INTRODUCTION

The City of Los Angeles Department of Building and Safety and the Structural Engineers Association of Southern California formed eleven subcommittees following the Northridge Earthquake. The purpose of each subcommittee was to review the seismic response of various building types and suggest any necessary code revisions to improve their performance. The Wood Frame Subcommittee made recommendations which eventually became interim law. Consistent interpretation of the interim rules and a basic understanding of their purpose is the goal of this training.

SCOPE

On April 22, 1994 the emergency enforcement measures for wood frame construction became effective. The original regulations reduced the allowable shear capacity and maximum permitted height to width ratios for drywall, stucco and plywood shear walls. Some panel edge members were required to be a minimum of 3 inch nominal width and increased detailing was required on the plans. The use of rotation to distribute diaphragm loads was also prohibited.

On May 20, 1994 the original measures were expanded to require common nails on plywood shear walls permitted after May 15,1994 and use reduced values for holdowns. Exceptions to some conditions were also allowed for non shear wall designated portland cement plaster walls, 3 inch width requirements, and one story non-habitable structures. Some plan requirements were further clarified.

On November 30, 1994 new provisions were added. Deputy inspection was required for higher shear strengths and values for 3-ply plywood panel construction were reduced. Larger plate type washers were required on the studs of holdowns and the sill plates of higher strength walls. The inter-departmental memos are attached in the Appendix[A1-6].

PLYWOOD SHEAR WALLS

The single most important part of the lateral force resistance system for most wood frame buildings is the shear wall. Horizontal diaphragms usually do not exceed their unblocked capacity in these buildings. The shear wall however is designed close to its allowable capacity for the given load. It is the essence of the lateral system and should get our closest scrutiny as code enforcers.

Most engineers choose a shear panel whose allowable value in Table 25 K-1[1] slightly exceeds the demand and then design the connections for the shear panel capacity instead of the calculated lateral load. This enables the structure to get full value out of the panel and assumes that all the factors of safety are equal for the wood and its connections. In fact, they are not, but this will remain the most frequent design technique until wood has commonly accepted load factors for all of its components.

The nail type, size and spacing usually determine the accepted strength of the shear panel for most wood frame construction. The shear capacity of the plywood can govern the allowable strength when the shear approaches 1000 lbs/ft. The tabulated design values in the code, all less than 1000 lbs/ft, are based on the lateral capacity of the nail and were modified to account for other factors developed in testing[12]. These other factors include the grade and thickness of plywood and the spacing and width of supports. See the attached sample calculation of shear wall capacity in the Appendix[A7].

HISTORY OF PLYWOOD SHEAR WALL TESTS

Previous American Plywood Association (APA) tests on shear walls were done with duplex, full and shortened common, and hot dip galvanized box and casing types of nails. Eight foot by eight foot plywood panels were fastened to mostly dry douglas fir lumber and loaded in one direction in a non-cyclic pattern.The tests were typically done in three ranges. The first was to the calculated allowable strength based on nail capacity. The second test was to twice this capacity. The final test was to failure of the plywood, its attachment or the boundary member. Summary of the test results are included in the Appendix[A8-10].

Some important observations were made about the failure mode in these tests. Most panel failure consisted of the fastener pulling out of the panel at its edges. Frequently the plywood would tear out. Sometimes the sheet would buckle and pull the entire nail out in withdrawal. Few cases showed actual shearing of the nail despite the capacity being so calculated. Also common and box nails of the same pennyweight had the same ultimate capacity. A noticeable drop in capacity was observed for the casing type nail. Conclusions were drawn from the observations and incorporated into the limits of Table 25-K-1.

Hot dip galvanized box nails were found to be nearly equivalent to common nails after nearly 60 additional tests.[12,A10a] The 8d box size showed equal strength to the common but had 25% more slip. Hot dip galvanized casing nails were found to be 60% as strong as both box or common. The size of the nail head became the important issue. It was concluded that the similar head size of the box and common explained the same strength and the smaller head of the casing nail, its reduction. Therefore panel configurations could use either galvanized box (hot dip) or common in the major part of the Table. Reduced values were shown for casing type nails. See the original Table 25-K-1 attached[A11].

Modeling Problems with the Previous Testing

Questions have arose about the propriety of these past static tests[18]. Recent independent testing suggests strength loss occurs at narrower panels[19]. Removal of the bearing provided by the test jig at the panel edge has already been shown to reduce capacity up to 30%[12]. Additional stiffness losses occur due to green lumber, box nails, edge restraint slippage and panel rotation. Green lumber has four times the nail slip of seasoned lumber[2]. Box nails have 25% more slippage than commons at the 8d pennyweight[12]. Slippage of holdown devices and only partial restraint at top plates of non-bearing walls allow additional rotation and deflection. The in place seasoning of green lumber creates slip at holdown anchors[20]. All of these issues raise questions on the completeness of the past tests as a model for present construction methods, system response and dynamic loading.

Present design standards better regulate stiffness criteria. Past tests dealt mostly with static strength concerns. Shear wall construction of today also requires careful and realistic analysis of deflection. Some engineers would be surprised to learn that their adequately strong panels also deflect 3 inches under actual seismic loading ! Proposed testing must include strength and stiffness concerns for a variety of panel constructions. The effect of green lumber, thinner nails, clipped nail heads, free panel edges, height to width ratio and anchorage slippage must be all independently investigated for their effects on strength and stiffness. The beneficial effects of larger headed nails and increased edge distances for fasteners for plywood and framing members should also be studied. *

The poor performance of some panels is suspected to be caused by dynamic degradation. Current effort is being made to specify acceptable dynamic test criteria (number of cycles, load duration and variation of intensity) to study degradation. In the interim, the subcommittee has recommended the use of the common nail, 75% of the UBC shear wall table values and a maximum 2:1 panel aspect ratio. All 3-ply panel construction which is less than 5/8 inch thick is also limited to an allowable of shear capacity of 200 lbs/ft.

NAIL SUBSTITUTIONS

Nail capacity (Z) is based on the diameter (D) for a given wood species group[3]. Douglas fir larch in group II has a capacity equation of $Z_{lbs} = 1650\, D_{in}^{3/2}$. Existing code[1] allowed the use of common or galvanized box nails on shear walls. Galvanized nails today are typically either electro- galvanized (EG) or hot dip galvanized (HDG) of the box type. Only the HDG has a noticeably improved diameter after processing and was the type used in previous plywood testing[12]. The common nail has the largest diameter of all and is 24% stronger in shear than the screw, ring or plain shank box or cooler nails in the 6d, 8d and 10d sizes. This is one reason the common nail has been selected by the subcommittee as the nail of choice for plywood shear walls. Reduced slippage is also a benefit.

Substitutions for the Common Nail

The substitution of common nails has occurred on construction sites for decades[9]. Floors are frequently fastened with wood screws, ring or screw shank cooler nails. Plain shank fasteners are not recommended on floor applications due to squeaking although short commons are sometimes used. Roofs and shear walls are nailed with coolers or short commons. Typical substitutions are short commons for 10d and coolers for 8d commons.

Framing members are connected with 16d sinker nails. All of these nails have different lengths or diameters (or both) than the common nail. See the attached Table 25G, UBC STD Table 25-17-H and excerpt from Federal Spec. FF-N-105B for further details[A12-13].

Problems with the Use of Nail Guns

Most plywood nailing of any volume is done by pneumatic tools commonly referred to as nail guns. The nails used in these tools may or may not have full heads. Some problems have occurred in the past when full headed nails were used in clips. The large bits of plastic required between nails sometimes broke up and jammed in the tool. For this reason some nail manufacturers clip the nail head to allow the nail shanks to be side by side and use glue and/or paper to form the clips. Other manufacturers have increased the plastic strength in the nail clip and continue using full head nails.

Some engineers are concerned that the reduced head size will affect the shear and slip capacity of the shear wall. This point has merit because full headed nails did perform better in shear wall tests than smaller headed nails such as the casing type[12]. However, current code allows these mechanically driven nails to have either T shaped or altered round heads[2]. The diameter of the shank and head must match Table 25-17-H for type but the tolerance for head variation is left up to the manufacturer This appears to contradict the ± 10% variation limit on head size. There are proposed code changes to require full head nails to eliminate this confusion and improve the connection strength.

Another area of concern for mechanically driven nailing is the difficulty in flush driving of the nail to the plywood surface. Air pressure fluctuates during tool use and it affects the driving pin of the tool. Excessive air pressure can overdrive the nail and low pressure will leave the nail above the surface. Careful setting of the operating range of the regulator on the air compressor is necessary for proper nail driving. Some nail guns require adjustment of the tool itself or an attachment to perform flush nailing.

Use of the Shortened Common Nail

Shortened common nails, referred to as short or plywood nails, may normally be substituted for full length common nails when allowed by the engineer or architect. They have tested equal in strength to full length nails and are less likely to split framing members in high strength applications[13]. For these reasons, they do not require wider framing members in the 10d at 3 inch spacing while full length 10d common nails do[1].

Shortened commons are designed to give the minimum required penetration for plywood nailing into Group II framing members. They come in three sizes: 2⅛, 2¼ & 2 ⅜ x .148 (10d common diameter) for use respectively on ½, ⅝, and ¾ inch plywood sheathing. The nails can be used on shear walls but with some limitations.

The required penetration for full lateral load value on a nail is 11 diameters for douglas fir larch (Group II), 13 diameters for hem-fir (Group III) and 14 diameters for California Redwood-Open Grain (Group IV). This is 1⅝, 1⅞, and 2⅛ inches respectively for the 10d diameter. Shortened commons will generally provide less than the required penetration for full value on redwood and hem-fir sill plates. Allowable shear wall capacity must be reduced due to the limited nail penetration in addition to the reduction for Group III (18%) or Group IV (35%) species framing members.

Shortened commons are not suitable for all conditions. They should not be used on plywood over existing space sheathing. The recommended method is to use full length 10d common or ring-shank nails into the framing members[A14-15]. Shortened commons are not recommended on roofs in general due to the loss in withdrawal capacity. The minimum recommended nail is the 8d common for even basic wind uplift with increased nail spacing for intermediate and high wind areas[25].

Use of the 16d Sinker Nail

The use of the 16d vinyl coated sinker nail is typical in wall, floor and roof construction. The general construction nailing requirements of the building code allow the use of common or box nails for these connections. See the attached Table 25-Q[A16]. The 16d sinker has a diameter between the box and common[23]. Although its length is ¼ inch shorter, it has the required penetration for full value when nailing the typical nominal 2 inch members together. It is 15% stronger in shear than the 16d box and is 87% of the 16d common shear strength[3] in these applcations. For these reasons, it is a suitable substitution and normally should be allowed. However an engineer or architect may require common type nails for all connections. Substitution of any material on a design by an architect or engineer requires their approval under registration, copyright and liability considerations.

But the use of the 16d sinker does affect some designed connections. The nailed sill plate of a shear wall, top plate chord splices and roof ceiling ties are common examples. Substitution of the 16d sinker for the 16d common can reduce the capacity of a nailed sill plate some 25% after considering penetration depth factor reductions. The reduction for the ties and splices is only 6%. As a side note, the 10d common has the same diameter as a 16d sinker. Values for lateral loads on 16d sinkers may therefore use the base value for the 10d common found in Table 25-G. This equivalent substitution is also authorized in some Simpson[21] manufactured connections.

LUMBER SPECIES SUBSTITUTIONS

The substitution of lumber becomes significant on plywood shear walls. The common field substitution of hem fir (Group III) for douglas fir (Group II) on pressure treated sill plates reduces the allowable shear capacity of the plywood wall 18%[1]. It also reduces the allowable load on the sill bolt from 5 to 11% depending on bolt size and sill thickness[2]. ✱
Plan check engineers and inspectors must check the species specified for the sill plate.

A large local lumber yard chain has reported that some 66% of the pressure treated and 20% of the standard sill plates purchased in the City of Los Angeles are **hem fir**.

Retrofitting of existing older buildings also frequently finds foundation grade redwood sills. The proper plywood shear capacity reduction for California Redwood (Open Grain) is 35%. The allowable sill bolt capacity is reduced from 11 to 15% in the full 2 inch width. Values for California Redwood (Close Grain) are somewhat better (18% reduction) but a lumber grader from an approved agency would have to certify this grain status on site.

SILL PLATE BOLTING

The anchorage of the sill plate itself is of major concern. Oversized holes drilled in the plate affect the quality of the joint by allowing excessive slippage. The code prescribes a maximum 1/16th and a minimum 1/32 inch allowable oversize hole[2]. Many contractors use larger holes for ease of installation of the wall or because the anchor bolts are not set or kept vertically plumb during concrete finishing. Inspectors can help reduce the problem by requiring all sill anchor bolts to be firmly attached to the form work before giving approval to pour the concrete. Contractors can also use the future sill plate as a template to maintain bolt position and determine end piece locations.

Construction errors or seismic upgrading may create oversized holes. Retrofit applications may require oversized holes for installation of mechanical or chemical anchors through existing plates. Oversized washers must be used when the hole in the sill is more than 1/16th inch larger than the bolt diamter unless the hole is filled with the chemical bonding agent. The washers that come with proprietary assemblies may need to be replaced.

The standard washer for sill plates has been a cut circular type. These "cut" washers are no longer allowed at the sill plates of shear walls with design loads exceeding 300 lbs per foot. Oversized washers must now be used instead to improve the shear capacity of the connection. See the following table and the attached test results and capacity chart.[A17-18]

BOLT DIAMETER$_{inch}$	PLATE SIZE$_{inch}$	THICKNESS$_{inch}$
1/2	2 X 2	3/16
5/8	2½ X 2½	1/4
3/4	2¾ X 2¾	5/16
7/8	3 X 3	5/16
1	3½ X 3½	3/8

MOISTURE CONTENT FASTENER REDUCTIONS

A common design error in wood frame connections is the omission of the moisture content reduction for fastener load capacities. The use of green or partially seasoned lumber affects the quality of mechanical connections as they dry. For nails, the typical reduction is 25%[2]. Most new construction uses surfaced green lumber because the lumber is exposed to moisture during construction. Connections of sill plates and doubled panel edge members should be designed accordingly. This includes Simpson Strong Tie Products (like A35F & MST straps) which are based on continuously dry conditions[21]. See UBC STD. Table 25-17-R. attached [A19].

NAILED SILL PLATE CONNECTIONS

Some engineers are under the mistaken belief that the code minimum sill plate nailing over wood floors is also sufficient at the base of shear walls. This is not the case. Even a 16d common at the 16 inch spacing is good for only 64 lbs./ft when nailed through a 5/8 inch plywood subfloor ! The limit of a nailed sill plate connection must be considered carefully since its capacity may be far below the actual shear wall strength. See the attached sample calculations and charts. [A20-22]

The new requirement for 3 inch nominal sill plates has caused some problems with new and exisiting construction. Large diameter nails, screws or bolts are required to nail down the sill plate over a wood floor. The larger nails require 3 inch or 4 inch minimum nominal width framing and the use of bolts or screws requires a lot of predrilling. In order to allow the use of the 16d nails to continue to attach sill plates, 3 inch nominal members will only be required on sill plates over concrete. This policy is in keeping with the new proposed code change. See the attached item # 18.[A23]

It is important to also note that the standard framing anchor cannot be used to attach the sill plate because some of the required nails would be in the plywood edge grain of the subflooring. Some help may be available if the engineer specifies a special length sill plate which extends into non-shear wall sections of the wall for the added nailing required. However, this solution is not available on shear walls with openings on each side that extend to the floor.

DOUBLE EDGE MEMBERS AND PANEL EDGE DISTANCE

Required shear wall strengths in excess of 300 lbs/ft now require nominal 3 inch wide framing members or blocking. An exception will be allowed for existing buildings to add another member to the original plate or stud. The design of the double stud or plate at panel edges must consider the nailing pattern on the panel. The connection of the two members must develop the same capacity as the boundary edge nailing and must be specified on the plans.

The minimum edge distance for nails was increased from ⅜ inch to ½ inch for all required 3 inch nominal or 2-2 inch nominal members. The present requirement in the code for 3 inch nominal width members at decreased nail spacing is to prevent the splitting of the framing member at ultimate capacity. Any plywood nailing less than 4 inches on center may also split the framing member during fabrication and should therefore be avoided when possible. See the attached table and sample calculation.[A24-25]

DIAPHRAGM TO SHEAR WALL CONNECTION

The connection of shear panels to roofs and floors may require the use of framing anchors or special length plywood due to the limited strength of a nailed connection to the available framing. Both methods are acceptable for engineering purposes. The mechanical connector is the more common and the preferred method for new construction. The connector should flushly attach half on the plate and half on the rim joist or blocking. The long dimension should be horizontal and not be nailed into the end grain of a joist or rafter. The connector may also be nailed into the side of the support member and the top plate but is more difficult to install in the field if the diaphragm is already sheathed.

Special length plywood may also be used in lieu of framing anchors or nailed sill plates if two lines of boundary nailing are provided. The boundary nailing is required at the plate and the plywood edge. Nails must be in blocking or a rim joist and should be staggered to prevent splitting during application. APA does not recommend this method with unseasoned lumber which is typical on new construction. Cross grain shrinkage of the rim joist or blocking may cause the plywood to buckle and then crack brittle finishes like portland cement plaster or masonry veneer. For this reason APA recommends a panel break at both plates for stucco walls and the use of dried lumber if the panel is attached to the joist or block. See the attached APA detail[A26]. Retrofit applications may omit this recommendation at the designer's option when the lumber has seasoned in place and will remain continuously dry. Some practicing engineers use a special detail[A27] for new construction that uses a stucco joint screed at the horizontal plywood joint. The plywood joint is spaced to account for the calculated shrinkage of the joist or rafter size.[19]

PLYWOOD PANEL CONSTRUCTION AND GRADE

A vast majority of existing plywood shear walls are constructed with 3/8 inch 3-ply C-D Exposure 1 panels. The panel is commonly referred to in the trades as 3/8 inch CDX. Designers have frequently chosen this panel for economic reasons. The Table 25-K-1 allowable value for 3/8 inch C-D grade is the same as the more expensive 15/32 when using the same nail size and spacing. Increased capacity can only be obtained by upgrading to the more expensive Structural 1 panels. Use of the thinner 3/8 plywood can also avoid extra furring required for wall finishes like portland cement plaster.

The performance of 3-ply construction has raised questions of its ultimate capacity. Horizontal tearing has occurred on some outer face plies above the inner ply seam. Values for all 3-ply panel construction were therefore reduced to 200 lbs/ft maximum. See the revised Table 25-K-1 attached[A28]. Panel values for 4 & 5 ply construction were not reduced because additional parallel plys reduce the chance of a local defect affecting the strength[28].See the attached sketch.[A29] The following panel information was furnished by a major plywood wholesaler and conforms to UBC STD TABLE 25-9-C (attached[A30]).

3 ply panel construction is used on 3/8 Struc 1, 3/8 C-D Exp 1, 1/2 C-D Exp 1,
4 ply panel construction is used on 1/2 Struc 1 (Southern Pine[1]), 5/8 C-D Exp 1[2]
5 ply panel construction is used on 1/2 Struc 1 (Douglas Fir), 5/8 Struc 1

1. Struc 1 in 1/2 inch thickness is normally available in 4-ply (Southern pine) because it is less expensive than the 5-ply (Douglas fir)
2. C-D Exp 1 allows 3 ply in 5/8 thickness under UBC STD 25-9 but manufacturers typically use 4 plys to obtain the required thickness

UBC Standard 25-9 based on US Product Standard 1-83 defines panel requirements for construction and industrial plywood. Some confusion has existed about the listed grades Structural 1 and Structural II. Some designers wrongly assume that Structural II is all plywood other than Structural 1. This is not the case and in fact Structural II is rarely manufactured[28]. The alternate specification to Structural 1 is normally Rated Sheathing[27].

The Structural 1 performance grade has enhanced racking and cross-panel strength properties. It uses only Group 1 species which gives it up to 46% increase in plywood shear through the thickness capacity over C-D Exposure 1 (APA Rated Sheathing). It is used on high strength diaphragms when the plywood shear strength governs the design instead of the allowable nail capacity.

Structural 1 is not always specified on shear walls. The frequently used UBC table values only give a 10% strength increase over all other performance grades. Structural 1 costs from 15% (3/8 inch) to 40% more (1/2 & 5/8 inch) than the normal alternate, CD Exposure 1. This averages only about $5 more a sheet and is a small investment in quality. The actual total benefit exceeds the 10% strength increase especially when you consider the reduction in nail slip (17%) and plywood shear (25%) deflections.[2]

PANEL HOLDOWNS

Bolted holdown connections were poorly installed in many buildings damaged by the Northridge Earthquake and the allowable values used in their design are based on non-cyclic static tests on steel jigs. For these two reasons, the values were reduced 25% and oversized plate washers are required on the vertical edge members. Inspectors are also to verify the 1/16th inch maximum oversized hole in the vertical edge member. The 1/16th maximum and 1/32 inch minimum oversize holes are already a code requirement for all bolted wood connections. The holdown bolt hole in the sill plate should be oversized to prevent combination shear and tension loading. A 1/2 inch oversized hole is recommended (1¼ inch max). Because unseasoned lumber will begin to shrink during construction, one last tightening of the holdown anchor bolts is required before permission to cover the rough frame is granted.

Use of the oversized plate washers and one last tightening of the bolt is an attempt to reduce the vertical slippage associated with bolted holdown devices. This often overlooked component of shear wall deflection is arguably the largest contributor under present design and construction practices.

Problems with Use of the Metal Strap

Another frequent holdown device is the metal strap for shear panels over wood floors. The allowable tension values published in most manufacturer's catalog do not take into account the clear span of the floor framing when involved in holdown use. The nails into the floor framing or top and bottom plates cannot help resist the uplift and must be neglected in determining the value of the strap to resist uplift. Calculations are required to determine the proper length of strap to clear span the floor framing and then engage sufficient length on a full height stud(s) at each end to resist the load. The number of nails into each stud should be clearly shown and the strap should be centered on the floor framing to equally engage the upper and lower studs. See the attached details.[A31-32]

Another problem with the use of the metal strap is the nailing conflict with the plywood edge nailing. Frequent construction practice places the strap over the plywood edge nailing. This usually violates the minimum nail spacing for both the strap and the plywood nails. Four inch nominal edge members may be required to accomodate the nailing of both the strap and plywood at its edge. The strap is better installed at any non-plywood side of the edge member.

Shear Wall Vertical Offsets on Multi-Story Buildings

Sometimes designers do not consider the lower elevation framing for transferring the uplift load into the story below. Full height studs are required and when they are not present, the shear wall length will need to be reduced. This happens when wall openings occur below upper story shear walls. Plan check and inspection staff must be careful that reduced width shear walls are shown on plan elevations and the holdown devices are properly <u>located at their actual ends</u> into full length studs both above and below. See the attached shear wall elevation. [A29]

ROTATION IN WOOD FRAMED BUILDINGS

The original version of the interim rules prohibited all rotational distribution of lateral forces. The interim law amended form allows the principle for one story uninhabited accessory buildings. The maximum diaphragm depth normal to the opening was set at 25 feet to prevent the creation of large torsional forces. In most cases, the remaining three walls are strong enough to maintain structural integrity although not to prevent damage. A problem can arise in High Wind Areas that use portland cement plaster shear walls. The in plane shear, with the added torsional component, can exceed the new recommended allowable for plaster. See the attached calculation[A40]. Therefore all accessory buildings with open fronts in High Wind Areas should use structural wood panel shear walls or be redesigned to a full box structure.

STUCCO & DRYWALL

Multi-story structures with only stucco or drywall shear walls performed poorly when tested by the Northridge Earthquake. Stucco or drywall are no longer allowed as wall bracing at the ground floor of all multi-story buildings. The allowable shear values for stucco and drywall were lowered to decrease their use as wall bracing (shear walls) and to encourage wood panel use. Buildings with plywood shear walls still standing in devastated neighborhoods attest to the superiority of plywood. Even though plywood walls had their share of problems, the material performed recognizably better in multi-story applications than either stucco or drywall.

PORTLAND CEMENT PLASTER

Poor stucco performance indicated problems with the lath fastening and furring from the building paper or felt backing. Large sections of lath and plaster detached from wood frame walls. This was typical of self furring type lath fastened by staples. This lath did not penetrate well into the middle of the plaster making the lath reinforcement ineffective. The stapled lath was also more likely to detach from the wood framing. For these reasons, the use of self furring lath and staple fasteners was removed from shear wall construction options in Table 47-I Item 1. Exterior lathing for non structural purposes may be by any method recognized in the code.

Narrow stucco sections also cracked to failure when remaining attached. The minimum height to width ratios was therefore reduced from 2:1 to 1:1 and the allowable shear value for stucco was reduced 50% to 90 lbs. per foot and is prohibited as wall bracing on the ground floor of multi-story buildings.

DIMENSION RATIOS

The reduction of allowable height to width ratios for plywood walls from 3½:1 to 2:1 is an attempt to limit deflection caused by panel bending and rotation and address narrow panel strength loss[19]. It also has the benefit of reducing the effects of vertical slippage from holdown devices. The vertical slip of the holdown anchor translates into horizontal displacement of the top of the shear panel by the ratio of the height to width of the panel. This significant component of shear wall deflection is hence reduced 43%.

Almost half of the allowable shear panel capacities in Table 25-K-1 allow deflection in excess of 0.005h story drift limits at a panel aspect ratio of 3½:1 This occurs without considering the contribution of holdown slippage. See the attached table.[A34] Holdowns are normally required on the narrow panels and can contribute significantly to their deflection. When the slippage effects of the common bolted holdown connections are included, *no panel configuration is known to meet the story drift limits.*

Increased panel widths as well as reduced allowables will decrease actual deflection. The panel configurations may have sufficient strength but the stiffness criteria of present code is not being met. One of the single greatest improvements in wood frame seismic performance may well come from the increased minimum aspect ratio of shear walls. See the attached shear wall deflection calculations.[A35-38]

MECHANICAL PENETRATIONS

All the good work of conscientious engineers and carpenters can be for nought if excessive mechanical penetrations penetrate shear walls. Large holes in beams or columns are rare because most tradesmen understand the vertical load paths intuitively. The same approach needs to be taken for that vertical diaphragm (ie cantilever wood panel beam). Large openings in diaphragms must be detailed on the plan with all of the appropiate opening reinforcement shown. This is done more often on floors and roofs but shear walls need the same attention. Simple tables and notes will help reduce a problem that stems from poor coordination of the mechanical and structural designs.

Building inspectors and good builders must pay careful attention to this ongoing problem. Guidelines in the past have not been uniform but some help is on the way. See the attached[A39] Table 23-1-Y-1 which proposes limits to the notching of shear wall plates. Other proposed code changes will limit all mechanical penetrations to no more than 20% of the shear wall length. Although these changes are not code yet, they are good advice now.

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The original version of the interim rules prohibited all rotational distribution of lateral forces. The interim law amended form allows the principle for one story uninhabited accessory buildings. The maximum diaphragm depth normal to the opening was set at 25 feet to prevent the creation of large torsional forces. In most cases, the remaining three walls are strong enough to maintain structural integrity although not to prevent damage. A problem can arise in High Wind Areas that use portland cement plaster shear walls. The in plane shear, with the added torsional component, can exceed the new recommended allowable for plaster. See the attached calculation[A40]. <u>Therefore all accessory buildings with open fronts in High Wind Areas should use structural wood panel shear walls or be redesigned to a full box structure.</u>

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The reduction in the allowable values of drywall partitions effectively removes their use as shear walls except in small one story structures. One value of 30 lbs. per foot now covers all conditions of blocking, nailing and number of layers. The quality of work of the blocked wall construction was poor and many installations are actually screwed instead of nailed. This also follows a more recent trend in the Uniform Building Code (UBC) to reduce allowable shears for gypsum products in Seismic Zones 3 and 4. Recent tests have indicated a weakness in dynamic cyclic applications[19]. Gypsum panels are sensitive to deflection and are not compatible with other materials like plywood or stucco.

Proposed code changes will allow higher allowable values to resist wind loads as well as the use of stapled lath. The practical effect in design will be small in our Seismic Zone 4. The seismic proposals show a increase to 60 lbs/ft shear capacity for gypsum lath, sheathing board, wallboard or veneer base but this is still too low to be useful on most structures

OPEN SOFT STORY DESIGN

Many of the hazardous apartment buildings now sitting vacant in the City are buildings with soft story conditions. Even the ones still standing have drifted beyond structural integrity under a moderate seismic event. Previous building code editions left it up to the designer to determine compatible deflection and allowable drift requirements. Not until the 1988 version of the UBC did the code attempt to quantify many of the structural irregularities and problems that performed poorly in this earthquake. Repairing of these hazardous buildings must meet the current seismic code standards.

A common situation involves a wide flange or pipe column acting as both vertical support and lateral brace. Sometimes there is an independent shear wall or frame but their interaction must be investigated. Common problems to investigate include compatible deformations, shear distributions based on relative stiffness, collector tie continuity, calculated story drift and P-Δ effects. A thorough understanding of and compliance to present code is therefore required for the repair of these damaged buildings.

Here is an example of what can be required. A three story apartment building partially collapsed when the many drywall (and few plywood) shear walls failed in the transverse direction. An analysis of the longitudinal direction revealed sufficient strength in the end frames but no apparent continuity tie between them in the building. Intermediate pipe columns in this same line were near their vertical capacity. When subjected to $3R_w/8$ deflections, the columns failed the P-Δ test for combined vertical and bending stress even though the end frames met the story drift limit.

In this case plywood shear walls are required on the ground floor by the new code and the second and third floors by load for the transverse direction. The longitudinal direction requires additional stiffness to limit the intermediate column drift. The existing frames could be stiffened or another frame added near the middle of the building. In either case, continuity ties would have to collect and distribute the lateral forces to the frames. They are required to be verified as existing during construction or detailed and built from the new repair plans.

PLAN REVIEW

Plan check corrections are sometimes written from the perspective of the building official and not the designer who must comply with them. The requirements are stated in code language instead of simple and clear English. The code language approach is understandable because plan check engineers sometimes have to defend their assertions from code sections. But building officials can help applicants obtain their permits when they write comments on the plans and calculations that are clearly referenced and easy to understand. These tend to be more helpful than the code language type corrections written alone on a plan check correction form.

PLAN REQUIREMENTS

Perhaps the greatest contribution a structural designer can make for quality of construction is a good set of plans. A great many poorly constructed buildings can be traced to an inadequate representation of the code requirements on the plans. Contractors and building officials then need to spend a lot of time on detail changes or omissions. A real effort in plan check for plan thoroughness can help minimize this problem.

Review of plans on damaged buildings as well as current practice indicates a need for better detailing of the lateral system. In a novel approach, the City is specifically requiring certain plan elements by law for the first time. These include elements of the framing and foundation plan. They also include required elevations and details. The following minimum requirements are stated in a broad interpretation of that directive.

FOUNDATION PLAN

1. The size, type, location and spacing of all sill plate anchor bolts
2. The location and type of all holdown anchor bolts embedded in concrete
3. The required depth of embedment, edge and end distance of anchor bolts
4. The column size and location of all braced or moment frames
5. Referenced details for:
    a) all grade beam and footing sections
    b) frame to foundation connection
    c) grade beam to caisson connection
    d) stepped stem wall on grade beam top
    e) embedment of bolts into footing at slab cold joint if used

FRAMING PLAN

1. The width, location and material of all shear walls
2. The width, location and material of all frames
3. Referenced details for:
   a) column to beam connection
   b) beam to wall connection
   c) wall brace for short studs when used at roof sections
   d) shear transfer details at floor/roof lines (top & bottom)
   e) required nailing and length for top plate splices
4. Shear wall elevations showing
   a) panel heights and widths
   b) location of holdowns, collectors, straps and framing anchors
5. Shear wall schedule with the following:
   a) correct shear wall panel capacity in lbs. per foot
   b) nail type, length, and pennyweight-(ie 8d common, 10d full length common)
   c) complete material specification-(Struc 1 C-D Exposure 1)
   d) material thickness-(15/32", 7/8")
   e) top & sill plate fastener type and on center spacing
   f) holdown type, bolt or nail sizes and required thickness of end stud(s)
   g) required location of 3x or 2-2x edge members
   i) required nailing connection of double end members(when used)
   j) required edge distance for nails on plywood and framing members
   k) sill plate material assumed in design
   l) required flush nailing at plywood surface.
   m) limits of mechanical penetrations

INSPECTION REQUIREMENTS

Every person involved in the construction process has a part in the pursuit of quality. The original designer starts the process and has the most liability. The plan reviewer spot checks the designer and the building inspector spot checks them both. The inspector must rely on both the plans and his or her own sense of the normal. When that sense is violated, questions should be asked because building inspectors are the eyes and ears of the building department.

The most frequent dilemma for the inspector is to know what is important for the limited amount of available time to inspect. The following guidelines should be helpful for structural issues on wood frame buildings.

1. *Make sure all properly sized anchor bolts are firmly attached to form work* with the proper concrete and wood edge distances before approval is given to pour. No exceptions should be allowed for holdown bolts. Because most pressure treated sill plates come in 14 or 16 foot lengths, expect to see a pair of bolts for the ends at these distances. An embedded J bolt is preferred to an added expansion or epoxy bolt.
2. The depth of the footings in firm competent soil may require you to ignore the 6-8 inches of loose topsoil on flat lots. Let the builder get a soils engineer if there is a desire for shallower footings. Normally the deeper the footing is, the better the soil properties are. *A typical 12 inch footing may be 18 inches below existing grade in order to be in good soil.*
3. Verify if *a top plate splice* detail exists on the plans and look for 16d nails coming through the lower of the two top plates at the splice. *Typical details require 3 stud spacing of the splice and a dozen or so 16d sinkers or commons on each side of it.*
4. *Several nails are required at roof or floor to wall connections.* Although Table 25-Q usually shows these connections with 3-8d common or box nails, many carpenters substitute 2-16d sinkers instead. This was a former option in the code but has been removed to improve the quality of the toe-nailed connection. Rafter and ceiling joist connections to the top plate requires 3-8d, carpenters use 2-16d at each end of the blocking, 1-16d for the blocking into the top plate, and a minimum of 3-16d between the ceiling joist and rafter. That's 14 nails within 16 inches! *The ceiling joist to rafter and ceiling joist lap connections at the roof level are important for roof and box frame stability.*
5. *Studs are normally assumed to be built full height unless otherwise noted.* However, a frequent practice by carpenters is to frame walls in two sections at the roof. The first section is the 8 ft height. Rafters and ceiling joists are framed next. Finally cripple studs are framed into the bottom of the parallel rafter usually without a top plate. This construction requires a brace and performed poorly in the Northridge earthquake. Sometimes 2x6 studs are required near the ridge of the roof. This is because the unbraced maximum height of a 2x4 stud is only 14 feet unless an engineered brace or stud design is otherwise approved.See the attached detail for wall bracing on short studs.

6. Shear panel construction is the most important part of the lateral system. *The nail size, spacing and edge distance are the most important elements.* Keep samples of common nail sizes for 6d, 8d and 10d to compare to the field specimens. The spacing is not as important as the number of nails it represents per foot of width. It is acceptable if the same number of nails are present and the spacing is approximately correct but do not count nails too close to the edge or that fracture the plywood surface. Nails that miss or are too close to the edge of the framing member or blocking should be removed.

7. Douglas fir-larch *sill plates* are critical for shear walls. The common field substitution of hem-fir reduces values 18% and is not normally considered by the designer or plan reviewer. Use of foundation redwood is worse with a 35% reduction in value. Read the Western Wood Products or other stamp on the piece to identify the species. *Return any engineered jobs to plan check* that do no show hem fir or redwood sill plates on the plans.

8. Grade of the plywood accounts for *10% of shear wall strength when Structural 1 is specified.* Look for the Structural 1 designation on the back of the panel. Exposure 1 is not Structural 1. Exposure 1 indicates exterior gluelines between the plies and it is required on all shear walls, floor and roof diaphragms. Normally, this requirement is not a problem since over 95% of all plywood panels are Exposure 1[26].

9. *A35F or increased sill plate nailing should be expected on all plywood shear panels* that span plate to plate on the same wall. Overlength sheets that break on the rim joist or blocking require edge nailing at the plate and edge of the plywood. That is four rows of nails at a break between two sheets. When the sheet is continuous over the floor/roof to wall connection, one row of nailing is required at the bottom plate, top plate and floor framing. That is a total of three rows at the floor diaphragm to shear wall connection.

10. Double or 3X members increase capacity 12%. Check *the nailing of the 2-2x pieces* together. The 16d sinker nail spacing on each side should be close to the shear wall nail spacing on panel seams or sill plates but *must be specified on the plans. See the table.*

SPECIAL DEPUTY INSPECTION REQUIREMENTS

Plywood shear walls will now require the use of a special deputy inspector when the design shear exceeds 300 lbs/ft. This is a new category of deputy inspector. The normal requirement for continuous inspection is not always required and some periodic special inspection is authorized under 91.306(e). The deputy inspector will be required in three distinct periods. The building inspector must verify the work of the s well as the work the deputy inspects.

The first duty of the wood deputy inspector will be to check the placement and size of anchor bolts for shear wall sill plates and holdown devices prior to the concrete pour. The next visit will be to insure the proper installation of sill plates during the initial phase of framing. Both of these inspections can be periodic. The actual installation of the plywood, its nailing and the connection of all hardware will be done under continuous inspection. The following is a general checklist for wood deputy inspection of shear wall systems.

PLAN & GENERAL REQUIREMENTS

1. All details and plans shall bear the approval stamp of the Department of Building & Safety and the stamp of an architect, civil or structural engineer registered in the State of California
2. All shear walls shall be of proper length and location as shown on floor and framing plans.
3. Any special conditions of the design or Department granted modifications of the code for the shear wall system shall be met.
4. All shear panel capacities greater than 300 lbs/ft require deputy inspection unless otherwise noted.

FOUNDATION

1. All shear wall sill plates shall be generally continuous and uninterrupted by utilities. The width of mechanical penetrations shall not exceed 20% of the shear wall length.
2. All foundation sill plates shall be connected to the foundation or foundation wall by minimum ½ diameter steel bolts with 7 inch embedment into **monolithic concrete** with bolt spacing at 6 feet on center maximum.

    EXCEPTION: Chemical, mechanical or bearing type anchors may be used when shown on the approved plans and all conditions of the Research Report are complied with.
3. The bolt size, type and spacing shall match the shear wall and foundation plan schedule.
4. All foundation sill plates require a minimum of two bolts per member.
5. All holdown anchor bolts shall have the minimum required embedment into monolithic concrete as shown in the Research Report and manufacturer's catalog.

WALL FRAMING

1. All framing dimensioned lumber shall be:
   a) **Douglas Fir-Larch unless otherwise noted on the approved plans**
   b) a minimum of 2 inch nominal thickness except as noted below
   c) free of defects such as dryrot, mildew, and excessive wane or warping
2. All studs and/or blocking at adjoining panel edges and bottom sill plates shall be 3 inch nominal or wider thickness when the required shear shown in the shear wall schedule exceeds 300 lbs/ft.

   EXCEPTION:Substitution of a double 2 inch member for the 3 inch required may occur on existing buildings when the original member is not replaced. The nailing of these substituted members together shall match the nail size and spacing requirements detailed on the plan.
3. Studs or blocking at adjoining panel edges shall be 3 inch nominal or wider thickness when:
   a) the plywood nail spacing is at 2 inch on center
   b) 10d full length common plywood nail spacing is 3 inches or less on center
   c) plywood is applied to both faces of the wall with nail spacing less than 6 inches on center on either side and panel joints occur on the same framing member for each face of the wall.
4 Framing members or blocking shall be provided at all panel edges
5. Studs shall not be cut or notched to a depth exceeding 25% of their width.
6. Studs shall not have bored holes exceeding 40% of their width (60% if doubled). Bored holes shall be a minimum of 5/8 inch from the edge and not be located in a cut or notched section.
7. Framing members which split during plywood nailing shall be replaced.
8. Mechanical penetrations shall not exceed 20% of the wall length.

PLYWOOD

1. Plywood thickness and grade shall match the shear wall schedule for the given location
2. Plywood shall be properly marked with a grade stamp showing:
   a) performance rating- Structural 1 or Rated Sheathing
   b) conformance to US Product Std. 1-83 (UBC STD. 25-9)
   c) Exposure 1 or Exterior durability (glueline) classification
3. Panel joints shall occur on the centerlines of studs or double edge member joints.
4. Panel joints shall occur on separate studs when plywood is applied on both faces of the wall if the studs are less than 3 inch nominal width and the nail spacing is less than six inches on center on either side of the wall.
5. Plywood shall not be cut out for ductwork, piping, electrical conduits or equipment without the approval of plan check and the architect and/or engineer.

PLYWOOD NAILING

1. Nails shall be driven flush and not fracture the surface of the sheathing.
2. All nails shall be removed if not properly located in a stud, plate or blocking.
3. All nails shall have the wire gauge (diameter) and full head size of the Common type.
4. The maximum field nailing is 12 inches on center for 16 inch on center framing
5. The panel edge nail spacing shall match the shear wall schedule for its location

   NOTE: Field tolerance of 1/3 of spacing requirement is allowed provided the equivalent number of nails is provided.
6. The minimum panel nail edge distance is 1/2 inch for design strengths over 300 lbs/ft (3/8 inch for all others).
7. Nails shall be staggered at panel joints for spacing of 2 inches on center and for full length 10d nails spaced 3 inches or less on center.
8. Nails shall be staggered on each face of the wall when plywood is applied to both faces with panel joints on the same framing member.
9. Edge nailing is required on the framing member connected to any holdown device.
10. When plywood is approved to connect the wall to the floor or roof without framing anchors or increased sill plate nailing, then the following conditions apply:
    - a) a row of edge nailing is required at the top and bottom plates of the shear wall
    - b) a minimum of one row of edge nailing is required at roof or floor framing members and at all panel edges
    - c) nails in blocking or a rim joist shall have the required nail end distance from the rim joist or blocking edge (3/4 inch for 6d, 8d and 13/16 inch for 10d).
    - d) nails in floor or roof framing shall not be located in the ends of rafters or joists
    - e) nails shall be staggered on floor or roof framing members
11. The diameter of a drilled hole,when used on seasoned framing members to prevent splitting of the wood, shall not exceed 75% of the nail diameter.
12. All nails shall be driven normal to the surface and shall not be toenailed except as required on panel edges.

SILL AND TOP PLATE CONNECTIONS

1. All foundation sill plates on concrete or masonry shall be treated Douglas Fir unless otherwise noted on the plans.
2. All foundation sill plates require a minimum of two bolts per member.
3. All anchor bolts shall be a minimum of 7 bolt diameters and a maximum of 12 inches from the end of the member.
4. All anchor bolts shall maintain a minimum of 3 bolt diameters from the concrete edge and 1½ bolt diameters from the wood edge of the sill plate.
5. All holes drilled in the plate for anchor bolts shall be a minimum of 1/32 and a maximum of 1/16 inch larger than the bolt diameter.

   EXCEPTION: Drilled holes for bolts subject to tension from holdown devices shall be 1/4 minimum and 1/2 inch maximum larger than the bolt diameter.
6. The use of standard circular cut washers on foundation anchor bolts is no longer allowed on shear walls with design strengths in excess of 300 lbs. per foot. The following table for oversized plate washers shall be used in its place:

BOLT DIAMETER$_{inch}$	PLATE SIZE$_{inch}$	THICKNESS$_{inch}$
1/2	2 X 2	3/16
5/8	2½ X 2½	1/4
3/4	2¾ X 2¾	5/16
7/8	3 X 3	5/16
1	3½ X 3½	3/8

10. All 2 inch nominal thickness sill plates over joists or blocking shall be face nailed with a 16d common, sinker or box nail at a minimum of 16 inches on center unless otherwise increased on the shear wall schedule.
11. All 3 inch nominal thickness sill plates over joists or blocking shall be attached as noted on the shear wall schedule. Increased blocking thickness shall be provided as required by the shear walll schedule.
12. All nails connecting sill plates shall be centered in the joist or blocking underneath and a minimum of 1 inch from their end.
13. All sill plates shall have increased nailing or framing anchors and be of the type and spacing as shown on the shear wall schedule.
14. All prefabricated framing anchors shall have current Research Report approval and be manufactured by a Department licensed fabricator
15. All framing anchors used as shear connectors shall have the long direction oriented horizontally and be attached directly to plate and roof or floor framing unless otherwise approved.
16. All top plate splices shall be nailed per the approved detail and be overlapped a minimum of 48 inches.

HOLDOWN CONNECTIONS

1. All prefabricated holdown devices shall have current Research Report approval and be manufactured by a Department licensed fabricator.
2. All drilled holes in studs for holdown devices shall be a minimum of 1/32 and a maximum of 1/16 inch larger than the bolt diameter.
3. All studs connected to the holdown device shall be of the width and number as shown on the shear wall schedule, manufacturer's catalog and current Research Report.
4. The holdown device shall normally be centered on the stud(s) to which it is connected but must maintain a minimum edge distance of 1½ bolt diameters.
5. When holdown devices are bolted to double 2 inch nominal members, the nail type and nail spacing of the double member shall be as noted on the shear wall schedule.
6. **All** washers on holdown device end studs shall be of the oversized type.
7. The proper sequence of bolted holdown installation is to first bolt and tighten the holdown to the studs. Then the tension bolt is tightened. This will help engage the stud bolts to help reduce slippage.
8. Anchor bolts placed in new concrete must have the proper embedment and cover as shown in the manufacturer's catalog and Research Report.
9. All prefabricated strap ties shall be attached to solid framing of the required width with the proper type and quantity of nails as shown in the Research Report, manufacturer's catalog, and structural calculations.
10. All prefabricated strap ties shall be centered vertically at the middle of the floor joists when used as holdowns.

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