Technical Paper on

OOPS TESTING OF TILT-UP TOP PLATE CONNECTIONS

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ABSTRACT

Concrete tilt-up buildings have historically been constructed by lifting (or "tilting up") precast concrete wall panels, then connecting the panels at the top with a wood or metal deck diaphragm to a ledger. This connection results in a portion of the wall (parapet) extending above the roof line. However, in some instances, the roof-to-wall connection consists of the use of a top plate bolted into the top of the precast panel (in lieu of the ledger) which is then connected to the roof diaphragm. This connection results in no parapet and provides advantages for drainage, height restrictions, and snow loading (no drift). In a wood roof diaphragm application with a top plate, out-of-plane seismic (OOPS) anchorage for the wall is sometimes provided indirectly through straps which are nailed to the top of the purlins or subpurlins and then nailed into the top plate. This paper provides a review of this top plate connection detail based on a limited testing program.

INTRODUCTION

Tilt-up structures have had poor seismic performance during previous earthquakes, which have resulted in the periodic modification of building codes to address these types of structures. Positive roof-to-wall anchors were implemented in the 1973 Uniform Building Code (UBC) following the 1971 San Fernando Earthquake. Subdiaphragm ties were required to transfer forces into the diaphragm in the 1976 UBC. Wall forces were increased due to the dynamic nature of a flexible diaphragm in the 1991 UBC, and again in general in the 1997 UBC. Various other requirements have also been added to the building codes over the years to address the performance of tilt-up buildings.

During the evolution of the building code requirements for tilt-ups, the design of the buildings was also evolving. One area of development was the elimination of the parapet and the use of a top plate connection (Figures 1 & 2). This connection has been challenged on many occasions as to whether it meets the code requirements for both strength and ductility. This connection is common in the Pacific Northwest (and to a lesser extent elsewhere) in buildings constructed since the mid-1970's.

In order to address the wood diaphragm top plate connection issue, six mock-up wood panels were constructed and tested. A description of the panels, testing, and results follows. An example is also provided for comparison of different standards.

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TILT-UP TEST PANELS

Six wood-framed test panels were fabricated based on construction documents for a tilt-up structure built in 1988. The building utilizes ½" plywood sheathing on 2x4 subpurlins located at 2'-0 on center, which in turn are supported by 4x14 purlins located at 8'-0 on center. The roof diaphragm is supported at the perimeter of the building by metal hangars connected to a 4x top plate which is bolted to the 5½" thick concrete wall with ¾" anchor bolts spaced at 4'-0 on center. Out-of-plane anchorage is provided by Simpson ST6224 straps located over each purlin at 8'-0 on-center and ST12 straps located over subpurlins at 4'-0 on-center (Photos 1, 2).

The test panels were constructed in a shop setting utilizing the same materials as specified on the construction documents. Quality control of the test panels is considered better than that which was likely provided in the field on a typical tilt-up building. Construction of the test panels was performed by a carpenter/framer who is experienced in the construction and retrofit of tilt-up buildings.

The six test panels were constructed as follows:

Panel No.	Member Size	Strap	Nailing
1	2x4 subpurlin	ST12	2 rows 10d @4" to top plate, 10d@6" to framing
2	2x4 subpurlin	ST12	2 rows 10d @4" to top plate, 10d@6" to framing
3	2x4 subpurlin	ST12	None to top plate, 10d@6" to framing
3A	2x4 subpurlin	None	2 rows 10d @4" to top plate, 10d@6" to framing
4	4x purlin	ST6224	2 rows 10d @4" to top plate, 10d@6" to framing
5	4x purlin	ST6224	2 rows 10d @4" to top plate, 10d@6" to framing
6	4x purlin	ST6224	None to top plate, 10d@6" to framing

Note: Test panel 3A was modified and reused from test panel 3.

ST12 and ST6224 straps were installed using Simpson N10 nails in each available hole. Straps were bent over the top plate 2" as shown on the original construction documents.

Test panels 3 and 3A were designed to test the individual contributions of the ST12 strap and the top plate nailing, both of which were provided in test panels 1 and 2. Test panel 3 (strap but no top plate nailing) was tested to failure of the ST12 strap. After which, the strap was removed, the top plate nailing installed, and re-tested as panel 3A (top plate nailing but no strap).

TILT-UP TEST PANEL PROCEDURE

Testing of the tilt-up panels was performed by Mayes Testing Engineers in Everett, WA on December 19, 2002. Test panels were laid flat on a concrete floor slab and bolted into the slab with ¾" anchor bolts at 4'-0 on center in order to replicate the connection of the top plate to the top of the concrete wall. Monotonic testing was performed using a 20 ton hydraulic ram with a 2 ksi gauge for Panels 1-3 (ST12 straps) and a 20 ton ram with a 5 ksi gauge for Panels 4-6 (ST6224 straps). Gauge readings were in psi, then converted to kips using calibration curves. (Photos 3, 4)

Load was applied to the framing member connected to the strap in a slow steady state. Ultimate load was considered achieved at the maximum obtained load, prior to a drop in load.

Results from the panel load tests are as follows:

Panel No.	Strap	Ultimate Load	Failure Mode
1	ST12	7,000 lbs	Pulled 2x subpurlin from strap and plywood
2	ST12	7,400 lbs	Pulled 2x subpurlin from strap and plywood
3	ST12	2,900 lbs	Strap pulled out of top plate
3A	None	7,600 lbs	Nails pulled through 2x4 subpurlin
4	ST6224	11,000 lbs	Top plate broke in cross grain bending
5	ST6224	10,600 lbs	Top plate split at anchor bolts
6	ST6224	9,000 lbs	Pulled nails out of 4x6 top plate

OBSERVATIONS DURING FABRICATION AND TESTING

During the course of fabrication and testing of the panels, several observations were noted and are included below.

1. The length of the ST12 strap is such, that when installed per the construction details, only 4 nails were provided into the subpurlins. (Photo 2)
2. The ST12 strap provided little additional strength to the overall connection. While test panel 3 indicated the strap provided 2,900 lbs to the connection, test panel 3A (without any strap) had an ultimate load similar to test panels 1 & 2 (7,000+ lb). Therefore, the primary out-of-plane load transfer is provided through nailing of the plywood to the top plate (cross grain). (Photo 5).
3. The ST6224 strap in panels 4-6 contained enough strength that the top plate failed. The wrap around design resulted in cross grain bending of the top plate. (Photo 6)
4. Splitting of the top plate could not be observed after the load was removed following failure of panels 4 & 5. Thus, visual observation of the connections following an earthquake may not be adequate to determine whether the top plate was damaged.
5. Prior to the “ultimate strength” failure of the connection, there was significant separation between the framing and the top plate, usually on the order of ½” to 1”.

EXAMPLE

The following example is provided as a comparison of different documents for the existing subject building.

Given: Tilt-up structure constructed in 1988 per the 1985 UBC, Seismic Zone 3.
Panel height = 26 feet, Thickness = 5½", No parapet

Tributary Weight (W) = (5.5in/12)(26'/2)(150 lb/ft) = 894 lb/ft

Demand: (Out-of-plane forces)

1985 UBC: Fp = ZICpWp = ¾(1.0)(.3)Wp = 0.225Wp
1988 UBC: Fp = ZICpWp = .3(1.0)(.9)Wp = 0.25Wp
1994 UBC: Fp = ZICpW x 1.5 = .3(1.0)(.9)(1.5)Wp = 0.375Wp
1997 UBC: Fp = (apCaIp/Rp)(1+3(hx/hr))Wp
= (1.5(.36)(1.0)/3.0)(1+3(26'/26')) = .72Wp (ult) or 0.51 (allowable)
FEMA-178: Fp = .67AvCcWp = .67(.3)(.9)Wp = 0.18Wp (ult) or 0.13 (allowable)
ASCE-31: Tc = YSdsWpAp = (.9)(.91)W = 0.82Wp (ult) or 0.59 (allowable), where
Ss=0.63 (475-yr for comparison), Fa=1.38 for Soil Class E

Capacity:
The capacity of the connection based on ASTM-D1764 is the lesser of (1) the actual load @ 1/8" deflection, or (2) the ultimate load divided by 3. For the testing conducted, deflection was significant (in some cases exceeded ½"), but not measured.

The tests conducted without diaphragm shear nails into the top plate (2 rows @ 4" o.c.) resulted in 2900 lb (Panel 3) and 9,000 lb (Panel 6) ultimate load for the ST12 and ST6224 straps, respectively. The resulting allowable loads would be 2900 lb/3 = 966 lb and 9,000 lb/3 = 3,000 lb, which is a good correlation with the published Simpson catalog values (ST12 = 900 lb, ST6224 = 2500 lb). However, because the straps are utilized in a non-standard installation (only 4 nails are installed in the ST12) the calculated allowable load based on N10 nails is 4 x 92 lb (1.33) = 490 lb.

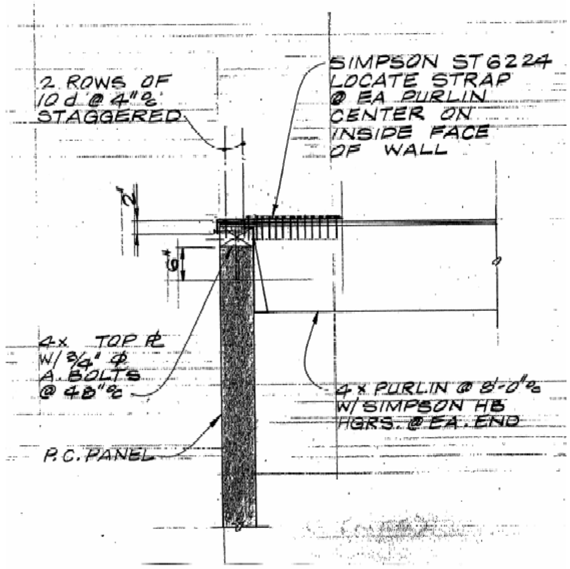
Demand/Capacity Ratios:
The following table represents the demand to capacity ratios for the connection.

	ST12		ST6224	
	Load (lbs)	D/C Ratio	Load (lbs)	D/C Ratio
Capacity	490		2,500	
1985 UBC	805	1.6	1,610	0.64
1988 UBC	895	1.8	1,790	0.72
1994 UBC	1,340	2.7	2,680	1.1
1997 UBC	1,840	3.8	3,680	1.5
FEMA-178	460	0.94	920	0.37
ASCE-31	2,090	4.3	4,190	1.7

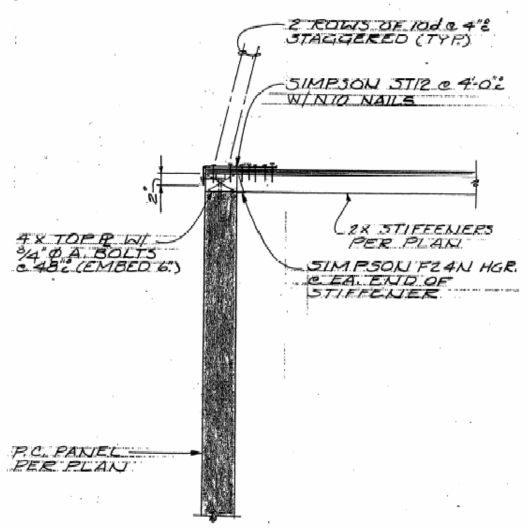
CONCLUSIONS/SUMMARY

Out-of-plane top plate connections continue to be provided in the design of new buildings and are present in possibly thousands of existing buildings. These connections are often noted as “deficiencies” in seismic reviews of existing buildings for lenders and potential owners, and have resulted in seismic retrofits to the connection. Therefore, the testing information in this report is intended to provide information about this type of connection for engineers (both those designing and those evaluating buildings) a basis to draw their own conclusions.

Based on the observations and testing, it is the opinion of the authors, that the connections do not provide the reliability that the building codes intend. While the ultimate loads achieved during testing correlated well with the anticipated allowable loads, the failure modes were also anticipated. Thus, under low force levels where the connections have a high overstrength factor (safety factor), the connections are anticipated to perform fairly well. However, under conditions where the forces are high (i.e. high ground motions, tall and/or thick wall panels, flexible diaphragms due to long spans, etc.) the reliability of these connections is poor. Engineers utilizing the detail for design should recognize the deficiencies associated with the connection. Engineers providing a review of existing details should also recognize the strength of the connection.



Purlin to Wall Connection
Figure 1



Subpurlin to Wall Connection
Figure 2

Construction of Test Panel
Photograph 1

ST12 Strap Installation
Photograph 2

Test Panel Setup
Photograph 3

Test Panel Setup
Photograph 4

Separation of Strap from 2x4 Subpurlin
Photograph 5

Cross Grain Bending of Top Plate
Photograph 6