to all of the requirements for ductile moment frames, we expect that the frames will exhibit a moderate degree of ductility and do not pose a great threat to life safety in moderate- to large-level earthquakes. In a very large earthquake, however, the frames could undergo large deformations and suffer significant damage, possibly leading to partial collapse of the building.

The library and administration building (Building A/B, see figures 4 through 6) has a blocked roof diaphragm of 3/8” plywood supported by 2x14 joists spaced at 16” centers. In the low roof areas, these joists span a maximum of 27 ft. between bearing walls. At the high roof, the 2x14 joists span 19 ft between the five pre-cast concrete frames, each of which has a single span of 52 ft in the building’s longitudinal direction (see figure 5). There are no interior shear wall lines. Collector elements from the concrete frames extend 20 ft in plan into the adjacent low roof areas. There is a concrete vault inside of the administration area, but these walls do not continue up to the roof and therefore do not act as shear or bearing walls for the building. Because of the long spans between the shear wall lines and frames, the diaphragm chords at the north end of the building could be overloaded during a major earthquake and the roof diaphragm could experience significant damage.

The three classroom buildings (Buildings C, D, and DR, see figure 1) are all of similar construction. They have blocked roof diaphragms of ½” plywood sheathing nailed with 8d nails at 6” spacing over open-web joists spaced at 24” centers that span up to 35 ft between 2x stud bearing walls and, in some cases, glue-laminated beams. The bottom flanges of the open-web joists are stabilized laterally, but are free to deflect in the plane of the truss. The central corridor roof is framed with 2x8 joists spaced at 16” centers with ceiling framing consisting of 2x6 joists spaced at 16” centers. Above the exterior doors, the roofline cuts inward, creating a notch in the roof (see figure 9). At these locations, there are metal straps and additional blocking that help to transfer the diaphragm chord forces. At buildings C and D, there are interior transverse shear walls. The middlemost of these shear wall lines has an offset between the wall on each side of the central corridor. Collector elements at the roof level allow the diaphragm to transfer the collector forces between the provided shear walls. At building C, the connection of this collector to the shear wall is not adequate, which could lead to roof diaphragm damage during an earthquake. At buildings C and D, the diaphragm spans on each side of the middle interior shear wall line are very long, ranging from 76 ft to 104 ft. This could result in the diaphragm chords and the shear walls along this wall line being overstressed during a code-level earthquake event.

The Performing Arts Building (Building E) has a blocked roof diaphragm of 3/8" plywood sheathing supported on 2x14 joists spaced at 16" centers that span 25 ft between pre-cast concrete frames. At low roof areas, 2x12 joists spaced at 16" centers span 5 ft between the transverse pre-cast concrete girders and exterior bearing walls. Unlike the other buildings, the Performing Arts Building has four two-bay concrete frames, each bay spanning 43 ft. Although there are pre-cast concrete girders running in the transverse direction between frames, they only occur at the exterior columns. The interior columns are laterally braced by the wood roof framing above, which is bolted to the frame at 4 ft centers.

There is a mechanical room mezzanine area on the south side of the Performing Arts Building. The mezzanine area has plywood-sheathed shear walls that support it laterally and non-bearing, non-lateral load resisting partition walls above the mezzanine level, which do not support lateral forces from the roof level. This provides lateral support for the mezzanine area and keeps its lateral force resisting system separate from the rest of the building.

The Cafetorium (Building F) has a blocked roof diaphragm of 3/8" plywood sheathing supported by 2x14 joists (2x12 at south wing) spaced at 16" centers. At the high roof area, joists span 26 ft between five pre-cast concrete frames that span 63 ft (see figure 12). At the west low roof area, the joists span 26 ft between glue-laminated beams that align with the pre-cast frames and between wood stud bearing walls. At the north and south low roof areas, wood stud bearing walls support the joists. At the north and south sides of the high roof area, the low roof frames into wood stud walls. The portion of these walls above the low roof level is sheathed with plywood and the portion below is unsheathed, causing the forces to be transferred upward from the low roof to the concrete moment frame.

A 6" thick concrete wall is located between the dining and kitchen areas of the Cafetorium. This concrete wall provides lateral support for the pre-cast concrete frames in the transverse direction on that line, whereas the east ends of the frames are laterally supported by plywood shear walls that are sheathed on both sides. It is unconventional for wood shear walls to be used for lateral support of concrete elements. However, there appears to be adequate strength in the wall to support the seismic loads in the transverse direction.

The Gymnasium (Building G) has low roof areas on all four sides of the high-roofed central gymnasium space. The high roof has a blocked roof diaphragm of ½" plywood sheathing over 2x12 joists spaced at 16" centers that span 21 ft. between four pre-cast concrete frames. The concrete frames are 27 ft tall and span 110 ft across the gymnasium space (see figure 15). Drilled piers under the frames are 36" in diameter and 22 ft. long.

The northern low roof area of the Gymnasium (see figure 13) has a blocked roof diaphragm of ½" plywood sheathing over 2x12 joists spaced at 16" centers that span 18 ft. between the concrete wall and a 2x6 stud wall. The roof joists are anchored to the concrete wall with steel angles spaced every 4 ft. At the other low roof areas, the roof framing is made up of open-web trusses joists that span up to 49 ft. The trusses are connected to concrete walls using steel angles spaced at 4 ft centers that are bolted to a piece of 2x12 which is nailed to the top chord of the truss. The bearing walls have plywood at the perimeter of the building are sheathed with 3/8" plywood and rest on typical grade beams.

Unlike other buildings on the campus, the Gymnasium concrete frames do not act as moment frames to resist the lateral loads. Instead, 8" thick pre-cast concrete walls around the perimeter of the central space act as shear walls. At the east and west walls, these concrete panels follow the sloped profile of the pre-cast frames, sloping inward from the low roof level to the high roof level (see figure 13). Along the north and south walls, 24" x 16" pre-cast pilasters are spaced at 14 ft centers between the pre-cast panels. The wall panels have dowels that hook into the slab-on-grade, which then has dowels into the foundation. Thus, the central portion of the building acts like a concrete shear wall building.

The anchorage of the Gymnasium concrete walls into the roof diaphragm occurs at steel ledger angles with metal plates welded to it at 4 ft centers. At walls perpendicular to the joists, the metal plates are bolted to the roof joists spaced at 4 ft centers with two 5/8" diameter bolts. The roof joists are also lapped and nailed to each other where they are supported at the interior frames, which creates a system of continuous cross-ties. Where the walls are parallel to the joists, the anchors are spaced at 24" centers and are connected to a single piece of blocking. Although the connection of this blocking into the diaphragm is sufficient to withstand the design forces, the aspect ratio of the sub-diaphragm that is supporting the wall loads is inadequate. This could lead to excessive wall deflections and the wall pulling away from the roof diaphragm, which could lead to partial roof collapse. This condition is partially mitigated by the fact that the wall can span horizontally between the pre-cast concrete frames, and therefore will probably not result in a collapse. The same poor anchorage detail also occurs at the east and west walls of the northern low roof area.

The Industrial Arts Building (Building H) has blocked roof diaphragm of 3/8" plywood sheathing over 2x12 roof joists spaced at 16" centers. At the high roof, these joists span 21'-8" between pre-cast concrete frames that are 55 ft long. The concrete frames have an asymmetrical shape, with the beam shaped so as to form a shallowly sloped roof and clerestory windows (see figure 18). At the front of the building, there are pre-cast concrete beams between the frames, which act as a moment frame in the perpendicular direction (see figure 18). At the rear of the building, lateral support for the concrete frames and roof is provided by plywood-sheathed shear walls. There appears to be adequate strength in the wall to support the seismic loads of the concrete frames in the direction perpendicular to the them.

The low roof areas of the industrial arts building are framed with 2x12 joists spaced at 16" centers that span between exterior 2x6 stud bearing walls and 7" by 21-1/8" glue-laminated beams. Where the low roof frames into the north-south walls of the high roof area, the wall studs are not continuous for the full height of the building. Rather, the low roof joists rest on top of a double top plate with another stud wall built above them in the manner of a two-story building. Collector elements in the low roof align with the edges of the high roof area.

At the north face of the low roof areas, the front of the building lacks adequate shear resistance (see figure 16). This could result in excessive diaphragm chord forces and large diaphragm deflections because the diaphragm is acting as a horizontal cantilever. At the rear wall of the building where the roof changes height, there is no collector element within the shear wall. This could result in some damage to the wall at this location and, if the tall and short portions of the wall become separated from each other, it could result in the loss of vertical support for some roof framing.

The covered walkways have 3x6 tongue and groove decking that typically spans 10 to 12 ft. between tube steel moment frames. The frames have 5x3x1/4 beams and 5x3x3/16 columns. These columns are embedded in 18" diameter, 9 foot. long drilled piers. The frames are typically 11'-5" wide and extend out past the outside edge of the roof sheathing. In the direction perpendicular to the frames, the columns act as cantilevers (see figure 21). Because of the cantilever action of the columns and their orientation with the narrow dimension in this direction, the covered walkways could experience large lateral deflections during an earthquake. Although this is not desirable, the flexural capacity of the columns is adequate to resist the anticipated seismic forces. The large deflections discussed above should be reduced by the covered walkway segments running in perpendicular directions that are connected to each other.

10.7 Basis of Evaluation

The document FEMA 310, Federal Emergency Management Agency, *"Handbook for the Seismic Evaluation of Buildings – A Prestandard,"* 1998, is the basis of our qualitative seismic evaluation methods to identify the structural element deficiencies. The seismic performance levels included in FEMA 310 allow the engineer the choice to achieve the Life Safety Performance or the Immediate Occupancy Performance. We have based our evaluation of school buildings on the Life Safety Performance level, which is defined as "the building performance that includes significant damage to both structural and nonstructural components during a design earthquake, though at least some margin against either partial or total collapse remains. Injuries may occur, but the level of risk for life-threatening injury and entrapment is low."

Because mitigation strategies for rehabilitating buildings found to be deficient are not included in FEMA 310 document, the California Building Code (CBC 2001) is used as the basis of our quantitative seismic evaluation methods and strategies for seismic strengthening of school buildings. The scope of our analyses were not to validate every member and detail, but to focus on those elements of the structures determined by FEMA 310 to be critical and which could pose life safety hazards. Element *strength* values not addressed in the California Building Code were based on the document FEMA 356, Federal Emergency Management Agency, *"A Prestandard and Commentary for the Seismic Rehabilitation of Buildings"* 2000.

10.8 List of Deficiencies

Building deficiencies listed below have corresponding recommendations identified and listed in Section 10.9, which follow the same order as the itemized list of deficiencies identified below. The severity of the deficiency is identified by a “structural deficiency hazard priority” system based on a scale between 1.0 and 3.9, which is described in Section 10.11. These priority ratings are listed in section 10.9. Priority ratings between 1.0 to 1.9 could be the causes for building collapses, partial building collapses, or life-safety hazards, if the corresponding buildings are subjected to major earthquake ground motions, which are possible at these sites. It is strongly recommended that these life safety hazards are mitigated by implementing the recommendations listed below.

Item	Building Structural Deficiencies
1.	At the exterior longitudinal walls of the Library and Administration building, the top plate splices are inadequate to transfer the roof diaphragm chord forces.
2.	At the library and administration, performing arts, cafetorium, and industrial arts buildings, the detailing of the pre-cast concrete frames may be inadequate to provide the required ductility during an ultimate level earthquake. There is no positive connection to resist uplift or jumping of the frame.
3.	At the library and administration, performing arts, cafetorium, and industrial arts buildings, the pre-cast concrete frames are seated in grout pockets. There is no positive connection to resist uplift or jumping of the frame.
4.	At the exterior walls of the classroom buildings (C and D and DR), the top plate splices are inadequate to transfer the roof diaphragm chord forces.
5.	The interior transverse shear walls near the center of the classroom buildings (C and D) are overstressed in shear. The holdowns are overstressed. The sill bolts are inadequate.
6.	The interior transverse shear walls near the center of classroom building C don’t align with each other. The collector connection between them is overstressed.
7.	At the gymnasium, the out-of-plane anchorage of the east and west concrete wall panels to the diaphragm does not continue far enough into the roof diaphragm. This condition occurs at the high roof and at the north low roof area. The aspect ratio of the sub-diaphragm is excessive and may lead to failure of the wall anchorage system.
8.	At the industrial arts building, the east and west wings have windows all along the north face of the building. There is a lack of lateral resistance along this line.
9.	At the south wall of the industrial arts building, there is no collector element where the wall changes height between the low and high roof areas. This may result in some damage to the building at this location. If the high and low roof areas of the building were to tear free from each other during an earthquake, there may be a loss of gravity support for some roof framing, resulting in a partial collapse of the roof.
10.	At the south wall of the industrial arts building, there is some moisture damage to the stud wall.
11.	At the portable classrooms, there is conduit running between the portable classrooms at the roof level.

10.9 Recommendations

Items listed below follow the same order as the itemized list of deficiencies identified in section 10.8 above.

Item	Recommended Remediation	Priority	Drawing Number
1.	Provide new CMST metal straps above the plywood sheathing at the roof.	1.2	2
2.	Provide additional concrete confinement and shear capacity by wrapping the pre-cast concrete columns and beams with a fiber reinforced polymer composite system. Wrap all 4 sides of the columns and 3 full sides of beams with as much as possible at the top of the beam.	1.8	2, 10, 12, 14, 16
3.	Provide new steel angles each side of each column and with expansion anchor into the column and foundation to resist uplift.	1.9	1, 9, 11, 15
4.	Provide new CMST metal straps above the plywood sheathing at the roof.	1.2	4, 6, 8
5.	Provide new plywood sheathing at the unsheathed face of the existing wall. Sister new 3x studs onto the existing 2x wall framing to comply with DSA requirements. Provide new holdowns at each end of wall. Provide new additional sill bolts at 48" o.c.	1.1	3, 5
6.	Provide additional nailing or LTP4 connectors at the existing collector connections.	1.3	4
7.	Provide new blocking between joists and continuous straps at 4'-0" o.c. to develop the anchorage forces back into the diaphragm at least 8 ft.	1.5	14
8.	Infill some existing windows with new plywood shear wall. Strengthen existing collector elements as required.	1.3	15
9.	Provide new blocking and collector straps in the existing wall at the discontinuity.	1.5	16
10.	Remove the existing wall and roof finishes and ceiling as required to repair the members with water damage. Ensure adequate waterproofing of the wall. Determine extent of damage in field.	2.5	16
11.	Provide flexible connections for conduit running between portables.	1.9	None

10.10 Portable Units

In past earthquakes, the predominant damage displayed by portable buildings has been associated with the buildings moving off of their foundations and suffering damage as a result. The portables observed during our site visits tend to have the floor levels close to the ground, thus the damage resulting from buildings coming off of their foundation is expected to be minimal. The life safety risk of occupants would be posed from the potential of falling 3 feet to the existing grade levels during strong earthquake ground shaking. Falling hazards from tall cabinets or bookshelves could pose a greater life safety hazard than building movement. The foundation piers supporting the portable buildings tend to be short; thus the damage due to the supports punching up through the floor if the portable were to come off of its foundation is not expected to be excessive.

Because of their light frame wood construction and the fact that they were constructed to be transported, the portable classrooms are not in general expected to be life safety collapse hazards. In some cases the portables rest directly on the ground and though not anchored to the ground or a foundation system could only slide a small amount. In these instances the building could slide horizontally, but we do not expect excessive damage or life safety hazards posed by structural collapse of roofs.

The regulatory status of portables is not always clear given that portables constructed prior to 1982 will likely have not been reviewed by DSA and thus will likely not comply with the state regulations for school buildings. Portables constructed after about 1982 should have been permitted by DSA. The permits are either issued as temporary structures to be used for not more than 24 months or as permanent structures.

10.11 Structural Deficiency Prioritization

This report hazard rating system is based on a scale of 1.0 to 3.9 with 1.0 being the most severe and 3.9 being the least severe. Based on FEMA 310 requirements, building elements have been prioritized with a low rating of 1.0 to 1.9 if the elements of the building's seismic force resisting systems are woefully inadequate. Priority 1.0 to 1.9 elements could be the causes for building collapses, partial building collapses, or life-safety falling hazards if the buildings were subjected to major earthquake ground motion.

If elements of the building's seismic force resisting system seem to be inadequate based on visual observations, FEMA 310 requirements and limited lateral (seismic) calculations, but DASSE believes that these element deficiencies will not cause life-safety hazards, these building elements have been prioritized between a rating low of 2.0 to 3.9. These elements could experience and / or cause severe building damage if the buildings were subjected to major earthquake ground motion. The degree of structural damage experienced by buildings could cause them not to be fit for occupancy following a major seismic event or even not repairable.

The following criteria was used for establishing campus-phasing priority:

First, the individual element deficiencies which were identified during site visit and review of existing drawings were prioritized with a rating between 1.0 to 3.9 and as described in this section.

Next, based on the school district's budgetary constraints and scheduling requirements, each school campus was given a phasing number between one and three. Phase 1A represents a school campus with severe seismic deficiencies, Phase 1B represents a school campus with significant seismic deficiencies and Phase 2 represents a school campus with fewer seismic deficiencies.

10.12 Conclusions

1. Given the vintage of the building(s), some elements of the construction will not meet the provisions of the current building code. However, in our opinion, based on the qualitative and limited quantitative evaluations, the building(s) will not pose serious life safety hazards if the seismic deficiencies identified in section 10.8 are corrected in accordance with the recommendations presented in section 10.9.

2. Any proposed expansion and renovation of the buildings should include the recommended seismic strengthening presented in section 10.9. Expansion and renovation schemes that include removal of any portion of the lateral force resisting system will require additional seismic strengthening at those locations. It is reasonable to assume that where new construction connects to the existing building(s), local seismic strengthening work in addition to that described above will be required. All new construction should be supported on new footings.

3. Overall, we recommend that seismic retrofit work for this school campus be performed in Phase 2.

10.13 Limitations and Disclaimer

This report includes a qualitative (visual) evaluation and a limited quantitative seismic evaluation of each school building. Obvious gravity or seismic deficiencies that are identified visually during site visits or on available drawings are identified and documented in this report. Elements of the structure determined to be critical and which could pose life safety hazards are identified and documented during limited quantitative seismic evaluation of the buildings.

Users of this report must accept the fact that deficiencies may exist in the structure that were not observed in this limited evaluation. Our services have consisted of providing professional opinions, conclusions, and recommendations based on generally accepted structural engineering principles and practices.

DASSE's review of portable buildings has been limited to identifying clearly visible seismic deficiencies observed during our site visit and these have been documented in the report. Portable buildings pose several issues with regard to assessing their life safety hazards. First, drawings are often not available and when they are, it is not easy to associate specific drawings with specific portable buildings. Second, portable buildings are small one story wood or metal frame buildings and have demonstrated fairly safe performance in past earthquakes. Third, there is a likelihood that portable buildings (especially those constructed prior to 1982) are not in compliance with state regulations, either because they were not permitted or because the permit was for temporary occupancy and has expired.

Appendix A – Figures

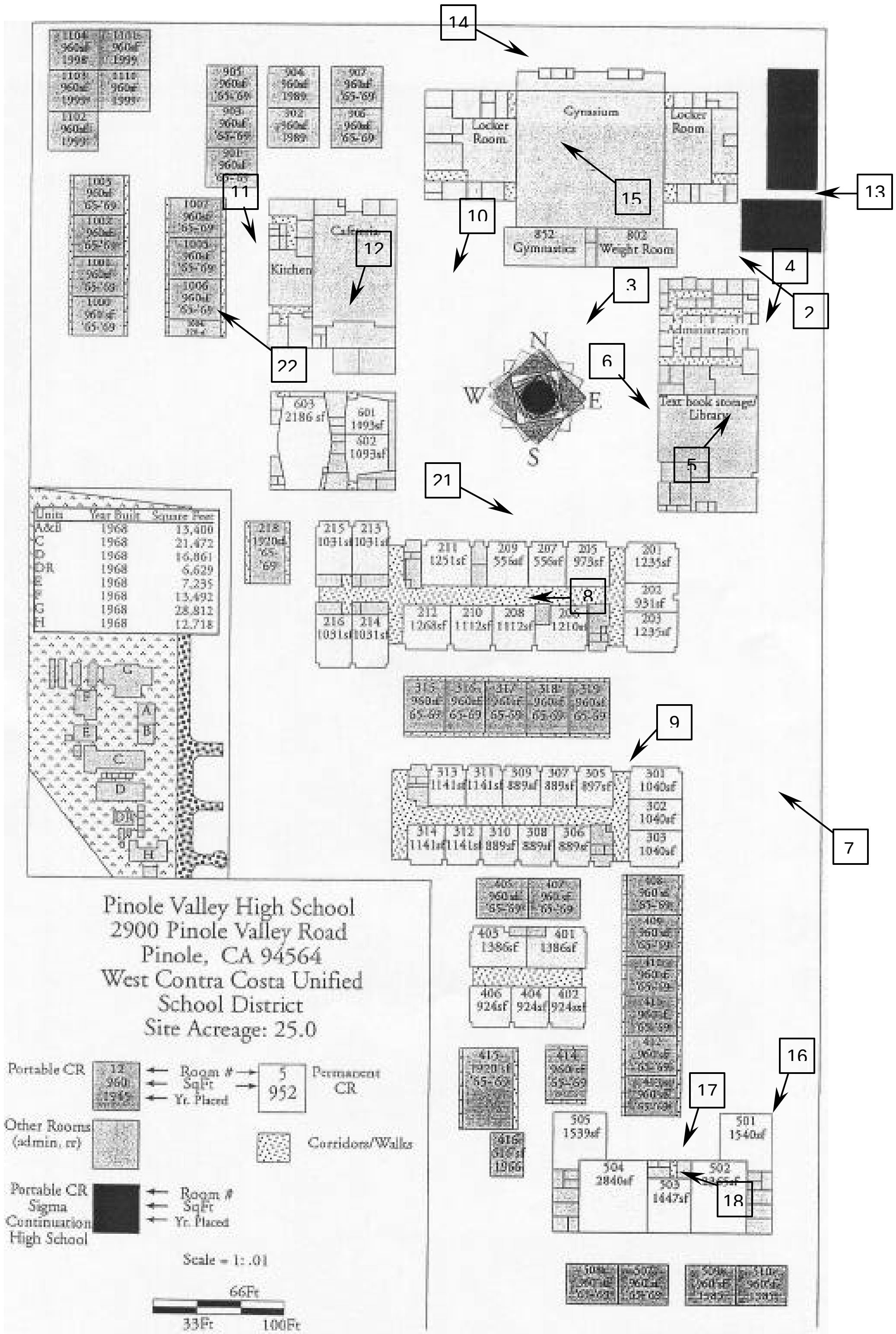


Figure 1: School layout plan

Figure 2: Front of school

Figure 3: Main quad area looking southwest

Figure 4: Northeast corner of the administration and library building

Figure 5: Clerestory windows at the library

Figure 6: West side of the administration and library building

Figure 7: East end of the classroom buildings C and D

Figure 8: Central corridor of classroom building

Figure 9: Setback in roof of the classroom building at the entrance

Figure 10: Front of the cafetorium and the performing arts building

Figure 11: Rear of the cafetorium and the performing arts building

Figure 12: Interior of the cafetorium

Figure 13: East side of the gymnasium

Figure 14: North side of the gymnasium

Figure 15: Interior of the gymnasium

Figure 16: Northeast corner of the industrial arts building

Figure 17: Front of the industrial arts building

Figure 18: Interior of the industrial arts building

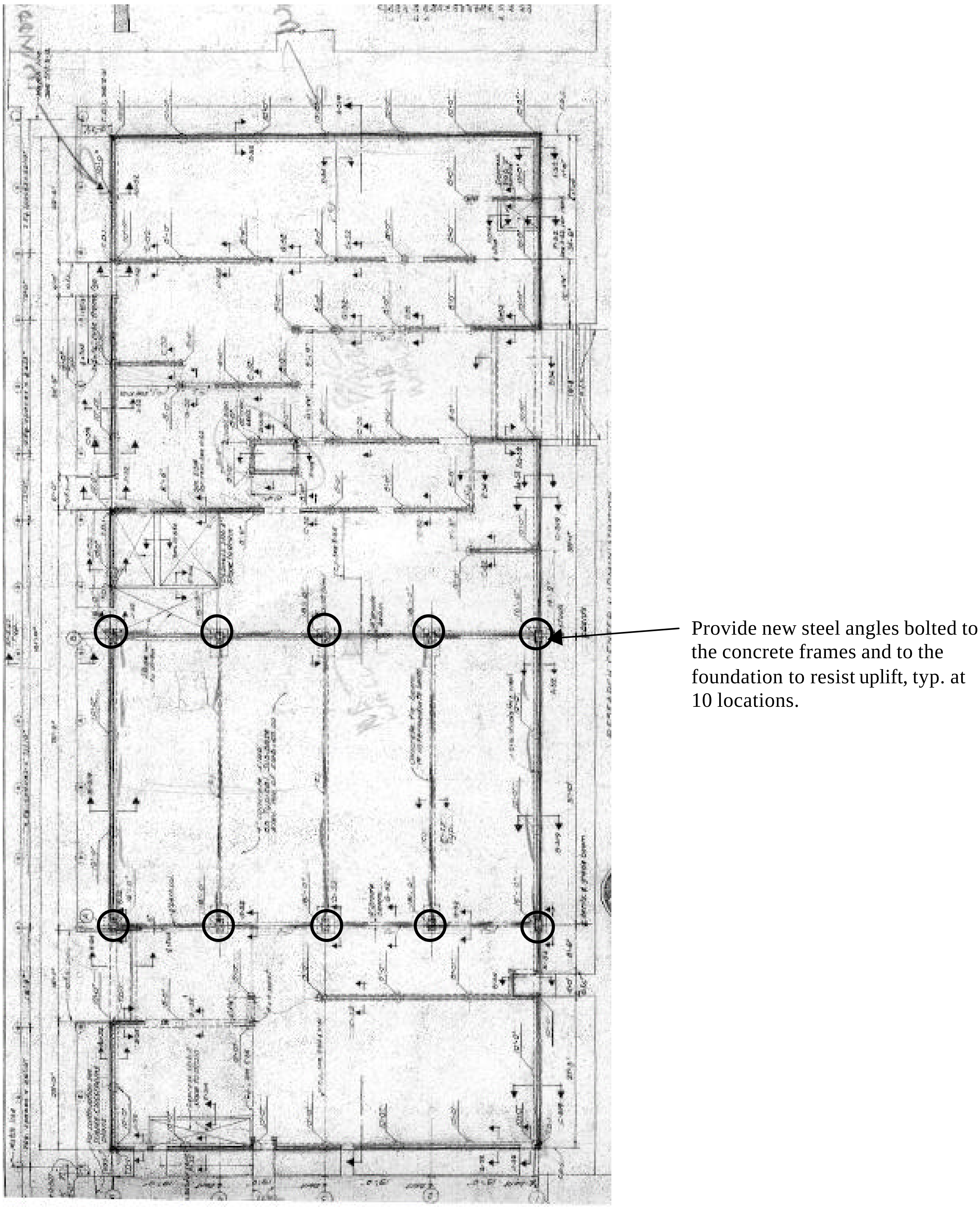
Figure 19: Deterioration of the wall in the industrial arts building

Figure 20: Deterioration of the wall in the industrial arts building

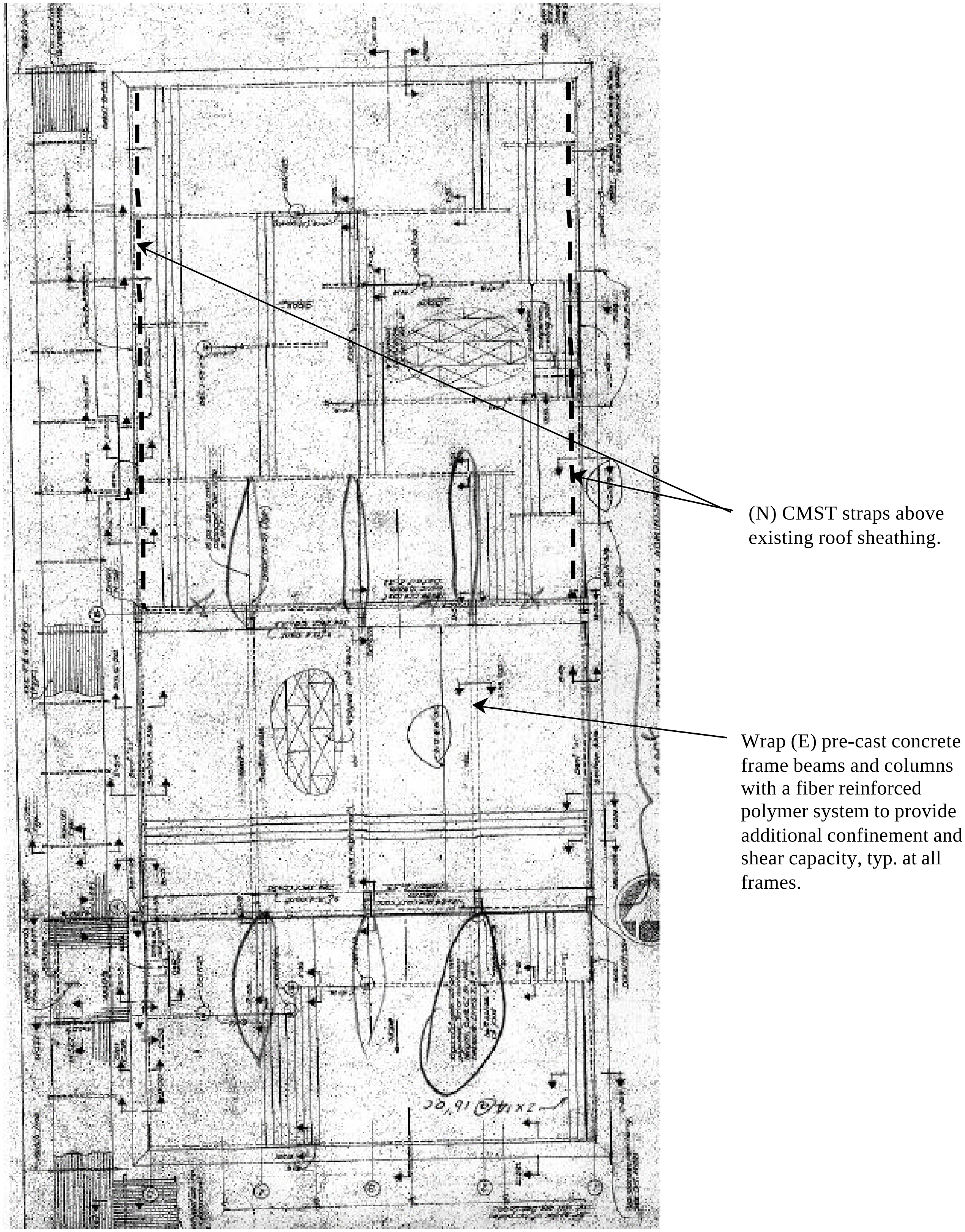
Figure 21: Typical covered walkway

Figure 22: Portable classroom units

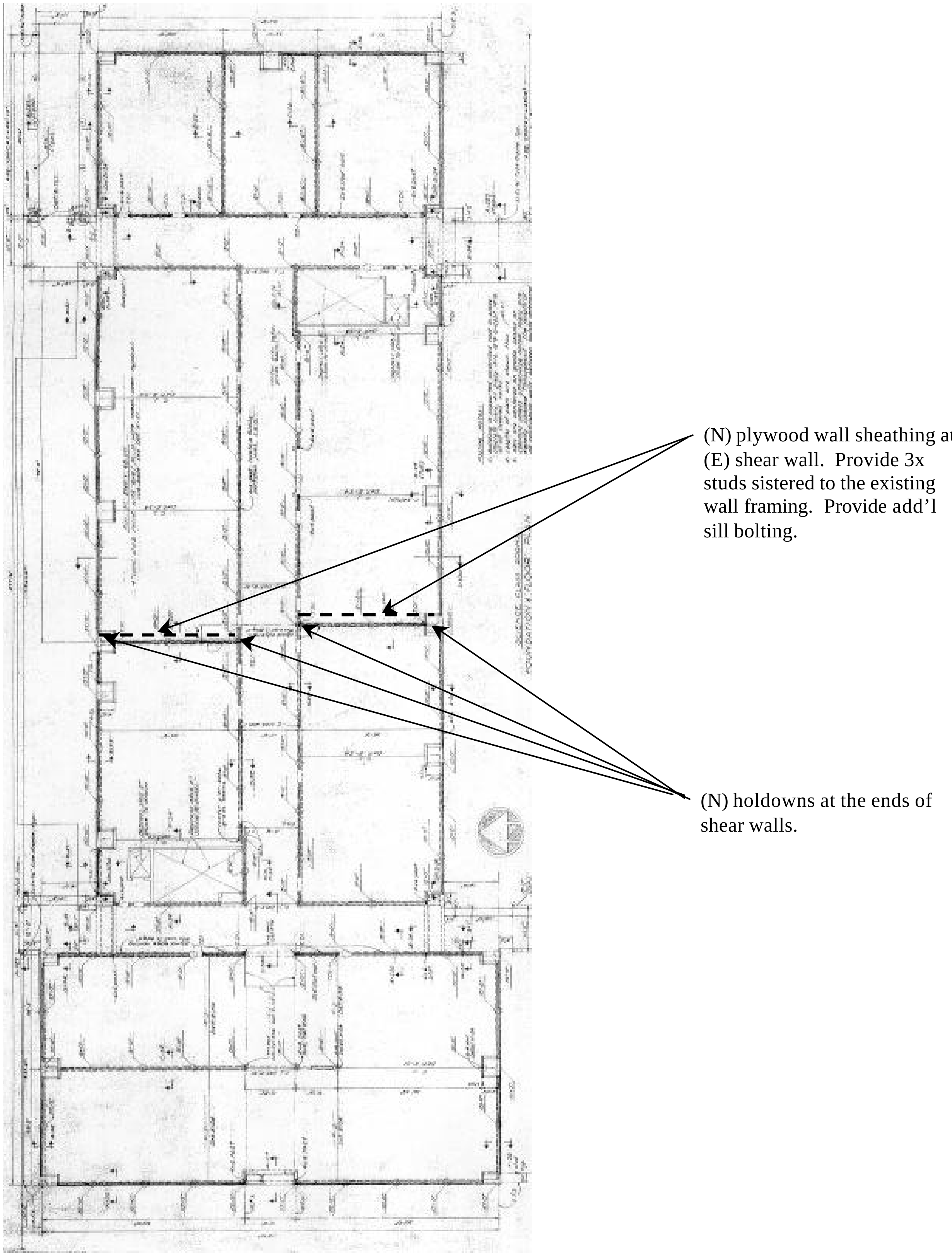
Appendix B – Drawings



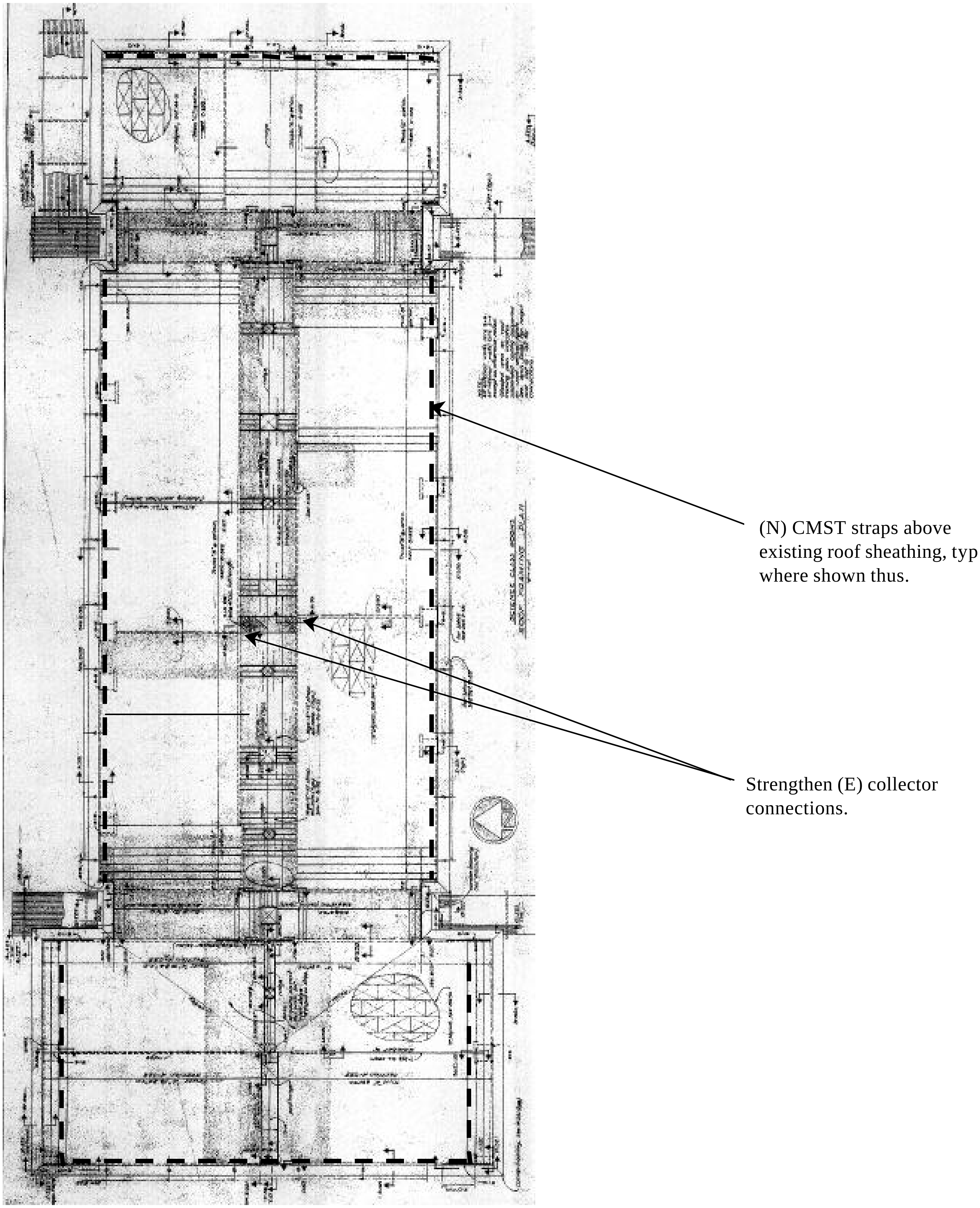
Drawing 1: Academic and library building (A/B) floor plan



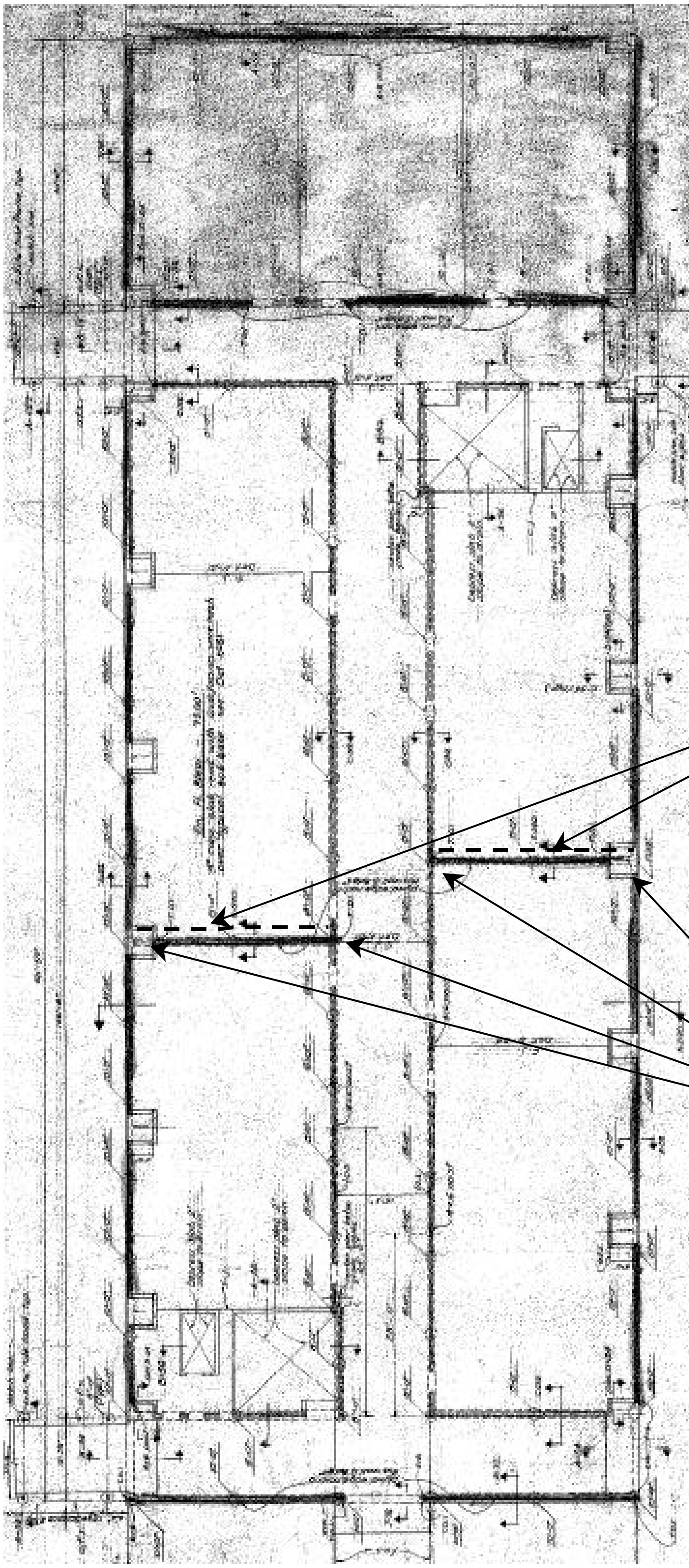
Drawing 2: Academic and library building (A/B) roof plan



Drawing 3: Classroom building C floor plan



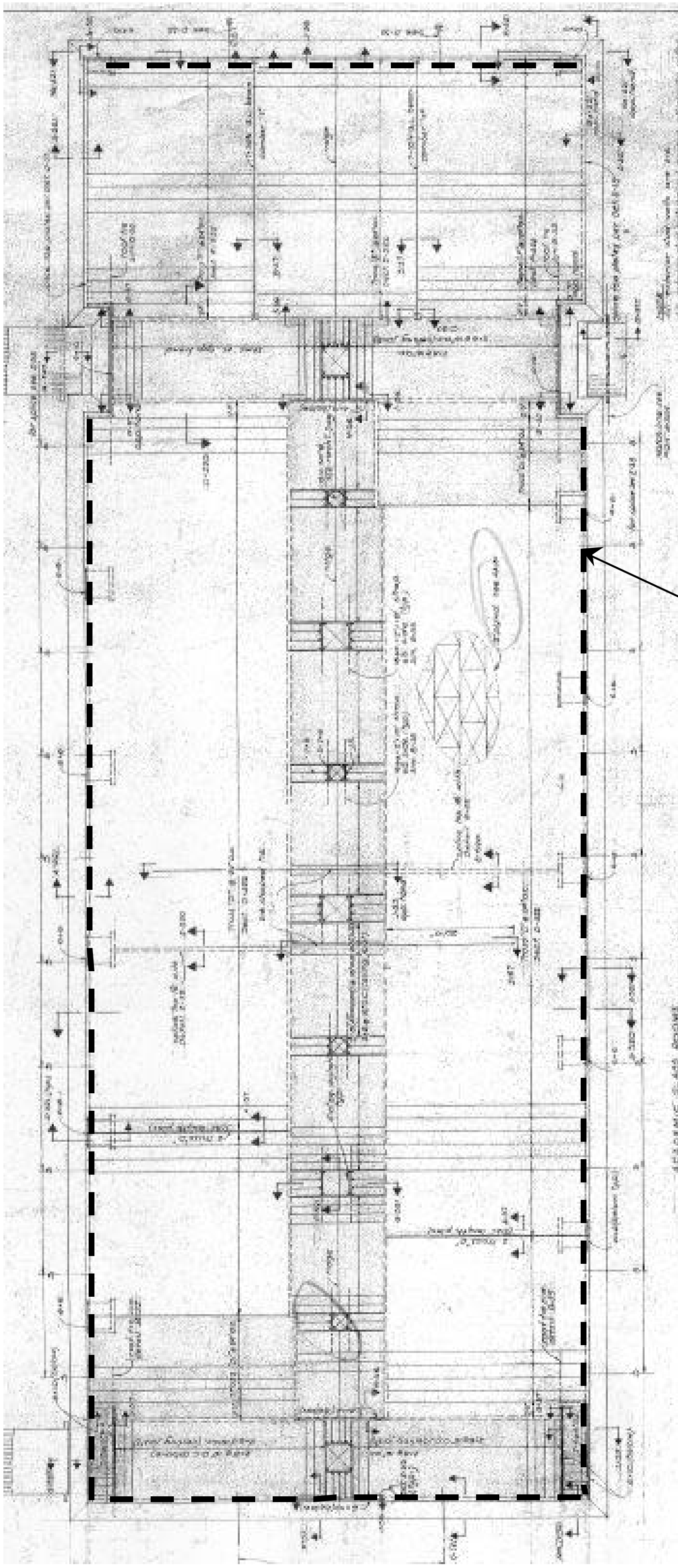
Drawing 4: Classroom building C roof plan



(N) plywood wall sheathing at (E) shear wall. Provide 3x studs sistered to the existing wall framing. Provide add'l sill bolting.

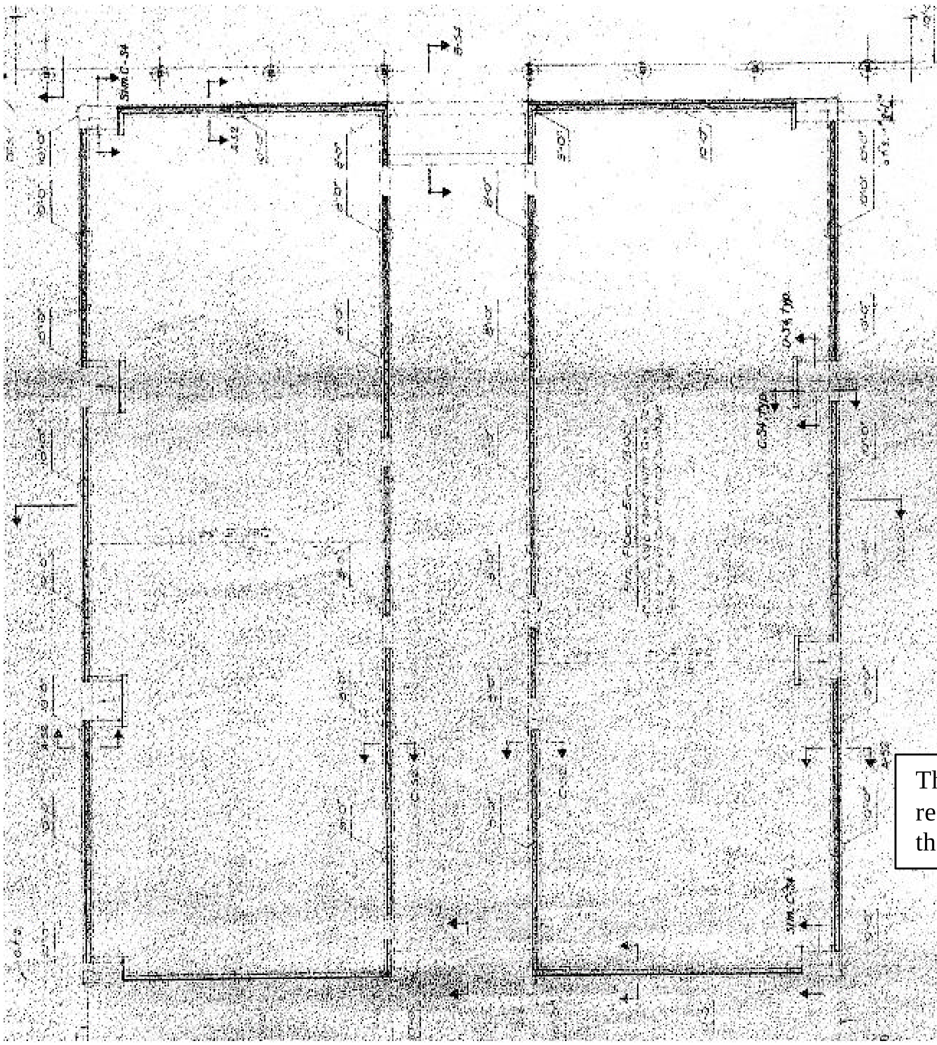
(N) holdowns at the ends of shear walls.

Drawing 5: Classroom building D floor plan



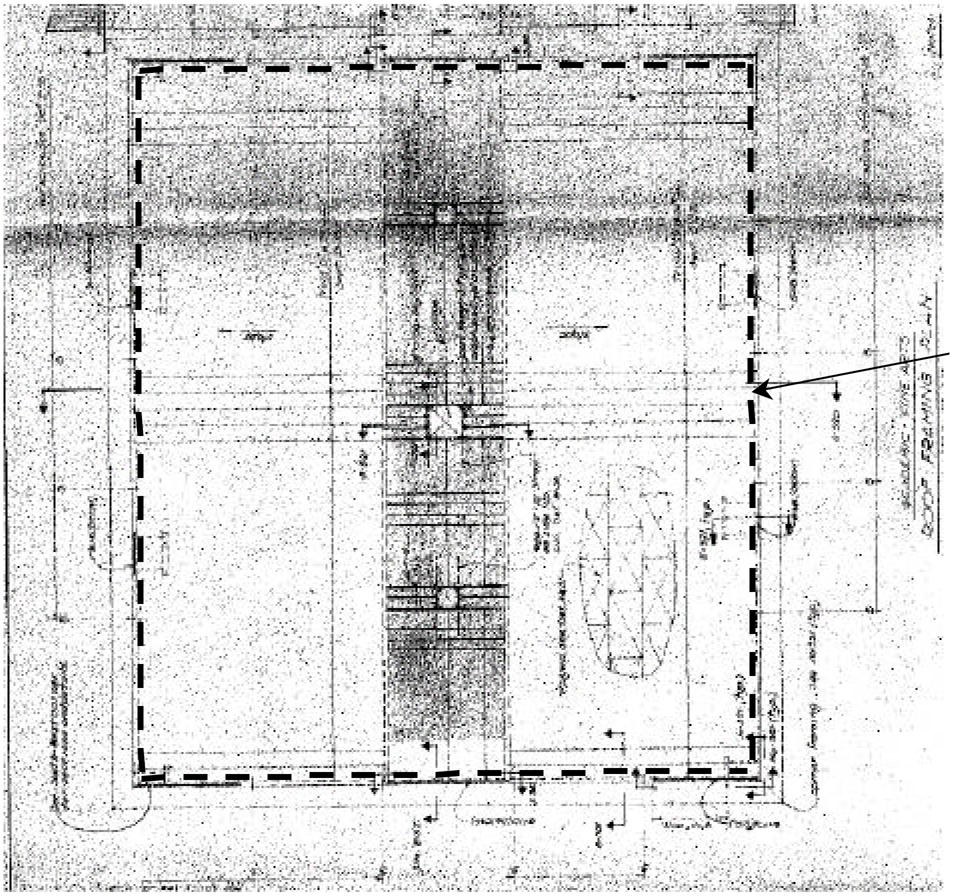
(N) CMST straps above existing roof sheathing, typ. where shown thus.

Drawing 6: Classroom building D roof plan

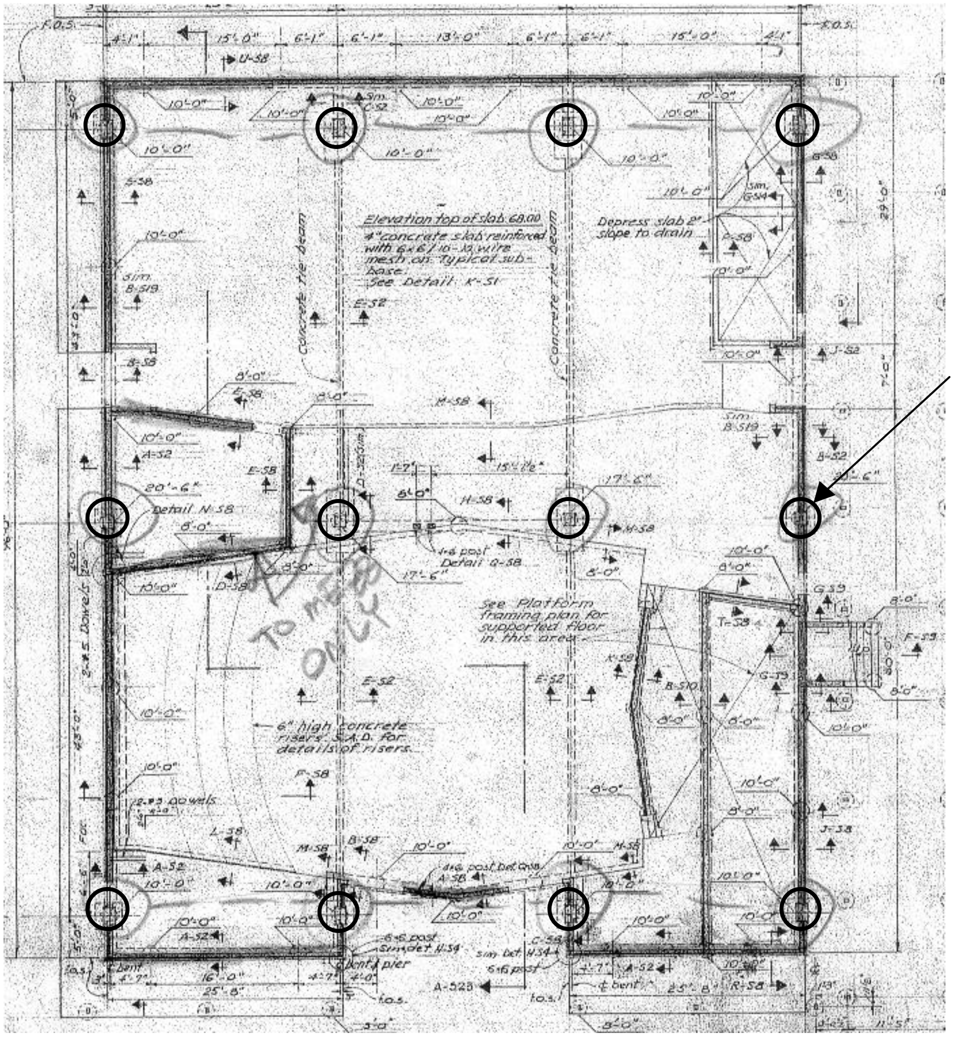


There is no seismic retrofit work recommended for this building at the foundation level.

Drawing 7: Classroom building DR floor plan

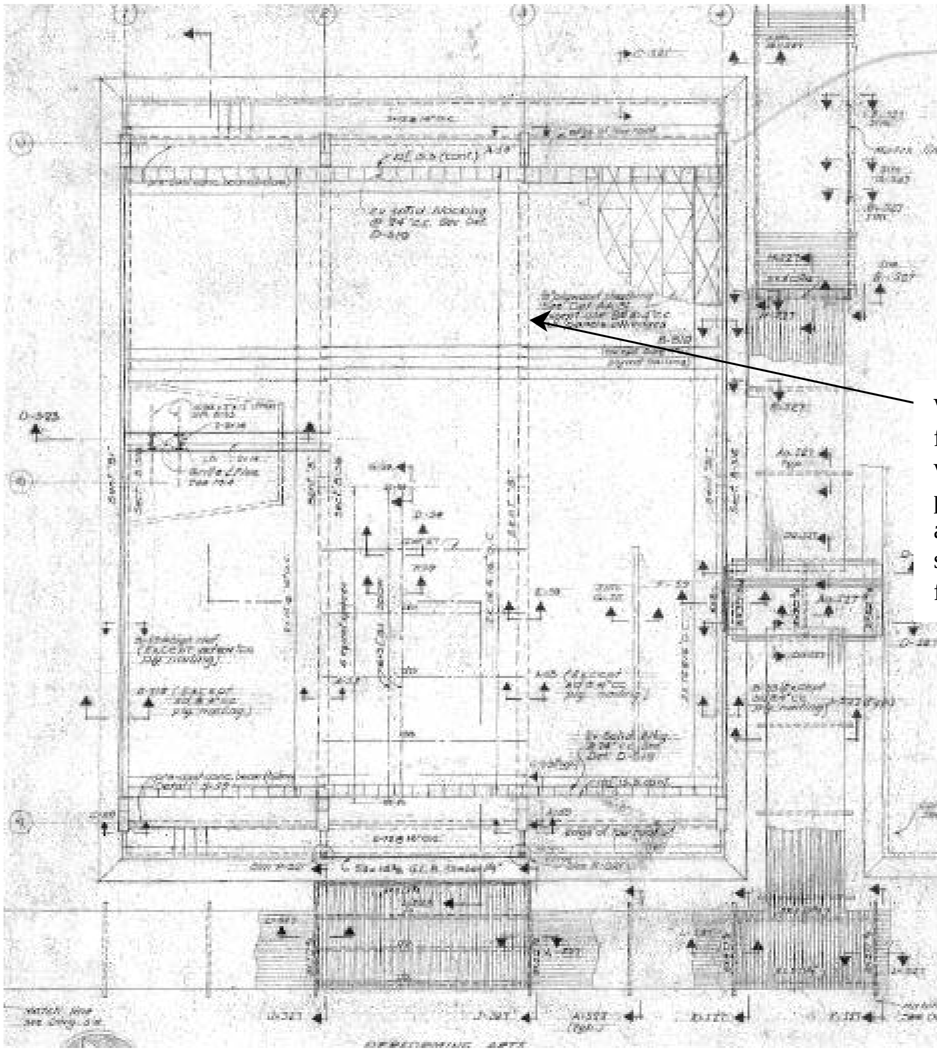


Drawing 8: Classroom building DR roof plan

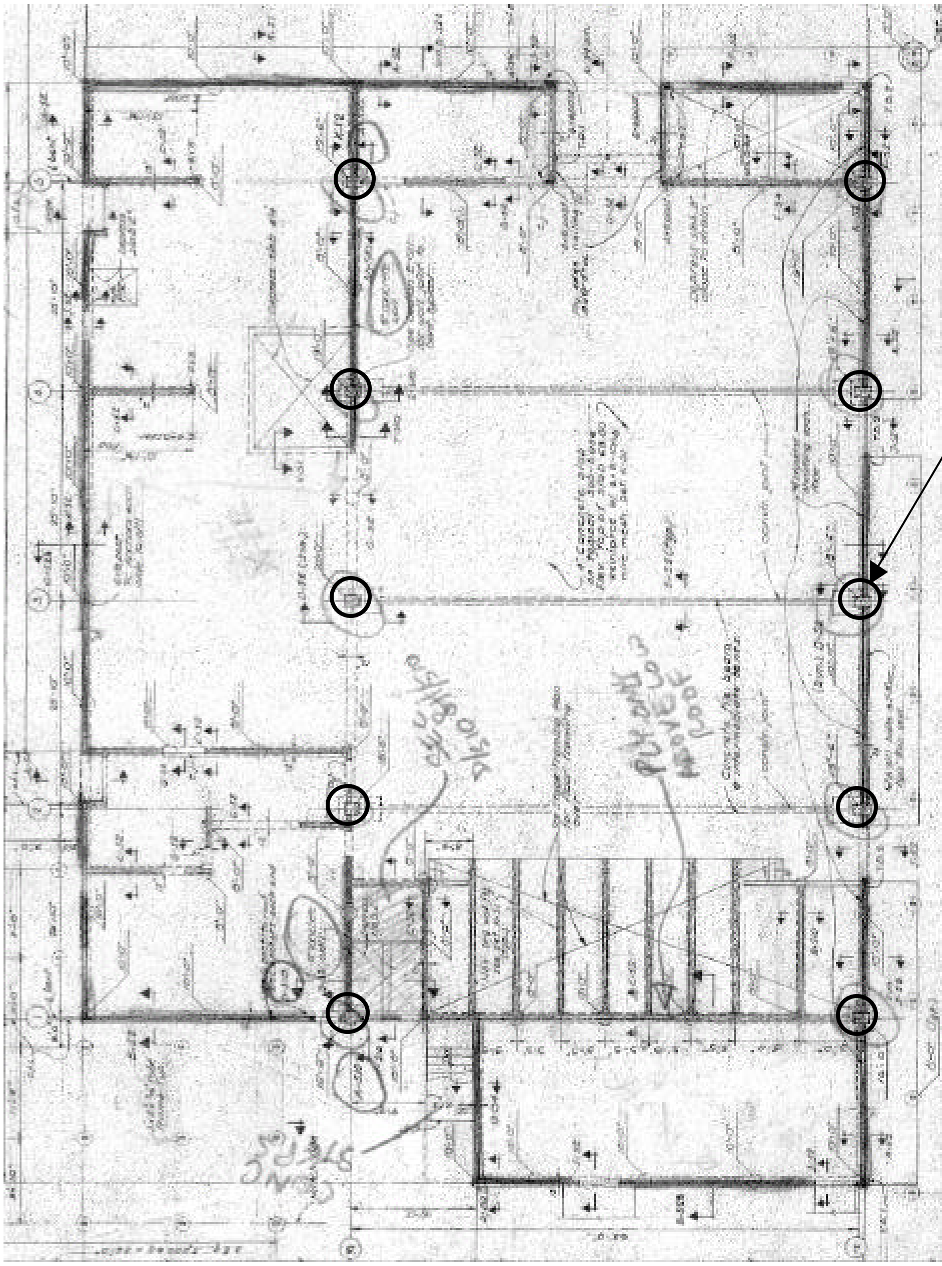


Provide new steel angles bolted to the concrete frames and to the foundation to resist uplift, typ. at 12 locations.

Drawing 9: Performing arts building (E) floor plan

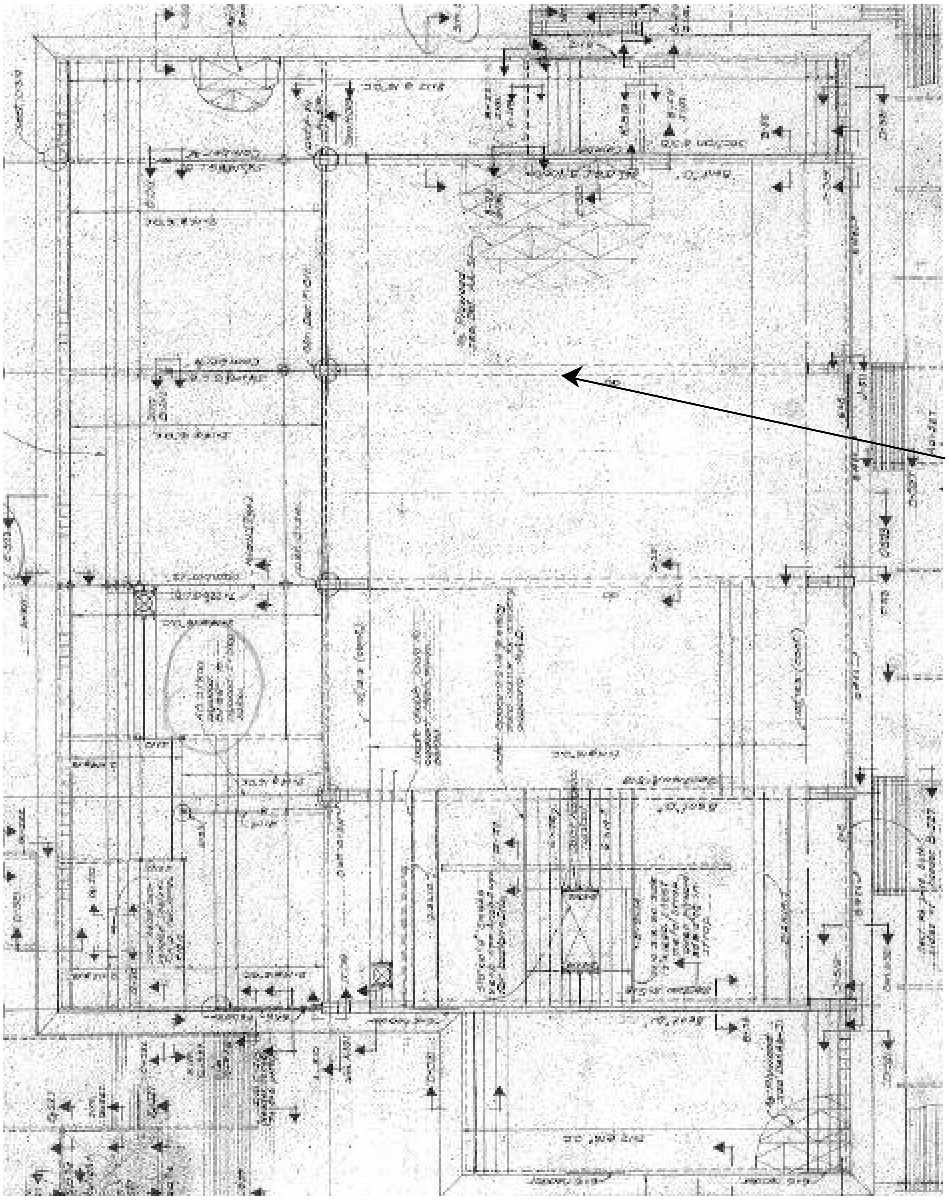


Drawing 10: Performing arts building (E) roof plan



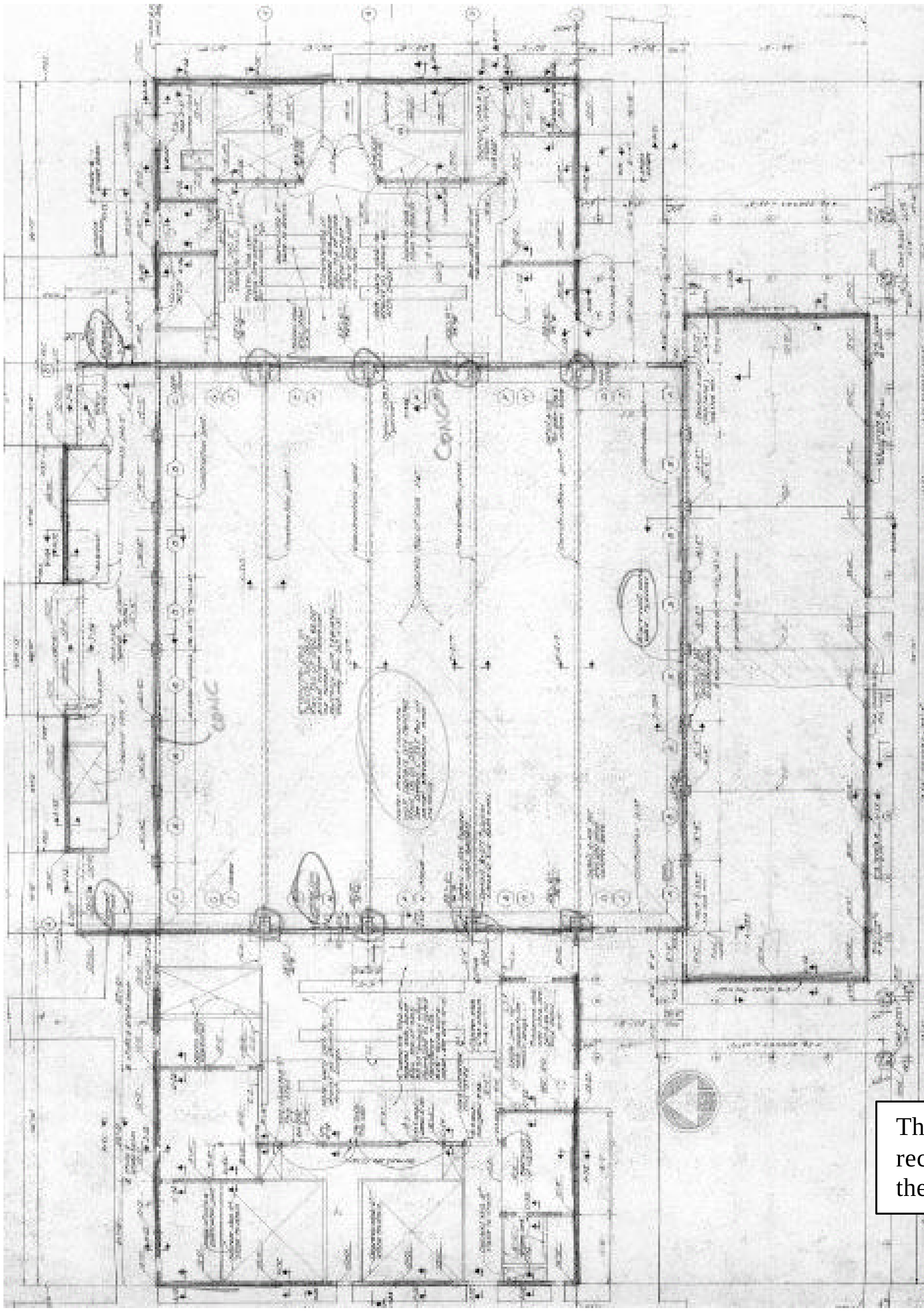
Provide new steel angles bolted to the concrete frames and to the foundation to resist uplift, typ. at 10 locations.

Drawing 11: Cafetorium (F) floor plan



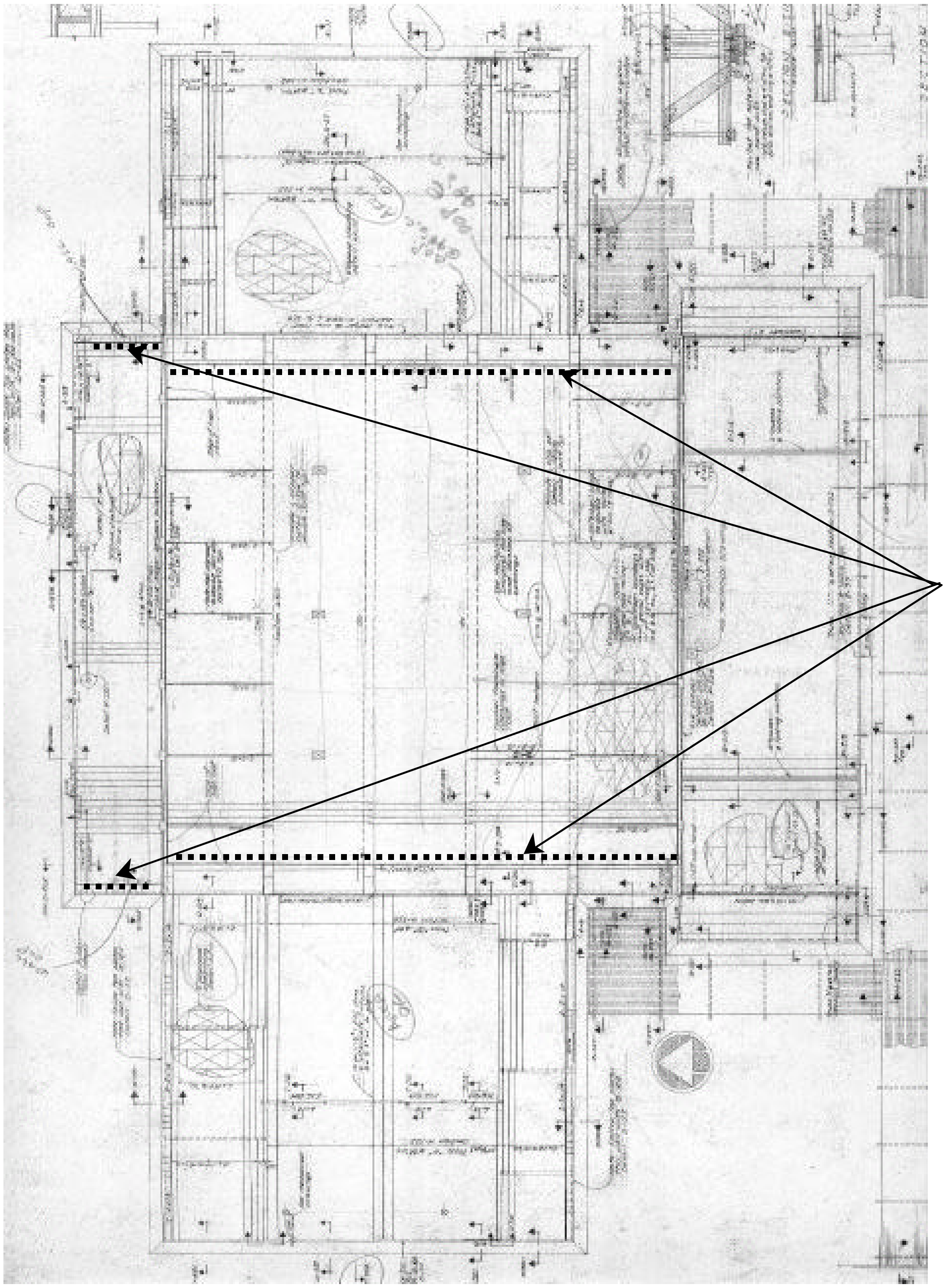
Wrap (E) pre-cast concrete frame beams and columns with a fiber reinforced polymer system to provide additional confinement and shear capacity, typ. at all frames.

Drawing 12: Cafetorium (F) roof plan



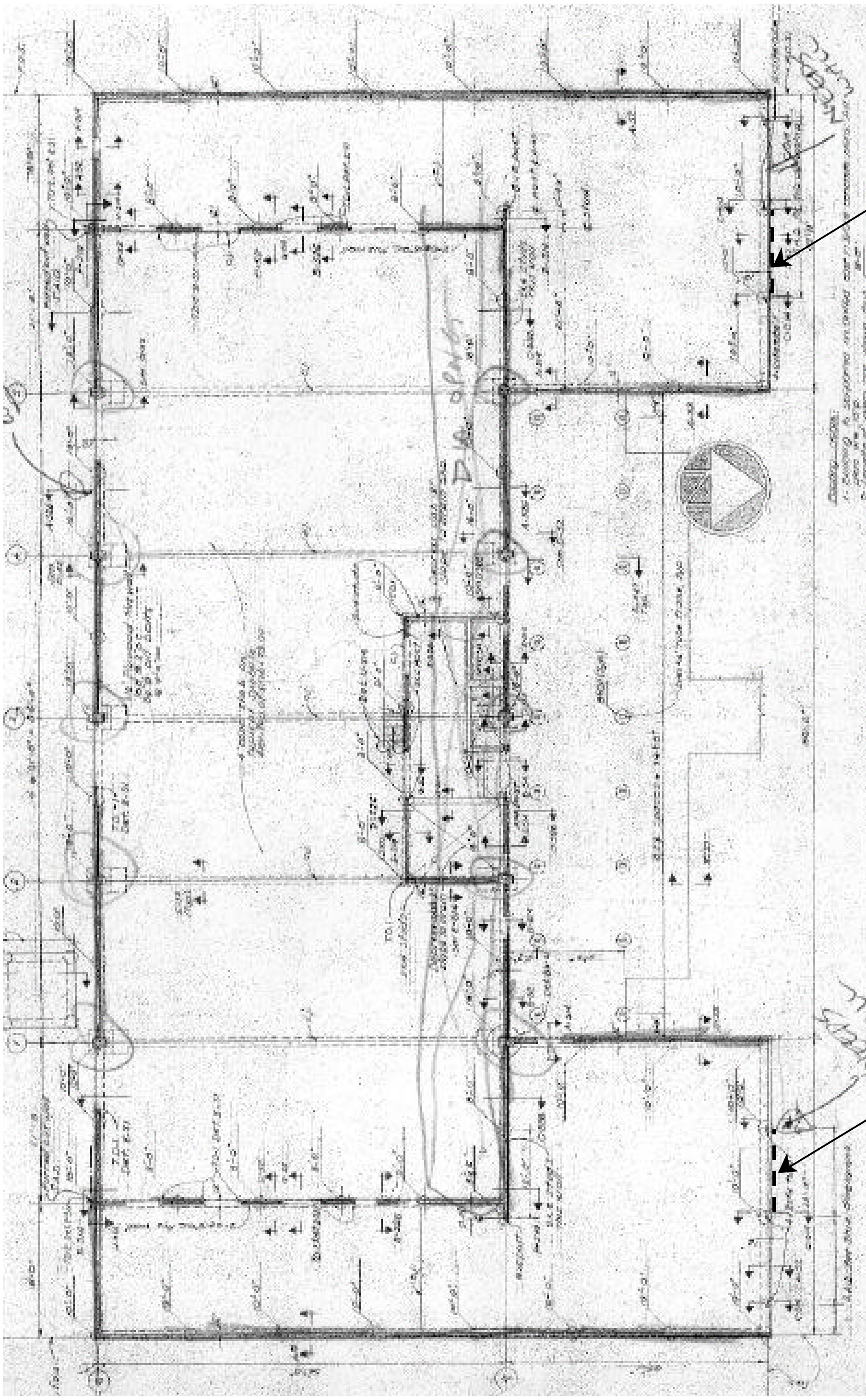
There is no seismic retrofit work recommended for this building at the foundation level.

Drawing 13: Gymnasium (G) floor plan



Provide (N) blocking between (E) roof joists and continuous metal straps spaced at 48" o.c. to develop the wall anchorage forces into the diaphragm and create a sub-diaphragm at least 8 ft. deep.

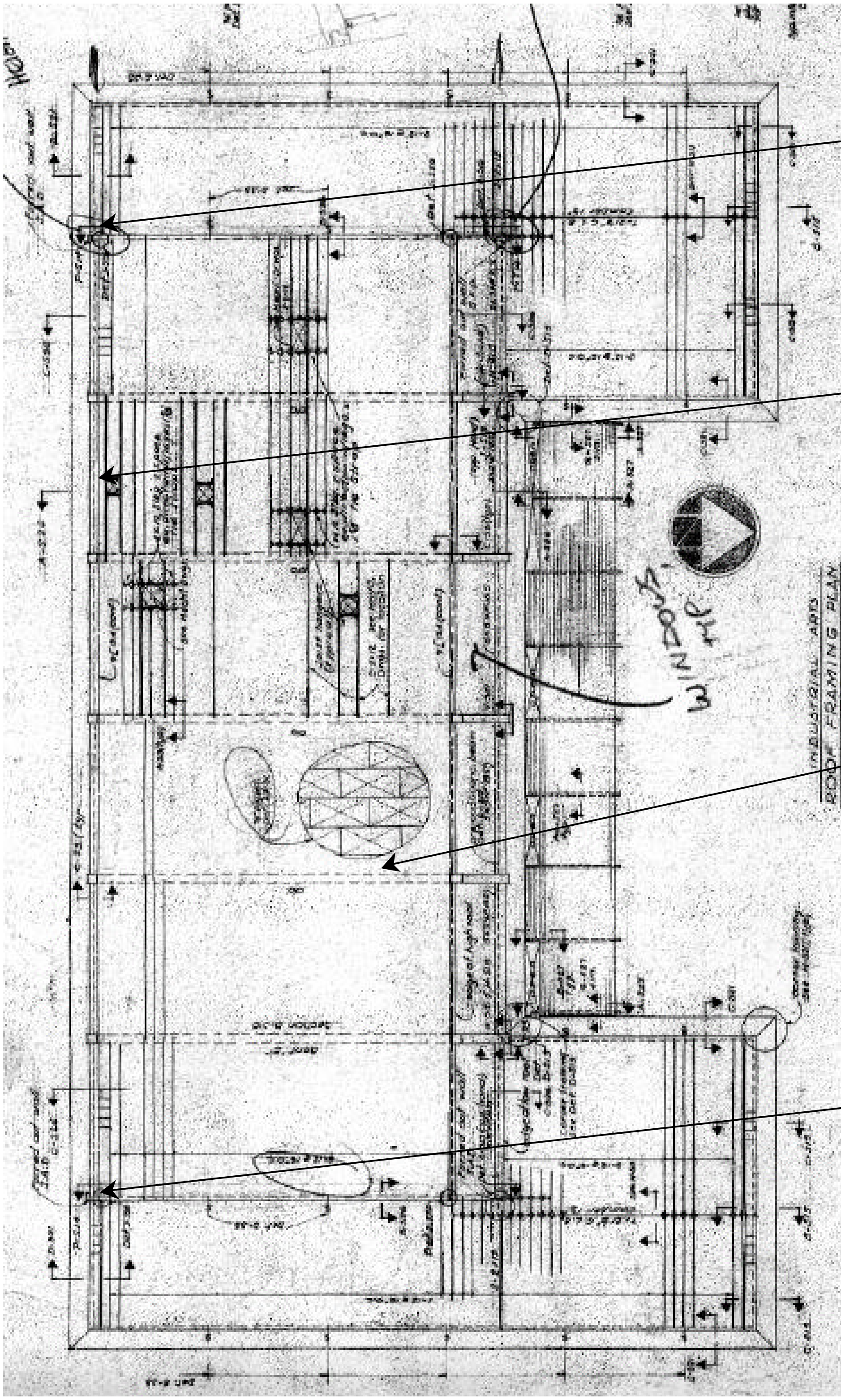
Drawing 14: Gymnasium (G) roof plan



Infill existing windows with (N) plywood shear wall. Provide additional sill bolts to the (E) foundation as required.

Infill existing windows with (N) plywood shear wall. Provide additional sill bolts to the (E) foundation as required.

Drawing 15: Industrial arts building (H) floor plan



Provide (N) blocking and collector straps in the (E) stud wall where the roof height changes.

Remove the existing wall and roof finishes and ceiling as required to repair the members with water damage. Ensure adequate waterproofing of the wall. Determine extent of damage in field.

Wrap (E) pre-cast concrete frame beams and columns with a fiber reinforced polymer system to provide additional confinement and shear capacity, typ. at all frames.

Provide (N) blocking and collector straps in the (E) stud wall where the roof height changes.

Drawing 16: Industrial arts building (H) roof plan