STRUCTURAL EVALUATION OF

DE ANZA 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

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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 De Anza 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 that could pose life safety hazards.

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

10.2 Description of School

The school was built in 1954. The original buildings are steel-framed structures with some concrete tilt-up walls (classroom buildings, administration and library building, auto shop building, cafeteria, and boiler house), a concrete locker rooms and gymnasium with steel framing at the barrrel vaulted gymnasium, and a wood-framed electrical service enclosure. There are nine main buildings (permanent structures), including the electrical service enclosure and boiler house, fifteen portable buildings, and numerous steel- and wood- framed covered walkway structures (see figure 1). There are two 1965 portables, nine 1966 portables, and four 1994 portables. The total square footage of the permanent structures is about 160,482 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 campus is located at a distance of about 3.4 kilometers from the Hayward fault. The table below shows the occupancy and structural system information that is used to determine the seismic design coefficient. 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.

Building	Occupancy	Importance	Structural System (longitudinal/transverse)	R_{long} / R_{trans}	V_{long} / V_{trans}
Classrooms	E-1	1.15	Concrete Bearing Shear Walls	4.5 / 4.5	0.383W
Admin. and Library	E-1	1.15	Steel Ordinary Moment Frame / Concrete Bearing Shear Wall	4.5 / 4.5	0.383W

Auto Shop	E-1	1.15	Steel Ordinary Braced Frame / Concrete Bearing Shear Wall and Diagonally-sheathed Shear Wall with some intermediate Ordinary Moment Frames	5.6 / 4.5	0.308W / 0.383W
Cafeteria	A-2.1	1.15	Concrete Shear Wall and Steel Ordinary Braced Frames / Concrete Bearing Shear Wall	5.5 / 4.5	0.314W / 0.383W
Gymna-sium and Locker Rooms	A-2.1	1.15	Concrete Bearing Shear Walls with concrete buttresses at the gymnasium in the transverse direction	4.5 / 4.5	0.383W
Boiler House		1.0	Concrete Bearing Shear Wall	4.5 / 4.5	0.333W
Electrical Service Enclosure		1.0	Diagonally-sheathed wood shear wall	4.5 / 4.5	0.333W

The seismic design coefficient in the 2001 CBC is calculated as:

$$V = \frac{2.5CaIW}{R} \quad \text{where Ca = 0.44 x 1.36 = 0.60}$$

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. De Anza High School; John Carl Warnecke, Architect, and Hall, Pregnoff, and Matheu, Structural Engineers; sheets 1-41, S1-S24; February 15, 1954.
2. "Measure D" - WCCUSD MIddle and High Schools– UBC revised parameters by Jensen- Van Lienden Associates, Inc., Berkeley, California.

10.5 Site Visit

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

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

The permanent buildings on the campus appear to be of similar construction and architectural style, with the exception of the gymnasium and locker rooms building. In general, the buildings are one story tall and have large numbers of exterior windows on the longitudinal sides and solid wall with occasional door openings at the transverse end walls. Most of the roof structure is framed with open-web joists that support wood roof diaphragms (see figure 3). The classroom, administration, and library areas typically have suspended ceilings, whereas the hallways and the cafeteria and gymnasium have acoustical tile ceilings.

There are six wings of the classroom building that are almost identical in construction. These areas are connected by a wide hallway area that houses the student lockers. At the ends of each of these wings, they are also connected to each other by exterior covered walkways, creating the effect of one contiguous roof structure with very large diaphragm openings at the courtyards between the classroom wings (see figure 4). The roof structure at the exterior covered walkways is exposed open-web joists that do not appear sufficient to restrain the individual classroom wings from moving somewhat independently of each other during an earthquake. This is a life safety hazard because the joists may be damaged and collapse during an earthquake. At the corner pilasters of all of the classroom wings, cracking was observed at the connection where the roof joist frames into the wall (see figure 5). This is a serious threat to life safety as the truss may lose gravity support if additional cracking occurs. Some vertical shrinkage cracks in the end shear walls were observed as well. The administration and library building is similar to the classroom buildings except that the ends of the building aren’t connected to the classroom building by exterior covered walkways. Also, there is evidence of some old fire damage to the roof of the classroom building near room 151.

The shop building has a multi-level roof profile. The outer sections of the roof are higher than the low area that runs longitudinally down the center of the building over the corridor (see figure 8). At one end of the building, the transverse shear wall is concrete and at the other end there is a wood-sheathed shear wall. There are visible cracks in the concrete end wall and pilasters and cracks in the stucco at the wood wall (see figures 9 and 10). Their locations at the change in roof height indicate that there may be some differential movement occurring between the high and low roof levels.

The cafeteria is a long, free-standing, rectangular building that is not connected to the other buildings on the campus. Unlike the buildings discussed above, the cafeteria building has a combination of braced frames and concrete shear walls in the longitudinal direction along the same gridlines. The cafeteria has a high roof with lower ceilings in the kitchen area and storage mezzanines above. The structural steel frames and roof diaphragm bracing are exposed to view in the cafeteria area and appear to be in good condition (see figures 13 through 15).

The gymnasium has a large barrel-vaulted open space with a wood roof and steel framing. The long skylight openings in the gymnasium roof have been infilled with straight sheathing over their entire length (see figure 18). The acoustical tile on the ceiling of the gymnasium shows evidence of water damage, primarily along the south side but also at the north side as well (see figure 19). The locker rooms have concrete shear walls and concrete roof diaphragms (see figure 20). There are some minor vertical shrinkage cracks and diagonal cracks near the corners of wall openings in the locker room and gymnasium concrete walls (see figure 22). Some of the skylight openings in the locker room areas have been infilled with plywood sheathing.

There are covered walkways attached to the buildings that interconnect all of the main buildings, as well as some freestanding covered walkways as well (see figures 21 and 23). The attached walkways are contiguous with the building roof framing at the gymnasium, classroom, and administration and library buildings. There are seismic separations between the covered walkway and the shops building and cafeteria. At the locker rooms, the covered walkway relies on a ledger connection for gravity support (see figure 24). The original construction drawings call for the covered walkway to be supported by a steel beam and two columns at each location where the existing ledger connections exist. This represents a life safety hazard because wood ledgers may fail due to cross-grain tension and the a section of the covered walkway might collapse.

10.6 Review of Existing Drawings

The original construction documents include multiple additive alternates and additional phases of work to be performed. With the exception of the construction of the fifth and sixth wings of the classroom building, none of the construction after Phase 1 appears to have been built. In particular, there are significant expansions of the auto shop and cafeteria buildings shown on the drawings that were never built.

The classroom buildings are all of almost identical construction. They have 3/8” plywood roof sheathing nailed with 5d nails at 4” o.c. over 2x6 T&G straight sheathing. The T&G sheathing spans 8 ft between open-web steel joists. These trusses span in the transverse direction between 6” wide-flange steel columns along the outside of the building and 8” thick concrete tilt-up wall panels along the building centerline. Where the interior trusses connect to the girder and wall, the bottom chord of the truss is welded to the support, causing the joist to have flexural stiffness at the support, particularly at the interior wall. This also will subject the bottom chord of the joist to out-of-plane wall seismic loading, which may cause buckling of the bottom chord of the joist. The steel columns rest on a 14” wide strip footing. As is typical throughout the campus, the slab-on-grade is 5” thick. There are also 8” thick tilt-up walls at the ends of the classroom wings and at the central corridor in the transverse direction. The tilt-up walls sit on a layer in a pocket between thickened slab sections, but do not have a direct shear connection to the foundation except at the pilasters. This represents a life safety hazard because the wall may slide during an earthquake. The pilasters at the ends of the wall panels rest on typical drilled piers that are 28” in diameter with a 48” diameter bell at the base. To the outside of the enclosed perimeter of the building, there is a 12 ft wide covered walkway that is framed with open-web joists spanning between steel columns and open-web truss girders. These open-web joists have small bars as a bottom chord member and do not have any bridging to provide lateral stability (see figure 4).

The classroom building wings are connected to each other by covered walkways at each end and corridor segments near the middle of the building that are framed contiguously with the main roof. These connecting walkways and corridors form courtyard areas that open to the sky (see figure 6). There is also a connecting corridor to the administration and library building (see figure 3). The classroom buildings could be viewed as being either three buildings that are linked together (i.e. no seismic joints) or as one large building that has significant diaphragm openings. The framing members at the exterior of the courtyard opening are open-web joists that will not perform well as collector elements (see figure 4). This is a life safety hazard because those joists may buckle and collapse during an earthquake. At the central corridor that connects the wings of the classroom building, there are some additional concrete shear walls that align with the exterior longitudinal walls of the classroom building wings. The roof framing does not appear to provide adequate collector capacity to allow these shear walls to carry the seismic load of the roof diaphragm tributary to these lines and may result in failure the collector along these lines. In the transverse direction, there are no continuous collector lines to connect the building wings to each other, which may result in tearing of the roof diaphragm and partial collapse of structure.

The administration and library building has a 2x6 T&G diagonally sheathed roof diaphragm over open web joists. The joists span 16 ft. between 16WF50 girders that in turn span up to 36 ft. in the transverse direction between steel columns on the building centerline and girders at the building exterior. The 14WF 30 exterior girders span as continuous beams over 6" steel wide flange columns that are spaced every 8 ft., making up a series of moment frames that provide lateral support for the building in the longitudinal direction. The bottoms of these columns are in grouted pockets and the beam-to-column connections at the top do not develop the capacity of either the beams or the columns. Also there is no lateral bracing of the moment frame bottom flanges or the tops of the columns. The lack the adequate strength and proper detailing of the moment frames constitutes a life safety hazard.

In the transverse direction, lateral support is provided by the 8" thick concrete tilt-up shear walls at the ends of the building. These walls have no direct shear connection to the foundation, having only a shear key into a thin layer of grout that was laid down at the time of erection, and have no restraint to keep them from sliding off of the footing in their out-of-plane direction. There is additional roof framing at the end bays to provide out-of-plane support for the concrete walls, but these ties do not continue across the length of the building to provide continuity. The exterior columns typically rest on a 14" wide strip footing and the interior columns are supported on drilled piers similar to the classroom building. At the north side, the ground slopes down away from the building. There is a suspended concrete slab at the first floor in this area and a small basement under the northwest corner of the building with concrete shear walls on all sides (see figure 7).

Near the center of the 221 ft long administration and library building, there is an attached corridor structure that connects the administration and library building to the classroom buildings. There are no continuous collector lines to transfer forces from the corridor to the main portion of the building in either the longitudinal or transverse directions.

The auto shop building has a multi-level roof that has high areas on the two sides and a low central area over the corridor that runs down the middle of the building (see figure 8). The roof has 2x6 straight sheathing that runs in the transverse direction over open-web joists. The open web joists span 24 ft. between 18WF50 beams (16WF40 at the low roof) that span about from 30 ft to 38 ft between 14WF43 columns. The columns are supported on typical 3'-8" square spread footings. There are skylights along the entire length of the auto shop building at one of the high roof areas and occur intermittently in the low roof area. To account for this weakened diaphragm, the building has 3 bays of diagonal L2x2x¼ "X" bracing just below the roof level (see figure 11). These braces help to distribute the diaphragm forces out to the longitudinal braced frames. There are 8 braced frames in total, two on each line on each side of the high roof areas. These chevron ST4WF8.5 braces are not directly connected to the roof diaphragm at the low roof level and lateral forces from the low roof level must be transferred through the bending of the columns up to the collectors at the high roofs (see figure 12). The brace connection into the column near the base does not align with the top of the slab, but rather is eccentric by about 5", causing bending in the column. Furthermore, the braces do not meet current slenderness or compactness requirements for seismic bracing and there is no lateral bracing of the roof beams where the chevron braces intersect them, which reduces the expected ductility of the braced frame system and constitutes a life safety hazard. In the transverse direction, lateral support for the building is provided by an 8" thick concrete tilt-up shear wall at the east end of the building, a diagonally sheathed stud wall at the west end of the building, and moment frames at the interior bays. The concrete shear wall has no direct connection to the foundation and is detailed similar to the administration and library building walls. The different stiffnesses of the lateral force resisting lines in the transverse direction will introduce torsion into the building motions. The moment frames do not have adequate capacity to support their tributary load laterally, but will help to reduce the deflections and forces in the roof diaphragm. The diagonally sheathed shear wall does not have holdowns at some locations.

The cafeteria building is a long, narrow structure with a tall interior open space. The roof diaphragm is made up of 2x6 T&G straight sheathing supported by open-web joists. These open-web joists span about 16 ft between steel bents (27WF94 beams and 14WF68 columns) that span the entire width of the building, 68'-8", in the transverse direction. These steel bents rest on typical 3'-6" square spread footings. In the transverse direction, there are roof trusses made up of L3x2x¼ "X" braces that transfer the diaphragm forces out to the exterior longitudinal wall lines. At these wall lines, there are partial height concrete walls along most of the length with full height concrete walls at each end of the building. There are also 4 bays of "X" braced frames with ST3.5I7.65 braces along each exterior longitudinal wall. A series of 10WF33 collectors connect the braced frames and concrete walls together, but appear to have very weak connections at the columns (see figures 14 and 15). In the transverse direction, the steel bents act as moment frames to help resist the lateral forces, but are too flexible to control the deflections sufficiently. There are also transverse concrete walls at each end of the building and at one interior transverse wall line. The 10" thick concrete walls at the north end of the building were cast in place and the ones at the south end of the building are 8” thick tilt-up walls. The cast walls have dowels into the foundation, whereas the tilt-up walls have no direct shear connection to the foundation and are detailed in a similarly dangerous manner as those in the administration and library building end walls (see figure 16).

The gymnasium building consists of two separate types of construction for the gymnasium area and the locker rooms. The main gymnasium area has a barrel vault roof that is sheathed with 2x6 T&G straight sheathing that is supported by 10WF21 steel beams. These steel beams span 20 ft. between three-hinged arches that are made up of 18WF50 girders (see figure 18). These arches then bear on curved 18” by 26” concrete buttresses at the outside of the building that deliver the arch force into the foundation (see figure 17). There are concrete tie beams in the foundation that resist the outward thrust of the arches in tension. In the transverse direction, the roof diaphragm spans only 20 ft between these arches. Because the roof diaphragm is interrupted by long skylights running the entire length of the building in the longitudinal direction, there are 3 bays of diagonal ST4WF8.5 “X” bracing that deliver the diaphragm forces to the longitudinal walls. Along the exterior longitudinal walls, there are 10” thick concrete walls that run the length of the building with only minor openings. In the transverse direction, there are 10” thick concrete walls at the ends of the gymnasium area and also at an interior location dividing the large and small gymnasium areas (see figures 2 and 18). Where these walls support the roof beams, the beams are embedded into the wall with angles welded onto the side to provide for anchorage in resisting out-of-plane forces. Because of the height of the gymnasium, the transverse walls are very slender at their highest point of about 31 ft., which constitutes a threat to life safety if the walls fail in flexure out-of-plane.

The locker room portion of the gymnasium building has two similar wings and a connecting corridor that runs alongside the north side of the gymnasium. The locker rooms have a 4½” thick concrete roof slab that spans 10 ft. between 12” by 16” beams. Those beams span 20 ft. between 14” by 20” girders that are supported by typical 16” square concrete columns. There are concrete shear walls, typically 8” thick, in both principal directions. The locker room roof has a number of large skylight openings that significantly decrease its shear capacity (see figures 20 and 21). There are hollow clay tile partition walls inside the locker rooms that are doweled into the floor slab.

The boiler house is a small free-standing structure with a diagonally sheathed roof supported by open web joists. Those joists span 16 ft. between 16WF36 beams that span 33 ft. between concrete walls. The floor slab is below the level of the adjacent grade. The south wall of the boiler house has windows along the entire length above a half-height concrete wall. The other three sides of the structure have 8” thick concrete tilt-up wall panels. This three-side box configuration will induce some torsion into the structure’s motion. At the east and west walls, the out-of-plane wall connection to the diaphragm introduces axial force into the open-web joists. Just outside the boiler house, there is an exhaust stack that is supported on a 7’-6” square footing (see figure 25).

The electrical service enclosure is a small wood-framed building (see figure 26). The roof has 1x diagonal sheathing supported by 2x8 joists at 24” o.c. The 2x8 joists span 13 ft. between exterior longitudinal walls. There are conventional diagonally sheathed shear walls along all four sides and also an additional interior transverse wall. There are 14" wide strip footings under all of the exterior and interior walls. There are no holdowns at the ends of the shear walls and there does not appear to be any additional reinforcement of the shear walls around openings. Because of the tall aspect ratio of the shear walls, the lack of holdowns represents a life safety hazard.

There are a number of free-standing covered walkways on the campus (see figures 21 and 23). These typically have 2x6 T&G straight sheathing spanning up to12 ft between 8WF24 steel beams. These steel beams span between 4" diameter cantilevered pipe columns. The base connection of the cantilevered column consists of a small thickened slab edge that has a pipe sleeve embedded in it. The column then is grouted into the pipe sleeve. This footing appears to provide much better column base fixity in the longitudinal direction of the covered walkway than in the transverse direction. The lack of adequate base fixity in this cantilever system constitutes a collapse hazard.

10.7 Basis of Evaluation

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

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

10.8 List of Deficiencies

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

Item	Building Structural Deficiencies
1.	Throughout the campus, the covered walkways are connected at each end to buildings. As the buildings move independently, the walkway may tear away and collapse.
2.	At the classroom buildings, the bottom chords of the open web joists are welded to the walls or columns in some locations, creating unintended moment connections. This also subjects the unbraced bottom chord of the truss to the out-of-plane wall forces from the tilt-up wall
3.	At the classroom buildings, there is no shear connection between the tilt-up walls and the foundation. The walls may slide during an earthquake.
4.	At the classroom buildings, the bolted connection from the tilt-up walls to the roof diaphragm may be insufficient to transfer the diaphragm shears into the wall.
5.	At the classroom buildings, the roof sheathing is 3/8" plywood over 2x6 straight sheathing. The plywood is nailed with 5d@4" o.c. This nailing is non-standard and is not included the UBC diaphragm shear tables. The roof diaphragm is insufficient to span 144' between the existing shear walls.
6.	In the longitudinal direction, the roof diaphragms of the classroom buildings are cantilevered off of the central wall line on both sides. This system lacks redundancy and results in diaphragm overstresses.
7.	The connecting area between the classroom buildings has a lack of sufficient collector elements. This area is only connected to the transverse walls through a truss that has little capacity to transfer axial loads
8.	There are short concrete walls at the core of the classroom building in the longitudinal direction, but the drag elements that deliver forces to them are inadequate to transfer the forces. Even if the collectors and their connections were strengthened, they would still be inadequate to carry the tributary loads.
9.	There is no lateral bracing of the bottom chords of the trusses at the attached covered walkways of the classroom buildings. This is a potentially unstable condition that could result in roof collapse.
10.	The classroom building wings are connected to each other by covered walkways at the ends. This results in seismic axial forces in the trusses, which do not appear to have adequate compression capacity or stability bracing.
11.	At the classroom buildings, there are no collector elements at the corners of the courtyard area openings. This will result in tearing and possible collapse of a truss. There is some existing cracking of the pilasters supporting the trusses at the corners of the courtyard openings.
12.	The out-of-plane anchorage of the classroom building transverse tilt-up walls is insufficient to resist seismic loads. There are no cross-ties between walls.
13.	At the administration and library building, the roof diaphragm is insufficient to span 221' between the existing transverse shear walls. The chords are also overstressed.

14.	The administration and library building has moment frames in the longitudinal direction at the exterior walls. There is a lack of moment frame stiffness and strength. This may lead to excessive deflections, damage to ceilings and walls, and potential collapse.
15.	At the administration and library building, the moment frame column to beam connections do not develop the capacity of either the beams or the column. Therefore, yielding will occur in the connection that may lead to a more brittle failure mode.
16.	At the administration and library building, the bottom flanges of the moment frame beams aren't braced laterally. Also, some of the moment frame joints aren't braced out-of-plane.
17.	The moment frame beams at the administration and library building are non-compact sections. Non-compact sections may exhibit poor ductility under cyclic loading.
18.	At the administration and library building, the base connection of the moment frame columns is a grouted pocket. This connection will degrade quickly under cyclic loading, losing fixity at the base
19.	The flexibility of the administration and library building moment frame system is incompatible with the stiff longitudinal shear walls in the adjacent classroom building. This may cause tearing and partial collapse in the connecting corridor structure as the buildings move relative to each other.
20.	At the administration and library building, the out-of-plane forces at the tilt-up walls are resisted by the top chord of the open-web roof joists. These roof joist top chords may be inadequate to carry the compression force and may buckle under seismic loading. Also, there are no continuous cross-ties and the sub-diaphragms may be overstressed. Also, the existing connections from the joists to the diaphragm are overstressed.
21.	At the connecting corridor between the classroom buildings and the administration and library building, there is a lack of collector elements to tie the corridor roof to the administration and library building in the longitudinal and transverse directions. This may cause tearing and partial collapse in the connecting corridor structure
22.	The transverse moment frames at the auto shop building are overstressed in flexure.
23.	The auto shop roof diaphragm has straight sheathing, which is highly overstressed. There are also excessive skylight openings in the roof.
24.	There is a concrete wall at the east end of the auto shop building and a diagonally sheathed wall at the west end. This disparity in stiffness will induce torsion into the building under seismic loading.
25.	At the auto shop building, the braced frame diagonals are made of T sections and are loaded eccentrically. These braces are overstressed. The braced frame diagonals also do not meet the compactness or slenderness requirements of the current code. This will lead to less post-elastic strength and ductility in the frame
26.	At the auto shop building, the roof beams are unable to resist the post-buckling unbalanced tension brace force and will result in negative post-buckling stiffness for the structure.
27.	At the auto shop building, the wall out-of-plane forces are resisted by the top chord

	of the open-web roof joists. These roof joist top chords may be inadequate to carry the compression force and may buckle under seismic loading.
28.	At the auto shop building braced frames, there is no bracing out-of-plane where the chevron braces intersect the beam.
29.	At the auto shop building and cafeteria braced frames, the braces intersect the columns about 5" above the slab level and frame into the weak axis of the column. This adds additional loads to the columns.
30.	At the administration and library building, the cafeteria, and the auto shop building, the shear wall connection to the foundation consists only of shear keys filled with grout. No dowels are present. The wall may slide in-plane or jump laterally off of the foundation
31.	The roof of the cafeteria has a 2x6 straight sheathing diaphragm. This diaphragm is highly overstressed.
32.	The cafeteria building has concrete shear walls and steel braced frames on the same line of resistance. Because of the large difference in stiffness between the concrete walls and the steel bracing, the walls will attract more seismic load. This will cause an overstress in the collector connections, which may result in a partial collapse of the building
33.	At the south wall of the cafeteria, the concrete shear wall doesn't have enough dead load to resist the seismic overturning moments.
34.	At the cafeteria, it is unclear from the construction drawings if there is anchorage of the wall to the roof diaphragm for resisting seismic out-of-plane forces. This may result in a partial collapse of the structure.
35.	At the cafeteria, the wall out-of-plane forces at the interior transverse concrete wall are resisted by the top chord of the open-web roof joists. These roof joist top chords may be inadequate to carry the compression force and may buckle under seismic loading. Also, there are no continuous cross-ties
36.	The hollow clay tile partition walls in the boys and girls locker rooms may not be properly braced and may collapse under seismic loading.
37.	The concrete roofs of the boys and girls locker rooms have multiple skylight openings. The roof diaphragms are overstressed.
38.	There is water damage to the acoustical tile ceiling in the gymnasium. This may indicate damage to the roof sheathing as well. Deterioration of the support members may result if not attended to.
39.	There is a ledger connection from the covered walkway to the concrete wall at each of the locker rooms. This connection is subjected to cross-grain tension due to seismic loading, which may result in loss of gravity support for the walkway roof. This is directly above a building exit. W
40.	The free-standing covered walkways between buildings are connected to each other without seismic joints between perpendicular segments. This may result in tearing of the roof and partial collapse.
41.	The cantilever columns at the free-standing covered walkways between buildings do not have sufficient fixity at the base to prevent excessive rotations that may lead to collapse.
42.	At the boiler house, the out-of-plane forces on the transverse walls are resisted by

	the top chord of the open-web roof joists. These roof joist top chords may be inadequate to carry the compression force and may buckle under seismic loading. Also, there are no continuous cross-ties
43.	The boiler house roof diaphragm is 2x diagonal sheathing and is moderately overstressed. This condition is aggravated by the fact that the building has walls only on 3 sides.
44.	There are no holdowns at the ends of shear walls at the electrical service enclosure. Some walls do not have enough tributary load to resist overturning loads
45.	At the portable classrooms, there is conduit running between the portable classrooms at the roof level.

10.9 Recommendations

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

Item	Recommended Remediation	Priority	Drawing Number
1.	Provide supplemental supports adjacent to each building so that damage will not lead to the collapse of the walkway. Reroute conduits and piping so that it is not supported by covered walkways or provide flexible connections.	1.2	None
2.	Retrofit the existing wall out-of-plane anchorage to the diaphragm. Strengthen the sub-diaphragms and collectors. wIf analysis of the trusses confirms that they were designed to carry gravity loads as simple spans, provide a connection to the supports that laterally stabilizes the bottom chord but doesn’t transfer axial loads into it.	1.2	2
3.	Provide intermittent steel angles from the wall to the adjacent slab and expansion anchor to tie it into the wall and slab.	1.5	1
4.	Provide new bolted connection of redwood sill into shear wall to transfer in-plane shears into the wall.	1.2	2
5.	Provide 2 bays of new interior transverse braced frames at each wing of the building to reduce the diaphragm span. Provide new collectors and renail the diaphragm as required along these lines.	1.1	1,2
6.	Provide 3 new braced frames and associated footings and at each exterior longitudinal wall of each classroom wing. Strengthen the collectors on these lines.	1.1	1,2
7.	Provide 3 new braced frames and associated footings and at each exterior longitudinal wall of each classroom wing. Strengthen the collectors on these lines.	1.1	1,2
8.	Provide 3 new braced frames and associated footings and at each exterior longitudinal wall of each classroom wing.	1.1	1,2

	Strengthen the collectors on these lines.		
9.	Provide new bridging at the open-web joists. Provide a minimum of two bracing points at each bottom chord.	2.0	2
10.	Provide new chord elements that are capable of transferring axial forces between the building wings.	1.2	2
11.	Provide new continuous collector elements, such as a continuous strap above the roof diaphragm.	1.0	2
12.	Provide additional out-of-plane anchorage of the walls at 8' o.c. Strengthen the sub-diaphragms and collectors.	1.1	2
13.	Provide 2 new interior transverse braced frames and associated footings to reduce the diaphragm span. Strengthen existing collectors.	1.1	3,4
14.	Provide 4 new braced frames and associated footings at each of the exterior longitudinal walls. Strengthen existing collectors.	1.0	3,4
15.	Provide 4 new braced frames and associated footings at each of the exterior longitudinal walls. Strengthen existing collectors.	1.0	3,4
16.	Provide lateral bracing at the midspan of the beam bottom flange and at the beam-column connections. With the addition of new braced frames, these collectors still need bracing	1.5	4
17.	Provide 4 new braced frames and associated footings at each of the exterior longitudinal walls. Strengthen existing collectors.	1.7	3,4
18.	Provide 4 new braced frames and associated footings at each of the exterior longitudinal walls. Strengthen existing collectors.	1.2	3,4
19.	Provide 4 new braced frames and associated footings at each of the exterior longitudinal walls. Strengthen existing collectors.	1.1	3,4
20.	Provide new additional out-of-plane anchorage of the wall to the diaphragm at 8' o.c., max. Strengthen the sub-diaphragms and collectors. Provide continuous cross-ties.	2.5	4
21.	Strengthen the existing collector elements and connections or provide new collector elements. Provide interior braced frames at the building as discussed above.	1.1	4
22.	Add 2 new braced frames and associated footings on each side of the central corridor (total of 4 frames). Strengthen existing collectors.	1.0	5,6
23.	Add a layer of new ½" plywood over the existing straight sheathing throughout the roof area. Strengthen connections to the existing wall lines.	1.8	6
24.	Provide interior braced frames at the building as discussed above. This will reduce the torsional effects to an acceptable	3.0	5,6

	level.		
25.	Provide new concentrically loaded braces with symmetric sections. Provide new connections to beams and columns.	2.0	5,6
26.	Provide new braces to eliminate the weak chevron frame configuration	2.0	6
27.	Provide new out-of-plane anchorage of the wall to the diaphragm at 8' o.c., max.	2.0	6
28.	Add new bracing of existing beam as required.	1.5	6
29.	Stiffen the column in the area between the brace workpoint and top of the slab. Extend this stiffened section above the workpoint and below the slab as required.	1.5	6,8
30.	Provide intermittent steel angles from the wall to the adjacent slab and expansion anchor to tie it into the wall and slab.	1.0	3,5,7,11
31.	Provide new plywood sheathing nailed over the existing T&G sheathing. Add a new interior transverse frame or wall and associated footing in the cafeteria area. To avoid interrupting the open cafeteria space, this brace might be placed at the exterior of the building, acting like a buttress.	1.2	8
32.	Strengthen the collector elements and their connections.	1.0	8
33.	Provide a new footing with additional weight to help resist overturning. Dowel this new footing into the existing footing.	1.1	7
34.	Investigate presence of out-of-plane wall anchorage. Provide new out-of-plane anchorage of the wall to the diaphragm if necessary.	1.0	8
35.	Provide new out-of-plane anchorage of the wall to the diaphragm	1.6	8
36.	Remove the existing partitions and build new partition walls using sheet metal studs.	1.0	9
37.	Provide diagonal bracing in some of the skylight bays.	1.1	10
38.	Repair the leaking roof and replace any deteriorated roof sheathing.	3.0	10
39.	Provide new secondary support columns and beam to prevent loss of vertical support.	1.3	10
40.	Provide additional gravity support columns so each side of the joint area is supported.	1.2	None
41.	Provide new welding and stiffeners at the column to beam connection of the corridors. Provide framing with moment connections in the transverse direction to stiffen the frame. Alternatively, provide new transverse thickened slab or grade beams to provide more rotational stiffness at the bases of the columns.	1.1	None
42.	Provide new out-of-plane anchorage of the wall to the diaphragm and continuous cross-ties.	1.3	11

43.	Provide new plywood sheathing over the existing diagonal sheathing	1.9	11
44.	Provide new holdowns at ends of shear walls.	1.2	12
45.	Provide flexible connections for conduit running between portables.	1.9	None

10.10 Portable Units

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

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

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

10.11 Structural Deficiency Prioritization

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

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

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

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

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

10.12 Conclusions

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

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

3. Overall, we recommend that seismic retrofit work for this school campus be performed in Phase 1A.

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.