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Title:	Capacity of Load-Bearing Housings: Preliminary Results from Load Testing		

Abstract

An initial series of screening tests was conducted in 2012 to assess the behavior and load carrying capacity of housings representative of butt cog style joist pockets cut into the sides of timber beams (see Fig. 1). The housings were cut in pairs on opposite faces of 8 inch by 8 inch timbers and tested in isolation such that the housings failed before the beams. The load-bearing surfaces of the housings were located at varying heights from the bottom of the beams but in all cases were at or below mid depth of the section. These tests served as a preliminary demonstration of behavior and provided a starting point for further testing and development of numerical modeling. This report presents preliminary findings and conclusions, and discusses possible direction for further research. The relatively consistent behavior demonstrated suggests that it may be possible to establish safe mechanics-based guidelines for design. The tests were limited in scope and number, and not conducted in a laboratory setting, so did not provide an adequate basis for definitive design guidelines, and none are provided here. Good agreement of test values with predicted capacity based on the 2012 NDS© Equation (3.4-7) for split ring connectors, shear plate connectors, bolts and lag screws was found for this data set.

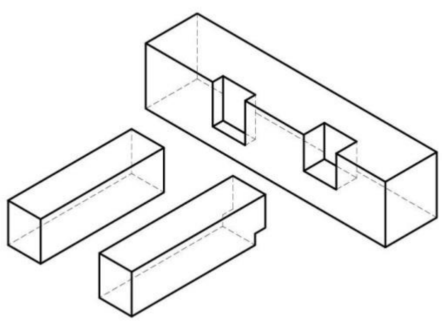

Figure 1 - Typical Butt Cog (Drop-in) Joist Housings

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Introduction

A long-standing challenge in the field of timber frame design has been assessing the load bearing capacity of joist and beam housings cut into the sides of receiving beams. In particular, for a given joist load, there has always been a question as to how much material in the receiving beam should be left below the joist bearing. There are no engineering guidelines that would permit quantitative assessment of the load bearing capacity of a given notched bearing configuration in a given species of beam. For investigators of old frames that are to be repurposed, and designers of engineered frames that incorporate joists supported in load-bearing housings cut into the sides of floor beams, this lack of a mechanics-based design method has been a concern for some time.

The *National Design Specification for Wood Construction*, NDS©, does not address partial width notches, whether loaded or not. In addition, the shear provisions relating to bolts loaded perpendicular to grain (Article 3.4.3.3) indicate that the safe load-carrying capacity of a beam is greatly diminished as the position of the bolts approaches the loaded face of the timber. By inference, load-bearing housings would be subject to similar limitations. However, the reduction in load-carrying capacity provided in the NDS© Equation (3.4-6), which would control at the housings nearest the end of a beam, would effectively prohibit load-bearing housings in timber framing if applied. Yet such housings have been used successfully for hundreds of years.

The *Standard for Design of Timber Frame Structures and Commentary*, TFEC 1-2010, sets certain limits on the proportioning of partial width notches on the sides and compression-tension faces of flexural members (see Section 2.3.4), and provides guidelines for analysis of members containing partial width notches, but does not directly address the load carrying capacity of housings cut in the side of members and loaded perpendicular to grain. As the Commentary points out:

> Partial-width notches on the lateral faces of bending members that extend below mid-depth of the member from the compression face are common in timber framing, for example where joists frame into the sides of beams. In such instances, bearing of the joist on the bottom surface of the notch may induce tension stress perpendicular to grain. While rules of thumb exist for the minimum distance from the loaded surface of the notch to the tension face of the member, no testing has been performed or analytical models developed to define the relationships between notch dimensions, clear distance to the tension face, mechanical properties of the wood (in particular tensile strength perpendicular to the grain), and safe load-carrying capacity of the notch. It is recommended that,

whenever possible, load-bearing notches not extend below mid-depth of the member.

The recommendation provided in the Commentary is conservative but runs counter to much traditional practice that, to the authors' knowledge, has not resulted in performance problems.

Because of the complexities of a numerical analysis and the fact that no numerical analysis could effectively predict the post-fracture behavior of such load bearing housings, the authors undertook a series of load tests.

Test Set-up and Loading Protocol

Three species of timber were used for the initial screening tests, Eastern White Pine, Red Pine, and Red Oak. All timbers were boxed heart and green at the time of testing (moisture content at the surface of the timbers was 40 percent and ranged to 100 percent plus at the pith). The grade of the Eastern White Pine timber based on the NELMA grading rules was judged by the authors to be No. 2, while the Red Pine and Red Oak were judged to be No. 1.

The housings were cut 2 inches in width as measured across the top width of the beam, leaving 4 inches of solid section in the center. See Fig. 2. The housings were 4 inches long measured parallel to the length of the beam. The depth of material left intact below the housings was varied from 2 inches to 3 inches and finally 4 inches to examine the change in capacity and behavior with varying depths of timber below the applied load.

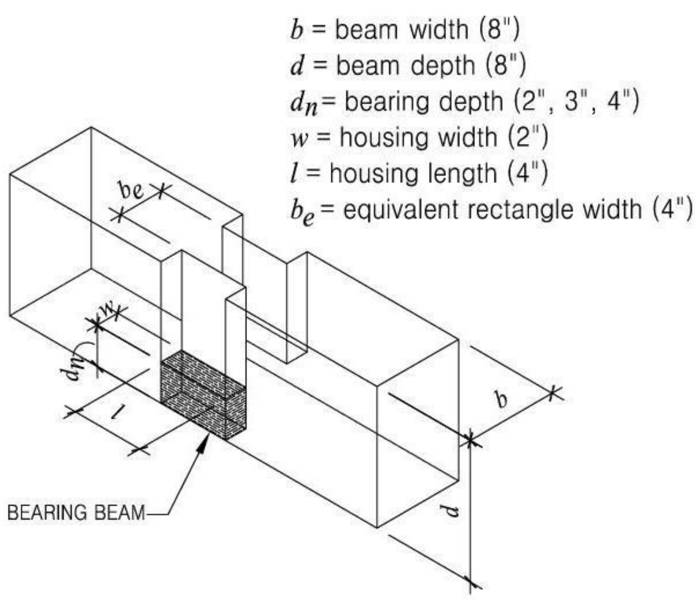

Figure 2 – Notation, Terminology and Dimensions

Seasoned White Oak loading blocks were inserted into the housings and used to load the bottom surface of the housing. The blocks were held 1/2 inch out from the back of the housing, that is the interior face, to represent more realistic loading conditions on the bottom of the housing that can be expected to result from construction tolerances, shrinkage of the beam, and rotation of the joists, all of which move the center of the load away from the back of the housing.

A hydraulic jack was positioned at the intersection of centerline of beam and centerline of the housings. Load was transferred from the jack to the two loading blocks via a 6 inch by 6 inch White Oak cross beam.

Specimens were clamped to a reaction beam with the hydraulic jack inserted between the two as shown in Fig. 3. The clamps were spaced typically 4 feet apart (4’-8” for the 4 inch bearing depth tests) but were not offset equally from the housings. Instead the clamps were typically 1'-4" and 2'-8" respectively from the centerline of the housings. In this way shear was higher on one side of the housing than the other, approximating the variation in shear that occurs at the last load-bearing housing adjacent a post or support for the beam. The offset in the position of the clamps also induced a higher shear for a given bending moment than a center point loading would generate. With the clamps installed at a distance of approximately $5d_n$ from the housings, it was intended they not influence local behavior at the housings.

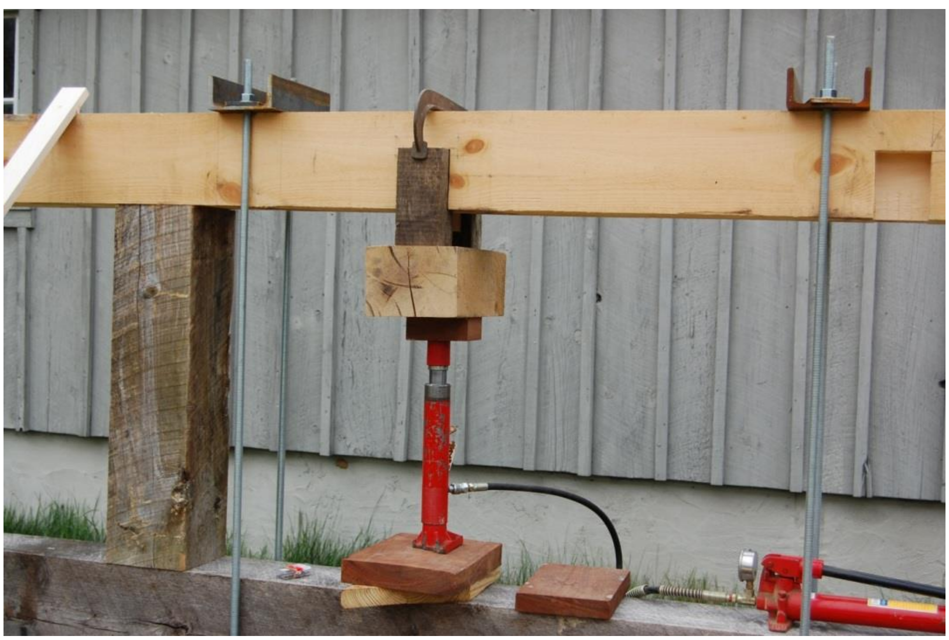

Figure 3 - Load Test Arrangement (Specimen Inverted)

The focus of this testing was to determine the behavior of the load bearing housing up to and including failure; for this reason deflection of the beam and the effect of the housings on beam behavior were not studied. To develop load-deformation curves the authors elected to measure separation of the "bearing beam" section from the main body of the timber as evidenced by horizontal fractures that developed parallel with the bearing surface at the face of the beam. Reference pins were driven into the beam at positions approximately 1 inch above and below the bearing surface on either side of the housing. In this way crushing of the beam fibers at the bearing surface did not influence the crack width measurement.

The load was increased incrementally, with the magnitude of the load step varying with wood species (500 pounds typically for Eastern White Pine, and up to 2000 pounds for Red Oak). The load was increased over a relatively short period of time at each of the steps, approximately 10 to 15 seconds. Once the next load level was achieved, the load was held constant until deflection readings stabilized. Thus the duration of the load test varied depending on the housing depth and the species being tested, ranging from about 40 minutes to 4 hours. Testing did not follow the procedures of ASTM D1761 *Standard Test Methods for Mechanical Fasteners in Wood.*

After the peak load was reached, load would fall off a certain percentage of the maximum but would not drop to zero. Additional load-point displacement was applied to observe the post-peak behavior, examine ductility and watch for any secondary increases in load carrying capacity. Testing was typically terminated when the gap between the bearing surface of the housing and the body of the beam reached 3/4 inch as measured at the exterior surface of the specimen.

Findings

Table 1 provides the average maximum test load for each of the three different species and housing depths. This simplified summary shows the basic trends as a function of housing beam depth *d* and the specific gravity of the species.

Figs. 4 and 5 provide graphic summaries of test load vs. crack width.

Table 1 – Average Maximum Test Values for Each Bearing Depth and Species (Pounds Per Housing, i.e. Half of Total Applied Load)

Species / Bearing Depth	2 inch	3 inch	4 inch
Eastern White Pine	3000	4750	6000
Red Pine	4000	6500	not tested
Red Oak	5000	6500	8000

As the load-deformation curves of Figs. 4 and 5 illustrate, the housings did not fail in a brittle manner, that is, the behavior did not remain linearly elastic up to peak load followed by complete loss of load carrying capacity. Rather, in all cases an indication that peak load was being approached was evidenced by a change from near linearly elastic behavior to a marked increase in crack width and length at the face of the beams with increments in load.

The mode of failure at the peak load varied primarily as a function of species. In Eastern White Pine, the weakest of the species tested, the failure was marked by a sudden rupture of the fibers in the bearing beam in the positive and negative bending regions, or in some cases by a shear failure where the fracture extending from the bearing surface would run out to the tension face of the beam. The fractures observed followed the grain of the timber and so slope of grain was an important factor locally for the white pine. The Red Oak specimens on the other hand, the strongest of the species tested, never failed due to a shear or flexural rupture. As the width and length of the fractures extending from the bearing surface increased past some critical length, the bearing beam eventually became too flexible to support the load and a steady decrease in load carrying capacity was observed with increasing deformation. The Red Pine was intermediate in behavior between the White Pine and Red Oak. Because the specimen was straight grained, its performance more closely resembled that of the Red Oak. (Note that the curves shown for Red Pine suggest a less ductile behavior than the Red Oak, however that is an artifact of the test set up. Multiple tests were run on the same timber and, in the case of the Red Pine specimen, fractures from the housing being tested grew in length until they intersected adjacent housings, which then changed the behavior of the bearing beam. Without these nearby housings having been present, the behavior likely would have appeared more like the Red Oak.)

The characteristic fracture that developed as the test proceeded exhibited a compound surface comprised of both a horizontal extension from the bearing surface parallel to the length of the beam and a diagonal sloping surface extending from the bearing surface at the back of the housing down and towards the center of the beam. See Fig. 6.

The load applied to the bearing surface of the housing induces tension perpendicular to the grain on the horizontal surface extending outward from the bearing surface into the surrounding wood, and induces rolling shear on the vertical plane below the back of the housing. Since the load is eccentric from the centroid of the beam, the load also induces rotation of the bearing beam, which generates tension perpendicular to the grain on the vertical surface at the back of the housing. This combination produces diagonal tension perpendicular to grain that causes the fracture surface at the back of the housing to angle downwards towards the tension face of the beam.

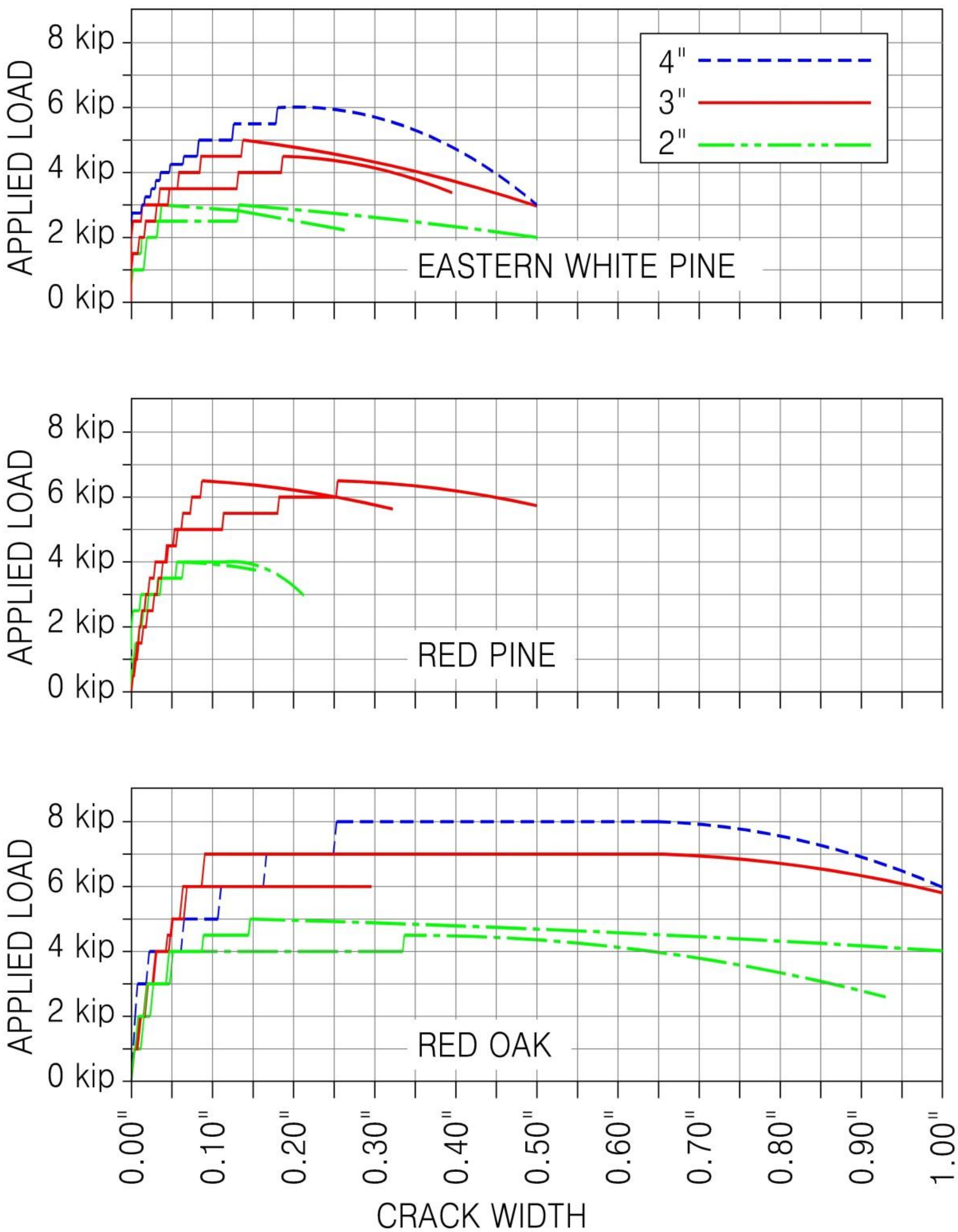

Figure 4 - Load - Deformation Curves, by Species for Various Bearing Depths Tested

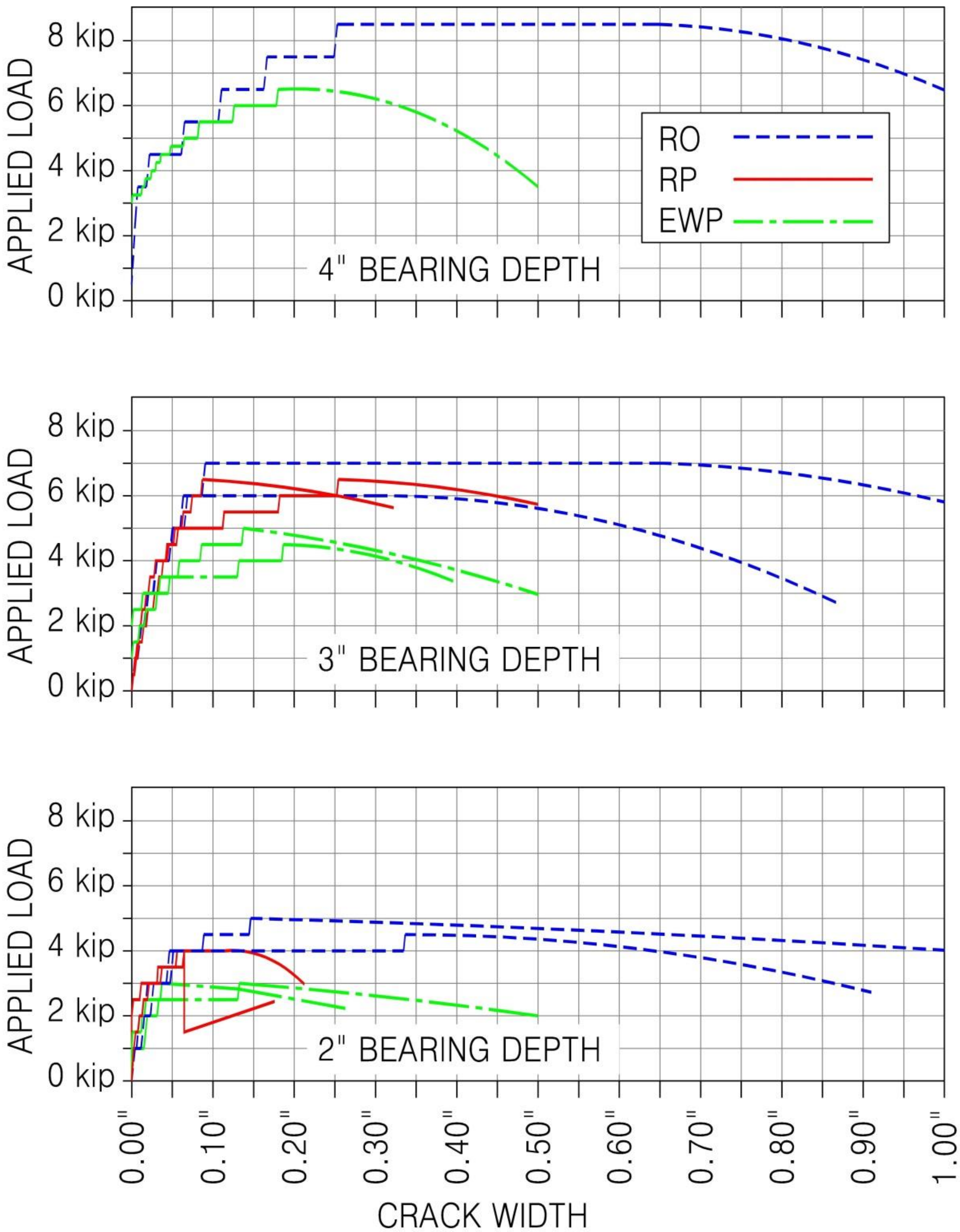

Figure 5 - Load Deformation Curves, by Bearing Depth, for Three Species Tested

As the width of the fracture increased, the direction of the fracture surface tended to change from being parallel to the length of the beam to running out towards the tension face. This characteristic form was repeated in virtually every test performed. The general form illustrated in Fig. 6 was typical of the 2 inch and 3 inch bearing depths, while the 4-inch bearing depth specimens tended to fracture straight across between housings. The fracture shown represents a snap shot during the testing. The actual shape and length of the fracture at peak load varied with species and bearing depth.

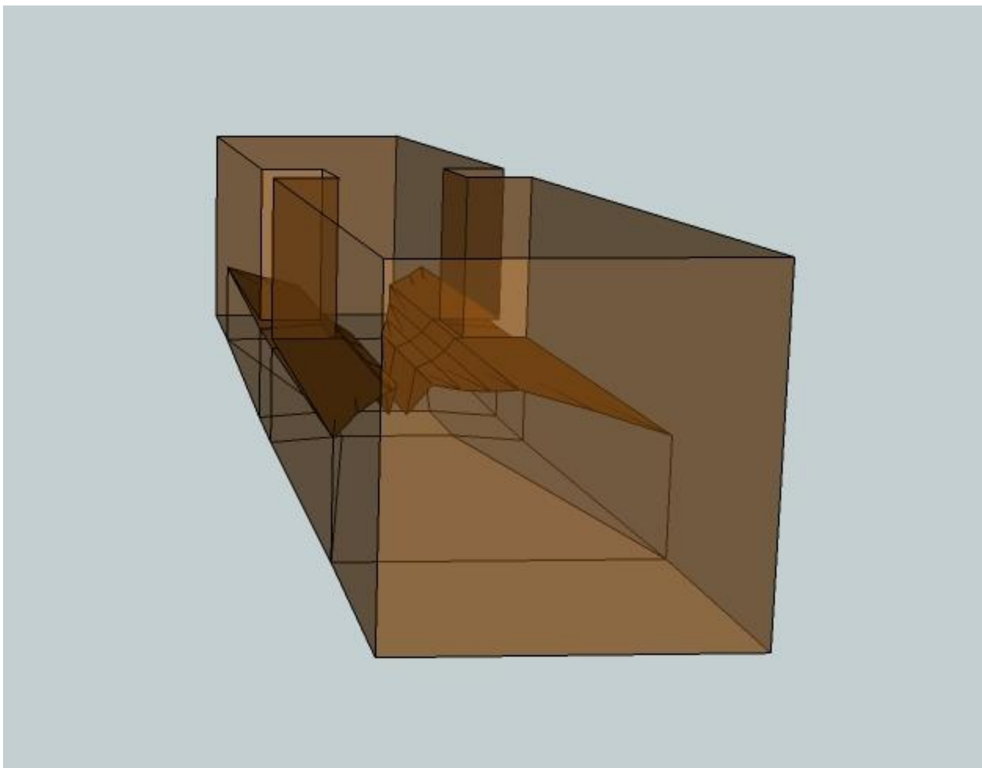

Figure 6: Fracture Geometry

Analysis

The test results presented in Figs. 4 and 5 make several aspects of load bearing housing capacity and behavior evident.

- The deeper the bearing depth (within the range tested here), the greater the load carrying capacity. The strength exhibited was approximately linearly proportional to the depth, not a geometric function of the depth, indicating that the flexural strength and stiffness of the so-called bearing beam were not primary factors in determining housing strength.

- The stronger the wood (as measured by tensile strength parallel to grain, tensile strength perpendicular to grain, and shear strength parallel to grain), the higher the load bearing capacity. The strength was not directly proportional to the nominal specific gravity of the species as listed in the NDS©, that is, the housing capacity did not increase from Eastern White Pine to Red Oak as much as the increase in specific gravity between the two species.
- The failure mechanism is more complicated than just a splitting perpendicular to grain failure, as evidenced by the fact that load capacity continues to increase after the initial development of the fracture plane. The geometry of the fracture plane suggests that tension perpendicular to grain, rolling shear, shear parallel to grain, and flexure all play a role, with different aspects being primary at different stages of deformation and fracture lengthening. Tension perpendicular to grain may initially be the stiffest and most directly engaged "spring" in the mix of resistance mechanisms, but it is also the weakest of the multiple mechanisms at work and is first to drop out. The absence of abrupt failure with the first appearance of fractures at the sides of the housings suggests that there are stronger though more flexible springs in the system that carry the load after failure of the stiffest spring. It appears that even at practical working load levels where the fractures, if any, have not extended very far, it is inappropriate to think of the material below the bearing surface as a beam supported at each end by tension. It more appropriately could be considered a thick plate supported on three edges with the strength and stiffness of the support varying around the edges.
- Dividing the peak load by a load duration factor of 1.5 to adjust for the short duration of the loading, and by a format conversion factor K_F of 3.32 and resistance factor ϕ of 0.65 for connections to convert from ultimate to working stress level (ignoring for the moment possible statistical variations that, with further testing, could define a 5 percent exclusion limit), produces an "allowable" load that is in the linearly elastic region (essentially uncracked behavior) of the housing. For example, dividing the 3 inch Red Pine average maximum load capacity of 6500 pounds by the above factors produces a working load of about 2000 pounds which can be seen in Figs. 4 and 5 to be in the elastic portion of the curve.

Table 2 shows the peak test values converted to implied allowable loads (ASD) by dividing the average peak test load by a load duration factor C_D of 1.5, a conversion factor K_F of 3.32 and a resistance factor ϕ of 0.65, and provides a comparison between these values and the allowable load that would result from applying NDS© Equations (3.4-6) and (3.4-7) to the beam shear capacity and treating the housing bearing depth d_n as equivalent to d_e used in those equations. The maximum shear produced in the beam in these tests was equal to three quarters of the total load applied due to the third-point loading used in the test set up. That is, V_{max} = 2/3 x 2•P, where P is the load applied at each housing, so:

$$P_{allowable} = \tfrac{3}{4} \text{ x } V'_n = \tfrac{3}{4} \text{ x } \frac{2}{3} * [F'_v * b * d_e] * [d_e/d]^2 \qquad \text{(per 3.4-6)}$$

$$P_{allowable} = \tfrac{3}{4} \text{ x } V'_n = \tfrac{3}{4} \text{ x } \frac{2}{3} * [F'_v * b * d_e] \qquad \text{(per 3.4-7)}$$

Table 2 – Comparison of Test Values (Converted to Allowable) with NDS© Equations (3.4-6) and (3.4-7) for Each Bearing Depth and Species, Pounds per Housing

Species/ Bearing Depth	**Eastern White Pine**			**Red Pine**			**Red Oak**		
	Test Allow.	**Eq. 3.4-6**	**Eq. 3.4-7**	**Test Allow.**	**Eq. 3.4-6**	**Eq. 3.4-7**	**Test Allow.**	**Eq. 3.4-6**	**Eq. 3.4-7**
2 in	926	63	1000	1235	65	1040	1543	78	1240
3 in	1466	211	1500	2006	219	1560	2006	262	1860
4 in	1852	500	2000		520	2080	2469	620	2480

This comparison shows that Equation (3.4-6) clearly does not relate to the capacities that were demonstrated in these tests. Equation (3.4-7) on the other hand shows good agreement with the test results, even though the NDS© indicates it should not be applied to connections closer than five times the beam depth to the support.

This comparison suggests that Equation (3.4-7) might be a reasonable starting point for a mechanics-based capacity formula in a design standard, and it is consistent with the finding that the load bearing capacity of a housing appears to be a linear function of the bearing depth. Also, allowable shear parallel to grain is a value readily obtained from the tables in the Supplement to the NDS© (as opposed to tension perpendicular to grain and rolling shear) and so is a desirable property to use in a design equation to predict safe capacity of load bearing housings.

Recommendations for Additional Study

Clearly for the three species tested considerably more data will be required to begin to understand the potential variability of peak load results for the same geometries. It will also be appropriate to test additional species that represent typical timbers used today in our industry, such as Douglas-fir, White Oak, Southern Yellow Pine and Yellow Poplar.

A broader range of geometries should be studied to verify whether the length of the housing parallel to the length of the beam, has any appreciable effect on capacity. Another relevant issue to be clarified is whether and how the ratio of the bearing depth to the overall beam depth affects capacity. Clearly the relationship between bearing depth and overall beam depth is complex. For purposes of evaluating load bearing housings, the most applicable range of d_n/d is from 0.15 to 0.5.

Determining how the width of the bearing surface, measured perpendicular to the grain of the beam, affects the housing capacity as that dimension changes from a practical minimum of, say, 1/2 inch up to the full width of the beam, as in the case of through-tenon mortises, would be of value in guiding development of predictive models.

The housings demonstrated varying degrees of ductility or toughness and suggested that higher strength woods have both higher load bearing capacity and increased toughness. Whether this behavior is a species dependent characteristic or can be related to specific gravity remains to be explored.

Since butt cog housings of the type tested here typically occur in multiples along the length of a beam, testing is needed to examine multiple housings and how cumulative shear in a beam affects the capacity of the load bearing housings closest to the end supports of a given beam. Testing is also needed to examine what effect spacing of the housings has on peak capacity for a given bearing depth.

We need to develop a better understanding of the failure mechanisms and process. Preferably we would be able with this information to develop a design formula that is conceptually better related than NDS© Equation (3.4-7) to the fracture plane that appears characteristic of the failure mechanism.

Conclusions

While the testing performed has been limited and relatively unsophisticated, the results do provide a preliminary indication of the capacity of load bearing housings taken in isolation, and of their behavior as the limit state is reached.

Once a reasonable database of test results across a variety of geometries, species, and spacings is available, the next task will be to take up numerical modeling so that reliable predictive design equations can be developed that apply across a broad range of geometries and species for use in timber frame design codes. The goal is to develop a capacity-based design formula for incorporation in *Standard for Design of Timber Frame Structures and Commentary*, TFEC 1, and the *Timber Frame Design Standard* (currently under development by the American Wood Council).