PROPOSAL 7-3 (2009)

SCOPE: Part 3 of the *2009 Provisions*

PROPOSAL FOR CHANGE:

Add the following as a Part 3 White Paper:

Special Requirements for Seismic Design of Structural Glued Laminated Timber (Glulam) Arch Members and Their Connections in Three-hinge Arch Systems

1.0 Scope.
These provisions are intended for the design and detailing of structural glued laminated timber (glulam) arch members and their connections that are part of the seismic force resisting system in three-hinge arch systems.

Seismic design coefficients for three-hinged wood arch systems shall be as specified in the applicable Building Code; or in the absence of such information, suggested coefficients are provided in Table 1.

Table 1. Seismic Design Coefficients for Glulam Arches.

Seismic Force Resisting System	R	Ω	C_d
1. Special glulam arch	2.5	2.5	2.5
2. Glulam arch not specifically detailed for seismic resistance - limited to seismic design categories A, B and C	2.0	2.5	2.0

Glulam arch systems shall comply with recommended detailing in *AITC 104-2003 Typical Construction Details*, requirements of the *2005 National Design Specification® for Wood Construction* (*NDS®*) including Appendix E, *ASCE 7-05 Minimum Design Loads for Buildings and Other Structures*, and the applicable building code.

Arch members and arch member connections shall be in accordance with the requirements of the *2005 NDS* including Appendix E Local Stresses in Fastener Groups. In addition, s*pecial glulam arch* systems shall be in accordance with the Sections 1.1 through 1.7.

1.1 Connection requirements.
Connections that are part of the *special glulam arch* seismic force resisting system shall be in accordance with requirements of *NDS* Chapter 10 for Mechanical Connections and additional requirements of this Section.

1.1.1 Arch Base. Arch base connections shall utilize a steel shoe assembly in accordance with *AITC 104 Typical Construction Details*. Timber rivets or dowel-type fasteners such as thru-bolts or lag screws shall attach the arch to the shoe. Dowel-type fasteners shall be chosen such that the expected yield mode is Mode III or Mode IV as defined in *NDS*. Timber rivet connections shall be designed to ensure that the expected strength limit state is characterized by rivet capacity.

1.1.2 Arch Peak. Connection of the arch at the peak shall utilize shear plates, bolts, steel dowels, or metal side plates or combination thereof in accordance with *AITC 104 Typical Construction Details.*

1.2 Nominal connection capacity. The nominal capacity of a connection shall be determined in accordance with the following:

(a) For dowel type fasteners: n x $Z(K_F)(l)(C_M)(C_t)(C_{eg})$ where n is the number of fasteners, Z is the reference lateral design value for a single fastener, and K_F, l, C_M, C_t, and C_{eg} are adjustment factors specified in *NDS* for format conversion, time effect, wet service, temperature and end grain, respectively.

(b) For timber rivets: (P_r or Q_r) x $(K_F)(l)(C_M)(C_t)(C_{st})$ where P_r is parallel to grain reference rivet capacity, Q_r is perpendicular to grain reference rivet capacity, and K_F, l, C_M, C_t, and C_{st} are adjustment factors specified in *NDS* for format conversion, time effect, wet service, temperature and metal side plate, respectively.

(c) For split ring and shear plate connectors: n x P x $(K_F)(l)(C_M)(C_t)(C_d)(C_{st})$ or n x Q x $(K_F)(l)(C_M)(C_t)(C_d)$ where n is the number of fasteners, P is the reference design value parallel to grain for a single split ring connector unit or shear plate unit, Q is the reference design value perpendicular to grain for a single split ring connector unit or shear plate unit and K_F, l, C_M, C_t, C_d and C_{st} are adjustment factors specified in *NDS* for format conversion, time effect, wet service, temperature, penetration and metal side plate, respectively.

1.3 Member requirements

Arch members that are part of the *special glulam arch* seismic force resisting system shall meet requirements of NDS and requirements of this Section.

1.3.1 Slenderness. The ratio of tangent point depth to breadth (d_t/b) shall not exceed 6, based on actual dimensions, when one edge of the arch is braced by decking fastened directly to the arch, or braced at frequent intervals as by girts or roof purlins. When such lateral bracing is not present, d_t/b shall not exceed 5.

1.3.2 End grain bearing. At the arch base, end grain bearing shall be on a metal plate with sufficient strength and stiffness to distribute the applied load. At moment splices, end grain bearing shall be on a metal plate when $f_c > (0.75)(F_c^*)$ as required in accordance with NDS 3.10.1.3.

1.3.3 Compression perpendicular to grain. Compression perpendicular to grain, induced at the arch base, shall be by a metal plate with sufficient strength and stiffness to distribute the applied load.

1.4 Member resistance.

1.4.1 Moment, Tension, Compression, and Shear. The arch member shall be designed to resist moment, tension, compression, shear, and applicable combinations of these induced by seismic forces determined in accordance with load combinations of Section 12.4.3.2 of *ASCE 7* (load combinations with over-strength) but need not exceed forces resulting from strength at connections determined in accordance with Section 1.4.2 (a).

1.4.2 Member Resistance at Connections. The member shall be designed for limit states of net section tension rupture, row tear-out, group tear-out as defined in *NDS* Appendix E, and shear in accordance with *NDS* 3.4.3.3 due to the seismic forces as determined by the lesser of:

a. The nominal connection capacity determined in accordance with Section 1.2 for LRFD, or the nominal connection capacity determined in accordance with Section 1.2 divided by 1.35 for ASD.
b. The required capacity resulting from load combinations of Section 12.4.3.2 of *ASCE 7* (load combinations with over-strength).

1.5 Transfer of forces to the arch members.

The diaphragm, members and connections shall be sized to transfer of out-of-plane wall and roof forces into the arch.

1.6 End fixity.

In accordance with assumed pinned behavior of a 3-hing arch, determination of reaction and arch member forces is based on assumed idealized pin behavior at the arch peak and base. Actual detailing may introduce partial moment fixity at reactions, and consideration shall be given to the effect of such fixity on member and connection response.

Arch moment splices shall utilize a metal bearing plate (when required), metal side plates, shear plates, bolts, steel dowels, timber rivets or combination thereof in accordance with *AITC 104 Typical Construction Details*. Design forces for determining the size and number of fasteners shall be based on load combinations of Section 12.4.3.2 of ASCE 7 (load combinations with over-strength) but need not exceed the member design force based on forces resulting from strength at connections (see Section 1.4.1 and 1.4.2 (a)).

Commentary

C1.0 Scope.

Special provisions are provided for the design of arch members and connections to resist seismic forces as part of a three-hinge arch system (see Figure C1.0). Such systems typically employ glued laminated timber Tudor arch members and are commonly used in church construction and other facilities intended for public assembly. Common features of these systems are the presence of 2x and 3x tongue and groove roof decking with wood structural panel overlay, longitudinal and transverse walls of light frame construction, or longitudinal and transverse masonry walls. Transverse end walls may or may not be designed as shear walls.

Special requirements apply to typical construction details used for over 50 years in three-hinged arch systems as outlined in *AITC 104 Typical Construction Details*. Typical arch base details in *AITC 104* are generally expected to produce good performance characteristics of connection yielding by either wood bearing or a combination of wood bearing and fastener yielding and will limit occurrence strength limit states of row tear-out, group tear-out, and net section tension rupture prior to connection yielding. The design requirements in this white paper utilize standard details that have been used successfully and that encourage a combination of wood bearing and metal fastener yielding modes at the base.

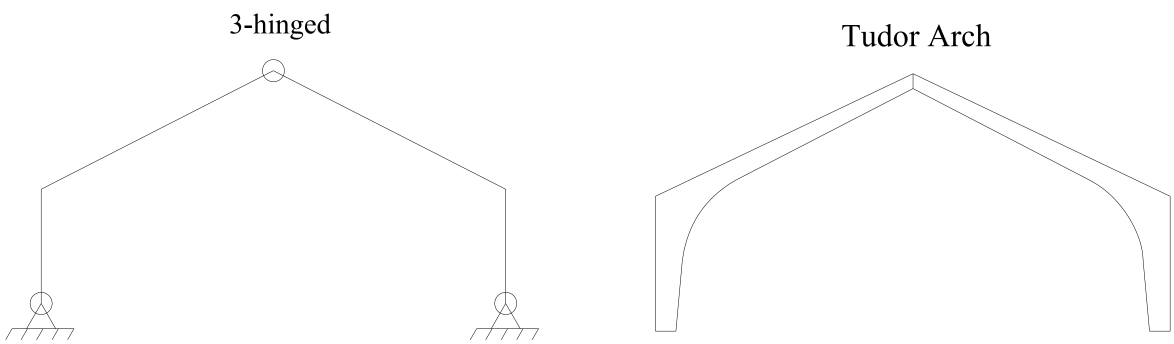


Figure C1.0. (a) 3-hinge arch, and (b) Tudor arch configuration.

C1.1. Connection requirements.

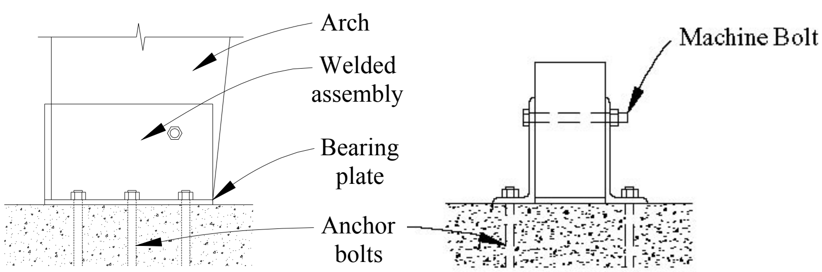
Ordinary load combinations (load combinations without over-strength) of *ASCE 7* are used to determine the size and number of fasteners in arch member connections at the base. Determination of the size and number of fasteners is not subject to special load combinations (load combinations with over-strength forces) to enable limited inelastic behavior of dowel-type fasteners (either by wood bearing or fastener bending) when coupled with wood member strength requirements of Section 1.4. This approach recognizes that wood connection strength is typically governed by wood failure mechanisms, not failure of the metal fasteners. For a given wood member cross-section, determination of the size and number of fasteners based on over-strength load combinations may not be beneficial to overall connection performance due to an increased number of fasteners and a reduction in wood member net section to accommodate the fasteners.

C1.1.1 The connection at the arch base utilizes a metal shoe (see Figure C1.1.1a and b) and typically employs a thru-bolt loaded in double shear. Placement of the bolt(s) is an important consideration. In-service drying of the member causes shrinkage which must be accounted for in the detailing of the connection to prevent splitting due to the development of tension perpendicular to grain stresses.

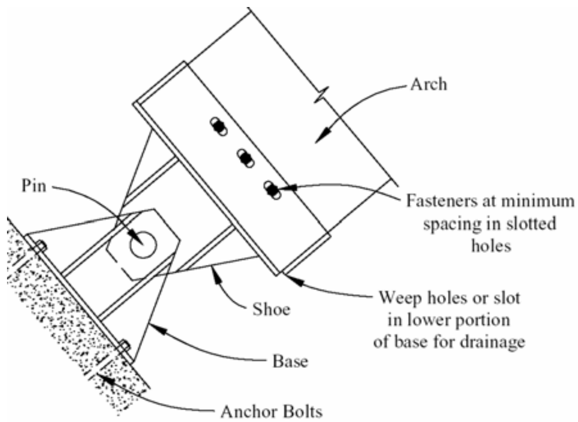
It is recommended that the bolt(s) be placed within 6 inches of the back of the arch if standard size holes are used. Where bolt(s) are placed farther than this from the back of the arch to resist the required loads, the designer should provide detailing to allow the wood to shrink without pulling away from the bearing seat. This may be accomplished through the use of slotted holes or oversized holes in the arch member. It is recognized that some movement of the arch at the base will occur before the bolt is engaged. This practice is used to prevent wood splitting due to occurrence of dimensional change under gravity loads. In some situations a bearing seat is also used at the inside face of the arch. In such a case, the bolt(s) is generally placed at the geometric center of the section with the hole(s) detailed to accommodate shrinkage.

Timber rivets as well as lag screws installed at each side of the arch base are expected to produce comparable performance provided that the controlling yield mechanism is based on dowel yielding or rivet capacity.

Under outward loads, the bending yield capacity of the plate at the back of the metal shoe will typically determine the size of the bearing area (i.e. the plate will yield before the wood reaches its design compression perpendicular to grain stress).



(a)



(b)

Figure C1.1.1. (a) Typical arch base with thru-bolt and (b) arch base with true hinge.

C1.1.2 The connection at the peak typically employs use of a shear plate or plates with thru-bolt(s) and is typically pre-fabricated in a manufacturing facility to establish proper fit and alignment. For arches with slopes of 3:12 and greater, typical connections employ shear plates and bolts or a combination of shear plates and bolts and dowels to transfer both horizontal and vertical forces. For low pitch (low slope) arches, steel side plates on each face are used in combination with shear plates. Figure C1.1.2 shows one example of a peak connection.

The bevel cuts shown at the top of the arch peak connection are used to minimize wood crushing and permit rotation due to downward deflection of the peak connection of deep members. They are not required for all designs but should be considered by the designer where significant rotation is expected. Bevel cuts are generally not used on the bottom side of the connection.

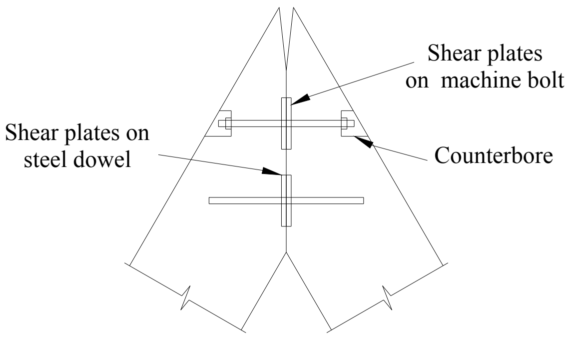


Figure C1.1.2. Typical arch peak connection detail.

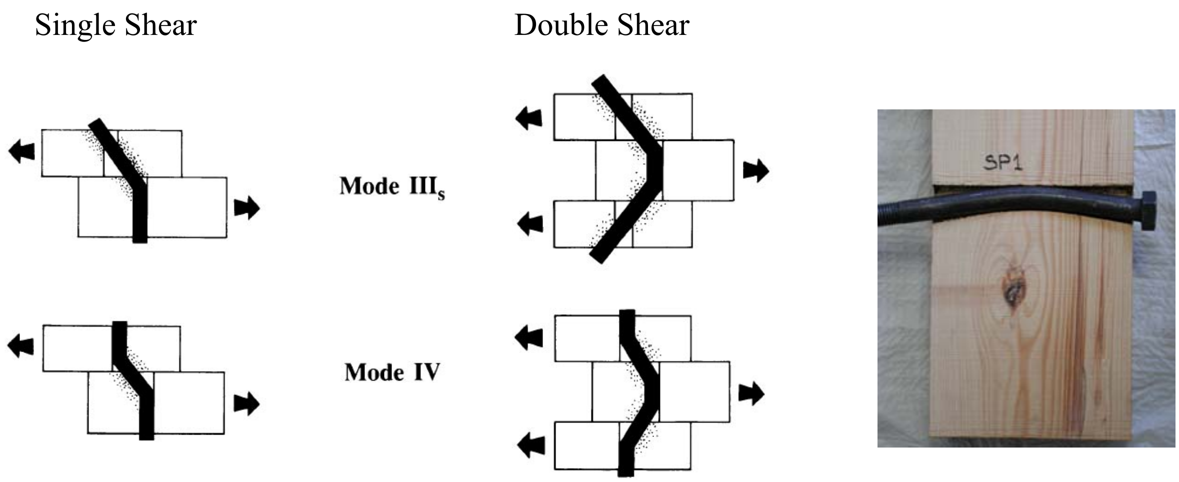
C1.2 Nominal connection capacity.
Determination of nominal capacity does not include adjustment factors for group action and geometry to more conservatively estimate nominal connection capacity. These factors are 1.0 or less in value and address wood strength limit states which are to be checked explicitly by provisions of Appendix E and shear provisions of NDS.

C1.3 Member requirements.
Prescriptive limits on d/b match those in *NDS 2005* for arches.

1.3.2 Consistent with typical construction details used for these systems, a metal plate with sufficient strength and stiffness to distribute the applied load is used at the base (see Figure C1.1.1) regardless of the level of stress in end grain bearing. This bearing plate also separates the arch from direct contact with the concrete, preventing moisture from wicking into the arch from the concrete.

C1.3.3 Compression stress perpendicular to grain in the arch member at the base should be through bearing on a metal plate with sufficient strength and stiffness to distribute the applied load.

C1.4 Member resistance.
Requirements of Section 1.4 are intended provide excess capacity in the member relative to connections, because little or no inelastic deformation is expected from the arch member itself except in bearing modes. Limited inelastic deformation can occur through wood bearing and fastener yielding in the connection region at the base (See Figure C1.3.3 (a) and (b) for examples of Mode III and Mode IV yielding and Figure C1.3.3 b for cyclic behavior of a bolted steel side plate to wood connection).



(a) **(b)**

Figure C1.3.3 (a) Mode III and Mode IV yielding for single and double shear connections, and (b) Mode IV yielding from a double shear connection test.

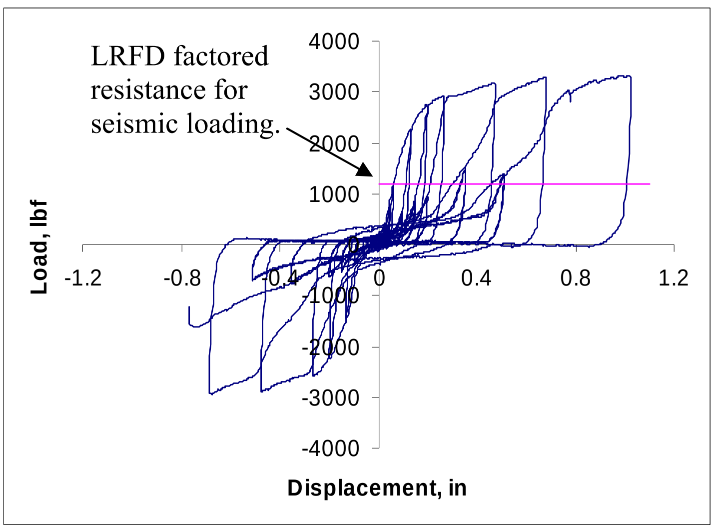


Figure C1.3.3b. **Cyclic curve for single shear bolted connection - Mode IIIs (3/8" diameter bolt, 4x6 wood member, ¼" steel side plate).**

C1.4.1 Arch member design strength must equal or exceed the force based on over-strength load combinations of *ASCE 7* but need not exceed nominal forces developed by connections in accordance with Section 1.4.2 (a). Design for bending, tension, compression and shear, per 1.4.1, is based on applicable net section or net bearing areas in accordance with NDS. Member design at connections, including provisions for shear at connections at member ends and local stresses in fastener groups, is in accordance Section 1.4.2.

C1.4.2 This section requires member design at connections for forces that can be developed in the connections or overstrength load combinations of ASCE 7 to increase capacity based on wood strength limit states relative to connection capacity and provide for limited inelastic behavior at base and peak connections by either wood bearing or fastener yielding or combination thereof.

In Section 1.4.2, required design wood strength at connections is taken as the lesser of (a) nominal strength of the connection for LRFD or the nominal strength divided by 1.35 for ASD, or (b) the force based on overstrength load combinations of *ASCE 7*. Case (b) will generally apply where loads other than seismic control the size and number of fasteners in the arch base. Where the connection has design strength in excess of that needed to resist seismic forces (e.g. forces from wind exceed calculated seismic force), it is only necessary to ensure that the wood member has sufficient design strength to resist loads from special load combinations, not the expected strength of the fasteners.

For ASD, wood strength limit states are checked using the nominal strength of the connection divided by a factor of 1.35. The 1.35 factor is specified to provide for consistent design whether provisions of ASD or LRFD are used. For member design (except compression perpendicular to grain) and connection design, the ratio of LRFD adjusted design value (10 minute basis) to ASD adjusted design value (10 minute basis) is: 2.16/1.6 = 1.35. The factor of 2.16 is the constant in the format conversion factor, K_F, and adjusts ASD reference design values (10 year basis) to LRFD design values (10 minute basis) and 1.6 is the load duration factor, C_D, which adjusts ASD reference design values (10 year basis) to ASD design values at a 10 minute basis.

C1.5 Transfer of forces to the arch members.
Adequate transfer of in-plane diaphragm forces and out-of plane wall and roof forces can be addressed by use of *NDS* for wood member and connection resistance and provisions of *ASCE 7*. For anchorage of concrete or masonry structural walls see *ASCE 7* Section 12.11 and 12.14.7.5, for bearing walls and shear walls see *ASCE 7* Section 12.14.7.6, and for non-structural components see *ASCE 7* Section 12.14.7.7.

C1.6 End fixity.
Three-hinge arch systems are designed assuming pin behavior when typical construction details of *AITC 104* are used; however, it is recognized that limited moment fixity is introduced at the arch base and arch peak connection regions by presence of connectors and bearing of the member cross section.

For example, at the arch base, rotation about the inside face of the arch at the base coupled with presence of connections in the arch shoe will provide moment fixity beyond the assumed condition of an ideally pinned joint. The intent of Section 1.6 is to consider the effect of such end fixity as the arch resists anticipated loading.

Precise estimates of anticipated deformations which may be detrimental to overall connection and member performance are difficult to predict. Their effect at the base connection is mitigated through 1) use of dowel fasteners in yielding mode, 2) increased strength of dowel fasteners loaded parallel to grain when compared to the same fastener loaded perpendicular to grain, 3) presence of localized bearing deformations about the arch base and surrounding the dowel, and 4) dowel placement. At the arch peak, tapering of the arch member minimizes fixity created by wood bearing as the arch deforms (see Figure C1.1.2).

Limited cyclic data of single shear, single bolt connections consisting of a steel side plate and a wood main member indicates an average displacement of 0.8” at maximum load (see Anderson, G.T., *Experimental Investigation of Group Action Factor for Bolted Wood Connections*, Thesis for Master of Science at Virginia Tech). For the particular connection tested (see Figure C1.3.3b), the ratio of average maximum strength to LRFD factored resistance was approximately 2.6. Displacement at maximum load and ratio of maximum load to the LRFD factored resistance will vary by connection configuration.

C1.7 Arch Moment Splice.

Large arches may employ arch moment splices in locations of reduced moment to facilitate shipping. Like the connection at the peak, these connections are typically pre-fabricated in a manufacturing facility to establish proper fit and alignment. Compression stress in the moment splice region is resisted by end grain bearing on a metal bearing plate between the connected members. Tension is taken across the splice by steel straps and shear plates, shear is taken by shear plates in end grain, and side plates are used to hold sides and tops of members in position.

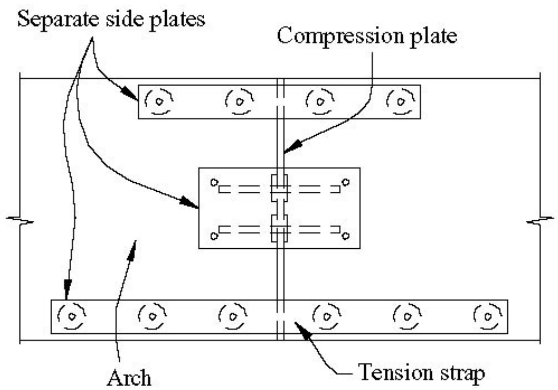


Figure C1.1.3. Typical arch moment splice.

References:

Anderson, G.T., *Experimental Investigation of Group Action Factor for Bolted Wood Connections*, Thesis for Master of Science at Virginia Tech, Blacksburg, Virginia.

AITC 104-03 Typical Construction Details, American Institute of Timber Construction, Denver, Colorado.

ANSI/AF&PA NDS 2005, National Design Specification® for Wood Construction (NDS), American Forest & Paper Association, Washington DC.

ASCE 7-05 Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, Virginia.

REASON FOR PROPOSAL:

Glulam arch structures are used with some regularity in churches and other public buildings and assembly areas; however, *ASCE 7* does not currently provide guidance on seismic design of these systems. Design recommendations were drafted with input from the American Institute of Timber Construction (AITC) and further developed by TS7. This proposal includes seismic design coefficients for two classes of three-hinge arch systems defined as: 1) *special glulam arch,* and 2) *glulam arch not specifically detailed for seismic resistance*.

For *special glulam arch systems,* required detailing enables limited inelastic behavior in connections through either wood bearing or fastener yielding. This is accomplished by requiring design of wood members at connections for the lesser of overstrength forces or the forces that can be developed in the connections. For *glulam arch systems not specifically detailed for seismic resistance*, use is limited to SDC A, B, C. Limitations on applicable SDC is analogous to the approach taken for steel systems not specifically detailed for seismic resistance and wood shear wall systems with other than wood structural panel. The value of R=2.0 is based on a relative comparison of R for *special* systems. For both system types, assumed system overstrength is Ω_0=2.5.

Comparison minimum base shear with past practice

Base shear assigned to "past practice" is based on provisions of 1997 UBC heavy timber braced frames in a bearing wall system (R = 2.8).

1997 UBC: $V = 2.5C_aIW/R$; $V = 2.5(0.44)(1.0)W/2.8$; $V = 0.39$ W (1)

Base shear in accordance with ASCE 7-05 is:

ASCE 7-2005: $V = C_sW = (2/3)F_aS_sW/(R/I)$; $R = (1.0)(2/3)(1.0)(S_s)W/V$ (2)

Solving R for S_s = 1.0 to 2.5 to represent 1997 UBC Zone IV gives the following range of R:

Range of R which provide base shear equivalent to 1997 UBC

Seismic Force Resisting System	R
Glulam arch, three-hinge	1.7 - 4.3

The value of R = 2.5 recommended for the 3-hinge arch structure falls within a broad range of R providing equivalent base shear with past practice defined by the 1997 UBC.