STRUCTURAL EVALUATION OF

RICHMOND HIGH SCHOOL

WEST CONTRA COSTA UNIFIED SCHOOL DISTRICT (WCCUSD)

For

WLC Architects
Kaiser Building
1300 Potrero Avenue
Richmond, CA 94804

By

DASSE Design, Inc.
33 New Montgomery Street #850
San Francisco, CA 94105
(415) 243-8400

October 17, 2002

DASSE Design Project No. 01B300x2

TABLE OF CONTENTS

Page No.

10.1 Introduction 1

10.2 Description of School 1

10.3 Site Seismicity 1

10.4 List of Documents 2

10.5 Site Visit 3

10.6 Review of Existing Drawings 5

10.7 Basis of Evaluation 8

10.8 List of Deficiencies 8

10.9 Recommendations 10

10.10 Portable Units 12

10.11 Structural Deficiency Prioritization 12

10.12 Conclusions 13

10.13 Limitations and Disclaimer 13

APPENDIX A: FIGURES

- Figure 1: School Layout Plan
- Figure 2: Campus Entrance
- Figure 3: Theatre Building (Unit D), exterior east wall
- Figure 4: Classroom Building (Unit D), exterior east wall
- Figure 5: Classroom Building (Unit D), exterior east wall
- Figure 6: Common Area (Unit D), interior southwest corner
- Figure 7: Common Area (Unit D), interior southeast corner
- Figure 8: Common Area (Unit D), interior at stage
- Figure 9: Theatre Building (Unit D), interior of auditorium
- Figure 10: Common Area (Unit D), interior facing clerestory windows
- Figure 11: Old Gymnasium Building (Unit B), exterior northwest corner
- Figure 12: Old Gymnasium Building (Unit B), exterior east wall
- Figure 13: Old Gymnasium Building (Unit B), exterior south wall
- Figure 14: Old Gymnasium Building (Unit B), exterior southeast corner of locker rooms
- Figure 15: Classroom Building (Unit C), exterior south wall
- Figure 16: Classroom Building (Unit C), exterior east wall
- Figure 17: Gymnasium Building (Unit E), exterior north wall
- Figure 18: Gymnasium Building (Unit E), interior southeast corner
- Figure 19: Gymnasium Building (Unit E), exterior south wall
- Figure 20: Gymnasium Building (Unit E), exterior southwest corner
- Figure 21: Music Building (Unit F), exterior northwest corner
- Figure 22: Shop Building (Unit A), exterior north and west walls
- Figure 23: Shop Building (Unit A), exterior north wall
- Figure 24: Shop Building (Unit A), interior at roof framing
- Figure 25: General (Unit D), conduit crossing seismic joint

APPENDIX B: DRAWINGS

- Drawing 1: Old Gymnasium Building (Unit B), Foundation Plan
- Drawing 2: Old Gymnasium Building (Unit B), Low Roof Framing Plan
- Drawing 3: Old Gymnasium Building (Unit B), High Roof Framing Plan
- Drawing 4: Classroom Building (Unit C), Roof Framing Plan
- Drawing 5: Classroom Building 100 (Unit D), Foundation Plan
- Drawing 6: Classroom Building 200 (Unit D), Foundation Plan
- Drawing 7: Classroom Building 100 (Unit D), Roof Framing Plan
- Drawing 8: Classroom Building 200 (Unit D), Roof Framing Plan
- Drawing 9: Classroom Building 300 (Unit D), Foundation Plan
- Drawing 10: Classroom Building 300 (Unit D), Roof Framing Plan
- Drawing 11: Classroom Building 400 (Unit D), Foundation Plan
- Drawing 12: Classroom Building 400 (Unit D), Roof Framing Plan
- Drawing 13: Classroom Building 500 (Unit D), Foundation Plan
- Drawing 14: Classroom Building 600 (Unit D), Foundation Plan
- Drawing 15: Classroom Building 700 (Unit D), Foundation Plan
- Drawing 16: Classroom Building 600 (Unit D), Roof Framing Plan
- Drawing 17: Classroom Building 700 (Unit D), Roof Framing Plan
- Drawing 18: Classroom Building 800 (Unit D), Roof Framing Plan
- Drawing 19: Gymnasium Building (Unit D), Low Roof Framing Plan
- Drawing 20: Gymnasium Building (Unit D), High Roof Framing Plan

10.1 Introduction

The purpose of this report is to perform a seismic assessment of the Richmond 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

The majority of the current campus dates back to 1968 when the largest portion of the school was demolished and reconstructed. The construction that took place at this time included Unit D (multiple classroom, library, commons, and theatre buildings) and Unit E (gymnasium and locker room buildings). All of these buildings are single story, and the majority are wood framed; however, there is a significant use of concrete frames and masonry walls. A few of the buildings on campus originated prior to this major rebuild. Unit A, the autoshop building, appears to have been a part of the original campus construction, built circa 1946. The old gymnasium and locker room building, Unit B, is a single story structure from 1953 that uses multiple construction materials. Another classroom building, Unit C, is a two story tilt-up concrete building that dates back to 1965, only three years before the major reconstruction.. Finally, the campus includes Unit F, the music building that was added in 1979, to complete the campus in its current form. In addition to these permanent structures, the campus also includes seven portable buildings. Six of these portable classrooms make up Omega Continuation School and were erected on following timeline: three between 1965 and 1969, one in 1994, one in 1998, and one at an unknown date. The final portable building serves as the adult education office and is believed to have been added sometime after 1979. Exclusive of these portables, the total square footage of the permanent structures is approximately 226,510 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 classroom buildings of Unit A, C, D, and F have an educational occupancy (Group E, Division 1 and/or 2) while the theatre and gymnasium buildings of Unit B, D, and E have an assembly occupancy (Group A, Division 1, 2, and/or 2.1). All of these buildings have an importance factor in the 2001 CBC of 1.15. The campus is located at a distance of less than 2.0 kilometers from the Hayward fault. Unit B resists seismic forces with a masonry shear wall system that has a response modification factor of R = 4.5. Concrete shear walls resist seismic forces in Unit C, giving that classroom building a response modification factor of R = 4.5. The non-ductile moment frames in the center portion of Unit D used to resist lateral forces are a system that is prohibited by the 2001 CBC in seismic zone 4. The plywood shear wall buildings of Unit D and E have a response modification factor R = 5.5. The theatre building of Unit D that uses masonry shear walls (some of which are also bearing walls) has 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 concrete and masonry shear wall buildings in the 2001 CBC is:

$$V = \frac{2.5C_a IW}{R} = \frac{2.5(0.44x1.50x1.15)W}{4.5} = 0.422W$$

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

$$V = \frac{2.5C_a IW}{R} = \frac{2.5(0.44x1.50x1.15)W}{5.5} = 0.345W$$

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. Richmond High School; Unit B; Schmidts & Hardman Architects; sheets A-1 – A-11, S-1 – S-5, H-1 – H-2, P-1 – P-2, E-1 – E-2; July 7, 1953; DSA application #11095.
2. Richmond High School; Units C; Akol & Associates/Jens Hansen & Associates; sheets A1 – A13, S1 – S4, P-1 – P-2, M-1 – M-2, E-1; February 8, 1965; DSA application #25770.
3. Richmond High School; Units D & E; Hardison and Komatsu Architects; Rutherford and Chekene Structural Engineers; sheets A-1.1 – A-11.3, S-1 – S-49, M-1.1 – M11.2, FP-1 – FP-9, E-1 – E-33, FS-1 – FS-9, L-1 – L-9; August 30, 1968; DSA application #30465.
4. “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 13th, 2002 and October 18th, 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

The shop building, Unit A, is a one story building that is isolated from the rest of the campus at the opposite end of the parking lot. According to one teacher, this building was originally an independent print shop that was acquired by the campus at some point. This information in combination with the fact that original construction drawings were unable to be located either by the district or the Division of the State Architect (DSA) suggests that this building may have never received DSA approval. If this is in fact the case, it is likely that DSA will require a mandatory seismic upgrade (per the provisions of the 2001 California Building Code) of the structure. The requirements of this type of upgrade would exceed both in scope and cost, the remediation procedures recommended by this report. The structure is observed to be of very old construction that uses a hybrid framing system. The roof is framed with riveted roof trusses that support wood beams. The diaphragm is composed of a combination of straight sheathing and steel tension rods that are located at the bottom chord of the steel trusses and span in the weak direction of the straight sheathing. The minimal strength of these two diaphragm systems appears to be inadequate in comparison to the expected seismic forces, and constitutes a life safety hazard. The roof is punctured with many openings, including a large skylight. Some of the building’s interior walls are unfinished revealing diagonal sheathing; however, these walls are not connected to the tension rods for the transfer of seismic forces. The exterior walls have a stucco finish, and have extensive window and garage door openings. Due to the large number of openings, the walls appear to have inadequate strength to resist the expected seismic forces, and consequently a life safety hazard is perceived. Unit A is shown in figures 22, 23, and 24.

The original gymnasium building, Unit B, is a combination of concrete, steel, and masonry construction. The taller gym area is supported by steel and concrete frames that are infilled with masonry. This masonry stops short of the roof, below continuous high windows that extend around the perimeter gym. This discontinuity forces lateral loads to be transferred solely through both strong and weak axis bending in the columns. Of the remaining locker room portion of the building, half is masonry construction, where further windows similarly inhibit the lateral strength of the building. The other half of the locker room portion differs from the rest of the building by its exterior stucco finish. Although masonry construction may be found beneath the stucco finish it clear that these two parts of the locker room were constructed separately. The lack of shear strength at the walls with extensive windows appears to constitute a life safety hazard in this building. Unit B is shown in figures 11, 12, 13, and 14.

Unit C, the two story building, was built of concrete tilt-up construction with a wood framed roof. A suspended T-bar ceiling was observed in the classrooms. On the exterior a cast-in-place concrete stair provides access to the second floor. Sufficient lengths of concrete wall exist to provide resistance to lateral seismic forces. Although the wall to diaphragm connection was not observable, it is noted that buildings of this type and age typically have insufficient anchorage for the concrete walls under out-of-plane loading. Unit C is shown in figures 15 and 16.

As the largest and most predominant building on campus, Unit D houses a number of functions. In addition to multiple classrooms, the cafeteria, administration offices, theatre, library, and a large commons area can be found in Unit D. The exterior of the building has a full height brick veneer up to an exterior soffit. The building is centered around a large common space with high ceilings. Surrounding this area, shorter classroom areas make up the perimeter, while the theatre wing extend from the south end. From a structural perspective, Unit D is composed of six separate structures, the seismic joints of which are apparent from the exterior of the building. In the classroom areas, the suspended ceiling is observed to be damaged in some areas. The interior walls in this area are drywall. At the central common space, the high roof is supported by concrete frames that extend both oriented both in a radial pattern and at the perimeter lines. Although it is not possible to observe the full detail of the construction, it is noted that concrete moment frame systems from this era traditionally have performed poorly in large seismic events. The hard ceiling in the common area is located at the structure, and the walls have interior wood paneling. The brick veneer that is typical at the exterior also occurs at some interior locations, most notably in the theatre and adjacent entrance area. Surrounded by common area, the circular library has a lower roof than the rest of the central area. High clerestory windows encircle this lower roof area facing into the common area, while additional tall, narrow windows occur between the library and common areas. The combination of these characteristics results in a system with a large diaphragm discontinuity and an unclear load path. Without the proper detailing this system could pose a life safety hazard. Additionally, hard conduits were observed crossing the various seismic separation joints. The inability of this conduit to withstand the differential movements of these separate structures is identified as a life safety hazard. Unit D is shown in figures 3, 4, 5, 6, 7, 8, 9, 10, and 25.

Unit E is the newer gymnasium building on campus. Supporting the roof, concrete frames spanning approximately 80 feet are exposed from the main gym below. Similar to the common area at Unit D, the hard ceiling is located at the structure level. Differing from the brick veneer of Unit D, the exterior of the new gymnasium has a stucco finish along. However, both buildings have the same exterior soffit at the roof level. Extensive lengths of wall indicate that the structure has adequate strength to resist lateral seismic loads. Unit E is shown in figures 17, 18, 19 and 20.

The music building located in Unit F appears that it may have been built by tilt-up construction. However, given its exterior stucco finish and similar appearance to the adjacent Unit E, it is possible that it may be wood framed structure. The main band room has a high acoustical tile ceiling, while the rear portion of the building has a mezzanine level that is used for storage.

Some high windows were observed near the roof level, but the large amount of solid wall suggests that the building has sufficient shear wall strength. Unit F is shown in figure 21.

10.6 Review of Existing Drawings

Construction drawings for Unit A, the shop building, were not available for review at the time of this assessment; therefore, no review was undertaken.

The old gymnasium building at Unit B is one story structure with two distinct wings. The north half of the shorter locker room wing was built prior to 1953. Although the original construction drawings are not available for this work, the building was upgraded during the construction of Unit D. Those drawings illustrate traditional wood framed construction with interior steel pipe columns. Plywood sheathing was added to both the roof and walls in the retrofit. The other half of the locker room wing is framed with 2x6, 2x8, and 2x10 roof joists spaced typically at 2’-0” centers. These joists are sheathed with 1” straight and diagonal sheathing. At the locations with straight sheathing, 5/8” diameter steel tension rods are provided for the transfer of diaphragm forces. In addition to being a poor system under cyclic loads, these rods lack adequate strength to resist the expected seismic forces. Steel trusses span the transverse width of the locker room to support the roof joists and bear on the masonry walls. These masonry walls serve as shear walls for the resistance of lateral forces; however, the presence of numerous window openings severely inhibits the strength of some of these walls in comparison to the expected earthquake forces. Additionally, the out-of-plane anchorage connections of these walls are inadequate at some locations.

The taller, gymnasium wing of Unit B is supported primarily by three interior steel frames and two concrete frames at the exterior walls. A 4-1/2” metal deck spans the substantial 20’ distance between frames. In the direction perpendicular to the frames, 3/4” and 1-1/8” diameter steel tension rods have been provided to transmit the diaphragm forces to the exterior walls lines. Based on the configuration of these rods, it appears that the original design intended to use both the exterior masonry shear walls and the interior steel frames to resist lateral forces. However, the performance of steel moment frames from this era is suspect, and thus should only be considered as secondary frames. Because the gage of the metal deck used is not apparent from the drawings, this deck as a diaphragm is considered inadequate to resist the expected seismic loads. The tension rods have a similar inadequacy. A band of high windows runs the complete perimeter of the building, which requires the lateral forces from the roof to be transferred through column shear in order to reach the walls. This system has a serious inadequacy of strength. At the high roof the masonry walls span horizontally between the columns. Both the out-of-plane flexural strength of these walls and their anchorage connections have inadequate strength relative to the expected seismic forces. The entire structure of Unit B is founded on a shallow foundation of spread and strip footings or varying sizes. Inadequacies in the strength of diaphragms, shear walls, masonry walls out-of-plane, and wall anchorage connections constitute life safety hazards at Unit B.

Built just prior to the major reconstruction, the classroom building of Unit C uses a structural system that is relatively unique to the campus. This two story building is a concrete wall structure that was built by the tilt-up method. The roof level is wood framed with a 1/2” plywood sheathed diaphragm. Due to the heavy nature of the concrete wall construction, the expected diaphragm forces at the roof level are very high resulting in a wood diaphragm with inadequate shear strength. Typical 2x14 roof joists are spaced at 2'-0" centers and span between the exterior concrete walls and two interior column lines. At the column lines, a combination of sawn and glue-laminated beams span between 6x6 wood posts. Differing from the roof framing, the second floor is composed of an 8" lightweight concrete structural slab. This one-way, flat slab spans between W18x64 steel beams on four interior lines and the exterior concrete walls. Below the second floor, the steel beams are supported by 6" diameter steel pipe columns. Lateral forces in this system are resisted solely by the four exterior concrete walls. The substantial wall lengths provide adequate resistance to the expected lateral forces. Furthermore, the cast in place concrete floor slab provides solid out-of-plane support for the walls; however, an adequate connection is absent at the roof level. The existing wall to roof anchorage connection fails to provide adequate strength relative to the expected out-of-plane seismic forces. Collector forces are adequately transferred by horizontal bars provided in the concrete walls at both the roof and floor levels. The building bears on standard 2'-0" and 2'-4" strip footings at the walls and 6'-0" square spread footings at the interior columns. The life safety hazards identified at Unit C are the inadequate diaphragm strength and the inadequate strength of the wall anchorage connections at the roof level.

Unit D includes a number of separate structures that utilize a variety of building materials. The typical perimeter buildings are single story structures that utilize typical wood framing. The roof framing consists generally of 2x12 joists at 16" centers, and is covered with 1/2" plywood sheathing. The joists are supported by typical 2x6 stud bearing walls and glue-laminated beams spanning between steel wide flange columns. Lateral forces are resisted by plywood shear walls composed of 1/2" plywood at exterior walls and 3/8" plywood at interior walls. Both the plywood diaphragms and shear walls are well designed and have relatively good nailing. However, in comparison to the seismic forces expected by today's standards these diaphragm and walls have deficiencies in strength in some locations. The plywood shear walls have positive connections to the foundation at critical locations. A double top plate is provided as a typical collector member at each wood framed wall. The splice of this double top plate has a limited capacity, but because the plywood shear walls are nearly continuous at most lines, the anticipated collector demand is small at most locations. In general the splices of collector members are well detailed, but in some discrete locations the splice strength is substantially inadequate in comparison to the expected seismic collector force. The wood stud walls bear on typical 1'-4" wide strip footings, while the columns are supported by a single 24"ϕ drilled pier at each location. At the wood framed portions of Unit D, deficiencies are found at some locations in the strength of the plywood shear walls and diaphragms. At discrete locations a life safety hazard is identified in the inadequate strength of collector splice connections.

Like the classroom buildings, the taller theatre building of Unit D is framed with 2x12 roof joists at 16" centers. These are supported by pre-cast concrete beam/column bents that span the 97'-2" transverse width of the auditorium. The roof diaphragm is sheathed with 1/2" plywood to match the other buildings; however, the vertical elements resisting the lateral seismic forces at the theatre building are masonry shear walls. In the transverse direction, this diaphragm spans an excessive 137'-3". Although the transverse concrete frames could reduce this diaphragm span, they lack both the stiffness and strength to provide adequate resistance. In comparison to the expected seismic loads, the roof diaphragm fails to achieve the strength demanded by this extreme span. While sufficient shear wall lengths are provided, the anchorage of these 8" CMU walls at the roof level is inadequate to resist the expected out-of-plane wall forces. The transfer of collector forces is adequately provided for by reinforcement in the top of the CMU walls. The CMU walls are founded on typical 1'-6" strip footings, and the concrete bents bear on a 30"ϕ drilled pier and pier cap at each column. Significant life safety hazards at the theatre building of Unit D exist in the inadequate diaphragm strength and the inadequate CMU wall to roof anchorage connection strength.

The central common area of Unit D, utilizes the structural framing methods previously described in a more complex configuration. Offset from the center of the overall space, the low roof of the circular library is framed with 2x12 roof joists at 16" centers that are supported by glue-laminated beams configured in a radial pattern. These glu-lam beams are carried by two central W27x84 steel beams and perimeter concrete columns. The remainder of the high roof is framed with 2x14 roof joists with spacing varying from 12" to 16" centers. At this level the roof framing is supported by pre-cast concrete beam/column bents similar to those at the theatre building. In this application the concrete bents are oriented radially in line with the glu-lam beams below. Additionally, at each corner of the common areas two level concrete frames are found in both of the principle axes. While both the high and low roof diaphragms are sheathed with 1/2" plywood, the difference in elevation between the two represents a drastic discontinuity. Collector forces are transferred through the concrete beams, whose strength is adequate at some locations when compared to the expected seismic forces. Lateral forces in the structure are resisted through the concrete bents acting as moment frames. For the time of their construction these concrete frames are well detailed, and do not have a brittle shear failure mode. Additionally, because the concrete bents were pre-cast in a shop it can be expected that the quality of the materials and construction was above average. However, in the context of these assets, concrete moment frame systems from this time period tend to behave with a non-ductile response. Because the interior concrete columns are not well braced out-of-plane, it is not expected that they will provide legitimate resistance to lateral loads. When the system is thus limited to resisting the expected seismic forces through only the exterior concrete frames, both these frames and the plywood diaphragms are found to have inadequate strength. Similar to the other buildings of Unit D, the concrete and steel columns bear on a drilled pier and grade beam foundation. The life safety hazards identified at the central common area of Unit D include the inadequate strength and ductility of the concrete moment frames, the inadequate strength and continuity diaphragm, and the inadequate strength of the collector beams.

Unit E was built along with Unit D and is composed of the same framing systems used there. The taller gymnasium portion of the buildings is framed with 2x12 roof joists at 16" centers carried by concrete beam/column bents. This system is nearly identical to the previously discussed theatre building, except that the 113' span here is slightly larger here. Over the lower locker room portion of the building, the typical 2x12 roof joists at 16" are supported by various bearing walls and glue-laminated beams. The roof diaphragm over the gym is sheathed with 1/2" plywood as used elsewhere, while the shorter diaphragm spans over the locker room area have 3/8" plywood sheathing. The large span across the gymnasium results in large diaphragm forces at the high roof and, thus, a diaphragm of inadequate strength. A combination of typical plywood shear walls and three concrete shear walls resist the lateral forces at Unit E. The plywood shear walls are composed of 1/2” plywood at exterior walls and 3/8” plywood at interior walls. The longest if the concrete walls serves as a divided between the high and low roof areas. When the expected seismic force from the low diaphragm is applied to this wall in the out-of-plane direction, it is found to have inadequate flexural strength. Additionally the anchorage connection of this wall to the high roof diaphragm lacks the strength required by the expected seismic forces. Double top plates or various wood beams serve as collector members to each of the shear walls; however, splices and connections of these members have inadequate strength at some of the low roof locations. The typical shallow foundation system used at Unit E is composed of 1’-4” strip footings and a drilled pier/pier cap at each of the column locations. The inadequacies in diaphragm strength, collector connection strength, wall flexural strength (out-of-plane), and wall anchorage connection strength constitute life safety hazards in Unit E.

Construction drawings for Unit F, the music building, were not available for review at the time of this assessment; therefore, no review was undertaken.

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.	Unit A: Walls with extensive window openings are likely to lack adequate shear strength to resist the prescribed seismic forces.
2.	Unit A: Horizontal truss composed of steel tension rods and straight sheathed diaphragms are likely to lack adequate strength to resist the expected seismic forces.
3.	Unit B: Masonry walls with extensive window openings lack adequate shear strength to resist the prescribed seismic forces.
4.	Unit B: Masonry wall anchorage connection lacks adequate strength to transfer the expected out-of-plane seismic forces.
5.	Unit B: Masonry walls lack adequate flexural strength to resist the expected out-of-plane seismic forces.
6.	Unit B: Horizontal diaphragm truss composed of steel tension rods and metal deck diaphragm lacks adequate strength to resist the expected seismic forces.
7.	Unit C: Plywood sheathed diaphragm lacks adequate shear strength to resist the expected seismic forces.
8.	Unit C: Concrete wall anchorage connection lacks adequate strength to transfer the expected out-of-plane seismic forces.
9.	Unit D, 100/200: Plywood sheathed walls lack adequate shear strength to resist the expected seismic forces.
10.	Unit D, 100/200: Plywood sheathed diaphragm lacks adequate shear strength to resist the expected seismic forces.
11.	Unit D, 300: Plywood sheathed walls lack adequate shear strength to resist the expected seismic forces.
12.	Unit D, 300: Plywood sheathed diaphragm lacks adequate shear strength to resist the expected seismic forces.
13.	Unit D, 300: Wood collector connections lack adequate strength to resist the expected seismic forces at critical locations.
14.	Unit D, 400: Plywood sheathed diaphragm lacks continuity and adequate shear strength to resist the expected seismic forces.
15.	Unit D, 400: Concrete moment frames lack adequate ductility and strength required to resist the expected seismic forces.
16.	Unit D, 400: Concrete collector beams lack adequate strength and lateral bracing to resist the expected seismic forces.
17.	Unit D, 500: Plywood sheathed walls lack adequate shear strength to resist the expected seismic forces.
18.	Unit D, 600/700: Plywood sheathed diaphragm lacks adequate shear strength to resist the expected seismic forces.
19.	Unit D 600/700: Masonry wall anchorage connection lacks adequate strength to transfer the expected out-of-plane seismic forces.
20.	Unit D, 800: Plywood sheathed diaphragm lacks adequate shear strength to resist

Item	Building Structural Deficiencies
	the expected seismic forces.
21.	Unit D, 800: Masonry wall anchorage connection lacks adequate strength to transfer the expected out-of-plane seismic forces.
22.	Unit D: Hard conduits crossing seismic separation joints lack the ability to withstand differential building displacements.
23.	Unit E, Gym: Plywood sheathed diaphragm lacks adequate shear strength to resist the expected seismic forces.
24.	Unit E, Locker Room: Wood collector connections lack adequate strength to resist the expected seismic forces at critical locations.
25.	Unit E, Gym: Concrete wall anchorage connection lacks adequate strength to transfer the expected out-of-plane seismic forces.
26.	Unit E, Gym: Concrete walls lack adequate flexural strength to resist the expected out-of-plane seismic forces.

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 plywood sheathing at existing wood stud walls with new hold-down anchors into foundation.	1.0	N.A.
2.	Remove existing straight sheathing and provide new plywood sheathing at roof with nailing into existing framing.	1.0	N.A.
3.	Provide new concrete wall in-fill at existing window locations with dowels into the existing masonry wall.	1.0	1, 2
4.	Strengthen concrete wall anchorage connection with new wall anchors and diaphragm ties.	1.1	2, 3
5.	Provide new steel “strong back” columns from the floor to roof diaphragm with anchors into existing concrete walls.	1.5	1
6.	Provide new double angle members and connections with anchorage into lateral load resisting members to create a new horizontal diaphragm truss.	1.5	2, 3
7.	Provide new double angle members and connections with anchorage into lateral load resisting members to create a new horizontal diaphragm truss.	1.5	4
8.	Strengthen concrete wall anchorage connection with new wall anchors and diaphragm ties.	1.1	4
9.	Provide new second layer of plywood sheathing on the backside on the existing wood shear walls with edge nailing to match the existing.	2.0	5, 6
10.	Provide new diaphragm nailing at existing plywood panel	2.5	7, 8

Item	Recommended Remediation	Priority	Drawing Number
	edges (transverse). Provide new plywood sheathing over existing attached with staples. Provide new framing members at new panel edges as required (longitudinal).		
11.	Provide new second layer of plywood sheathing on the backside on the existing wood shear walls with edge nailing to match the existing.	2.0	9
12.	Provide new diaphragm nailing at existing plywood panel edges at critical locations.	2.5	10
13.	Provide new metal strapping at critical locations.	1.5	10
14.	Provide new plywood sheathing over existing attached with staples. Provide new framing members at new panel edges as required.	1.1	12
15.	Provide new concrete shear walls at perimeter frames and interior “ring” locations. Provide new dowels into the existing concrete columns. Provide new concrete footings.	1.1	11
16.	Provide new concrete collector beams at the perimeter lines. Provide new concrete collector beams at interior “ring”.	1.3	12
17.	Provide new second layer of plywood sheathing on the backside on the existing wood shear walls with edge nailing to match the existing.	2.0	13
18.	Provide new diaphragm nailing at existing plywood panel edges (longitudinal). Provide new plywood shear walls at existing partition wall locations. Provide new concrete footings (transverse).	2.5	14, 15, 16, 17
19.	Strengthen concrete wall anchorage connection and plywood roof diaphragm with new wall anchors, diaphragm ties, and diaphragm nailing as required.	1.1	16, 17
20.	Provide new double angle members and connections with anchorage into lateral load resisting members to create a new horizontal diaphragm truss.	1.1	18
21.	Strengthen concrete wall anchorage connection with new wall anchors and diaphragm ties.	1.1	18
22.	Provide new flexible conduit connections across seismic separation joints.	1.9	N.A.
23.	Provide new plywood sheathing over existing attached with staples. Provide new framing members at new panel edges as required.	1.5	20
24.	Provide new metal strapping, blocking, and/or collector beam at critical locations.	1.5	19
25.	Strengthen concrete wall anchorage connection and plywood roof diaphragm with new wall anchors, diaphragm ties, and diaphragm nailing as required.	1.1	20

Item	Recommended Remediation	Priority	Drawing Number
26.	Provide new steel “strong back” columns from the floor to roof diaphragm with anchors into existing concrete walls.	1.5	20

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 at this school campus be performed 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

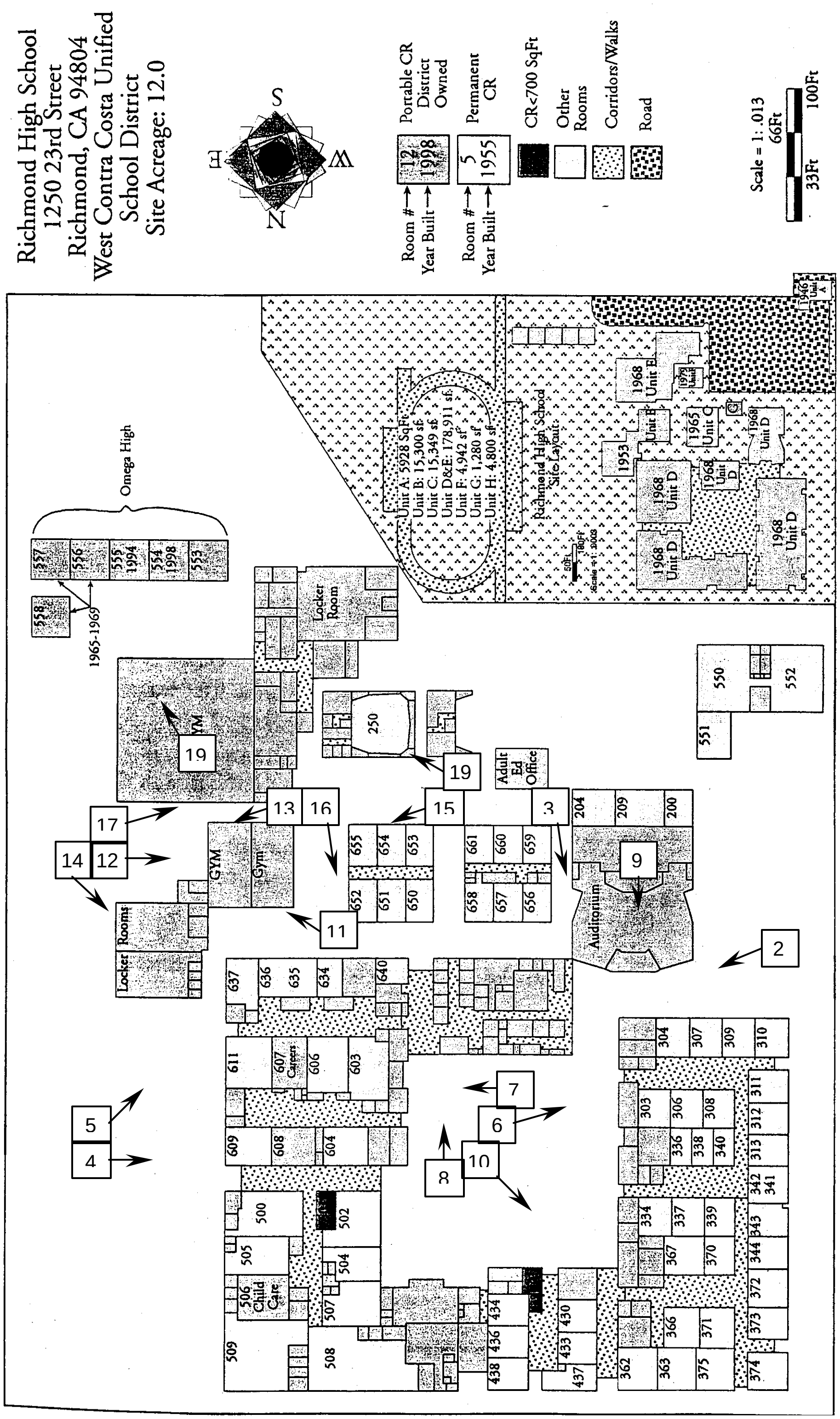


Figure 1: School Layout Plan

Figure 2: Campus Entrance

Figure 3: Theatre Building (Unit D), exterior east wall

Figure 4: Classroom Building (Unit D), exterior east wall

Figure 5: Classroom Building (Unit D), exterior east wall

Figure 6: Common Area (Unit D), interior southwest corner

Figure 7: Common Area (Unit D), interior southeast corner

Figure 8: Common Area (Unit D), interior at stage

Figure 9: Theatre Building (Unit D), interior of auditorium

Figure 10: Common Area (Unit D), interior facing clerestory windows

Figure 11: Old Gymnasium Building (Unit B), exterior northwest corner

Figure 12: Old Gymnasium Building (Unit B), exterior east wall

Figure 13: Old Gymnasium Building (Unit B), exterior south wall

Figure 14: Old Gymnasium Building (Unit B), exterior southeast corner of locker rooms

Figure 15: Classroom Building (Unit C), exterior south wall

Figure 16: Classroom Building (Unit C), exterior east wall

Figure 17: Gymnasium Building (Unit E), exterior north wall

Figure 18: Gymnasium Building (Unit E), interior southeast corner

Figure 19: Gymnasium Building (Unit E), exterior south wall

Figure 20: Gymnasium Building (Unit E), exterior southwest corner

Figure 21: Music Building (Unit F), exterior northwest corner

Figure 22: Shop Building (Unit A), exterior north and west walls

Figure 23: Shop Building (Unit A), exterior north wall

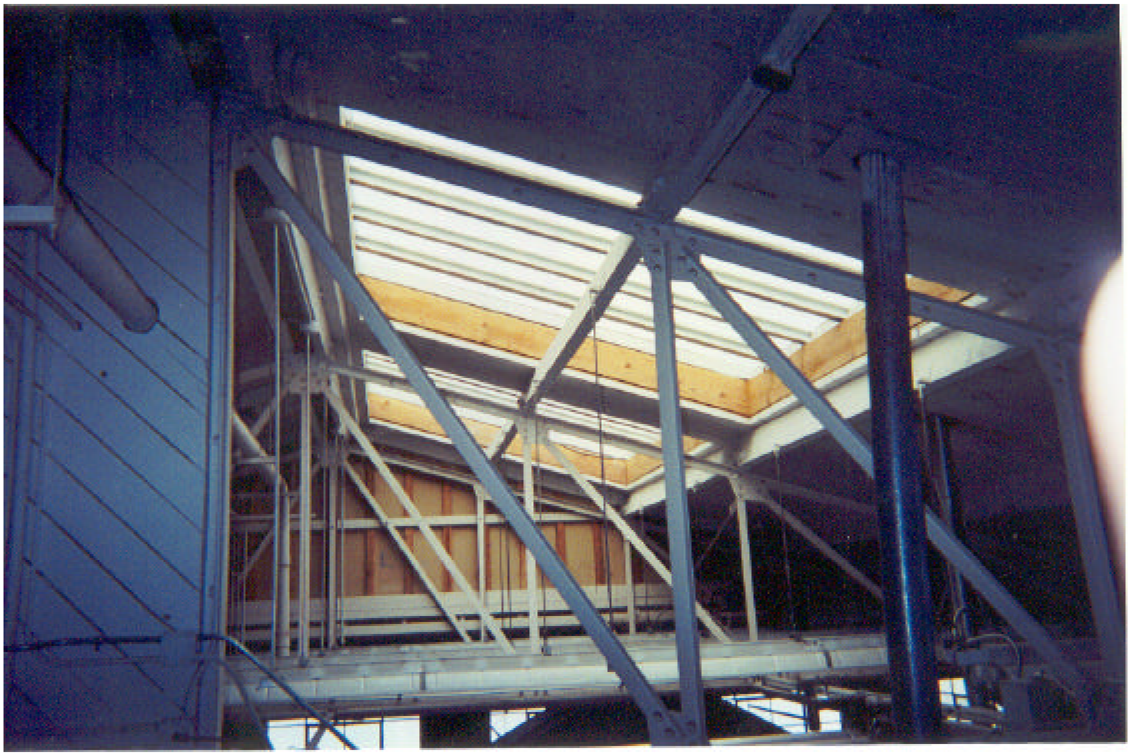
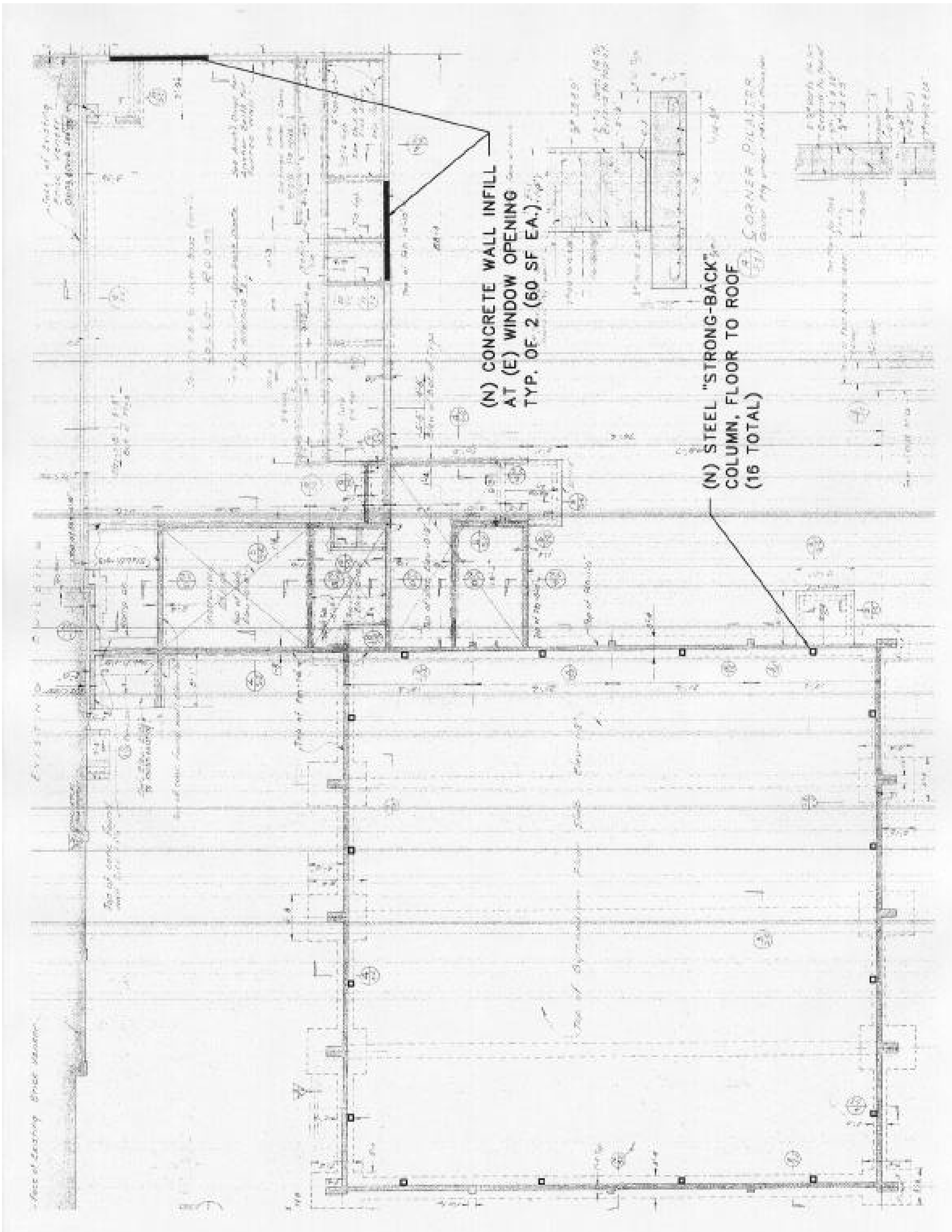


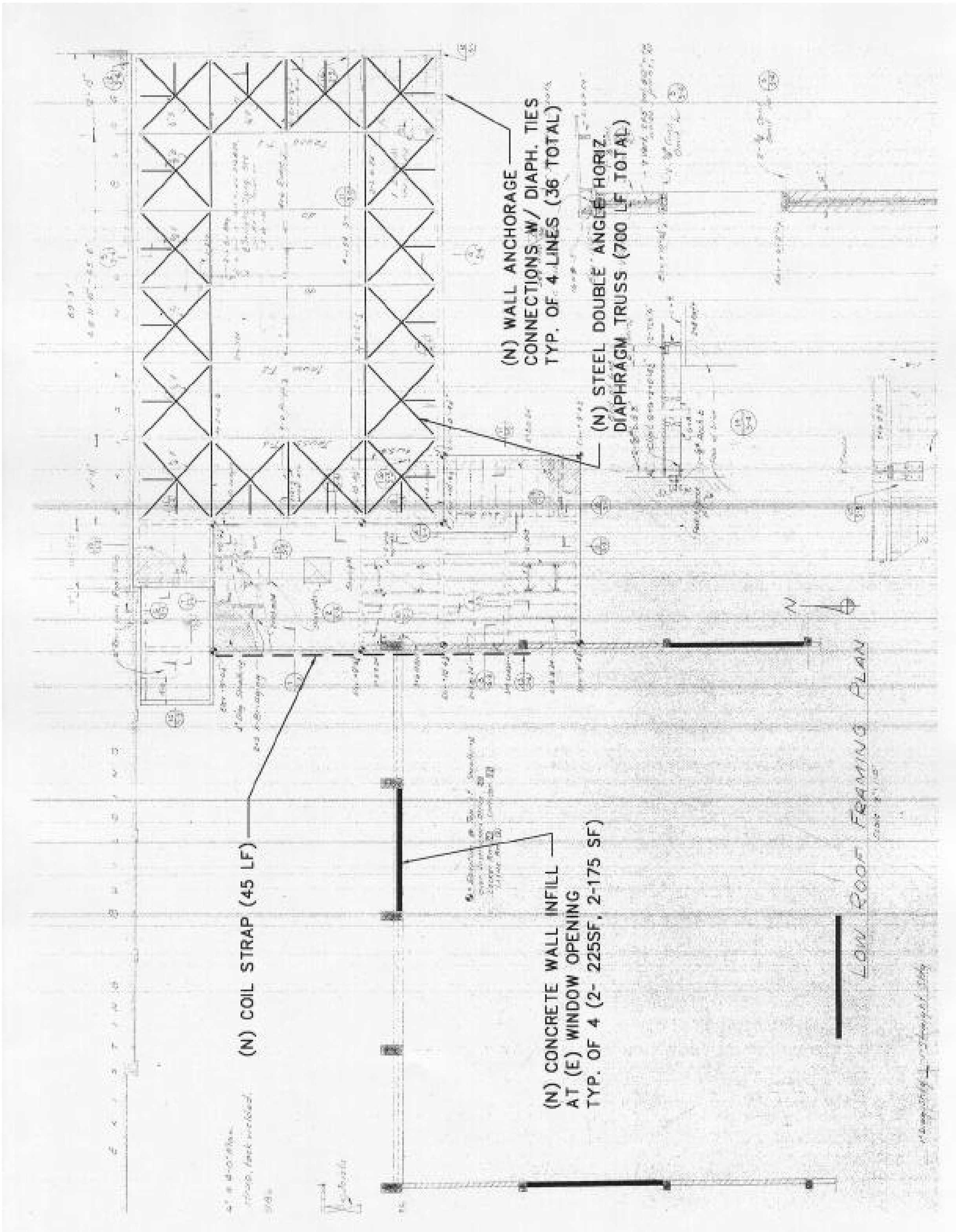
Figure 24: Shop Building (Unit A), interior at roof framing

Figure 25: General (Unit D), conduit crossing seismic joint

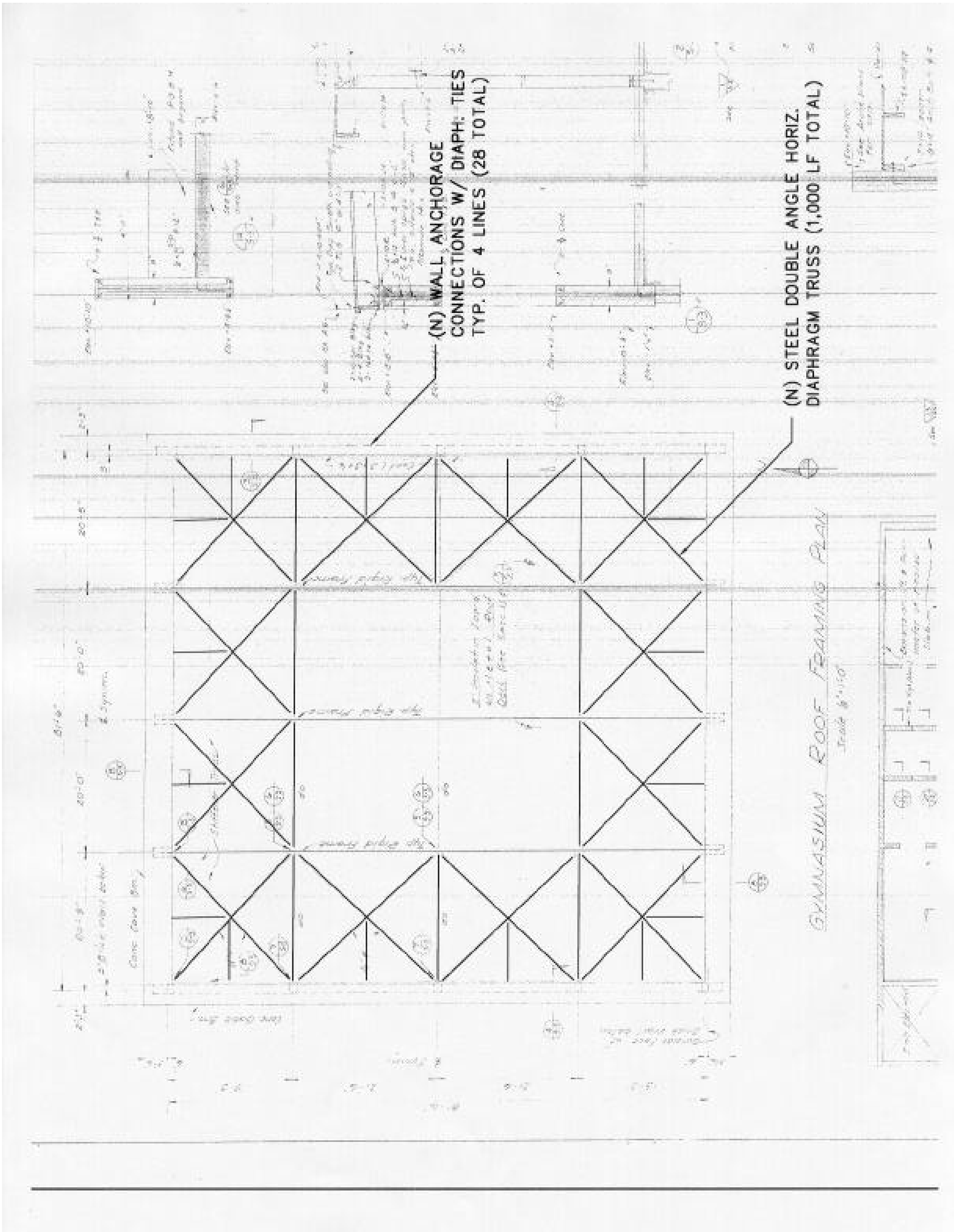
Appendix B: Drawings



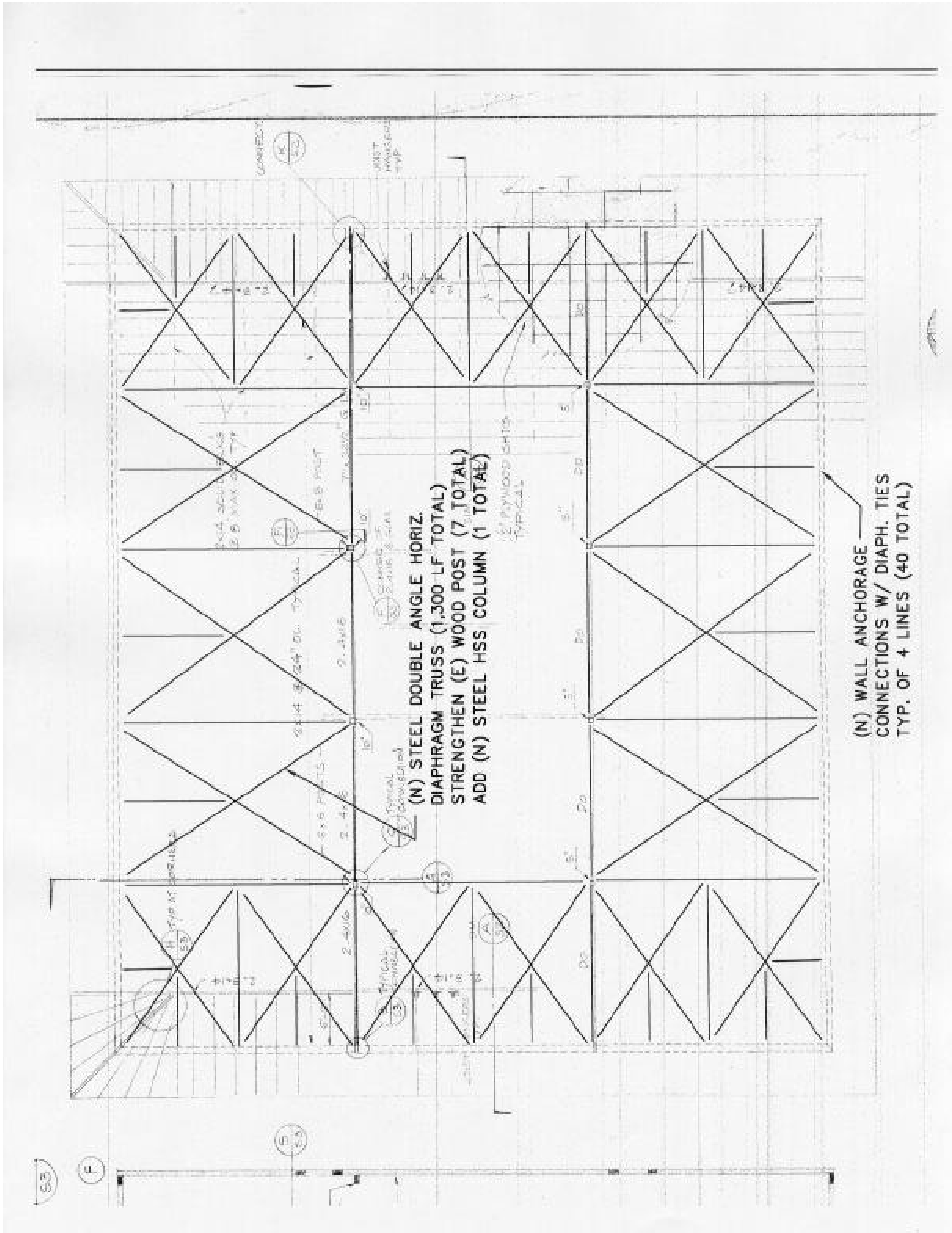
Drawing 1: Old Gymnasium Building (Unit B), Foundation Plan



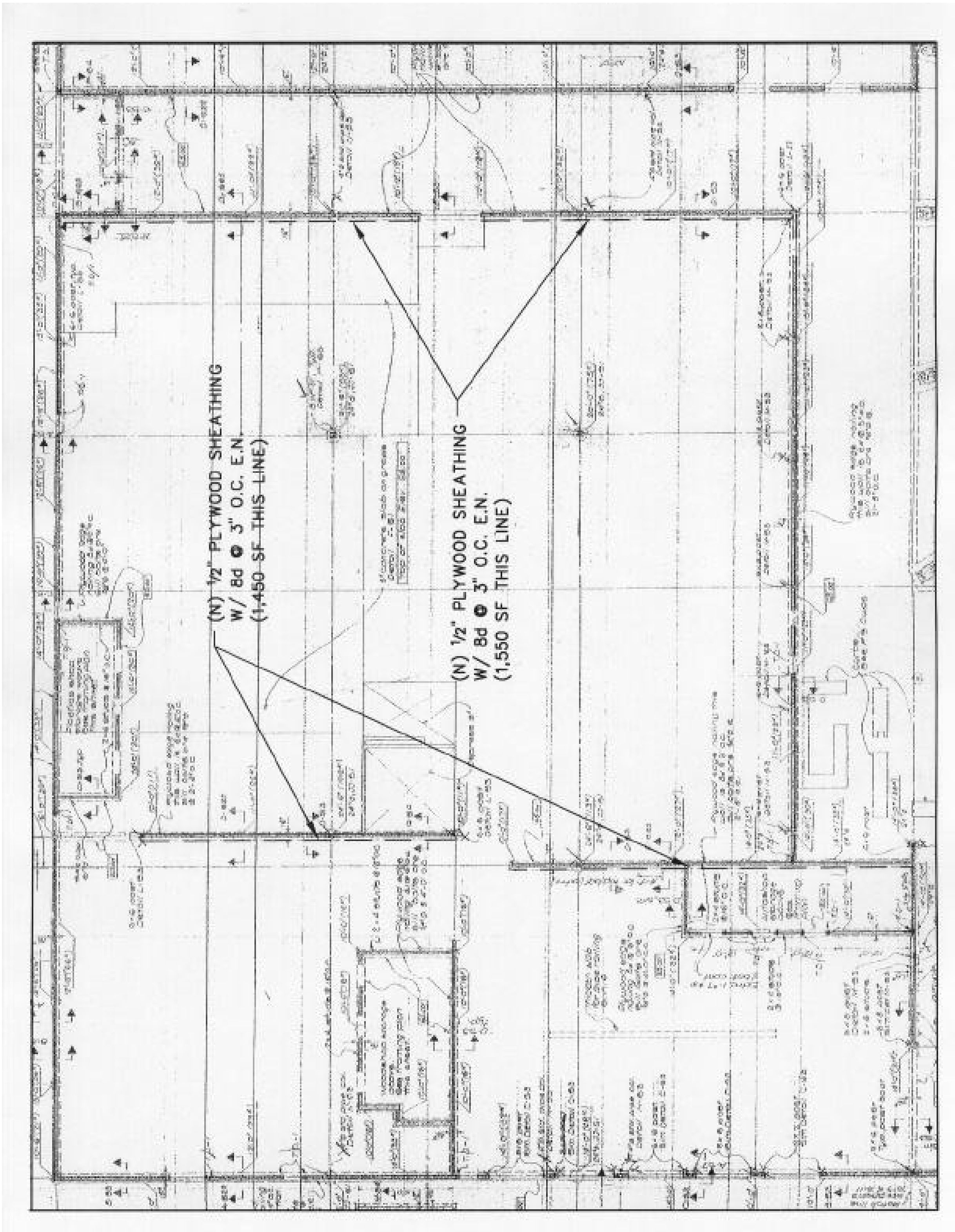
Drawing 2: Old Gymnasium Building (Unit B), Low Roof Framing Plan



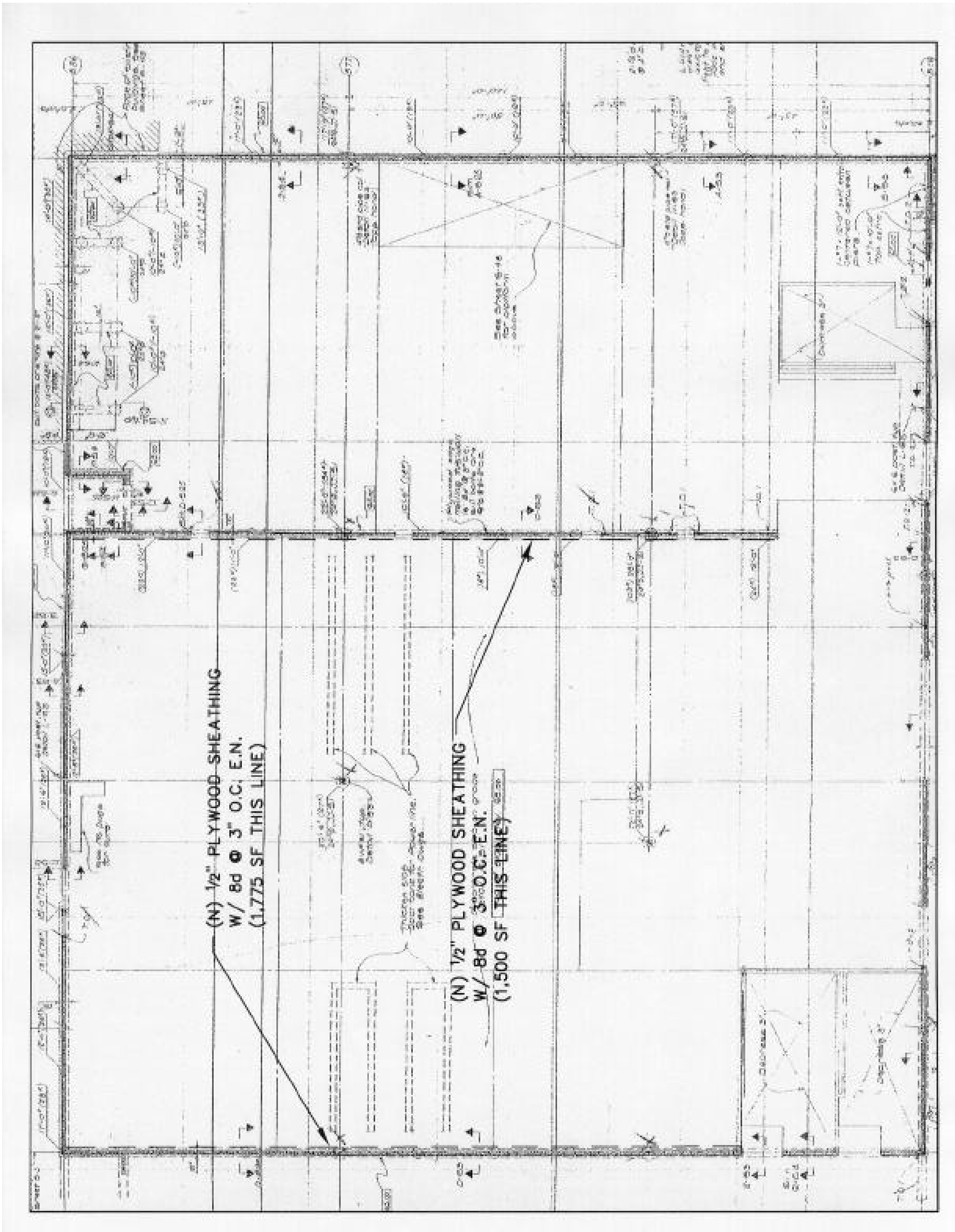
Drawing 3: Old Gymnasium Building (Unit B), High Roof Framing Plan



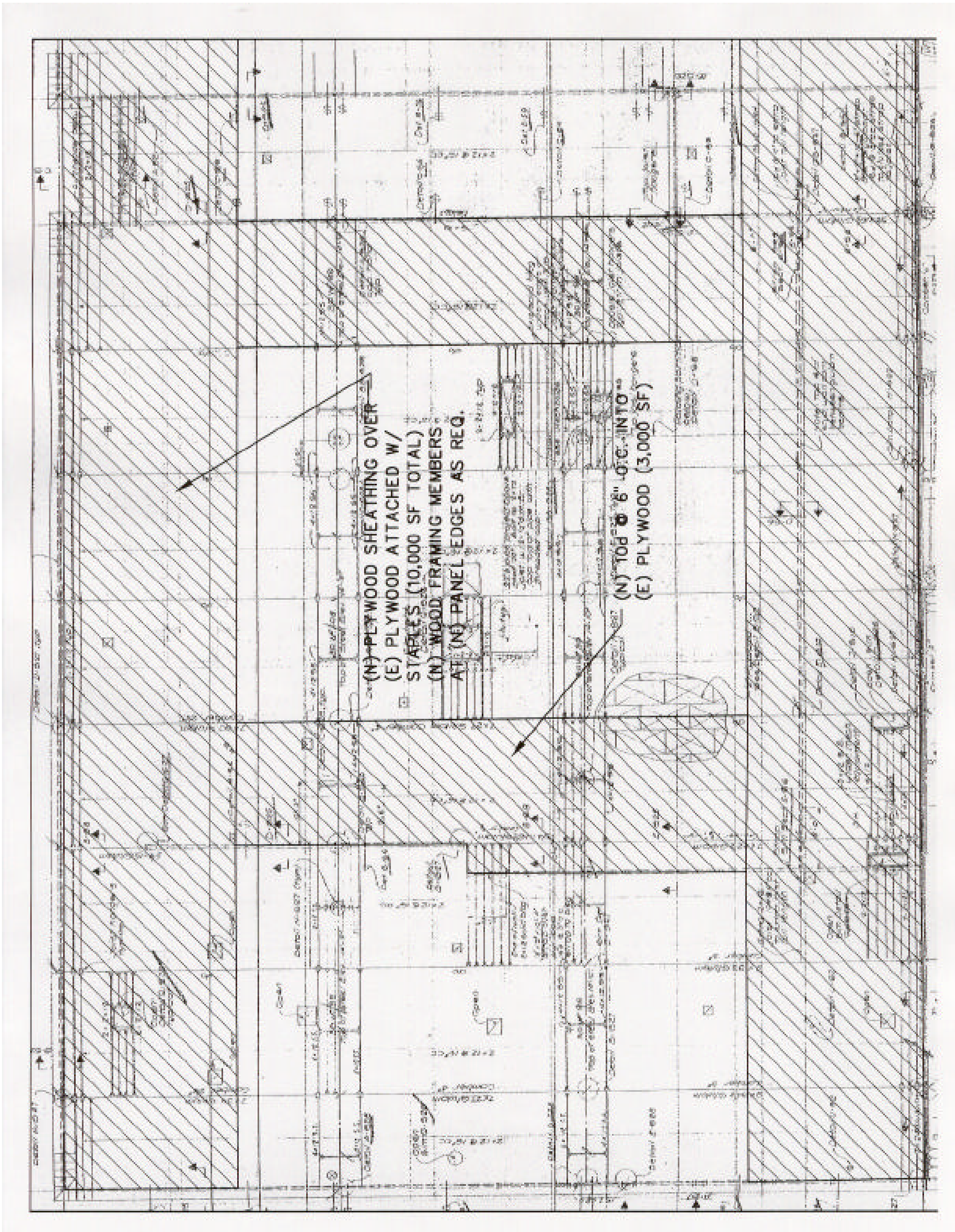
Drawing 4: Classroom Building (Unit C), Roof Framing Plan



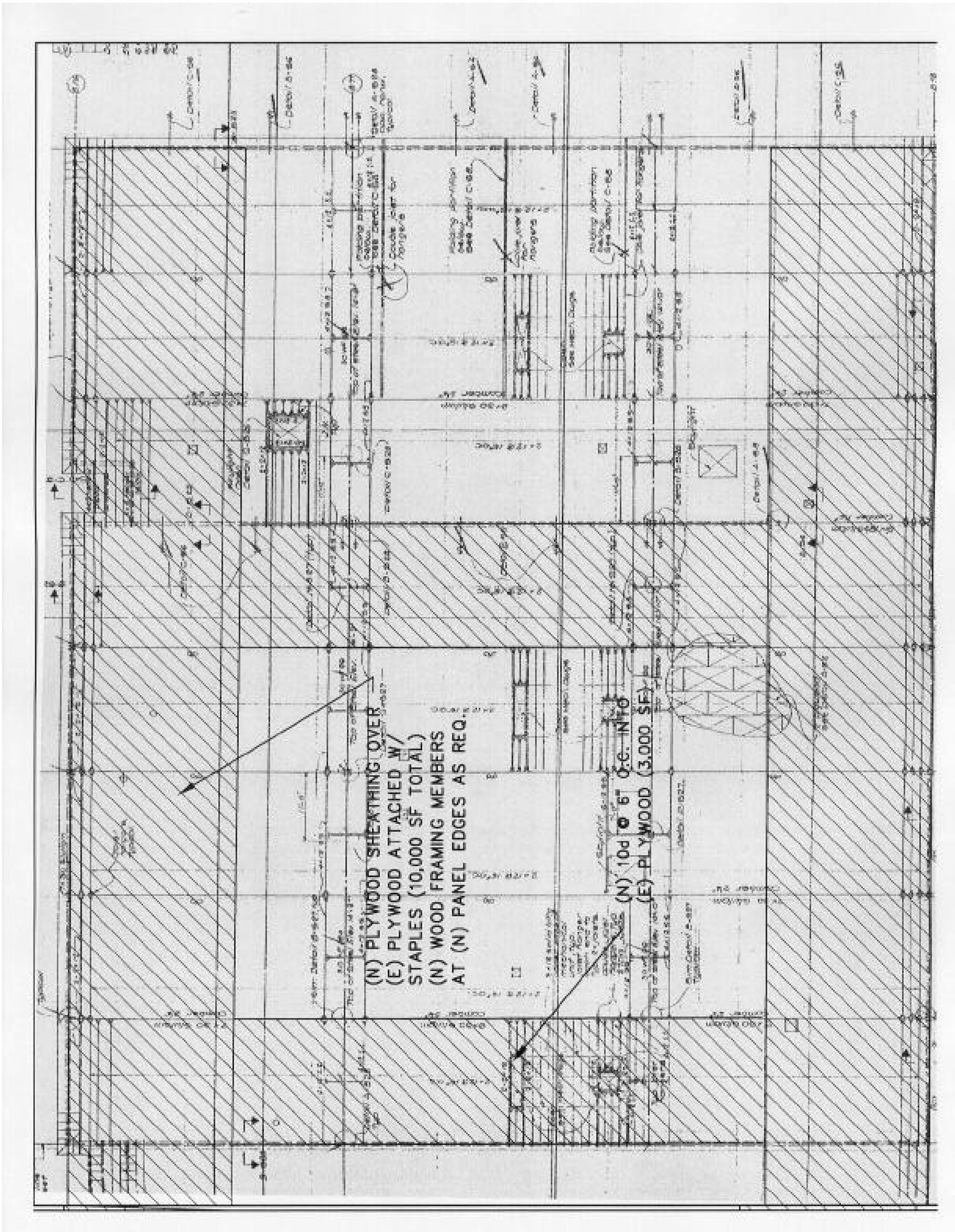
Drawing 5: Classroom Building 100 (Unit D), Foundation Plan



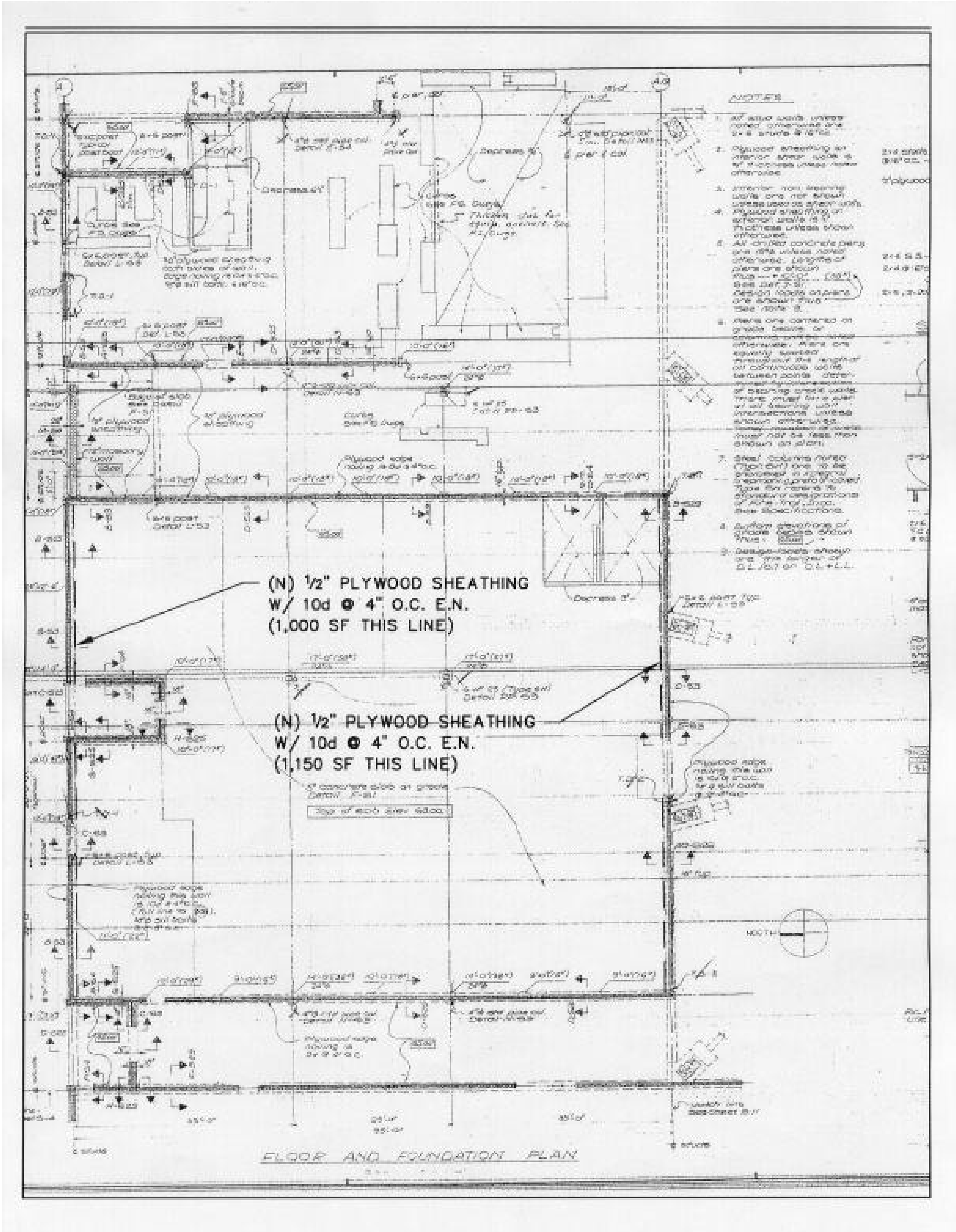
Drawing 6: Classroom Building 200 (Unit D), Foundation Plan



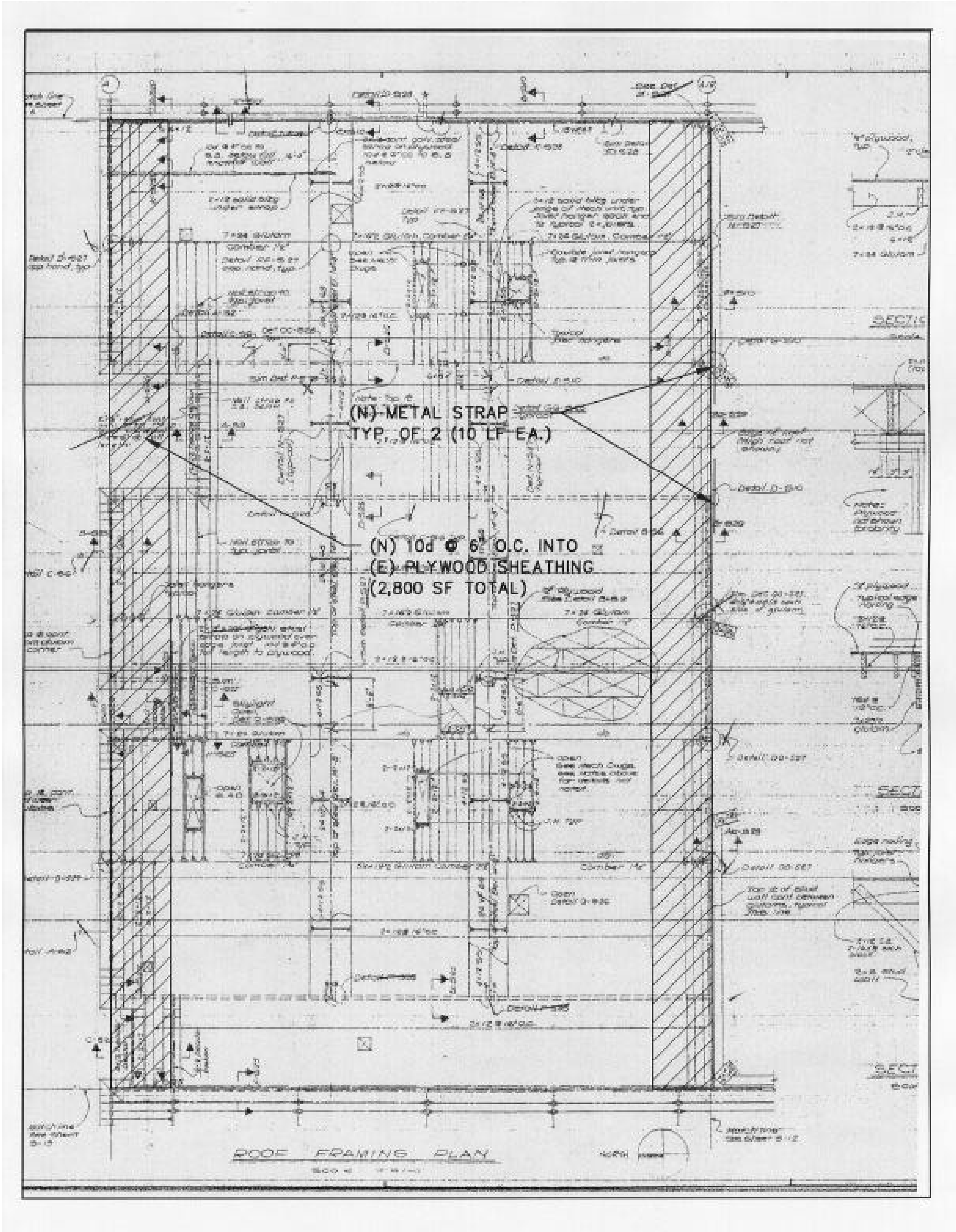
Drawing 7: Classroom Building 100 (Unit D), Roof Framing Plan



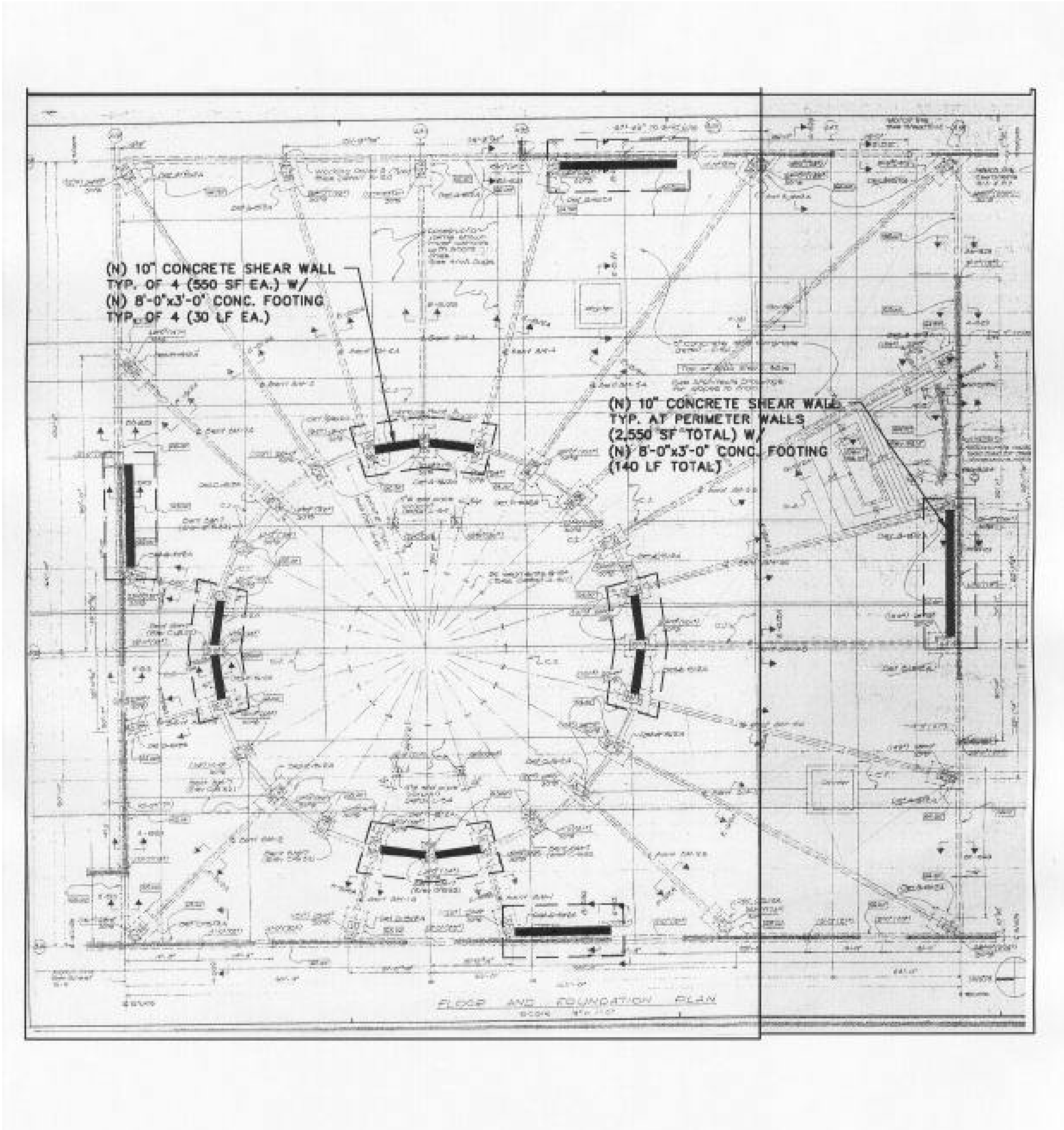
Drawing 8: Classroom Building 200 (Unit D), Roof Framing Plan



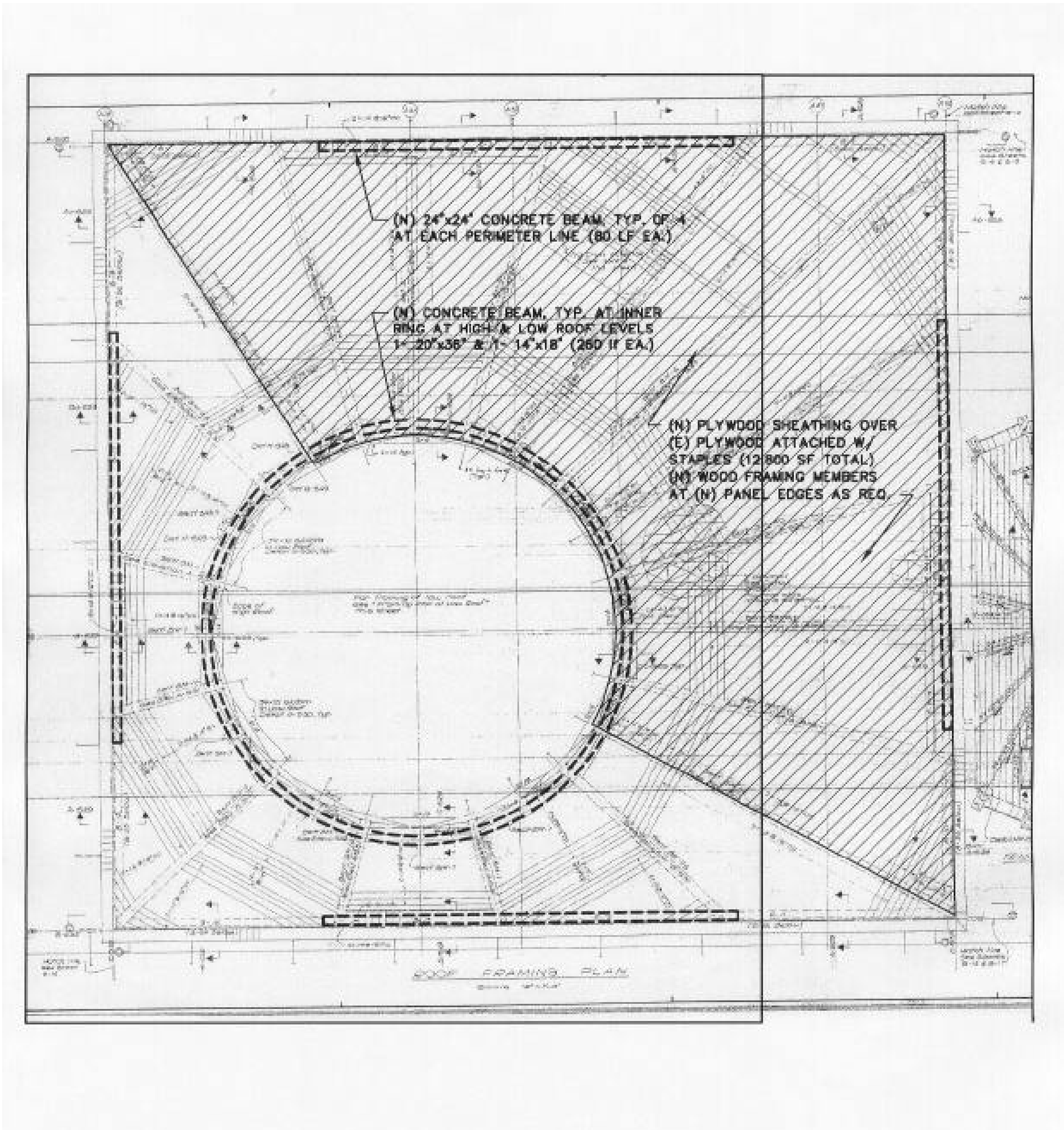
Drawing 9: Classroom Building 300 (Unit D), Foundation Plan



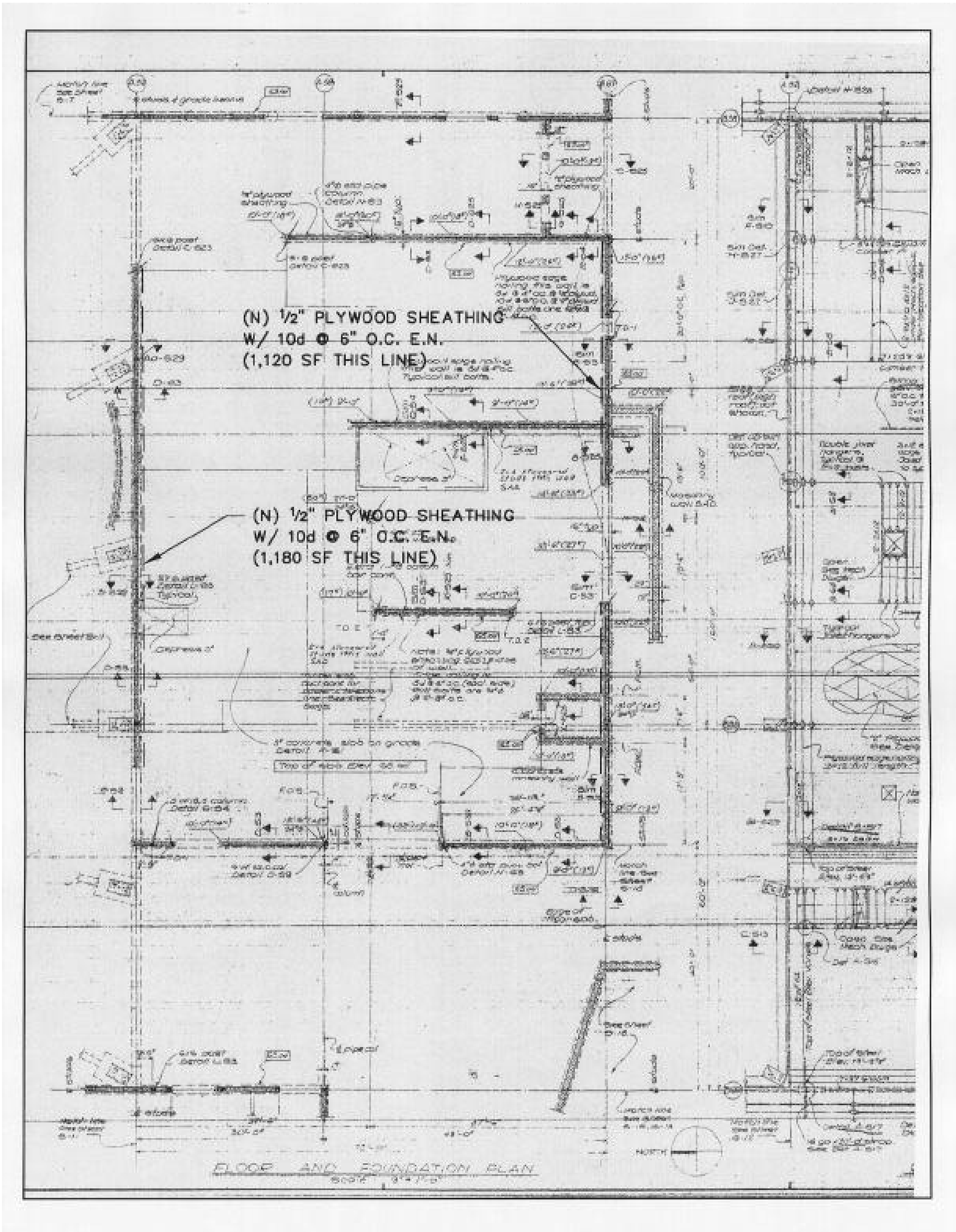
Drawing 10: Classroom Building 300 (Unit D), Roof Framing Plan



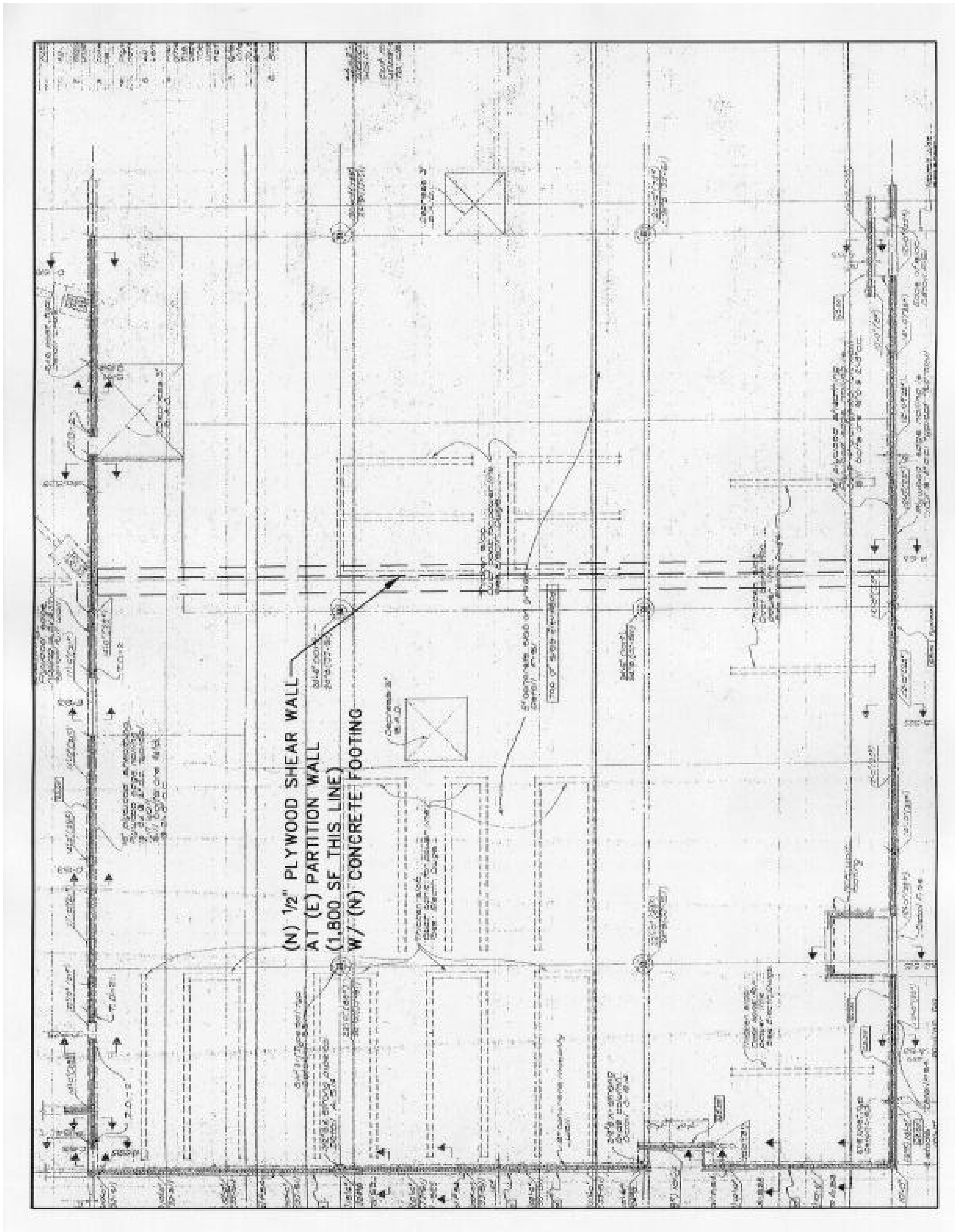
Drawing 11: Classroom Building 400 (Unit D), Foundation Plan



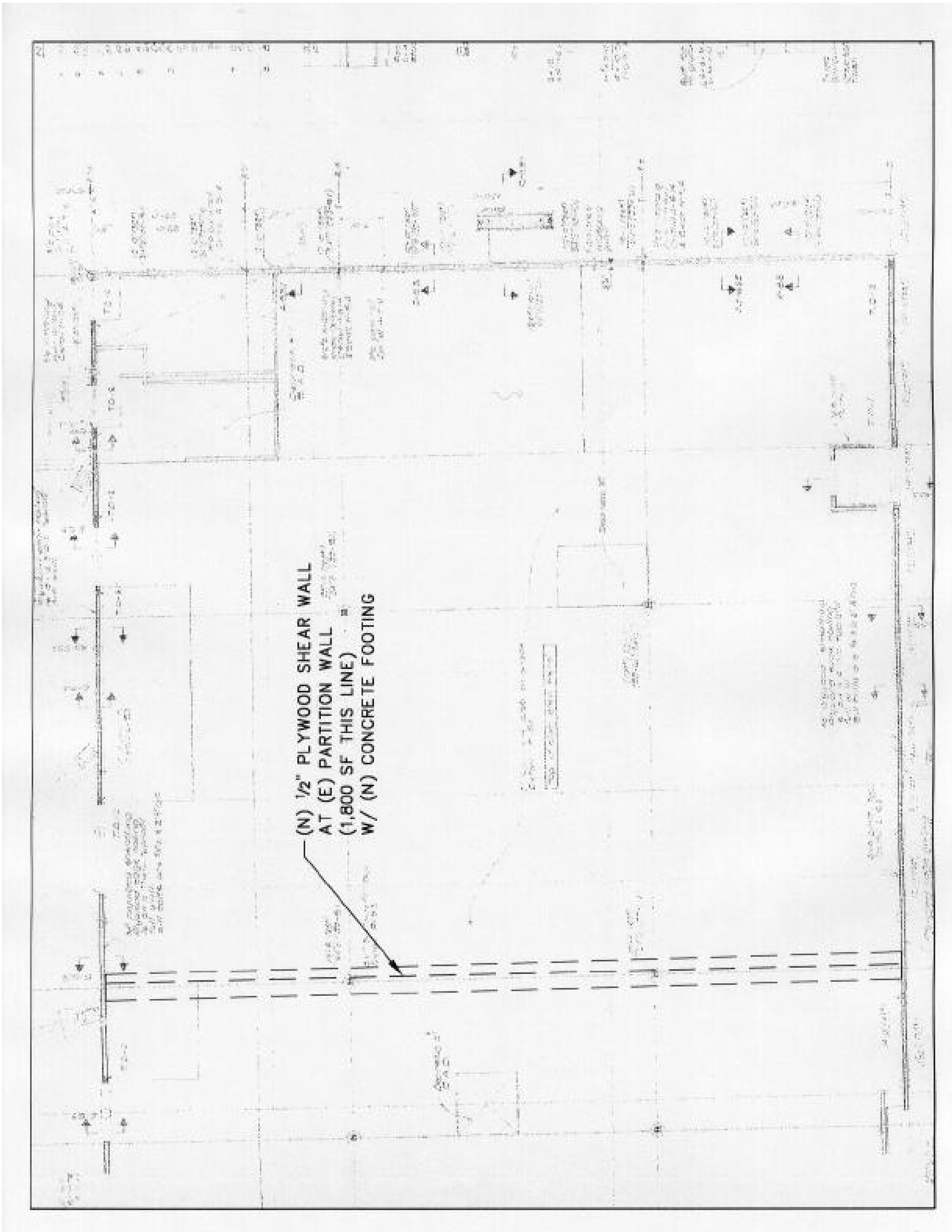
Drawing 12: Classroom Building 400 (Unit D), Roof Framing Plan



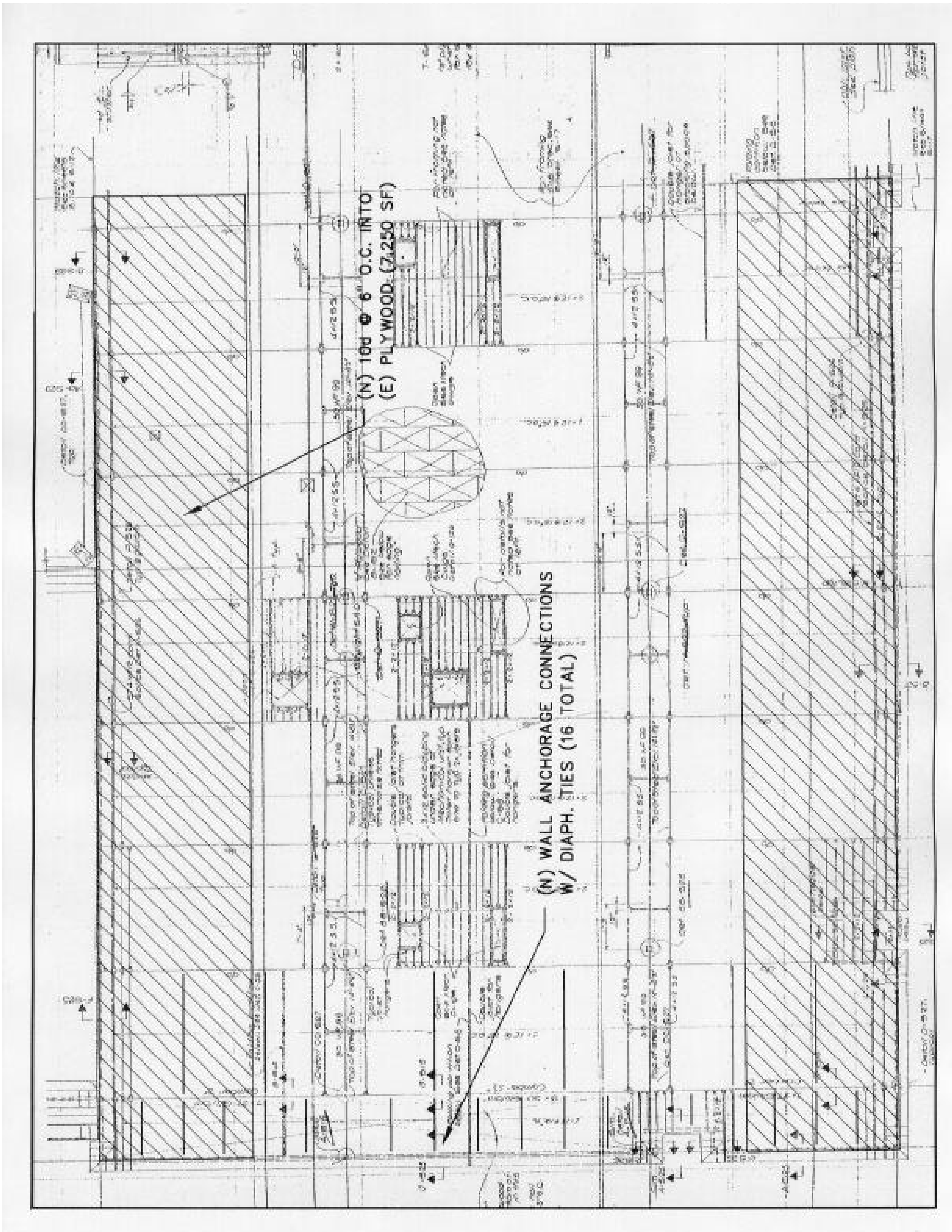
Drawing 13: Classroom Building 500 (Unit D), Foundation Plan



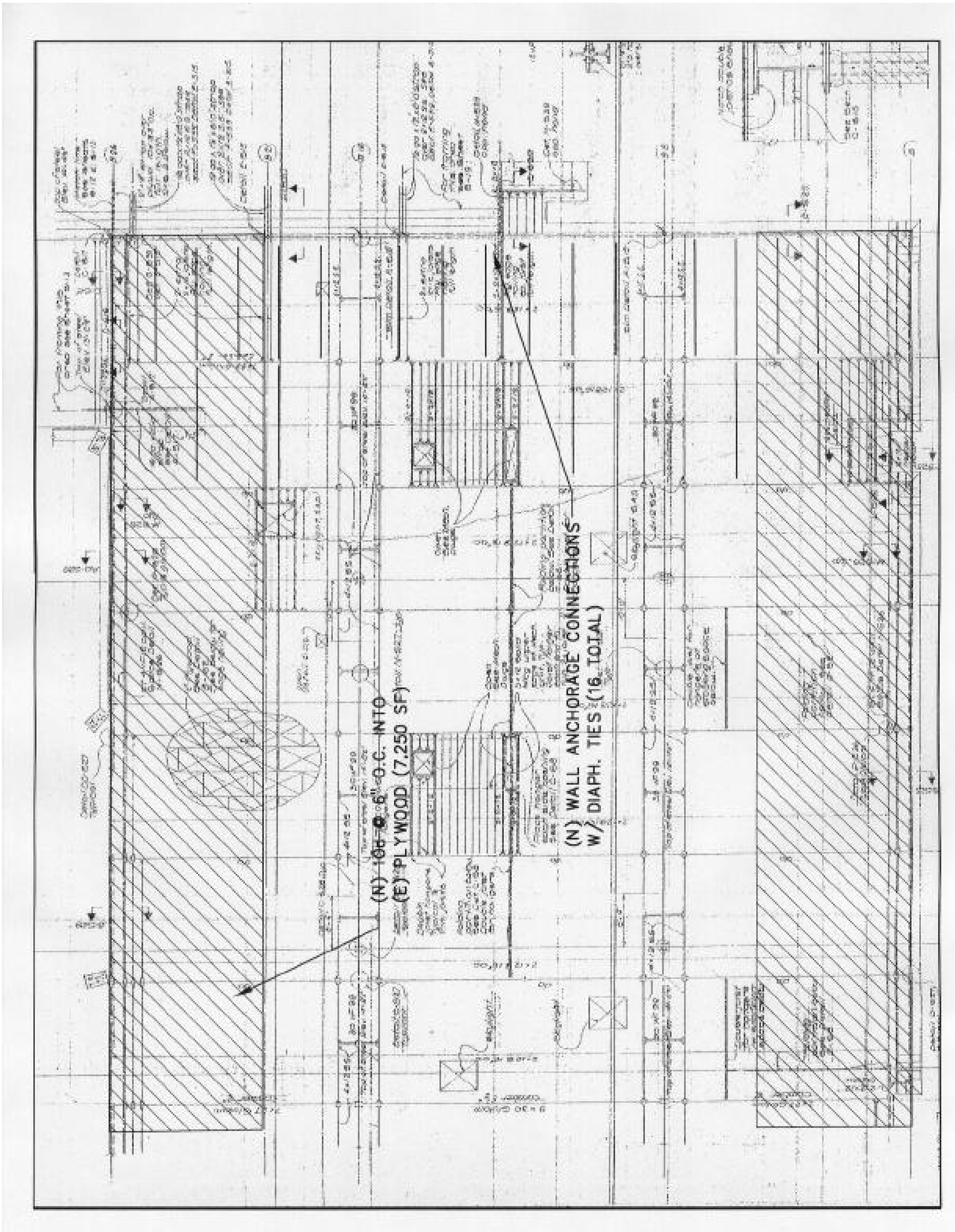
Drawing 14: Classroom Building 600 (Unit D), Foundation Plan



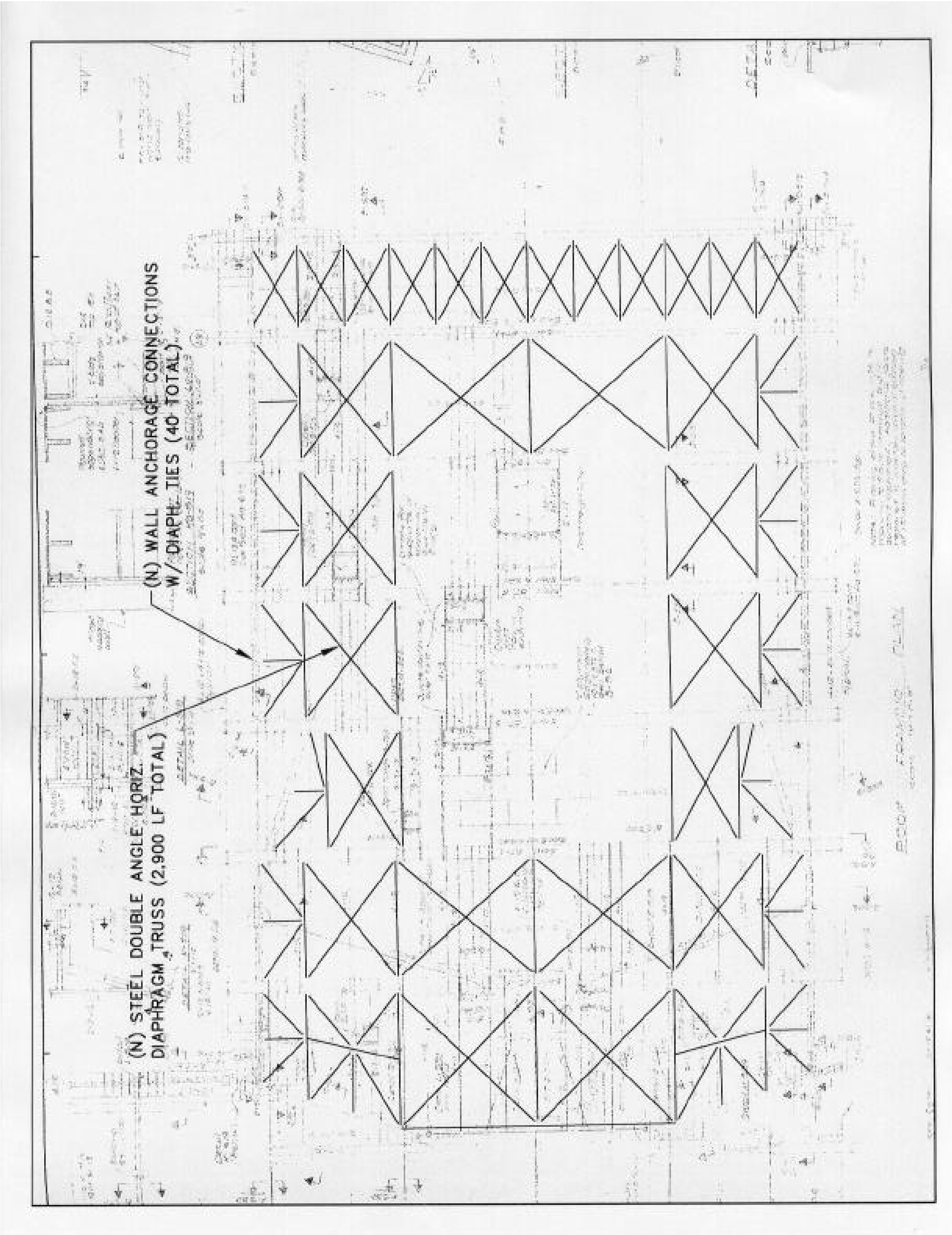
Drawing 15: Classroom Building 700 (Unit D), Foundation Plan



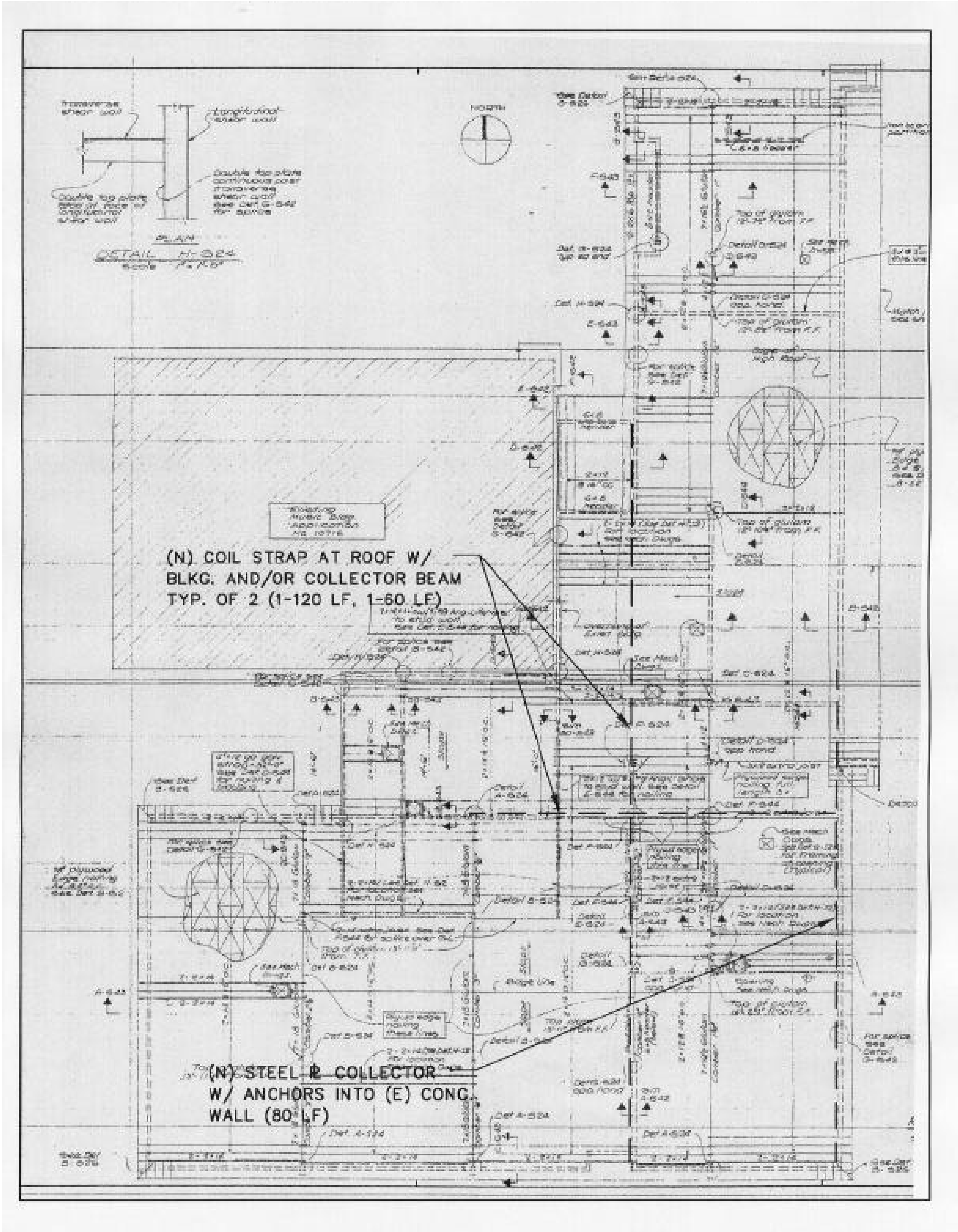
Drawing 16: Classroom Building 600 (Unit D), Roof Framing Plan



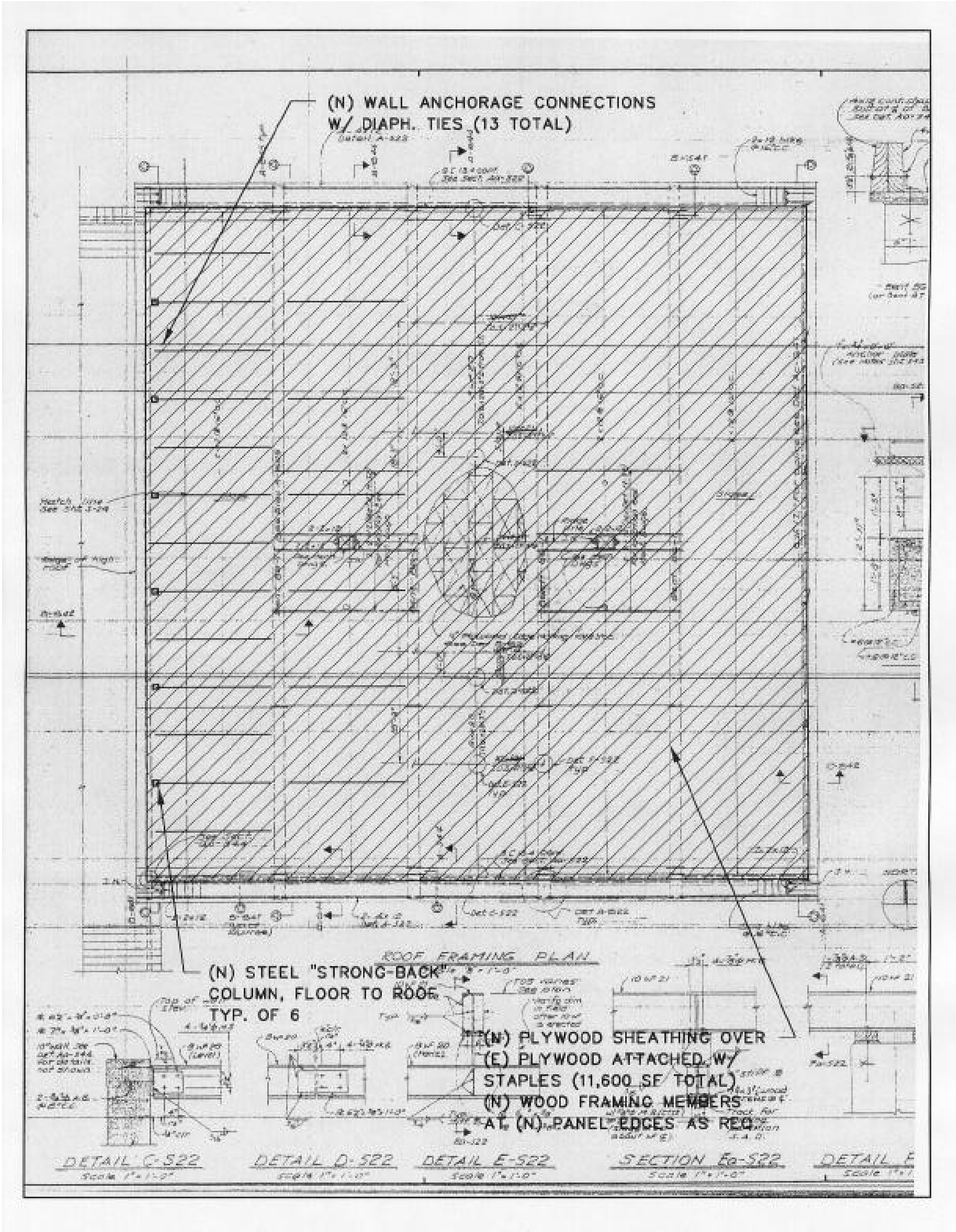
Drawing 17: Classroom Building 700 (Unit D), Roof Framing Plan



Drawing 18: Classroom Building 800 (Unit D), Roof Framing Plan



Drawing 19: Gymnasium Building (Unit D), Low Roof Framing Plan



Drawing 20: Gymnasium Building (Unit D), High Roof Framing Plan