STRUCTURAL ENGINEERING INSTITUTE

CASE STUDY

Structure:	Large Retail Building
Location:	Yucca Valley, California
Completion Date:	About 1990
Date of Failure:	June 28, 1992
Failure Investigated By:	Gregg E. Brandow, PhD, SE Brandow & Johnston, Inc.
Causes of Failure:	Lack of Understanding of Seismic Performance and Design Requirements

The failure of this large retail building during a large earthquake was caused by a significant deficiency in the structural design which demonstrated a lack of understanding of the seismic performance of this class of building and the design requirements for the structural system. The building was located in Yucca Valley in the California desert and the failure occurred during the Landers Earthquake of 1992. The design was performed by a Licensed California Professional Engineer.

BUILDING DESCRIPTION

This large retail building is shown in the photograph and has overall dimensions of 293 feet by 223 feet, as shown in Figure 1. The building is single story except for a mezzanine that extends across the north bay of the building. The roof utilizes steel bar joists framing to steel truss girders with a typical bay size of 41 feet by 38 feet 8 inches. An inch and one-half thick metal deck spans between the joists and supports insulation and roofing. The roof framing is supported on steel pipe columns and on concrete masonry walls located on the perimeter of the building and on the interior in the building on grids 2 and 3 and grids D1, D2, D3, and F. The foundations are a bearing type foundation with isolated spread footings under the columns and continuous spread footings under the walls.

The seismic resistance of the building relies on the strength and stiffness of the concrete masonry walls which are reinforced and partially grouted. The metal deck roof forms a diaphragm that distributes the seismic inertia forces to the walls. This type of structural system relies on the structural detailing to ensure that the seismic forces can be collected through the flexible roof diaphragm and transferred to each wall based on the location and stiffness of the walls.

THE COLLAPSE DURING THE LANDERS EARTHQUAKE

The roof collapse between grid 2 and 3, as shown in the photograph, occurred because of two modes of failure. The first is due to the roof diaphragm pulling away from the wall due to east-west diaphragm deflection. This deflection was probably on the order of 4 to 6 inches and was caused by the acceleration of the roof and the east wall away from the west wall on grid 2. The west wall was unable to deflect with the roof due to the stiffening effect of the cross walls on grids C2, D1, D, D3, and F. The weak connections between the roof and the walls contributed to the failure. The second mode of failure is a shear failure along grid 2 due to the lack of welding between the ledger angle and the imbeds from grid A to C2, the omitted shear transfer detail between C2 and G1, and the lack of diaphragm chord and collector design along grid 2. All the shear transfer at grid 2 was forced into the joist connection to the wall, which was already inadequate for the pulling forces from the diaphragm perpendicular to the wall.

The failures at the framing to wall connections at grid 3 between D and F are due to the incompatibility of the flexible diaphragm and the rigid interior masonry walls. The movement of the flexible roof diaphragm ripped the framing members and their connections from the stiff walls. The connections were anchored into unreinforced masonry units and thus the failures were quite brittle.

The failure of the diaphragm connection to the rear wall at grid H is a result of the very weak connection to the masonry wall. The anchor bolts failed by breaking out of the inside face of the wall.

DESIGN DEFECTS

The seismic design of this building was based on the assumption of a flexible roof diaphragm which was assumed to span across the building between the masonry walls on grids A and H and between 2 and 8. For E-W forces, the design ignored the stiff walls on grid lines C2, D1, D2, D3, and F which restrained the roof diaphragm and caused large restraint (collector) forces at these walls. With the lack of design, there were no drag members or connections which resulted in the diaphragm pulling away from these walls which then allowed the diaphragm to span between the exterior walls, as shown in Figures 2 and 3. The diaphragm deflected which caused the framing to loose vertical support as it pulled away from the walls. The following were the design deficiencies:

1. For east-west seismic forces, the diaphragm deflection of several inches is incompatible with the relatively rigid masonry walls on grids C2, D1, D2, D3 and F. For north-south seismic forces, the diaphragm deflection is incompatible with the masonry wall on grid 3. These masonry walls should have been accounted for in the seismic design and appropriately designed and detailed with collectors, as required by UBC section 2312(e)4 which states:

   > **Distribution of horizontal shear.** Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

   In another section of the UBC, section 2312(j) 2D, the requirements for diaphragms is explained:

   > Diaphragms supporting concrete or masonry walls shall have continuous ties between diaphragm chords to distribute, into the diaphragm, the anchorage forces specified in this chapter. Added chords may be used to form sub-diaphragms to transmit the anchorage forces to the main cross ties. Diaphragm deformation shall be considered in the design of the supported walls.

2. Diaphragm chord and collector detailing is lacking at the diaphragm boundary at grid 2. This seriously limits the ability of the diaphragm to distribute seismic forces to the masonry shear walls. This deficiency is in violation of UBC Section 2312(a) which reads as follows:

   > Every building or structure and every portion thereof shall be designed and constructed to resist stresses produced by lateral forces as provided in this section.

   The collapse of the roof, the tear in the roof diaphragm, and the pilaster damage at grid 2 and D3 are caused to some degree by this defect.

3. The diaphragm-wall connections are not detailed in a manner to assure an adequate transfer of forces between the wall and the trusses, as shown in Figure 4, truss girders or the bridging. Anchor bolts are so close to the face of the block that the connection capacity is very small compared to the capacity required by UBC Section 2310:

> **Anchorage of Concrete or Masonry Walls**
>
> **Sec. 2310.** Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter or a minimum of 200 pounds per foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet. Required anchors in masonry walls or hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall.

4. The diaphragm shear transfer between the roof diaphragm and the interior masonry walls requires a connection between the metal deck and the top of the wall.

CONCLUSIONS

The intent of the seismic design standards in the Uniform Building Code and the standards of practice in California is to have acceptable performance of buildings in earthquakes and prevent collapse. The failures that occurred at this building are unacceptable and are a result of an apparent lack of understanding of the seismic lateral load resisting system required for this type of building and the UBC requirements for the design and detailing of the building combined.

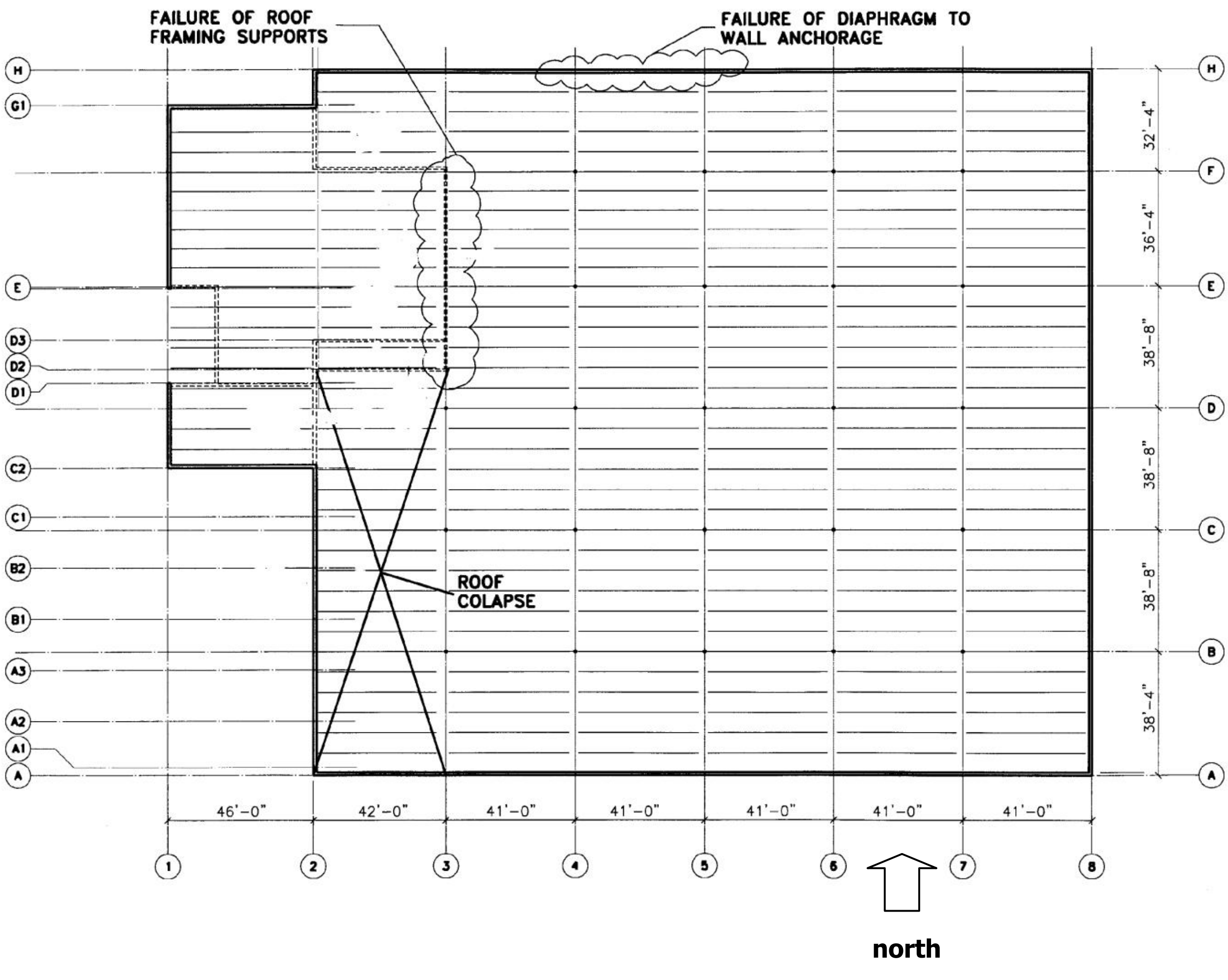

FAILURE OF ROOF FRAMING SUPPORTS
FAILURE OF DIAPHRAGM TO WALL ANCHORAGE
ROOF COLAPSE
H
G1
E
D3
D2
D1
C2
C1
B2
B1
A3
A2
A1
A
F
D
C
B
32'-4"
36'-4"
38'-8"
38'-8"
38'-8"
38'-4"
46'-0"
42'-0"
41'-0"
41'-0"
41'-0"
41'-0"
41'-0"
1
2
3
4
5
6
7
8
north

FIGURE 1

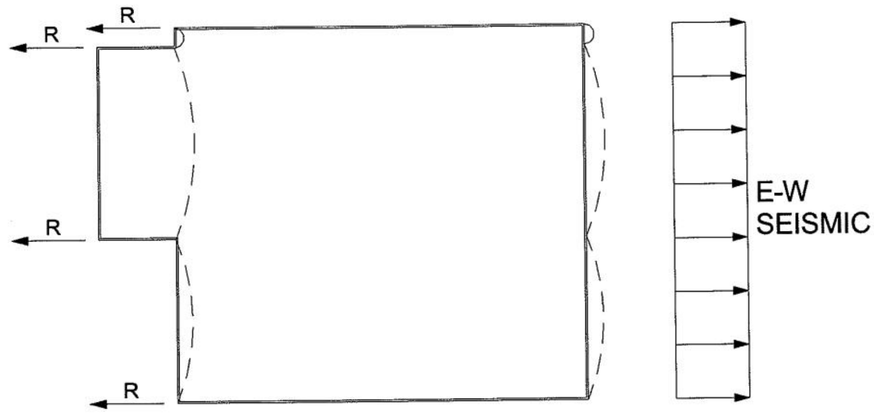


Figure 2: Before Wall Anchorage Failed

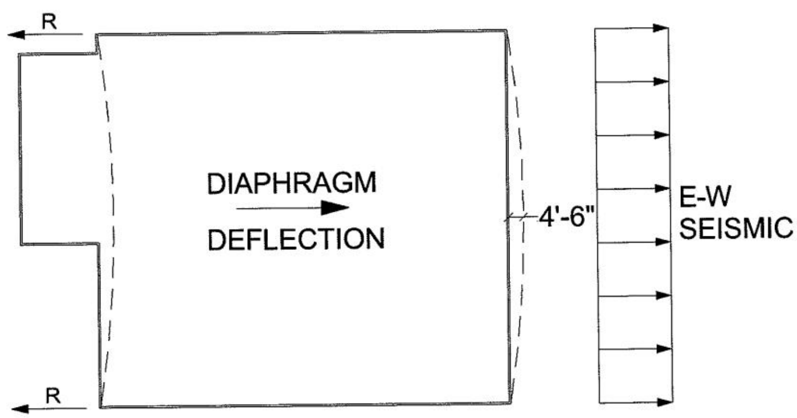
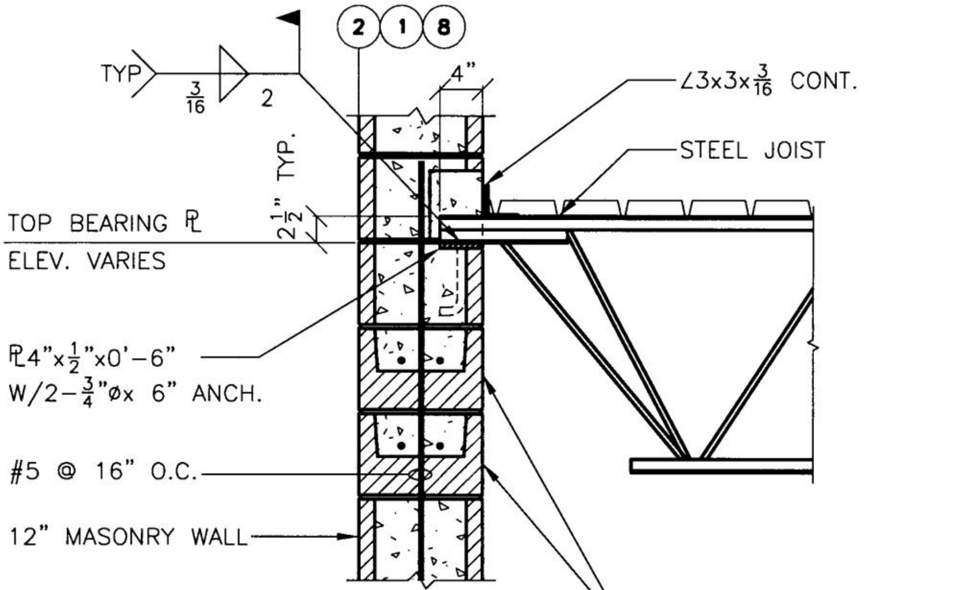
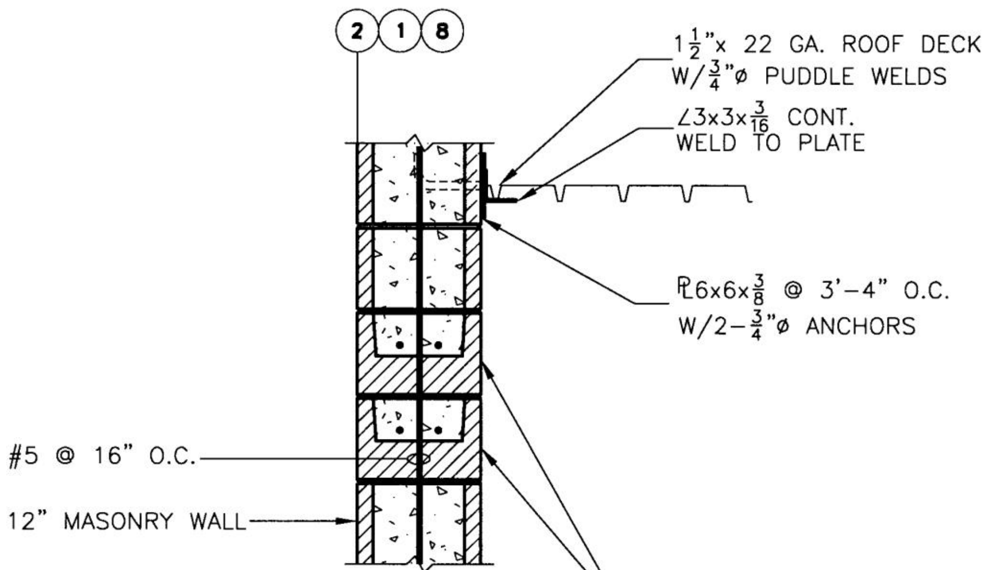


Figure 3: After Wall Anchorage Failed



SECTION



SECTION 8B

FIGURE 4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U-HAUL