PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

Evaluation and Application of Concrete Tilt-up Assessment Methodologies

Timothy Graf

and

James O. Malley

Degenkolb Engineers
San Francisco

Sponsor:
California Energy Commission

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Pacific Earthquake Engineering Research Center
College of Engineering
University of California, Berkeley

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ABSTRACT

This report covers Task 1 of the Pacific Earthquake Engineering Research (PEER) Center Task 509 — Evaluation and Application of Concrete Tilt-up Assessment Methodologies. The objective of Task 1 of Lifelines 509 is to summarize the research findings and results of previous PEER research projects on concrete tilt-up buildings and to assess the report findings on their impact upon current design codes and guidelines. From all four reports evaluated, some suggestions are presented for changes in design. Ideas for future PEER research not specific to any of the reports are also presented.

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EXECUTIVE SUMMARY

This report covers Task 1 of the Pacific Earthquake Engineering Research (PEER) Center Task 509 — Evaluation and Application of Concrete Tilt-up Assessment Methodologies. The objective of Task 1 of Lifelines 509 is to summarize the research findings and results of previous PEER research projects on concrete tilt-up buildings and to assess the report findings on their impact upon current design codes and guidelines. These research projects are (1) *Seismic Performance of Tilt-up Buildings* by John F. Hall; (2) *Building Vulnerability Studies: Modeling and Evaluation of Tilt-up and Steel Reinforced Concrete Buildings* by John W. Wallace, Jonathan P. Stewart, and Andrew S. Whittaker; (3) *Stiffness of Timber Diaphragms and Strength of Timber Connections* by Gerard C. Pardoen, Daniel Del Carlo, and Robert P. Kazanjy; and (4) *Seismic Performance of an Instrumented Tilt-up Building* by James C. Anderson and Vitelmo V. Bertero. Each report covers primarily the performance of existing older pre-1997 tilt-up construction. Little research was done in these reports on the performance of newer post-1997 tilt-up buildings, so this topic will not be specifically addressed in depth in this report.

This report is organized into six main sections. The first two sections offer a brief overview of tilt-ups, the scope of this report, and short synopses of the reports in this task. The third section consists of comparison of the data to guidelines (FEMA 356) and established codes (2000 IBC; 1997 UBC). This section comprises conclusions drawn by Degenkolb Engineers and is not the work of the original authors. The fourth section lists a series of questions for the authors concerning their reports. The fifth section has suggestions for future PEER research presented by the authors and added to by Degenkolb and other practicing engineers. The final section consists of suggestions for practicing engineers on how to incorporate the ideas presented in these four reports. Additional, more complete summaries of all the reports are also included in the appendices.

The data from the reports were evaluated to test the validity of design procedures and to identify what changes, if any, could be made to current design methodologies.

The Hall report, when compared to 2000 IBC requirements and FEMA 356, had out-of-plane wall anchor force estimates slightly below code values for the longitudinal walls but had much higher anchorage loads for the transverse walls. The analytically determined period values from the Hall report were shorter than those obtained from FEMA 356.

The Wallace report showed the importance of including provisions on near-field effects because of their influence on building performance and the importance of obtaining an accurate diaphragm stiffness.

The Pardoen report's experimentally determined diaphragm yield strength values did not compare well with code and FEMA 356 values; however, the experimentally determined ultimate capacities were fairly close to FEMA 356 values. Some discrepancies also exist between the experimentally determined diaphragm backbone curves and those prescribed by FEMA 356.

In the Anderson report, the analytically determined values for building periods generally led to period values that were very close to the instrumentally recorded values but much shorter than those of FEMA 356.

Each report raised questions regarding assumptions and procedures in the research process, and the questions are presented for the authors to consider. Most of the questions from the Hall, Wallace, and Anderson reports cover modeling assumptions and inputs used in the computer models. Questions on the Pardoen report, the only report focusing on laboratory testing, are related to test assumptions and setup.

From all four reports evaluated, some suggestions are presented for changes in design. The Hall, Wallace, and Anderson reports found that analyses can be as simple as a two-dimensional (2D) shear beam model or as complicated as a three-dimensional (3D) finite element model. Although both predict overall building response quite well, the shear beam model does so more quickly. These reports and the Pardoen report also found that the strength and ductility of the roof connections affect the performance of the building and that the design of the connections is very important. While these reports support current methods used in structural engineering, no major changes to the current codes and guidelines were recommended due to these studies.

The reports also illuminated ideas for further research of tilt-up building construction. Research suggestions for the authors of the reports ranged from studying the anchorage forces along the short side of the building versus the long side; rerunning analysis done in the reports using the current building code, FEMA 356 values, or updated data; and conducting more laboratory testing of tilt-up components. The reports also led to many ideas for future PEER research not specific to any of the reports.

The many suggestions for further tilt-up construction research include instrumentation of more tilt-up buildings, studies of connection improvement in existing tilt-ups, better period estimation, more and better documentation of damage in tilt-ups, the strength of connections in cross-grain ledger bending, and amplification of forces at pilasters.

After reviewing the four reports, several important points emerged for practicing engineers to consider in evaluating an existing tilt-up building:

- Out-of-plane wall anchor forces on the short side of a tilt-up can be larger than on the long side due to the diaphragm remaining elastic during shaking in the longitudinal direction.
- Near-fault effects can have a significant effect on the seismic performance of tilt-ups.
- The addition of sliders or the removal of the beam seat bolts under an existing GLB at the top of a pilaster can decrease the wall anchor demand.
- While all the research seems to have captured global performance well, the primary element of importance in tilt-up design still lies in the design of the connections of the diaphragm to the walls. Strong, stiff connections are needed for good building performance during an earthquake.
- The building period determined by analytical models and the recorded building period are shorter than the periods produced by FEMA 356 or dynamics. Using either of these methods may lead to an unconservative design. Using the acceleration from the plateau of the response spectrum is recommended unless a more detailed computer model suggests otherwise.

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1 Introduction

1.1 TILT-UP HISTORY

According to the Structural Engineers Association of California (SEAOC) *Guidelines for Seismic Evaluation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*, "tilt-ups were first used in the early 1900s as an efficient method of fabricating durable wall panels used in military structures." Tilt-ups have since become widely used for low-rise warehouses, and other large industrial-type structures. These buildings can be constructed relatively cheaply and quickly, and allow for quick occupation. The building name, "tilt-up," derives from the method in which the building is constructed. The wall panels, usually of reinforced concrete, are constructed on the ground and then tilted into place on-site. A roof then connects the wall panels. On the West Coast the choice of roof construction in the past was usually wood frame because of the availability of the material. Metal decking and joists can also be used for the roof, and are becoming more common.

After the 1994 Northridge, California, earthquake tilt-ups were discovered to have several vulnerabilities. According to SEAOC tilt-ups are "widely recognized as having a significant risk of sustaining life-threatening damage, including partial or complete collapse during moderate-to-strong earthquakes… primarily due to inadequate anchorage for walls into the horizontal roof and floor diaphragms."

Since this unfortunate discovery, many research projects have been conducted on tilt-ups and their behavior. Some of these studies have been conducted under the Pacific Earthquake Engineering Research (PEER) Center. PEER has chosen four reports on tilt-ups for study and evaluation under PEER Task 509.

1.2 PEER PROJECT OVERVIEW

PEER Task 509 covers the evaluation and application of concrete tilt-up assessment methodologies. Task 509 is broken up into two tasks:

Task 1

Summarize the research findings and results of previous (PEER) research projects on concrete tilt-up buildings. These research projects are documented in the following reports:

(1) *Seismic Performance of Tilt-up Buildings* by John F. Hall

(2) *Building Vulnerability Studies: Modeling and Evaluation of Tilt-up and Steel Reinforced Concrete Buildings* by John W. Wallace, Jonathan P. Stewart, and Andrew S. Whittaker

(3) *Stiffness of Timber Diaphragms and Strength of Timber Connections* by Gerard C. Pardoen, Daniel Del Carlo, and Robert P. Kazanjy

(4) *Seismic Performance of an Instrumented Tilt-up Building* by James C. Anderson and Vitelmo V. Bertero

The goal of Task 1 is to provide input to existing codes (e.g., International Building Code) and guidelines (e.g., FEMA 356) that will improve the seismic assessment methodologies of concrete tilt-up and other low-rise, rigid-wall, flexible diaphragm buildings.

Task 2

Apply the building assessment methodology developed in PEER Task 507*, Advanced Seismic Assessment Guidelines* by C. Allin Cornell, Paolo Bazzurro, Charles Menun, Maziar Motahari, to a concrete tilt-up building in the Pacific Gas & Electric (PG&E) building inventory. The tilt-up buildings selected are in Hollister, and Redlands, California. Alternatively, another PG&E tilt-up may be considered. [The alternative tilt-up was chosen, the PG&E Fremont building.]

Predicting the post-earthquake functionality of utility structures is a crucial step in evaluating the likelihood of the electric distribution network being able to provide gas and electricity to its customers. The final product of the guidelines is a set of fragility curves for structural limit states directly related to post-earthquake building occupancy status tags: namely, green, yellow, or red.

In this part of the project, we will apply the previously developed guidelines to one of PG&E's concrete tilt-up buildings. The objectives are (1) to identify potential difficulties that structural engineers would encounter in using the procedure described in the *Advanced Seismic Assessment Guidelines*; (2) to recommend possible revisions to the procedure to address any identified difficulties; and (3) to identify and make recommendations on other issues related to assessing the seismic reliability of utility structures and systems.

This report will focus on Task 1, the summarizing and evaluating of previous PEER reports on tilt-up construction. Task 2 will be covered in a future report.

1.3 REPORT ORGANIZATION

The report for Task 1 is organized into six main sections. The first two sections offer a brief overview of tilt-ups, the scope of this report, and short synopses of the reports in this task. The third section consists of comparison of the data to guidelines (FEMA 356) and established codes (2000 IBC; 1997 UBC). This section comprises conclusions drawn by Degenkolb Engineers and is not the work of the original authors. The fourth section lists a series of questions for the authors concerning their reports; the fifth section offers suggestions for future PEER research. The authors presented some ideas for additional research in the reports and Degenkolb Engineers and other practicing engineers have added to this list. The final section consists of suggestions for practicing engineers on how to incorporate the ideas presented in these four reports.

1.4 COMMON ASSUMPTIONS

Each report had some common assumptions about the tilt-ups that were modeled and studied:

- All the reports evaluated in this report concern the performance of existing older pre-1997 buildings. Existing older tilt-ups generally have solid panels and columns between panels. Construction from the late 1990s to present-day construction was either not investigated deeply or at all in the reports. Newer buildings have different connection details, are usually lacking columns between tilt-up panels, and often have more frame-like panels.
- Each building model was regular in plan and one-story tall, so no vertical or horizontal irregularities were included.
- All buildings, since they were modeled after West Coast tilt-ups, were assumed to have wood diaphragms. Metal decking is also used but more commonly in other regions of the country.
- No eccentric connections were modeled.

1.5 CODES AND GUIDELINE OVERVIEW

Three main documents were used to compare to the results obtained from the four reports. Two codes, the 1997 UBC and the 2000 IBC, and one guideline, FEMA 356, were used to calculate values that were compared to the report results obtained analytically or experimentally. The 1997 UBC and the 2000 IBC are used in the design of new buildings, while FEMA 356 is used to retrofit existing structures. The IBC has updated their code to the 2003 IBC, but it has not been widely adopted yet. The code contains revisions from the 2000 IBC, including some updated provisions that apply specifically to tilt-ups. For example, the out-of-plane anchorage force has been changed in the 2003 IBC to include a 1.4 increase when steel anchors are used, which was already established in the 1997 UBC.

A brief overview of the history of code development as it pertains to tilt-up design is given below. A more detailed account can be found in the SEAOC publication *Guidelines for Seismic Evaluation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*. The summary below will focus on two main aspects of the tilt-up design, the wall anchorage force and the base shear. Additional miscellaneous code provisions are also included.

Wall Anchorage: The 1927 UBC recognized the need for wall anchors and continuous ties, but not until the 1937 UBC was an out-of-plane force added. In 1949 a minimum pressure was added and later modified to a linear load in the 1958 UBC. The 1971 San Fernando, California, earthquake highlighted undesirable building performance that led to many additions to the UBC in the next few years. In the 1973 UBC the formula for the design force for wall anchorage was changed to reflect the effects of soft soils and included the importance of buildings. In 1979 the design force formula was changed once again. This revision did not include soft soil effects, but in high seismic areas the force was increased 50%. The anchorage design force remained basically unchanged until the 1997 UBC, which introduced two equations to determine the design wall anchorage force. These equations took into account the material type used in the anchorage and assigned a load factor depending on the material.

The 2000 IBC is very similar to the 1997 UBC but does include some difference for tilt-up design, especially for wall anchorage. The 2000 IBC uses the same load factor for all materials, and the out-of-plane force equation is not dependent on the height of the anchor with respect to the roof height. The 2003 IBC does include the material load factors that are used in the 1997 UBC, but otherwise the equation remains unchanged.

Base Shear: Early versions of the base shear formula in the UBC took into account, in addition to the weight, varying factors such as allowable soil pressure (1937), and number of stories above the considered level (1949). The 1961 UBC introduced a formula that accounted for building type and height. This formula increased base shear 20–30% for tilt-ups. The 1976 UBC increased the base shear an additional 40%. This formula took into account the soil conditions as well as the importance of the building. The base shear formula remained basically unchanged until the 1997 UBC in which two equations were introduced that took into account not only soil conditions but also near-source factors.

The 2000 IBC base shear equation is a simple equation with only two variables. The equation does not take into account soil condition and near-fault effects. Base shear in the 2003 IBC has changed slightly. Base shear can be determined by either using the base shear formula of ASCE 7, which is essentially the same formula as the 2000 IBC, or by a simplified method which does not take into account soil effects, near-fault effects, or building importance.

Miscellaneous: The 1973 UBC stated, "wood ledgers shall not be used in cross-grain bending," and the 1976 UBC prohibited cross-grain tension. The 1973 UBC also added tie requirements at the top of pilasters, where cracking can occur. In the 1976 UBC the subdiaphragm concept was introduced due to complaints about the expense of previous UBC versions requiring continuous cross ties across the width of the building. The 1991 UBC included provisions for detailing narrow piers more like frames. At this time tilt-ups were transitioning from large warehouse structures with solid walls to office structures with larger and more frequent opening in the panels. These office buildings behaved more like frames, and needed to be detailed as such.

The guideline also used in this document, FEMA 356, has also changed since its inception. FEMA 356 is the result of a progression of studies over the last 20 years that somewhat parallels the UBC development. The beginnings of FEMA 356 were in its immediate predecessor, FEMA 273, and in ATC 14 and ATC 20. The changes in these documents are not as clear-cut as the code changes, so no summary is provided.

2 Report Synopses

Below are brief synopses of the four reports reviewed for PEER Task 509. More detailed summaries of the reports can be found in the appendices.

2.1 *SEISMIC PERFORMANCE OF TILT-UP BUILDINGS* BY JOHN F. HALL

The report by John Hall studied the behavior of concrete tilt-up buildings by the use of a nonlinear 3D model and two simple nonlinear 2D "shear beam" models for each direction of the building. The example building is similar to the PG&E Meter Repair Facility in Fremont, California. Roof-to-wall connections and seat connections of the glulam beams (GLB) were included in the models to predict connection demands. Several analysis iterations of the 3D model were conducted with varying connection strength and input ground motions, to determine the sensitivity of the results to variations to the inputs.

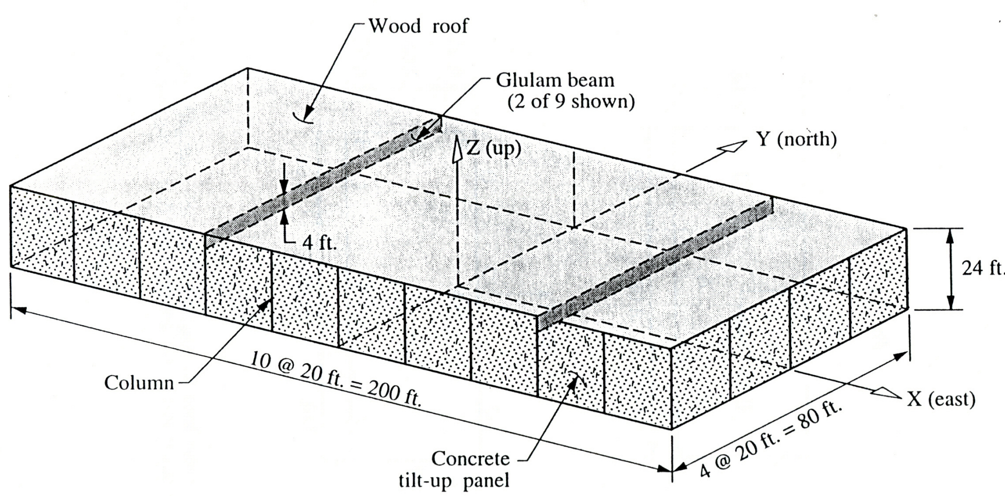


Figure 1. Computer model layout

The results from the analytical testing indicated connection demands might be increased by frame action between the glulam beams and the walls. (Note: GLBs used in the model were approximately 4 ft deep) They also indicated that the 2D shear beam model does a reasonable job of predicting overall building behavior, but does not predict connection demands well.

2.2 *BUILDING VULNERABILITY STUDIES: MODELING AND EVALUATION OF TILT-UP AND STEEL REINFORCED CONCRETE BUILDINGS* BY JOHN W. WALLACE, JONATHAN P. STEWART, ANDREW S. WHITTAKER

The report by John Wallace, Jonathan Stewart, and Andrew Whittaker studied the behavior of concrete tilt-up buildings by the use of a nonlinear 2D model analyzed using the program DRAIN 2DX. The report also offers an overview of previous experimental tests and results by other researchers. Most of the tests focused on wood diaphragm testing. The results obtained from these previous studies are used as input and guidelines for the nonlinear model. The model was constructed of three individual 2D models, each representing a particular aspect of the building. The three models consisted of one model representing the diaphragm, one representing the internal frame of the longitudinal walls, and one the transverse walls.

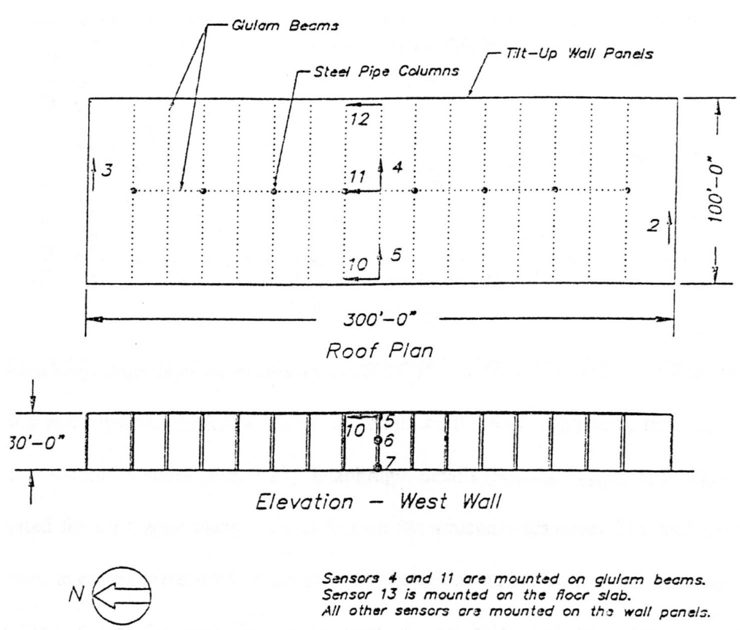


Figure 2. Hollister building layout

To test the validity of the modeling methodology, the properties were set to replicate the conditions of the PG&E building in Hollister. The Hollister building contains strong motion instruments, and the building response has been recorded in several earthquakes. The model adequately predicted overall building behavior for these earthquakes. The model was also used to test the impact of each building element on overall performance. These runs indicated that diaphragm stiffness and near-field motions have a considerable influence on building response.

2.3 *STIFFNESS OF TIMBER DIAPHRAGMS AND STRENGTH OF TIMBER CONNECTIONS* BY GERARD C. PARDOEN, DANIEL DEL CARLO, ROBERT P. KAZANJY

This report studied the behavior of concrete tilt-up buildings by the use of physical models. The tests were not concerned with overall building performance, but rather the performance of individual components of the structure. The components tested were the diaphragm, GLB to pilaster connections, purlin to wall connections, and subpurlin to wall connections. Test specimens were made to replicate "old" (original to early 1970's) construction, and "new" (current to mid-to-late 1980's) construction.[1] (See Degenkolb Note below) Some old connections, especially purlin and subpurlin to wall connections, are not presently considered to be connections at all. Many old connections relied on cross-grain bending, which is not recommended by today's standards.

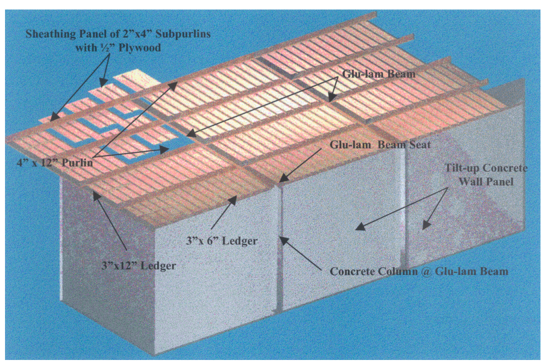


Figure 3. Components of tilt-up

The testing showed that more closely spaced subpurlins in a diaphragm do not contribute much to strength and that new GLB connections are generally stronger than their old counterparts. However, the old GLB to wall connections studied may not be as strong as new connections, but they may be more ductile. New connections for purlins and subpurlins are much better than the old connections, since the old connections relied solely on cross-grain bending, which is considered zero strength.

[1] The use of "old" and "new" connections in this report is misleading. Both old and new connections are pre-1997 and do not reflect current construction. New connections are more reflective of mid-1980's construction.

2.4 *SEISMIC PERFORMANCE OF AN INSTRUMENTED TILT-UP BUILDING* BY JAMES C. ANDERSON AND VITELMO V. BERTERO

The report by Anderson and Bertero studied the behavior of concrete tilt-up buildings by the use of linear and nonlinear 3D models analyzed using the commercial program SAP2000. Data used for comparison were taken from an instrumented building, constructed in the early '70s in Redlands, California, that was subjected to ground motions in four past earthquakes, with some remedial work performed after one of the stronger earthquakes. Wall anchor connections and soil properties were included in both models by the use of linear springs. Nonlinear connections were modeled with the use of the Pardoen test data, and analyses were performed with either old or new connections. To model the nonlinear behavior of the diaphragm, a continuum of truss elements with nonlinear properties had to be used.

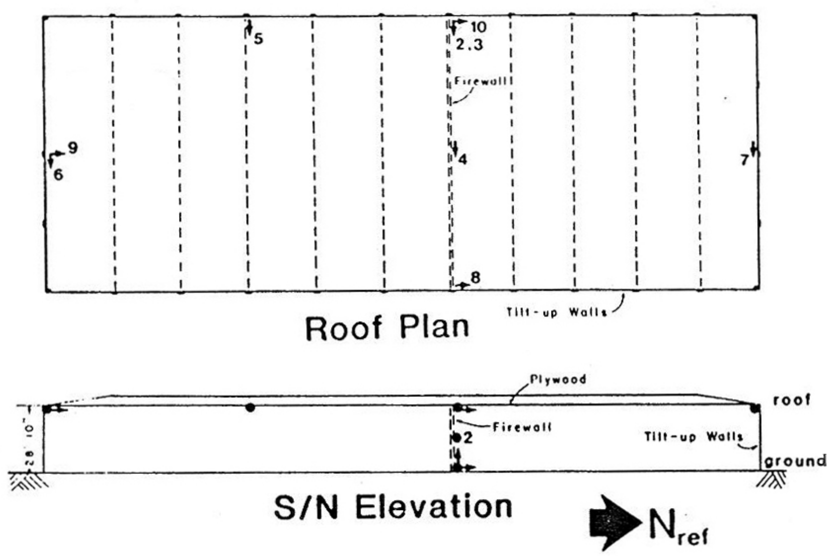


Figure 4. Redlands building layout and sensor locations

Several runs of the model were done, using the ground motions to which the Redlands building was subjected. Other runs were also made using pulse-type ground motions to investigate near-field effects. The models predicted the behavior of the building quite well. The runs indicated that nonlinear behavior in the structure could have a significant effect on the force and displacement demands of the components of the building.

3 Evaluation of Reports

The following sections have been produced by Degenkolb Engineers to compare the results and conclusions obtained in the reports to current documents used in design and analysis. The information is not the work of the original authors, but our interpretation of the data.

3.1 HALL REPORT

Some important aspects of the Hall report were the analytically determined out-of-plane anchorage loads, the building periods, and the results concerning frame action in the transverse direction. The anchorage loads and the period values were taken from the report and compared to code design values. The effect of frame action on design is also discussed.

The maximum wall anchor loads analytically determined from the Sylmar-Northridge ground motion response in the Hall report at the north and south walls (corresponding to a ground motion in the transverse direction of the building) are slightly lower than the 2000 IBC and FEMA 356 values for the design anchor force. However, the analytically determined wall anchor loads at the east and west walls (corresponding to a ground motion in the longitudinal direction of the building) are much higher than the 2000 IBC and FEMA 356 calculated values. The 1997 UBC provides a higher value for anchor loads in both directions, due to the material factor used for steel. Note that FEMA 356 has different values for the two performance objectives, Life Safety (LS) and Collapse Prevention (CP). The CP values for wall anchor loads in the east-west walls are closer to the analytically determined values than the IBC standard value. The 2000 IBC and FEMA 356 are fairly accurate in predicting wall anchorage forces for motion in the transverse direction of the building, but are unconservative in predicting wall anchorage loads for motion in the longitudinal direction. The difference may be due to the diaphragm remaining elastic during shaking in the longitudinal direction due to the diaphragm overstrength. The 1997 UBC is conservative in both directions.

Note: The Sylmar ground motion is a more severe ground motion than either the prescribed code spectrum or the spectrum generated by FEMA 356. Since only one, very strong ground motion was used, the comparison results do not necessarily reflect behavior that will occur in all buildings. In order to make a more detailed assessment, more data are needed.

Table 1A. Analytically determined and FEMA 356 wall anchor loads

	Max. Analyt. Det. (lb/ft)	FEMA 356 LS (lb/ft)	FEMA 356 / Analyt. Det.	FEMA 356 CP (lb/ft)	FEMA 356 / Analyt. Det.
N & S Walls	800	825	1.03	930	1.16
E & W Walls	1150	825	0.72	930	0.81

Table 1B. Analytically determined 1997 UBC and 2000 IBC wall anchor loads

	Max. Analyt. Det. (lb/ft)	2000 IBC (lb/ft)	2000 IBC / Analyt. Det.	1997 UBC (lb/ft)	1997 UBC / Analyt. Det.
N & S Walls	800	825	1.03	1390	1.74
E & W Walls	1150	825	0.72	1390	1.21

Tables 1A and 1B Notes:

- FEMA 356 forces are calculated from Equation 2-6 from Section 2.6.7.1 — out-of-Plane Anchorage to Diaphragms, with a minimum of 400 lb/ft or 400 S_{XS}.
- 2000 IBC forces are calculated from Equation 16-64 from Section 1620.2.1 — anchorage of concrete or masonry walls, with a minimum load of 200 lb/ft per 1604.8.2.
- 1997 UBC forces are calculated per the requirements of Section 1633.2.8.1, with a minimum unfactored load of 420 lb/ft for high seismic areas.
- 1997 UBC values include a 1.4 increase for the strength design forces for steel elements of the anchorage system.

The building periods for the PG&E Fremont building determined in the report are shorter than those calculated using the FEMA 356 approximate building period equation (Eq. 3-8) or using dynamics equations. (See Appendix E for a detailed calculation.) The yield value that Hall assumed in his model does not correlate well with the working stress value he has provided. Appendix E also contains an explanation of how a different diaphragm yield was chosen for the FEMA 356 period calculation. A period calculated directly from dynamics is slightly shorter than the FEMA 356 period, but the results are close. Using either the FEMA 356 or the dynamics method results in a period that is longer than the analytically determined period, and may cause some unconservatism in design. A longer period may cause the acceleration to be lower, since the period is farther down the spectrum, and the forces will also be lower. Current design practice assumes that the period is on the plateau of the design spectrum, resulting in the maximum acceleration of the response spectrum. It appears from this model that using either

FEMA 356 or dynamics for period approximation is not ideal, and computer models may yield the best period approximations.

The 1997 UBC or the 2000 IBC period equation does not consider the effect of the diaphragm at all. Because of this these codes do not offer accurate period values and were not compared in the table.

Table 2. Building periods in the transverse (north-south) direction

Analytically Determined Period (s)		FEMA 356 Period (s)	Period from Dynamics (s)	FEMA 356 / Analytically Determined		Dynamics / Analytically Determined	
Low	High			Low	High	Low	High
0.67	0.70	1.18	1.12	1.76	1.69	1.67	1.60

Table 2 Notes:

- The FEMA 356 period is calculated using Method 3 — Approximate (Eq. 3-8) for estimating a structure's period.
- The period from dynamics is calculated by assuming that the diaphragm acts as an oscillating, simply supported beam of uniform mass and stiffness.
- A sample calculation using each method, using Equation 3-8 of FEMA 356, and using dynamics equations can be found in Appendix E.
- One conclusion by Hall that is not normally considered in design is the effect of frame action on connection demands. Hall stated that frame action between the GLBs and the supporting walls can lead to frame action in the connection and increase demands.

Practicing engineers do not usually consider frame action in design. In current construction GLBs are used less and steel trusses are becoming more common. If GLBs are used, they are generally set into a steel bracket in the wall and the bottom of the GLB is bolted into the bracket. The connection at the top to the roof is limited to nailing, and does not provide a strong connection. Thus, the connection has little to no moment capacity, and frame action is not accounted for.

In existing buildings frame action may be a problem, since there is a connection at both the top and bottom of the GLB, and because the Hall report suggests that the cause of the frame action is due to the connection at the base of the GLB to pilaster. According to Hall's results removing this connection will eliminate the frame action. Removing this connection is done in retrofit schemes by removing the bolts from the GLB seat. However, this is not done to prevent increased forces due to frame action but to prevent pilaster failure. Many observed failures of the GLB connection have occurred in the pilaster underneath the GLB seat due to lateral load being transferred through the seat to the pilaster. By removing the bolts, the lateral demand is removed from the pilaster and the connection is not as stiff, so the demand is not as large. As a result, the problem of increased demand due to frame action is taken care of indirectly in some building retrofits, although it is not the intended purpose.

Some limitations of the report are:

- In the initial model, the diaphragm strengths and connections were based on the 1994 UBC requirements. The increased 1997 UBC requirements were not investigated and may provide better performance in the model.
- Connection strengths in the model of the PG&E Fremont building were based on test data from the UC Irvine (UCI) tests (Pardoen Report). The UCI test data used were based on a limited number of tests and may not be indicative of average strengths of connections or connections commonly used.
- The software used for analysis was written specifically for this report, but may not be applicable for all cases.
- The 2D model provides good global performance, but demands in the connections cannot be accurately estimated.

3.2 WALLACE ET AL. REPORT

The Wallace report did not present much of its data in tabulated or graphical form. Because the data were not easily available for comparison to code values, no direct comparisons could be made to code requirements.

The review of previous experimental studies conducted by other researcherspresented in the report did not note that any major changes in tilt-up design have been made due to those studies. Limited information was known about the design of the Hollister warehouse building that was modeled in the report. Because of the lack of information, the properties of the building components were assumed to be similar to the PG&E Fremont building. The connections used in the Fremont building were replicated and tested in the Pardoen report and used as input for this one. The diaphragm strengths were based on an equation created in a previous report by Hamburger (Hamburger et al. 1996) that is highly dependent on using a control specimen to normalize the equation. The Pardoen results were used to normalize the equation, and any potential concerns with the results of the Pardoen tests will carry over to the results of the computer model.

The strength of the Wallace report lies in the conclusions that are drawn on the important aspects in tilt-up modeling. A sensitivity study was completed with the model to show what aspects of the building most affected total performance. From this study, it was shown that two of the most important inputs for building performance are the diaphragm stiffness and the ground motion inputs. Near-field results motions were shown to have a significant effect on diaphragm response.

Some limitations of the report are:

- The 2D model provides good global performance, but demands in the connections cannot be accurately estimated.
- The model does not capture the increase in damping due to nonlinear behavior.

3.3 PARDOEN ET AL. REPORT

Of the two focuses of the Pardoen report, the diaphragm testing will be evaluated more closely than the connections. Connection strengths directly depend on the anchor used, which will differ from building to building. Only two types of connections were tested in each connection test, and this small sample set was thought to be insufficient to develop major conclusions on how design should be done. Some of the connection tests are also atypical and thus not indicative of common practice.

In the report Pardoen stated "the estimated yield loads for all six diaphragms exceeded the '97 UBC loads by 27% – 56%." The load capacities given in the 1997 UBC are allowable values and are not a clear indication of the actual strength. A more direct comparison would be the FEMA 356 yield loads, which estimate the expected yield strength. Table 4 shows FEMA 356 expected yield loads are all approximately 40% – 60% higher than yield loads determined by testing. However, these values are dependent on the original author's choice of a yield point. The yield values have been taken directly from the Pardoen report and no clear indication is given of how these values were chosen. Table 5, the ultimate strength comparison, shows much better correlation between the experimentally determined values and the values obtained from FEMA 356.

The diaphragm panels that were tested were relatively small compared to the diaphragm of an actual tilt-up building. The panels were also set-up and tested differently from common diaphragm assumptions in a tilt-up building. The panels were attached to the floor and cantilevered up. An actuator then applied load cyclically at the top (Figure 1). The way the test panels were set up, they more closely resembled cantilevered shear walls. Common assumptions in diaphragm design are: (1) The diaphragm is simply supported at both ends and spans laterally as a deep beam (2) Lateral forces are applied as a distributed load along the length of the diaphragm. Neither of these assumptions was used in the test set-up and may have affected the results and made them difficult to compare to diaphragm results obtained from code procedures. Since the test setup appears more like a shear wall, the results were also compared to the values obtained from code procedures for a shear wall.

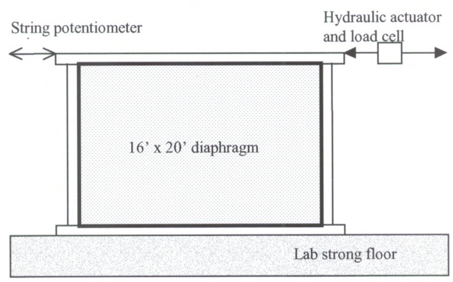


Figure 5. Test setup

Table 3. Test panel summary

	Panel size	Sub-purlin	Nails	Sheathing	E.N.	Cont. E.N.
RD1	20 x 16	2x4 @ 16"	10d	½"	6" o.c.	6" o.c.
RD2	16 x 20	2x4 @ 16"	10d	½"	6" o.c.	6" o.c.
RD4	20 x 16	2x4 @ 16"	10d	½"	6" o.c.	6" o.c.
RD5	20 x 16	2x4 @ 24"	10d	½"	6" o.c.	6" o.c.
RD3	20 x 16	2x4 @ 16"	10d	½"	3" o.c.	2" o.c.
RD6	20 x 16	2x4 @ 24"	10d	½"	3" o.c.	2" o.c.

Table 4A. Test yield results and published allowable / yield values per diaphragm requirements

	Exp. Det. Yield (lb/ft)	FEMA 356 Q_{CE} (lb/ft)	FEMA 356/ Exp. Det.	LRFD (lb/ft)	LRFD / Exp. Det.	1997 UBC & 2000 IBC (lb/ft)	UBC & IBC / Exp. Det.
RD1	406	646	1.59	420	1.03	320	0.79
RD2	500	646	1.29	420	0.84	320	0.64
RD4	406	646	1.59	420	1.03	320	0.79
RD5	469	646	1.38	420	0.90	320	0.68
RD3 *	938	1277 / 1462	1.36 / 1.56	830 / 950	0.88 / 1.01	640 / 730	0.68 / 0.78
RD6 *	938	1277 / 1462	1.36 / 1.56	830 / 950	0.88 / 1.01	640 / 730	0.68 / 0.78

* The two numbers, Number A / Number B, in these rows are explained below.

For diaphragms where 10d nails at adjoining panel edges are at 3″ o.c., 3x framing is required. 2x framing was provided for the subpurlins, so the full strength for panel edge nails at 3″ o.c. was not used, and the strength for nails at 4″ o.c. was used (Number A). However, the documents used provide a strength for panels where 2x framing is used for 10d nails at 3″ o.c.; that value is also noted (Number B).

Table 4B. Test yield results and published allowable / yield values per shear wall requirements

	Exp. Det. Yield (lb/ft)	FEMA 356 Q_{CE} (lb/ft)	FEMA 356 / Exp. Det.	LRFD (lb/ft)	LRFD / Exp. Det.	1997 UBC & 2000 IBC (lb/ft)	UBC & IBC / Exp. Det.
RD1	406	542	1.33	440	1.08	340	0.84
RD2	500	542	1.08	440	0.88	340	0.68
RD4	406	542	1.33	440	1.08	340	0.84
RD5	469	542	1.16	440	0.94	340	0.72
RD3 *	938	812 / 1058	0.87 / 1.13	660 / 860	0.70 / 0.92	510 / 665	0.54 / 0.71
RD6 *	938	812 / 1058	0.87 / 1.13	660 / 860	0.70 / 0.92	510 / 665	0.54 / 0.71

* The two numbers, Number A / Number B, in these rows are explained below.

For shear walls where 10d nails at adjoining panel edges are at 3″ o.c., 3x framing is required. 2x framing was provided for the subpurlins,. so the full strength for panel edge nails at 3″ o.c. was not used, and the strength for nails at 4″ o.c. was used. (Number A). However 3x framing was assumed to be provided at the boundaries and continuous panel edges, and if 3x framing is provided at the other framing members the second value will be obtained. (Number B).

Notes for Tables 4A and 4B:

- It was assumed that nailing at 2″ o.c. was provided around the panel boundaries of RD3 and RD6.
- The IBC diaphragm values taken from Table 2306.3.1 of 2000 IBC, the IBC shear wall values taken from Table 2306.4.1 of 2000 IBC, and both are "recommended shears."
- The UBC diaphragm values taken from Table 23-II-H of 1997 UBC, the UBC Shear Wall values taken from Table 23-II-I-1 of 1997 UBC both are "allowable shears."
- The IBC and UBC shear values are identical to those in values from the "Structural-Use Panel Shear Wall and Diaphragm Supplement" from the *ASD Manual for Engineered Wood Construction.*
- The FEMA 356 diaphragm values are taken from the "Structural-Use Panels Supplement," Table 5.5 of the *LRFD Manual for Engineered Wood Construction,* and modified per FEMA 356.
  - Example: Panel RD1 per LRFD has a "factored shear resistance" of 0.42 k/ft; FEMA 356 states ϕ is to be taken as unity to determine the yield (Q_{CE}). The values obtained from the LRFD document already contain $\phi = 0.65$, so the 0.65 must be divided out.
    $\therefore$ 420 lb/ft / 0.65 = 646 lb/ft
- The FEMA 356 shear wall values are taken from the "Structural-Use Panels Supplement," Table 5.4 of the *LRFD Manual for Engineered Wood Construction,* and modified per FEMA 356.
  - Example: Panel RD1 per LRFD has a "factored shear resistance" of 0.44 k/ft; FEMA 356 states ϕ is to be taken as unity and the value to be multiplied by 0.8 for plywood to determine the yield (Q_{CE}). The values obtained from the LRFD document already contain $\phi = 0.65$, so the 0.65 must be divided out.
    $\therefore$ 440 lb/ft / 0.65 *0.8= 542 lb/ft

Table 5A. Test ultimate results and FEMA 356 ultimate values per diaphragm requirements

	Load (lb/ft)		
	Test Ultimate	FEMA 356 Ultimate	FEMA 356/ Exp. Determined
RD1	1000	969	0.97
RD2	1100	969	0.88
RD4	1250	969	0.78
RD5	719	969	1.35
RD3	1625	1916 / 2192	1.18 / 1.35
RD6	1500	1916 / 2192	1.28 / 1.46

Table 5B. Test ultimate results and FEMA 356 ultimate values per shear wall requirements

	Load (lb/ft)		
	Test Ultimate	FEMA 356 Ultimate	FEMA 356/ Exp. Determined
RD1	1000	813	0.81
RD2	1100	813	0.74
RD4	1250	813	0.65
RD5	719	813	1.13
RD3	1625	1218 / 1587	0.75 / 0.98
RD6	1500	1218 / 1587	0.81 / 1.06

Notes for Table 5A&B:

- Ultimate Load per FEMA 356 is 1.5 * Yield (Q_{CE})

Table 6. Ultimate to yield ratios for diaphragm and shear wall requirements

	Exp. Ult. / Exp. Yield	FEMA 356 Ult. / FEMA 356 Yield
RD1	2.46	1.50
RD2	2.20	1.50
RD4	3.08	1.50
RD5	1.53	1.50
RD3	1.73	1.50
RD6	1.60	1.50

The diaphragm panels were tested cyclically and a backbone curve was created from the hysteretic data. By using FEMA 356 recommendations from Section 8.4 and modeling parameters from Table 8-4, a code backbone curve was created and overlaid using both the diaphragm and shear wall code procedures. Figures 1 and 2 show that the FEMA 356 backbone for a diaphragm is slightly unconservative when compared to the test data. FEMA 356 estimates a higher diaphragm capacity than the test results indicate. Again, this fact may be due to the test setup and loading of the test panels. However for panel RD3, using a lower diaphragm yield strength due to the presence of 2x framing (See note below Table 4A), the FEMA 356 diaphragm backbone closely follows the experimentally determined backbone. The yield points and ultimate points are still higher than those determined in the experiment, but the stiffnesses are similar.

When a backbone is created using FEMA 356 for a cantilevered shear wall panel the results are more conservative for yield strengths, but the deformation capabilities seem a bit larger than those indicated by the tests.

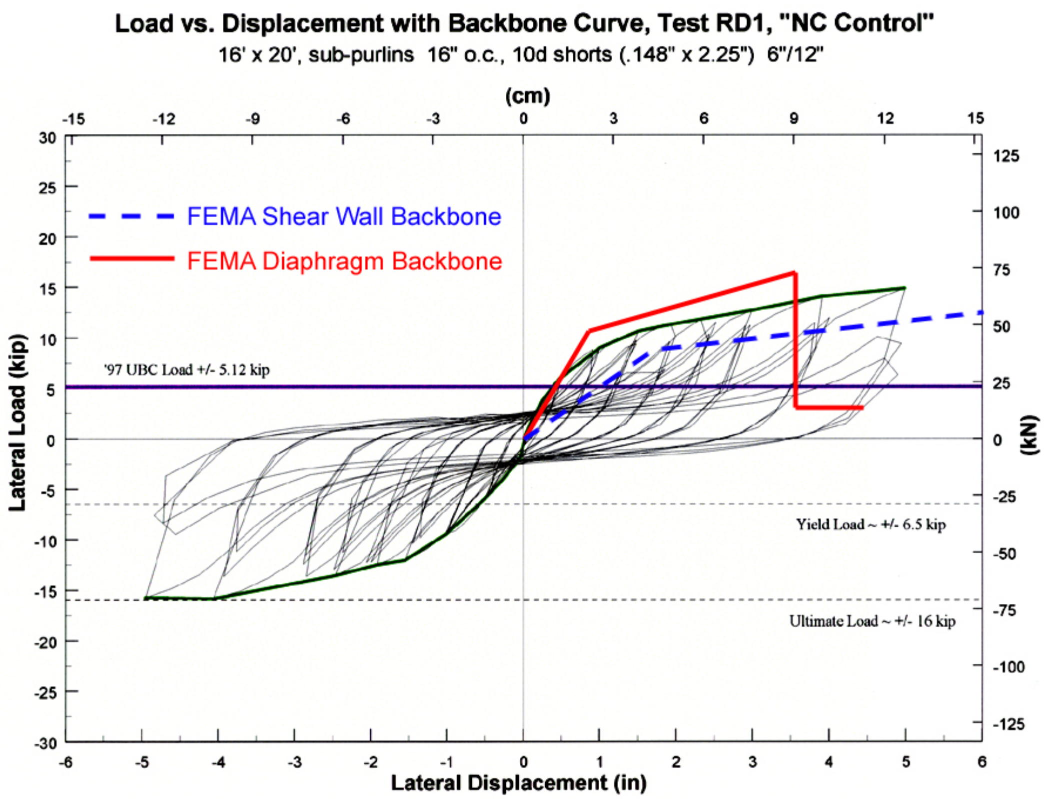


Figure 6. Backbones for panel RD1

Table 7. Backbone points for panel RD1

		Yield	Ultimate	Residual
Experimentally Determined	V	6.5 k	16 k	Not tested
	Δ	0.6 in.	4.1 in.	Not tested
FEMA 356 Diaphragm Requirements	V	10.3 k	15.5 k	3.1 k
	Δ	0.89 in.	3.56 in.	4.45 in.
FEMA 356 Shear Wall Requirements	V	8.6 k	13.0 k	2.6 k
	Δ	1.67 in.	6.84 in.	8.51 in.

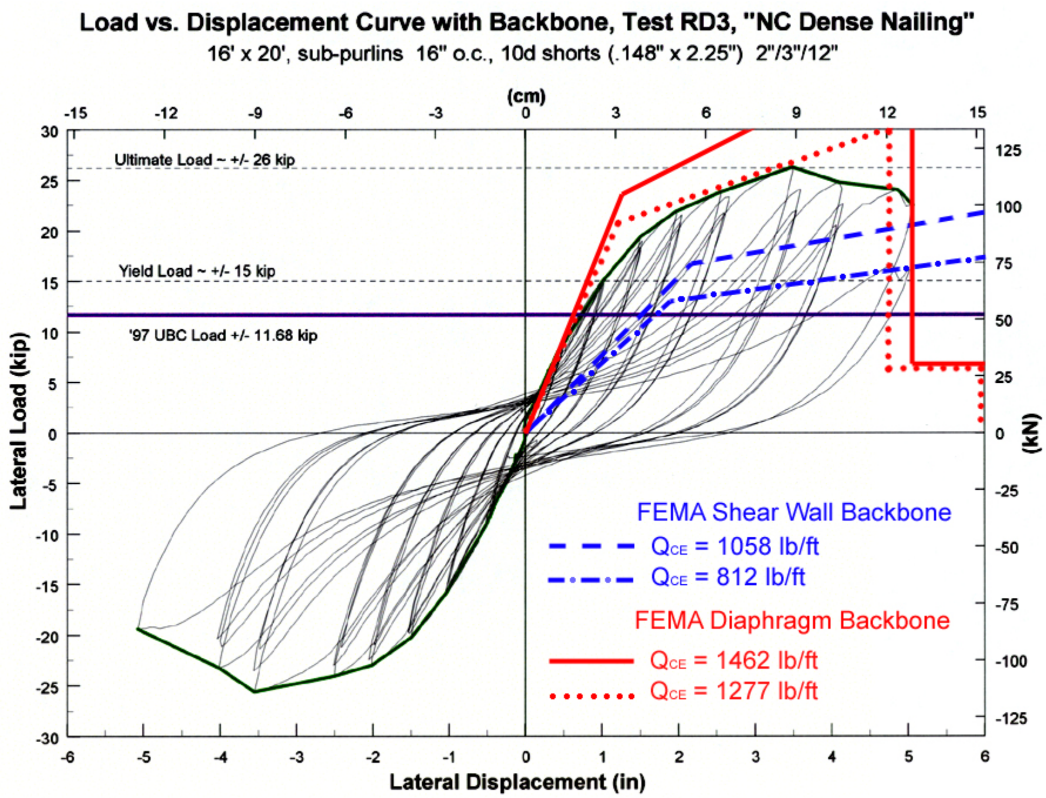


Figure 7. Backbones for panel RD3

Table 8. Backbone points for panel RD3

		Yield	Ultimate	Residual
Experimentally Determined	V	15 k	26 k	Not tested
	Δ	1.0 in.	3.5 in.	Not tested
FEMA 356 Diaphragm Requirements	V	20.4 k / 23.4 k	30.6 k / 35.1 k	6.1 k / 7.0 k
	Δ	1.18 in. / 1.26 in.	4.72 in. / 5.04 in.	5.90 in. / 6.30 in.
FEMA 356 Shear Wall Requirements	V	13.0 k / 16.9 k	19.5 k / 25.4 k	3.9 k / 5.1 k
	Δ	1.91 in. / 2.12 in.	7.83 in. / 8.69 in.	9.74 in. / 10.81 in.

Notes for Tables 7 and 8:

- The experimentally determined yield and ultimate displacements are visually approximated from the backbone curves obtained from testing.

Both the strength comparison and backbone comparison do not give a clear indication of the prediction capabilities of the codes. There is not a clear correlation between the test data and the code values. The tests were not necessarily set up according to the assumptions used in design and this may have caused some discrepancies in the results.

Connection tests have poor correlation to the values published by the connection manufacturer, Simpson. For the GLB seats, Simpson does not publish a connection strength for the entire connection subject to horizontal loading, but only the allowable horizontal bolt loads. Failures in the field have shown that the weak point is the connection of the seat to the pilaster. This is not reflected in either the testing in the Pardoen report or in the Simpson values. For the purlin and subpurlin anchors, the values published for the connection are for that strap alone, based on the allowable strength of the nail group. The tests conducted included other connections as well. The purlin test with the PA-18 strap also included two angles that increased the strength of the connection. The subpurlin tests with the PAT-18 strap included a joist hanger that would provide an additional, but quite small, amount of strength to the connection.

Table 9. Connection strengths and manufacturer values

	Anchor Name	Avg. Exp. Det. Ultimate (k)	Simpson Allowable (k)
GLB Seat	GLB-5A	31.3	No value given
	GLB-512**	36.3	8.260
Purlin & Subpurlin Anchors	PA-18	20	1.690 *
	PAT-18	7.5	0.985 *

Notes for Table 9:
* Simpson allowable loads have been calculated by dividing the 33% or 60% increase for seismic or wind from the published values.
** According to author a GLB-512 connection was used, however no Simpson anchor can be found with that name. From pictures in the report it appears that a Simpson GLBT512 was used.

Some limitations of the results are:

- A small sample set of specimens was tested, and the repeatability of the tests cannot be verified.
- Connection tests were lacking cyclic testing data as well as shear strength of the connections.
- There was a lack of documentation (photographs, detailed written descriptions, etc.) for the failure modes, especially in the glulam beam to pilaster tests.
- The small-size specimens may not accurately depict behavior of much larger diaphragms in actual buildings.

3.4 ANDERSON ET AL. REPORT

The focus of the Anderson report was on varying the properties in a model created in SAP 2000. The wide range of data available in the report makes it difficult to compare all the experimentally determined data with code values.

Ductility demands were reported often in the report, and were usually somewhat high. Ductility in anchors is something that cannot and should not be relied upon. In design, the wall anchors need to remain elastic.

The building period was easily extracted from the report data and could be compared to code values. The building that was used as the basis for the SAP model was instrumented and the response of the building was recorded for several earthquakes. From the recorded response the Redlands building period was calculated in one direction. The SAP model also produced periods for the building. The range of the period values is wide due to the varying properties in the different models. The fire wall in the building was chosen not to be included in the SAP model, so results in the east-west direction may be affected by its absence.

In this comparison periods recorded from the building are available, so the comparison has a basis in reality. Like the period comparison in Hall both the FEMA 356 and dynamics periods are larger than the periods given in the report. The FEMA 356 period and the dynamics period are very close. The FEMA 356 and dynamics periods in the north-south direction correlated more closely than the period in the east-west direction. The much larger FEMA 356 and dynamics periods in the east-west direction could cause design forces to be underestimated. The FEMA 356 or dynamics period would be farther out on the design spectrum and lead to lower accelerations. Assuming that the period lies on the plateau of the spectrum would lead to a conservative result. In the calculation of the FEMA 356 period and the dynamics period, the fire wall was neglected. This will make the period longer in the east-west direction than if the fire wall was considered to split the diaphragm into two spans. More data and a more detailed look at how the fire wall affects building behavior are needed to decide if the FEMA 356 or dynamics equation will always be higher than that derived from an analytical model.

Table 10. Building periods

	Recorded Period (s)		Analytically Determined Period (s)		FEMA 356 Period (s)	Period from Dynamics (s)
	Min	Max	Min	Max		
N-S Direction	0.33	0.75	0.39	0.46	0.64	0.61
E-W Direction	0.40	0.67	0.29	0.42	1.89	2.01

Table 11. Ratio of FEMA 356 period values to recorded and analytically determined building periods

	FEMA 356 / Recorded		FEMA 356 / Report Analytically Determined	
	Minimum	Maximum	Minimum	Maximum
N-S Direction	1.94	0.85	1.64	1.39
E-W Direction	4.73	2.82	6.52	4.50

Notes for Tables 10 and 11:

- The recorded period is derived from seismic data taken from accelerometers installed in the building.
- The FEMA 356 period is calculated using Method 3 — Approximate (Eq. 3-8) for estimating a structure's period.
- The period from dynamics is calculated by assuming the diaphragm acts as an oscillating, simply supported beam of uniform mass and stiffness.
- A sample calculation using each method, using Equation 3-8 of FEMA 356 and using dynamics equations, can be found in Appendix E.

Table 12. Ratio of dynamics period values to recorded and analytically determined building periods

	Dynamics / Recorded		Dynamics / Report Analytically Determined	
	Minimum	Maximum	Minimum	Maximum
N-S Direction	1.85	0.81	1.56	1.33
E-W Direction	5.03	3.00	6.93	4.79

The report also stated that at GLB to pilaster connections, forces can be as much as three times higher at the middle of the diaphragms than at the ends, and that the code, which uses a uniform load, will be conservative at the ends and much less conservative in the middle. This effect was taken into account when writing the code prevision. The uniform out-of-plane load is based on the acceleration of the middle of diaphragm, so a non-uniform out-of-plane load was recognized; however, a uniform distributed load is much easier to calculate and design for.

Some limitations of the results are:

- The inelastic diaphragm model was derived from the results in Pardoen's report. These results may not be indicative of larger diaphragm strengths, and may not be applicable in the SAP model.
- The inelastic connection properties were also derived from the Pardoen report. The values obtained from the report are not for cyclic testing, and the results may not be valid for use in a seismic model.
- The effect of the fire wall on building performance was not investigated in the report. It appears that it has some effect on the structure and may cause the results to vary.

4 Questions Raised by Review of the Reports

Some aspects of the reports were not entirely clear, and further explanation is necessary based on a list of questions below for the authors.

4.1 HALL REPORT

- Connections in the buildings were modeled with a connection both at the top and the bottom of the glulam beam. Looking at current connections used in typical construction, this assumption does not appear to be correct. Some connections may involve a connection at both the top and the bottom, but this may not provide sufficient strength for frame action. Why were connections placed at both the top and bottom of the GLB in the model?
- Where did the ultimate displacements used for the connections in models D1, D2, and D3 come from?

4.2 WALLACE ET AL. REPORT

- Will more testing help improve the Hamburger equation mentioned in the report? Can it be made more general for use in design?
- The wall anchors were assumed to be rigid in compression. This assumes there is no gap at the end of the member. Why was the gap not included?
- What was the PGA for the Loma Prieta ground motion used?

4.3 PARDOEN ET AL. REPORT

- In the diaphragm specimens that were made, it is not clear what the size of the boundary timber members and the members along the continuous joint were. What size were they?

- In current codes or guidelines when using nailing at 2″ o.c. for the continuous joint or using 10d nails, 3x framing is required. From the report it appears that only 2x framing was provided in the interior and an unknown framing size was used along the boundaries. Was the effect of framing size considered in the analysis?
- Most failures seen in the field concerning GLB to pilaster connections occur in the pilaster and not in the GLB. The failures are due to insufficient confinement or edge distances in the pilaster. Why wasn't this failure mode investigated?
- The failure mode in the purlin-to-wall connections where the plywood was spliced is not a failure mode that is seen in the field. Could the method of load application have influenced this failure mode? Was the load applied by pulling on the plywood, or were the purlins pulled?
- It is unclear from the report where the failures occurred. Better documentation, written and photographic, would be helpful.

4.4 ANDERSON ET AL. REPORT

- The presence of damage in the connections is an important aspect in determining the previous performance of the building in an earthquake. Is it possible to re-visit the Redlands site and do a more thorough examination of the wall anchorage connections, inspect the diaphragm for damage, inspect the fire wall for damage, and document any retrofit or repair work done in the building? More detail is needed on the existing construction to do a full evaluation of the building.
- Why was the fire wall in the Redlands building not included in the computer model? It appears to have had a significant effect on the performance of the building. If connection details were not available, assumptions could be made and differing models could be used to determine how the wall will affect performance.
- In the conclusions, item 5 notes a 65% reduction in stiffness of the Redlands building after the Big Bear earthquake. Could this be from loosening or yielding of the nails only?
- Why is horizontal shear in the wall anchors an issue? Shouldn't ledger bolts be taking most of the horizontal shear from the diaphragm into the wall?
- The issue of mass participation is perplexing. The base shear in one direction should not be so different from the base shear in the other direction. Was this investigated further

with simpler models to verify that it is correct? It also seems there are a lot of extra modes of the model that are not of interest, could these items be related?

- What was the PGA for the Lucerne, Takatori, and Los Gatos ground motions used in the pulse analyses?

5 Suggestions for Further Research

While the reports have addressed many of the issues about the seismic performance of tilt-up buildings, they have also created a number of questions that could use further investigation. The following lists suggestions for further research in tilt-up building performance.

5.1 HALL REPORT

- The original calculations of the capacity of the anchorage and strength of the diaphragm for the models were developed using the 1994 UBC requirements; changes in the code since then should lead to better seismic performance. Rerunning the analysis of the building using the 1997 UBC and the 2003 IBC capacities would better reflect performance of tilt-ups constructed to meet current codes.
- Based on the results of this analysis, the forces obtained from codes or guidelines for wall anchors are still too low on the short side of the building. Further investigation into what code forces would be appropriate for each direction of the building is needed.

5.2 WALLACE ET AL. REPORT

- Rerun the analysis of the Hollister building without the additional 1.33 increase in stiffness of the diaphragm because of the presence of roofing materials. A comparison of forces used in design to forces experienced in the building would be helpful.

5.3 PARDOEN ET AL. REPORT

- All connection tests performed were pull tests. In order to gain a better knowledge of the connection performance seismically, cyclic tests should be performed on all components.
- More tests are needed for all components to verify the repeatability.

- According to computer models created in other reports, the frame effect of the GLB to the pilasters places additional demand on the connections. Testing to see if this frame action can actually be developed in a connection could determine its effect on design. If frame action is a problem an improved connection should also be tested.
- Efforts in connection testing should be done in conjunction with the vendors. Connection strengths published by the vendors do not always reflect the strength of the whole connection, but rather just an aspect of the connection (i.e., strength of the bolts or nails in shear). Future research into connection testing could be conducted with the vendors to ensure that the performance of the entire connection was evaluated.
- More tests are needed with more typical types of hardware: tension tie, twisted strap, hold down. Vendors may have more information as to which connections are most frequently used. The failure mode for new connectors was not even associated with the hardware, but rather the nailing in the perpendicular subpurlin. This may not be a realistic failure mode.
- The effects of wall anchor spacing on performance should also be considered.

5.4 ANDERSON ET AL. REPORT

- Additional data gathered from an additional, more thorough, site visit should be added to the model and the model rerun.
- A simpler mode could eliminate the superfluous modes that are present in the detailed model. This could also pin down the issue of different base shears in different directions.

5.5 FUTURE PEER RESEARCH SUGGESTIONS

- More instrumented tilt-up buildings would improve the ability to predict tilt-up performance.
- It appears that the period of a tilt-up is not easily calculated by the methods proposed by the codes or guidelines. A more thorough investigation of tilt-up building period may be helpful to determine a more accurate equation.
- If frame action is a problem in the GLB connections, pulling out the bottom bolts will keep moments from developing at the connection. This will also require sufficient

bearing length for the displacement of the GLB seat. Testing and documenting the performance of such a connection would be helpful.

- Modeling the effect of a bad wall anchor on the rest of the wall anchors may be worthwhile. Would the bad anchor produce a "zipper" effect in the rest of the anchors? This may be a simple addition to already existing analytical models.
- None of the reports mentioned continuity ties. With detailed models, like the ones used in the Hall, Wallace, and Anderson reports, an estimate of continuity tie forces and whether or not ties are needed, should be relatively easy to extract from the model. Further investigation of tie forces would be helpful.
- Amplification of connection demands at pilasters has been brought up in several reports, but nothing in-depth was investigated. This is an issue the code addresses by saying it should be accounted for, but provides no prediction method. Prediction of amplification at the pilasters would be helpful to quantify and some guidance is sorely needed.
- Some recommendation for realistic periods for rigid wall / flexible diaphragm buildings is needed. The question tied into this is how much diaphragm deflection is acceptable? How much gap should be provided from nonstructural elements like storage racks which are also experiencing large deflections on their own?
- More documentation of earthquake damage in existing buildings is needed to try to establish if more damage has been observed for wall anchors on the short sides of buildings. These observations are complicated by the fact that the short sides are typically framed with purlins, while the long sides are framed with GLBs and subpurlins. This difference in framing precludes making a comparison.
- The use of the Pardoen tests in many of the computer models is not entirely justified. Some of the connections tested are not typical and do not reflect the connections in a typical building. The models also do not incorporate the fact that deflections beyond a certain amount (estimated at 3/8″) results in permanent damage to nails at the ledger.
- Subpurlins without anchors were not included in some, if not all, computer models. According to the Pardoen tests, these subpurlins also have stiffness and strength and will help resist loads.
- More testing of what deformations and loads are associated with cross-grain bending. If the connections have the strength that is shown in the Pardoen report, failures should not

occur in moderate earthquakes or in moderate seismic zones. This does not seem consistent with observations, where failures have been observed at about 0.2g. This type of damage obviously is somewhat dependent on different things such as nail size and spacing, ledger size, location of ledger bolts, quality of construction, and condition of materials, and could be investigated to provide a better picture of the failure mode. Nails into ledgers are also subject to diaphragm shear simultaneously. This effect should be considered as well.

- Some important aspects of anchorage connection should be tested, such as eccentric connections, stiffness of ledger nails versus wall anchors.
- All of the reports summarized the analysis of rectangular buildings. Regular buildings make up a percentage, but not all, of tilt-up buildings. The analysis of irregular tilt-ups could be an interesting topic.
- The effect of soft soil on anchorage demands would offer great insight into anchorage design.
- The effects of site period on building response should be studied in more depth. For example, will a tilt-up on bay mud with a site period comparable to the building period obtain serious damage? The effects of different foundations, mat or spread footing, should also be included.

6 Implications for Assessment and Design

6.1 HALL REPORT

- The building period determined by Hall's model is shorter than the periods produced by FEMA 356 or dynamics. Using either of these methods may lead to an unconservative design. Using the acceleration from the plateau of the response spectrum is recommended, unless a more detailed computer model shows otherwise.
- A simple "shear beam" model used for evaluating diaphragms reasonably predicts displacements and shear strains when compared to the results of a 3D finite element model. This shows a general analysis of a building can be done quickly and easily with a high degree of accuracy for the overall performance of the building.
- Vertical ground motion accelerations produce little demand on diaphragm to wall connections. Connections do not need to be analyzed for an upward acceleration.
- Frame action should be included in the building model. In a building with deep members, the possibility for frame action at the connection is possible and may increase the demand at the connection.
- Connection strengthening could lead to improved overall building response.

6.2 WALLACE ET AL. REPORT

- Minor variations in elastic stiffness of the diaphragm can have large effects on the maximum displacements. A stiff diaphragm can drastically reduce displacements. Having an accurate portrayal of the properties of the diaphragm becomes important in the analysis of the building.
- Connection strengthening could lead to improved overall building response.

- Near-field motions have the greatest impact on the diaphragm. Building shears and displacements can be doubled from non-near-field buildings. Careful attention should be paid to the location of the building and the seismicity of the area.
- A 2-D beam model represents diaphragms reasonably well. Again, this shows a general analysis of a building can be done quickly and easily with a high degree of accuracy.
- A hysteretic damping element for computer modeling could improve post-yield displacement correlation. Yielded building elements cause the performance of the building to differ dramatically. Using an element in the model that simulates the increased damping of a yielded element allows the building performance during an earthquake to be more accurately modeled.

6.3 PARDOEN ET AL. REPORT

- The reduction of subpurlin spacing in a diaphragm from 24″ o.c. to 16″ o.c. does not result in significant improvement in capacity for densely nailed panels. Capacity of a diaphragm comes from the edge nailing of the plywood sheets and not the field nailing.
- new glulam to pilaster connections, i.e., Simpson SST GLBT, are more likely to see cross-grain tension due to the layout of the bolts than to old connections, i.e., Simpson SST GLB. The designer must be careful when choosing a connector, and the behavior of the connection and how demands will be affected must be understood.

6.4 ANDERSON ET AL. REPORT

- The building period determined by Anderson's model, and the recorded building period are shorter than the periods produced by FEMA 356 or dynamics. As stated previously, using either of these methods may lead to an unconservative design. Using the acceleration from the plateau of the response spectrum is recommended unless a more detailed computer model shows otherwise.
- At GLB to pilaster connections, forces are as much as three times higher at the middle of the diaphragms than at the ends. Assuming a constant wall anchorage force along the length of a wall, as the codes do, can lead to an ultra conservative design at the ends of a wall, and a much less conservative design near the middle.

- Dense nailing in diaphragms and current commonly used anchorage connections are better able to resist pulse-type earthquakes. The diaphragm is much stronger and stiffer, and will yield lower displacements and strains. With a stronger diaphragm, stronger connections will be needed to resist the increased forces generated by the diaphragm.
- Constructing a 3D model is tedious and time consuming and does not necessarily yield more accurate results for overall building performance (i.e., diaphragm displacements, building period, etc.) than a 2D model. Connection performance may be more accurately estimated in a 3D model, but the report results do not give a clear indication that it does.

REFERENCES

ABK Joint Venture. 1981. *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Building; Diaphragm Testing*. Research Report ABK-TR-03.

American Forest and Paper Association and American Wood Council. 1999. *Allowable Stress Design: Manual for Engineered Wood Construction*. Washington, D.C.

American Forest and Paper Association and American Wood Council. 1996. *Load and Resistance Factored Design: Manual for Engineered Wood Construction*. Washington, D.C.

Anderson, James C., and Vitelmo V. Bertero. 2004. *Seismic Performance of an Instrumented Tilt-up Wall Building.* PEER report 2004/04. (forthcoming). Berkeley, CA: Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Clough, Ray W., and Joseph Penzien. 1993. *Dynamics of Structures — Second Edition*. New York: McGraw-Hill, Inc.

Federal Emergency Management Agency (FEMA). 2000. *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. FEMA Publication 356. Washington, D.C.

Fornseca, F. S., S. L. Wood, and N. M. Hawkins. 1996. *Measured Response of Roof Diaphragms and Wall Panels in Tilt-up Systems Subjected to Cyclic Loading*. *Earthquake Spectra* 12(4). Earthquake Engineering Research Institute.

Fornseca, F. S. 1997. *Cyclic Loading Response of Reinforced Concrete Tilt-up Structures with Plywood Diaphragms*. Urbana, IL: University of Illinois, Urbana.

Hall, John F. 1999. *Seismic Response of Tilt-Up Buildings — Draft*. Pasadena, CA: Caltech.

Hamburger, R. O., S. K. Harris, S. C. Martin, D. L. McCormick, and P. G. Somerville. 1996. *Response of Tilt-up Buildings to Seismic Demands: Observations and Case Studies from the 1994 Northridge Earthquake*. San Francisco, CA: EQE International.

Hrennikoff, A. 1941. Solution of Problems of Elasticity by the Framework Method. Transactions, ASME, *J. Applied Mechanics.* December.

International Code Council, Inc. (ICC). *2000 International Building Code*. Falls Church, VA.

International Conference of Building Officials (ICBO). *1997 Uniform Building Code*, *Volume 2*. Whittier, CA.

Pardoen, Gerard C.; Daniel Del Carlo, Robert P. Kazanjy. 1999. *Stiffness of Timber Diaphragm and Strength of Timber Connections*. Irvine, CA: University of California, Irvine.

Structural Engineers Association of Northern California. 2001. *Guidelines for Seismic Evaluation of Tilt-Up Buildings and Other Rigid Wall / Flexible Diaphragm Structures*. San Francisco, CA.

Wallace, John F., Jonathan P. Stewart, Andrew S. Whittaker. 1999. Building Vulnerability Studies: *Modeling and Evaluation of Tilt-up and Steel Reinforced Concrete Building.* PEER report 1999/13. Berkeley, CA: Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Appendix A Summary: *Seismic Performance of Tilt-up Buildings* by John F. Hall

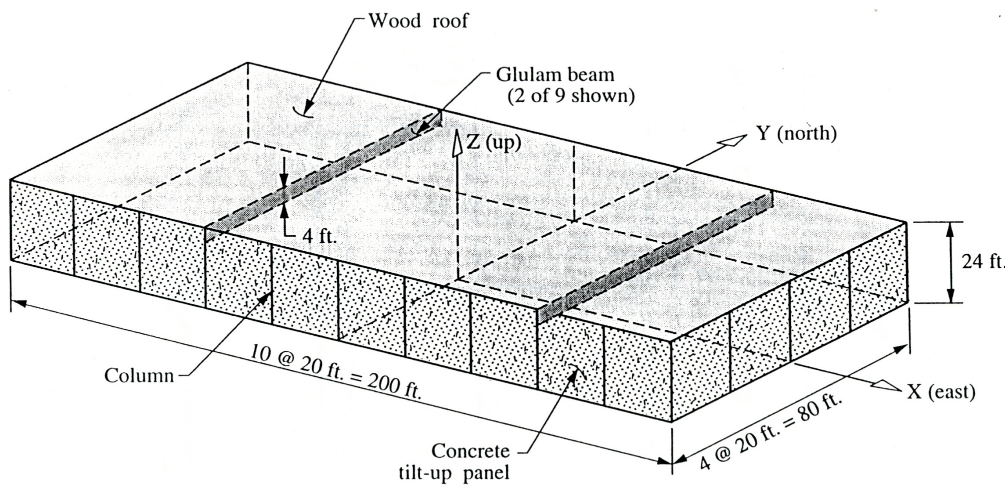
A simple shear beam model is often used to model and evaluate the diaphragm of a tilt-up building. To validate that model and to investigate other modeling aspects of a tilt-up, a 3D finite element model was created, and the results from the two models compared. The main points of investigation are the bending effects in the wall panels, the frame action between the walls and deep beams (placing demands on the connections not recognized in the code), and the effect of the three components of ground motions.

The building model is 80 ft x 200 ft, with the long direction oriented along the x-axis (east-west). The model is similar to the PG&E Meter Repair Facility in Fremont, California, constructed in 1972. In the model:

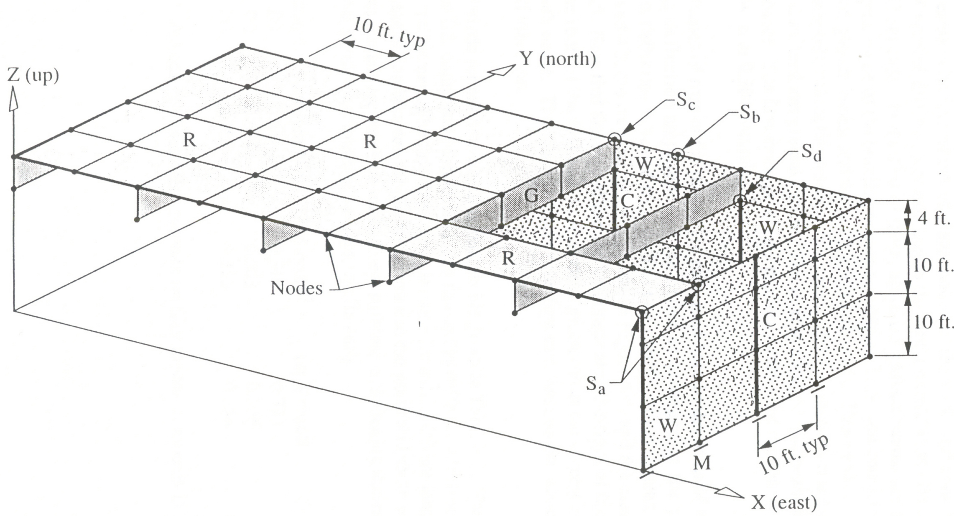
- The 1994 UBC was used to determine the shear strength of the diaphragm.
  - The capacity of the center portion of the diaphragm was lower than those adjacent to the end walls, due to lower shear demand in the middle of the diaphragm.
- The shear connections between the wall and the diaphragm are assumed stronger than the boundary nails of the diaphragm.
- Cast-in-place column elements (pilasters) are placed between wall panels.
- The diaphragm was attached to the tilt-up panels at several locations by nonlinear spring elements representing out-of-plane wall connections.
  - There are three locations: roof node to wall or wall/column node, top GLB/roof node to wall node, and bottom GLB node to wall/column node.
  - GLB connections were typically at 20 ft o.c., with intermediate diaphragm connections at the midpoint between GLBs.
  - Vertical DOFs have been slaved, and axial springs perpendicular to the wall are modeled to be subjected to tension only.

- For horizontal shear at the GLB ends, a spring is placed only at the top node, corresponding DOFs at the bottom GLB node and the adjacent wall/column node are slaved.

- The base of the wall is fixed in all degrees of freedom except for a rotation about the horizontal axis in line with the wall. The rotation is resisted by a elastic-perfectly-plastic bending moment spring.



Layout of computer model



Section of computer model, showing locations of connection springs

Three initial models were made:

- D1, the standard model with design indicative of the '94 UBC.
- D2, where the code factors for the design of diaphragm to wall connections for out-of-plane forces were doubled, thus doubling the strength and stiffness of the connections.
- D3, the same as the standard model except sliders had been placed beneath the glulam beam (GLB) seats to eliminate frame action.

Initial modal analysis showed:

Model	**Transverse (N-S) Period (seconds)**	**Explanation of Period**
D1	0.69	—
D2	0.67	The period is shorter due to stiffer connections
D3	0.70	The period is longer due to lack of frame action

Ten ground motions (GMs) were chosen for dynamic analysis of the models:

- 1978 Tabas (TAB)
- 1989 Loma Prieta at Lexington Dam (LEX) and at Los Gatos (LGP)
- 1992 Landers at Lucerne Valley (LUC)
- 1994 Northridge at Sylmar (SYL) and at Rinaldi Receiving station (RRS)
- 1995 Kobe at Takatori (TAK) and at Japan Meteorological Association (JMA) (GM is at soft soil site)
- 1979 Imperial Valley at Bonds Corner (BCR)
- 1940 Imperial Valley at El Centro (ELC * 1.5) (This GM was scaled amplitude-wise by 1.5)

Each GM has a primary component, a minor component, and a vertical component.

Peak ground accelerations are as follows:

Ground Motion	TAB	LEX	LGP	LUC	SYL	RRS	TAK	JMA	BCR	ELC*1.5
Acceleration, g	0.90	0.45	0.60	0.71	0.84	0.90	0.75	0.89	0.76	0.52

Note: PGAs were estimated from ground acceleration time histories in report.

The models were tested under various GMs and the model runs were indicated by model type first, directions of GM (the underlined entry indicates that the primary component was in that direction), and GM used. Ex.: D1XYZ(SYL) indicates that model D1 was subjected to the

Sylmar GM minor component in the x-direction, the primary component in the y-direction, and the vertical component in the z-direction. D2X(TAK) would indicate that the D2 model was excited by the Takatori GM primary component in the x-direction only.

In model run D1Y(SYL) the roof displacement was about 8 in. and the response was very nonlinear, although still below ultimate. The reduction of strength in the center portion of the diaphragm resulted in a more uniform distribution of shear strains. The displacement of the wall anchor connections at the roof level usually exceeded the ultimate displacement capacities. The roof and seat connections of the GLBs did not only resist out-of-plane forces, but also transferred moment as part of the frame mechanism. The moment transfer was the dominating response. The frame action appears to be an important mechanism in the design of tilt-ups.

Two tests were run, D1XY(SYL) and D1XYZ(SYL), to investigate the effect of multiple components of GMs on analysis results. Results from these tests were virtually unchanged by adding additional components. The vertical acceleration added little moment to the connections. From these results it can be seen that the multiple components of GMs are not needed; only the primary component of GM needs to be applied to each axis.

In model run D2Y(SYL) the elongations in the roof connections were significantly reduced with the stronger and stiffer connections. The roof displacement and shear changed little from the standard model, this indicates connection strength does not reduce roof lateral displacement.

In model run D3Y(SYL) the elimination of frame action caused moderately larger roof displacements and shear strains. However, elongations in the roof-to-wall connections are reduced because the contribution from frame moment transfer is not present. The maximum elongation in the roof connections still exceeds the ultimate capacity. Because of the sliders placed under the GLB seats, a reduction in bending demands of the tilt-up panels occurred due to the absence of frame action.

Two models were run with GMs along the long direction of the building, D1X(SYL) and D2X(SYL). There is no frame action in this direction due to the single connection of the purlin-to-wall connection. In the model D1, the elongations in the connection are very large and greatly exceed the standard ductile range of most standard connections. In the D2 model, the stronger connections lead to much smaller elongations.

Cases D1Y, D2Y, and D3Y were run for all ten primary components of GMs. The differences between the results for each model are similar to the differences observed for the models under the Sylmar GM. Very large displacements occurred under the Rinaldi Receiving Station GM, the Takatori GM, and Japan Meteorological Association. Under these large displacements, failure of the diaphragm due to large shear strains is a distinct possibility. There is a need to limit the displacement of the roof to prevent severe damage or collapse.

Cases D1X, D2X, and D3X were run for all ten primary components of GMs. Again, the differences observed in the behavior of the models was similar to the differences seen in the models under the Sylmar GM.

In order to compare the results and complete the task outlined in the scope, a simplified 2D model was created for the diaphragm in the x-direction and one in the y-direction. Model A1 was to represent the diaphragm with shaking along the y-axis (north-south), and model B1 was to represent the diaphragm with shaking along the x-axis (east-west).

Model	2D Model Period	Period from FEM analysis
A1	0.77 seconds	0.69 seconds
B1	0.28 seconds	0.30 seconds

The longer period in model A1 is due to the lack of extra bending stiffness provided by the N-S walls (which were not modeled) including the frame action from the GLB connection. The smaller period of model B1 is due to the different roles the walls played in the periods of model D1.

Model B1 accurately depicts roof displacements and shear strains, but is not as good for prediction of connection elongations. Model A1 shows poor agreement with model D1 for displacements and elongations peaks. Poor performance is observed in model A1 due to lack of frame action in the model. Since frame action is not a problem along the x-axis, model B1 performs better. Elongations are sensitive to modeling assumptions, since the connection forces are related to the wall inertial forces, which are acceleration derived.

In an attempt to limit roof displacements, two interior walls were added to the model along the centerline to reduce the diaphragm span. This option was chosen over strengthening the diaphragm, which could increase the horizontal shear demand on connections in the east and west walls. The model created was denoted “D4.” Roof to wall connections to the east and west walls were twice the code value to counter possible increased connection demands, sliders were placed at the GLB seats, and the capacity of the roof connections to north and south walls was doubled. The weaker center portion of the diaphragm was retained to reduce the loads delivered to the buttress walls (i.e., the diaphragm was not renailed). Buttressing reduced the period to 0.42 seconds from the 0.67–0.70 second range. It also reduced the maximum displacement to 4.29 in. from 8–9 in. Buttressing is effective in reducing the maximum displacements and the maximum elongations.

Two additional models were made reflecting the construction of the PG&E Fremont building. Model D5 used properties obtained from testing at UC Irvine for old construction, and model D6 used the same roof properties as D5 but with stronger diaphragm to wall connections. The properties of the stronger connections were based on the tests at UC Irvine of the new connections. The results of the analysis were consistent with those from the previous runs of the model, but the connection strengths, with the exception of D5 and D6, are comparable or exceed model D2. Weak connections of the roof to the east and west walls in model D5 suffered large elongations. This may mean that the original Fremont structure is vulnerable during an earthquake.

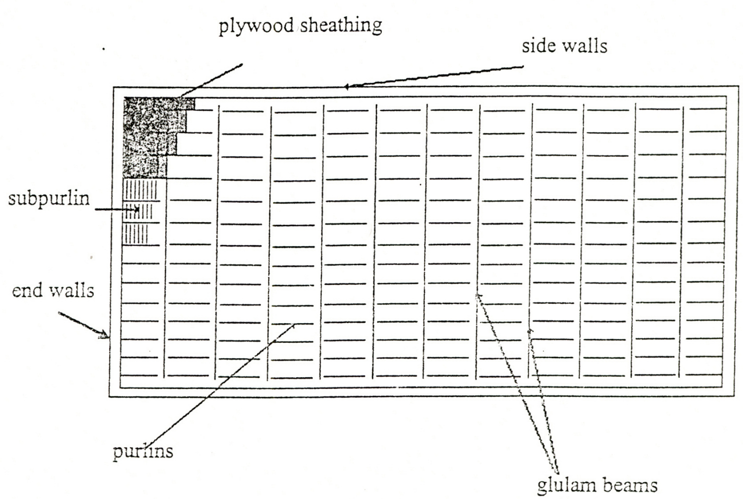
Conclusions derived from the analyses include:

1. The level of response of a tilt-up building to a particular GM correlates to the response spectrum ordinate of the fundamental mode in the direction being excited.
2. Conservative results for design/evaluation can be obtained by applying the principal horizontal ground motion singly in separate analysis for each horizontal direction.
3. In model D1 (based on '94 UBC) some motions place large demands on diaphragm to wall connections. These connections would need to be more ductile to survive. The connection demands are reasonable when strengths are doubled (model D2).
4. The frame action between the GLBs and the supporting walls increases connection demands. The tensile component of the moment between a beam and a wall adds to the tensile force that develops when the diaphragm and wall want to move apart. A possible design fix to this problem is to reduce the depth of the GLB by adding interior columns. Also, replacing the GLB seat connection with a slider to allow free movement reduces the demand of the connection
5. The lateral deflection of the roof under near-source ground motions can be quite large, especially for excitation in the short direction when the roof vibrates perpendicular to the long span.

6. Reducing diaphragm displacements can be accomplished by adding buttress walls, but can increase diaphragm shear strains due to the buttress walls attracting large forces and the diaphragm not being strengthened.
7. GMs applied in the long direction cause smaller roof displacements, but this may be the controlling case for roof-to-wall connections. The cause is the potential for higher accelerations of the roof when the building is excited in the long direction.
8. The responses of model D1 to 1.5*El Centro GM are in a reasonable range.
9. Simplified models based on the shear beam concept with added masses predict roof displacements and shear strains that are roughly consistent with results of FEM models. However, demands on diaphragm to wall connections show greater disagreement. Connection demands are related to acceleration response, which is sensitive to modeling, and no frame action exists in the simplified models.
10. There is an imbalance in connection strengths, weak connections for the short walls, and strong connections for the long walls.
11. The favorable performance of models with stronger connections shows the benefit of the retrofit of connection strengthening.

Appendix B Summary: *Building Vulnerability Studies: Modeling and Evaluation of Tilt-up and Steel Reinforced Concrete Buildings* by John W. Wallace, Jonathan P. Stewart, and Andrew S. Whittaker

Many buildings PG&E owns are tilt-up built before implementation of modern provisions for seismic design. These buildings are therefore susceptible to significant damage in moderate to strong earthquakes. To evaluate this type of building for safety, a series of 2D nonlinear analyses were run to discover any potential vulnerabilities. The PG&E Repair Facility in Fremont, California, was chosen as a typical tilt-up building. Another PG&E building, in Hollister, was instrumented and subjected to moderate ground shaking during the 1989 Loma Prieta earthquake. The structural systems for both the Fremont and Hollister sites are similar, so conclusions from Hollister can be used to assess the Fremont site.



Typical layout of a tilt-up

A study of soil-foundation-structure interaction and its effect on tilt-up buildings were also studied.

The structural response generally seen in tilt-up buildings is that the end walls behave as essentially rigid elements without significant amplification of the motion from the base to the top. However, the diaphragm will have significant acceleration. Accelerations of 2 to 3 times the measured accelerations at the ends have been recorded at the mid-span. From testing done at the University of Illinois (Fonseca 1997; Fonseca et al. 1996) it was concluded that the behavior of the roof diaphragm is a major factor in the building response. From the force-displacement data it was seen that the stiffness of the structural system degraded as the amplitude of the imposed displacements and the number of loading cycles increased.

Failure modes in these buildings center around the connection of the diaphragm to the walls. The glulam beam (GLB) to pilaster connection and the purlin and subpurlin-to-wall connection are the main connection and failure areas. Especially in older buildings the lack of wall anchorage is a problem and nail pullout and cross-ledger bending can often occur.

Experimental studies typically concentrate on four areas: (1) the response of individual nailed connections (2) the response of tilt-up wall panels (3) the response of diaphragms (4) the response of complete tilt-up systems. The main focus of this report was the diaphragm and connection tests, since these aspects generally dominate the response of the building.

Previous tests have been performed by the ABK (1981) research program and consisted of the dynamic and static loading of fourteen 20 ft by 60 ft diaphragms. The test sample diaphragm N best represents tilt-up construction with 1/2 in. plywood, 8d nails at 4 in. o.c. edge nailing, and 12 in. o.c. intermediate nailing. This diaphragm obtained a yield force of 12 k with an initial stiffness of 20 k/in. and post-yield stiffness 35% of the original. Most tests were conducted without roofing material, but one series of tests showed that unblocked diaphragms with roofing material obtained an increase in overall stiffness of about 33% at a displacement level of 0.3 in. and an even higher contribution at higher displacement levels. Because only a small amount of tests were conducted, the contribution of roofing is not certain and should be neglected.

Other tests, conducted by the University of California at Irvine (UCI), were with six 20 ft by 16 ft panels. Four specimens were tested with 6 in. o.c. edge nailing, and two specimens were tested with 2 in. o.c. edge nailing. Those with 6 in. nailing obtained an average yield of 10 k, an initial stiffness of 12.5 k/in., and a post-yield stiffness of 15%. Panels with 2 in. nailing obtained an average yield of 20 k, an initial stiffness of 16 k/in., and a post-yield stiffness of 35%.

An attempt to generalize the diaphragm stiffness and strength was made by using equations in a previous report by Hamburger (Hamburger et al. 1996) and extrapolating from the data obtained from the UCI tests.

The equations as modified by Wallace by using the UCI test data are:

K_d = 12.5 k/in. x (D/16 ft)/(L/20 ft) x ($F_{plywood}$ / 0.5 in.) x $F_{nailsize}$ x (6 in. / s) x $F_{roofing}$
Fy = 10k x (D/16 ft) x ($F_{plywood}$ / 0.5 in.) x $F_{nailsize}$ x (6 in. / s) x $F_{roofing}$

Note: The original EQE equations are derived from testing done by ABK (1981), and are as follows:

K_d = 21.2 k/in. (H / L) F_{ns} F_{th} F_d F_r
F_y = 8.48 k (H) F_{ns} F_{th} F_d F_r

Where:
H = depth of the diaphragm element in feet / 24 feet
L = width of the diaphragm element in feet / 24 feet
F_{ns} = edge nail spacing in inches / 4 inches
F_{th} = plywood thickness in inches / ½ inch
F_d = 1.0 for 8d or 1.33 for 10d nails
F_r = 1.2 for presence of roofing

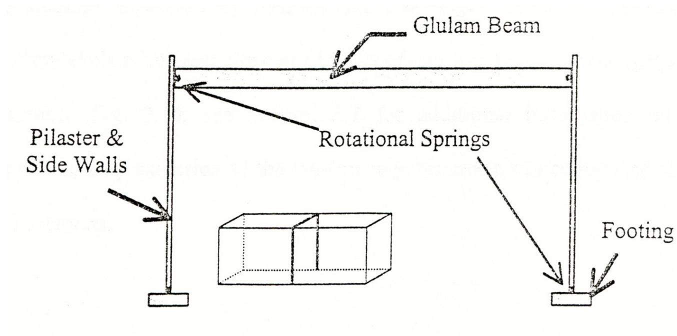
The first panel was seen as the "control" panel and was used to normalize the equation for that series of tests. The equation was used and compared to actual results obtained from the UCI tests and the ABK tests. The computed data do not always agree with the test results and can vary from 3% to 74%. The limited data and lack of a systematic evaluation of parameters in the existing test programs inhibit the ability to create a more comprehensive equation.

When modeling a tilt-up building, the most widely used procedure is to model the diaphragm as a system of inelastic spring or truss elements. However, a way to determine connection forces in these models is not well developed. Models used in previous studies have omitted end walls, side walls, soil foundation structure interaction, connections and GLBs.

A simple nonlinear model of a tilt-up can be constructed using three 2D models. These three models are:

(1) The diaphragm for in-plane stiffness and strength of the roof.

(2) The internal frame to model the side walls, out-of-plane bending stiffness of the roof, connections between the roof and side walls, internal steel columns supporting the roof, and wall to foundation details.



Internal frame model

(3) The end walls to model the stiffness end walls, foundation and the soil.

In the diaphragm model nonlinear elements which allow only bilinear flexural response were used to model the in-plane behavior of the diaphragm. The diaphragm is basically modeled as a shear beam. The yield strength and stiffness of each "beam" in the diaphragm model are selected to represent the behavior of the diaphragm. The diaphragm properties were calculated using a shear stiffness of a fixed-fixed beam and the Hamburger equation for the yield strength.

The internal frame model included the out-of-plane bending stiffness of the side walls, bending stiffness of the diaphragm and GLBs, and stiffness of connections between the roof and the side walls.

- Equivalent columns, with a tributary width per ACI's T-beam requirements, were used to represent the walls and the pilasters.
- The moment-curvature relationship of the pilaster-footing connections was computed by the extension of the tension reinforcement.
- The GLB seat was assumed to be rigid and the connection strength of the pilaster-GLB connection was limited by yielding of the tension connection in the plane of the roof. Rotation in the GLB connection was from extension of the straps or hold-downs. The GLB connection assumes that other failure modes, cross-grain bending of the ledger and splitting at the pilaster tip, are prevented. The beam seat should be checked after analysis to make sure it can develop the required forces.
- The side wall-purlin/subpurlin connection strength was limited by yielding of the tension connections in the plane of the roof, rotation was from the extension of the straps, and the connection was assumed rigid in compression.

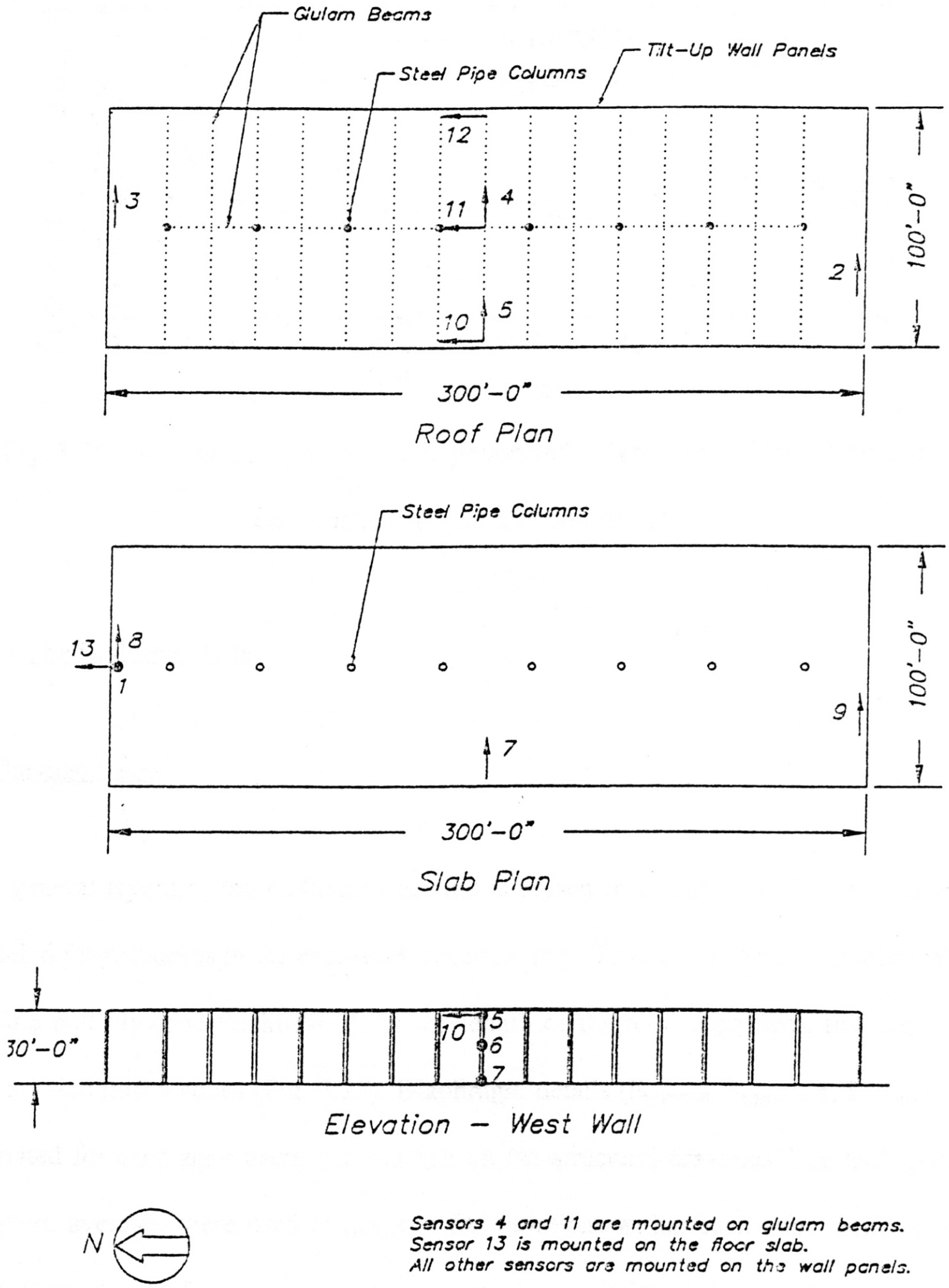
All components of the internal frame were placed together in a frame model and analyzed. The resulting pushover curve was simplified into a bilinear curve. The bilinear curve was then used to define a bilinear spring element in the diaphragm model to represent the internal frames.

Three models of the end walls were used: one where the diaphragm was fixed at the top of the walls and the flexibility of the walls neglected, another model where the elastic bending and shear stiffness of end walls were modeled with a fixed base, and a model that included the effect of the flexibility of the foundation and soil.

In addition to hysteretic damping associated with nonlinear behavior, Rayleigh damping of 2% was used as well as damping from soil-foundation-structure interaction for walls on a flexible base.

The mass contribution of the out-of-plane walls was calculated and added to the weight of the diaphragm. For the end walls it was assumed that either shear deformations or rotation at the base of the wall occurred. Both of these cases lead to a triangular displaced shape of the wall, and because of this, two-thirds of the mass of the end walls were lumped at the ends of the diaphragm model.

To test the validity of the model, the conditions of the PG&E building at Hollister were replicated and the results from the model then checked against data from instruments on the building. The Hollister building contains 13 strong-motion instruments that have recorded responses for the 1948 Morgan Hill, 1986 Hollister, and 1989 Loma Prieta earthquakes. The Loma Prieta ground motion was used, since the levels of shaking in the other earthquakes were relatively small. No peak ground acceleration was given for the record used.



Layout and location of instrumentation in Hollister building

For the diaphragm model due to the variation of nail spacing, the diaphragm was divided into four zones of strength and stiffness. Roofing was present at the site and accounted for in the model. An average of 15% post-yield stiffness was used.

For the internal frame the equivalent columns used for the walls were assumed to be fully cracked. The connection for the pilaster-footing connection was not known, so a connection detail was used from the PG&E Fremont building. The roof-wall connections were a single bolt through the GLB seat hardware and straps on top of the subpurlins were assumed.

The complete model was run for three different cases: no end walls, end walls fixed at the base, and end walls on a flexible base. All three models yielded essentially the same response. Each had a period of 0.66 seconds. The strain calculated in the connection straps was about 0.5% (4.75 ε_y); this number does not seem excessive but there were no data to support this conclusion. In the model of the Hollister building, the critical case for the diaphragm shear is not the diaphragm section near the end walls, rather the diaphragm sections at the ends of the two weakest diaphragm element groups.

From the study of the model of the Hollister building, it could be seen that:

- The displacement correlation for peak cycles was relatively good; however, the model was slightly stiffer.
- The displacement correlation beyond 15 seconds was poor.
- Yielding in the diaphragm was not observed.
- The mid-diaphragm displacement response was not significantly influenced for models that included the end walls.
- The mid-diaphragm displacement response was not influenced by soil-foundation-structure interaction due to the building geometry and favorable soil conditions.

Each element of the model was varied in strength and stiffness to observe the model response.

- The elastic stiffness of the diaphragm was varied +/- 15%. The maximum displacement increased almost 50% with a 15% decrease in diaphragm stiffness, and yielding occurred in the diaphragm. With a 15% increase in stiffness, deflection decreased by only 12%, and no yielding occurred. These analyses indicate that a small change in diaphragm stiffness can lead to significantly improved displacement response correlation. Overall, the diaphragm model reasonably represents the diaphragm behavior.
- Connection stiffness and strength were also studied over several conditions. Stiffnesses were varied from 0.6 x 10^6 to 2.0 x 10^6 and completely rigid, and strengths were varied from 890 k to 10,0000k. The results of analyses indicate the effective stiffness and yield

strength of the springs representing the internal frame are relatively insensitive to changes in connection stiffnesses. However, connection strength can affect changes in yield strength of the internal frame. Changes in the frame stiffness were generally less than 5%, while frame yield strength varied by as much as 30%. The stiffness of the internal frame relies mostly on the wall panels and the GLB, while strength depends primarily on the connection strength This potential variation in yield strength of the internal frame should be considered in assessing connection performance.

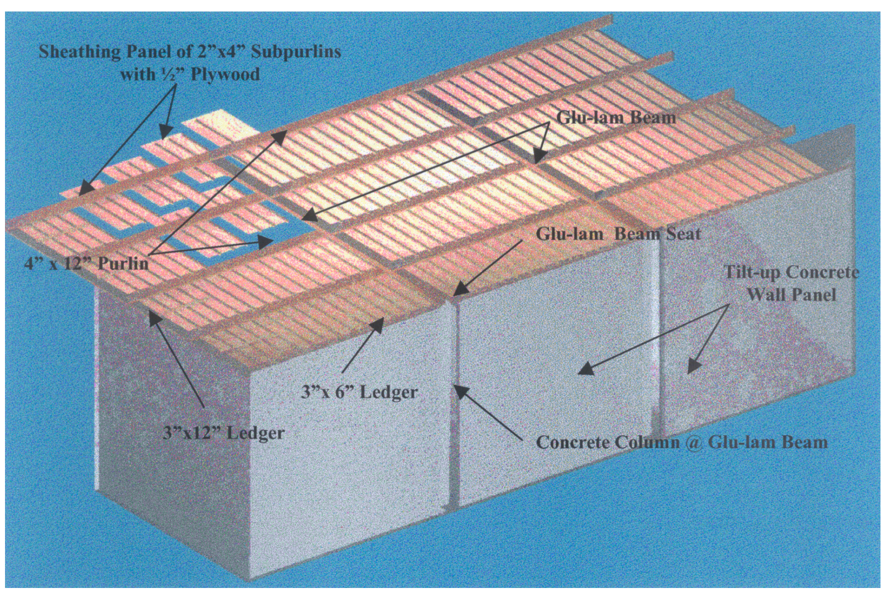
- The effect of wall tributary length was also investigated. Initially ACI's T-beam requirements were used to choose the width, but a model was made with the full tributary width (halfway between pilasters). The building period decreased 11% and the correlation between responses was worse.
- The stiffness of the GLB in the internal frame was increased 100% to account for the contribution of the out-of-plane bending of the roof diaphragm to the lateral stiffness of the frame. This increase made little difference in the effective stiffness and yield strength. When modeling the internal frame, it was recommended to consider the GLB stiffness only and neglect the contribution of the diaphragm.
- From previous runs it could be seen that damping increases after the peak cycles, possibly due to diaphragm damage (i.e., nail deformation or slip). With the original 2% damping, the correlation was good until after the peak. When damping was increased to 10% the correlation at peak is not good, but the correlation after the peak is much better. In order to capture this effect, an element that allows hysteretic damping after inelastic diaphragm response could be used to model the diaphragm.
- The soil properties were also varied, and were modeled as a very flexible base. The building period increased from 0.70 seconds to 0.72 seconds, and had an 18% increase in peak displacements. Soil-foundation-structure interaction is unlikely to be significant for tilt-up buildings.
- One of the most significant effects on tilt-ups is near-field motions. Several ground motions were chosen and scaled to the average PGA of the ground motions selected. Shears and displacements in the diaphragm increased 30% to 60%, and internal spring forces increased 9% to 16%. Near-field motions have the greatest impact on diaphragm response of the variables explored.

The report came to the following conclusions:

- The simple model is capable of representing measured mid-diaphragm response reasonably well.
- Sensitivity studies indicate that diaphragm stiffness has a considerable impact on the response of tilt-ups.
- The Hamburger equation, when modified, gives a reasonable approach for predicting diaphragm strength and stiffness based on current information.
- Near-field motions have a significant influence on the response of the building.

Appendix C Summary: *Stiffness of Timber Diaphragms and Strength of Timber Connections* by Gerard C. Pardoen, Daniel Del Carlo, and Robert P. Kazanjy

An experimental program was run at the University of California, Irvine, to complement the analytical studies. The PG&E building in Fremont, California, the Gas Meter Repair Plant, was chosen as the experimental focus. Several expansions have occurred on the site and construction of the diaphragm and diaphragm connections vary. In-situ conditions were replicated and tested. The major emphasis of this study was (1) to determine the cyclic load deformation stiffness characteristics of the roof diaphragm of the PG&E Fremont building and (2) to determine the roof to wall connection strength of the existing connections in the PG&E Fremont building.



Components of a tilt-up

To determine the diaphragm stiffness five 16-ft x20-ft panels and one 20-ft x16-ft were constructed. To test connection strength, twelve roof-to-wall connections were tested cyclically. These connections included the glulam beam-to-column connection, purlin-to-wall connection, and the subpurlin-to-wall connection. Test specimens were made to replicate old (original to early 1970's) construction[2], and new (current to mid-to-late 1980's) construction, and were tested using a simple pull test.

STIFFNESS OF ROOF DIAPHRAGMS

All diaphragms tested were constructed with 10d nails and Structural I sheathing. Four new panels panels were constructed (RD1, RD2, RD3, and RD4) with subpurlins at 16″ o.c. Two old panels were constructed, with subpurlins at 24″ o.c. The only difference between the panels is the spacing of the subpurlins.

ATC-24 was used at the loading procedure for the panels except for panel RD4 where an ersatz "fling" displacement time history was used: where the "fling" was a 2% drift displacement in the "pull" direction immediately followed by loading in the "push" direction until significant load decay occurred and the panels were then reloaded in the "pull" direction.

Panel	Height x Width	Edge Nailing	Continuous Edge Nailing	Sub-purlin spacing	UBC Allowable (lb/ft)	Yield Load (lb/ft)	Ultimate Load (lb/ft)
RD1	20 ft x 16 ft	6 in. o.c.	6 in. o.c.	2 x 4 @ 16 in. o.c.	320	406	1000
RD2	16 ft x 20 ft	6 in. o.c.	6 in. o.c.	2 x 4 @ 16 in. o.c.	320	500	1100
RD3	20 ft x 16 ft	3 in. o.c.	2 in. o.c.	2 x 4 @ 16 in. o.c.	730	938	1625
RD4	20 ft x 16 ft	6 in. o.c.	6 in. o.c.	2 x 4 @ 16 in. o.c.	320	406	1250
RD5	20 ft x 16 ft	6 in. o.c.	6 in. o.c.	2 x 4 @ 24 in. o.c.	320	469	719
RD6	20 ft x 16 ft	3 in. o.c.	2 in. o.c.	2 x 4 @ 24 in. o.c.	730	938	1500

From testing it was observed that:

- Panel RD2 was stiffer and obtained a higher strength than panel RD1, but when these values were normalized with respect to racking edge length and drift, the stiffness and deformation values were within 10% of each other.
- Panels with more nails at panel edges (denser nailing pattern) had higher ultimate loads as expected.

[2] Degenkolb Note: The use of "old" and "new" construction in this report is misleading. Both old and new connections are pre-1997 and do not reflect current construction. New connections are more reflective of mid-1980's construction.

- The testing procedure affects the results. The panel tested using the fling procedure showed a 25% increase in ultimate load and a 50% increase in lateral displacement.
- The increase in strength in the new panels was not due to the increase in subpurlins, rather the increase in nailing due to the addition of more subpurlins. It appears that the subpurlins contribute more to the lateral load performance of a lightly nailed shear panel than a more densely nailed one.

The observed yield loads exceeded the UBC code values by 27%–56%. (Note: UBC values are allowable values.)

STRENGTH TESTS OF GLULAM BEAM-COLUMN CONNECTIONS

Two double-ended columns were constructed with one glulam beam (GLB) connection at each end. One column, with dimensions 12 in. x 12 in., was constructed with two old connections, Simpson GLB seat GLB-5A, while the other, with dimensions 12 in. x 16 in., was constructed with two new connections, Simpson GLB seat GLB-512. Both columns had the same reinforcement, 4-#7 longitudinal bars, 4-#3 hoops at 4 in. o.c. near the end, and #3 hoops at 12 in. o.c. for the middle section of the column. The primary concern was the strength of the connection between the GLB and the pilaster, so the concrete column was attached to the lab's strong wall while the GLB was pulled monotonically.

Sample	Beam/Column Size	Beam Seat	Bolts	Ultimate / NDS Load (k)
GLB1	5 1/8" x 12" / 12" x 12"	GLB-5A	(2) 5/8 in.	29.9 / 5.99
GLB2	5 1/8" x 12" / 12" x 12"	GLB-5A	(2) 5/8 in.	32.8 / 5.99
GLB3	5 1/8" x 12" / 12" x 16"	GLB-512	(2) 3/4 in.	35.0 / 8.26
GLB4	5 1/8" x 12" / 12" x 16"	GLB-512	(2) 3/4 in.	37.5 / 8.26

The test results showed:

- The new connection has a higher yield strength and ultimate strength, and has a more clearly defined bilinear elasto-plastic behavior.
- The old connection never experienced complete failure or fracture, but the new connections failed in cross-grain tension induced by the rotation of the bolt couple relative to the GLB. A split occurred along the line of bolts in the new connection.
- The differences in the connections may have caused the failure in the new connection, while the old connection did not fail. The old connection used 5/8 in. bolts, while the new connection used 3/4 in. bolts. The old bolt pattern was skewed at 45°, whereas the

new connection had the bolts lined up vertically. This vertical alignment caused the large cross-grain tension responsible for the failure.

- The old connection bolts yielded and caused local crushing, which permitted rotation without large cross-grain tension forces.
- The old connections may have been more ductile than the new stronger connections.

STRENGTH TESTS OF PURLIN-TO-WALL CONNECTIONS

Four test panels were constructed to test the strength of the purlin-to-wall connections, two representing old construction, and two new construction. The old connection was merely a joist hanger to support the purlin, while the new connection had a strap on the top of the purlin embedded into the concrete and a double L-bracket connecting it to the ledger.

Sample	Length x Width	Ledger	Anchor Bolts	Straps	Edge Nailing @ Ledger	Edge Nailing @ 4x	Ultimate Load (k)
PW1	8 ft x 4 ft	3 in. x 12 in.	(4) 5/8 in.	(2) PA-18	10d @ 6 in. o.c.	10d @ 6 in. o.c.	20.5
PW2	8 ft x 4 ft	3 in. x 12 in.	(4) 5/8 in.	(2) PA-18	10d @ 6 in. o.c.	10d @ 6 in. o.c.	20.5
PW3	8 ft x 4 ft	3 in. x 12 in.	(2) 5/8 in.	None	10d @ 6 in. o.c.	10d @ 3 in. o.c.	5.1
PW4	8 ft x 4 ft	3 in. x 12 in.	(2) 5/8 in.	None	10d @ 6 in. o.c.	10d @ 3 in. o.c.	3.1

The tests showed:

- The new connection with the Simpson PA-18 strap and double L-bracket had significantly higher yield strength and a more pronounced trilinear failure. Failure of this connection was nail pullout and nail pullthrough at the subpurlin where the plywood was spliced.
- The old construction failed by cross-grain splitting of the ledger.

STRENGTH TESTS OF SUBPURLIN-TO-WALL CONNECTIONS

Four test panels were constructed to test the strength of the subpurlin-to-wall connections, two representing old construction, and two new construction. The old connection was merely a joist hanger to support the purlin, while the new connection had a strap on the top of the subpurlin embedded into the concrete.

Sample	Length x Width	Ledger	Anchor Bolts	Straps	Edge Nailing @ Ledger	Average Yield Load (k)	Average Ultimate Load (k)
SW1	4 ft x 4 ft	3 in. x 6 in.	(1) 5/8 in.	(2) PAT-18	10d @ 6 in. o.c.	3.5	7.5
SW2	4 ft x 4 ft	3 in. x 6 in.	(1) 5/8 in.	(2) PAT-18	10d @ 6 in. o.c.		
SW3	4 ft x 4 ft	3 in. x 6 in.	(1) 5/8 in.	None	10d @ 6 in. o.c.	3.0	4.2
SW4	4 ft x 4 ft	3 in. x 6 in.	(1) 5/8 in.	None	10d @ 6 in. o.c.		

The test results showed:

- The new connections with the Simpson PAT-18 straps had a slightly higher yield and a 75% increase in ultimate strength. The failure was nail pullout and pullthrough.
- The old connection failed at lower loads due to cross-grain bending and splitting of the ledger.

Appendix D—Summary: *Seismic Performance of an Instrumented Tilt-up Building* by James C. Anderson and Vitelmo V. Bertero

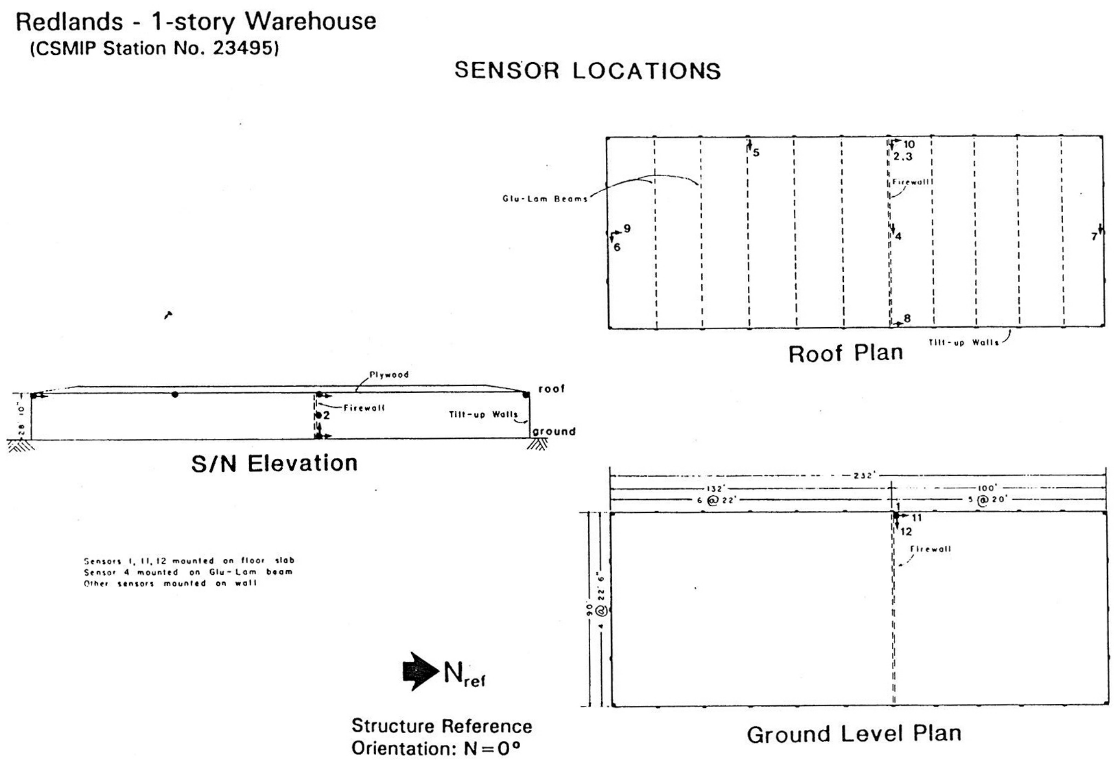
An instrumented tilt-up building in Redlands, California, was subjected to four ground motions and provided sufficient data to compare with computer models. The data from the instrumented building helped the computer model to be more realistic. The design of the building was completed in 1971, and was probably designed per the lateral force provisions of the 1969 [*sic.* No code in 1969; 1970 was code year.] Building Code. The building is approximately 232 ft by 98 ft, with the long direction oriented in the north-south direction. The building is divided almost in half by a bearing stud partition that acts as a fire wall. Walls and framing are typical of tilt-up construction of this vintage; concrete walls with cast-in-place pilasters between individual panels and the glulam beams (GLBs) are supported by the pilasters and framing spans between GLBs. The diaphragm is 1/2 in. structural plywood.

The building was instrumented with 12 strong-motion accelerometers. The building is located almost halfway between the San Jacinto and San Andreas faults, and has been through four significant ground motions:

- 07/08/1986 Palm Springs at a distance of 6.2 miles from epicenter.
- 06/28/1992 Landers — at a distance of 46 miles west from epicenter (this ground motion produced the largest displacement in the east-west direction).
- 06/28/1992 Big Bear — at a distance of 24 miles west of the epicenter (this ground motion produced the strongest accelerations and largest displacement in north-south direction).
- 01/17/1994 Northridge — at a distance of 76 miles southeast of the epicenter.

Peak Ground accelerations recorded at the Redlands site by the instruments at the base of the building (Instruments 11 and 12) are as follows:

Ground Motion	Palm Springs	Landers	Big Bear	Northridge
Acceleration, g	0.04	0.12	0.18	0.07



Layout of instrumentation locations for Redlands building

OBSERVED DAMAGE

On August 8, 2001, a site survey was conducted by the authors to determine the extent of the damage from the four earthquakes. A close inspection of the roof diaphragm was not possible due to limited roof access and interior finishes obstructing the view. No significant structural damage to the remainder of the building was visible and occupants reported no severe damage from the earthquakes over the years. Minor cracking, however, was observed in some of the structural walls.

SPECTRAL ANALYSIS OF RECORDED DATA

The recorded accelerations were run through a single-degree-of-freedom oscillator with 5% critical damping with variable circular frequency to generate the linear elastic response spectrum. Response spectrum from roof sensor 5 and floor sensor12 under the Landers GM indicates a fundamental period in the transverse (E-W) direction of about 0.4 seconds and sensors 9 and 11 indicate a fundamental period of about 0.35 seconds in the longitudinal (N-S) direction.

Fourier transfer functions (FTFs) in the north-south direction using sensor 9 at the roof and sensor 11 at the base were calculated. FTFs in the east-west direction using sensor 5 at the roof and sensor 12 at the base were also calculated. Results are summarized below:

Seismic Event	Fundamental Period (sec)		Relative Stiffness		Percent Stiffness Reduction	
	N-S	E-W	N-S	E-W	N-S	E-W
Palm Springs	0.35	0.40	1.00	1.00	0 %	0 %
Landers	0.33	0.46	1.12	0.75	0 %	25 %
Big Bear	0.75	0.54	0.22	0.55	78 %	45 %
Northridge	0.60	0.67	0.34	0.35	66 %	65 %

The increase in period during the Big Bear earthquake could be indicative of damage occurring and possibly loosening of nails and connections in the diaphragm. The slight decrease in period in the north-south direction for the Northridge earthquake is probably due to remedial work done in the building after the Big Bear earthquake.

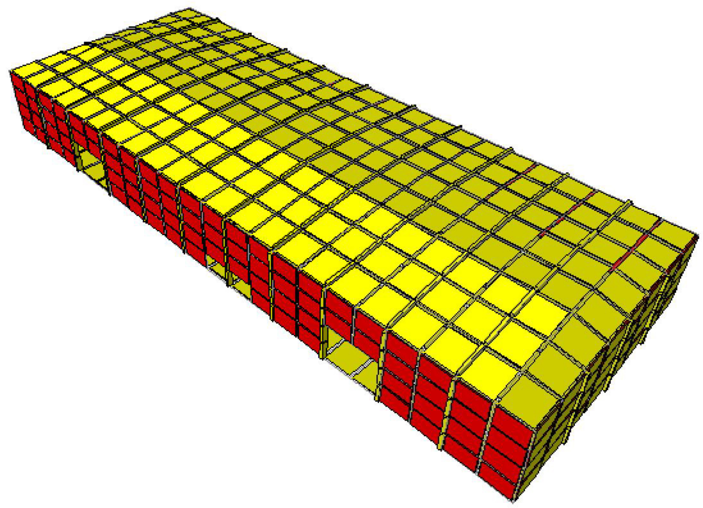
ELASTIC DYNAMIC ANALYSIS

An elastic dynamic analysis was done of the same building using a 3D model constructed in SAP 2000. Only four sheets of drawings were available, so assumptions were made and standard details for tilt-up buildings were used. Three model variations were considered:

(1) The original elastic model — model is built as drawings indicate.

(2) 1st Variation — wall panels are not connected at corners of building.

(3) 2nd Variation — walls panels are not connected to one another by pilasters, and GLBs are supported in the middle of the panel.



SAP 2000 model

The models contain the slab-on-grade supported by elastic springs, concrete walls, concrete pilasters (only in models 1 and 2), timber GLBs, timber roof purlins, and the plywood rood diaphragm. The wood stud bearing wall (fire wall) in the building was not included in the analytical model due to lack of details.

The analysis of the original 3D model indicates the 6th mode is the first significant mode in the east-west direction at a period of 0.39 seconds, and the 9th mode is the first significant mode in the north-south direction at 0.33 seconds. Comparing these results from the response spectrum analysis and the FTFs:

Analysis Procedure	Transverse (E-W)	Longitudinal (N-S)
Response Spectrum	0.40 s	0.35 s
Fourier Transfer Function	0.40 s	0.33 s
SAP2000 Model	0.39 s	0.29 s

To verify the model, time histories were run for each of the four earthquakes. For each ground motion, the two horizontal accelerations were applied simultaneously. Sensor 12 was used for the east-west direction and sensor 11 was used for the north-south direction. From the time history analyses it seems that calculated displacement histories show better correlation with recorded data than do the calculated acceleration histories. Interaction between the north wall and the fire wall may have influenced the comparison of peak acceleration and displacement values at locations near the two walls.

The response of the model to the largest earthquake, Big Bear, was studied and a more in-depth comparison between the response to this earthquake and the predicted response can be found in the report

The base shears obtained for each ground motion were compared to the design base shear per the 1997 UBC.

Ground Motion	Base Shear per 1997 UBC (k)		Base Shear per SAP 2000 Analysis (k)	
	E-W	N-S	E-W	N-S
Palm Springs	247	247	175	25
Landers	247	247	450*	70
Big Bear	247	247	470**	60
Northridge	247	247	120	40

* Landers exceeds code value on 6 excursions. The damage will be limited and may actually be contained in the inherent overstrength of building.

** Big Bear exceeds code value on 9 excursions. The effect is slightly greater than Landers.

In calculating the base shear, the acceleration is multiplied by the modal participation factor for the mass of the structure. The mass participation in the transverse (east-west) direction is three times that of the longitudinal (north-south) direction, so the base shear in the transverse direction is three times larger. This effect is currently not covered in the static equivalent lateral load procedure of the '97 UBC, and could be an important factor in design.

Looking at the forces in the walls, it was seen that the in-plane shear of the walls is much less than the expected strength, and the out-of-plane bending demands are also much less than the estimated capacity. But the in-plane shear of the diaphragm exceeded its strength by two times.

	GLB to Pilaster Axial (k)	In-plane Wall Shear (k/in.)	Out-of-plane Wall Moment (k-in./in.)	In-plane Diaphragm Shear (k/in.)
Estimated Capacity	26/34*	1.74	3.0	0.085
Landers	13.6	0.15	1.4	0.175
Big Bear	14	0.18	1.4	0.175

* Old connection idealized strength from UCI tests/new connection idealized strengths from UCI tests.

Although the idealized yielding of the old specimens was estimated to be about 26 kips, the actual yielding started at about 13 kips. The new specimens behaved practically elastically up to a load of 26 kips and the idealized yielding strength was about 34 kips.

PULSE-TYPE GROUND MOTIONS

The effect of tilt-ups to pulse-type ground motion was also studied for buildings with near-field conditions. Three pulses were selected for analysis:

- the Lucerne ground motion during Landers
- the Takatori Station ground motion during Kobe
- the Los Gatos Presentation Center ground motion during Loma Prieta

No peak ground accelerations are given for the ground motions used.

Linear springs were included in the model as connection elements to obtain connection forces. The two components of ground motion were applied in the principal directions of the building. The base shears obtained were:

Ground Motion	Base Shear per 1997 UBC (k)		Base Shear per SAP 2000 Analysis (k)	
	E-W	N-S	E-W	N-S
Lucerne	247	247	880	340
Takatori Station	247	247	2300	390
Los Gatos	247	247	1667	425

The Los Gatos ground motion had a significant amount of excursions above the design base shear, and significant nonlinear behavior is expected.

Connection demands for each pulse ground motion are as follows:

	GLB to Pilaster			Purlin to Pilaster/Wall Connection		
	Axial (k)	Vertical Shear (k)	Horizontal Shear (k)	Axial (k)	Vertical (k)	Horizontal (k)
Estimated Capacity	26/34	Not given	Not given	Not given	Not given	Not given
Lucerne	26	5	36	24	5.5	44
Takatori Station	69	7.5	90	23	14	99
Los Gatos	46	6	60.5	22	10	72

	In-plane Wall Shear (k/in.)	Out-of-plane Wall Moment (k-in./in.)	In-plane Diaphragm Shear (k/in.)
Estimated Capacity	1.74	3.0	0.085
Lucerne	0.42	8.1	0.360
Takatori Station	0.90	10.4	0.900
Los Gatos	0.60	12.6	0.600

NONLINEAR ANALYSIS

Analyses up to this point indicated that components of the tilt-up building may experience nonlinear behavior during moderate or strong earthquakes. To investigate the demands on the building, a nonlinear model of the building was constructed. To model inelastic behavior in components, bilinear springs were used as replacements for the elastic members. Bilinear springs were placed at the connections between the walls and the roof. The nonlinear properties of the spring were taken from the UCI tests by Pardoen. The horizontal shear component of the connections were taken as elastic, since no experimental data was available.

Representative input for connections is similar to the old type GLB and purlin connections:

GLB to pilaster: P_y = 13 k

	Elastic Stiffness, K_1 (k/in.)	Inelastic Stiffness, K_2
Axial	60	0.134 K_1
Vertical Shear	60	0.134 K_1
Horizontal Shear	30	1.0 K_1 (linear)

Purlin to pilaster/wall: P_y = 3 k

	Elastic Stiffness, K_1 (k/in.)	Inelastic Stiffness, K_2
Axial	33	0.035 K_1
Vertical Shear	33	0.035 K_1
Horizontal Shear	30	1.0 K_1 (linear)

New connections:
GLB to pilaster: P_y = 34 k

	Elastic Stiffness, K_1 (k/in.)	Inelastic Stiffness, K_2
Axial	44	0.030 K_1
Vertical Shear	44	0.030 K_1
Horizontal Shear	30	1.0 K_1 (linear)

Purlin to pilaster/wall: P_y = 18 k

	Elastic Stiffness, K_1 (k/in.)	Inelastic Stiffness, K_2
Axial	120	0.035 K_1
Vertical Shear	120	0.035 K_1
Horizontal Shear	30	1.0 K_1 (linear)

Results from Big Bear, with old connections

	GLB to Pilaster			**Purlin to Pilaster/Wall Connection**		
	Axial (k)	**Vertical (k)**	**Horizontal (k)**	**Axial (k)**	**Vertical (k)**	**Horizontal (k)**
Inelastic Model	12	1.75	2.1	3.05	2.0	12
Elastic Model	13.5		20	5.5	3.0	20

	GLB to Pilaster			**Purlin to Pilaster/Wall Connection**		
	Axial	**Vertical**	**Horizontal**	**Axial**	**Vertical**	**Horizontal**
Displacement Demand	0.26 in.	Elastic	Elastic	0.17 in.	Elastic	Elastic
Ductility Demand	1.5	Elastic	Elastic	1.9	Elastic	Elastic

Results from Los Gatos, with old connections

	GLB to Pilaster			**Purlin to PilasterWall Connection**		
	Axial (k)	**Vertical (k)**	**Horizontal (k)**	**Axial (k)**	**Vertical (k)**	**Horizontal (k)**
Inelastic Model	19	4.7	5.5	13.2	5	29
Elastic Model	46	6.1	62	22	10	72

	GLB to Pilaster			**Purlin to Pilaster/Wall Connection**		
	Axial	**Vertical**	**Horizontal**	**Axial**	**Vertical**	**Horizontal**
Displacement Demand	1.0 in.	Elastic	Elastic	10 in.	Nonlinear	Elastic
Ductility Demand	4.0	Elastic	Elastic	100	Nonlinear	Elastic

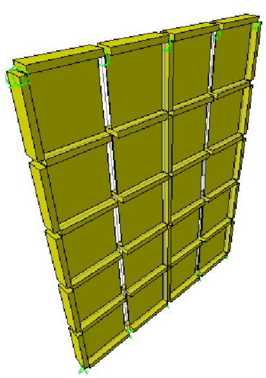
Results from Los Gatos, with new connections

	GLB to Pilaster			Purlin to Pilaster/Wall Connection		
	Axial (k)	Vertical (k)	Horizontal (k)	Axial (k)	Vertical (k)	Horizontal (k)
Inelastic Model	33	5.0	7.5	18.3	7	39
Elastic Model	46	6.1	62	22	10	72

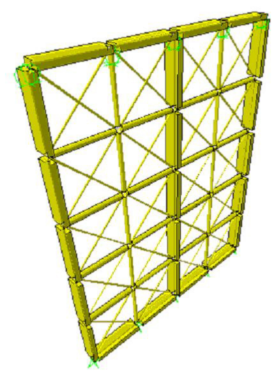
	GLB to Pilaster			Purlin to Pilaster/Wall Connection		
	Axial	Vertical	Horizontal	Axial	Vertical	Horizontal
Displacement Demand	1.0 in.	Elastic	Elastic	0.5 in.	Elastic	Elastic
Ductility Demand	2.0	Elastic	Elastic	3.3	Elastic	Elastic

NONLINEAR DIAPHRAGM

The previous elastic model also showed that the diaphragm should experience some inelastic behavior as well. To model the inelastic diaphragm an assembly of nonlinear beam and truss elements will be used. This model was suggested by Hrennikoff (Hrennikoff 1941), and is used to approximate the behavior of a 2D continuum. Although not identical, the model will provide a very good approximation. To calibrate the model, the data from the UCI diaphragm tests were used. Two models of diaphragms were chosen with a sparse nailing pattern of 6 in. o.c., and a dense nailing pattern of 2 in. o.c.



Diaphragm modeled with shells



Hrennikoff diaphragm model

The diaphragm was replaced with the Hrennikoff model by using the previous nonlinear model. The building period increased in both directions due to the more flexible connections from the UCI testing and diaphragm.

Model	N-S Period	E-W Period
Linear	0.39 s	0.29 s
Nonlinear	0.46 s	0.42 s

In a nonlinear static pushover analysis, there was little difference between the two connection types when the sparsely nailed diaphragm was used. The sparsely nailed diaphragm nonlinearity began around 50 k and was not capable of developing the necessary lateral force capacity of the building within 16 in. of displacement with either type of connection. The densely nailed diaphragm nonlinearity began around 150 k and with the new connections was able to develop the design strength in 8 in. of roof displacement.

A 3D time history analysis was run on the nonlinear model using the Landers ground motion. The displacement correlation is very good between the model and the recorded data.

Results from Big Bear, with dense nailing, and old connections

	Maximum Base Shear (k)	
	E-W	N-S
Code Shear	247	247
Nonlinear Model	220	60
Linear Model	480	60

	GLB to Pilaster			Purlin to Pilaster/Wall Connection		
	Axial (k)	Vertical (k)	Horizontal (k)	Axial (k)	Vertical (k)	Horizontal (k)
Inelastic Model	9	2.4	0.49	3.25	4.25	0.6
Elastic Model	13.5	1.6	19	5.5	2.8	20

	GLB to Pilaster		Purlin to Pilaster/Wall Connection		Diaphragm
	Axial	Vertical	Axial	Vertical	
Displacement Demand	0.18 in.	Elastic	0.26 in.	1 in.	0.078 in.
Ductility Demand	Weak NL	Elastic	2.2	7.6	4.5

Note: Horizontal component constrained to be elastic

Results from Los Gatos ground motion responses

	GLB to Pilaster			Purlin to Pilaster/Wall Connection		
Condition	Axial (k)	Vertical (k)	Horizontal (k)	Axial (k)	Vertical (k)	Horizontal (k)
Elastic	46	6.1	62	21.9	9.9	71.9
Old Connections	19	4.6	5.5	13.2	5	29
New Connections	34	5	7.5	18.5	7	39
Dense Nailing / Old Connection	18.0	8.5	2.8	4.9	11	2.8
Dense Nailing / New Connection	16.0	13.2	0.8	19.25	22	1.0
Sparse Nailing / Old Connection	24.0	6.9	6.5	3.9	10.1	7.0
Sparse / Nailing New Connection	35.0	7.0	6.8	21.0	26.0	7.0

Condition	Maximum Base Shear (k)	
	E-W (Transverse)	N-S (Transverse)
Code Shear	247	247
Elastic	1600	420
Old Connections	820	346
New Connections	540	220
Dense Nailing / Old Connection	810	340
Dense Nailing / New Connection	530	200
Sparse Nailing / Old Connection	310	150
Sparse / Nailing New Connection	455	160

Displacement ductility

	GLB to Pilaster			Purlin to Pilaster/Wall Connection			Diaphragm
Condition	**Axial**	**Vert.**	**Horiz.**	**Axial**	**Vert.**	**Horiz.**	**Shear**
Max Capacities	6.8/3.1*	---	---	13.5/9.4*	---	---	4.5
Old Connections	4.0	Elastic	Elastic	100	13.1	Elastic	Elastic
New Connections	2.0	Elastic	Elastic	3.3	Elastic	Elastic	Elastic
Dense Nailing / Old Connection	3.6	Elastic	Elastic	8.0	56.7	Elastic	18.3
Dense Nailing / New Connection	Elastic	Elastic	Elastic	3.5	7.4	Elastic	4.3
Sparse Nailing / Old Connection	5.7	Elastic	Elastic	9.0	48.0	Elastic	36.8
Sparse / Nailing New Connection	Elastic	Elastic	Elastic	3.5	5.0	Elastic	20.7

* Designates old ductility capacity / new ductility capacity

CONCLUSIONS

Analysis of Recorded Response:

- Significant amplification of base acceleration occurred between roof and base in the out-of-plane to the walls. The amplification may be as high as 5.65 times.
- Almost no amplification of the acceleration occurred at the base to the top of the in-plane of walls.
- There was little change in response over the entire period range for in-plane accelerations for different spectra. However, out-of-plane comparisons indicate amplification of accelerations for building periods of less than 1 second.
- At 65% reduction in stiffness, the transverse period increased from 0.4 s to 0.67 s, and the longitudinal period increased from 0.35 s to 0.75 s —all occurred due to cumulative shaking of the building.

3D Elastic Dynamic Analysis:

- The SAP 2000 model gave very close approximations to the fundamental modes derived from field data.
- The participating mass was 46.3 % in the transverse direction and 15.5% in the longitudinal direction. The base shear is equal to the mass participation times the base acceleration, so the base shear in the transverse direction will be almost three times larger, which is seen in the models.
- Comparisons between the recorded accelerations and displacements and calculated ones showed good agreement for all four recorded ground motions.
- The Palm Springs and Northridge base shear was well within the code base shear, while the Landers and Big Bear base shears exceeded the code shear a limited number of times.
- The in-plane shear force demands of the walls were more than twice the code requirements; however, calculations indicate substantial overstrength for in-plane shear. The out-of-plane moment demands were almost half of capacity. The connection demands were within capacity, so the diaphragm is a critical component of the system.
- A site visit indicated no continuity at corners, but this has a limited effect on accelerations and displacements. Although the base shear was increased in the north-south direction by 30%, there was a significant increase in the out-of-plane moment in the corner panels.
- Pulse-type ground motions place extreme demands on buildings. All demands were significantly higher than the code requirements. The only adequate aspect of building was in-plane strength of walls.
- The axial force in the GLB to pilaster connection at the mid-point of the wall may be as much as 2.7 times the value at the end of the wall.

3D Nonlinear Static and Dynamic Analysis:

- The Hrennikoff model was used to represent the roof diaphragm, and the major connections of the diaphragm to the walls were modeled using nonlinear springs. The walls were kept as elastic nonlinear elements. The out-of-plane bending demand was reduced to near-capacity values, and the model was effective in predicting displacement and ductility demands.

- The axial force in GLB at mid-length was reduced to a maximum of 1.7 times, a 37% reduction from elastic.
- The Big Bear ground motion had the maximum axial demand in the GLB connection with a displacement demand of 0.26 in. and a displacement ductility demand of 1.5. The purlin to pilaster displacement demand of 0.17 in. and displacement ductility demand of 1.9.
- Under pulse-type ground motions the old GLB connections yielded an axial displacement demand of up to 1 in., with a displacement ductility demand of 4. The purlin to pilaster connection had an axial displacement of 10 in. with a displacement ductility of 100. Connections along the shorter side have high in-plane shear forces that are concurrent with axial force.
- The new GLB connections had the same displacement demand, but displacement ductility was reduced to 2. The new purlin to wall connection reduced the displacement demand to 0.5 in., and ductility demand to 3.3.
- By using a pushover analysis and the nonlinear Hrennikoff truss model, a close approximation to test results can be obtained.
- The maximum base shear decreased below the code required, and all old connections to GLB remained elastic or very weakly nonlinear. The purlin connections had maximum displacement of 1 in. and ductility of 7.6, and the nonlinear Hrennikoff elements had a displacement demand of 0.78 in. and ductility of 4.5.
- With the densely nailed nonlinear diaphragm, and old connections, the base shear in the E-W direction about was about 2.4 times the code value. The GLB connection displacement demand was 0.89 in. and the ductility was 3.6. These values are less than those obtained from the UCI tests. The purlin connection had a displacement demand of 1.2 in. (close to test value) and a ductility = 5.2 (much lower than the 13.2 from tests).
- With the densely nailed, nonlinear diaphragm and new connections, the base shear is almost equal to the code requirement. All GLB connections remained elastic and purlin demands were reduced. The ductility demand of the diaphragm was 4.3, which compares

well to tests. Buildings with these characteristics should be able to resist a pulse-type earthquake.

- The diaphragm with sparse nailing and old connections had a base shear of about 1.7 times code. The GLB connection displacement and ductility demand was less than achieved in tests. The purlin connections were critical and high displacement and ductility demands on the roof diaphragm may not be sustainable.
- The diaphragm with new connections and sparse nailing had the base shear reduced so that only a few excursions were above code base shear. The GLB connections remain elastic and the purlin connection demands are lower. The diaphragm ductility demands are again high and may not be sustainable.

RECOMMENDATIONS

- Connection Testing
    - Only axial tests were performed, but no vertical and horizontal shear tests. These tests are needed.
    - The connection of the GLBs directly to walls, with no pilasters, needs to be studied.
    - Nailed connections tend to loosen under cyclic loads; this aspect needs to be tested.
- Diaphragm Testing
    - There is a lack of documentation for the failure modes of the UCI tests.
    - There also was no evaluation of repeatability of results.
    - More tests are needed.
- Pushover Analysis
    - The analysis can be dependent on location and magnitude of lateral forces, since the diaphragm may not be capable of delivering forces to the lateral-resisting elements.
- Tilt-up Wall Panels
    - The study focused on buildings with pilasters between TUW panels; current buildings do not have pilasters, merely a chord connection at top and possibly a steel splice plate at mid-height.
    - A detailed analysis of this type of building is beyond the scope of the report. However, a simple analysis run showed no significant changes.
- Instrumentation Program
    - Currently there are no buildings instrumented with segmented panels, so no data exist for comparison.

- Vertical Accelerations
    - The roof response to a vertical ground motion may have a significant effect on the GLB and purlin connection, but this was beyond the scope of the study. This response may influence connection design and should be investigated.

Appendix E Calculation of a Tilt-up Building Period Using FEMA 356 and Dynamics

Building periods can be estimated several different ways, but one equation will not calculate the period for every building type. This section will provide a sample calculation by using Equation 3-8 of FEMA 356 and common dynamics equations.

E.1 PERIOD PER FEMA 356:

FEMA 356 provides a method specifically for tilt-ups, and gives a more reliable answer than by using the simplified method presented in most codes for all building types. For buildings with single-span flexible diaphragms the period can be estimated by the FEMAapproximate period equation, Equation 3-8:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}$$

where:

Δ_w = In-plane wall displacement due to the weight tributary to diaphragm

Δ_d = In-plane diaphragm displacement due to the weight tributary to diaphragm

Note: When calculating Δ_d the diaphragm shall be considered to remain elastic under the prescribed lateral loads.

If we assume the tilt-up walls to be solid with few openings, they will be very stiff. Therefore, Δ_w will be very small and its contribution can be neglected. That leaves us with:

$$T = (0.078\Delta_d)^{0.5}$$

To calculate the deflection of the diaphragm due to the tributary weight, we first need to define the stiffness of the diaphragm. This can be accomplished by using Equation 8-4 of FEMA 356:

$$\Delta_y = \frac{5v_y L^3}{8EAb} + \frac{v_y L}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

where:

A = Area of diaphragm chords cross section, in.^{2}
b = Diaphragm width, ft
E = Modulus of elasticity of diaphragm chords, psi
e_n = Nail deformation at yield load per nail, in.
G = Modulus of rigidity of wood structural panels, psi
L = Diaphragm span, distance between shear walls or collectors, ft
t = Effective thickness of wood structural panel for shear, in.
v_y = Shear at yield in the direction under consideration, lb/ft
Δ_y = Calculated deflection of diaphragm at yield, in.
$\Sigma(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support

If we assume that the tilt-up walls will be acting as diaphragm chords, the first and last terms can be neglected. The first term, $5v_y L^3/8EAb$, deals with the chord deflection. Since the chord is a portion of the tilt-up panel, and the panel is very stiff, the deflection will be negligible. The last term, $\Sigma(\Delta_c X)/2b$, deals with the chord-splice slip. If we assume the panels are attached together rigidly with pilasters, the chord-splice slip will be negligible as well. This leaves us with:

$$\Delta_y = \frac{v_y L}{4Gt} + 0.188 L e_n$$

The yield load of the diaphragm, V_y, is equal to $2v_y b$. By using this and the yield deflection, we can calculate the diaphragm stiffness, k_d.

$$k_d = \frac{V_y}{\Delta_y}$$

Using the diaphragm stiffness, the deformation of the diaphragm under the tributary load can be calculated.

Using the values provided by Hall in his report, a period using FEMA 356 can be obtained.

e_n = 0.08, 8d nails assumed
G = 90,000 psi
L = 200 ft
t = 0.278 in., 3/8″ unsanded plywood assumed

v_y = Hall does not determine the diaphragm strength from the diaphragm tables, but rather determines a design working stress for the diaphragm of 670 lb/ft from the 1994 UBC. Using the *Allowable Stress Design Manual for Wood Construction* the diaphragm configuration that most closely corresponds to the design working stress is a 3/8″ plywood diaphragm using 8d nails with 2″ o.c. boundary nailing, and 3″ o.c. nailing at other panel edges, and 3x framing provided throughout. This corresponds to a recommended shear of 675 lb/ft. Using this same diaphragm configuration the *Load Resistance Factor Design (LRFD) Manual for Wood Construction* (AF&PA / ASCE Standard 16-95) gives a factored shear resistance of 880 lb/ft. Converting this to an expected value, v_y, by using FEMA 356 Section 8.3.2.5 we obtain:

v_y = 675 lb/ft / 0.65 = 1038 lb/ft.

$$\Delta_y = \frac{1038\, lb/ft \times 200 ft}{4 \times 90{,}000 psi \times 0.278\, in} + 0.188 \times 200 ft \times 0.08 = 5.08\, in$$

$$V_y = 2 \times 1038\, lb/ft * 80 ft = 166.1\, k$$

$$k_d = 166.1\, k / 5.08\, in = 32.7\, k/in$$

According to Hall the weight tributary to the diaphragm is 585.4 k. Using the stiffness and the weight, the elastic deflection can be obtained:

$$\Delta_d = 584.4\, k / 32.7\, k/in = 17.87\, in$$

Plugging this into the approximate period equation we obtain:

$$T = (0.078 \times 17.87\, in)^{0.5} = 1.18\, s$$

E.2 PERIOD PER DYNAMICS:

The period can also be obtained by assuming that the diaphragm acts as an oscillating, simply supported beam of uniform mass and stiffness. The circular frequency can be determined and then the period from the frequency. Using an equation from *Dynamics of Structures; (2nd ed.)* Section 8-6, Example E8-4, the simple dynamics equation for the circular frequency of an oscillating spring is:

$$\omega^2 = \pi^4 \frac{EI_{eff}}{\overline{m} L^4}$$

Where:

ω = Circular frequency of the diaphragm, rad/s
E = Elastic modulus of the diaphragm, ksi
I_{eff} = Effective moment of inertia of the diaphragm, in.^{4}
$\overline{m}$ = Distributed mass of the diaphragm, k·s^2/in.^{2}
L = Length of the diaphragm, in.

The effective moment of inertia of the diaphragm can be back-calculated from the diaphragm deflection under a known load and the equation for the deflection of a simply supported beam:

$$\Delta = \frac{5wL^4}{384EI_{eff}}$$

Where:

w = Distributed load on the diaphragm, k/in.

The procedure for calculating the diaphragm deflection is similar to the procedure using in the FEMA 356 calculation. However, the diaphragm deflection from the *LRFD Manual for Wood Construction* will be used instead. This equation does not calculate the deflection at yield but rather any deflection under an applied load. The diaphragm deflection, Equation C9.5-1, is as follows:

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

Where:

A = Area of the chord cross section, in. ^{2}

b = Diaphragm width, ft

E = Elastic modulus of the chords, psi

e_n = Nail deformation, in.

G = Modulus of rigidity of the plywood, psi

L = Diaphragm length, ft

t = Effective thickness of plywood for shear, in.

v = Maximum shear due to unfactored design loads in the direction under consideration, lb/ft

Δ = Calculated deflection, in.

$\Sigma(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by it's distance (ft) to the nearest support

e_n = $(V_n / 857)^{1.869}$ for 8d nails in Green / Dry Wood

And V_n= load per nail

Since this equation does not yield a linear stiffness, the maximum value allowed for the nail deformation, e_n, will be used.

V_n max = 220 lb/nail

This corresponds to a load placed on the diaphragm ends of:

v = 220 lb/nail x 6 nails/ft = 1.32 k/ft

With two sides, each 80 ft long:

V = 1.32 k/ft x 80 ft x 2 = 211.2 k

And:

$e_n = (220/ 857)^{1.869} = 0.079$

Using the same assumptions that the tilt-up panels will act as the chords, the deflection equation reduces to:

$$\Delta = \frac{v\ L}{4Gt} + 0.188 L e_n$$

$$\Delta = \frac{1320\ lb/ft \times 200\,ft}{4 \times 90{,}000\ psi \times 0.278\ in.} + 0.188 \times 200\,ft \times 0.079 = 5.61\ in.$$

Using this deflection for the deflection of a simply supported beam, the effective moment of inertia can be found:

$$\Delta = \frac{5wL^4}{384EI_{eff}}$$

Where:

$$w = \frac{211.2\ k}{200\,ft \times 12\,in./ft} = 0.088\ k/in.$$

$$L = 200\,ft \times 12\ in./ft = 2400\ in.$$

$$E = 1{,}700\ ksi$$

$$5.61\ in. = \frac{5 \times 0.088\ k/in. \times (2400\,in.)^4}{384 \times 1700\ ksi \times I_{eff}}$$

$$I_{eff} = 3.986 \times 10^6\ in.^4$$

Using the effective moment of inertia in the dynamics equation:

$$\omega^2 = \pi^4 \frac{EI_{eff}}{\overline{m}L^4}$$

Where:

$$\overline{m} = \frac{584.4\ k/2400\ in.}{386.4\ in./s^2} = 6.302 \times 10^{-4}\ \frac{k \times s^2}{in.^2}$$

We obtain:

$$\omega^2 = \pi^4 \frac{1700\ ksi \times 3.986 \times 10^6\,in.^4}{6.302 \times 10^4\ \frac{k \times s^2}{in.^2} \times (2400\ in.)^4} = 31.572\ rad^2/s^2$$

$$\omega = 5.619\ rad/s$$

Using simple relationships, the building period, T, can be obtained:

$$T = \frac{2\pi}{\omega}$$

$$T = \frac{2\pi}{5.619\ rad/s} = 1.12\ s$$

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