STRUCTURAL EVALUATION OF

JOHN F. KENNEDY HIGH SCHOOL

WEST CONTRA COSTA UNIFIED SCHOOL DISTRICT (WCCUSD)

For

WLC Architects
Kaiser Building
1300 Potrero Avenue
Richmond, CA 94804

By

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DASSE Design Project No. 01B300x2

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10.1 Introduction

The purpose of this report is to perform a seismic assessment of the Kennedy High School in Richmond, 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, which 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

This school was built in 1965. Buildings are numbered from I to IX. Building I, II & III are two story classroom buildings with tilt-up exterior walls with wood floor and roof. Covered walkway were added at the north and west side of the Arts, Crafts and Home Making building (building IV) in the year 1965. Building V is a one story gymnasium building that has both concrete and wood construction. Building V1 is a one story Industrial Arts building with exterior concrete walls and with a wood roof. Building V11 is called Multi-Use building with exterior concrete walls and with a wood roof. Building V111 and 1X are one story wood framed buildings. There are also four toilet buildings labeled Buildings ‘A”, ‘B’, ‘C’ & ‘D’ which are adjacent to buildings V111 and 1X. All the buildings are connected through covered walkways. There are several portable buildings located behind the gymnasium building and these portable buildings vary in dates from 1965 to 1967. The total square footage of the permanent structures is about 207,200 square feet.

10.3 Site Seismicity

The site is a soil classification S_D in accordance with the 2001 California Building Code (CBC) and as per the consultants, Jensen Van Lieden Associates, Inc.

The main building and classroom building have an educational occupancy (Group E, Division 1 and 2 buildings) and the multi-purpose building and the Gymnasium building have an assembly occupancy (Group A, Division 3), all have an importance factor in the 2001 CBC of 1.15. The campus is located at a distance of about 2.6 kilometers from the Hayward fault. The wood framed building utilize a plywood shear wall system to resist lateral loads. This has a response modification factor R = 5.5. Conversely, the concrete shear wall buildings have a response modification factor R = 4.5. 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 for the plywood shear wall buildings in the 2001 CBC is:

$$V = \frac{2.5CaIW}{R} = \frac{2.5(0.44x1.44x1.15)W}{5.5} = 0.33W$$

The seismic design coefficient for concrete shear wall buildings in the 2001 CBC is:

$$V = \frac{2.5CaIW}{R} = \frac{2.5(0.44x1.44x1.15)W}{4.5} = 0.40W$$

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. John F. Kennedy High School – Donald Francis Haines – Zaven Tatarian & Associates, Architects – Sheets A-1 to A-5, A-101 to A-107, A-201 to A-207, A-301 to A-305, A-101 to A-107, A-401 to A-406, A-501 to A-513, A-601 to A-607, A-701 to A-709, A-801 to A-803, A-901 to A-904, A-1001 to A-1002, A-1101, A1105, A1106 & A1107
2. John F. Kennedy High School – Chin and Hensolt Structural Engineers – Sheets S-1 to S-12, S-101 to S-107, S-201 to S-205, S-301 to A-304, S-401 to S-402, S-501 to S-507, S-601 to S-605, S-701 to S-707, S-801 to S-803 & S-901 to S-903. DSA application numbers 25925, 34890, 66096 and 01-100731.
3. Alterations – Seismic Bracing (increment 1) - John F. Kennedy High School Deferred Maintenance Project – HTI Architects; Simpson Gompertz and Heger – Structural Engineers – sheets JFK-S0.0, JFK S0.1, JFK S0.2, JFK S1.1 to S1.6, JFK S8.1 to S8.6.
4. Alterations – Seismic Bracing (increment 1) - John F. Kennedy High School Deferred Maintenance Project – HTI Architects; Simpson Gompertz and Heger – Structural Engineers – sheets JFK-S0.0, JFK S0.1, JFK S0.2, JFK S1.1 to S1.6, JFK S8.1 to S8.6 dated December 1998.
5. Alterations – Seismic Bracing (increment 2) - John F. Kennedy High School Deferred Maintenance Project – HTI Architects; Simpson Gompertz and Heger – Structural Engineers – sheets S0.1, S0.2, S0.2, S1.1, S1.2, S3.1 to S3.3, S5.1, S8.1 to S8.4 dated December 1998.

10.5 Site Visit

DASSE visited the site on August 9th 2002 and November 26th 2002. The main 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

This entire campus was re-roofed approximately in year 2000 per the district maintenance staff. Buildings I and II (facing Cutting Boulevard) are two story buildings see (figures 2&3) with exterior concrete tilt-up walls and with interior plywood shear walls in the transverse direction. The roof and the second floor have plywood sheathing. Both of these buildings were seismically retrofitted in 1988. There is seismic joint between these buildings and the connecting corridor. The building exterior typically has a stone aggregate finish. Covered walkways have plaster finishes at ceilings and are connected at either end to the main buildings. There are no door or window openings on the north wall of these buildings except for the full height door opening (see figure 3 & 4), which leads to entrance lobby and connects with buildings I and II. The entrance lobby has a pre-cast stair that leads to the second floor of buildings I & II (see figure 5). The second floor corridor has continuos window openings on the south longitudinal wall (see figure 6). There is a staircase at the west side of building I.

Building III has not been built, but there is a room noted as 305 per campus key plan in that location.

Building IV is called Arts, Crafts and Home-Making Building (see figure 7). This building was originally constructed in 1956 and some minor modifications were done such as adding partition walls in 1965. New covered walkways were added to the west and north side of the building in 1965. Minor modifications were performed in year 1998 to this building. The building has perimeter concrete walls with steel framing and metal deck at the roof. There are numerous openings on the south longitudinal wall of this building (see figure 8). The building has a metal deck roof with steel trusses supported on steel columns on one side and corridor shear walls on the other side (see figure 9).

Building V is a Gymnasium Building. This building has perimeter concrete walls and a plywood roof at the low roof covering Girls and Boys locker rooms. The roof of the gymnasium building has glu-lam beams spanning 105 feet with metal deck acting as the diaphragm (see figure 10). The north, west and east walls are plywood shear walls, and the south wall is a concrete wall. Glu-lam beams are supported on steel wide flange columns.

Building VI is called the Industrial Arts building. It is a one story building with perimeter concrete walls and with wood framing at the roof and with plywood roof sheathing. There are numerous door and louver openings on the south wall (see figure11). The exterior pre-cast concrete walls have a stone aggregate finish. Two covered walkways connect to this building from the north side and one covered walkway connects to the east side as well. The covered walkway at the north side has a 2” seismic joint.

Building VII is called Multi-Use Building consisting of a cafeteria with an adjacent kitchen wing, an instrumental music building with an adjacent theater (choral and dramatic arts room – see figure 13). It is essentially a single story building with perimeter pre-cast concrete walls. Some of the transverse walls in the cafeteria and kitchen wing and the instrumental music buildings have plywood shear walls. The south and the east walls of the cafeteria/kitchen buildings have numerous openings. The roof of the multi-use building consists of wood framing with glulam beams and plywood roof sheathing (see figure 12).

Building VIII and IX are called Science Wing and Resource Materials Center respectively. They are single story plywood shear wall buildings (approximately octagon shaped in plan) with cement plaster finish on the exterior. The roofs have a combination of wood and steel framing with plywood roof sheathing.

There are numerous covered walkways, which envelop the perimeter of the buildings (see figures 14, & 15). These walkways have plastered ceilings and are supported on pre-cast concrete columns that taper top and bottom. The walkways have seismic joints at various locations.

Toilet Building “C” was seismically retrofitted in year 1998. It was also re-roofed in the same year.

Portable buildings located behind the gymnasium building have numerous openings and do not have enough length of shear walls to adequately resist seismic loads. Steel roof beams have rusted (see figure 16) and are badly in need of repairs.

10.6 Review of Existing Drawings

Building I and II: These are two story buildings with 8” exterior concrete tilt-up panels on the north, east and west sides and with plywood shear walls on the south longitudinal wall. Interior transverse walls are plywood shear walls (3/8” plywood sheathing typical). Both these buildings have one 6” thick pre-cast concrete shear wall in transverse direction (north-south) between ground level to second level only. Exterior 8” thick pre-cast concrete wall have #4 reinforcement at 10” centers in each direction. Gravity framing at the second floor consists of a combination of steel beams and girders with secondary wood framing supported on tube steel columns. Floor sheathing consists of ½” plywood sheathing. The roofs have wood framing with 27-1/8” glu-lam girders and 8x16 joists spanning 28 feet and with 2x16 secondary joists at 24” centers spanning 28 feet. The roof has ½” plywood sheathing. A connecting corridor on the south side connects to two story toilet buildings “A “ and “B”. The stair leading to the second floor of the toilet buildings is a pre-cast concrete stair. The roof of the connecting corridor is lower than the roof of buildings I and II . The plywood shear walls on the south side does not have adequate shear strength due to numerous openings. This shear wall should be strengthened with additional nailing at panel edges, staggered with existing nailing.

Buildings I and 1I were seismically retrofitted in 1998. However, this retrofit work primarily addressed the inadequate anchorage for the perimeter concrete tilt-up walls for out-of-plane loads due to seismic forces. Seismic retrofit design completed in 1998 was performed to meet the 1995 CBC.

In Building I, the interior transverse (north-south) shear walls in the second story do not line-up with the walls in the first story creating an out-of-plane offset that results in a discontinuity in seismic load path. This irregularity is called a plan irregularity and could result in overstressing of the second floor diaphragm and the 3x16 joists that support the upper plywood shear walls in the upper story. Additionally, hold downs required at the ends of the shear walls in the second story are not present. This irregularity represents a life safety hazard. New steel braced frames and foundations as shown in appendix “B” drawings should be provided from the ground to the roof level.

Building II, which resembles building I in lateral and gravity framing system, also has plan irregularities similar to those described for building I with out-of plane offsets of vertical shear wall elements and thus posing a life safety hazard.

The corridor framing consists of 2x8 joists at 24” centers at the roof and at16” centers on floor. The floor consists of a 5/8” plywood diaphragm covered with 1-5/8”concrete fill and the roof has ½” plywood sheathing acting as the diaphragm.

The entrance lobby roof connects buildings I and II and has a big opening in the center, which is framed by 7”x27-1/2” glu-lam beams all around the openings. Secondary framing consists of 2x8 joists (spanning 13’) at 24” centers. There are glu-lam beams (5-1/4”x17-3/4”) on north (on grid A/2) and on the south (on grid C/2). Glu-lam beams support a 2x4 stud wall (with 3/8” plywood sheathing) which extends from the ceiling level to the roof level and acts as a collector to drag seismic loads in east-west direction to the tilt-up concrete shear wall on line A/2. However, this glu-lam beam extends about 3” north of the tilt-up panel edges and does not have a positive connection between the panel and the glu-lam beam. The lack of adequate connection poses a life safety hazard and new positive connections to the tilt-up panels need to be provided. Also, connections between glu-lam beams on this line need to be strengthened to provide continuity. This is true for glu-lam beams on line D/2, which acts as collector elements and hence connections to the tilt-up panels need to be strengthened. Additionally, continuity should be provided across the glu-lam beams by providing steel straps. Pre-cast concrete columns at the main entrance are slender 23 feet tall columns. The connection of the columns at the base is described under the covered walkway section later in this section. Connections at the base need to be strengthened to make sure that these columns (4 total) can adequately resist seismic loads due to their own self weight through cantilever action.

At the full height doors at the front entrance, C9x13.4 mullions with ¼” thick cover plate supporting metal screens also need to be strengthened, since their slenderness ratio exceeds 200.

The foundation system for buildings I and 1I consists of a 8” thick slab on grade with 3’ wide by 18” deep continuos footings with 4-#5 top and bottom reinforcement. Footing also have #3 ties at 18” centers. The pre-cast concrete tilt-up walls have #4 reinforcement at 12” centers horizontal dowels at their bottom connecting to the slab-on-grade. This is the detail used for all the buildings with pre-cast concrete tilt-up panels. Vertical dowels tying the slab-on-grade to the footing consists of #4 reinforcement at 18” centers. The dowels transfer shear forces from the pre-cast walls to the footings. There are no hold-down reinforcement to connect the pre-cast walls to the footings to resist uplift forces due to seismic loads at any of the buildings.

Hold down anchors need to be provided for tilt-up panels adjacent to door openings when the openings are located close to the ends of the panels. In these cases, part of the panels adjacent to the openings will experience uplift loads during seismic events and could experience severe damage and possible failure of the wall sections. This is a possibility for all of the buildings in the campus that have tilt-up panels with the conditions described above.

Building II: This building which was noted as “Alternate A” in the existing drawings, has never been built. Instead, there is a small one story building numbered 305 on the campus key plan. Since we do not have any drawings for this building, no review was undertaken.

Building IV: This building was originally constructed in 1956 with minor modifications performed in 1965. The exterior walls are 8” thick pre-cast concrete tilt-up walls with #4 reinforcement at 18” centers in the horizontal and vertical directions with a double curtain mat of reinforcement. Interior tilt-up walls are 6” thick with #4 reinforcement at 12” centers in each direction. The south exterior longitudinal wall below the large windows typically consists of 8” thick masonry walls with #4 reinforcement at 24” centers horizontal and vertical and with cells fully grouted. This wall has extensive window openings (full height windows –see figure) and lacks adequate length of shear wall to resist seismic loads. Concrete shear walls are made of tilt-up panels that are 18 feet to 32 feet wide and are connected together with a poured in place concrete column (thickness of the concrete columns are equal to the tilt-up wall). Interior longitudinal tilt-up shear walls have numerous door and window openings. All the tilt-up walls have hold down anchorage which consists of 1-1/8” bolt (12”long) embedded into the footings with ½”x4” base plates attached to the bottom of the panels. Panel reinforcement consisting of #8 x2’ long bars are welded to the base plates. The bottom of the panel share access holes 6” wide x 4” deep to install the nuts and washers on the anchor bolts which were then filled with dry pack after panels were erected.

The roof framing consists of metal deck with steel the roof trusses spaced at 7’-6” that span approximately 49 feet. Continuos bridging are provided at 10 feet centers. The trusses are supported on steel beams (8WF24) which are supported by steel pipe columns (3-1/2” dia.). Interior tilt-up panel shear walls (6” thick) require a collector to drag the roof diaphragm loads to the shear walls.

The foundation consists of continuos spread footings 14” wide by 12 ‘ deep at transverse walls and 4’ wide by 1’-10” deep at longitudinal walls supporting 8” block walls. There is typically a 3’ deep stem wall (8” thick) above the footings with #4 at 24” vertical reinforcement and #4 reinforcement at 16” horizontal. Pre-cast concrete tilt-up walls sit on top of these stem walls. The stem walls are connected to a 5’ thick slab on grade with #4 vertical reinforcement at 24” centers.

Building V: This is a Gymnasium Building with a high roof which also has attached locker rooms with low roof. The gymnasium roof framing consists of shaped glu-lam beams, 12-1/4" wide by 6' deep spanning 105 feet at a spacing of 20 feet with 5-1/4"x16-1/4" glu-lam blocking at the ridge. The glu-lam beams are supported at their ends by steel columns (8WFx24). The roof diaphragm consists of metal deck spanning 20'. The gauge and depth of the metal deck is not clear from the existing drawings. Capacity of the deck (due to its excessive span) is questionable to support both gravity) and lateral seismic loads. Further investigation is required by opening the roofing at critical areas to check the deck span and welding and to check any retrofit work has been done to strengthen the metal deck.

The low roof has wood framing with 7"x 26" continuos glu-lam beams spanning 68 feet at 20 feet spacing with intermediate 4" diameter pipe column supports. At end wall, 8x10 wood posts support glu-lam beams. Secondary joists are 2x12 at 24" centers. The low roof diaphragm consists of ½" plywood with panel edge nailing of 8d at 5"centers.

The north, east and west walls of Gymnasium are plywood shear walls with 3x10studs at 16" centers at east and west walls, and 2x8 studs at 16" centers at the north wall. These walls have 3/8" plywood sheathing on both sides of the wall. The east, west and north walls of gymnasium do not have adequate length to resist prescribed seismic loads and this may pose a life safety hazard. These walls need to be strengthened by providing additional nailing staggered with existing nailing. The east and west walls do not have adequate strength to resist prescribed seismic loads below the low roof (locker room roof) and ground level. Top plates of the east and west walls of gymnasium appear to be discontinuous as the deep glu-lam beams (6 feet deep) interrupt them. A bent plate strap connecting the top plates on either side of the 6' deep beams should be added to provide continuity.

The low roof of the locker room typically has perimeter pre-cast 6" thick concrete shear walls. Connections of concrete tilt-up walls to the roof diaphragm lack adequate strength to resist prescribed lateral loads. This may pose a life safety hazard and the connections should be strengthened. Collector connections need to be strengthened on line D between lines 6/5 to 7/5 and on lines 3/5 and 4/5 as they appear to be inadequate to resist seismic loads.

The plywood diaphragm between D/3 and P/5 in the locker room area has inadequate strength to resist prescribed seismic loads in east-west direction due to the long span of the diaphragm. New plywood shear walls, hold downs and collector connections should be provided as shown in drawing appendix "B".

The plywood diaphragm at the dance floor area, weight room and wrestling area between grids B/5 and D/5 has inadequate strength to resist prescribed seismic loads. New plywood sheathing should be added to existing partition walls between 6/5 and 7/5 in the music room, which will create new shear walls and reduce the diaphragm spans. New plywood shear wall should be added on line 5/5 between grids B/5 and D/5 at the weight room and on line 3/5 between grids B/5 and D/5 at the wrestling area.

Building VI: This building is called the Industrial Arts building and is one story with perimeter pre-cast concrete tilt-up walls 8" thick with pre-cast concrete pilasters on the north, south and west walls only. The roof framing consists of 7"x30-3/4" glu-lam beams spanning in north south direction 52'span at 12'-8" centers which are supported at the perimeter by pre-cast walls. At the interior of the building, steel beams are supported by wide flange girders (24WFx76), which are supported on steel columns (8WF24). The roof diaphragm consists of ½" plywood. Roof joists are 2x8 and 2x10 spaced at 24" centers.

There are numerous openings in the south wall with six roll-up doors to provide access for the auto shop. There are door and window openings in the east and west walls. Pre-cast concrete columns or pilasters adjacent to door openings do not appear have hold down anchors which poses a life safety hazard as these columns will experience uplift forces during a strong seismic event. This is true for door openings in tilt-up panels as the area adjacent to the openings can experience uplift forces and hence needs to be retrofitted with hold down anchors. Existing drawings do not show hold down anchors in the pre-cast concrete tilt-up panels and at any of the buildings.

Connections of the concrete tilt-up walls to the roof diaphragm lack adequate strength to resist lateral loads, which poses a life safety hazard. The connections should therefore be strengthened. The roof diaphragm has inadequate strength to resist prescribed seismic loads and should be strengthened.

The foundation for this building is similar to those of buildings I and II.

Building VII: This is a large building consisting of a multipurpose room (MPR) with a 21feet high tall roof area and with an adjacent low roof over the kitchen area on the south side. Covered walkways are on the west and north sides. At the east end of the multi-purpose room is the music room with a 14 feet tall roof. Next to the music room is the theater building with a roof height of 21feet. The perimeter of the building has 8" thick concrete tilt-up panels with a stone aggregate finish on the exterior. Details of the pre-cast panels are similar to those at buildings I & 1I. Interior shear walls have plywood sheathing on both faces of the wall, which are oriented in the north south direction.

The roof of the MPR has ½" plywood (blocked) diaphragm. The roof framing consists of 2x12 joists at 24" centers, spanning between shaped glu-lam beams (11"x 37-47" deep) spanning about 81' between grids B/7 to M/7 and spaced at 12' centers. Glu-lam beams are supported on 6WFx20 steel columns at both the ends on lines M7 and B/7. Kitchen roof framing has a ½" plywood diaphragm with 3x12 at 24" secondary joists framing into glu-lam beams at one end and pre-cast walls at the other end. There is a mechanical well in the center of the kitchen roof.

The MPR has discontinuous shear walls in the east–west direction on lines B/7 and M/7 which poses a life safety hazard. A new lateral system such as a steel braced frames with new foundations should be added along the above grid lines to resist prescribed seismic loads and provide a complete lateral load path from the cafeteria roof to the foundation. It should be noted that on line M/7, there is a short piece (7'-7" length and 9-6" tall) of 10" thick poured in place concrete wall near line 10/7 in north-south direction, which is connected to exterior pre-cast concrete wall. Since the pre-cast walls do not have hold downs to resist seismic overturning forces, the poured in place concrete wall will experience uplift forces. Also, the poured in place wall is about 9-6" tall and supports 2x8 stud wall with plywood sheathing. The 2x8 stud wall on line M/7 is a bearing wall and has 3/8" plywood sheathing from low roof (kitchen roof) to MPR roof (high roof) in east-west direction and thus making it a discontinuous shear wall. The C12x20.7 collector on line M is overstressed as it has to span 12 between the roof joists and requires lateral bracing to reduce the unbraced length.

In the north-south direction for the MPR, lateral resistance is provided by wood shear walls on line 13/7 & 8/7 and by a pre-cast tilt-up wall on line 10/7. Wood shear walls consists of 2x6 studs at 16" centers with 3/8" plywood on both the faces of wall. There is a 22' wide door opening in the middle of this wall on line 13/7 and uplift anchors (hold downs) have been provided at the ends of the openings. However, hold down anchors have not been provided next to the door openings between grids B/7 and E/7 and they need to be provided. The pre-cast wall on line 10/7 between grid lines L/7 and H/7 is a 14' wide panels (panel # 728) with a door opening near the panel edge on line H/7. A similar situation occurs on panel between E/7 and A/7 on line 10/7. These panels will experience uplift forces due to seismic loads and hold downs should be provided at the ends of wall and adjacent to the openings. The collector element on line 10/7 consisting of C8x11.5 channel needs to be laterally braced in order to avoid reliance on cross grain bending of the nailers attached to the channel. Angle bracing (kickers) should be provided from 2x12 roof joist down to the channel member.

On the east side of the MPR, at the low roof area between 13/7 and 14/7 and lines N/7, there are four very short panels (4' long panels) that are required to support tributary roof seismic loads but as well as wall out-of-plane loads. These panels should be retrofitted with hold down anchors mitigate life safety hazard.

At the kitchen area low roof, perimeter pre-cast tilt-up panels and the interior plywood shear walls (2x6 studs with 3/8" ply. on both sides) in the north south direction provide the lateral resistance for this area. The collector connections at the plywood shear walls near P/7 & M/7 in north-south direction on grid 12/7, need to be strengthened.

The Music room between grids 6/7 and 8/7 has a wood framed roof which is about 7 feet lower than the MPR roof. This roof has a ½' plywood diaphragm with 2x12 joist at 24" centers 16.5 feet spans and supported by 7"x 24-5/8" glu-lam beams which span the width of the building and are supported by 8x6 wood posts.

Perimeter tilt-up concrete panels in east-west direction provide lateral resistance to the music room. In the north-south direction, plywood shear walls (2x6 studs with 3/8" ply. for both faces) on line 8/7 together with tilt-up panels on line 6 provide the lateral resistance. A collector on line 6 (at mechanical well roof level) consisting of 2 –2x12 joists lack continuity at line F1/7 and require a steel strap to transfer seismic forces across the 6x8 post located at gird F1/7. Also the collector splice consisting of 12-16d nails is inadequate to transfer the roof diaphragm (music room) forces to the tilt-up panels on line 6.

Adjacent to the music room is the mechanical well between grids 5/7 and 6/7 where the roof is approximately 5 feet lower than the roof of music room. The music room roof has 5/8" plywood sheathing with 2x14 joists at 12" centers. These joists are supported by 6x14 beams spanning the width of the room (15'-4") and supported by 6x6 posts. Tilt-up panels on lines 5/7 and 6/7 provide lateral resistance to seismic loads in the north-south direction. Wood shear walls (2x6 studs with 3/8" ply. on both faces of wall) between lines G/7 and F/7 provide lateral resistance in the east-west direction. Tilt-up panels (#722 & #710) have door openings close to the panel edges and should be retrofitted with hold down anchors.

Next to the mechanical well is the theater area (between grids 1/7 to 8/7) consisting of Choral and Dramatic Arts rooms. The roof has ½" plywood sheathing with 2x12 joists at 24" centers. The joists are supported by 7"x 27-1/2" glu-lam beams spanning the width of the room (32') except for the glu-lam in the center which is 9" x 32-1/2". There is a low roof area to the west (between grids 1/7 and 3/7). Roof framing at this low roof area consists of 2x6 joists (spanning 14') which are supported at their ends by a wood shear wall and by perimeter tilt-up panels.

Tilt-up panels on lines C/7 and K/7 provide the lateral resistance in east west direction. Additionally there are two wood shear walls below the low roof area between 1/7 and 2/7. Tilt-up panels on lines 2/7 and 5/7 provide the lateral resistance in the north-south direction. The diaphragm spans 72' in north-south direction and may be slightly inadequate, from seismic forces originating at perpendicular tilt-up panel. The 2-2x8 top plates that act as a collector on line 3 are interrupted by 32" deep glu-lam beams. However, continuity is provided by 3/8" thick steel plates attached (bolted) to top and bottom plates on either side of the glu-lam beam which are welded to 3/8" thick plates attached to the sides. The collector on line 5/7 is inadequate and requires strengthening.

Connections of concrete tilt-up walls to the roof diaphragm lacks adequate strength (for all the buildings denoted by #V11) to resist lateral loads. This may pose life safety hazard and the connections should be strengthened.

The foundation is similar to those described for buildings I and II.

Buildings VIII and IX: These two buildings are similar in shape. Both are one story wood framed buildings which have footprints roughly polygon in shape. Exterior walls of both buildings have 3/8" plywood sheathing on the exterior. The layout of interior shear walls is different in either building. Building IX, being a library building, has fewer interior shear walls than building VIII.

Roof framing in building VIII consists of 2x12 joists at 16" centers with22' spans, which are supported by glu-lam beams. The glu-lam beams are supported by 6x6 wood post. The typical span of glu-lam (5-1/4"x 24-1/4") beams are 36'. The roof framing is symmetrical (with minor exceptions) about the ridgeline which is oriented in the north-south direction. The roof sheathing consists of ½" plywood. The roof connection (on lines B/8 and B/9) utilize 5'' angle clip at 4 feet centers which do not have adequate capacity to transfer out-of –plane forces. The nails would be in withdrawal (detail 15/S12), which is not a desirable connection. The connections need to be strengthened (at all location in other buildings where this detail 15/S12 has been used).

The roof framing in building IX is 2x14 joists at 16" centers from A/9 to F/9. Joists are supported by 2x6 bearing walls. From grid F/9 to J/9, 2x12 joist at 24" centers (spanning 18 feet) frame into into 5-1/4”x19-3/8” glu-lam beams (span 44’) or to W24x68 steel beams (span 56’). Wood posts (6x6) support the glu-lam beams and W5x16 columns support the steel beams. Collector connections need to be retrofitted for the interior shear walls on line F/9. The roof sheathing is ½” plywood. There is a roof opening (24’x13’) on the north side centered on the ridgeline. The roof diaphragm strength may be inadequate to resist prescribed seismic forces in north- south direction. Shear walls strength may be inadequate to resist prescribed seismic forces (walls on grid 2/9 and 9/9 between A/9 & C/9 and walls on grids 5/9 and 7/9 between D.5/9 and E.5/9). The collector connection is not adequate to resist seismic forces on lines 2/9 and 9/9. Collector connections were not provided to drag the seismic loads into the wall on grids 5/9 and 7/9 between D.5 & E.5 and J/9. This may pose a life safety hazard. Additional shear walls need to be added to reduce the diaphragm spans and new collector elements should be added. At the collector connection of tilt-up panels on line C/9 to 2-2x16 members, the existing drawings calls for a detail, which is not applicable to this condition. This connection needs to verified and strengthened as required.

On both the buildings, a covered walkway extends on all the four sides of the building. On the north side of the buildings, covered walkways have seismic joints.

Foundation consists of typical spread footing under columns and continuous spread footing under walls with 4” slab on grade.

Covered walkway framing consists of shaped 2x8 joists at 24” centers spanning 9 feet which are supported at one end by the exterior shear wall of building and at the other end by 3x12 joists. The 3x12 joists are supported by 12” x 12”pre-cast concrete columns. The roof has 3/8” plywood sheathing. At the top and bottom of the pre-cast columns, there are pipe and tube steel inserts, which project from the top by about 1” and the bottom of column by about 8”. At the bottoms, TS4x4x3/8 insert are connected to the footings through base plates and anchor bolts. At the tops, ½” plates are welded to the cap plate of insert, which in turn is used to support the wood joists. The tube steel columns are not adequate to resist is forces due to moment from cantilever action and the connection needs to be strengthened as this may pose a life safety hazard. This is true for *all covered walkway columns* in the campus.

The column reinforcement consists of 4 -#6 vertical reinforcement with #3 reinforcement ties at 8” centers. The lateral system for the covered walkways is provided by the pre-cast concrete columns by resisting the lateral loads through cantilever action.

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 for 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.	All Buildings – For all tilt-up panels, no hold-downs are provide to resist uplift forces due to seismic loads..
2.	Buildings I & 1I - Interior transverse plywood shear walls have plan irregularity. This results in overstressing of the 2nd floor diaphragms and overstressing of the 4-3x16 joists that support the 2nd floor shear walls. Additionally, holdown ties at the 2nd floor shear walls are not present.
3.	Building I&1I – Entrance lobby roof - On lines A and C glu-lam beams lack continuity and needs to be positively connected to develop connector force to transfer roof diaphragm forces.
4.	Building I & II – Entrance lobby Columns – Pre-Cast columns lacks adequate strength at bottom connection to transfer seismic forces. Also, the steel channel mullions supporting entrance metal screen are slender and needs to be strengthened.
5.	Building II - Plywood shear walls at 1st floor at the South face of the building lack adequate shear strength to resist prescribed seismic forces.
6.	Building IV – Longitudinal wall on the south has excessive openings and lack adequate length of shear wall to resist seismic forces.
7.	Building IV – Collector connection to pre-cast concrete wall on north longitudinal wall is inadequate.
8.	Building IV – Interior shear wall in transverse direction needs a collector to drag

	seismic loads.
9.	Building V - Plywood shear walls along East and West boundary of gymnasium lack adequate strength to resist prescribed seismic forces
10.	Building V - Pre-Cast concrete wall anchorage connections at low roof area lack adequate strength to resist the prescribed out-of-plane seismic forces.
11.	Building V - Metal deck at Gym roof spans 20’ and the capacity of the deck to support gravity and lateral loads is questionable.
12.	Building V – Roof diaphragms between grids B to D and between grids 6 to 9 & I to 4 and roof diaphragm between grids A to C & grids 4.2 to 5.8, lack adequate strength to resist prescribed seismic forces.
13.	Building V – Roof diaphragm between grids 6 to 9 & 1 to 4 and D to P lack adequate strength to resist prescribed seismic forces.
14.	Building V – Collector connection on line D/5and 7/5 is inadequate and needs to be strengthened
15.	Building V – Top plates at gymnasium high roof are discontinuous and thus not providing continuity of chords required to resist seismic loads.
16.	Building VI - Pre-Cast concrete wall anchorage connections to roof lack adequate strength to resist the prescribed out-of-plane seismic forces
17.	Building VI - In-plane shear-transfer connections between diaphragm and shear walls lack adequate strength to transfer prescribed seismic forces.
18.	Building VI - Roof diaphragm lacks adequate shear strength to resist prescribed seismic forces.
19.	Building VII - Pre-Cast concrete wall anchorage connections to roof lack adequate strength to resist the prescribed out-of-plane seismic forces
20.	Building VII – Collector connections along Grids 5, 6 and 10 have inadequate strength to resist the prescribed seismic forces.
21.	Building VII - Collectors at grid 10 and grid M at cafeteria roof have inadequate strength to resist the prescribed seismic forces.
22.	Building VII - Collector connection at grid line 11.8 for plywood shear wall running north- south direction lacks adequate strength.
23.	Building VII – Shear walls on lines B/7 and M/7 supporting cafeteria roof are discontinuous and there is no path to transfer roof diaphragm forces to foundation.
24.	Building VII – Connection between roof and plywood shear wall is inadequate to resist seismic force.
25.	Building IX – Diaphragm is overstressed between grids 1 to 10 for seismic loads in north-south direction.
26.	Building IX – Connection detail called out at grids 2/C & 9/C are not applicable as the collector connection is to pre-cast panel and not to plywood wall as detailed in the referenced connection detail.
27.	Building IX – Collector connection at grid F /7 is inadequate to transfer seismic

	loads.
28.	Portable building has numerous openings and lack adequate length of shear wall to resist seismic force. Also, steel roof beams have rusted and are badly in need of repairs.

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.	All Buildings – For all tilt-up panels, check adequacy to resist seismic uplift forces and provide holdowns as required at the ends of panels and adjacent to openings	1.2	1
2.	Provide new steel braced frames from ground to roof level	1.0	1
3.	Provide positive connection between the glu-lam beams and the pre-cast panel to transfer collector force. Also provide strap across glu-lam beams to provide continuity	1.1	2A
4.	Strengthen the connection at the base of pre-cast columns by reinforcing the TS 4X4 and by providing additional anchor bolts. Also, strengthen existing channel member.	1.5	2
5.	Provide additional nailing at panel edges and wall perimeter to strengthen existing plywood shear wall	1.5	2
6.	Fill-in existing window for two bays at each end with new 8" thick wall and strengthen foundation as required and provide collector connection	1.1	3
7.	Strengthen collector connection to pre-cast concrete wall on north longitudinal	1.5	3
8.	Provide new collector to transfer seismic forces	1.5	3
9.	Strengthen the plywood shear walls with additional nailing at the panel edges and wall perimeters, staggered with the existing nails	1.1	4
10.	Strengthen the concrete wall anchorage connection with additional wall anchors, diaphragm ties, and diaphragm nails at plywood diaphragm as required.	1.0	5
11	Verify manufacturer's allowable metal deck diaphragm values. Field verify at critical locations the welding of deck to support members and check to see if any retrofit work has been done to the metal deck	1.0	5

12.	Provide new plywood shear wall at locations shown on drawings to reduce the span of diaphragm	1.1	4
13.	Provide new plywood shear wall at locations shown on drawings to reduce the span of diaphragm	1.1	4
14.	Strengthen collector connection	1.5	4
15.	Provide bent strap plate to provide continuity	1.5	5
16.	Strengthen the plywood shear walls with additional nailing at the panel edges and wall perimeters, staggered with the existing nails	1.0	6
17.	Strengthen connection by providing additional Simpson L50 angles	1.2	6
18.	Strengthen diaphragm by providing additional nailing and provide new blocking as required	1.5	6
19.	Strengthen the plywood shear walls with additional nailing at the panel edges and wall perimeters, staggered with the existing nails	1.0	7
20.	Strengthen connection by providing steel strap as required	1.5	7
21.	Provide angle kickers from roof joists down to channel collectors to laterally brace the collector members	1.5	7
22.	Strengthen connection by providing steel strap as required	1.8	7
23.	Provide new steel braced frame and foundations at lines M and B from ground to support cafeteria high roof.	1.0	8
24.	Provide Simpson L50 to supplement existing L50 angle and provide new blocking as required. This is to be provided wherever detail 15/S12 is called out on the drawings.	1.2	9
25.	Provide new shear wall at locations as shown in drawings	1.1	10
26.	Field verify connection detail provided and strengthen as required.	1.8	11
27.	Strengthen connection by providing new steel strap and blocking as required	1.5	11
28.	Portable building needs to be removed	1.2	N/A

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 be performed to this school campus in Phase 1B.

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

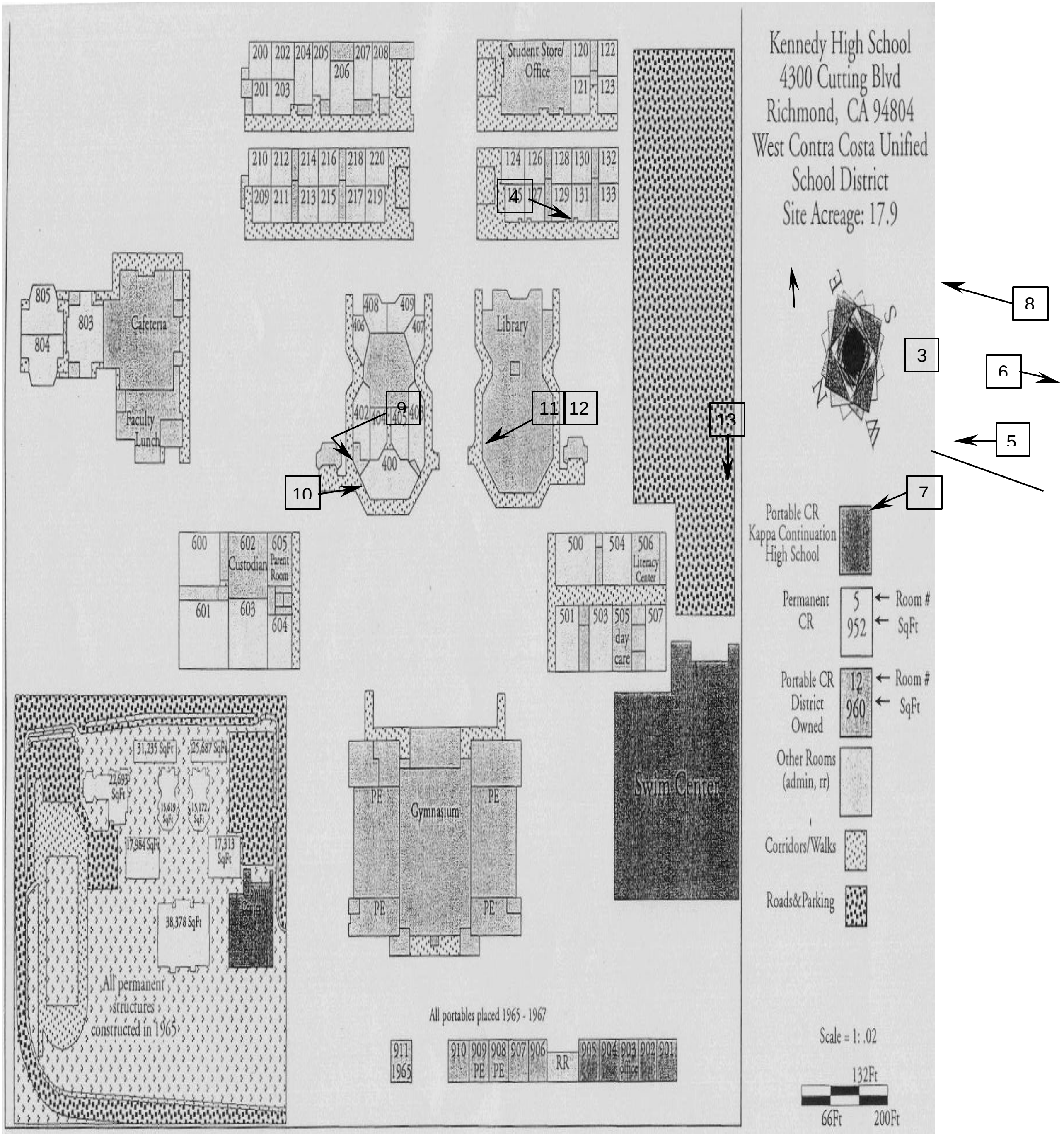


Figure 1: School Layout Plan

Figure 2: Building 11 North Side

Figure 3: Main Entrance from Cutting Boulevard

Figure 4: Front Entrance Door with Tall Pre-Cast Columns

Figure 5: Main Entrance Lobby.

Figure 6: Building 1 Second Floor Corridor

Figure 7: West Elevation of Building IV

Figure 8: South Wall of Building IV

Figure 9: Roof Framing of Building IV

Figure 10: Gymnasium Building Roof Framing

Figure 11: South Wall of Industrial Arts Building

Figure 11: Roof Framing of Multi-Purpose Room

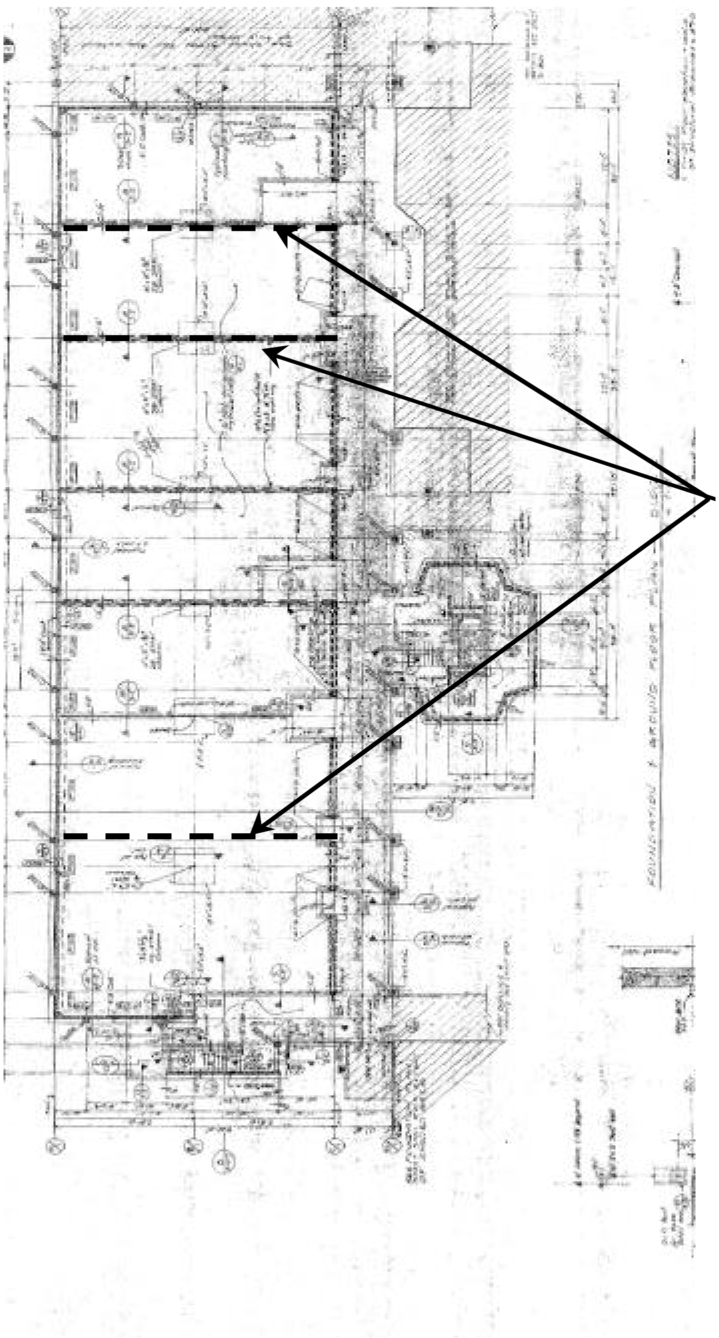
Figure 13: South West Elevation of Theater Building

Figure 14: Typical Covered Walkway with Seismic Joint

Figure 15: Covered Walkway Connecting Buildings 1 & 11


Figure 16: Exterior Wall of Portable Building

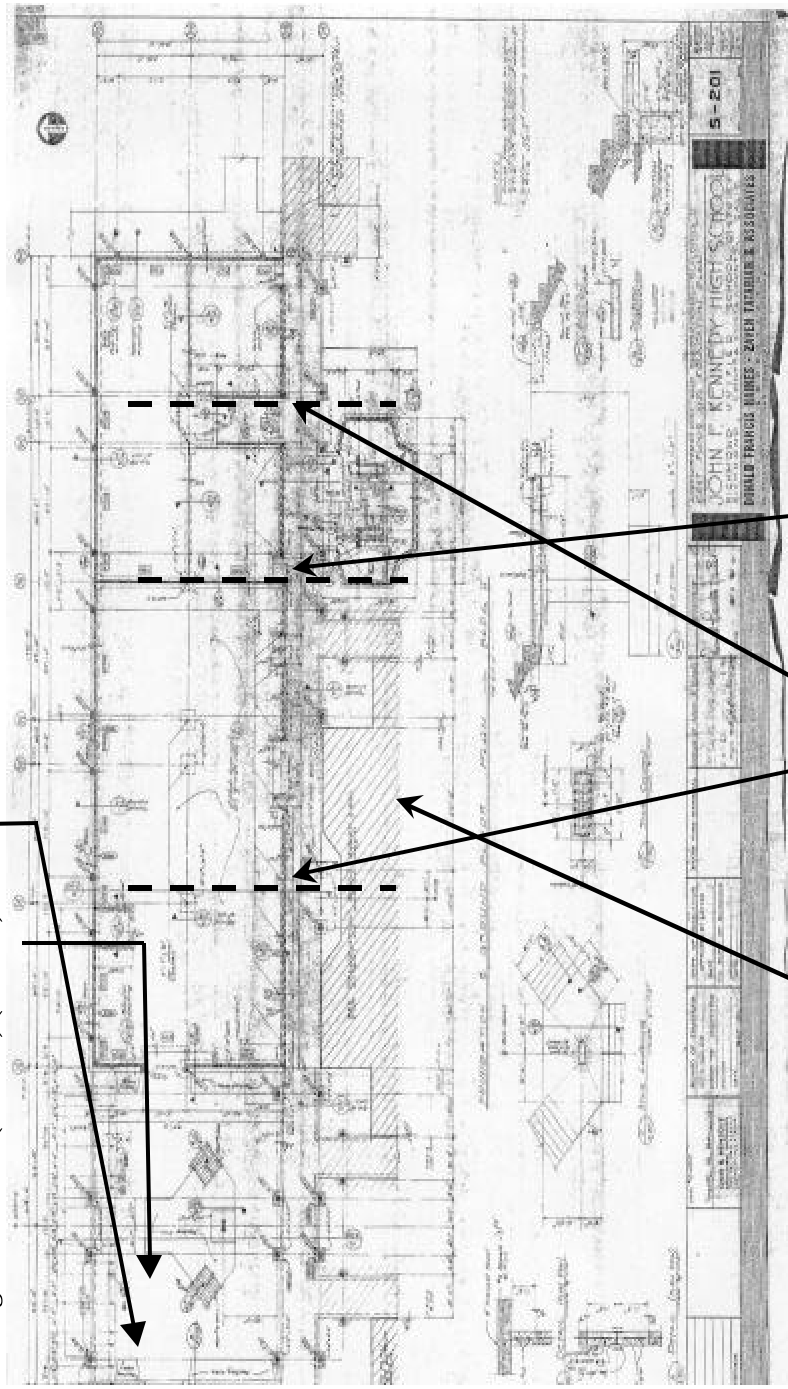
Appendix B – Drawings



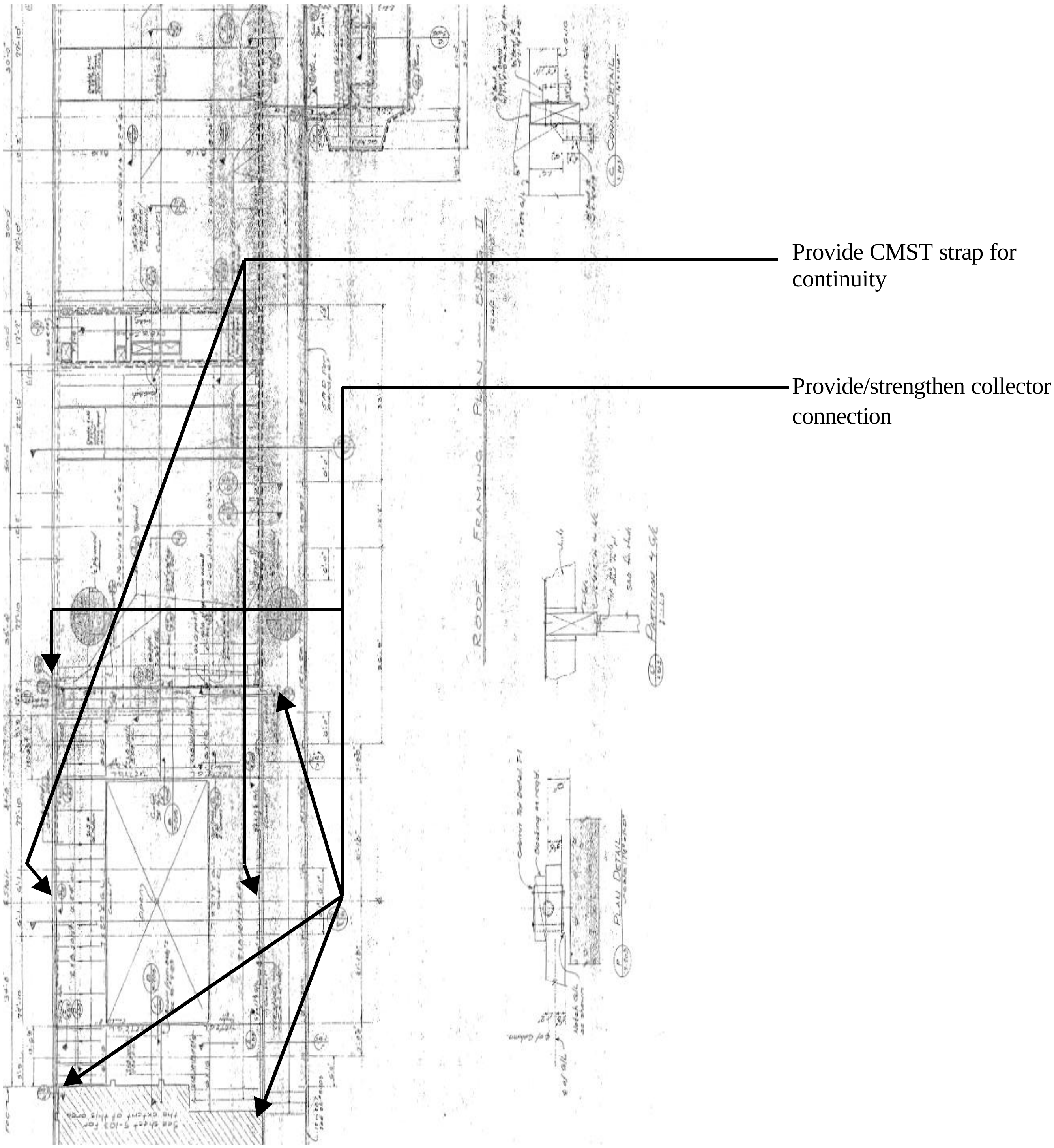
Provide new steel braced frame from ground to roof level along these grid lines. Provide new foundations for braced frames.

NOTE:
Check adequacy and provide hold-downs as required at the ends of panel and adjacent to openings for all tilt-up panels. Typical for all buildings.

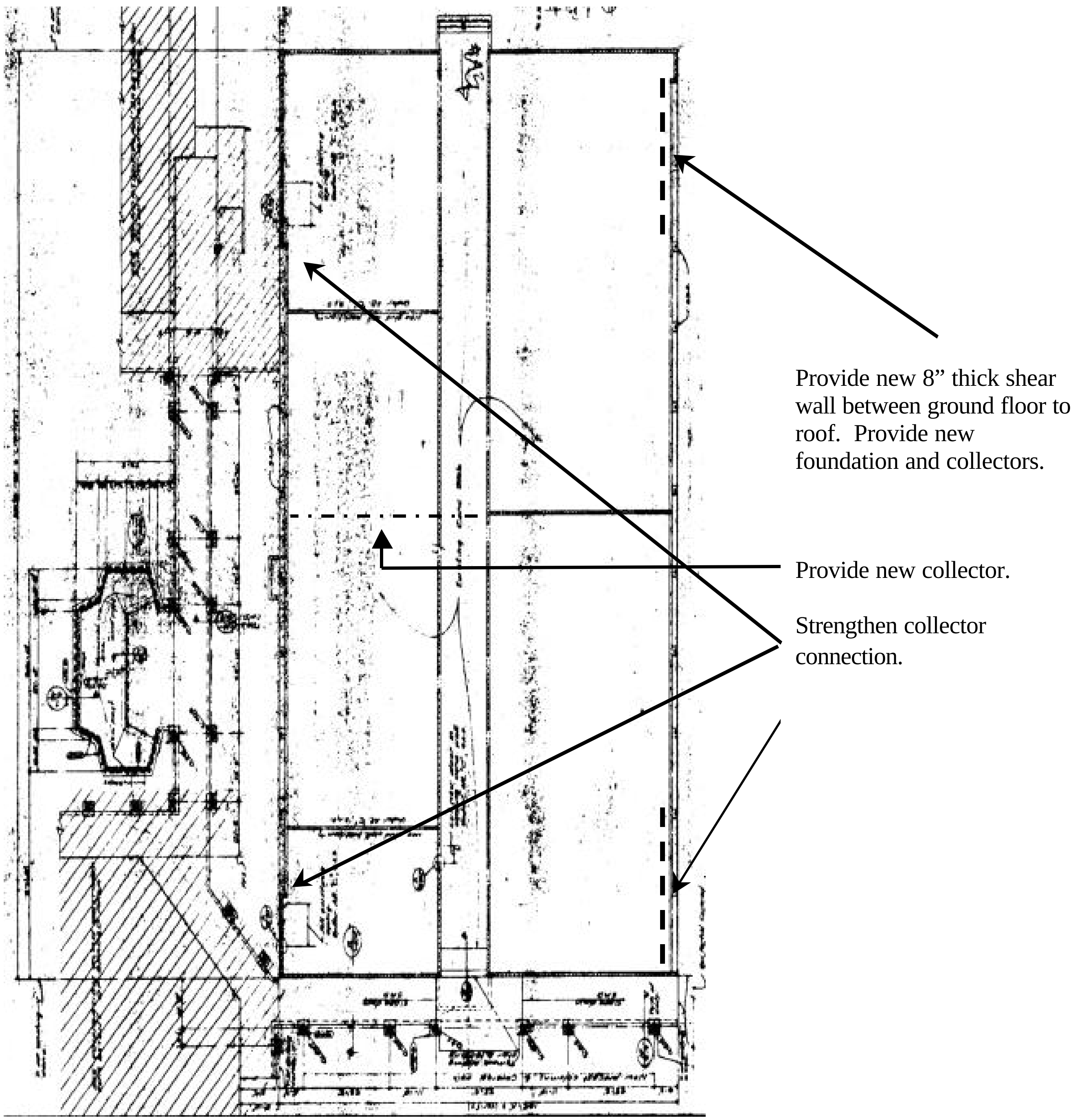
Drawing 1: Building I Foundation Plan



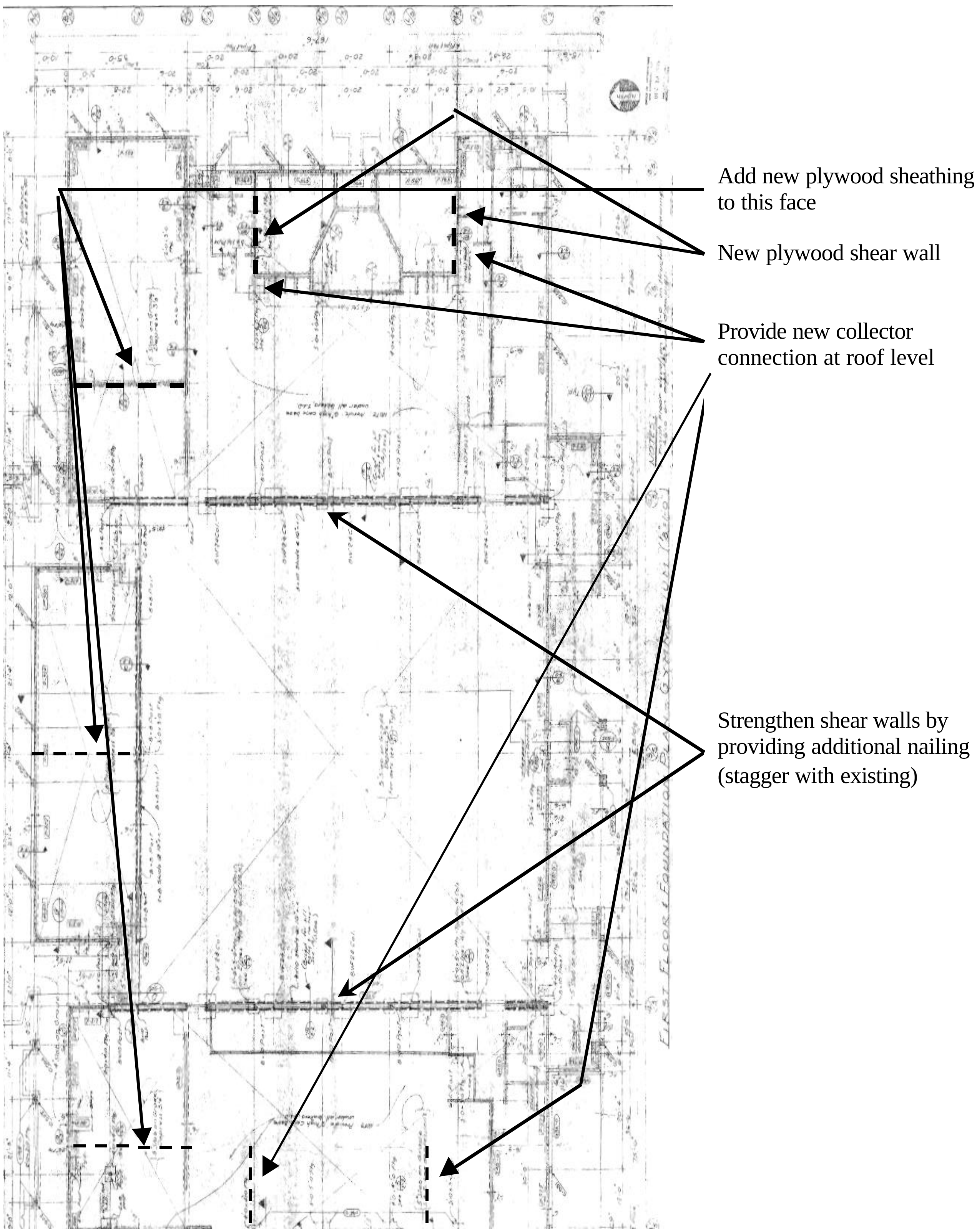
Drawing 2: Building II Foundation Plan



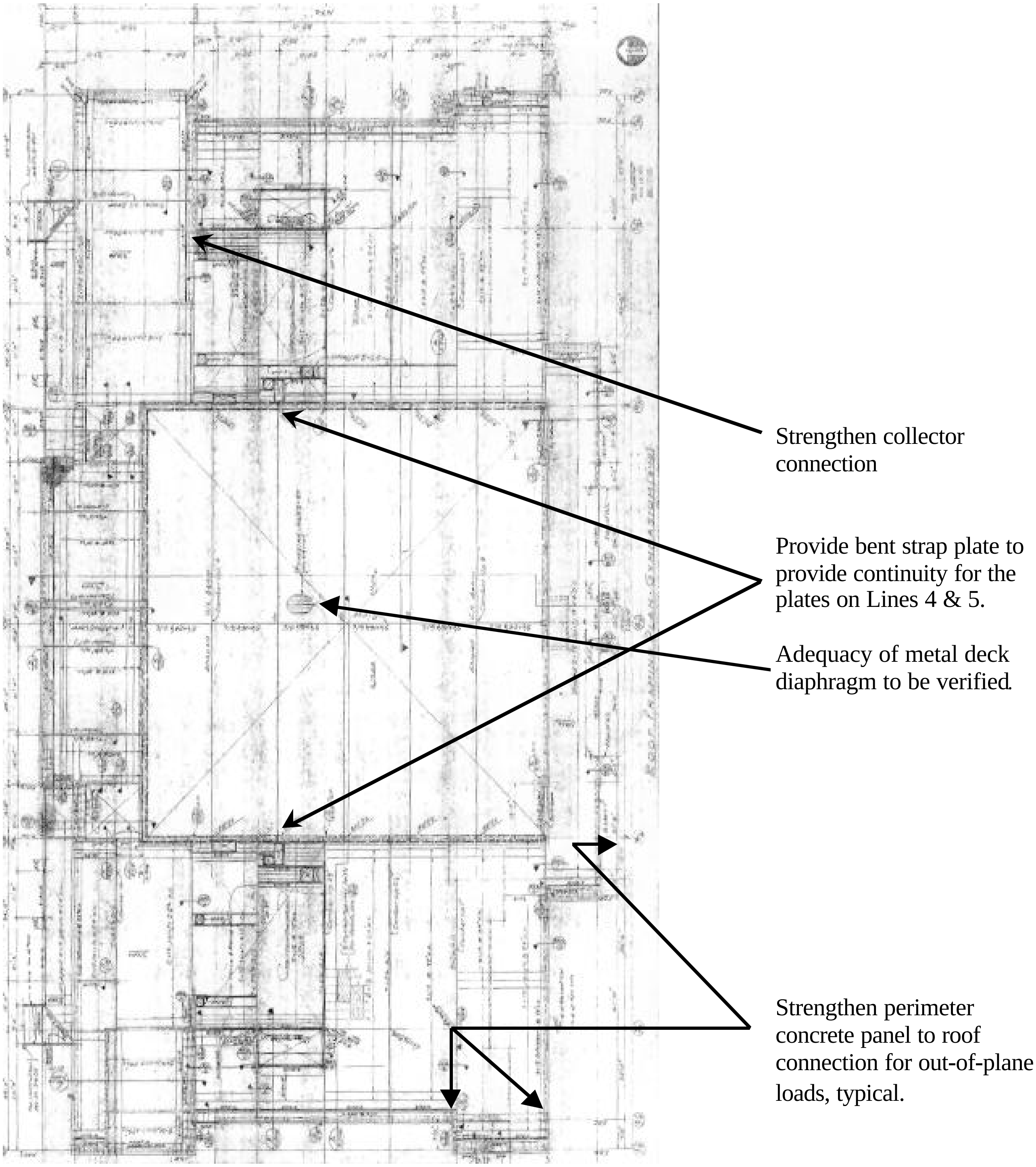
Drawing 2A: Building II Foundation Plan



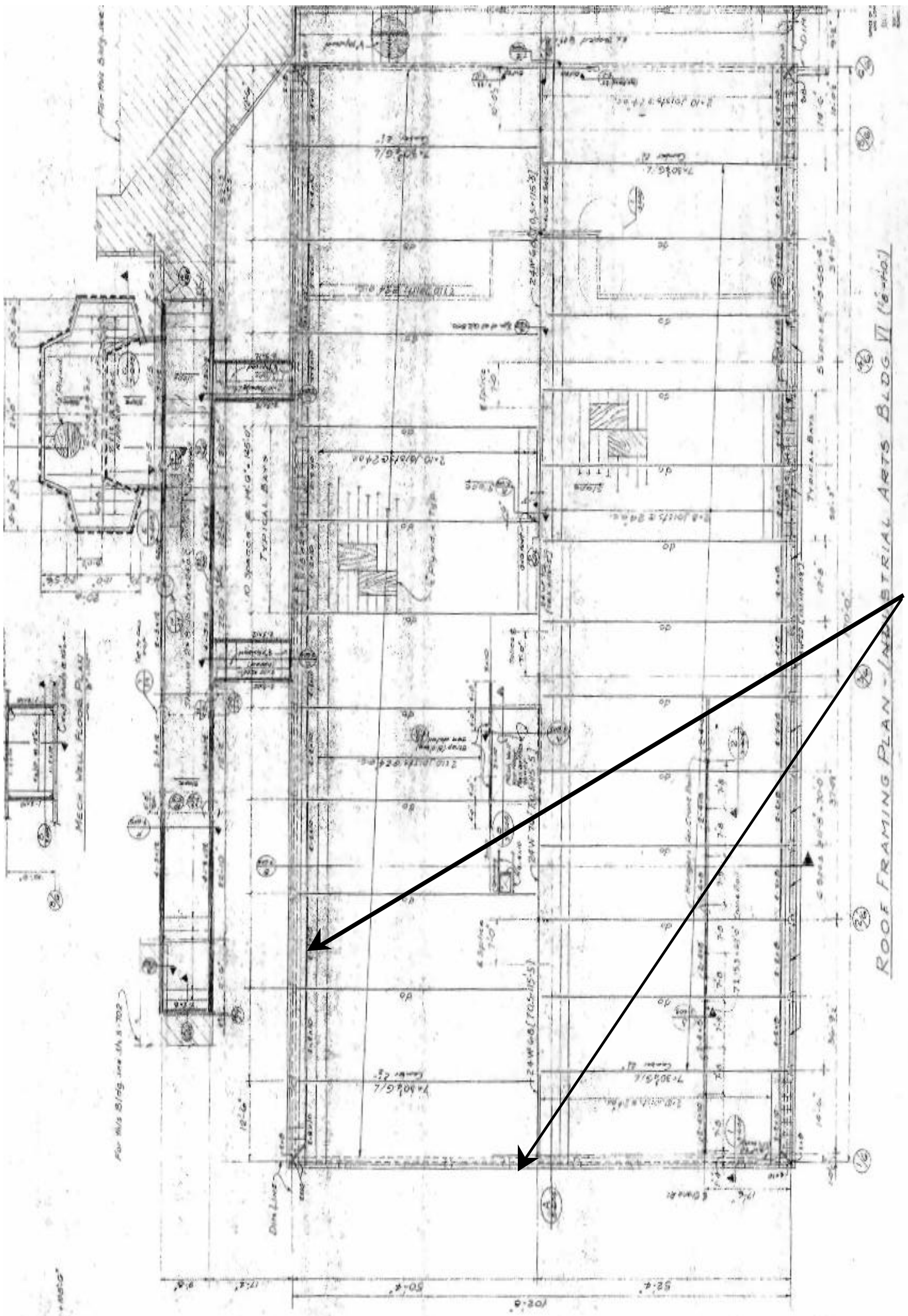
Drawing 3: Building IV Foundation Plan



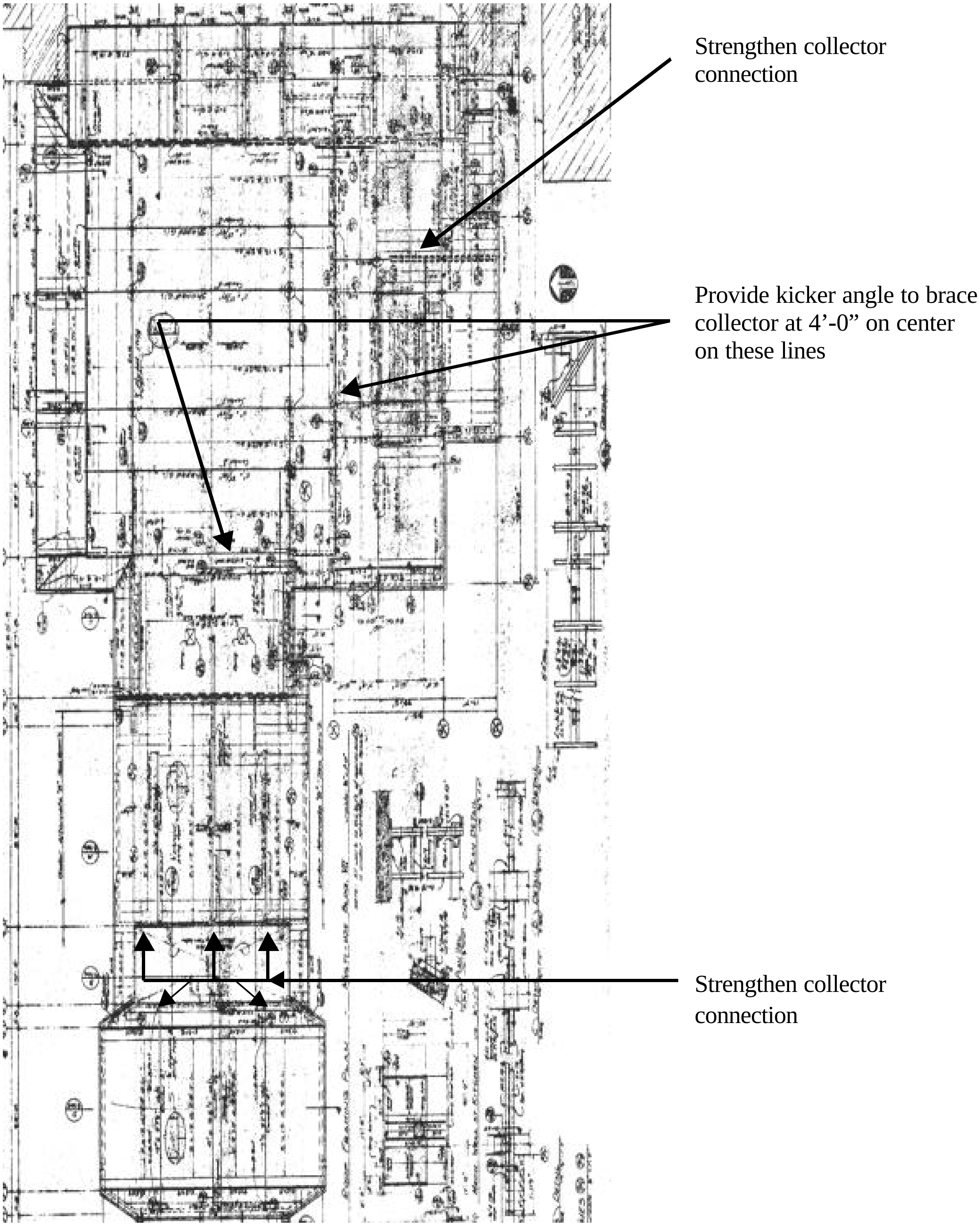
Drawing 4: Building V Foundation Plan



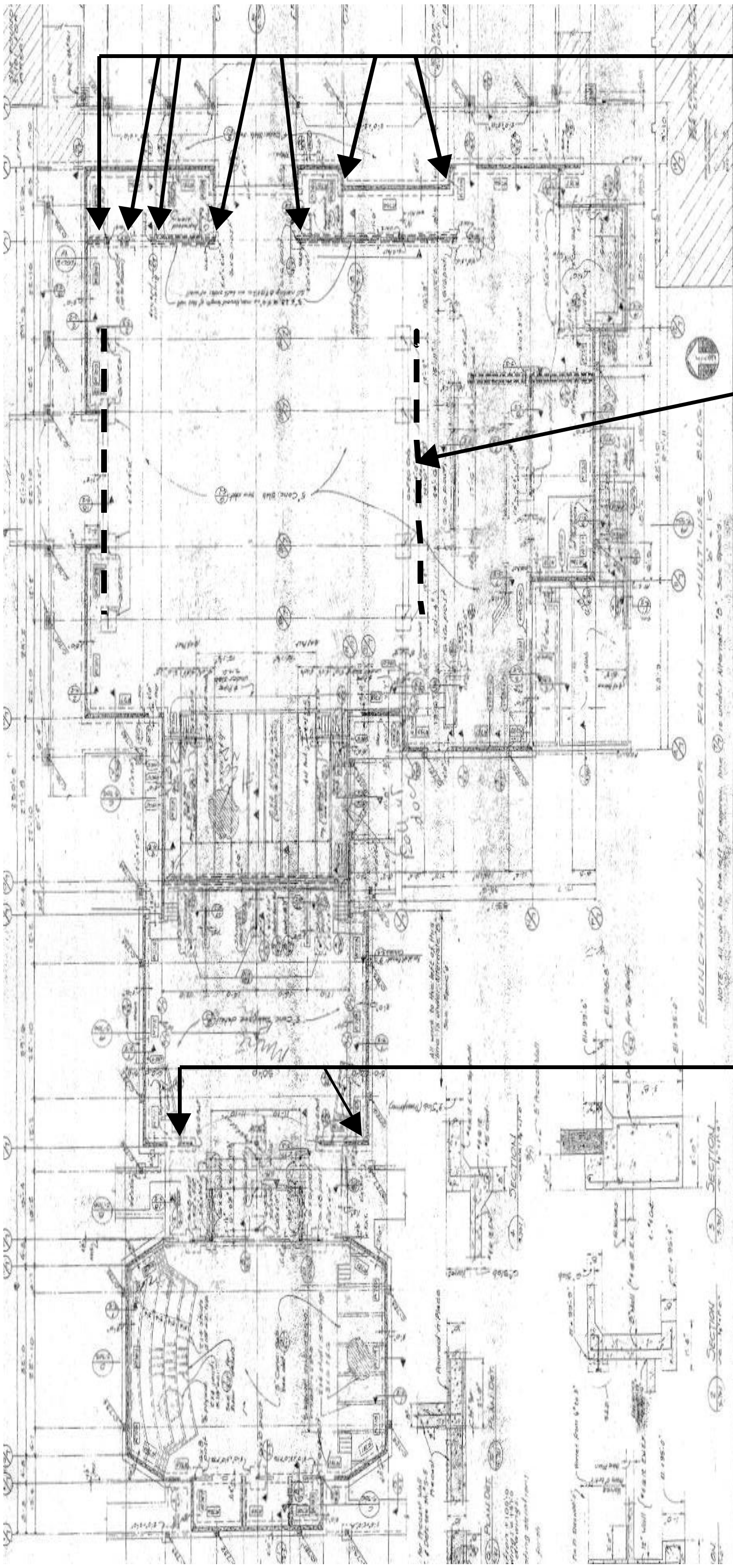
Drawing 5: Building V Roof Plan



Drawing 6: Building VI Roof Plan



Drawing 7: Building VII Roof Plan



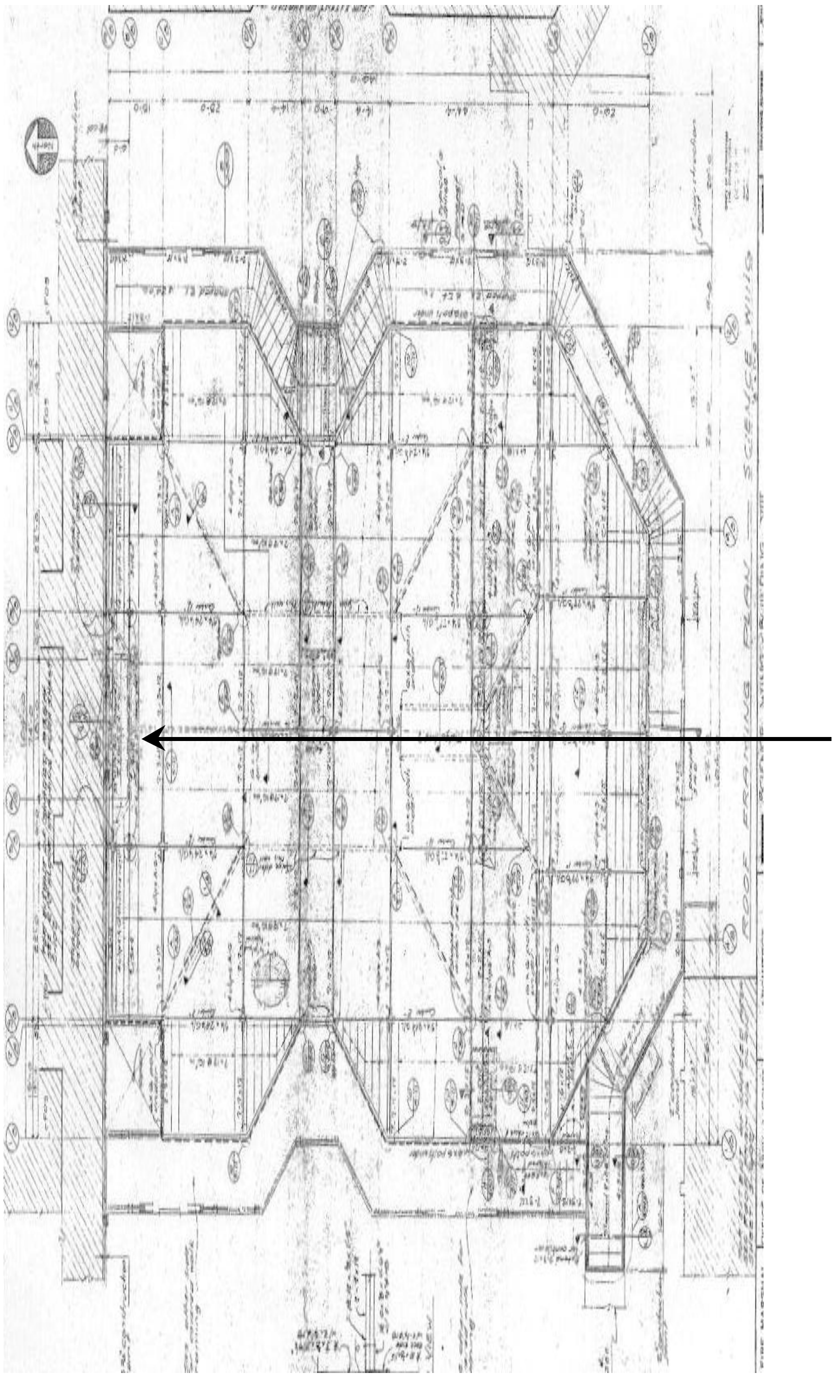
Provided hold-down anchors at ends of tilt-up panels

Provide new braced frame on lines B/7 & M/7 from ground to multi[purpose high roof and collectors as required.

Provide hold-down anchors at ends of tilt-up panels.

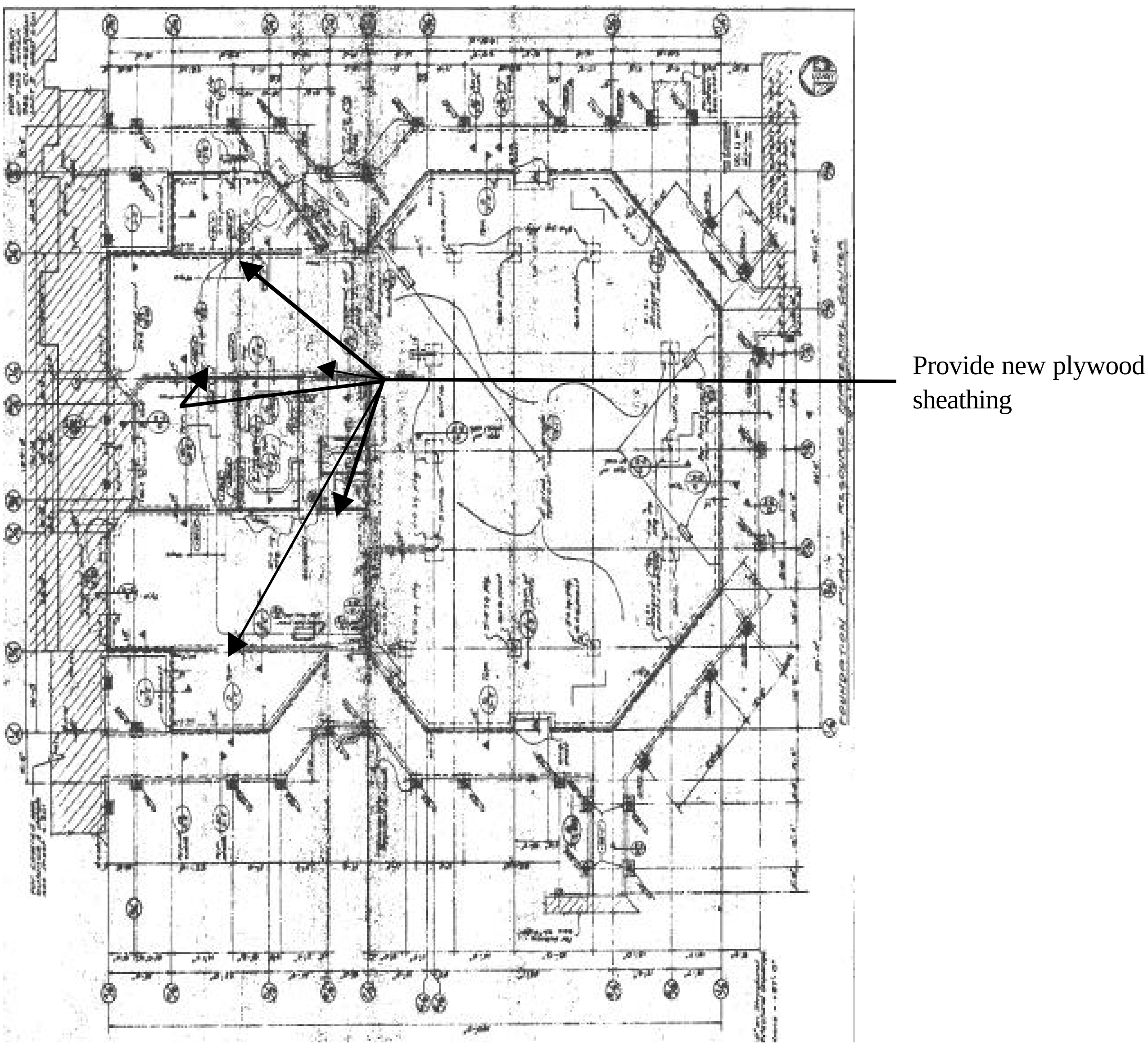
NOTE: Strengthen connection between perimeter tilt-up panel and roof (for wall out-of-plane loads) typical.

Drawing 8: Building VII Foundation Plan

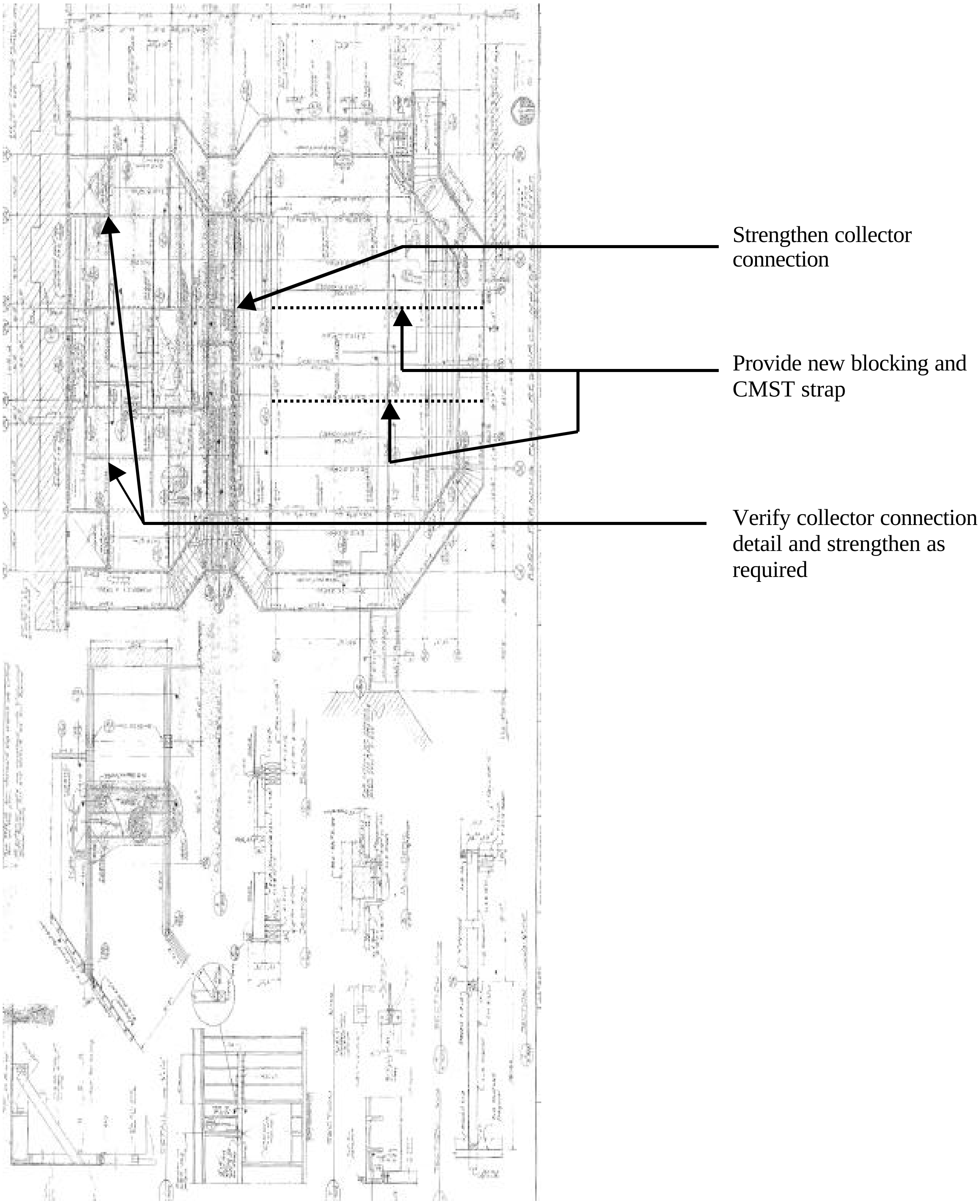


Strengthen roof to shear wall connection on this line

Drawing 9: Building VIII Roof Plan



Drawing 10: Building IX Foundation Plan



Drawing 11: Building IX Roof Plan