STRUCTURE FOR AN ARCTIC HOUSE

prepared for:

CANADA MORTGAGE AND HOUSING CORPORATION

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STRUCTURE FOR AN ARCTIC HOUSE

EXECUTIVE SUMMARY

Canada Mortage and Housing Corporation commissioned the development of a prototype Arctic house which has been designed and built in North Vancouver on property belonging to the Council of Forest Industries of British Columbia adjacent to their research and development laboratory. The structure is of interest technically because it is supported on four* spot footings rather than the conventional concrete perimeter foundation. To do so, the longitudinal exterior walls have been rigidly connected to the roof and main floor to form a "tube" or "monocoque" which functions much the same way as the unibody of a car or the fuselage of an aeroplane. The purpose of the spot footings and monocoque is to enable the building to withstand differential movement of foundations on discontinuous permafrost. The structure is "stick built" and uses spruce plywood, spruce dimension lumber, 20 gauge galvanized sheet steel and a large number of power driven nails.

Most components were prefabricated by students at B.C. Institute of Technology, and put together by a carpenter** and helper at the job site.

From an architectural point of view, the building provides 1000 square feet of main floor space, and 500 square feet on the second floor, interrupted only by the legs of the rear frame. Partitions can thus be put anywhere, or the floor plan left open. The foundations and structural frame cost approximately $18,000.00.

* - the building could be modified to rest upon three foundations.

** - Dave de Goutier of Ladner, B.C.

INTRODUCTION

Arctic foundation designs are vulnerable to the unique permafrost soil conditions and related problems of the north and have a history of less than satisfactory performance. Differential displacement due to frost heaving and permafrost degradation in northern soils has been the major problem. The effect of peat in the northern provinces and swelling clays in the Winnipeg region has been serious racking and subsequent severe damage to the foundation, building frame, openings and finishes. The loss of air tightness and on-going repairs serve to increase operating costs. A reliable, simple and cost-effective solution to this problem is highly desirable. The cost of piles and additional building materials required to solve the problem in northern and remote areas is high and therefore it would be more cost effective if the structural capacity of these materials could be optimized.

Piles, and pad and wedge foundation systems are in common use in Arctic and remote locations with some success. Alternative foundation design concepts have been tried in various northern locations, including three bearing-point metal tube spaceframe, and a buried pad and pier arrangement.

Stress skin panels and plywood box beams illustrate how standard sheathing materials have been utilized for a greater structural purpose. This project demonstrates how normal Canadian stick frame construction techniques and materials can be engineered to produce a stronger building that can withstand severe structural loading. In this case the loading is due to concentration of weight into spot footings. A similar approach could be used to produce buildings with a superior resistance to wind or earthquake loads.

DESIGN

The prospect of supporting 50 tons* of house on 4 spot foundations without recourse to heavy support beams was fairly daunting. However, after several attempts, some of which never left the calculation pad, let alone the drawing board, a workable concept emerged. The house would have to be stick-built, meaning only commonly used house building materials: plywood, dimension lumber, and nails, to which was added thin sheet steel to reinforce connections. It was apparent that the main floor would have to be hung from the side walls and that the side walls (and one end wall) would have to transfer these loads to the foundations. Openings in these walls would have to be minimal and no jogs permitted. Three quarter inch spruce plywood would do the job and calculations showed that this thickness in the main floor, roof and side walls would form an adequate monocoque shell.

As in all engineered wood structures, the connections were difficult: effective nailing required much more than the conventional 3/8 inch edge distance between nail and sheathing panel edge, so wider framing was required. To span the floor 28 feet without using proprietary trusses meant plywood web beams, and it was the selection of an "I" section for the floor beams that suggested the framing system for the entire house. The critical connection of the floor beams to the studs backing the side walls could be made by using the plywood as a tongue secured to framing lumber on either side by nails acting in double shear. (see figure 1)

*- see Engineering Section

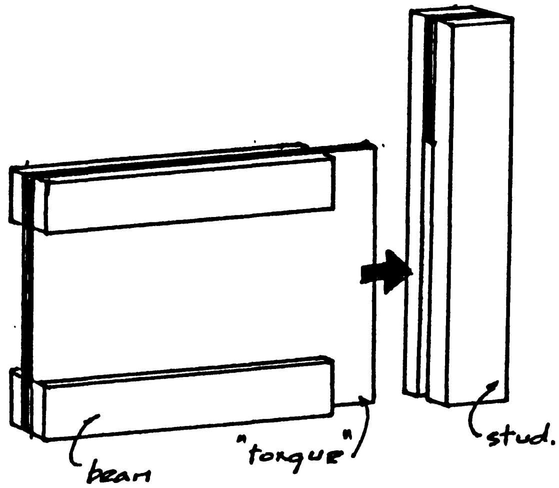


figure 1

This principle was applicable throughout the entire cross section of the house and benefits resulted from provision of significant end fixity in all members. The relatively large contacting areas in the tongued connections could also accommodate large numbers of nails without having to impose tight dimensional tolerances on nail spacing. (see figure 2).

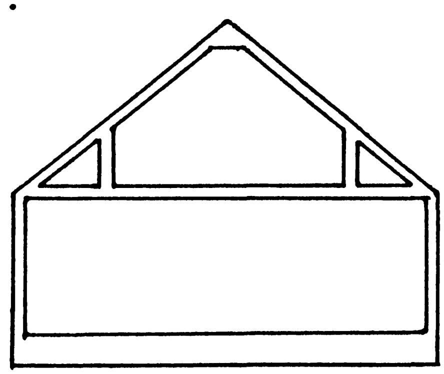


figure 2

Having thus "doubled up" on studs, rafters and floor beams, the need for economy dictated that the framing be spaced at 4 feet on centre. However, because the open floor plan placed minimal restrictions on interior arrangement, a 4 foot module could be made to work. The problem now was that every 8 feet, at the ends of the plywood panels, there was no effective connection. It became necessary to invent "reinforced sheathing", by introducing sheet metal gussets connecting the corners of the plywood panels. (see figure 3)

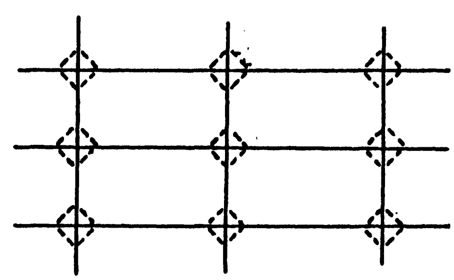


figure 3

Sheet metal splices were already planned for the "I" beams supporting the main floor. Four sheets of 20 gauge (.036 inch) galvanized steel are incorporated in a "sandwich" splice at midspan of the beams. It was known from previous work that a nail could be driven quite easily through two thicknesses of lumber, one thickness of plywood, and three thicknesses of sheet steel. (see figure 4)

DESIGN (continued)

Note how the sheet metal "bursts" at the nails to increase the area of bearing on the wood. The sheet metal is the same material that is used in gang-nail connections except that, unlike the latter, very little strength is lost to perforations.

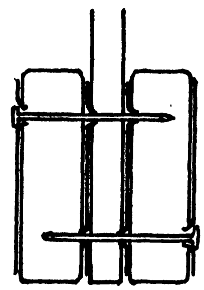


figure 4

The sheet steel sheathing reinforcement acts directly with the plywood and the lumber framing serves only as backing. Thus the splitting of wood is not critical and the concentration of nails may be increased at will.

Now we had a monocoque tube. But how to support it?, and where?

Figure 5 illustrates the conceptual frame which serves several functions.

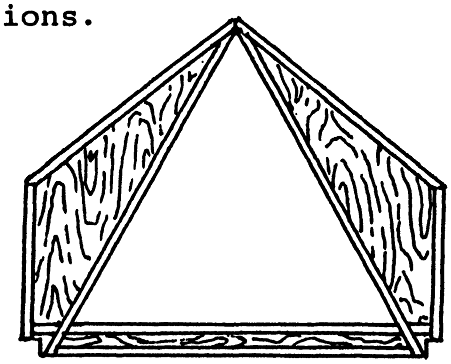


figure 5

DESIGN (continued)

(1) To transfer wall and beam loads to compression members which could be supported by spot foundations.

(2) To provide torsional stability. Try twisting rectangular tubes and compare the stiffness of an open ended tube with that of one with closed ends.

(3) To provide the house with lateral stability. This is in keeping with other features of the design which are intended to resist high wind loads: The house sits as close to the ground as possible, lacks roof overhangs and has low side walls and a pitched roof.

(4) To allow for openings. Because the side walls are so highly stressed, openings in them must be kept small. Access and decent light are available only through the end wall frames.

The location of footings, and hence the location and number of frames was designed to minimize the stresses in the monocoque. Figure 6 compares stress producing parameters resulting from the simplest arrangement (footing at each corner) with those in the final structure:

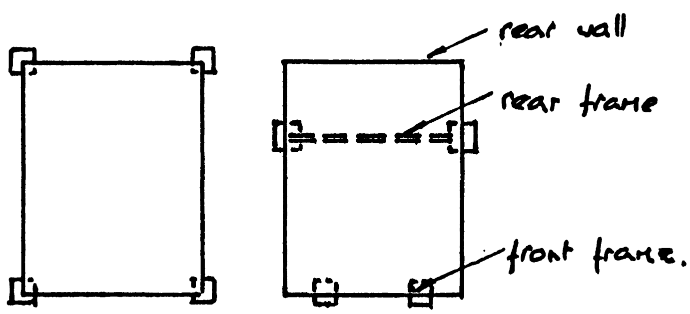


figure 6

			Reduction
Flexure:	400 K'	180 K'	55%
Shear:	46 K'	38 K'	18%
Torsion:	622 K'	168 K'	73%

Thus the basic arrangement was determined by engineering considerations. The resulting floor plan however, provides width for rooms on either side of central access, and enough length for two rooms in front of the rear frame and one room between the rear frame and the rear wall. Technically only two frames are required. However, the rear frame is a lamination of two frames, and the rear wall, which carries very little load, is fabricated the same way as the front frame for convenience of construction.

The size of footings is dictated by a design bearing pressure of 1500 pounds per square foot. Analysis of the effect of wind pressure upon the building shows that although the building is heavy enough, and the rear footings are sufficiently far apart, to resist a considerable overturning force, provisions must be made to prevent the building from sliding. The lower beam provides resistance to sliding as well as helping to distribute the vertical load into the footing. Also shown are auger anchors to provide additional resistance to uplift and overturning.

Both upper and lower beams project beyond the footing to provide for a pair of hydraulic jacks which would be used to level the building. After jacking, the level would be maintained by placing blocks of the appropriate size between the footing and the upper beam.

With some modification to the threaded rod arrangement, the upper beam could be supported temporarily on needle beams while the lower part of the foundation was being replaced.

CONSTRUCTION

Before the ink had dried on the drawings there were two major changes: 1), the B.C. Institute of Technology agreed to have their carpentry students prefabricate the structure, and 2), the designer's intention to use common nails was thwarted by everyone else on the project wanting to use a power nailer.

The first change meant producing shop drawings, and the second resulted in doubling the number of nails.

Shop drawings turned out, in retrospect, to be invaluable. Whereas such components as the floor beams would have been prefabricated in any case, the frames and so forth would have been very difficult to make given only the dimensions shown on typical house plans. Furthermore, with the geometry established and all components prefabricated, drawings for use on site by the carpenter could be very simple; limited to overall dimensions and connection details. Every effort was made to keep detailing simple. A convention whereby the edges of plywood panels were shown as shaded lines, and lumber and sheet metal outlines in full lines, worked well, but the typical nailing detail was misinterpreted, as were such niceties, common in engineering, as tolerances and handing.

While the students were assembling components in the B.C.I.T shop, the carpenter had the site stripped, concrete foundations (with provisions for hydraulic jacks) made, a drained lime rock surface placed, and hardware fabricated. After the components arrived on site, the floor beams were set on temporary beam supports, blocking and bracing installed, sheet metal reinforcement placed, and plywood sub-floor nailed in place.

The nails throughout the job were by Bostitch. All were .12 inches in diameter, and only two lengths were used; 3½ inches for the components and 2½ inches for the sheathing. Nails were restricted (except at some splices) to the outer third of the lumber members leaving the middle strip available for bolts. The nailing gun worked extremely well; in fact too well. There was a noticeable tendency both in the shop, and on the site, to drive more nails than called for on the drawings.

One of the advantages of using an "I" configuration with plywood sandwiched between lumber, is that splitting of the wood is immediately apparent, (which is not the case with box type plywood components). In any event splitting did not occur anywhere. Nail spacing in lumber was 3 inches and, into metal gussets, 2 inches.

The floor now formed a platform on which the frames could be nailed or bolted together, and pockets around the perimeter of the floor were cut to accommodate frames and studs. The fully assembled main frame was too heavy to lift by hand, so a crane was brought in to lift all the frames into position. With the frames and studs in place, wall sheathing was installed, and then the problems began.

Dimensional inaccuracies in the frames (a jig should have been used) and absence of a traditional upper wall plate made the setting of the attic beams, (which again required a crane) rafters, and upper floor joists extremely difficult, and much time and effort was spent before the roof was completed. With the benefit of hindsight it is clear that, 1) floor beams must be accurately positioned, and 2) attic beams should be erected before the side walls.

The use of 24 foot lengths of lumber (attic beams, ceiling joists and frames) is another feature that was questioned. It was found, however, that the cost of splicing shorter pieces was such that the extra effort in finding 24 foot lengths of lumber is well worth while.

COST

The imposition of a $15,000 budget early on in the design process had beneficial results. The combination of a tight budget with COFI's requirement for clear floor space forced the designer to take various decisions which seemed radical at the time, but now can be seen to have resulted in an unorthodox, but practical, and economic, structure.

Actual costs were as follows:

Lumber		$2,592.98
Plywood		$4,046.23
Nailer & Nails		$1,529.58
Hardware		$1,817.96
Equipment Rental		$1,158.00
Concrete & Gravel		$ 438.10
Other - Caulking & Hydraulic Jacks		$ 172.87
SUB - TOTAL:	MATERIALS	$11,755.22
	LABOUR	$ 6,000.00
TOTAL		$17,755.22

ENGINEERING

In addition to the previously described optimizing of the arrangement of the foundations, it should be pointed out that the cantilever action of both monocoque and attic beams tends to localize the maximum bending moments, and although there is a joint right at the root of the rear cantilever, it is preferable, in spite of a general design philosophy of keeping things uniform, to introduce increased sheathing reinforcement (in the form of 24 inch steel gussets) at this one section, rather than go for completely uniform, but excessive, reinforcement throughout. In fact a major feature of the design is the repetition of details. The structural arrangement is such that the same nail spacing applies to most members.

The rear frame carries 75% of all loads, and that it does so, is clearly reflected in its construction. Consisting of two frames bolted together, it provides:

(1) 8¼ inch thickness to resist compression buckling
(2) Four vertical members to which the side walls can transfer their load through a double row of nails.
(3) An easy bolted connection to the floor beam.

Other features to be noted are:

(1) Front and rear walls and the two wythes of the rear frame all have the same external geometry which means only one frame jig is required.

ENGINEERING (continued)

(2) The use of full size panels wherever possible cuts down the number of connections.

(3) The sheet steel reinforcement can be introduced in any joint without disruption of wood or plywood arrangement.

(4) The sheet steel reinforcement can be bent and thus transfer shear around awkward corners such as at eaves or ridge.

(5) From long experience in engineering light timber frame structures, the writer derives considerable reassurance from the "doubling" of the lumber in structural members. There is always the chance that a single "low strength" piece of lumber can critically weaken a structure.

The "I" section beams do have one disadvantage, and that is that they tend to buckle. For this reason the design is such that members brace one another wherever possible:

: Floor beams are braced by steel tube bracing at the lower flange and the main floor at the top flange.

: The attic beams are braced by the hangers which run between the rafters and the ceiling joists.

: The frames are braced by the attic beams.

The following is a digest of the actual calculations and contains some simplifications and approximations.

Loads are as follows:

i)	Structure as built (spruce at 25 pcf)	18K
ii)	Anticipated finishes (floor finishes, partitions, gypsum wall board, glazing, interior finish)	: 10K
iii)	Snow load (40 psf GSL 26 psf on roof)	: 26K
iv)	Upper Floor occupancy load (@ 30 psf)	: 16K
v)	Main Floor occupancy load (@ 40 psf) x .75, area reduction factor)	: 30K
		100K
	=	50 Tons

Loads are distributed to the main structural elements as:

i)	Floor beam (main floor loads)	: .19 Klf
ii)	Half monocoque (half main floor and quarter of roof)	: .74 Klf
iii)	Attic beam (half upper floor and quarter of roof)	: .52 Klf

Reaction from side walls and attic beams:

i) To rear frame (.74 + .52 x $\frac{36 \times 18}{24}$ = 34.0K

ii) To front frame (.74 + .52 x 36 - 34 = 11 K

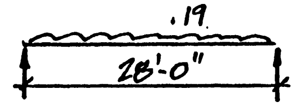
TOTAL 90 K*

* This weight is 10 K short of 100 K total because some loads are supported directly by the frames.

ENGINEERING (continued)

Shear force and bending moment diagrams

FLOOR BEAM



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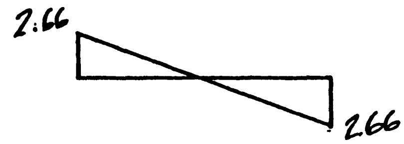
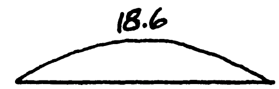
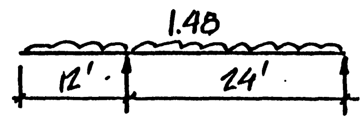


BMD



MONOCOQUE



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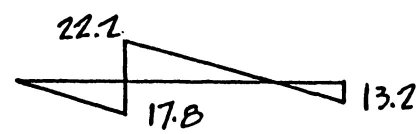

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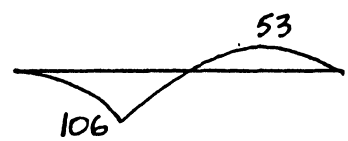
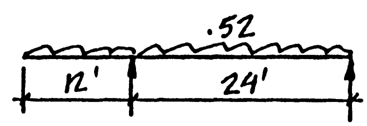
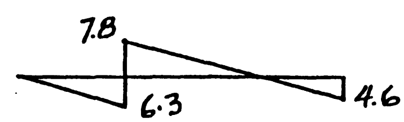


ATTIC BEAM



SFD



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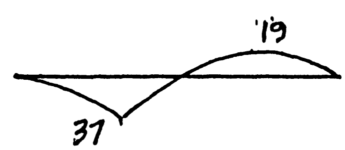


ENGINEERING (continued)

Shear Stresses

Floor beam : $\frac{2.66}{24 \times .75}$ = .148 Ksi

Side wall : $\frac{22.2}{2 \times 102 \times .75}$ = .145 Ksi

Attic beam : $\frac{7.8}{48 \times .75}$ = .220 Ksi

Stresses due to bending

Floor beam : $\frac{18.6 \times 12}{18.5 \times 2 \times 1.5 \times 5.5}$ = .73 Ksi

Monocoque : $\frac{106 \times 2^*}{12 \times 28 \times 12 \times 0.4}$ = .13 Ksi

Attic beam : $\frac{37 \times 12}{42.5 \times 2 \times 1.5 \times 5.5}$ = .63 Ksi

* factor to allow for shear lag

Compressive Stresses

Rear frame : $\frac{34.0}{4 \times 1.5 \times 5.5}$ =1.03 Ksi

Front frame $\frac{11.0 \times 2^*}{3 \times 1.5 \times 5.5}$ =0.91 Ksi

* total front frame reaction : torsion condition

Maximum Torque : 11.0 x 2 x 7.5 = 168 K'

Torsion Constant : $\frac{2 \times .75^2 \times 336^2 \times 120^2}{.75 \ (336 + 120)}$ = 5.35×10^6 (approx.)

Torsion Shear : $\frac{168 \times 12 \times 12}{2 \times .75 \times 336 \times 120}$ = .033 Ksi

Torsion Rotation : $\frac{168 \times 12 \times 24 \times 12}{5.35 \times 10^6 \times 94}$ = .0011 radians

Torsion Deflection : .0011 x 14 x 12 = .19 inches

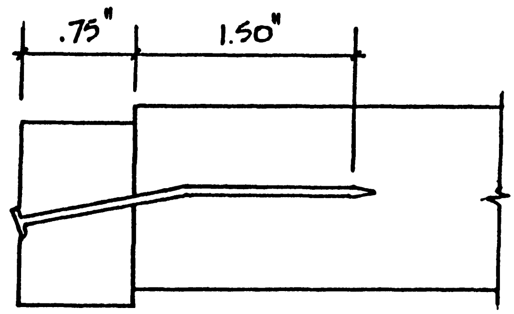
ENGINEERING (continued)

Nailed Connections

The decision to change from 3½ inch common nails to Bostitch machine driven nails not only necessitated an increase in the number of nails, but also put the design of the connections beyond the scope of the current edition of CSA 086, the Canadian Code for Engineering Design in Wood. In a search for guidance, 1986 U.S.D.A. publications by Aune and Patton-Mallory, (A, P-M) appeared to offer a means of determing ultimate capacities for laterally loaded nailed connections with other than common nails.

The parameters required in this method are simple: nail yield stress (F_Y), wood embedding stress (F_E), nail diameter (d), and constants for various joint configurations.

CASE 1



Sheathing connection

2½" Bostitch nail (d = .12")

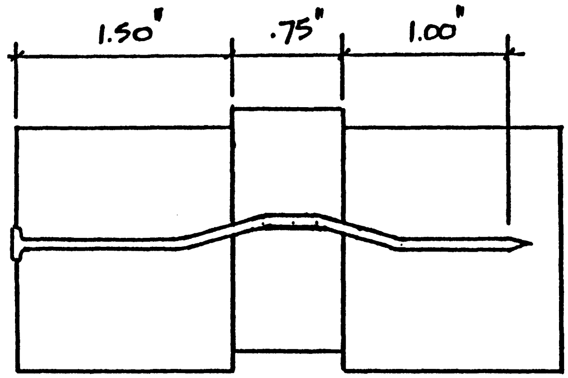
Ultimate capacity $V_u = C_c\ F_E\ d\ 1$

$= .45 \times 4.2 \times .12 \times .75$

$= .170$ K

ENGINEERING (continued)

CASE 2



Tongue connection

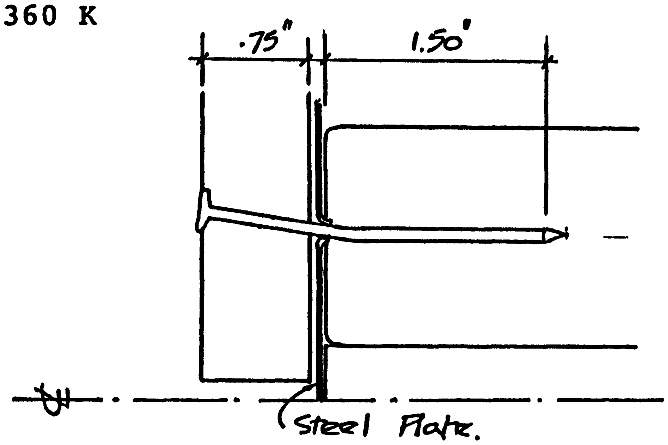
3½" Bostitch nail (d = .12)

$$V_u = Cm\sqrt{F_y F_E}\, d^2$$

$$= 1.16\sqrt{112 \times 4.2} \times .12^2$$

$$= .360\ K$$

CASE 3



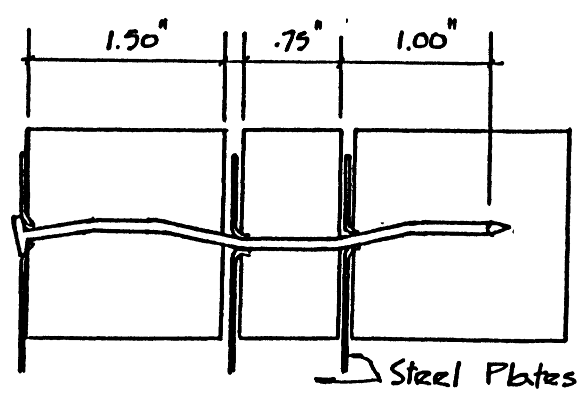
Reinforced sheathing

2½" Bostitch nail

$$V_u = .75\ (1.15 + 1.63)\sqrt{112 \times 4.2} \times .12^2$$

$$= .325\ K$$

CASE 4



Beam Flange Splice

3½" Bostitch nail

$$V_u = (2 \times .82 + .58)\sqrt{112 \times 4.2} \times .12^2$$

$$= .693\ K$$

ENGINEERING (continued)

Now, working from the previously derived shear force and bending moment diagrams, the connection loads will be determined, and then, from the number or spacing of nails in each component on the drawings, a "factor of safety" (FOS) will be calculated, equal to the ultimate nail capacity (V_u) multiplied by the number of nails, and divided by the applied load.

Floor Beam:

1) Maximum horizontal shear $= \dfrac{18.6 \times 12 \times 2}{18.5 \times 14} = 1.72$ Klf

number of nails per foot $= 2 \times 2 \times 2 = 8$

FOS $= \dfrac{.360 \times 8}{1.72} = 1.67$

2) Flange splice load $= \dfrac{18.6 \times 12}{18.5} = 12.1$ K

Splice plate stress $= \dfrac{12.1}{4 \times 5.0 \times .036} = 16.6$ Ksi

Number of nails in splice $= 3 \times 2 \times 2 \times 3 = 36$

FOS $= \dfrac{.693 \times 36}{12.1} = 2.06$

3) Stud Connection: load $= 2.66$ K

Number of nails $= 2 \times 2 \times 2 \times 2 = 16$

FOS $= \dfrac{.360 \times 16}{2.66} = 2.15$*

* because this connection is critical, it was subsequently reinforced with sheet metal gussets.

ENGINEERING (continued)

Monocoque:

1) Maximum side wall shear $= \frac{22.2}{2} = 11.1K$

No. of "reinforced" nails $= 2 \times 2 \times 6 \times 2.8 = 67$

No. of "sheathing" nails $= 2 \times 4 \times 3.4 = 27$

FOS $= \frac{67 \times .325 + 27 \times .17}{11.1} = 2.37$

2) Maximum roof tension $= \frac{106}{12} = 8.8\ K$

No. of "reinforced" nails $= 8 \times 26 = 208$

FOS $= \frac{208 \times .325}{8.8} = 7.7$*

* - allows for shear lag

3) Maximum torsion per foot $= .033 \times .75 \times 12 = .297$ Klf

effective "reinforced nails"

per foot $= \frac{26}{2 \times 4} = 3.25$

FOS $= \frac{3.25 \times .325}{.297} = 3.55$

Attic Beams:

1) Maximum horizontal shear $= \frac{37.0 \times 12 \times 2}{42.5 \times 12} = 1.74$ Klf

number of nails per foot $= 2 \times 2 \times 2 = 8$

FOS $= \frac{.360 \times 8}{1.74} = 1.65$

Application of the Aune, Patton-Mallory ultimate capacity equations to simple configurations using common nails indicates that the A,P-M values are about 3 times the commonly established working loads for common nails. Thus the design of the nailed connections in the monocoque building can be seen to be less conservative than the sizing of the structural numbers which generally comply with engineering code requirements.

Before construction commenced a series of preliminary tests on connections were run, not only to confirm load values but also to check out the performance of the nailing gun, check that the nails would not split the wood, and that nail location tolerances could be maintained without marking nail location with a template.

Test were performed in the COFI laboratory by the writer, assisted by the carpenter for the project, and laboratory staff.

First, the Bostitch nails themselves were tested in bending and the yield strength was found to be in excess of 100 Ksi, the figure of 112 Ksi used in preceding calculations being adopted from the Aune, Patton- Mallory report.

Next the two specimens illustrated on drawing 2020-1, were tested to check out the strength of the reinforced sheathing connection. Load/slip lines are shown on graph No. 1. Note that the A,P-M ultimate load applies only to lateral nail loading and that laterally loaded nailed connections develop additional strength in with-drawal when deformation becomes sufficiently large.

Finally, two "tongued" double shear connections were tested. Specimen data and results are shown on graphs No. 2 and 3.

In performing the test, the results of which are shown on graph No. 3, an attempt was made to find an elastic yield point. Altering the A,P-M equation by substituting elastic for plastic modulus of the nail, fibre stress at proportional limit for embedding strength of the wood, and adjusting the configuration factor,

$$V_p = C_{CP}F_c d\, l_{SMIN}$$

$$= .67 \times 3.7 \times .12 \times .81 = .240$$

Furthermore, the deflection of the nail at this load can be approximated by the expression:

$$\text{deflection} = \frac{F_c d\, l^3_{SMIN} l_M}{54\ EI}$$

and substituting numerical values in this,

$$\text{deflection} = \frac{3.7 \times .12 \times .81^3 \times .75}{54 \times .295}$$

$$= .011 \text{ inches}$$

This value compares well with observed cyclical deflection shown on graph No. 3. Non-cyclical deformation is probably attributable to "setting" strain in the wood. Note that this separation of wood strain and nail yield is an excellent reason for testing nailed connections in double shear.

Although full scale testing of the completed structure is problematic, and beyond the scope of this report, two simple tests have been performed to date.

First string lines were set along each of the side walls, and deflection at the rear wall of the building was noted as the temporary supports of the rear wall were removed.

The immediate deflection was less than 0.25 inches and after 5 months this figure had increased to 0.46 inches.

Second, the northerly of the two posts supporting the front wall was raised 1½ inches by a hydraulic jack and the resulting deflection of the southerly post was measured. The immediate deflection was 0.25 inches (equivalent to .0014 radians) and after 5 months this figure had increased to 1.0 inches.

GOWER YEUNG & ASSOCIATES

CNS 29.5.89.

CMHC OTTAWA : MONOCOQUE #1.

PRELIMINARY TEST #1.

TESTED 25 MAY '89.

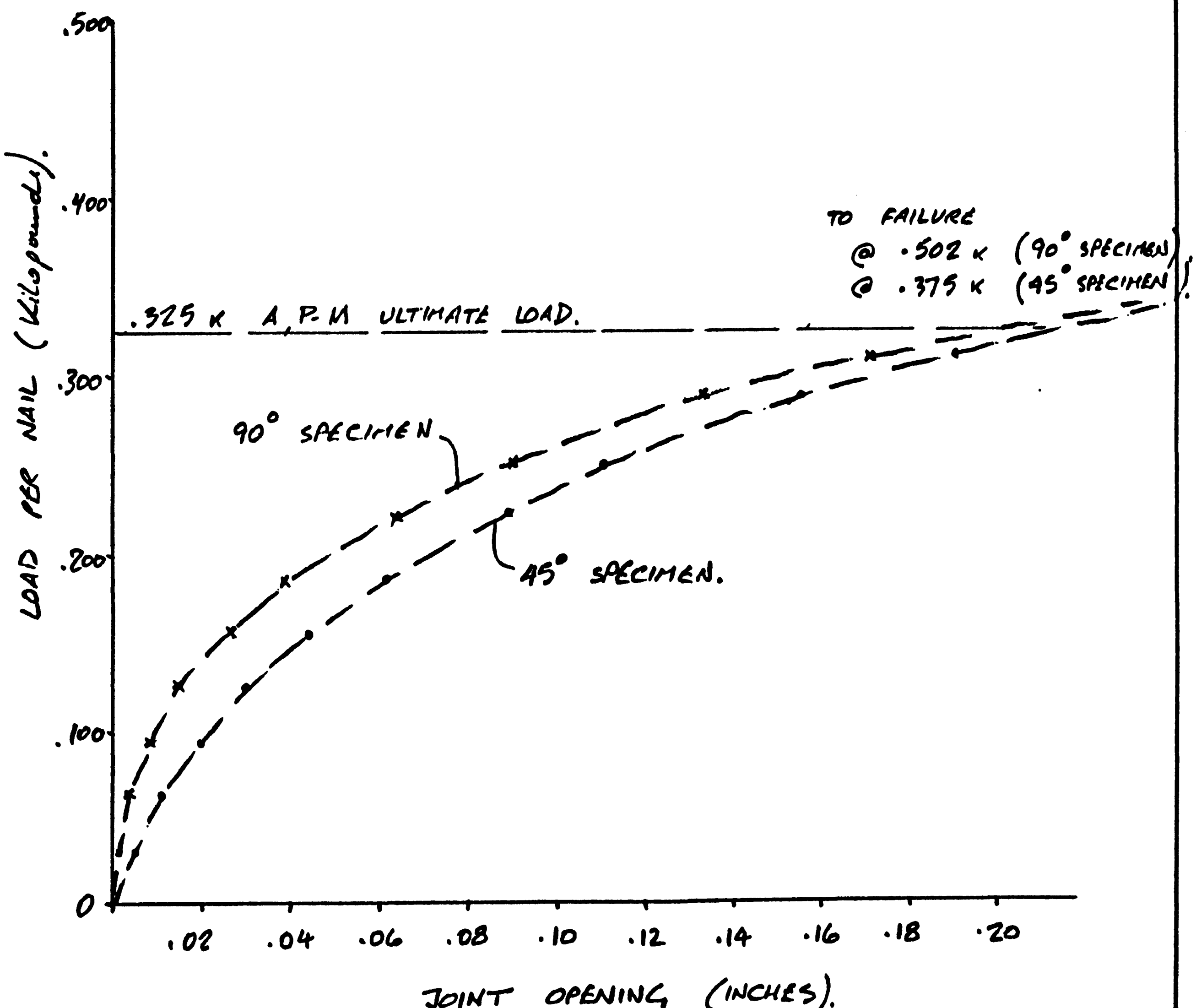
EQUIPMENT : TINIUS OLSEN

: DIAL GAUGES.

SPECIMENS : SEE DRG 1891. P9.

RATE OF LOAD APPLICATION

: APPROX. 20 SECONDS PER LOAD INTERVAL.



GRAPH No 1.

GOWER YEUNG & ASSOCIATES. CNS 29.5.89.

CMHC OTTAWA : MONOCOQUE #1.

PRELIMINARY TEST #2

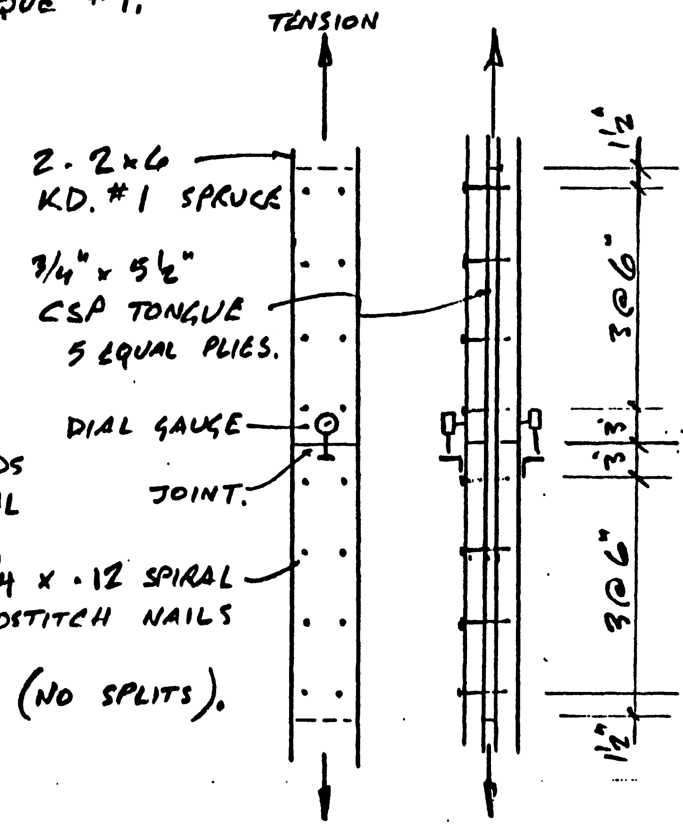
TESTED 11 MAY '89.

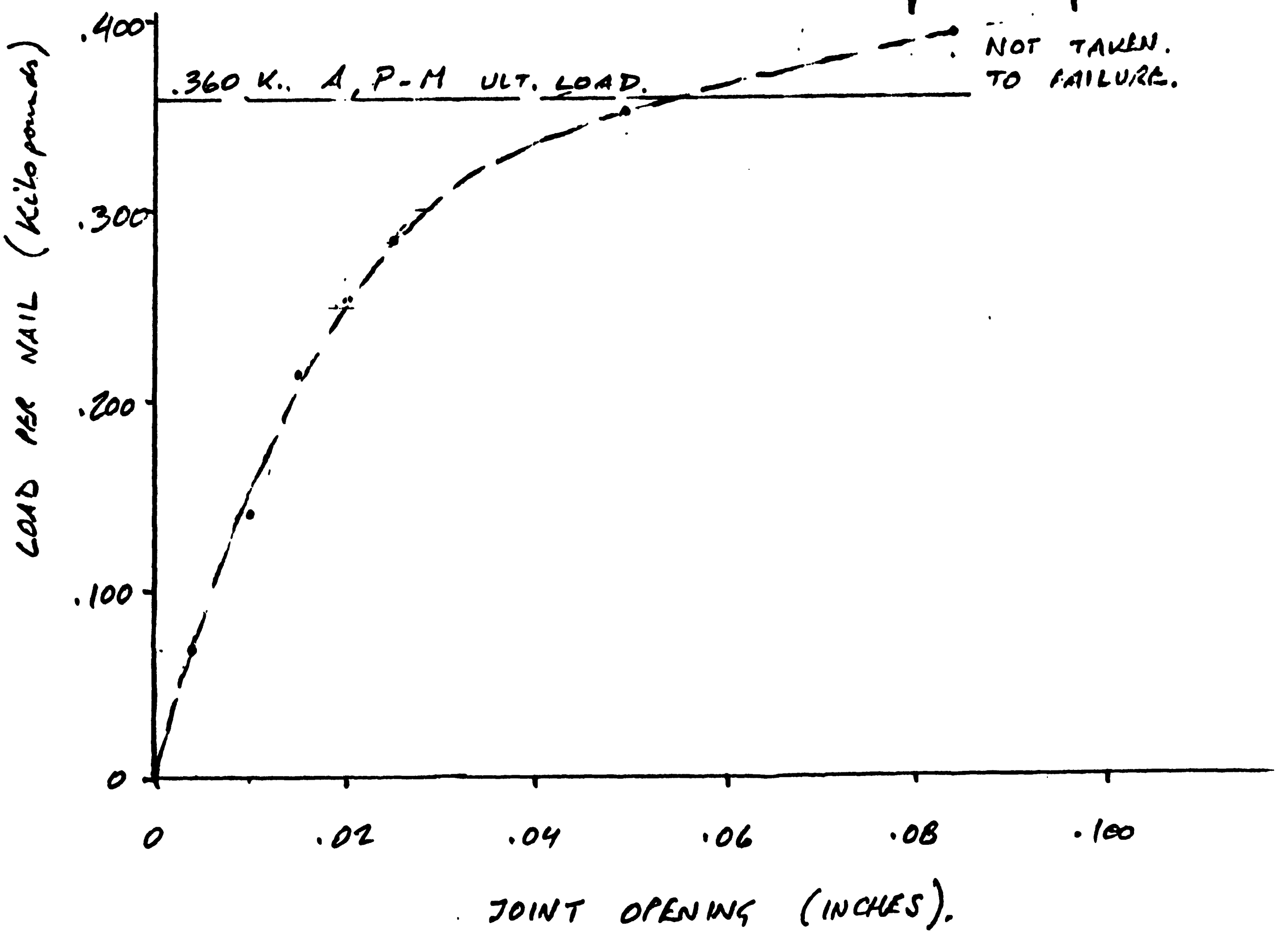
EQUIPMENT : TINIUS OLSEN,

DIAL GAUGES.

RATE OF LOAD APPLICATION :

: APPROX 20 SECONDS PER LOAD INTERVAL





GRAPH No. 2.

GOWER YEUNG & ASSOCIATES. CNT 29.5.89

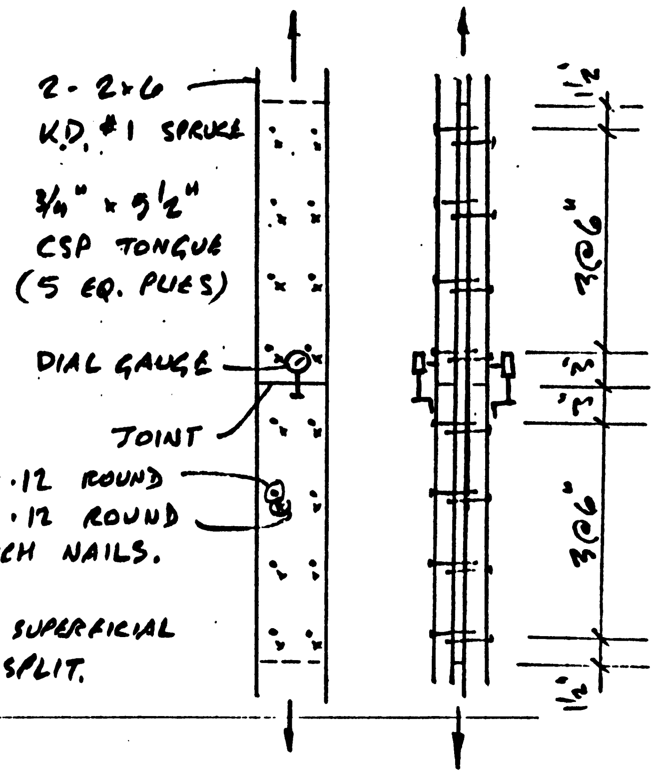
CMHC OTTAWA : MONOCOQUE #1.

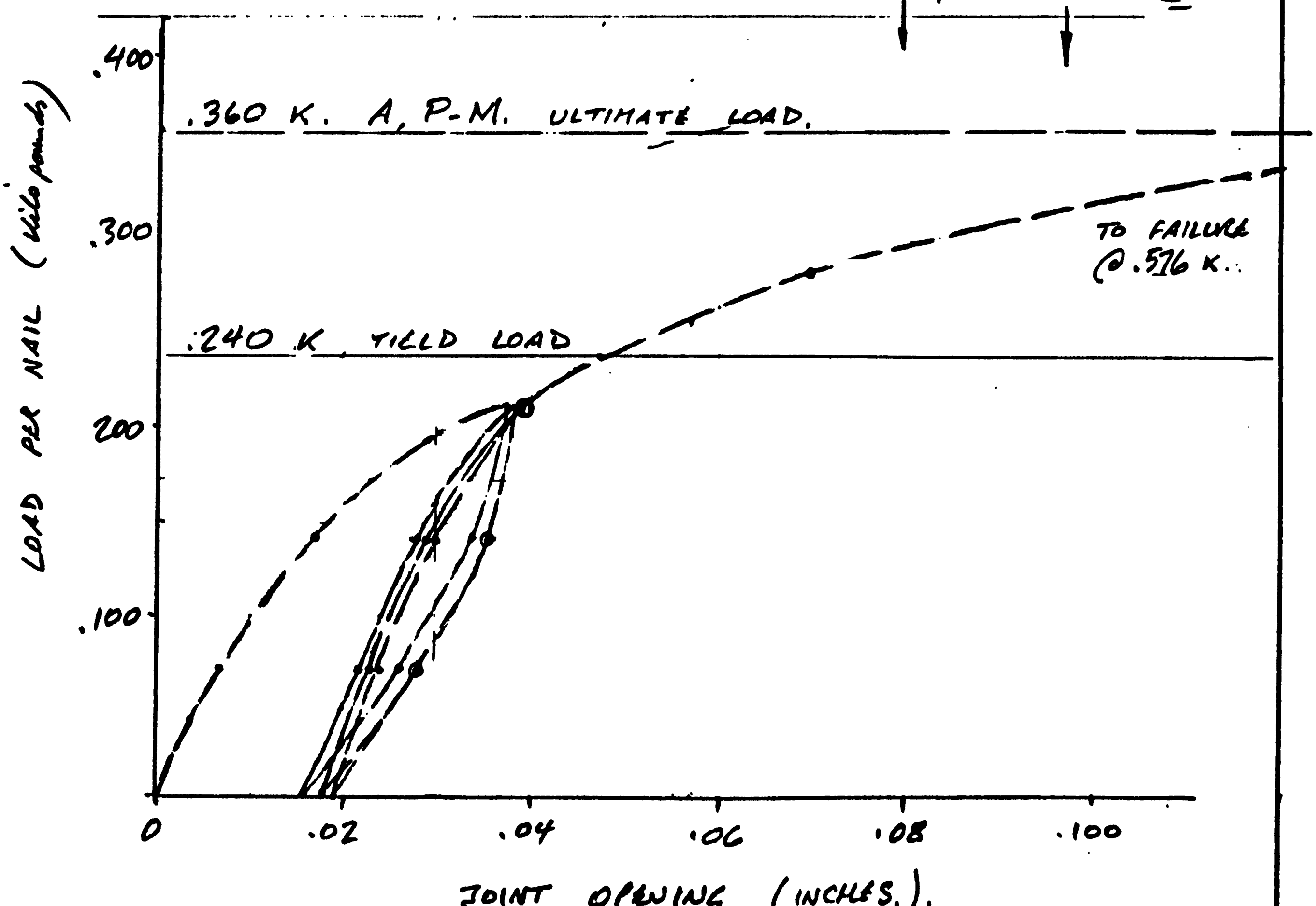
PRELIMINARY TEST #3

TESTED 26 MAY '89

EQUIPMENT : TINIUS OLSEN
DIAL GAUGES.

RATE OF LOADING :
30 SEC. / LOAD INTERVAL
CONTINUOUS.





GRAPH No 3.

CONCLUSIONS:

From an architectural point of view the monocoque house is truly an example of form following function. In addition to the distinctive appearance deriving from the structural function of the frames, the exposed floor beams, steep roof pitch, lack of overhangs, and limited wall openings, are all intended to meet the functional requirements of an Arctic environment, at minimum cost, and without recourse to sophisticated technology.

The structure is, of course, only part of a house, and it is hoped that this structure will not cause insuperable difficulties to those who design and specify finishes, insulation, services and all the other aspects of the finished house.

An unforeseen bonus from having to meet Council of Forest Industries' requirement for an open floor plan, is the flexibility to arrange the interior to suit individual requirements. The only constraint in arrangement is the rear frame, which effectively separates the "front" from the "rear" of the building. As previously mentioned, COFI preferred to have the open area at the front, but there is a choice of orientation and that choice is purely architectural.

This, then, summarizes aspects of the fully developed concept.