A los asistentes a los cursos del Centro de Educación Continua

La Facultad de Ingeniería, por conducto del Centro de Educación Continua, otorga constancia de asistencia a quienes cumplan con los requisitos establecidos para cada curso. Las personas que deseen que aparezca su título profesional precediendo a su nombre en el diploma, deberán entregar copia del mismo o de su cédula profesional a más tardar 15 días antes de la terminación del curso, en las oficinas del Centro, con la Sra. Sánchez.

El control de asistencia se efectuará al terminar la primera hora de cada día de clase, mediante listas especiales en las que los interesados anotarán personalmente su asistencia. Las ausencias serán computadas por las autoridades del Centro.

Se recomienda a los asistentes participar activamente con sus ideas y experiencias, pues los cursos que ofrece el Centro están planeados para que los profesores expongan una tesis, pero sobre todo para que coordinen las opiniones de todos los interesados constituyendo verdaderos seminarios.

Al finalizar el curso se hará una evaluación del mismo a través de un cuestionario diseñado para emitir juicios anónimos por parte de los asistentes.

Las personas comisionadas por alguna institución deberán pasar a inscribirse en las oficinas del Centro en la misma forma que los demás asistentes.

USOS ESTRUCTURALES DE LA MADERA

Fecha	Hora	Duración	Tema	Profesor
Lunes	9-9:30	30 min.	I. INTRODUCCION	Ing. Francisco Robles
			II. LA INDUSTRIA DE LA MADERA Y RECURSOS FORESTALES	
"	9:30-10:30	1Hs.	a) Desde el punto de vista oficial	Ing. Salvador Vazquez R.
"	10:30-11:30	1Hs.	b) Desde el punto de vista de la industria	Sr. Carlos Rodríguz A.
"	11:30-13:00	1:30Hs.	III. PROPIEDADES DE LA MADERA	Ing. Ramón Echenique
			COMIDA	
"	15-17	2:00Hs.	IV. ELEMENTOS DE MADERA Y TRIPLAY	Ing. Federico Martínez de Hoyos
"	17-18	1 Hs.	V. TABLEROS DE MADERA	Ing. Federico Martínez de Hoyos
Martes	9-10:30	1:30Hs.	VI. CRITERIOS DE DISEÑO	Ing. Roberto Meli
			VII. SISTEMAS ESTRUCTURALES DE MADERA Y TRIPLAY	
"	10:30-12:30	2:00Hs.	a) Techos económicos en claros pequeños y grandes. Vigas de madera y vigas de alma abierta. Armaduras de madera	Ing. Robert G. Sexsmith.
"	12:30-13	30 min.	Discusión	
			COMIDA	
"	15-17:30	2:30 Hs.	b) Sistemas para vivienda. Techos y sistemas de pisos de triplay. Armaduras de techo. Problemas de seguridad contra incendio. Consideraciones económicas y factibles en México.	Ing. Francisco Robles. Dr. Ramón Echenique.
"	17:30-18	30 min.	Discusión	

Fecha	Hora	Duración		Tema	Profesor
Miercoles	9-11	2Hs.	c)	Puentes de madera y concreto	Ing. Robert G. Sexsmit
"	11-13	2Hs.	d)	Conexiones. Clavos, pegamento, pernos, anillos, placas de cortantes, placas con clavos	Ing. Jehová Guerrero y Torres.
				COMIDA	
"	15-18	3Hs.	e)	Estructuras de placas plegadas de triplay	Dr. Roy Hooley
Jueves	9-10:30	1:30Hs.	VIII.	LA MADERA COMO GENERADORA DEL ESPACIO ARQUITECTONICO	Arq. Jaime Ortiz M.
			IX.	EJEMPLOS DE PROYECTOS DE CONSTRUCCION DE ESTRUCTURAS DE MADERA	
"	10:30-13	2:30Hs.	a)	Auditorio en Puebla	Ing. Federico Martínez Hoyos. Ing. Jehová Guerrero y Torres.
				COMIDA	
"	15-16	1:00Hs.	b)	Instalaciones Portuarias, Postes y Torres en Madera	Dr. Roy Hooley
"	16-18	2 Hs.	c)	Estructuras para Puentes	Dr. Roy Hooley
Viernes	9-11:30	2:30Hs.	d)	Estado del Arte en EUA: Laminación, Preservación, Reglas de graduación, Diseño.	Sr. Karl Lindberg
"	11:30-13	1:30Hs.	X.	PROTECCION DE LA MADERA	
				COMIDA	
"	15-18	3 Hs.		Mesa Redonda.	

INTRODUCCION

Francisco Robles

> "La madera es el único material vivo que se emplea en la construcción, y, como todo lo que proporciona la vida, es algo menos rígido que los otros. El atractivo que tiene la madera procede, en gran parte, de sus cualidades vitales."
>
> Eduardo Torroja, en
> "Razón y ser de los tipos estructurales"

1. CONSIDERACIONES GENERALES

El interés actual en la madera como material estructural obedece en gran parte a que es un material vivo. En efecto, como ha dicho el Dr. Blomquist, "la madera es el único recurso natural renovable que puede producirse y manejarse como una cosecha y que tiene buenas propiedades estructurales". En esta época en que nos preocupa por una parte la crisis de energéticos y de minerales y por otra la creciente contaminación ambiental, parece que debe estar en auge un material como la madera, cuya transformación en material de construcción implica menor consumo de energía y menor contaminación del aire y del agua que los que caracterizan a la fabricación de materiales tales como el acero, el cemento, el aluminio y los tabiques. Debido a la ligereza de la madera, el ahorro de energéticos se consigue no sólo en los procesos de elaboración sino también en el bajo costo de transporte.

Por otra parte contribuye al interés de la madera como material estructural, el que la producción de productos derivados de la madera puede dar origen a multitud de industrias que podrían constituir una importante fuente de ingreso para los ejidos y para la población rural en general. Una ventaja de las industrias de la madera es que por regla general requieren inversiones iniciales bajas. Además las plantas pueden ser relativamente pequeñas sin menoscabo de su eficiencia.

La naturaleza viva de la madera se refleja en lo complejo de su estructura. Tanto sus cualidades como sus limitaciones se desprenden de esta estructura. En las palabras de Torroja: "Las fibras son la característica constitutiva esencial de la madera. La fibra le da su belleza, su expresión resistente, su estructura vital". La estructura fibrosa de la madera es el origen de su naturaleza anisótropa, que constituye un inconveniente desde el punto de vista de su uso como material estructural: es resistente a los esfuerzos normales paralelos a las fibras, pero es débil ante estas acciones en sentido perpendicular a ellas. También es baja su resistencia a esfuerzos cortantes paralelos a la fibras. Por otra parte es en las fibras donde reside el atractivo estético de las variadas texturas de la madera.

Una ventaja importante de la madera es su ligereza. Es de los materiales que puede desarrollar una mayor fuerza de tensión o compresión por unidad de peso.

Constituye una limitación de la madera la forma en que se encuentra en la Naturaleza: en piezas rectas de longitud mayor que sus dimensiones transversales. Tanto el tamaño como la forma imponen restricciones a las escuadrías posibles.

Desventajas adicionales de la madera son su tendencia a los cambios volumétricos con los cambios de humedad del ambiente, el aumento progresivo de las deformaciones bajo carga con el tiempo, la dificultad de realizar uniones adecuadas, el peligro de pudrición bajo la acción de determinados organismos vivos y el peligro de incendios. Estos inconvenientes, que pueden contrarrestarse en grado razonable, a veces se exageran. Incluso el comportamiento ante incendios, puede ser más favorable que el de otros materiales. La durabilidad puede ser considerable. . Existen techos de madera en Inglaterra que datan del siglo XIII y es frecuente encontrar elementos de chicozapote en las ruinas mayas que todavía cumplen con su función estructural.

No obstante sus inconvenientes, el interés de la madera como material estructural parece claro. Sorprende que en México, cuyas reservas forestales son apreciables, su uso esté restringido a la construcción (de viviendas rudimentarias,) de cimbras y obras falsas para estructuras de concreto, y, ocasionalmente, de algún techo para instalaciones industriales o centros de reunión, y a la elaboración de durmientes de ferrocarril para la trasmisión y distribución de energía eléctrica.

En este curso se explorarán las causas del escaso aprovechamiento de la madera, se señalarán aplicaciones en que la madera puede ser de interés y se harán sugerencias sobre formas en que el uso de la madera puede fomentarse.

2. ALGUNOS ANTECEDENTES HISTORICOS

La madera fue el primer material utilizado por el hombre con capacidad para resistir tensión y compresión, y, por lo tanto, flexión. Se mencionan algunos ejemplos de usos de la madera en épocas pasadas.

2.1 Viviendas paleolíticas de Rusia. Los indicios más antiguos del uso de la madera. Unos 20 siglos de antigüedad.

2.2 Palafitos y viviendas comunales del neolítico

2.3 Arsenal de Pireo

2.4 Puentes galos y chinos

2.5 Puente de Trajano.- Diseñado por Apolodoro. 99 d. de J.C. 20 pilas y un claro total de aproximadamente un kilómetro.

2.6 Antigua catedral de San Pedro en Roma. 326 d. de J.C.

2.7 Techos de madera en Inglaterra.- Viviendas y edificios religiosos. Abadía de Westminster (S. XIII). Diseñados empíricamente.

2.8 Techos de madera en China.

2.9 Techos de armaduras durante el Renacimiento.- Palladio, S. XVI .

2.10 Obra falsa para levantar el Obelisco de la Plaza de San Pedro.- Se cambio en S XVI desde su antiguo lugar en el Cerco Máximo.

Peso: 327 ton. Proyectó Doménico Fontana.

2.11 Puentes cubiertos suizos y americanos.- Puente de Schaffhausen en Suiza. 120 metros. Construido inicialmente con un apoyo al centro. S.XVIII. Destruido por los franceses en 1799.

Puentes semejantes fueron frecuentes en Nueva Inglaterra en el S. XIX.

El objeto de la cubierta era proteger las uniones.

3. EVOLUCION DE LAS FORMAS ESTRUCTURALES EN MADERA

3.1 El poste.- El árbol vivo sugiere el uso estructural de la madera como soporte vertical: el poste empotrado en el suelo. Puede usarse sin labrar.

3.2 La viga.- El árbol caído sugiere las posibilidades de la madera como viga.

Los primeros puentes y techos se hicieron con troncos sin labrar. Siguió el empleo de madera aserrada. Un paso importante fue el invento del machihembrado que permite la transmisión transversal de cargas.

3.3 La armadura.- Las limitaciones en escuadría de la madera restringen los claros en que pueden usarse elementos sujetos a flexión. La triangulación se desarrolló como una forma de salvar claros grandes con poco peso. En las armaduras triangulares el material se usa con gran eficiencia puesto que los miembros trabajan en compresión o tensión uniformes en toda su sección y longitud. Los primeros intentos de triangulación se hicieron con madera para techos de dos aguas. En los puentes de madera la triangulación ha sido siempre un elemento esencial. Aun hoy es uno de los recursos estructurales más comúnmente utilizadas.

3.4 Madera contrachapada (triplay)

El hecho de estar formada la madera contrachapada por capas alternadas con las fibras normales entre sí convierte a la madera en un material isótropo en el plano, obviándose así uno de sus inconvenientes. Aplicaciones interesantes de la madera contrachapada, son en combinación con madera ordinaria para formar vigas, en sistemas estructurales basados en el concepto de "stress-skin", y en techos de placas plegadas.

3.5 Madera laminada

Los elementos laminados están formados por tablas encoladas, con sus tablas en la misma dirección. La madera laminada encolada ha aumentado las posibilidades mecánicas de la madera al independizar las dimensionales totales de la viga de las impuestas por el tamaño de los troncos de los árboles.

Además los elementos de madera laminada puede construirse con curvas lo que incrementa la libertad en formas estructurales.

3.6 El arco

La técnica de la madera laminada encolada ha hecho posible la construcción de arcos de cerca de 100 m. Un ejemplo de arco interesante es el cimbra del puente de Plougastel, hecha con maderas delgadas clavadas y con ensambles a compresión a base de mortero (Freyssinet).

3.7 Estructuras a base de lamelas

La estructura de lamelas permite hacer techos de claros grandes utilizando elementos de pequeñas dimensiones.

3.8 Materiales diversos derivados de la madera

Existe una multitud de productos derivados de la madera que tienen aplicaciones estructurales. Su interés es doble: permiten aprovechar los desperdicios de madera y pueden diseñarse para que reunan propiedades específicas requeridas para determinada función estructural.

3.9 Paneles sandwich

Están formados por capas exteriores de un material resistente (triplay, láminas de derivados de la madera) y capas internas ligeras (poliestireno, poliuretano, papel con estructura de panal, etc.).

4. CAMPOS DE APLICACION DE MADERA

4.1 Vivienda

4.2 La madera como auxiliar en la construcción.- Obras falsas, cimbras, andamios.

4.3 Postes, pilotes y durmientes

4.4 Construcciones provisionales

Puentes de caminos de bajo costo, puentes provisionales. Edificios provisionales.

4.5 Construcción ligera. Aulas, talleres.

4.6 Obras portuarias

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PROPIEDADES DE LA MADERA

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Podemos definir a la madera como un conjunto de células huecas y alargadas cementadas entre sí. En el árbol vivo las funciones principales de la madera son las de sostén y conducción de soluciones. Las paredes celulares de grosor variable según la especie de madera están estructuradas a base de tres componentes principales, celulosa que se puede decir que es el armazón, las hemicelulosas que funciona como matriz y la lignina que es el cementante de todos los componentes.

La madera es un material anisotrópico, o sea que todas sus propiedades varían de acuerdo con sus tres ejes estructurales, los cuales forman ángulos rectos entre sí.

El eje longitudinal o axial (L) puede definirse como aquél que corre paralelamente a lo largo del tronco o de las fibras, el radial (R) es perpendicular al longitudinal, paralelo a los rayos (una línea recta de la médula o centro del árbol a la superficie del tronco), y el tangencial (T) perpendicular al axial y al radial y tangente a los anillos de crecimiento o circunferencia del tronco. En forma similar la madera tiene tres planos: el transversal (TR) delimitado por los ejes tangencial y radial; el radial (RL) comprendido entre los ejes radial y longitudinal, y el tangencial (TL), que se forma con la intersección de los ejes tangencial y longitudinal.

Existen dos grandes grupos de árboles de donde proviene la madera:

a) Las angiospermas, latifoliadas, hojosas o de hoja caduca. Ejemplos de este grupo son las maderas de caoba, encino, fresno, cueramo, etc.

b) Las gimnospermas o coníferas. La madera de oyamel, cedro blanco, sabino, pino, etc., son ejemplos de este grupo.

En México la madera de pino es la más bundante en el mercado y la más comúnmente usada en la construcción. En la actualidad para el mercado nacional, las calidades de la madera no se clasifican en base a sus posibles usos estructurales, sino únicamente desde el punto de vista del uso que se le pueda dar en la manufactura de muebles, canceles, alacenas, etc. El sistema de clasificación en uso, especialmente en la Ciudad de México, segrega las piezas en madera de primera, segunda, tercera y de construcción o cimbra, según la cantidad y severidad de defectos. Por lo tanto la llamada madera de construcción es la de más baja calidad, la que es prácticamente inservible para usos de carpintería comunes y corrientes. Más adelante se presentará una clasificación de la madera desde el punto de vista estructural que podría ser adoptada por usuarios, distribuidores y productores.

Propiedades físicas de la madera.

El peso total de la madera es la suma de los pesos de agua y de sustancia madera. El agua puede contribuir significativamente al peso total de la madera, llegando en algunas especies a más del 200 porciento. Es por esto que los valores de densidad son de poco valor como índices de las características físico-mecánicas de la madera si no se establece el contenido de humedad al que se hizo la medición.

La madera de pino que comúnmente se usa en la construcción tiene densidades que van de 0.40 a 0.55 gr/cm^3. PA/VV.

El contenido de humedad es la relación que existe entre el peso del agua en la madera respecto al peso anhidro de la misma. El contenido de humedad de la madera recién aserrada puede tener valores

$$\text{Contenido de humedad \%} = \frac{\text{Peso del agua}}{\text{Peso de la madera anhidra}} \times 100$$

de 150 a 200 porciento o más. Lo mismo sucede con la que está sumergida en agua. La que generalmente se usa en la construcción tiene contenidos de humedad de 7 a 50 porciento aproximadamente.

La humedad en la madera puede estar localizada principalmente en dos sitios. En los huecos de las fibras como agua libre y dentro de las paredes de las fibras como agua fija. Al secarse la madera el agua libre en los huecos de las fibras es la primera en perderse, y más tarde la que se encuentra en las paredes de las fibras. El contenido de humedad de la madera correspondiente a la humedad que queda únicamente en las paredes celulares se le llama punto de saturación de la fibra (PSF), siendo el intervalo de valores para la madera de pino de 25 a 30 porciento. Las propiedades de la madera CAMBIAN NOTABLEMENTE a contenidos de humedad inferiores al PSF.

Una característica importante de la madera es su higroscopicidad. Es decir que tiene la capacidad de tomar o dejar escapar humedad hasta que se balancee con la de la atmósfera. Este punto de balance se le conoce como contenido de humedad en equilibrio. Es por esto que el contenido de humedad de la madera por abajo del PSF variará según la temperatura y humedad relativa del medio ambiente.

Al cambiar de contenido de humedad la madera por abajo del PSF, el agua fija dentro de las paredes de las fibras también varía. El aumento o disminución de agua causa que las paredes de las fibras aumenten en dimensión o se contraigan. Consecuentemente los cambios dimensionales de la madera ocurren cuando varía su contenido de humedad por abajo del PSF.

La expresión que se usa para valorizar las contracciones de la madera es la siguiente:

$$\text{Cambio dimensional } \% = \frac{\text{Dimensión A - Dimensión B}}{\text{Dimensión A}} \text{ X } 100$$

Dimensión A= Es aquella de mayor magnitud, la cual generalmente es la que la muestra tiene cuando su contenido de humedad es superior al PSF

Dimensión B= Es la menor, que por lo general es la que la muestra tiene cuando su contenido de humedad es inferior al PSF

Los cambios dimensionales en la dirección longitudinal (L) son insignificantes, ya que pueden tener valores hasta de 0.9%. En la dirección radial (R) son del orden de 2.4 a 11% y en la tangencial (T) de 3.5 a 15%. Para la madera de pino del país los valores aproximados son: L= 0.3%, T= 8%, R= 4%.

Como era de esperarse, siendo la madera anisotrópica, sus coeficientes de expansión térmica varían según los ejes principales de la misma. La expansión térmica perpendicular a las fibras es de 10 a 15 veces mayor que en la dirección longitudinal. En la gran mayoría de los casos los cambios dimensionales debidos a variaciones de temperatura no se toman en cuenta, por su poca magnitud en la madera o porque los cambios dimensionales causados por variaciones de humedad son mayores y encubren los térmicos. Por ejemplo una viga de madera de 0.5 x 0.5 x 10 m expuesta a un cambio de temperatura de 50°C sufrirá un cambio dimensional perpendicular a las fibras (R y T) de 0.16 cm y a lo largo (L) de 0.19 cm.

La conductividad térmica en la dirección axial (L) es aproximadamente 2.5 veces mayor que las direcciones transversales (R y T).

La madera es un excelente aislante a corriente eléctrica en el estado anhidro, mas al aumentar su contenido de humedad, su conductividad aumenta significativamente. La resistividad (^{1}/conductividad) de la madera anhidra varía entre 3×10^{17} y 3×10^{18} ohm-cm comparándose favorablemente con la de bakelita que es de 1×10^{12} ohm-cm. Madera con un contenido de humedad de 30% tiene una resistividad de 1×10^{6} ohm-cm. La resistividad de la madera en la dirección axial (L) es aproximadamente la mitad que en la perpendicular a las fibras (R y T).

Respecto al aislamiento de sonido, la madera por sí sola al igual que otros materiales, no constituye una barrera contra el sonido, pero cuando se combina con diversos elementos se puede obtener una unidad estructural con propiedades satisfactorias de aislamiento. El problema de absorción de sonido es diferente que el de aislamiento, ya que este último requiere de materiales pesados y densos, mientras que el primero necesita de materiales blandos y porosos.

Propiedades mecánicas de la madera

Como la madera tiene tres ejes principales (anistropia): Longitudinal o axial (L), radial (R) y tangencial (T), las propiedades mecánicas son diferentes en dirección de cada uno de estos ejes aun cuando en muchas ocasiones estas diferencias entre radial y tangencial son mínimas, por lo que se ha optado por hablar únicamente de las resistencias mecánicas en dirección paralela y en dirección perpendicular a las fibras.

La resistencia de la madera en tensión paralela a las fibras (T 11) es la más alta de todas las resistencias de este material. La madera tiene una deformación plástica mínima cuando la madera se sujeta a este tipo de esfuerzo. Comúnmente la resistencia en T 11 puede ser 40 veces mayor que perpendicularmente a ellas ($T\perp$). Por lo regular se entiende que los resultados de pruebas de flexión estática (esfuerzo al momento de la ruptura EMR) valorizan conservadoramente la madera en T 11. El rango de valores para madera con un contenido de humedad de 12 porciento es de 300 a 3000 kg/cm^2 según la densidad de la especie. La madera de pinos mexicanos tiene valores alrededor de 800 kg/cm^2 a un contenido de humedad de 12 porciento.

En el caso de compresión paralela a las fibras (C 11), la relación esfuerzo deformación en contraste a la de T 11, la porción plástica de la curva es mayor y tiene un límite de proporcionalidad bien definido. Aunque las resistencias son diferentes para T 11 y C 11, los módulos de elasticidad son iguales. La resistencia en C 11 es de 3 a 10 veces mayor que la dirección perpendicular ($C\perp$). El intervalo de valores de EMR cuando la madera tiene un contenido de humedad de 10 porciento se estima que es de 100 a 1600 kg/cm^2. Las especies de pino del país tienen valores cercanos a 150 kg/cm^2 a un contenido de humedad de 12%.

La resistencia en compresión perpendicular a las fibras está íntimamente relacionada a la dureza y resistencia al corte perpendicular a las fibras ($CR\perp$). Cuando la madera se comprime perpendicularmente a las fibras, la tendencia es la de compactar las fibras e incrementar la densidad conforme va aumentando la carga, razón por la cual se puede decir que el máximo esfuerzo es imposible determinar, por lo que se trabaja con el esfuerzo al límite de proporcionalidad (ELP). Entre las maderas mexicanas pueden encontrarse especies con valores de 22 a 225 kg/cm^2 a un contenido de humedad de 12%. La madera de pino del país a un contenido de humedad de 12 porciento tiene valores cercanos a 60 kg/cm^2.

Cuando se impone a la madera un esfuerzo cortante en dirección perpendicular a las fibras (CR$\perp$), su resistencia es muy grande y nunca sucede ruptura en el plano transversal (TR) ya ésta únicamente se presenta en C$\perp$o corte paralelo a las fibras (CR 11). Cuando el esfuerzo cortante es paralelo a las fibras y el plano donde este se aplica es el radial (RL), en ocasiones la resistencia se ve afectada muy seriamente por la presencia de pequeñas rajaduras o grietas resultado del secado de la madera. Entre las maderas mexicanas pueden encontrarse especies con valores de 22 a 225 kg/cm^2 a un contenido de humedad cercano al 12 porciento, para la madera de pino al mismo contenido de humedad los valores están cercanos a 40 kg/cm^2.

Los valores de dureza son indicadores de la resistencia de la madera a indentarse y a la abrasión. Para determinar la dureza en la madera se utiliza el método Janka. La dureza en las superficies radial (RL) o tangencial (TL) es prácticamente igual, sin embargo la que se presenta en la transversal (TR) es por lo general mayor que la lateral. Para especies mexicanas con contenido de humedad de 12 porciento, la dureza lateral (RL y TL) va de 109 a 1548 kg y la de los extremos (TR) de 152 a 1550 kg. La madera de pino mexicana tiene una dureza lateral de 360 kg y en las superficies transversales de 460 kg, a un contenido de humedad de 12 porciento.

Si se compara la madera de una conífera de EE.UU. (Douglas fir) y acero estructural con bajo carbón, puede verse que para pesos iguales de ambos materiales la madera es 16 veces más eficiente que el acero en flexión estática. La razón principal de esto estriba en el hecho de que conforme la densidad de un miembro decrece, el volumen, el área transversal y su momento de inercia aumentan. Generalmente en flexión estática la fractura total en la madera no es instantánea, sino que se desarrolla poco a poco. Las especies mexicanas tienen un intervalo de EMR de 300 a 2100 kg/cm^2 y de módulo de elasticidad (ME) 40 000 a 300 000 kg/cm^2, a contenidos de humedad de 12%. Se estima que la madera de pino nacional tiene valores de ME de 100 000 kg/cm^2, y de EMR de 850 kg/cm^2 a un contenido de humedad de 12%.

La madera es más resistente a una carga de impacto que a una aplicada estáticamente, siendo en flexión 50 a 60 porciento más resistente al impacto.

Existen varios factores de importancia que afectan las carácterísticas mecánicas de la madera. A continuación discutiremos brevemente los más importantes.

La madera proviene de árboles que son organismos y como todo ser viviente exhiben variabilidad natural. Así tenemos que las características mecánicas de la madera de muestras de un mismo árbol, y entre árboles de la misma especie exhiben diferencias. En base a estudios hechos en otros países, podemos dar como ejemplo, que el módulo de elasticidad en flexión estática puede tener un coeficiente de variación de 22% y el EMR de 16%, esto es para la madera de una especie.

La resistencia mecánica es proporcional a la densidad, de donde resulta que un aumento o disminución en la densidad de la madera tiene como consecuencia igual efecto en la resistencia.

El contenido de humedad al igual que la densidad, es un factor de suma importancia en la resistencia mecánica de la madera. Cuando su contenido de humedad es superior al PSF la resistencia mecánica es la misma para todos los contenidos de humedad hasta la saturación total. Por abajo del PSF la resistencia mecánica aumenta conforme la madera tenga menor humedad. Por ejemplo si una muestra se seca hasta un contenido de humedad de 8%, el cambio de humedad que sufrirá a partir de un PSF de 50% será de 22% y es posible que el EMR se incremente 88 porciento, 4 porciento por cada porciento de contenido de humedad que disminuye a partir del PSF.

La influencia de la temperatura sobre las propiedades mecánicas puede ser considerable, y la magnitud de este cambio depende de la combinación de tiempo y contenido de humedad cuando la madera se expone a temperaturas extremosas. La resistencia al impacto es la característica que más es afectada por las bajas y altas temperaturas. Cuando la madera tiene un alto contenido de humedad, o se calienta en atmósfera de gran humedad, la pérdida de resistencia es mayor que si la atmósfera y madera tuvieran humedad menor.

Otro factor que afecta la resistencia mecánica de la madera son los defectos naturales que son aquéllos que se forman cuando el árbol está en pie y los artificiales que se producen durante los procesos de corte, secado, preservación, etc.

Algunos de los defectos naturales más comunes son nudos, desviación de la fibra y bolsas de resina.

Los nudos son los defectos naturales más comunes e importantes, ya que se trata de porciones de ramas que quedan incluidas dentro de la madera al crecer el árbol en diámetro. Los nudos producen el efecto detrimental de la desviación de las fibras en su cercanía y recordemos que la madera es menos resistente en la dirección perpendicular a las fibras que en la paralela, además actúan como reductores de área de resistencia, ya que cuando el nudo es del tipo "flojo" se puede asumir que en ese lugar existe un orificio.

Algunos árboles crecen de tal manera que sus fibras están dispuestas en espiral a lo largo del tronco. Al aserrarse la madera ésta presenta grandes desviaciones en dirección de las fibras, lo que afecta grandemente la resistencia, ya que la madera es menos resistente en la dirección perpendicular a las fibras. La desviación de la fibra se expresa como la relación entre 1 cm de desviación de la fibra de la arista ó eje de la pieza y la distancia dentro de la cual ocurre esta des viación. Una desviación de 1/15 reduce el EMR en 11 porciento y el ME en 6 porciento, para flexión estática.

Las bolsas de resina son defectos de relativa poca importancia y su efecto sobre la resistencia depende de la abundancia, tamaño y localización de las cavidades en la pieza de madera.

Entre los defectos artificiales más comunes tenemos desviación de la fibra, grietas y alabeos.

La desviación de la fibra es uno de los defectos artificiales más comunes y se origina al aserrar mal el árbol o al volver a aserrar las tablas sin precaución de que las fibras corran paralelamente a los cantos y superficies de las piezas elaboradas. El efecto sobre la resistencia es a causa de las diferencias en resistencia de la madera entre la dirección longitudinal y las transversales.

Las grietas por lo regular aparecen durante el proceso de secado y su magnitud y frecuencia depende primordialmente de la especie, tamaño de la pieza y precauciones durante el secado. Su influencia negativa sobre la resistencia de la madera es poco importante en T 11 y en C 11 y C⊥. Sin embargo reduce notablemente la resistencia en T⊥ y CR 11. En el caso de flexión estática el efecto sobre la resistencia depende mucho sobre la localización de las grietas, ya que entre más cerca estén del plano neutral donde el corte es máximo, su efecto es mayor; en las superficies donde los esfuerzos de tensión y compresión son máximos, los efectos de las grietas son poco importantes.

Los alabeos son defectos que se originan durante el secado y se consideran defectos, ya que al eliminar las distorsiones es necesario remover material. En ocasiones la severidad del defecto es tal, que hace la pieza prácticamente inservible para la construcción y muchos otros usos.

A continuación se presentarán una serie de tablas y diagramas mostrando esfuerzos permisibles según calidad para la madera de pino del país. Estos esfuerzos los definimos como aquéllos que pueden ser sostenidos permanentemente con seguridad por un componente estructural de cierta calidad.

La derivación de estos esfuerzos se basó en resultados de pruebas de laboratorio realizadas con madera de pino libre de defectos y con pequeñas probetas. La madera provino de árboles de Durango, Chihuahua, Michoacán y Veracruz (14 especies). Están basados en una muestra reducida, por lo tanto los esfuerzos que se proponen se pueden considerar como conservadores. Los esfuerzos ya fueron modificados para tomar en cuenta la variabilidad natural de la madera, la permanencia de la carga y se les aplica un factor de seguridad.

Las calidades propuestas dependen en el número de anillos de crecimiento por cm, profundidad de fisuras o grietas, desviación de la fibra, dimensiones de bolsas de resina y localización, dimensión y frecuencia de nudos en la madera.

Esfuerzos según calidad para la madera de pino

"VERDE" a contenidos de humedad superiores al 18%

Clasificación	Flexión estática ó Tensión 11 kg/cm2	Compresión 11 kg/cm2	Compresión 1 kg/cm2	Corte 11 kg/cm2	Módulo de elasticidad promedio kg/cm2	mínimo kg/cm2
Calidad 75 F-75	67.5	41.25	11.25	10.5	45000	22500
Calidad 65 F-65	58.5	35.75	9.75	9.10	39000	19500
Calidad 50 F-50	45	27.5	7.5	7	30000	15000
Básico F-100	90	55	15	14	60000	30000

"SECO" a contenidos de humedad inferiores al 18%

Clasificación	Flexión estática ó Tensión 11 kg/cm2	Compresión 11 kg/cm2	Compresión 1 kg/cm2	Corte 11 kg/cm2	Módulo de elasticidad promedio kg/cm2	mínimo kg/cm2
Calidad 75 F-75	78.75	52.5	16.5	11.25	54000	24750
Calidad 65 F-65	68.25	45.5	14.3	9.75	46800	21450
Calidad 50 F-50	57.5	35	11	7.5	36000	16500
Básico F-100	105	70	22	15	72000	33000

Esfuerzo según calidad= Es aquél esfuerzo que puede ser sostenido permanentemente con seguridad por un componente estructural de cierta calidad.

Dimensiones máximas o mínimas de defectos, excepto nudos.

TIPO DE DEFECTO	CALIDAD 75 F-75	CALIDAD 65 F-65	CALIDAD 50 F-50
Velocidad de crecimiento (mínimo)	16 anillos /5 cm	12 anillos /5 cm	8 anillos /5 cm
Fisuras o grietas (profundidad máxima)	1/4 del grosor	1/3 del grosor	1/2 del grosor
Desviación de la fibra (no mayor de	1 en 14	1 en 11	1 en 8
Gema (no mayor de	1/8 de cualquier sup.	1/8 de cualquier sup.	1/4 de cualquier sup.
Bolsas de resina (menos de 3 mm ancho y profundidad máx. de	1/4 del grosor	1/3 del grosor	1/2 del grosor

Dimensiones máximas de nudos permisibles

Ancho nominal de la superficie		CALIDAD 75 F-75			CALIDAD 65 F-65			CALID F-	
		Nudos en el canto o en el borde de la viga	Nudos en la zona central de la viga o en cualquier superficie de un miembro en compresión	Nudos en las aristas de las vigas o en cualquier superficie de un miembro en tensión	Nudos en el canto o en el borde de la viga	Nudos en la zona central de la viga o en cualquier superficie de un miembro en tensión	Nudos en las aristas de las vigas o en cualquier superficie de un miembro en tensión	Nudos en el canto o en el borde de la viga	Nudos zona de la en cu super un mi comp
		mm	mm	mm	mm	mm	mm	mm	
1	25.4	6	6	--	10	10	3	13	
1-1/2	38.1	10	10	--	13	13	6	19	
2	50.8	13	13	3	19	19	10	25	
2-1/2	63.5	16	16	6	22	22	13	32	
3	76.2	19	19	10	29	25	16	38	
4	102	25	25	13	38	35	19	51	
5	127	32	32	16	48	44	25	64	
6	152	38	38	19	57	51	29	76	
7	178	41	44	22	60	60	32	83	
8	203	44	51	29	67	67	38	89	
9	229	48	54	32	70	73	44	92	
10	254	51	60	35	76	79	51	98	
11	279	51	64	38	76	86	54	102	
12	305	54	70	41	79	92	60	108	

Dos o más nudos de dimensión máxima no se permiten en una longitud de 305 mm. Para miembros sujetos a flexión y simplemente las dimensiones de los nudos permisibles se pueden aumentar cuando se localizan en los tercios de los extremos. Estos se pue proporcionalmente a los extremos en dimensiones 25 porciento mayores a las que aparecen en la Tabla.

PROTECCION DE LA MADERA

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La madera como todos los materiales de construcción, es susceptible a deteriorarse con el tiempo; sin embargo, siendo ésta de origen orgánico, son los organismos los principales causantes de su deterioro, aunque en ciertos casos agentes físicos como fuego é intemperismo, pueden ser los principales destructores de la madera.

Intemperismo.- La madera expuesta a la lluvia, sol, viento, polvo, etc., con el tiempo su color se transforma en grisáceo, a causa de que las capas superficiales se deterioran por las hinchazones y encogimientos que experimenta con los cambios de humedad, además de que los polisacáridos de la madera han venido sufriendo hidrólisis, proceso que se acelera por la energía de las radiaciones infrarojas y ultravioletas del sol. Este tipo de deterioro es relativamente sencillo de evitar cubriendo la madera periódicamente con capas de pintura o barniz, las cuales actúan como barreras a los rayos del sol y retardan la penetración de humedad y por lo tanto reducen los cambios dimensionales.

Fuego.- Como los principales componentes químicos de la madera, lignina, hemicelulosas y celulosas, son combustibles, la madera es también combustible. Pero no porque tenga esta propiedad quiero decir que si se usa en una estructura, ésta no puede ser resistente al fuego en caso de incendio.

Para el caso de la madera y su resistencia al fuego, es muy importante la relación entre forma y dimensión de la pieza. Una astilla prende fácilmente, el fuego se propaga con rapidez y se consume en segundos o minutos. En el caso de una pieza grande con mucho volumen en comparación a su área (una columna de 10x10x400 cm) esta se prende con mayor dificultad que una astilla, la propagación de la flama se reduce considerablemente y se consume muy lentamente. Una pieza de estas características forma una capa de carbón en la superficie que actúa como aislante, además a causa de la baja conductividad térmica de la madera, el interior se conserva a temperaturas bajas, aunque en el exterior se esté quemando, por lo que la pieza conserva por mucho tiempo gran parte de su resistencia mecánica.

Hasta la fecha, no se ha encontrado ningún tratamiento que convierta a la madera en un material incombustible. Pero tampoco existe estructura alguna que sea cien por ciento a prueba de incendios, no importa si está hecha con materiales incombustibles. Sin embargo a la madera se le puede aplicar ciertos tratamientos para retardar su ignición y la propagación de flamas.

Por lo tanto, a una estructura de madera se le puede dar resistencia al fuego mediante tratamiento de la madera de compuestos químicos hidrosolubles retardantes de fuego, con recubrimientos a base de vermiculita, asbesto, yeso, etc., mediante los cuales se pueden formar barreras temporales que protejen la madera del fuego y utilizan piezas de madera de dimensiones y formas que den un gran volumen con un mínimo de superficie.

Hongos.- Son plantas que a diferencia de las plantas verdes o con clorofila, no pueden manufacturar sus alimentos, por lo tanto son parásitos y se alimentan de materia orgánica. Los hongos producen exoenzimas que descomponen la celulosa, hemicelulosas y lignina de la madera, en compuestos menos complejos que pueden utilizar en su nutrición.

La gran mayoría de los hongos que manchan o deterioran la madera necesitan de ciertas condiciones para su crecimiento, si ALGUNA DE ELLAS SE MODIFICA NEGATIVAMENTE EL HONGO NO PUEDE DESARROLLARSE.

1. Alimento.- El alimento en la madera siempre está presente, ya que las mismas paredes celulares y contenidos de las células son el alimento.

2. Humedad.- Los hongos necesitan de cierta humedad, y cuando la madera tiene contenidos de humedad menores al 15 porciento, el hongo no se desarrolla.

3. Oxígeno.- Estas plantas necesitan de un mínimo de aire (oxígeno) dentro de la madera, el cual se estima que es igual al 20 porciento del volumen de la pieza de madera. Es por esto que piezas saturadas y sumergidas en agua no tienen espacios con aire, y por lo tanto no pueden ser deterioradas por hongos.

4. Temperatura.- El rango de temperatura óptima para el desarrollo de los hongos en la madera es de 23 a 33°C.

Existen tres tipos principales de deterioro por hongos. Un grupo de hongos que viven de las sustancias almacenadas en cierto tipo de células de la madera. Estos hongos manchan a la madera, mas no reducen su resistencia mecánica. Otro grupo causa pudriciones, o sea que manchan y destruyen la madera al alimentarse de los componentes de las paredes celulares. Este segundo grupo causante de las pudriciones blancas y pardas es el más importante por la gran cantidad de daños que causan. El tercer grupo de hongos es el causante de las llamadas pudriciones blandas, que es un tipo de pudrición muy especializado y únicamente ocurre cuando la madera está sujeta a temperaturas y humedades altas, como las que existen en las estructuras de torres de enfriamiento. Este grupo de hongos mancha y destruye la madera.

Para prever el daño por hongos se podría utilizar un o una combinación de métodos. En algunos casos es conveniente usar madera de especies que tengan gran durabilidad natural. El aumento en durabilidad natu ral de éstas especies se debe a que por razones poco conocidas el duramen o centro de los árboles queda impregnado con sustancias químicas que son eficaces preservadores. Otra forma de proteger la madera de hongos es el diseñar las estructuras de tal forma para que el contenido de humedad de la madera se mantenga a menos de 15 porciento. En algunos casos esto consiste en cubrir las tuberías de agua fría con algún aislante para que la humedad que se condensa sobre ellas no gotee a la madera y que el agua de lluvia no se acumule en recovecos, etc. Si la madera va a estar en contacto directo con el suelo o expuesta a la intemperie, su contenido de humedad probablemente sobrepase el 15 porciento, por lo que para protegerla se le puede someter a tratamientos superficiales o a presión con preservadores, según el riesgo a daño por hongos a que esté expuesta la madera. Si el riesgo es alto, la madera debe tratarse con métodos a presión; si el riesgo es menor entonces el método de inmersión podría ser el más conveniente. Cuando la aplicación se hace con brocha o con aspersión, la protección que se obtiene es mínima, por lo que únicamente se recomienda para casos de muy poco riesgo.

El tipo de solución preservadora que se vaya a emplear depende en muchas ocasiones del uso final de la pieza. Si no se va a pintar y la apariencia no es importante, la creosota o pentaclorofenol disueltos en aceites oscuros podrían usarse. En cambio, si las piezas de madera se van a pintar o la apariencia es determinante, entonces lo mejor sería utilizar soluciones de sustancias tóxicas en agua o en aceites ligeros o claros.

Insectos.- A estos organismos se les considera como segundos en importancia a los hongos por los daños que causan. Los insectos más importantes son las termitas o polilla y existen dos tipos principales, las termitas subterráneas que son las más destructoras y la polilla de la madera seca.

Las termitas subterráneas construyen su nido bajo el suelo o en pedazos de madera en contacto con este a fin de hacer túneles hasta los sitios donde encuentran alimento. Estos insectos de cuerpo blando, son muy susceptibles a los cambios de temperatura y de humedad, es por esto que construyen túneles para mantener dentro de ellos condiciones óptimas de medio ambiente. Consumen la parte interna de la pieza dejando un cascarón en el exterior que los proteje de la luz y cambios de temperatura y humedad. Cuando la madera no está en contacto directo con el suelo construyen túneles sobre el tabique o concreto hasta llegar a la madera y mediante ellos mantienen la comunicación con el nido y las condiciones ambientales deseadas.

Las polillas de la madera seca no necesitan conexión alguna con el suelo y resisten bien los cambios de temperatura y humedad. No son tan dañinas ni numerosas como las subterráneas y su presencia se nota cuando los miembros aladas (palomillas de San Juan) emergen de la madera a través de pequeños orificios para ir a otros sitios a depositar huevecillos o empezar una nueva colonia.

El control más efectivo de las termitas subterráneas es la prevención. Cuando sea posible las construcciones de madera no deben estar en contacto con el suelo a menos que la madera esté apropiadamente impregnada con algún preservador. Se deben diseñar los cimientos con obstáculos contra termitas, inspeccionar periódicamente y destruir túneles que se localicen sobre cimientos de concreto o mamposteria, remover toda la madera enterrada cercana a la construcción, y en áreas de alta incidencia de daño envenenar el suelo alrededor de la estructura o utilizar alguna especie con gran resistencia natural al ataque por termitas.

Para evitar el daño por polilla de la madera seca, se pueden utilizar especies resistentes a ella o se le puede aplicar a la madera un tratamiento superficial efectivo con algún preservador.

Barrenadores marinos.- Estos organismos desde la antiguedad son famosos por los daños que causan a embarcaciones de madera é instalaciones marinas, sobre todo aquéllas que se encuentran en mares tropicales o en zonas costeras salobres.

Los dos tipos principales de barrenadores son moluscos y crustáceos y en ambos el deterioro consiste en que cavan túneles ya sea para alimentarse de la madera o para usarlos como morada y digerir el plankton marino y los hongos que crecen en las paredes de los túneles, los cuales pueden variar en longitud y en diámetro, según el organismo y condiciones ambientales en las que se desarrolla.

Los métodos más efectivos de protección en contra de los taladradores marinos consisten en impregnar la madera con sales hidrosolubles de cobre-cromo-arsénico o con una mezcla de creosota y alquitrán de hulla. Existen también especies cuya madera tiene gran resistencia natural al ataque de barrenadores marinos.

La preservación de la madera en México.- Las soluciones de preservadores más conocidas y usadas en México son a base de creosota, pentaclorofenol y sales de cobre-cromo y arsénico tipo C.

La creosota es un producto de la destilación de carbón bituminoso, consistente en una mezcla de unos 40 importantes compuestos tóxicos a hongos e insectos. Su aplicación por lo general es con métodos a base de presión. Una desventaja para ciertos usos es que lo sucia que deja a la madera la imposibilita para pintarla, además del mal olor que despide.

El pentaclorofenol es un compuesto de cloro y fenol en forma de polvo grisáceo. Es soluble en aceites y generalmente se aplica en concentraciones del 5 porciento. Se pueden utilizar aceites ligeros claros con lo que se obtienen buenas apariencias de la madera tratada, además puede pintarse. Su aplicación puede ser por imersión, aspersión o a base de métodos a presión.

Las sales hidrosolubles de cobre cromato y arsénico, comúnmente llamadas sales CCA, vienen en dos tipos A y B; ambos contienen básicamente los mismos elementos tóxicos a los organismos destructores de la madera pero en diferentes proporciones, razón por la cual la madera tratada con el tipo A necesita una mayor cantidad de sales por unidad de volumen. Los dos tipos son efectivos. La madera tratada con estas sales hidrosolubles queda limpia y se le puede aplicar toda clase de acabados. Por lo general la madera se impregna con métodos a base de presión. Una desventaja es que es necesario volver a secar la madera después de tratada. Las sales CCA tipo C tienen características similares.

Los retardantes de fuego más efectivos son soluciones solas o en combinación de fosfato mono y dibásico de amonia, sulfato de amonia, bórax, ácido bórico y cloruro de cinc. El más eficaz es el fosfato de amonia, ya que no únicamente reduce la inflamabilidad de la madera sino que previene la formación de brasa. El mejor método de aplicación es el de presión.

En la actualidad en México existen firmas distribuidoras de los preservadores y retardantes de fuego mencionados. También existen plantas de impregnación a presión en Mazatlán, Sin., Guadalajara, Jal., Durango, Dgo., Parral, Chih., Chihuahua, Chih., Las Vigas, Ver., y en la Ciudad de México.

Resumiendo, se puede decir que aunque la madera es un material orgánico combustible y susceptible a ser deteriorado por organismos, ésta puede tener gran durabilidad y permanencia en estructuras de madera.

Para evitar el deterioro causado por hongos se recomienda que, cuando sea posible, la madera se mantenga constantemente a un contenido de humedad inferior al 15 porciento. Cuando esto sea imposible y dependiendo del riesgo a que está sujeta, debe tratarse con un preservador adecuado, o escoger una especie con gran durabilidad natural.

Respecto a los insectos, especialmente las termitas subterráneas, se recomienda prever el daño, ya que el control una vez que han atacado la madera es mucho más difícil. Una recomendación especial es que toda estructura de madera se inspeccione cuidadosamente dos o tres veces al año, con objeto de detectar algún inicio de daño causado por agentes biológicos, ya que es muy fácil controlar el daño en sus etapas iniciales.

La madera es muy resistente al fuego cuando las piezas son de grandes dimensiones. Para madera de piezas pequeñas estas pueden impregnarse con algún retardante de fuego. Además lo más importante en los incendios son los contenidos de las estructuras y no el de los materiales de que está hecha la estructura.

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USOS ESTRUCTURALES DE LA MADERA

GRITERIOS DE DISEÑO PARA ESTRUCTURAS DE MADERA

ING. ROBERTO MELI.

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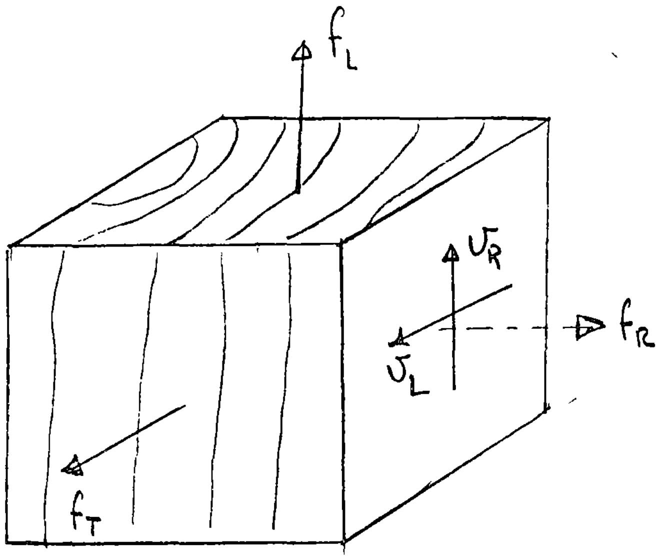
CRITERIOS DE DISEÑO PARA ESTRUCTURAS DE MADERA

1. COMPORTAMIENTO MECANICO

En una sesión anterior se han tratado las propiedades mecánicas de la madera. Para entender el comportamiento estructural de la madera y plantear los criterios de diseño es importante recalcar lo siguiente:

a) La madera es un material anisotrópico (tiene distintas propiedades en distintas direcciones). Puede considerarse que existen dos direcciones ortogonales con esfuerzos distintos: paralelo al grano y perpendicular al grano).

b) Por lo anterior los esfuerzos que hay que considerar en el diseño son los siguientes:



f_L: esfuerzo longitudinal de tensión o compresión paralela al grano

f_T: esfuerzo transversal perpendicular al grano. (f_R: esfuerzo radial, se toma igual al transversal, aunque sea en general menor)

τ_L: cortante longitudinal (deslizamiento a lo largo del grano)

τ_R: cortante transversal (la resistencia es mayor que en el caso anterior.

Para el triplay hay que considerar dos tipos de esfuerzo cortante

τ_T en el plano

τ_R en el espesor

2. ESFUERZOS DE DISEÑO

El diseño se basa en esfuerzos permisibles que son valores reducidos con respecto a las que se obtienen en los ensayes estándar; estos a su vez se corrigen por las distintas condiciones de trabajo que pueden presentarse.

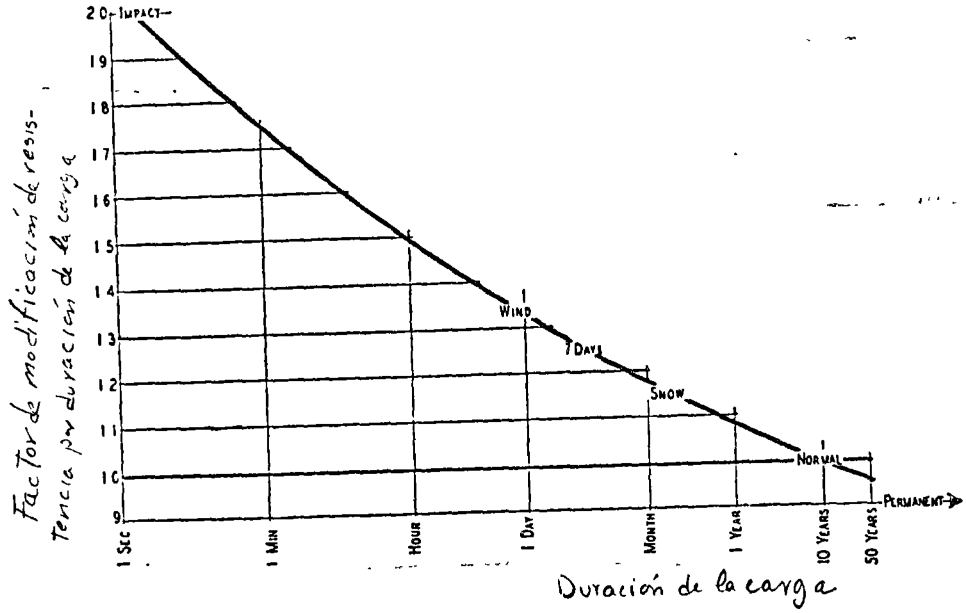
Los esfuerzos permisibles están reducidos con respecto a los de ensayes estándar tomando en cuenta:

a) La variabilidad propia del material. Los valores medios obtenidos en los ensayes deben reducirse considerando que las propiedades varían de un árbol a otro. Esta reducción depende de la variabilidad de la propiedad en cuestión para la especie de que se trate. Para la resistencia en compresión y cortante este factor es del orden de 0.75.

b) Los defectos de las piezas. Las piezas de madera tienen defectos que reducen su resistencia: nudos y rajaduras que no existen en los especímenes sobre los que se realizan las pruebas estándar. La importancia de estos defectos se juzga mediante una clasificación. En los países en que el empleo estructural de la madera está muy difundido, los elementos de madera pasan a través de un proceso de clasificación en el cual, con base en el número, posición y dirección de nudos y rajaduras y en la densidad de los anillos de crecimiento y en la dirección del grano, se estima la reducción que estos defectos implican en la resistencia de la pieza con respecto a un elemento sin defectos. De esta forma cada elemento de madera sale con un sello que especifica sus esfuerzos básicos. El factor reductivo por este efecto varia entre 0.40 y 0.75 según el grado.

En México no existe un sistema de clasificación que se base específicamente en las propiedades estructurales. La clasificación en madera de primera, segunda y tercera atiende esencialmente al aspecto de las piezas.

c) La duración de la carga. En los ensayes estándar la carga se aplica rápidamente, mientras que los esfuerzos permisibles se refieren a cargas sostenidas (del orden de 10 años). El efecto de la duración de la carga se aprecia en la figura siguiente, de la que se deduce que la reducción correspondiente a 10 años con respecto a una carga rápida es del orden del 50 por ciento.



d) El factor de seguridad. Tomando en cuenta las distintas incertidumbres que existen en el diseño (magnitud de cargas, métodos de análisis, procedimientos constructivos) es necesario tomar en cuenta un factor de seguridad que depende del tipo de esfuerzos y que varía entre 1.5 y 2.

Como resultado de tomar en cuenta todos los factores anteriores las propiedades en especímenes estándar se ven reducidas por factores que varían entre 5 y 10, según el grado, para los esfuerzos de tensión y compresión. La tabla siguiente muestra algunos valores comparativos tomados del Reglamento Británico.

COMPARACION ENTRE ESFUERZOS EN ESPECIMENES ESTANDAR Y ESFUERZOS BASICOS ESPECIFICADOS

Especie	Compresión ‖ grano		Cortante		Módulo Elasticidad	
	f_e	f_b	v_e	v_b	E_e	E_b
Abeto Douglas	540	54 - 106	103	7.8 - 13.8	140,000	120,000
Western Hemlock	450	42 - 85	67	6.4 - 11.7	120,000	100,000
Redwood	450	39 - 81	93	6.4 - 11.7	100,000	850,000

El subíndice e indica esfuerzos medios en ensayes estándar

El subíndice b indica esfuerzos de diseño: se indican los límites para los distintos grados considerados.

Esfuerzo en kg/cm^2

Datos del Reglamento Británico CP-112

En México, el Reglamento del D.F., especifica esfuerzos permisibles para los distintos tipos de madera a través de correlaciones entre dichos esfuerzos permisibles y la densidad de la madera. Las correlaciones no son malas pero lo que falta es una clasificación confiable.

La tabla siguiente da los esfuerzos propuestos por el Reglamento del D.F.; estos esfuerzos se refieren a maderas de primera y deberán modificarse multiplicándolos por los factores especificados en la tabla para otros tipos de madera.

ESFUERZOS PERMISIBLES SEGUN EL REGLAMENTO PARA EL D.F.

Concepto	Valor en kg/cm^2 Para cualquier γ	Para $\gamma = 0.4$
Esfuerzo en flexión o tensión simple	$196\ \gamma^{1.25}$	62
Módulo de elasticidad en flexión o tensión simple	$196{,}000\ \gamma$	79,000
Esfuerzo en compresión paralela a la fibra	$143.5\ \gamma$	57
Esfuerzo en compresión perpendicular a la fibra	$54.2\ \gamma^{2.25}$	7
Módulo de elasticidad en compresión	$238{,}000\ \gamma$	95,000
Esfuerzo cortante	$35\ \gamma^{1.25}$	10

Para maderas selectas, se pueden incrementar en un 30% los valores anteriores. Para maderas de segunda, se tomará el 70% de los valores consignados en la tabla. Para maderas de tercera, se tomará el 50%.

Esfuerzos permisibles para distintas maderas mexicanas se dan explicitamente en las Especificaciones para estructuras de madera de la Secretaría de Obras Públicas dichos esfuerzos se presentan en la tabla siguiente:

ESFUERZOS UNITARIOS PERMISIBLES EN Kg/cm²

ESPECIE	Calidad	Paralelamente a la fibra				Comp. normal a la fibra	Módulo de elasticidad
		Flex.	Tens.	Comp.	Cort.		
Pino blanco (*P. arizonica*)	1a.	80	65	60	6.0	18.0	85 000
Pino lacio (*P. michoacana*)	2a.	60	55	50	6.0	18 0	
Pino ayacahuite							
Pino prieto (*P. chihuahuana*)							
(*P. dou lasiana*)							
Pino real (*P. engelmanni*)							
(*P. herrerai*)	1a.	90	75	70	8 0	20.0	90 000
(*P. ortiguillo*) (*P. lawsoni*)	2a.	70	65	60	8.0	20	90 000
Chalmaite blanco (*P. montezumae*)							
(*P. pseudostrobus*)							
(*P. strobus chiapenis*)							
Cedro rojo o blanco							
Pino chino	1a.	100	85	80	9	20 0	100 000
Ocote chino (*P. leiophilla*)							
(*P. lumholtzii*)	2a.	75	70	65	9	20.0	100 000
Pino blanco (*P. durangesis*)	1a.	110	90	85	9	25 0	100 000
Pinabete	2a.	85	75	70	9	25 0	100 000
Bari (*Cordia gerascanthus*)	1a. 2a.	130.0 100.0	110 95	100 80	10 10	30 0 30.0	110 000
Cocoíte (*Gliricida sepium*)							
Dzalán (*Lisyloma babamensis*)	1a.	120.0	100	95	10	25.5	100 000
Guayacán (*Guaiacum oficianale*)	2a.	90	85	75	10	25.5	
Jobo (*Spondias lutea*)							
Encino	1a. 2a.	120 0 90.0	100 85	95 75	10 10	25 0 25 5	100 000
Huapaque (*Ostrya guatemalensis*)	1a. 2a.	150 0 110 0	125 0 105 0	115 95	12 12	30 0 30 0	110 000
Zapotillo (*Erythroxylon ellipticum*)	1a. 2a.	135 0 100 0	110 95	100 80	10 10	25 0 25 0	110 000

En cuanto a la modificación de los esfuerzos básicos por diversas condiciones de trabajo, estos son los principales factores que hay que considerar según el Reglamento del D.F.

a) Las condiciones de humedad. Tratándose de maderas saturadas o sumergidas, el esfuerzo de compresión paralelo a la fibra debe reducirse 10%; el de compresión perpendicular a la fibra, 33%; y los módulos de elasticidad, 10%. En realidad la reducción de la resistencia con el contenido de humedad es continua y esta especificación es una simplificación gruesa. La mayoría de los Códigos distinguen la madera protegida de la humedad de la expuesta a la intemperie.

b) La duración de las cargas. Cuando la duración de las cargas no exceda el lapso indicado a continuación, se incrementarán los esfuerzos permisibles según la siguiente tabla:

15% para dos meses de duración:

25% para 7 días de duración:

50% para viento o sismo:

100% para impacto

Los incrementos anteriores no se aplican a los módulos de elasticidad en el cálculo de deflexiones.

c) La posibilidad de redistribución de la carga. Cuando 4 o más miembros puede considerarse que están trabajando en conjunto para soportar una misma carga, los esfuerzos permisibles pueden incrementarse en un 10% (Reblamento Británico).

3. PROCEDIMIENTOS DE DISEÑO

Los procedimientos de diseño para madera se basan en las fórmulas conocidas de resistencia de materiales, por lo tanto solo se destacarán aquí algunos puntos particulares sobre los que hay que poner la atención.

3.1 Diseño de vigas (Miembros en flexión)

Hay que revisar:

a) Flexión: con la fórmula de escuadría. El esfuerzo admisible se reduce para miembros de mucho peralte (mayor que 30 cm) debido al efecto del gradiente de esfuerzos. El factor de reducción por peralte es el siguiente:

$$F = 0.81 \frac{(h^2 + 922)}{(h^2 + 588)}$$

b) Cortante horizontal (longitudinal), hay que comparar el esfuerzo cortante máximo (igual a 1.5 veces el promedio) con el permisible

c) Aplastamiento en los apoyos: Los esfuerzos permisibles en compresión se aumentan dependiendo del tamaño del apoyo.

d) Deflexión: con las fórmulas usuales. La deflexión permisible se fija generalmente en 1/360 veces el claro. Para el efecto de cargas sostenidas el módulo de elasticidad debe reducirse hasta el 50% según algunos reglamentos.

En elementos de piso la deflexión es el efecto que rige el diseño en un gran número de casos, especialmente porque se relaciona en forma directa con las vibraciones que se presentan en los sistemas de piso. En elementos de madera laminada es usual especificar contraflechas que reducen la importancia de este factor.

e) Pandeo lateral: debe proveerse contraventeo en vigas peraltadas; hay problemas si la relación peralte a ancho excede de 3

3.2 Diseño de columnas (Miembros en flexocompresión)

El diseño está regido en general por pandeo excepto para columnas muy cortas. Se emplea la fórmula de Euler que, considerando un factor de seguridad de 2.75, da lugar a que el esfuerzo permisible sea

$$F_a = \frac{0.3\,E}{(K\ L/d)^2}$$

Las columnas pueden ser simples (de un solo elemento) o espaciadas (de varias piezas unidas por medio de bloques de empaques). Para estas últimas se aplican las mismas fórmulas pero con factores notablemente mayores.

Para efecto combinado de carga axial y flexión se aplica la conocida fórmula de interacción

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

3.3 Conexiones

Las estructuras de madera están armadas a base de piezas relativamente pequeñas. Uno de sus puntos críticos son las uniones entre las distintas piezas; estas deben diseñarse de manera que sean capaces de trasmitir los esfuerzos de uno a otro elemento.

Por la importancia del diseño de las conexiones en estructuras de madera, este se tratará específicamente en otra sesión de este curso.

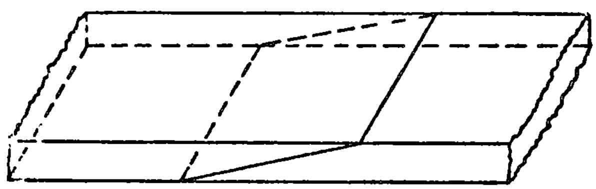
4. MADERA LAMINADA

La madera laminada está formada por piezas delgadas (tablones o duelas) pegadas para formar vigas o muy frecuentemente elementos curvos. Es importante notar que en todos los lechos la dirección de las fibras es la misma, a diferencia de lo que ocurre para el triplay.

Su diseño difiere poco del de estructuras de madera sólida; hay que tomar en cuenta los hechos siguientes:

a) La clasificación es distinta que para madera. Como se trata de piezas que trabajan esencialmente en flexión es importante que la madera mejor esté colocada en los extremos y las reglas de clasificación se basan en los defectos de las partes extremas esencialmente.

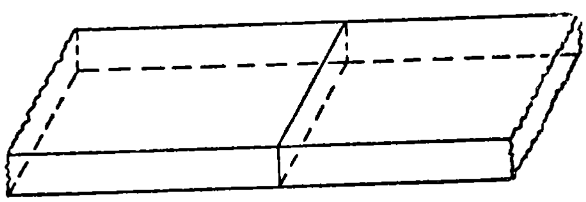
b) La unión longitudinal de las piezas reduce la resistencia. Es conveniente el empleo de juntas inclinadas como la de la figura; mientras más in clinadas sean las juntas menor es la reducción de esfuerzos que se pide por ese efecto. Cuando las juntas longitudinales son verticales, la reducción es mayor y depende del número de juntas por unidad de longitud (ver figura).



JUNTA INCLINADA

Factor de reducción de resistencia por juntas inclinadas

Inclinación de la junta	Reducción
1 a 12 o menos	0.85
1 a 10	0.80
1 a 8	0.75
1 a 5	0.60



JUNTA VERTICAL
(poco recomendable)

Factor de reducción de resistencia por juntas verticales

Espaciamiento de las juntas	Factor de Reducción
30 t	0.90
20 t	0.80
10 t	0.60

t es el espesor de la laminación.

TIPOS DE JUNTA LONGITUDINAL EN MADERA LAMINADA

c) Para piezas con curvatura importante, se introducen esfuerzos que reducen la resistencia de las piezas. El factor de reducción que se propone es el siguiente:

$$\frac{f'}{f} = \left[1 - 2000\ (t/R)^2\right]$$

De particular importancia son las tensiones radiales (en la dirección del peralte) que suelen ser críticas por la baja resistencia de la madera para tensión normal al grano. Las fórmulas dadas por los reglamentos para considerar este efecto no son muy exactas (ver referencia 1).

5. MADERA CONTRACHAPADA (Triplay)

Se trata de paneles compuestos por cierto número (impar) de lechos, colocados alternando la dirección de las fibras.

Los elementos de triplay pueden emplearse para trabajar con cargas normales a su plano (en sistemas de piso) o más frecuentemente con cargas en su plano (como diafragmas).

Los esfuerzos permisibles dependen del grado de triplay (se clasifica en grados A, B, C y D generalmente).

Para su trabajo en compresión y tensión y para el cálculo de deflexiones, solo se consideran efectivos los lechos con las fibras en la dirección de los esfuerzos. Existen tablas que dan las propiedades geométricas equivalentes de las secciones en función del número de láminas.

Por lo anterior, al usarlo como placa (losa) hay que orientar las fibras de los lechos exteriores en la dirección del claro.

Para cortante el esfuerzo permisible perpendicular a los lechos es del orden del doble del que se admite en el plano de los lechos.

6. COMPORTAMIENTO SISMICO DE ESTRUCTURAS DE MADERA

La experiencia de daños observados por efecto de sismos intensos muestra que el comportamiento de las estructuras de madera se compara muy favorablemente con el de otros materiales especialmente la mampostería y el concreto. Las razones de este desempeño satisfactorio son principalmente las siguientes:

a) El tipo de estructuración da lugar a una muy alta capacidad de absorción de la energía producida por el sismo, a través de movimiento de las juntas.

b) La resistencia de la madera ante cargas aplicadas en forma rápida es muy considerable.

Los puntos que hay que cuidar en el diseño de estructuras de madera en zonas sísmicas son:

a) En anclaje adecuado de las estructuras en su base y el diseño de las conexiones.

b) La rigidización lateral; mucho de los daños observados en estructuras de madera durante temblores se deben a que las deformaciones excesivas sufridas por las construcciones producen daños en elementos no estructurales aunque no afectan mayormente a la estructura.

c) Es importante que los pisos y techos funcionen como diafragmas rígidos para trasmitir las fuerzas sísmicas a los elementos resistentes en cada dirección. Para lograrlo debe proporcionarseles rigidez y resistencia al cortante en su plano. Los techos de armaduras de madera deben contraventearse. En pisos de parquet las duelas deben unirse para formar un elemento continuo.

d) Debe proporcionarse una liga adecuada entre muros y techos y entre muros transversales para evitar fallas por volteamiento de los muros y para una transmisión apropiada de las cargas laterales.

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Economical Roofs-Short and Long Span

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Timber is a very economical material for use in roof structures in a wide variety of spans. Its use results in very light roofs and rapid construction times. In this discussion we shall examine some specific systems and the major considerations for choosing and proportioning them.

A. Beam and Joist Systems.

Let us first consider short span Beam and joist systems. By "short span" we mean spans of about 6 meters or less, where sawn beams can be used. In this span range, spans should be chosen as long as possible within the limits of available lumber length, or to suit the architectural requirements.

Consider the floor system of Fig. 1. It is a typical Beam and joist system supporting a plywood or shiplap floor.

A live load of about 200 kg/m^2 is assumed. The design follows briefly to remind us of the design principles to be used.

Flooring

Plywood flooring 5/8 to 3/4" will suffice for the flooring. Face grain should be in the span direction, and unsupported edges should be tongue and groove or supported with blocking between the joists (unless floor underlayment is to be attached on top). Such connections permitt the floor to act as a shear diaphram for lateral force design.

Joists

The joists carry live load 200 kg/m^2. Dead load of flooring about 15 kg/m^2, piping, ductwork, etc. 15 kg/m^2 and selfweight estimated at 25 kg/m^2. Load duration is normal, conditions are dry. Joists are at 41 cm, therefore carry a D + L line load of

$$w = (250\ kg/m^2)\ (0.41) = 100\ kg/m$$

There are design aids available which directly give required joist size. Let us check flexure, assuming No 1 k.d.s.p. (No. 1 kiln dried Southern Pine). f_b = 115 kg/m^2 x 1.15where the 15% increase in allowable stress is permitted in specifications for load sharing systems.

The required section modulus is 220 cm^3 . Tables of section modulus for standard lumber sizes are available. From these we find that 2 x 8 joists have S = 13.1 in = 215 cm^3. Shear will not likely govern. Deflection should be checked and limited to about L/360. . avoid "bouncy" floors.

If we check deflection we shall find it too large. 2 x 10 joists are needed to meet the deflection criterian. Dead load of joists is now 15 kg/m^2. Beams can now be designed on a total load basis of kg/m^2 live, kg/m^2 dead load.

Beams

Total load is 245 psf on tributary width 4.8 m, giving a line load 5500 kg/m . Selfwt is about 140 kg/m. Total load 5600 kg/m, on a span of 5.5 m .

Flexure, using f_b = 115 kg/cm^2 gives a Section modulus of 3850 cm^3.

A lox 14 beam is needed; if this is too large a size, a higher grade lumber might be used. Beams can be made up of a pair of narrower members bolted to one another.

End shear is 3300 kg giving a shear of 4.9 kg/cm^2(less than allowable).

Deflection will not govern this member.

Columns

Columns will have a total load of 6500 kg. Assume column length is 2.7m (9 f_t) .

A 4 x 4 column has e/d = $\frac{9(12)}{3.5}$= 31 which is less than the maximum L/d= 50.

At this slenderness, the allowable column stress is

$$f = \frac{0.3\ E}{(e/d)^2} = 549 \text{ psi} = 38.5 \text{ kg/cm}^2$$

and the required area is $\frac{6500}{38.5}$ = 170 cm^2.

A 6 x 6 column provides 193 cm^2 and has lower L/d . Use 6 x 6 column.

We now have a design:

Joists	2 x 10	16" o.c. span 16' (4.8 m)
Beams	10 x 14	span 18' (5.4 m)
Columns	6 x 6	

Bearing stress on beam-col. is

$$\frac{14400}{30} = 480 \text{ psi} = (34 \text{ kg/cm}^2)$$

Depending on the specification used thisis a bit high. A column cap could be used of higher grade softwood timber, hardwood, or steel.

Columns will require a base plate of some type. Lateral support must be provided to the system, possible by braces between columns and beams.

We have an efficient system, of available materials, that is very fast to erect with nominally skilled labor.

Medium Spans

When spans exceed about 6 m we must go to laminated timber members.

Consider the problem of framing a large manufacturing area with a flat roof "post and beam" system. Let us assume that we can have column locations as shown in Fig. 2.

Main beam lines cross the

Structure at 7.2 spacing and the main beam spans are 9.6 m - 12 m - 9.6 m

Secondary beams at 2.4 m span 7.2 m between main beam lines. Plywood stressed skin panels are to be shop fabricated to span 2.4 m between beams.

Design of the beams for a roof dead + live load of 150 psf results in members 3 1/4 x 14 5/8. The odd numbers are standard sizes fabricated from a member 16" wide with 9-1 5/8" laminations. The beams rest in metal beam hangers attached to the main girders. The roof will provide lateral support to the beams, after it is attached. The beams weigh 7.3 kg/m^2.

The main beams can be efficienly designed as cantilevers with a central suspended span. An optimum cantilever distance can be found to minimize the size of the side span members.

This is about 1.5 m for the spans shown with uniform load.

The central suspended span is then 9 m. For a uniform dead and live load (includ ng 24' beams and the decking) of 156 kg/m^2 these beams become 5 1/4" x 24 3/8". The side spans work out to about the same size because of the benificial effect of cantilevering.

Spaced laminated timber columns can be chosen to support the loads. Lateral forces must be accomodated with end walls using the floor as a shear diaphragm.

The structure is light, fire resistant, and easy to erect with a small crane.

Many variations to this basic system can readily be imagined.

B. Trusses

Timber trusses are a very common form. Thousands of railway and roadway bridges and many large span roofs. have been constructed of them. A few important forms are shown in Fig. 3. The key features to watch for are to arrange for short stocky compression members where possible, easy connections, and forms that minimize chord stresses. The picned truss is useful for short spans (60 ft or less) where a sloping roof is desired. When very short spans such as for houses, are involved, the entire truss can be made of about 2" lumber with plywood gusset plates for connections. For example, the top chords might be 2 x 6 , bottom chord 2 x 6, and webs 2 x 4. For larger spans the top and bottom chords are spaced members, with single webs the width of the space. Bolt and split ring connectors are used. The bottom chord may be spliced at midspan with wood pieces providing a lap joint.

The flat pratt truss has similar chords and webs-spans can be great because web length does not increase with span as in the pitched truss. Chords may have more than one splice.

Compression webs are short, the longer sloping webs being in tension.

A variant of this, called a howe truss, reverses the slope direction. The sloping members are in compression, and the vertical tension webs are steel rods that pass through a single solid chord with threaded ends to anchor on a plate that bears on the outer faces of both chords. The compression members are connected on dowels. This system uses large timbers but has very simple connections. The compression member connection will only carry compression, therefore we have to watch for stress reversal under partial span loading.

On long spans the bowstring truss makes sense because under uniform loading the web forces are negligible. It acts as a tied arch. Small webs with light connections are then possible. This form should be especially attractive in Mexico, where you do not have large nonuniform snow loads. Web design here would be governed by nonuniform wind suction across the upper surface. The chords of the bowstring would be laminated, while the webs can generally be sawn lumber. Bowstring trusses can reach spans of over 200 ft.

All of these trusses might fit well with the practice here of using stone or brick walls. The trusses can bear on the top of the wall. Because of their lightness, adequate tiedown against wind suction is essential.

The depth-span ratios for these trusses should be a little greater than for similar steel trusses to keep chord sizes down. About 1: 5 or 6 is good for pitched trusses, 1:6 or 8 for bowstrings, 1: 8 or 10 for flat trusses. Roofs should have enough slope to guarantee drainage and avoid ponding.

Deflection is a much a result of joint displacement as of member length changes, thus the number of panels should be kept small. Joints are a major source of expense, another reason to minimize their number.

Design of trusses is a straightforward process. The choice of joint system is an important step. A few joint types are shown in Fig. 4 It can be seen that joint type is dependent on type of chords, (single or multiple) and type of webs (tension rods or sawn lumber). Lateral bracing must be provided for trusses, often by diagonals across pairs of trusses. This might be done in end bays, and other trusses braced by the roof beams and roofing, and by horizontal members at lower panel points. Choice of roofing system must take this requirement for bracing into consideration.

C. Arches and Portal Frames.

The variety of arches and frames possible in wood is endless. The story is best told in photographs. Most forms are three hinged, with maximum size of members limited by clearance requirements during shipping.

Let us look at a 60 m span parallel arch system as a typical example of arch construction.

Assume parallel arches at 6m spacing, 60 m span, 24 m height, 100 kg/m^2 loading or 100 kg/m^2 suction due to wind on leeward side. Analysis leads to a required section about 11" x 45 1/2" at the thickest part. The member can be tapared to a size governed by shear at the ends (about 11" x 24" at crown, 11" x 30" at base).

The analysis simply follows the usual procedure for force analysis. The section is governed by combined bending and axial load. Shear is checked at the ends. Lateral bracing is necessary, and can be provided by beams in beam hangers placed during arch erection.

Overall buckling of the arch should also be checked.

Light arches at close spacing have been found useful for farm storage structures. One firm in New York State has produced a series of designs for spans 9 to 24 m. The arches are gable arches at 2 or 4 ft.

spacing, and can be erected by field crews without cranes. They provide large farm storage buildings rapidly and cheaply. They can be roofed with plywood or light decking.

Radial arches are a common form. The one at Puebla is discussed later in this course.

Portal frames are typically used in modern church structures. Here, the architect is almost unlimited possibilities.

A problem unique to these forms is the small radius of curvature at the knees.

This results in radial tension and must be carefully accounted for. In general, radial tension arises whenever we have flexure on curved beams. Remember that wood is very weak in radial tension.

The various roof forms all have advantages worth summarizing. They are very fast to erect when fabricated in a shop in advance. They do not require highly skilled labor. They provide light, strong roofs of any span desired. They perform better in fire than steel construction if large lumber sizes are used. They provide a degree of insulation and sound deadening. If forests are properly managed, they use a renewable resource.

TIMBER TRUSS DESIGN

The design of timber trusses is a broad subject, and it is impossible to provide more than a general discussion of truss design here, highlighting design features peculiar to timber construction

Timber trusses may be classified according to several criteria

Form	Bowstring, pitched, flat, inverted bowstring, lenticular or sawtooth
Web System	Howe, Pratt or Warren (with or without verticals)
Materials	Sawn, glulam or mixed materials
Type of Fastening	Timber connectors (split rings and shear plates), or bolted joints.
Chord Arrangement	Monochord (having chord members made up of a single piece, Double Leaf (having chords of two leaves, usually with web members between), or Multi-leaf (having chords of three or more leaves).

Selection of Truss Type

Sometimes roof shape will dictate the truss type, as in a sawtooth roof More often the type of truss is selected by the designer for best economy, together with appearance requirements.

The truss type universally found to be most economical for uniform roof loads is the bowstring Here the top chord is of very nearly the same shape as an ideal arch, so that for uniformly distributed loads, stresses in chords are almost uniform throughout the truss, and web stresses are low. These features are particularly important in timber truss design because of the need for keeping connections between members simple and compact

When a flat or pitched roof is required, or when loads are transmitted to the truss of panel points rather than uniformly along the top chord, flat or pitched trusses may be used. Often it is more economical to provide buildups from bowstring trusses even for these roof shapes. Sometimes flat trusses are not needed even when a flat roof appearance is desired, if parapets or adjoining buildings control the outward appearance.

The web system used should in general permit timber web members to be used with greatest efficiency and most convenient connections Warren web systems, usually with verticals, are found best for bowstring trusses, Howe web systems for rod-and-timber trusses.

Glulam permits higher stresses than sawn timber, can be curved to any shape, can be varied in cross-section throughout its length, and can be of almost any length and size needed for the design. Sawn timber is limited in length and extent of available sizes, and is relatively prone to checking and splitting in larger sizes. Sawn members are less expensive than glulam, however, where they will serve. Thus most trusses of substantial span, or designed to carry substantial loads, are combinations of glulam and sawn timber, often including steel rod tension members or steel gusset plates, or both

To carry a given load at a joint, split-ring and shear-plate connectors are generally most economical for shop-fabricated joints, but bolted joints are to be preferred as a rule where a few holes must be bored in the field If considerable fieldboring is necessary, connector joints may be best for field work also since the necessary grooving tools can be provided

In general, monochord trusses will carry a load at least equal to that sustained by double- or multi-leaf trusses having the same cross-sectional chord area However, splices may be more cumbersome in monochord trusses, and in large or heavily-loaded trusses the chords themselves may become inconveniently large Many factors enter into the determination of the number of leaves in a truss, and it is wise to calculate stresses before deciding on truss details.

Layout

For flat trusses, the depth should be 1/8 to 1/10 of the span For pitched trusses, the roof slope should be at least 4 12 In bowstring trusses, the radius of the top chord is usually made equal to span (depth 0 134 span), although a greater radius may be used for low-rise trusses, and a shorter radius for high-rise trusses.

Panel spacing may be controlled by the desired location of purlins or concentrated loads, or by secondary bracing, usually panel length should be selected for best economy of the whole structure Usually the fewest panels that will give reasonable compression chord and web sizes should be used. For bowstring trusses, with top chords uniformly loaded, this usually means a panel length of 8 to 12 feet depending on truss span

Trusses should be cambered in the shop to offset initial deflection resulting from the setting of joints and to provide good appearance under dead and live load. Recommended camber is given on page 212.

Design

Timber trusses are usually considered as pin-connected trusses, and continuity is often taken into account in calculating combined bending and axial load in chord members, where roof loads are applied directly to chords Stress diagrams are drawn for balanced and unbalanced live loads, wind loads, and special loads Recommended loading combinations are given on page 46

Members are designed for the allowable unit stresses given on pages 11 to 13. Connectors are designed according to the data given beginning on page 142.

For a given combination of live, dead, wind or earthquake loads, the duration of such combined load can be only as great as the shortest duration of any of its components, and the working stress permitted for the shortest duration in the combination should be applied to the member stress induced by the combination of loads of various durations Thus for a member under dead load only, 90% of the tabulated allowable unit stress should be used, for a member under dead, floor and snow loads combined, 115% of the tabulated value should be used, for a member under wind, dead and wind, or dead, live, snow, and wind, 133% of the tabulated load should be used. A check should be made on individual combinations to determine which will govern. Factors for duration of load are given on page 13

In the design of bowstring trusses, when radius equals span, the central angle formed by the truss is 60°, and the top chord compression in the end panel is numerically equal to the total load on the truss. Because the circular top chord is very close to a flat parabola in shape, the top chord stress is nearly uniform throughout its length. Bottom chord stress in such a bowstring truss varies from 90% to 95% of the total load on the truss

If the roof load is applied directly to a bowstring top chord through decking or joists, the direct bending is equal and opposite to the bending caused by eccentricity of the curved chord, and the chord need be designed only for axial load and for the relatively small bending moment which results from the dead load of truss and suspended ceilings. If the roof load is applied to a curved top chord through purlins at panel points only, bending is induced in the curved top chord equal to the amount of eccentricity multiplied by chord stress, multiplied by a factor of 0.85 for continuity, the member must therefore be designed for combined bending and compression.

If the roof load is applied directly to a straight chord, as in a flat or pitched truss, the ord must be designed for combined bending and axial load.

In bowstring trusses, the greatest chord stresses are produced by full balanced vertical ad, the greatest web stresses are caused by unbalanced vertical load.

etails

Bowstring Trusses. Monochord bowstring trusses may have either continuous glulam ords, or sawn chord segments abutted at panel points. Web members may also be single-af, fastened to chords by steel straps or gusset plates, or may be double-leaf, placed out-de the single-leaf chords, and fastened by bolts and connectors.

Double-leaf bowstring trusses usually have single-leaf web systems, connector-joined chords. Since web stresses are low, a connector assembly on a single bolt will usually uffice for the web-to-chord joint. The eccentricity resulting from such connections will ot usually produce shear stresses that are too high, and they can be calculated and allowed or in design.

Multi-leaf trusses are built much the same as double-leaf trusses, the web members aving one less leaf than the chord. Double- or multi-leaf chords may be of curved glulam r of overlapping segments of sawn timber. Occasionally where web stresses are high, the eb member may have one more leaf than the chords.

Web members or chords having more than one leaf may be designed as spaced columns, rovided the end fixity and spacer-block requirements are met. Spaced column design is iven in CSA Code O86, and data are given on pages 122, 123, 128 and 129.

Flat and Pitched Trusses. Monochord flat or pitched trusses are usually of the rod-nd-timber Howe type, with diagonal web members dapped into chords, and threaded rods xtending through the chords. Such joints can be designed to avoid eccentricity. Daps hould not be made more than one-quarter the depth of the chord member. When calculating et section, both the hole and the dap should be deducted from gross section.

Double- or multi-leaf flat or pitched trusses are sometimes designed, but usually only or light loads. Heavy loads lead to large web stresses, for which the rod-and-timber type of truss is better adapted.

The use of multiple connector-and-bolt assemblies in web-to-chord joints should be avoided unless the members have been fully seasoned before fabrication to the moisture content they may be expected to attain in service. In seasoning, wood shrinks across the grain but not along the grain. Thus the shrinkage in members permitted to season after fabrication and which meet at an angle to each other causes distortion of the bolt pattern. The resulting "shrinkage stresses" cause splitting and lessen the load-carrying capacity of the joint.

In outdoor structures, and even in indoor structures of constant occupancy, some seasonal moisture changes in the wood occur, and thus joints which would result in "shrinkage stresses" should be avoided whenever possible. When they cannot be avoided, wood should be seasoned to as nearly its final equilibrium moisture content in the structure as possible, stitch bolts across the ends of web members, and saw kerfs to control location of splits, may be helpful. The harmful effects of shrinkage stresses are at a minimum when only one connector assembly is used in the joint, or when the members to be joined are parallel and are alike in moisture content and shrinkage characteristics, as in a chord splice.

Splices and Heel Joints

Splices in chord members should be made between panel points whenever possible. Trusses should be made symmetrical about their centre lines with respect to splices and joints to take advantage of production and assembly economies.

Compression splices in chords may be made by installing 18-gauge galvanized sheet metal separators tightly fitted between abutting ends to prevent interlocking of end grain. Splice plates bolted to the outside of single-leaf chords, or between the leaves of double- or multi-leaf chords, serve only to hold the abutting ends in position and do not transfer load. Thus the use of split rings in such splices is unnecessary.

Similar splices may be made using steel splice plates, when truss thickness is a factor.

Tension splices may be made using wood splice plates and split rings, or steel splice plates and shear plates. In multi-leaf trusses wood splices, or a combination of both wood and steel splice plates, may be used. The arrangement of splice plates and connectors should be such that the load is transferred as evenly as possible between chord sections, without eccentricity, this usually requires symmetry about each of three planes through the centre of the joint.

Consideration should be given to the possibility of differential shrinkage between splice plates and chord, as when unseasoned sawn splice plates are used with seasoned glulam chords, or steel splice plates are used with sawn chords. The effect will be greatest when members are deep and gage lines of connectors are widely separated, it can be minimized by using two narrow, parallel splice plates of wood, or two steel straps.

Heel connections must transfer the horizontal component of load from the top chord of a bowstring or pitched truss (or the end diagonal of a flat truss) into the bottom chord, and the vertical component to the supporting column or pilaster. For light loads, wood heel splices and split ring connectors may be used, with light steel bearing plates between truss and support. For larger trusses and heavier loads, welded steel assemblies are used, in which the top chord or diagonal bears directly on a steel heel plate, to which straps are welded which transfer load to the bottom chord through shear plates.

The number of splices to use depends upon the availability of commercial lengths of timber and the lengths which are convenient to ship and handle in assembly and erection. They should, of course, be as few as possible.

Western lumber species (Douglas fir, western hemlock) are usually readily available in lengths up to 20 feet, and are obtainable on order with reasonable ease up to 32 feet. Lengths over 24 feet, however, usually carry a premium price. Eastern species (eastern spruce, jack or red pine) are not readily available in lengths greater than 16 feet.

Glulam lengths are limited only by the size of the plant in which they are produced, shipping restrictions and handling ease. The maximum length for any given truss member will depend to some extent upon its cross-sectional size and hence its flexibility. Trusses of 80-foot to 100-foot span will probably require one chord splice, and more than 100 feet two or more chord splices.

Truss Spacing

Distance between trusses is controlled by the most economical arrangement for the loads to be carried and the other services to be performed by the roof structure, such as support of crane rails or ceiling. All other requirements being equal, best economy depends upon the members which rest upon the trusses and support the roof deck.

For sawn joists of eastern species, the maximum practical spacing, and usually the most economical, is 16 feet. For sawn joists of Douglas fir or western hemlock, 20 feet is the maximum and usually most economical. The spacing of trusses which support laminated or plank decks directly should be the maximum allowable for the species and thickness of decking used. Spacing of trusses at less than 12 foot centres, regardless of decking requirements, is seldom economical.

When glulam purlins are used, the spacing is limited only by good economy, since glulam members can be made to any length. Spacings up to 30 feet have been found economical. For buildings of any extent, where truss spacing is not controlled by other considerations, an investigation of cost of roof structure for different truss spacings and purlin arrangements may be advisable.

Longitudinal sway bracing, perpendicular to the plane of the trusses, is usually provided by sawn X-bracing in a vertical or near-vertical plane. Recommended X-brace and strut sizes are given on page 213.

Wind Bracing

Lateral wind bracing may be provided by end walls or intermediate walls or both, provided the diaphragm action of the roof or the horizontal bracing is adequate to transfer wind loads to the walls, and provided the walls themselves are adequate. Knee braces between trusses and columns may be installed to provide resistance to lateral loads to each building bent.

Horizontal framing between trusses consists of struts running between trusses at bottom chord level, and diagonal tie rods, often of steel with turnbuckles for adjustment. The arrangement provides a horizontal truss and should itself be analyzed for the required wind loads to determine member sizes and connections. Struts may be of sawn of glulam timbers, fastened by clip angles to truss chords, or may be tee struts of two planks spiked together and to the truss, snugly fitted between truss chords. Recommendations for strut sizes are given on page 213.

When knee braces are used, the truss, knee brace and column should be analyzed together as a building bent both for the wind loads applied and for vertical loads. Bending actions in knee braces should be considered and caution exercised to prevent splitting of the knee brace because of overstressing in combined bending and axial stress.

Fire Resistance

When trusses are to be incorporated in a building of "heavy timber" fire classification, the minimum truss member size will be specified by the applicable building code. The National Building Code of Canada (1953) Subsection 4.1.3 5 requires nominal dimensions of single roof truss members to be at least 4" x 6" in unsprinklered buildings, this is a common building code requirement. Other requirements deal with spaced members, sprinklered buildings and other details, the building code in force should be consulted before proceeding with design. Best economy of trusses built to qualify as "heavy timber" will be realized when the member size required for fire resistance is not greater than that required for structural adequacy, this may dictate the most practical spacing of trusses and the purlin size. Data on the fire performance of timber is given on page 262.

Standard Trusses

Some timber fabricators provide standard trusses for the more common roof loads and spans. Such standard trusses may well be more economical than trusses especially designed for a specific building, because of the advantages of quantity production, they should be considered wherever they may prove suitable.

TIMBER ARCH DESIGN

The design of three-hinged arches of the Tudor or Gothic type is not difficult, but in some cases it may become quite involved. Experience in arch design will permit the designer to avoid onerous calculations of non-critical conditions for the particular size and shape of arch being considered.

Generally speaking, vertical snow loads will govern at roof slopes under about 40° and wind loads at greater roof slopes, for customary building proportions.
In Mexico wind will probably govern.

In the following pages, the complete design of a Tudor arch by exact methods is detailed, an approximate design method suitable for roof slopes under 40°, and for preliminary design purposes, is also given. This latter method is usually sufficient for architectural purposes, but critical factors should be checked by exact methods before proceeding.

EXACT DESIGN METHOD

Basic Assumption: The designer must first make certain assumptions regarding (a) variation in arch depth, (b) radius, and (c) whether haunch, if any, will be solid or open.

Most haunched arches are tapered from the haunch toward the base and toward the crown, so that at any point along the axis the section will suffice for combined bending and compression, and so that the sections at the base and crown will be adequate for shear. In practice, the base and crown depths are usually $\frac{2}{3}$ to $\frac{3}{4}$ of those at the tangent points to avoid excessive deflection. If maximum taper is desired, stresses must be checked at a number of points along the axis.

Usually it will be sufficiently close to assume the arch axis to be 5 inches in from the outer face of the arch at the heel and crown and 8 inches at the tangent points, but if an exceptionally deep or shallow arch is required these distances may vary.

Sometimes an arch curved from base to crown is required. Such arches are usually of uniform section, in which case a conservative assumption as to depth must be made.

To permit the use of $1\frac{5}{8}$-inch laminations a radius of 40' 0" may be used if the curve is continuous to the base or crown or a 32' 0" radius if the curve stops short of these points. However, a much shorter radius is usually desired, a typical one in common usage being 9' 4" for $\frac{3}{4}$-inch laminations if the curve stops short of the base or crown or 12' 6" if it is continuous to an end. See page 209 for limiting radii. The shorter the radius the more costly will be the arch, particularly if laminations less than 1" nominal thickness are required.

When the roof slope is flat the haunch may be very deep, and it may be economical to leave the haunch open above the actual structural member required. The roof is carried by a sloping beam supported at the inner end by the arch and at the outer end by the wall.

Design Procedures The following steps are generally applicable, except for the determination of wind pressure and the method of combining forces due to various conditions of loading, these may vary according to the governing by-law. Here allowable loads and combinations thereof are in accordance with the National Building Code of Canada and stresses according to the C S A Code of Recommended Practice for Engineering Design in Timber.

1 Determine dead, snow and wind loads and their points of application

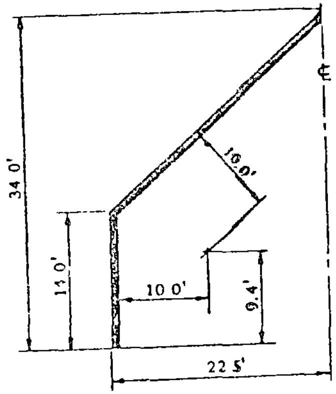
(a) Dead load per sq. ft of roof area-arch may be estimated at 5 lbs per square foot, roof and ceiling actual dead load. See page 266 for weights of building materials

(b) Snow load in pounds per square foot of horizontal projection multiplied by factor [1 – 0 0233 (roof slope in degrees – 20)] (see page 45)

PRELIMINARY ARCH DESIGN

When preliminary sections are required and the roof slope is under about 40°, wind will obably not govern and the following simplified design, which takes into account vertical ads only, may be used. The method is given in conjunction with the same example used the exact design method previously given.

1. Sketch the outline of the arch as before, assuming 9' 4" minimum radius. The dimensions, loads, and strength data are:



Dimensions:

Span 45.0 ft, haunch height 14.0 ft, crown height 34.0 ft, spacing of arches 14.0 ft

Loads.

DL 15 psf + LL 40 psf = Total vertical load 55 psf.

Allowable stresses (normal duration of load + 15% for snow):

Bending: (2400) (1.15) = 2760 psi
Compression parallel. (1600) (1.15) = 1840 psi
Longitudinal shear. (165) (1.15) = 190 psi
Compression perpendicular (390) (1.15) = 450 psi

Assume the centre of curvature and scale vertical distance from elevation of arch base.

2. Determine horizontal thrust at arch base, by summation of moments for half arch, or by formula:

$$H = \frac{\text{Roof load x arch spacing x span}^2}{\text{8 x crown height}}$$

$$H = \frac{(55)\,(14.0)\,(45.0)^2}{(8)\,(34.0)\,(1000)} = 5.7 \text{ kips}$$

3. Determine moment at tangent point (opposite centre of curvature) of wall arm.

$$M = (5.7 \text{ kips})\,(9.4 \text{ ft})\,(12)\,(1000) = 642{,}000 \text{ lb. inches}$$

4. Determine required section modulus at that point:

$$S = \frac{M}{F_b} = \frac{642{,}000}{2760} = 233 \text{ inches}^3$$

5. Determine arch size at tangent. From "Properties of Sections", 5¼" x 16½" has a section modulus of 238.2 inches³. In this case, since the roof slope exceeds 40° slightly and wind may be governing, it is best to assume an arch of one or two laminations greater depth. This would give 5¼" x 18" estimated arch size at tangent point.

6. Determine arch size at base, by formula

$$d = \frac{3}{2}\ \frac{\text{Horizontal reaction}}{\text{Allowable shear stress x arch width}}$$

$$d = \frac{(3)\,(5{,}700)}{(2)\,(190)\,(5.25)} = 8.6 \text{ inches}$$

This is the amount required for shear strength alone. As in the "exact" method, make arch depth at base ⅔ of the depth at tangent points, or about 11¼". Crown depth should be the same as base depth.

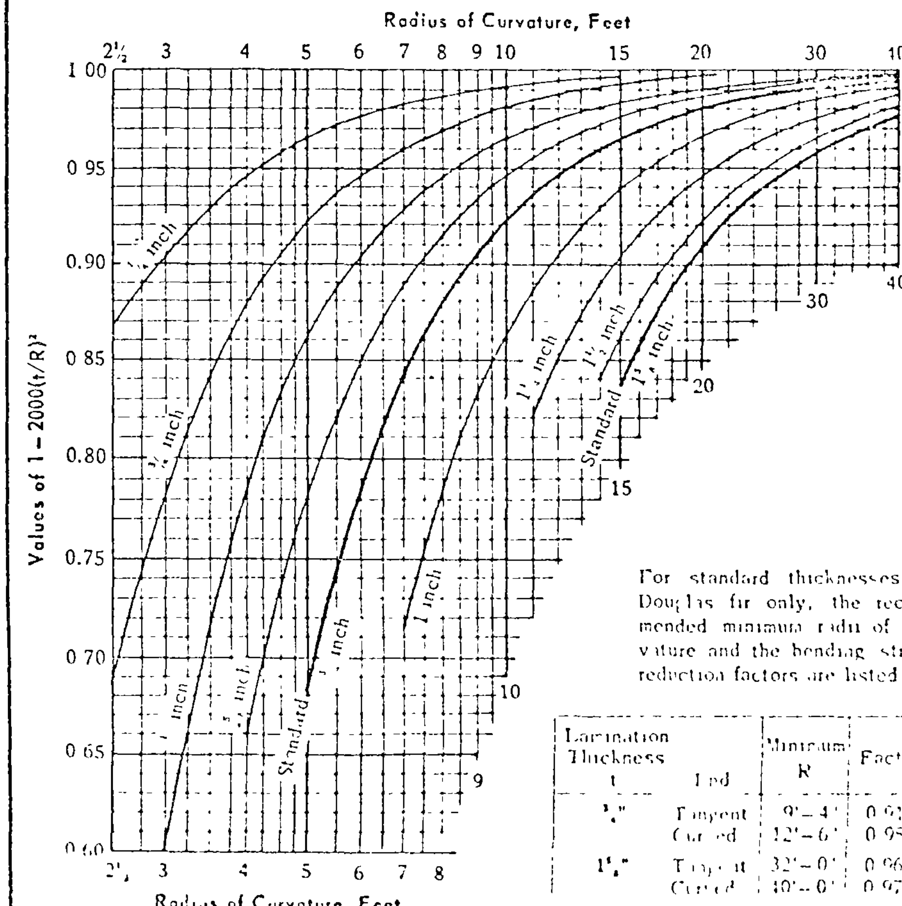
It should be noted that this method yields only approximate results, and this only when the roof slope is flat enough so that wind does not govern (about 40° or less). In this example wind was governing, and the calculated section would have been too small, had extra laminations not been added arbitrarily. However, this method provides approximate sizes quickly, for preliminary estimates.

BENDING STRESS REDUCTION FACTOR FOR CURVED MEMBERS

In accordance with CSA O86–1959, the working stress in bending, for the curved portion only of members, shall be modified by multiplication by the curvature factor

$$1 - 2000(t/R)^2$$

where t = the thickness of the lamination in inches, and
R = the radius of curvature of the lamination in inches.

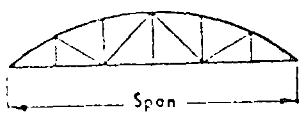


For standard thicknesses of Douglas fir only, the recommended minimum radii of curvature and the bending stress reduction factors are listed

Lamination Thickness t	End	Minimum R	Factor
¾"	Tangent	9'–4"	0.910
	Curved	12'–6"	0.950
1½"	Tangent	32'–0"	0.96[illegible]
	Curved	40'–0"	0.977

CAMBER OF TRUSSES AND BEAMS

...WSTRING ROOF TRUSSES



Segmental Overlapping Sawn Timber Chords

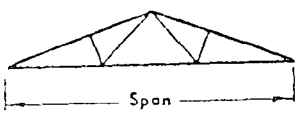
Camber in bottom chord only, equal to ⅜ inch per 10 feet of span.

Continuous Glulam Top Chord

Camber in bottom chord only, equal to ¼ inch per 10 feet of span.

...ANGULAR ROOF TRUSSES

...here feasible, camber ...top and bottom chords.



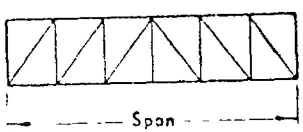
Sawn Timber or Rod and Sawn Timber

Camber in top chords equal to ⅜ inch per 10 feet of span, camber in bottom chord equal to ½ inch per 10 feet of span.

Glulam Timber or Rod and Glulam Timber

Camber in top chords equal to ¼ inch per 10 feet of span, camber in bottom chord equal to ⅜ inch per 10 feet of span

...VE AND PRATT ROOF TRUSSES



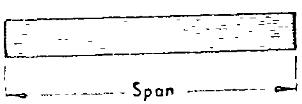
Sawn Timber or Rod and Sawn Timber

Camber in top and bottom chords equal to ½ inch per 10 feet of span.

Glulam Timber or Rod and Glulam Timber

Camber in top and bottom chords equal to ⅜ inch per 10 feet of span.

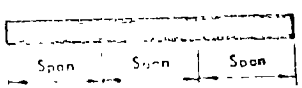
...PLE GLULAM AND OTHER BUILT-UP BEAMS



Camber equal to deflection due to twice the dead load.

Alternatively, camber of ⅜ inch per 10 feet of span will suit many conditions of loading.

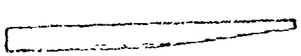
...O-SPAN AND OTHER CONTINUOUS GLULAM BEAMS



Camber built into these structures should account for the loading combinations on the various spans.

With roof spans, the centre-span camber should be greater than the adjacent-span camber to prevent excessive overloading due to unforeseen live loads as, for example, rain water accumulation.

...ANTILEVERED GLULAM BEAMS



Inadequate camber in cantilevered beams should be avoided for pleasing appearance

The advice of the fabricator should be sought for camber requirements in these structures

ROOF TRUSS BRACING

During the erection of a structure such as a building, bracing is required between trusses to provide lateral stability and to resist buckling of truss chords under compression.

Vertical, or near-vertical, crossed sway bracing, acting in tension only, is usually located in alternate bays to align each pair of trusses erected and to prevent overturning of these trusses Each truss should be braced on at least one side.

Horizontal struts, butted between bottom chords of trusses and continuous from end to end of the structure, assist in positioning the lower chord. In a similar manner, occasional purlins may be used to position the top chords of adjacent trusses.

A built-up member of two planks in the form of a "T" can be used to resist compression and, if designed properly, tension as well. To be effective, these struts must be fitted snugly between trusses, and where tension forces have to be resisted, too, must be secured by adequate fastenings to each other or to the bottom chord.

The following table indicates for various truss spacings and spans the recommended size of members to be used in T-struts and vertical sway bracing. Lumber used for these purposes should be of "Construction" grade (B.C.L.M.A.) or equivalent per CSA O141-1958, Specification for Yard Lumber.

These member sizes have been arrived at through field experience and are satisfactory for bracing and alignment of trusses during erection.

If laterally applied forces can exist, such as wind loading, then a horizontal bracing system, independent of the above ordinary truss bracing, must be designed and installed to transmit the forces to the walls and thence to the foundations.

Truss Span	Member	Truss Spacing					
		14'	16'	18'	20'	22'	24'
30' – 59' One line of bracing	T-strut Top piece	2 x 6	2 x 6	2 x 8	2 x 8	2 x 10	2 x 12
	Bottom	2 x 6	2 x 6	2 x 8	2 x 8	2 x 10	2 x 12
	X-bracing	2 x 6	2 x 6	3 x 6	3 x 6	3 x 6	3 x 6
60' – 89' Two lines of bracing	T-strut Top piece	2 x 6	2 x 6	2 x 8	2 x 10	2 x 10	2 x 10
	Bottom	3 x 6	3 x 6	3 x 8	3 x 8	3 x 8	3 x 10
	X-bracing	3 x 6	3 x 6	3 x 6	3 x 6	3 x 6	3 x 6
90' – 120' Three lines of bracing	T-strut Top piece	2 x 8	2 x 8	2 x 10	2 x 10	2 x 10	2 x 10
	Bottom	3 x 8	3 x 8	3 x 8	3 x 8	3 x 10	3 x 10
	X-bracing	3 x 6	3 x 6	3 x 6	3 x 6	3 x 6	3 x 6

Notes

1 Struts should be assembled with nails at one-foot centres and should be spiked to trusses with at least 3 nails The nail length should equal twice the thickness of the top piece of the T-strut

Alternatively, rectangularly cross-sectioned struts may be used in which case the truss connection is made with clip angles and machine bolts or by means of a properly designed hanger.

2 Sway bracing should be bolted at each end to truss web members with ¾ inch machine bolts, and be fastened at their intersection with a spacer block and one ¾ inch machine bolt

Alternatively, clip angles and bolts may be used to connect bracing to trusses.

Steel bar with turnbuckles and clips for bolting to web panel points may be substituted for wooden sway bracing members

SHOP DETAILING PRACTICE

Good drawing office practice is equally applicable to timber and to other materials. 'e following practices are generally followed by members of the Canadian Institute of ..ber Construction, and serve as a reliable guide for designers and detailers in all offices.

eneral

The methods of preparing shop details will depend upon the nature and complexity of e project. The following general rules apply, however, to most detailing jobs.

1 Careful planning of drawing and layouts is necessary for ease of reading and good appearance.

2 Repetition, unnecessary pictorial details, drawings more easily described by words, and excessive fabrication details should be avoided.

3. The principle of symmetry, standard symbols, and simplicity in erection drawings should be used to full advantage.

4 All information necessary in order to fabricate and erect should be shown or referred to on the drawings.

ypes of Drawings Required

Architectural drawings will usually show the general shape of structural members, and eir relationship to other elements of the building. Detailed drawings are necessary to ibricate and erect the structural members. Such drawings are of three types. framing etails, hardware or connection details and erection or layout drawings.

Framing details are needed for the fabricating plant to manufacture members of the quired shape, trim them to the required size, and do the necessary boring, routing or apping. The information on the framing details should refer only to the actual framing of the item All daps, cuts and counterbores should be accurately detailed. Standard etailing symbols should be used, as given on page 210.

Framing details for trusses are usually made as part of the assembly drawing for the uss, and plywood patterns are made to ensure accurate fit. When the truss is symmetrial, only half need be shown, if a small sketch showing piece marking is included.

For simple jobs, it is usually sufficient to draw elevations of typical frames or bents. ll information is provided on the elevation, such as geometry, fabricating details, and ocation of hardware A list of connecting hardware is shown next to each joint. Separate etails should be provided for cross bracing, struts, wall connections, or other types of embers used A separate detail should be shown for whatever cannot be taken care of y the elevation of the bent, such as a purlin hardware connection detail or end cutting etail.

In more complex jobs, framing members in planes normal to principal bents must be hown on additional elevations. Jobs of this kind would include buildings with wall wind racing, girder-and beam buildings, and others These additional elevations may encompass he whole building or they may be partial elevations, sufficient only to detail the member equired to be framed.

For still more complex jobs, such as skew frames or hip and valley roofs, it often is ound convenient to show large joint details and indicate the hardware at the joint. Lumber details are then shown separately with all information required for fabrication but without hardware

Hardware details are for the most part drawings of the fabricated and often welded steel parts used for connecting timber members, they are needed for the steel fabricator to work from Since the steel is often not fabricated in the same shop as timber, all necessary dimensions should be shown on these details without need for reference to other details. Plywood templates may be used when necessary to ensure accurate fit of bolts or lag screws. Hole locations and dimensions should be shown as in standard steel detailing. Standard welding symbols should be used When standard steel connections for which drawings are available are called for, they need not be detailed but are merely referred to by number.

Erection details are needed by the erection crew at the jobsite. An erection plan is drawn for the roof and each floor of a structure. This plan locates the centre lines of framing members and their location in relation to each other and to the rest of the structure. Member piece marks are shown on erection details. Anchor bolts are usually not detailed on the erection plan since they are indicated on elevations. Anchor-setting plans, may, however, be shown in these drawings, or on a separate anchor-setting drawing, when requested.

Layout of Drawings

For simple jobs, it is often possible for all three types of details to appear on one sheet. For more complex jobs, better practice dictates that separate sheets be used.

Information pertinent only to one kind of detail should not be repeated on other detail drawings, e g., a reference dimension essential to the erection plan is of no use to users of framing details.

Details should be arranged on the sheet so that they are easily read, so that there is adequate clearance between drawings and dimension lines, and so that enough space is allowed for the later possible addition of needed notes or sketches.

Scale

The scale selected for drawings will depend to some extent on the size of sheet and size of structure, but should always be large enough to allow easy reading. The following scales are usually satisfactory

Framing plan elevations.	¼" or ⅛"	= 1' 0"
Framing details	½" or ¾"	= 1' 0"
Hardware or steel details.	1" or 1½"	= 1' 0"
Erection plans.	$^1/_{16}$" or ⅛"	= 1' 0"

Piece Marking

Each member should bear a piece mark, which is shown on the framing details or hardware details, erection plans, and is marked on the member itself. All pieces bearing the same mark number are interchangeable, and conversely no two pieces which differ in any way should bear the same mark number.

Piece mark numbers usually consist of a letter designating the type of member, and a number, e g., beam markings might be shown as B1, B2, B3, etc. If two members differ only in that they are right- and left-hand, they are usually so marked, e g., B1L and B1R If there are only small differences from a standard member, a letter may be suffixed, e g., B2, B2A, B2B. Steel parts are given the same type of piece marking. Standard hardware such as bolts and timber connectors are not piece marked.

When two pieces are matched in the shop and should be similarly matched in the field, each piece should bear a separate mark number so that they will be erected properly.

Material Lists

Material lists are necessary for completion of a job They may be shown on the shop drawings, or may accompany the drawings as separate sheets Lists of the hardware, ich as bolts, washers, connectors, plates, or welded assemblies, which are required for simple connection should be shown opposite the joint on the framing details.

Clearances

The following clearances should be observed in detailing:

1. Bolt holes in wood or steel should be 1/16 inch oversize, anchor bolt holes may require additional clearance.
2. Holes for drift bolts should be the same size as the drift.
3. Holes for lag screws in softwood should be bored so that the lead hole in the main member for the shank is the same size and depth as the shank, and for the threaded portion approximately 3/4" of shank diameter, holes in side members should be 1/16 inch oversize, as for bolts.
4. Daps should be 1/16" greater in size than the member which frames into the dap.
5. Side clearances between members should be 1/8 inch, or between member and steel shoes or hangers, 1/16 inch.
6. Where members must span exactly between two fixed points an end clearance of 1/8" should be allowed.

Dimensioning

Standard dimensioning practice should be followed. Whenever possible, holes should be located on gauge lines. All gauge lines and other fabricating dimensions should be referred to the same datum plane Fractional dimensions should be shown to the nearest 1/16 inch

Sizes of member cross-sections should be given in inches. Actual cross-section size of glulam members should be given, and nominal size of sawn members. Sometimes it is necessary to surface sawn members to standard finished glulam sizes, in such cases the actual size should be given, with the notation "actual". All other dimensions should be given in feet and inches.

Radius and location of inner edge of curved members should be shown, except that when radius is large as in a cambered beam, ordinates may be shown instead. Where only centre ordinate for nominal camber of a beam is given, the beam will be cambered to a parabolic profile

When a member, truss or arch is symmetrical about a centre line, the symmetry should be noted on the centre line and dimensions given on one side only, referred to the centre line.

Notes and Special Data

Material requirements may be given on the material lists and need not be repeated on the drawings

Additional data which is required to describe fully the structural members and which may be included in notes on drawings or on material lists, are as follows.

1. Grade for sawn members, or stress-grade for glulam members.
2. Service grade (interior or exterior) for glulam members.
3. Appearance grade for glulam members.
4. Finish (seal coat, stain, varnish, etc) if required.
5. Preservative treatment if required
6. Wrapping, crating, or shipping requirements

MINIMUM RADII OF CURVATURE

The assembly layup for curved members is governed by the same considerations of species, grade and end joints that apply to straight members, but the maximum thickness of the laminations is governed by the curvature to which the laminations are to be bent.

Lumber that is to become part of a curved laminated member must be unheated and dry when bent after glue has been applied The practice of steaming thick lumber to permit bending to sharp curvature has no application in glued-laminated construction.

Standard lamination thicknesses are ¾ inches and 1⅝ inches. Generally, the most economical members can be produced from laminations of standard thickness.

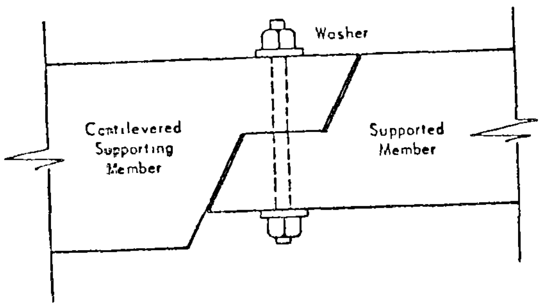
Lamination thicknesses other than standard require resurfacing, thus a greater quantity of laminating stock of standard thickness must be consumed to build the required section. When for some reason a radius less than the recommended minimum for ¾-inch laminations is required, it is advisable to choose the greatest possible radius and to consult with laminating plants before preparing specifications.

Minimum Bending Radii Recommended for Douglas Fir and Clear Straight Grained White Oak at a Moisture Content of approximately 10 Percent

Lamination Thickness t	Recommended Minimum Radii of Curvature		
	White Oak	Douglas Fir	
		Tangent Ends	Curved Ends
¼ inch	1' – 6"	2' – 7"	2' – 7"
⅜	2' – 6"	4' – 0"	4' – 7"
½	3' – 7"	6' – 0"	7' – 2"
⅝	4' – 10"	7' – 8"	9' – 10"
¾ (Standard)	6' – 1"	9' – 4"	12' – 6"
1	7' – 9"	15' – 0"	20' – 4"
1¼	11' – 8"	20' – 8"	28' – 0"
1½	14' – 10"	27' – 6"	35' – 6"
1⅝ (Standard)	16' – 0"	32' – 0"	40' – 0"

SUSPENDED BEAM CONNECTIONS

..is type of connection may be made using bolts or threaded rods, with round or square special plate washers, between e cantilevered and suspended portions of beam arrangements shown in pages 106 to 112. Other types of suspended ...m connections may be made provided they resist adequately the vertical and horizontal forces imposed.



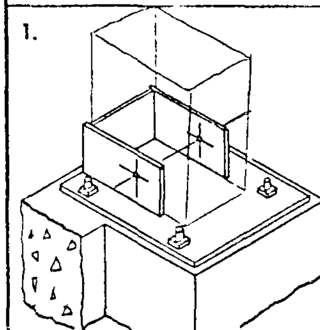
Arranging beams so that rod or bolt is in tension avoids shear at notched corners. Two bolts may be used with rectan-gular washers when loads are great and beam dimensions permit, washer dimensions must be adjusted accordingly. Side plates similar to those shown on page 234 may be added to transfer lateral loads across the joint. Another type of sus-pended beam connection may be made similar to the concealed purlin hanger shown in drawing 20 on page 232.

SQUARE PLATE				ROUND PLATE			
Rod or Bolt Load, Lb	Rod Diameter, In.	Side of Square Washer, In.	Minimum Washer Thickness, In	Rod or Bolt Load, Lb.	Rod Diameter, In.	Washer Diameter, In	Minimum Washer Thickness, In
Tensile Strength of Steel = 20,000 psi							
2,560	½	2½	¼	2,560	½	2⅞	5/16
4,110	⅝	3¼	⅜	4,110	⅝	3⅝	⅜
6,140	¾	3⅞	7/16	6,140	¾	4½	½
8,520	⅞	4⅝	½	8,520	⅞	5¼	9/16
11,200	1	5⅜	9/16	11,200	1	6¼	⅝
14,100	1⅛	6⅛	⅝	14,100	1⅛	7	¾
18,060	1¼	7	¾	18,060	1¼	7⅞	13/16
21,430	1⅜	7⅝	13/16	21,430	1⅜	8½	⅞
26,260	1½	8⅜	⅞	26,260	1½	9½	1
35,430	1¾	9¾	1	35,430	1¾	11	1⅛
46,710	2	11⅛	1⅛	46,710	2	12⅝	1 5/16
Tensile Strength of Steel = 18,000 psi							
2,310	½	2⅜	¼	2,310	½	2¾	5/16
3,700	⅝	3	5/16	3,700	⅝	3½	⅜
5,530	¾	[illegible]	[illegible]	5,530	¾	4¼	7/16
7,680	⅞	[illegible]	7/16	7,680	⅞	5	½
10,090	1	[illegible]	[illegible]	10,090	1	5¾	⅝
12,690	[illegible]	[illegible]	[illegible]	12,690	1⅛	6⅜	11/16
16,260	[illegible]	[illegible]	11/16	16,260	1¼	7½	[illegible]
19,290	[illegible]	[illegible]	[illegible]	19,290	1⅜	[illegible]	[illegible]
23,640	[illegible]	8	13/16	23,640	1½	9	[illegible]
31,890	[illegible]	[illegible]	15/16	31,890	1¾	[illegible]	[illegible]
42,040	[illegible]	[illegible]	[illegible]	42,040	2	12	[illegible]

TYPICAL CONNECTION DETAILS

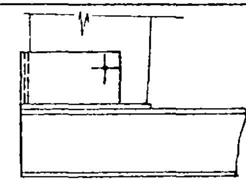
ARCH SHOES

1.



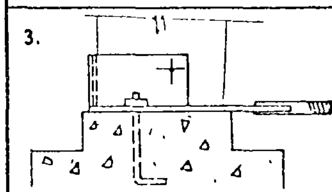
The most common type of connection between the b... of arches and the foundation, this is also the basic de... for the variations shown in (2), (3), (4) and (5) below. T... shoe is for use when the horizontal component of the a... reaction is transferred to a masonry pier through arc... bolts, and the foundation must be designed to carry ... horizontal thrust. Piers may be designed as cantilevers, they may be tied together across the building through ... floor slab. This type of shoe is suitable for horizon... thrusts up to about 10 kips. Dimensions and carrying ca... cities are given on page 236. A concealed arch shoe may ... made by slotting the arch along its centre line and usin... single plate in the slot in lieu of the side plates.

2.



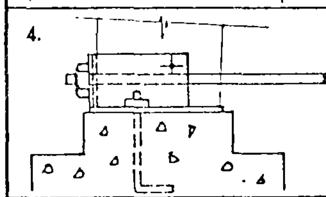
For use when the arch is supported on a steel fl... beam. This is the same assembly as in (1) above, exc... that the anchor bolts are omitted and the base plate ... made narrower. The shoe is welded to the floor beam, wh... carries the horizontal thrust by tying across to the oppos... side of the building.

3.



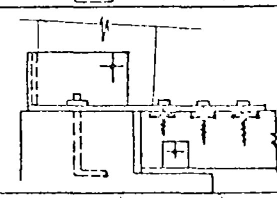
For concrete foundations not designed to carry h... zontal thrust. This is the same design as in (1) abo... except that anchor bolts and foundation need not carry ... horizontal thrust. The base plate should be extended ... enough so that the tie rod does not interfere with the b... of the arch. A short rod stub should be welded to the pl... and threaded for a turnbuckle.

4.



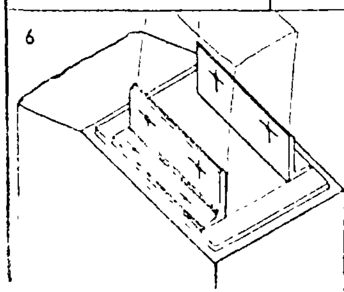
A variation of (3) having the rod extending through ... arch can be used when the floor construction permits, ... in joist floors.

5.



For use with glulam floor beams. This is the desig... (1) above, with the base plate extended far enough to pe... installation of lag screws and shear plates in the top of ... glulam beam, to carry the horizontal thrust. Anchor b... can be lighter than in (1) since they do not carry horizo... load. Alternatively, the floor beam may be extended so ... the arch rests directly upon it rather than upon ...

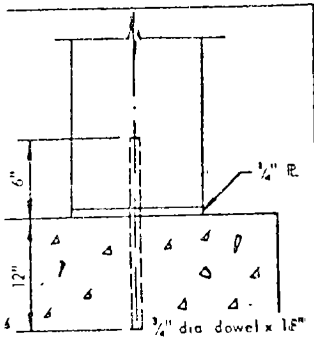
6



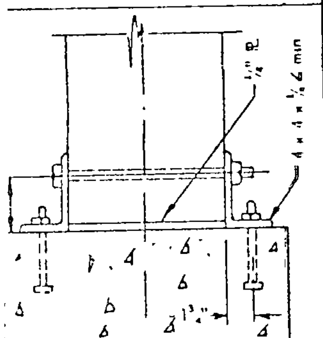
For foundation arches, where the bearing s... inclined so that loads parallel to the bearing are ... small. Similar to a column base, the side plates ... clip angles or welded lu... Shear plates may be re... in the arch to resist loads across the bearing at face, ... when necessary anchor bolts must also be de... to resist such loads. For large foundation arches ... connection may be desirable.

TYPICAL CONNECTION DETAILS

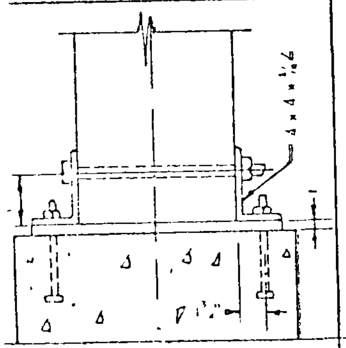
COLUMN BASES



The simplest of column bases, this may be used when uplift forces and horizontal forces are negligible. It is most frequently used for small columns.

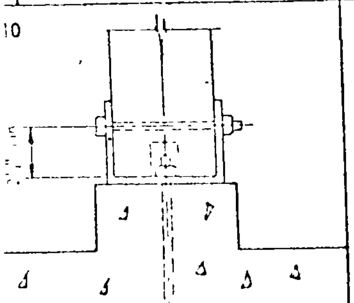


When uplift or horizontal forces, or both, may exist, a column base similar to this should be used. When necessary for greater end distance in the column, the upstanding leg of the angle may be increased. One or more bolts may be used through the column, with shear plates if necessary for the horizontal load to be transmitted.



For the same uses as (8) above, where it is desirable to provide uniform bearing surface under column and angles, or where it is necessary to distribute the vertical reaction over a larger area than that of the column. The base plate should be a minimum thickness of $\frac{1}{4}$-inch, but where the load must be distributed the thickness depends upon the area over which the load must be spread.

10

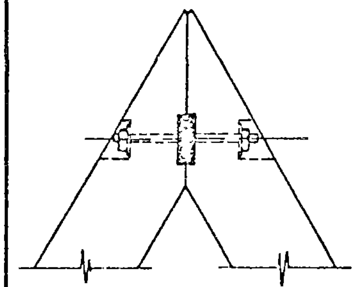


For use where there may be some horizontal load or uplift, but where the plinth is limited in size. One or more bolts may be used through column and embedded in masonry, as required, if needed, shear plates may be used in the column.

TYPICAL CONNECTION DETAILS

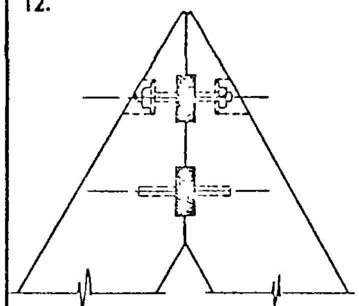
ARCH PEAKS

11.



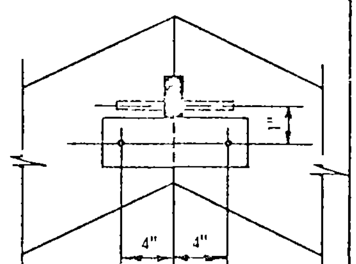
For steep arches, where bolt length is not too great, this connection transfers both horizontal and vertical loads. It consists of two shear plates back-to-back, and a machine bolt or threaded rod and washers counterbored into the arch. To avoid local crushing at peak under deflection, tips of arch halves are sometimes beveled off; sometimes a $\frac{1}{8}$-inch spacing washer is used for the same purpose.

12.



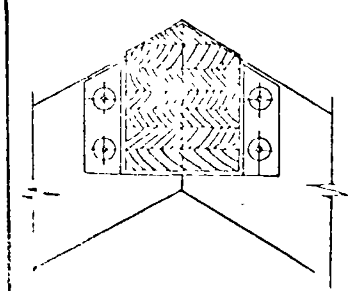
When the vertical shear is too great for one pair of shear plates, an additional pair of shear plates centered on a $\frac{3}{4}$" x 6" dowel are used. The load-carrying capacity of shear plates perpendicular to grain in end grain is about three-fourths of that perpendicular to grain in side grain.

13.



When the slope of the arch is such that bolt or rod lengths as in (11) or (12) above would be excessive, shear plates back-to-back on a $\frac{3}{4}$" x 6" dowel are used in conjunction with a $\frac{1}{4}$-inch tie plate and through bolts. When appearance is not a major factor plates on the face of the arch may be used, but if a concealed connection is desired a $\frac{3}{4}$-inch bent plate may be dapped into the top of the arch and secured with lag screws and shear plates.

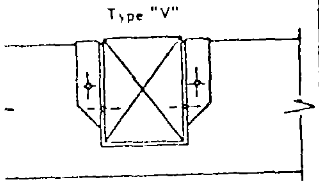
14.



Here a hanger for the ridge purlin is combined with the peak connection. The arrangement is essentially a side plate on each arch face bolted through the arch, with shear plates on the bolts when necessary to transmit vertical loads and shear between arch halves.

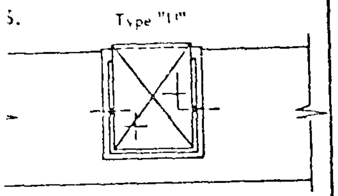
TYPICAL CONNECTION DETAILS

BEAM OR PURLIN CONNECTIONS



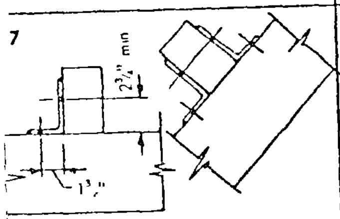
The simplest type of purlin hanger is the Type "V" strap hanger. It may be folded from plate, but is usually of welded straps. It may be single or double, and may be bolted or spiked. The hanger may be dapped into the top of the supporting beam or arch. If the purlin is of sawn timber it should extend about ⅛-inch above the supporting member for each 4 inches of purlin depth; if glulam, both members should be flush. Dimensions and load carrying capacities of Type "V" purlins begin on page 238.

5.



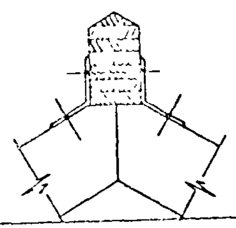
When somewhat neater appearance is desired, the Type "U" hanger may be used. Here the supporting strap is welded to a face plate, which may be nailed to the supporting member and may be lag-screwed into the top of the member. Dimensions and load carrying capacities of Type "U" purlins begin on page 239.

7



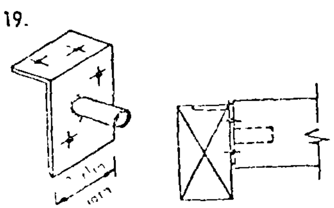
When purlins are supported on top of the beam or arch, clip angles made the simplest connection. For slopes under 45 degrees, one clip angle is usually sufficient; for greater slopes two should be used.

18.



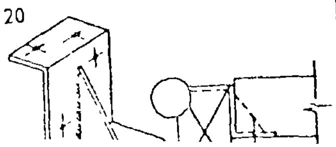
For ridge purlins supported on top of arches, bent plates may be used in place of the clip angles in (17) above. The arch is cut off so that the ridge purlin bears directly on the arch, the usual arch peak connection must also be made.

19.



For light purlin loads, one type of concealed purlin hanger may be made from welded plate and pipe. On steep slopes, the supporting arch may be dapped to receive the hanger, providing stability. Deep purlins may require two pipes. The hanger must be nailed or screwed to the end of the purlin before erection.

20

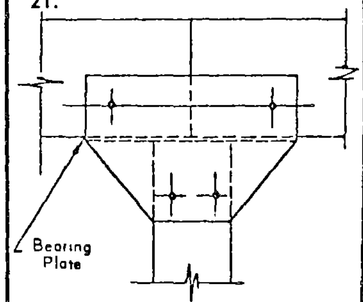


For heavier purlin loads, an alternate type of concealed purlin hanger is welded from steel plate. The end of the purlin receives a saw kerf into which the web plate fits, and both purlin and supporting beam or arch may be dapped to provide flush surfaces. Such concealed purlin hangers should always be full purlin width. This type of purlin ... also be adapted for ... suspended beam ...

TYPICAL CONNECTION DETAILS

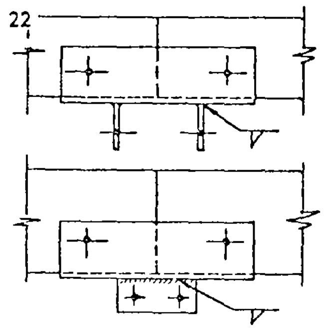
COLUMN CAPS AND BEAM SEATS

21.



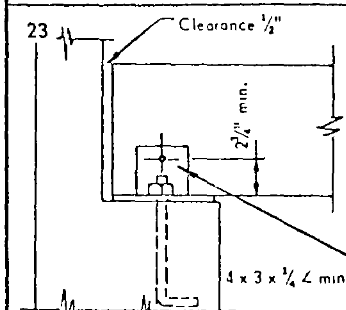
This simplest column cap is adapted to connectio... in which the beam and column are the same width. T... bearing plate may be loose or it may be welded to t... side plates, which are bolted through column and bea... Plate thickness are ¼ inch in most cases, unless requir... bearing area and overhang calls for thicker plate. Bolts a... for positioning and transfer of lateral loads only, vertic... loads are carried directly in bearing through the beari... plate. A concealed column cap may be made by using... single plate placed in a mortise cut in beams and colum... in lieu of two side plates.

22



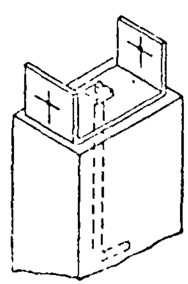
These two alternatives to (21) are useful when t... column and beam are of different widths. They must ... welded assemblies. Minimum edge and end distances f... bolts in timber members should be observed in these a... in (21).

23



When a beam bears on masonry, it should always h... at least a ¼-inch bearing plate, and thicker bearing pla... if it is necessary to distribute the load over a greater ar... Anchor bolts should extend through bearing plate and c... angle, sometimes lugs may be welded to bearing plate. ... bearing plate with concealed dowel, similar to (7), ... sometimes used.

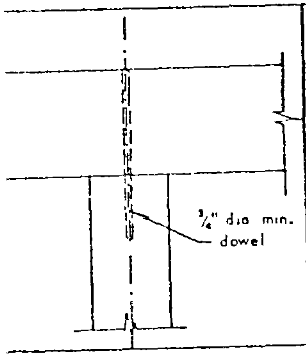
24.



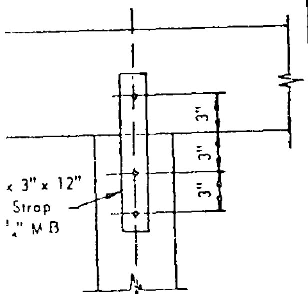
Where the bearing area is not wide enough for c... angles, a bent bearing plate is used, with the bottom ... the beam routed out to receive the anchor bolt head.

TYPICAL CONNECTION DETAILS

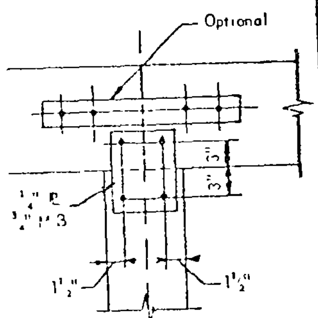
COLUMN CAPS



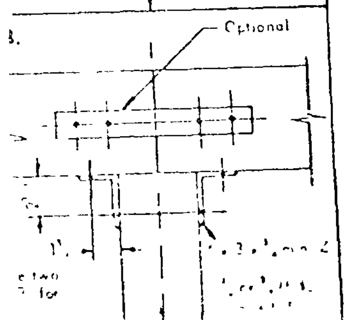
In this simplest beam-to-column connection for beams continuous over the column, the dowel may be driven in prebored holes from the top of the beam, or it may extend partially through the beam, with hole in beam bored only part way through. Either round dowels or spiral dowel made from square bars may be used.



For beams continuous over the column, this connection provides for uplift. A loose bearing plate may be used where the column cross-section provides inadequate area for bearing on the beam in compression perpendicular to grain.



3.

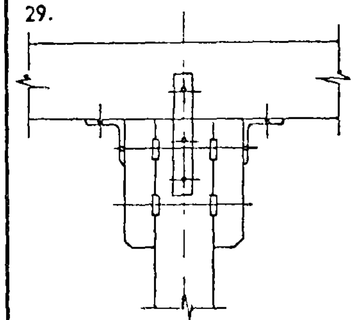


These column connections may be used either for continuous beams, or where beams abut over the column A splice plate on each side of the beam should be used across such butt joints to tie the members together and to transfer lateral loads. Usually ¼" x 3" strap is sufficient, unless lateral loads require heavier steel, bolts only, or bolts and shear plates, may be used. If the column cross-sectional area is insufficient for bearing in compression perpendicular to grain in the beam, loose bearing plates may be used, to distribute the load An alternative splice plate method calls for dapping the top of the beam for a single splice plate, fastened with lag screws with or without shear plates, depending on the load to be transferred. In such cases, holes should be slotted to avoid restraint under load.

TYPICAL CONNECTION DETAILS

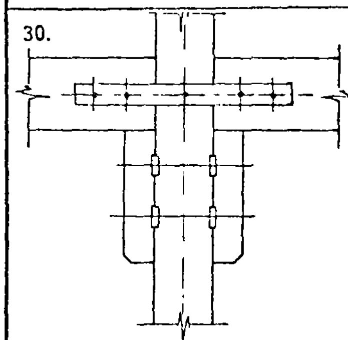
BEAM CONNECTIONS TO COLUMNS

29.



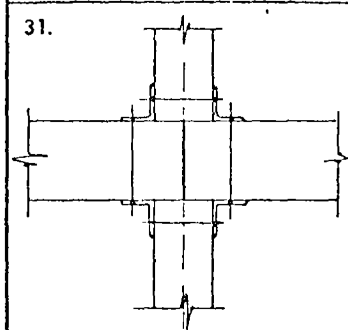
As an alternative method of distributing the load over a beam area greater than that of the supporting column, corbels may be used, with or without an additional bearing plate. The load carried by the corbels is carried into the main column through split rings. Dimensions of strap and angles are similar to those given in column caps (26) and (28).

30.



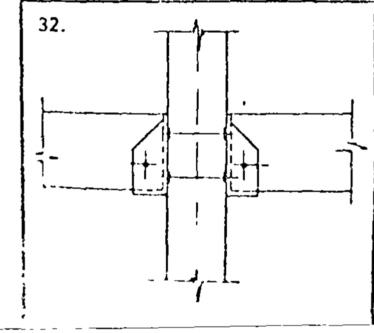
When the column continues through, in multi-story buildings, this variation may be used. With glulam columns, the column size may be stepped down at the elevation of the corbel.

31.



When beams abut at a column in multi-story buildings, clip angles may be used above and below the beam, with a bearing plate of required thickness for the needed distribution of load.

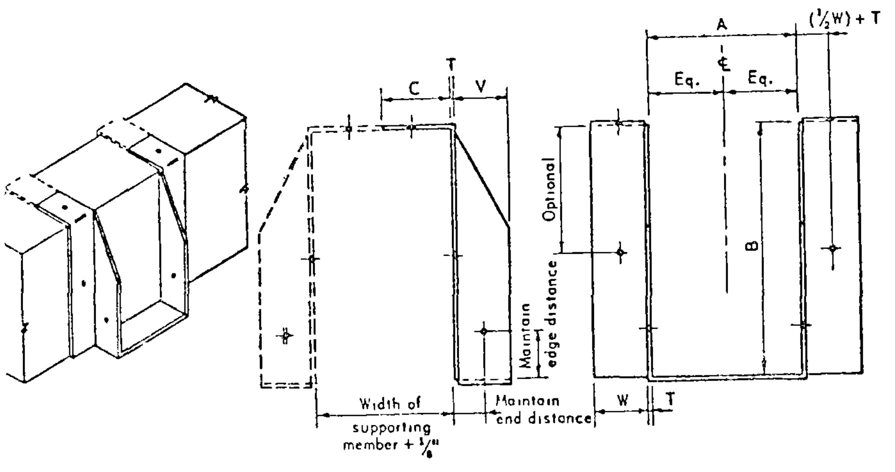
32.



As an alternate to (30) above, beam or purlin hangers similar to the Type "U" hanger of (16) can be used. Here the reaction must be transmitted to the column through shear plates. The column may also be stepped in size at the elevation of the bottom of the beam.

TYPICAL PURLIN HANGERS

SINGLE OR DOUBLE HANGERS, TYPE "V"



Nominal Size In Depth	A	B	C	W	V	T	Max Load Kips	Weight of Single Hanger Lb
6	2 3/4	5 3/8	2 1/2	1 1/2	2	[illegible]	1.5	1.5
8	2 3/4	7 1/4	2 1/2	1 1/2	2	[illegible]	1.5	2.0
10	2 3/4	9 1/16	2 1/2	1 1/2	2 1/2	[illegible]	1.9	2.5
12	2 3/4	11 1/8	2 1/2	1 1/2	2 1/2	[illegible]	1.9	3.0
14	2 3/4	13 1/8	2 1/2	1 1/2	3	[illegible]	2.3	3.8
16	2 3/4	15	2 1/2	1 1/2	3	3/16	2.3	6.8
6	3 3/4	5 3/8	2 1/2	2	2	1/8	2.1	1.8
8	3 3/4	7 1/4	2 1/2	2	2	1/8	2.1	2.4
10	3 3/4	9 1/16	2 1/2	2	2 1/2	1/8	2.6	3.2
12	3 3/4	11 1/8	2 1/2	2	2 1/2	1/8	2.6	3.6
14	3 3/4	13 1/16	2 1/2	2	3	3/16	3.1	6.8
16	3 3/4	15	2 1/2	2	3	3/16	3.1	7.9
6	5 3/4	5 3/8	2 1/2	2	2	5/32	3.2	2.4
8	5 3/4	7 3/16	2 1/2	2	2	5/32	3.2	3.1
10	5 3/4	9 1/8	2 1/2	2	2 1/2	3/16	3.9	4.9
12	5 3/4	11 1/16	3 3/8	2	2 1/2	3/16	4.0	5.9
14	5 3/4	13	3 3/8	2	3	1/4	4.8	9.8
16	5 3/4	14 15/16	3 3/8	2	3	1/4	4.8	11.2
8	7 1/4	7 1/16	4 3/8	2	2	3/16	4.3	4.5
10	7 1/4	9 1/8	4 3/8	2	2 1/2	3/16	5.4	5.4
12	7 1/4	11	[illegible]	2	2 1/2	[illegible]	5.4	8.5
14	7 1/4	13	4 3/8	2	3	[illegible]	6.5	10.7
16	7 1/4	[illegible]	[illegible]	2	3	[illegible]	6.5	11.8
10	9 1/4	9 1/16	4 3/8	[illegible]	3 1/2	[illegible]	8.5	9.7
12	9 1/4	11	4 3/8	[illegible]	3 1/2	[illegible]	8.5	11.4
14	[illegible]	13	[illegible]	[illegible]	[illegible]	[illegible]	8.5	[illegible]

GLULAM PURLINS

Actual Purlin Size, In Width	Depth	A	B	C	W	V	T	Max. Load Kips	Weight of Single Hanger Lb
3 1/4	4 7/8	3 5/8	4 3/4	2 1/2	1 1/2	2	1/8	2.7	1.4
	6 1/2	3 5/8	6 3/8	2 1/2	1 1/2	2	1/8	2.7	1.9
	8 1/8	3 5/8	8	2 1/2	1 1/2	2	1/8	2.7	2.3
	9 3/4	3 5/8	9 5/8	2 1/2	1 1/2	2 1/2	1/8	2.9	2.7
	11 3/8	3 5/8	11 3/16	2 1/2	1 1/2	2 1/2	3/16	2.9	4.7
5 1/4	6 1/2	5 5/8	6 3/8	2 1/2	2	2	5/32	3.9	2.8
	8 1/8	5 5/8	8	2 1/2	2	2	5/32	3.9	3.3
	9 3/4	5 5/8	9 9/16	2 1/2	2	2 1/2	3/16	3.9	5.1
	11 3/8	5 5/8	11 3/16	2 1/2	2	2 1/2	3/16	3.9	5.8
	13	5 5/8	12 13/16	2 1/2	2	2 1/2	3/16	3.9	6.6
	14 5/8	5 5/8	14 3/8	3 3/8	2	3	1/4	5.2	11.2
	16 1/4	5 5/8	16	3 3/8	2	3	1/4	5.2	12.2
	17 7/8	5 5/8	17 5/8	3 3/8	2	3 1/2	1/4	5.2	14.2
7	8 1/8	7 1/4	7 15/16	3 3/8	2	2	3/16	5.2	4.4
	9 3/4	7 1/4	9 9/16	3 3/8	2	2 1/2	3/16	5.2	5.5
	11 3/8	7 1/4	11 1/8	3 3/8	2	2 1/2	1/4	5.2	8.4
	13	7 1/4	12 3/4	3 3/8	2	2 1/2	1/4	5.2	9.4
	14 5/8	7 1/4	14 3/8	4 3/8	2	3	1/4	7.6	11.3
	16 1/4	7 1/4	16	4 3/8	2	3	1/4	7.6	12.4
	17 7/8	7 1/4	17 9/16	4 3/8	2	3 1/2	5/16	7.6	18.1
9	9 3/4	9 1/4	9 1/2	4 3/8	2	2 1/2	1/4	6.8	7.9
	11 3/8	9 1/4	11 1/8	4 3/8	2	2 1/2	1/4	6.8	9.2
	13	9 1/4	12 3/4	4 3/8	2 1/2	2 1/2	[illegible]	8.5	[illegible]
	14 5/8	9 1/4	14 3/8	4 3/8	2 1/2	3	[illegible]	8.5	[illegible]
	16 1/4	9 1/4	15 15/16	4 3/8	2 1/2	3	[illegible]	[illegible]	[illegible]
	17 7/8	9 1/4	17	4 3/8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

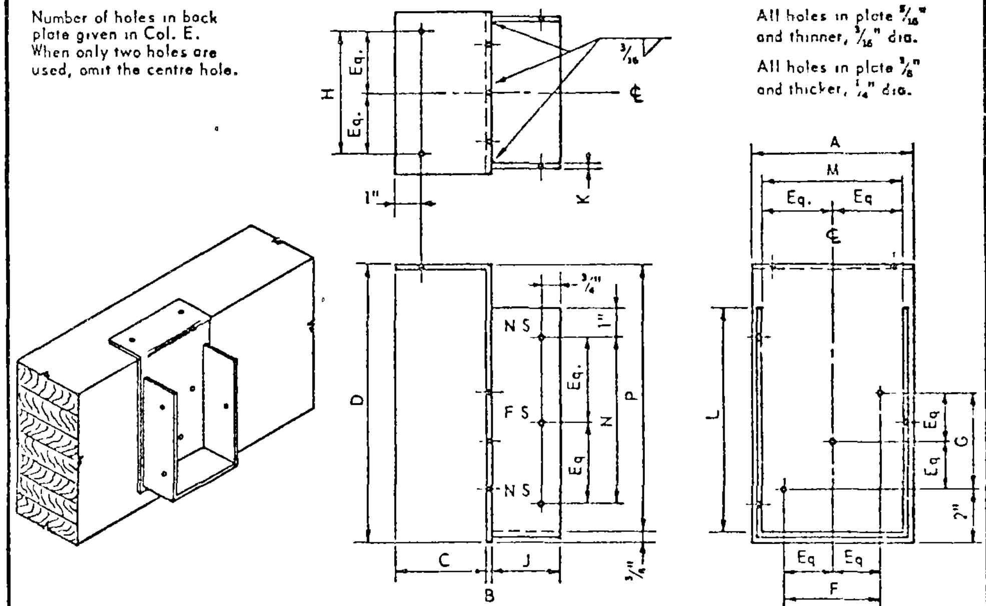
TYPICAL PURLIN HANGERS

SINGLE HANGERS, TYPE "U"

Number of holes in back plate given in Col. E. When only two holes are used, omit the centre hole.

All holes in plate 5/16" and thinner, 3/16" dia.

All holes in plate 3/8" and thicker, 1/4" dia.



Purlin Size, In. Width	Depth	Max. Allow Reaction Kips	Back Plate, Inches A	B	C	D	E	F	G	H	Hanger Plate, Inches J	K	L	M	N	P	Wt Each Lbs
3 1/4	4 7/8	1.0	4	1/4	2 1/2	5 1/4	2	1 1/2	2	2 1/2	2	3/16	3 1/4	3 3/8	1 1/4	4 7/8	3.4
		1.5	4	5/16	2 1/2	5 1/4	2	1 1/2	2	2 1/2	2	3/16	3 1/4	3 3/8	1 1/4	4 7/8	4.0
		2.0	4	3/8	2 1/2	5 1/4	2	1 1/2	2	2 1/2	2	3/16	3 1/4	3 3/8	1 1/4	4 7/8	4.5
	6 1/2	1.0	4	1/4	2 1/2	6 7/8	2	1 1/2	2 1/2	2 1/2	2	3/16	4 7/8	3 3/8	2 7/8	6 1/2	4.1
		1.5	4	5/16	2 1/2	6 7/8	2	1 1/2	2 1/2	2 1/2	2	3/16	4 7/8	3 3/8	2 7/8	6 1/2	4.8
		2.0	4	3/8	2 1/2	6 7/8	2	1 1/2	2 1/2	2 1/2	2	3/16	4 7/8	3 3/8	2 7/8	6 1/2	5.4
	8 1/8	1.0	4	1/4	2 1/2	8 1/2	2	1 1/2	3	2 1/2	2	3/16	6 1/2	3 3/8	4 1/2	8 1/8	4.9
		1.5	4	5/16	2 1/2	8 1/2	2	1 1/2	3	2 1/2	2	3/16	6 1/2	3 3/8	4 1/2	8 1/8	5.7
		2.0	4	3/8	2 1/2	8 1/2	2	1 1/2	3	2 1/2	2	3/16	6 1/2	3 3/8	4 1/2	8 1/8	6.5
	9 3/4	1.0	4	1/4	2 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	6.2
		1.5	4	5/16	2 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	7.1
		2.2	4	3/8	2 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	8.0
		0.7	4	1/4	3 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	6.4
		1.1	4	5/16	3 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	7.4
		1.6	4	3/8	3 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	8.1
		2.4	4	1/2	3 1/2	10 1/8	3	1 1/2	3 1/2	2 1/2	2 1/2	3/16	8 1/8	3 3/8	6 1/8	9 3/4	10.3
	11 3/8	1.0	4	1/4	2 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	7.2
		1.5	4	5/16	2 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	8.2
		2.2	4	3/8	2 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	9.2
		0.7	4	1/4	3 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	7.1
		1.1	4	5/16	3 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	8.5
		1.6	4	3/8	3 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	3/16	9 3/4	3 3/8	7 3/4	11 3/8	9.6
		[illegible]	4	[illegible]	3 1/2	11 3/4	3	1 1/2	4	2 1/2	2 1/2	[illegible]	9 3/4	3 3/8	7 3/4	11 3/8	[illegible]

ARC AND GAS WELDING

UNIT STRESSES AND WELDING SYMBOLS

Unit Stresses of Weld Metal

SHEARING — On section through throat of fillet weld, or on faying surface area of plug or slot weld . . . . 13,600 psi

On section through throat of butt weld . . . . 13,000 psi

(Stress in a fillet weld shall be considered as shear on the throat, for any direction of applied stress. Neither plug nor slot welds shall be assigned any values in resistance to stresses other than shear.)

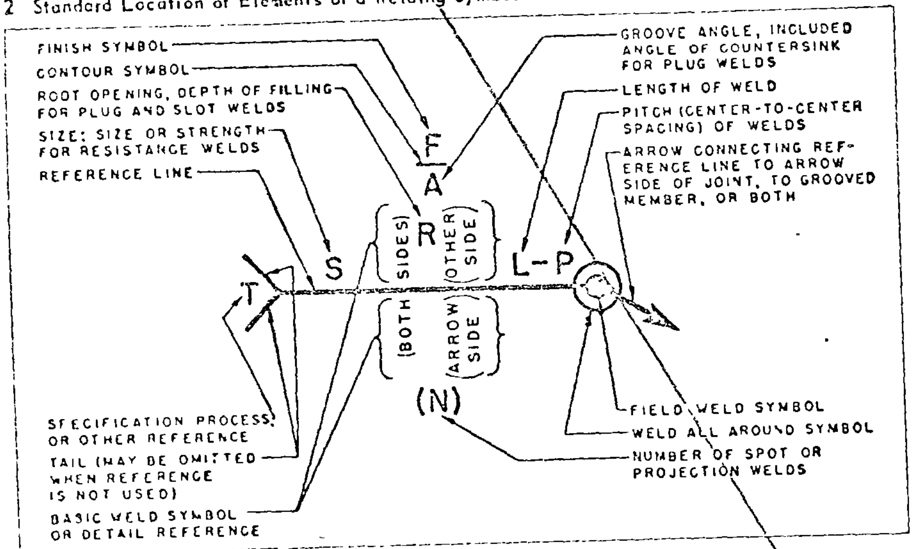
TENSION — Butt welds, section through throat . . . . 20,000 psi

COMPRESSION — Butt welds, section through throat (crushing) . . . . 20,000 psi

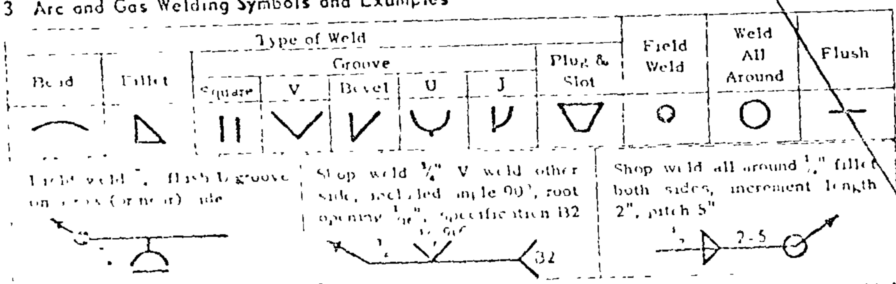
BENDING — Fibre stresses in butt welds, due to bending, shall not exceed the above values prescribed for tension and compression, respectively.

— from AISC Specification, 1947

2 Standard Location of Elements of a Welding Symbol



3 Arc and Gas Welding Symbols and Examples



— Extracted with permission from ASA Standard Z32.2.1–19[illegible] published by the American Society of Mechanical Engineers

RAILROAD CROSSTIES AND SWITCH TIES

Dimensions for Standard Crossties and Switch Ties

The standard crosstie length in Canada is 8 feet except in Newfoundland where 7 feet is standard. The Canadian standard switch tie is 7 in x 9 in., lengths from 8' 6" to 16' 6" depending on the turnout for which required.

Size No. 1 crossties are widely used under heavy traffic combined with higher speeds.
Size No. 2 crossties are used under main line traffic and branch lines.
Size No. 3 crossties are used on some branch lines and sidings.

Size	Kind of Wood	Sawed or Hewed Top Bottom & Sides	Sawed or Hewed Top & Bottom Only
1	Soft	9" to 9½"; 7" to 7½" — 9" to 9½", 7" min; 7" to 7½" — 9" to 9½", 8" min	
1A	Soft		7" to 10"; 7" to 7½"
2	Soft or Hard	8" to 8½"; 6" to 6½" — 8" to 8½", 6" min; 6" to 6½" — 8" to 8½", 7" min	
2A	Soft		6" to 10"; 6" to 6½"
3	Hard	7" to 7½"; 6" to 6½" — 7" to 7½", 6" min	
3	Soft	8" to 8½", 5" min; 6" to 6½"	
3A	Soft		11" max, 5" min; 6" to 6½"

Species of Wood

Hardwoods and softwoods are used for both crossties and switch ties. Hardwoods are usually ordered mixed allowing the following species oak, beech, birch, and maple. Softwood species specified are usually one or more of the following Douglas fir, larch, hemlock tamarack, lodgepole pine, jack pine and red pine.

Preservative Treatment

The major Canadian railways have standardized on the use of pressure treated ties. The standard preservative used is a mixture of 50% creosote and 50% petroleum [illegible]. The recommended retention is 7 lb of preservative per cubic foot of wood. The [illegible] life of pressure treated ties in Canada is more than thirty years. This is three to [illegible] the best service life that can be realized using untreated ties or ties treated with [illegible] treatment.

Canadian Standard Association Specification

The recognized specification covering pressure treated crossties and switch ties is CSA Spec O80 C6, Preservative Treatment of Crossties and Switch Ties by Pressure Processes. Other CSA Specifications related to wood preservation are listed on page [illegible]

ROUND TIMBER POLES
FOR UTILITIES AND BUILDINGS

Dimensions

Tables of dimensions for poles of various species classed as above are provided in applicable Canadian Standards Association Specifications. The following table is an extract from CSA Spec O15 3, Physical Properties of Jack, Lodgepole and Red Pine Poles and Reinforcing Stubs (Third Edition), to which reference should be made for similar tables giving dimensions of lodgepole pine and red pine poles. Such tables for other species are given in appropriate CSA specifications.

DIMENSIONS OF JACK PINE POLES

Figures Based on Ultimate Fibre Stress of 6,400 Pounds per Square Inch

Class		1	2	3	4	5	6	7	8	9	10
Minimum circumference at Top (In.)		27	25	23	21	19	17	15	15	15	12
Length of Pole (Feet)	*Ground Line Distance from Butt (Feet)	Minimum Circumference at Six (6) Feet from Butt (Inches)									
16	3½					22 0	20 5	19 0	17 5	Less than Class 8 poles of same length	Less than Class 8 poles of same length
18	3½			27 5	25 5	23.5	21 5	20 0	18 5		
20	4	33 0	31.0	29 0	26.5	24 5	23 0	21 0	19.5		
22	4	34 5	32.5	30.0	28.0	26 0	24.0	22 0	20.5		
25	5	36 0	34.0	31 5	29.5	27 0	25 0	23.0	21.5		
30	5½	39 0	36.5	34 0	32 0	29.5	27.0	25 0	23.5		
35	6	42 0	39 0	36 5	34 0	31 5	29.0	27.0	25.0		
40	6	44.0	41.5	38 5	36 0	33 0	30 5	28.5			
45	6½	46 0	43.5	40.5	37 5	35 0	32.0	30.0			
50	7	48 0	45.0	42 0	39 0	36.5	33.5	31.0			
55	7½	50 0	47.0	43 5	40 5	37.5	35.0				
60	8	51.5	48.5	45 0	42 0	39 0	36 0				
65	8½	53 5	50.0	46 5	43 5	40 0					
70	9	55 0	51.5	48.0	44 5	41 5					

Note Dash lines indicate length limits.

* The figures in this column were used in determining the required circumferences at ground line and should be used whenever a definition of ground line is necessary.

Lengths

Poles are produced in lengths from 16 feet to 70 feet or more, depending on the species The standard lengths are 16, 18, 20, 22, 25, and 30 feet, and longer in 5 foot increments

Pole strength Classes

Eight different pole strength classes have been adopted, each having a different breaking load value For two additional classes, 9 and 10, top circumferences only are specified In the cases of classes 1 to 8, the breaking loads have been used to compute the necessary circumferential measurements. Thus a pole of any given length and class has a fixed strength regardless of species, for engineering design and practical purposes although the dimensions will vary with the species because of the inherent differences in strength of the various species.

For purposes of calculating strength, it is assumed that the load is applied two feet from the top of the pole and that the pole breaks at the ground line. The following table gives the approximate breaking load for Classes 1 to 8

Pole Classes and Approximate Breaking Loads

Class of Pole	Approximate Breaking Load
1	4,500 pounds
2	3,700 pounds
3	3,000 pounds
4	2,400 pounds
5	1,900 pounds
6	1,500 pounds
7	1,200 pounds
8	960 pounds

Species of Wood

In Canada poles are normally produced from jack pine, red pine, lodgepole pine, Douglas fir, western larch or western red cedar.

Preservative Treatment

Service conditions under which poles are normally used are such that they are exposed to decay. In order to obtain the best service life and to maintain the strength of poles, they should be properly treated with an approved preservative.

Preservatives commonly used are creosote, or a 5% solution of pentachlorophenol in oil.

The best service life is obtained from full length pressure-treated poles. A retention of six to eight pounds of preservative per cubic foot of wood is recommended

The heartwood of western red cedar has a good natural resistance to decay, but the sapwood has considerably less This species of pole has normally been used with a butt treatment only in accordance with Canadian Standards Association Specification O80 C7. There has been a substantially increasing trend toward full-length treatment of this species.

Service Life

Due to varied service conditions, qualities of different species, etc., it is impossible to estimate service life accurately. The following table gives a conservative estimate of pole service life that may be expected under average service conditions.

Full Length Pressure Treated Poles	40 to 50 years
Butt Treated Western Red Cedar Poles	25 to 35 years
Untreated Poles	4 to 20 years

(Some species are more decay-resistant than others, when untreated)

Canadian Standards Association Specifications

The recognized specifications covering round timber poles and their preservative treatment are as follows

- O15.1 The Physical Properties and Preservative Treatment of Eastern White Cedar Poles.
- O15 2 The Physical Properties and Preservative Treatment of Western Red Cedar Poles.
- O15 3 The Physical Properties and Preservative Treatment of Jack, Lodgepole and Red Pine Poles and Reinforcing Stubs
- C15(E) The Physical Properties and Preservative Treatment of Douglas Fir Poles
- O80 C4 Preservative Treatment of Poles by Pressure Processes
- O80 C7 Preservative Treatment of Incised Pole Butts by the Non Pressure Process, Western Red Cedar and Northern White Cedar Poles

Other CSA Specifications related to wood preservation are given on pages 260 and 261

ROUND TIMBER PILES

The following table showing sizes of piling normally available is from Appendix A of CSA specification O56.1962.

TABLE A1
SIZES OF TIMBER PILES

Size Designation	Size 14	Size 13	Size 12	Size 11	Size 10	Size 9
Diameter at Extreme Butt or Large End Inches	14	13	12	11	10	9
Length Feet	Diameters at Tip Small End Inches					
Up to 20	10	10	9	8	7	6
20 to 34	10	9	8	7	6	6
35 to 44	9	8	7	6	–	–
45 to 59	8	7	7	–	–	–
60 to 69	8	7	6	–	–	–
70 to 89	7	6	–	–	–	–
90 to 105	6	5	–	–	–	–

Notes 1 Diameters are minimum except tolerances allowed by Clause 4 2.1 5 will apply.

2 Maximum diameter at butt shall not exceed 20 inches for any size.

3 The Table is written in such a way that the butt size governs in most cases other than for very long piles, and tip sizes shown are only restrictive in excluding abnormal piles.

When design requirements are such that tip size only governs the selection of the size of the pile, it is recommended that minimum tip size only be specified with maximum butt diameter not to exceed 20 inches.

When selecting a piling species to meet design and length requirements it is advisable to determine its availability or permit the use of one or more alternative species having sufficient strength.

Extremely long piles are available only in Douglas Fir, Western Red Cedar, Pacific Coast (Western) Hemlock, and Western Larch Species

Preservative Treatment

Pressure treatment of timber piles should be specified when a permanent foundation is required The two exceptions to this statement are (1) when piles are permanently and completely submerged in fresh water, and (2) when piles are entirely embedded in earth and cut off below ground water table If there is any possibility of the water table fluctuating or being lowered in the future, pressure treated poles should be specified.

The required retention of preservative varies with species and service conditions, the following table shows recommended minimum retentions

Minimum Retention of Creosote, Pounds Per Cubic Foot of Wood

Species	General Use	Coastal Use*
Coast Douglas Fir	8	12 Full Cell
Larch	10	12 Full Cell
Lodgepole & Jack Pine	8	12 Full Cell
Red Pine	10	16 Full Cell
Tamarack	8	12 Full Cell

* Coastal use refers to use in salt water where timber is subject to attack by marine borers

Canadian Standards Association Specifications

The recognized specifications covering piles and their pressure treatment are as follows

O56 Standard Specification for Round Timber Piles
O80 C3 Preservative Treatment of Piles by Pressure Processes
O80 C12 Specification for Creosoted Wood Foundation Piles.

Other CSA Specifications related to wood preservation are listed on pages 260 [illegible]

PART IV

TIMBER TECHNOLOGY

This part contains condensed information on timber technology with which the designer should be familiar in order to make the most effective use of timber as a structural material.

TIMBER CONSTRUCTION AND FIRE SAFETY

The strength of materials at ordinary temperatures bears little resemblance to the load-carrying capacity under fire conditions, where temperatures may be of the order of 1500° to 1800°F. Unlike many other materials, wood in construction sizes retains its strength and shape for prolonged periods of severe heat exposure. Timbers do not expand or distort in a fire. For this reason, adjoining walls, floors, and roofs are not affected beyond the local zone of fire exposure.

In the important fire protection aspect of structural endurance, timber performs better than most non-combustible building materials. Although the surface may get charred, the undamaged wood below the surface retains its strength and will support a load equivalent to the capacity of the uncharred wood size. In actual fires, the extent of charring is seldom more than one-half inch deep. For this reason, fire exposed timbers often remain intact with a serviceable safety factor.

Fire endurance or collapse resistance of load bearing timber assemblies is a matter of the thickness of wood and the number of faces exposed to fire, plus the actual load carried by the member. The same thickness of wood will have various fire endurance times or "ratings" dependent on where it is used e.g., a column, beam, floor, roof, wall, partition etc

Consequently building codes specify minimum nominal dimensions to ensure adequate fire endurance in exposed structural elements of timber. Buildings of timber construction are usually designated as one of three general code types Wood Frame Construction, Ordinary Construction, and Heavy Timber (also called Mill) Construction

Wood Frame Construction has exterior walls of wood and structural framing of smaller dimensions than Heavy Timber Ordinary Construction has masonry exterior walls and structural framing of wood of smaller dimensions than Heavy Timber. Framing of Ordinary and Wood Frame Constructions is often provided with fire resistant coverings, where the occupancy risk or building height warrants this added protection.

Normally, framing of Heavy Timber Construction is exposed for purposes of appearance and economy. The superior fire resistance of Heavy Timber Construction is due to the use of timbers of massive size Section 4 1 3 5 of the National Building Code of Canada (1953) is a typical example of the specification for Heavy Timber Construction. Glued-laminated timber construction is also recognized in Section 4.1 3 5 as equivalent in fire performance to sawn timbers of the same cross-sectional dimensions.

Fire Protection

The extent of structural fire endurance desirable beyond building code requirements is a matter of economic balance between the risk inside the building and the available fire protection facilities. Experience indicates that too much emphasis on the fire resistance of the structural frame results in a false sense of security. The emphasis is wrongly placed on so called "fireproof" buildings rather than fire-safe buildings. Good building design and adequate structural endurance, plus automatic detection and extinguishing facilities where needed provide maximum fire safety at minimum cost

No occupied building is "fireproof", because each contains its share of fire risk in the form of materials and processes inside the building. The ease of ignition and rate of burning, of these building contents determine the "degree of risk" and extent of protection required. The basic role of the building is to resist heat collapse and thus safe[illegible] exit during the evacuation period and subsequently permit effective extinguish[illegible]

The fuel [illegible] by the surface charring of wood is far outweighed by its [illegible] only a fraction of the heat produced by the normal contents [illegible]

building However, where conditions of use require relatively non-combustible building materials, fire retardant pressure treatments are highly effective. Several commercially available pressure treatments have been tested by the Underwriters' Laboratories of Canada and are listed in their publications.

Damage control in the first instance begins with the contents, which are much more susceptible to ignition and fire spread than the structure. In fact, timber buildings usually require severe fire exposure from the contents to maintain combustion. When the contents of a building are on fire, a "furnace effect" is produced with the intense interior heat attacking the structure.

At this point the most important fire protection factors are prompt detection and extinguishing operations The speed and effectiveness with which a fire is discovered and extinguished will govern the extent of damage to both contents and structure.

Building materials can neither detect nor extinguish a fire but they can play a significant role in facilitating evacuation and fire fighting operations. In planning fire safe buildings, structural endurance, exits, detection and extinguishing facilities must be integrated for balanced protection.

Fire Safe Design

Maximum protection of the occupants and the property can be achieved by taking advantage of the fire-endurance properties of wood and by close attention to specific design details.

These include:

1. Adequate and accessible exits for evacuation of the occupants and entry by the fire department.
2. Enclosing stairwells, elevator shafts and all other openings to upper storeys.
3. Eliminating or protecting openings in partitions, walls and other horizontal dividers.
4. Isolating hazardous processes by use of detached buildings or fire resistive barriers.
5. Sub-dividing buildings as much as possible to minimize the value in any one area.
6. Installing roof venting equipment and suspended fire curtains from the roof, where division walls would interfere with essential production facilities.
7. Firestopping concealed spaces and suspended ceilings.
8. Providing windows or access panels in basement walls to permit fire department entry and to facilitate venting smoke and gases.
9. Installing automatic alarms and/or sprinklers, where the degree of risk and values warrant special protection.

REFERENCE PUBLICATIONS

The following references are generally helpful in the study of timber engineering. The reader should be cautioned, however, that detailed data in some of these publications are not in agreement with recognized Canadian practice.

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Wood-Frame House Construction, by Anderson and Heyer U S Dept. of Agriculture Handbook No. 73. Superintendent of Documents, Washington, D.C.: 1955.

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Piling

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Fire in Buildings, by Bird and Docking. A. & C. Black (London): 1949.

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Fir Plywood Guide. Plywood Manufacturers Association of B.C., Vancouver.

SOURCES OF INFORMATION

Forest Products Laboratories of Canada, Ottawa and Vancouver (Miscellaneous publications),

U S Forest Products Laboratory, Madison, Wisconsin (Miscellaneous publications),

Forest Products Research Society, Madison, Wisconsin (Publishes Journal, monthly),

American Wood Preservers Association, New York (Publishes Proceedings, annually).

PART V

REFERENCE DATA

This part contains miscellaneous data which may be needed for occasional or frequent reference, assembled here for convenience.

1

STRUCTURAL DESIGN

VOL. 3 BEHAVIOR OF MEMBERS AND SYSTEMS

Chapter 19. Structural properties of Engineering Materials.

19.4 TIMBER

Wood has been used in structures for thousands of years with an excellent record of performance and durability. Timber is readily available in most parts of the world in a variety of species. It has a relatively high ratio of strength to weight, it is efficient in resisting both tensile and compressive stresses, and it can absorb considerable energy prior to fracture. It is less sensitive to fatigue problems than metals. Timber is quite resistant to deterioration, and, when used in large solid members, it can withstand instense temperatures. Modern glue-laminating techniques makes it possible to fabricate members of nearly any shape and size. Timber is also available in sheet form (plywood). Finally, it has a natural esthetic appeal and is often utilized in an internally exposed condition for roof structures.

Some of the shortcomings of timber include its low elastic modulus, its susceptibility to volume changes (swelling and cracking), and deterioration under certain environmental conditions, and its variability in mechanical properties, not only from species to species but also within a given species. Reliable grading techniques for timber are still evolving. Fortunately, the strength variability effect is minimized in glued-laminated timber because it is made up of many small pieces of high-quality lumber. Connections to transfer tensile or shearing loads are difficult to make and are usually expensive. Finally, specific grades of timber used in a set of design calculations may not be readily available at the time of construction, thereby necessitating design changes.

Wood is a cellular material composed of cellulose, lignin, and small quantities of other materials. The cell walls are made of cellulose, stiffened and bound together by lignin. The cellular structure produces anisotropy; that is, a dependence of mechanical properties on direction. Anyone working with wood soon notices that its stiffness and strength along the direction of the grain are considerably higher than perpendicular to the grain. We shall explore this complex behavior both qualitatively and quantitatively in this section.

Wood contains large amounts of water in its natural state and normally is partially dried before use in a structure. As it dries below the *fiber saturation point* (25 to 30% moisture content), it increases in both strength and stiffness. Most dried timber has a moisture content of less than about 20%. If it is to be glued, timber should be dried to about 12% moisture content. Timber changes dimensions with changing moisture conditions, and the degree of dimensional change is a function of grain direction. Longitudinal shrinkage is negligible in most species, and the tangential shrinkage is about twice the radial shrinkage (directions are referred to the annual growth rings of the tree).

NOTA: No se incluyen las figuras mencionadas en el texto.

2

The two classes of trees are *hardwood* (or deciduous) and *softwood* (or coniferous). We shall restrict our attention to the softwoods since they encompass nearly all structural timber. The term "soft wood" does not necessarily mean that the wood is softer than a hard wood; also, soft woods are often stronger than hard woods. Douglas fir and Southern pine are among the most widely used species and have similar properties. Other species that are becoming more important in construction because of the scarcity of Douglas fir include hemlock, spruce, larch, the various pines (Ponderosa, white, sugar, and Norway), and the other firs (balsam and white).

Mechanical Properties

We wish to explore a number of important factors that influence the mechanical properties of timber and the response of timber to load, including:

(a) Density variations and inherent variability within a given species.
(b) Moisture content.
(c) Direction of loading with respect to the direction of the grain.
(d) Types of stress (tension, compression, or shear).
(e) Duration of loading.

Prior to presenting detailed discussion on the properties of one popular species – Southern pine – we should think about how timber responds to stress and how it fails when loaded to its ultimate capacity.

The internal structure of wood is composed of a large number of tiny fibers oriented with their length parallel to the vertical axis of the tree. The severe directionality of fibers is immediately evident in the grain patterns in a section of timber. Each fiber can be considered as a very small hollow column with lateral support provided by the connected adjacent fibers.

It is instructive to idealize a piece of wood loaded in either uniaxial tension or compression as a handful of drinking straws bound loosely together with rubber bands and loaded in the same manner. The straws have little strength and stiffness when compressed perpendicular to their long axis, but they show considerable strength and stiffness in axial tension or compression. Failure of the straws in compression parallel to their long direction is by buckling of the individual straws. Wood behaves in much the same fashion. It fails by crushing at low stress levels when compressed perpendicular to the grain, by tearing of the fibers at high parallel-to-the-grain tensile stress, and by successive buckling of the fibers when loaded in compression parallel to the grain. The parallel compression strength is about 5 to 10 times the perpendicular compression strength, while the modulus of elasticity may differ by as much as 50 to 1 for the two directions.

Fig 19.17

The parallel compression failure of wood (see Figure 19.17) has been mathematically modeled by Pincus [1967], who showed that the proportional limit of timber can be predicted by an analysis of long tubular columns embedded in an elastic medium. This analysis lends further strength to the concept of wood fibers acting as miniature Euler columns.

The behavior of wood loaded perpendicular to the grain is characterized by an increase in stiffness after the cavities of the fibers are compressed; this occurs when the thickness is reduced to about ⅓ the original value. The capacity of this dense, compressed material is very high, but it normally cannot be utilized in design because the corresponding deformations are too high. Wood loaded in side compression is used in bearing shims and in grillages where its compressibility and eventual increasing stiffness permits appropriate load sharing and redistribution.

Timber loaded to failure in bending (Figure 19.18a) exhibits the typical load-deflection behavior shown in Figure 19.18b. The initial elastic response is followed by inelastic behavior with appreciable strain prior to failure. At the proportional limit the extreme fibers on the compression side of the beam begin to buckle, throwing more load on their adjacent interior fibers. The buckled fibers continue to carry a stress near their buckling stress because the adjacent fibers provide some support and restrain against failure. This process spreads toward the neutral axis as the bending moment is increased, and the stress distribution just before failure is as illustrated in Figure 19.18c. We see that the compressive stresses are nonlinear, the neutral axis is slightly below mid-depth, and the maximum tensile stress is about 50% larger than the peak compression. The first visible distress in the beam is often tearing and failure of tensile fibers, even though the failure really starts with overstressing in compression.

Deep beams tend to have lower ultimate bending stresses than shallow beams because the flatter strain gradient and the increased depth of the deeper beams results in less-effective restraint of the critically stressed compressive fibers by adjacent fibers. This phenomenon leads to the so-called "depth effect" that must be accounted for in designing beams (Section 29.1)

The various mechanical properties of wood are determined from tests (ASTM D143) performed on small specimens that are free of defects such as knots and cracks. The resulting values are therefore much higher than the allowable stresses that we use for members containing defects.

The directional properties of shortleaf Southern yellow pine and their strong dependence on moisture content are illustrated in Figures 19.19 and 19.20, where eight different stress conditions are portrayed. Two different stress values are given for some conditions; f'_{prop} is the proportional limit stress while f'_u is the stress at failure of the specimen. We are most interested in values for dry wood (Figure 19.20) because structural timber in thicknesses up to four in. is usually kiln dried prior to usage and then kept dry in the finished structure. The strong dependence of properties on density of the wood fiber is presented in Chapter 29.

Several observations are in order here:

(a) The strengths in longitudinal compression (case 1) and in bending (case 7) are at least 50% greater than the corresponding proportional limits

(b) The bending test proportional limit and strength values (case 7) for the small clear specimens are higher than the longitudinal compression values (case 1). This is because clear wood is much stronger in uniaxial tension than in compression. Members with defects (such as knots) on the tension side may fail suddenly in tension and show a substantially reduced modulus of rupture.

(c) Strength values in large sawn timbers would normally be substantially less than the values given in Figures 19.19 and 19.20 for the small clear specimens. A major reason for the reduced strengths is the presence of natural defects, such as knots, sloping grain, splits, and shakes (cracks) and variations in soundness of the wood through the thickness of the member, as well as the depth effect described earlier.

19.19
19.20

(d) Timber is very weak in tension perpendicular to the grain (case 6) and relatively weak in perpendicular compression (cases 2 to 4) and in horizontal shear (case 8).

(e) The perpendicular compression test results (cases 2 to 4) are influenced by the specimen configuration; the fibers in the 2 in. long unloaded end regions give support to the central loaded fibers and raise the proportional limit stress about 50% above that for a 2 in. cube loaded in uniform perpendicular compression. The strength value also depends upon the direction of the growth rings with respect to the compression. In softwoods, the minimum perpendicular compression strength occurs when the loading direction is at an angle of about 45° with the annual growth rings (case 4).

(f) The modulus of elasticity E_L for longitudinal compression (case 1) is about 10% higher than the bending modulus (case 7). Approximate values of E for perpendicular compression loading are $E_R = 0.10E_L$ for loading radial to the growth rings (case 2) and $E_T = 0.05E_L$ for loading tangential to the growth rings (case 3).

(g) Poisson's ratio is also highly directional, with approximate values of $\nu_{TL} = 0.025$, $\nu_{RL} = 0.04$, $\nu_{LT} = 0.5$, $\nu_{LR} = 0.35$, $\nu_{RT} = 0.5$, and $\nu_{TR} = 0.3$ for a softwood such as Southern pine. The first subscript is the direction of stress (T = tangential, L = longitudinal, and R = radial, all with respect to the grain direction), and the second is the direction of deformation. Longitudinal deformation is largely independent of tangential and radial perpendicular compression

(h) The shearing modulus G for timber is highly directional and must be associated with the appropriate shearing stresses and strains in the three planes defined by longitudinal, radial, and tangential directions (as in stress cases 1, 2, and 3 for compression). A mean value of $G = E_L/16$ may be used in computing torsional and shearing deformations of timber members.

(i) The compressive strength of timber surfaces loaded at angles other than 0° or 90° to the grain is needed in truss joints and other types of structures with inclined members. The *Hankinson formula* is a commonly used empirical relationship that provides a continuous transition from longitudinal compression to perpendicular compression:

move down to level of eqn.

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta} \qquad (19.4)$$

where N is the strength on the inclined surface, Q is the strength perpendicular to the grain, P is the strength parallel to the grain, and θ is the angle between the grain direction and the load.

(j) The sharp influence of moisture content on timber strength is evident from a comparison of Figures 19.19 and 19.20. Reducing the water content from the green condition (81%) to a dry state (12%) nearly doubles the proportional limit and strength values in both compression and bending. Further drying will continue to increase strength. We should note, however, that dry timber may occasionally have less shock resistance (energy absorption capacity) than green wood because the latter will deflect much more prior to fracture. Empirical expressions exist (Gurfinkel [1973]) for converting strength data measured at a certain moisture content to values at different moisture levels.

The transformation of the test results of Figures 19.19 and 19.20 into values suitable for allowable stresses for design purposes will be discussed in Chapter 29.

The variability of timber density for Southern pine is illustrated in Figure 19.21*a*, where frequency distributions are given for small clear specimens and for lumber. The mean value for both distributions is about 0.52. Both stiffness and strength rise with increasing density; in fact, the modulus of rupture R of kiln-dried Southern pine has been determined to be a linear function of specific gravity G, with $R = 20.3G + 2.5$ for clear specimens and $R = 21.4G - 4.7$ for lumber, where R is in ksi. A frequency distribution of modulus of rupture values measured on green Douglas fir specimens cut from a single tree is given in Figure 19.21*b*. Southern pine would yield essentially identical results. It is evident that variation in density is largely responsible for in-species variation in mechanical properties of small clear specimens. This variation and the influence of defects must be taken into account in establishing values of allowable design stresses. The problem is handled by grading the lumber as it comes from the sawmill.

Timber shows pronounced sensitivity to duration of loading, particularly in its resistance to bending loads. For example, the maximum load that can be carried for one month is on the order of 70% of the load that can be carried for one minute. Test results by Wood [1951] on Douglas fir beams are shown in Figure 19.22. The reasons for this behavior are not fully understood, but the time-dependent inelastic deformations of the cellulose component, which also leads to creep deformations in timber structures, must be an important factor.

Timber has high durability when kept in a continuously submerged condition. Deterioration can be severe, however, under alternating wet and dry conditions or at a transition zone such as near the waterline in a marine pile. Preservative chemical treatments are used widely to make wood resistant to fungi (decay), insects, and marine borers. Creosote solutions, oil-borne chemicals, and water-borne inorganic salts are forced into the timber by pressure to produce a highly resistant product known as *treated timber*. Properly treated timber will last nearly indefinitely under adverse conditions.

19.5 SUMMARY

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We have seen that from the standpoint of the structural engineer, the most important material properties are E, f_y, f_u', failure criteria for combined stress states, ductility, and the various time and environment dependent phenomena such as creep, strain rate, ~~length~~ duration of loading, and moisture level. For a given structural geometry, the structural deformations and the deflections in the elastic range of behavior are a function of E, the modulus of elasticity. The modulus also controls a structure's buckling capacity. The ability of a structure to reach a certain value of overload is normally a function of the yield strength f_y, the ultimate strength f'_u, and failure criteria for combined stresses while the performance and safety of the structure after it achieves the overload, or under reversing heavy loads, depends primarily on the material ductility and its ability to absorb energy. Both deformations and stress distributions are influenced by time of loading and other environmental factors whenever we use concrete, timber, or plastic in a structure.

A structure can be no better than the material from which it is built. Thus it is imperative that we have an intimate knowledge of the response of materials to load.

CHAPTER 29

Behavior of Timber Structures

29

Behavior of Timber Structures

Timber is one of the most versatile of all natural building materials. It is efficient in carrying tensile, compressive, and bending stresses, and it has a high strength/weight ratio. It can be used in a wide variety of structural forms (see Figure 29.1 and Figures 2.4, 4.13, 4.26, and 6.17 of Volume 1) to produce rather unique architectural effects. The structural properties of timber are presented in Section 19.4. In this chapter we shall consider the major aspects of behavior of timber members and their connections.

Timber is used in many shapes and sizes ranging from thin plywood to beams several feet deep. Ordinary solid-sawn members are available in standard sizes such as 2 by 4s, 4 by 8s, and up to as large as about 12 in. by 14 in. The dimensions given for sawn timbers are nominal; that is, the member is typically 1/2 in. smaller in actual dimensions after it is surfaced (planed). A 10 in. by 12 in. timber that has been surfaced on all four sides measures 9-1/2 in. by 11-1/2 in. The actual dimensions must be used in all calculations.

Many of the fundamental behavior and design concepts utilized for steel and concrete structures are useful in designing timber structures. The major new aspect is the strong directionality (anisotropy) of timber. Its strength properties vary considerably depending upon the angle between the loading and the grain, and whether the stress is compressive, tensile, or shearing. We shall assume that Section 19.4 on basic mechanical properties has been studied prior to studying this chapter.

The behavior of connections requires special attention. As we shall see in Section 29.5, connections for timber are not amenable to ordinary linear elastic stress analysis. Instead, we must develop an understanding of the complex behavior and possible failure modes of the various forms of connections and rely on extensive test data for establishing design information.

The establishment of appropriate factors of safety in timber structures is difficult because of the inherent variability in strength properties as well as the dependence of strength on such environmental factors as duration of loading and moisture content. The natural variability in timber is partially accounted for by rather elaborate grading rules that are applied during the production of "stress graded" lumber. The grading is done on the basis of density, straightness of grain, severity and location of defects, and other factors that affect basic strength properties. Grading rules are established and enforced by a number of commercial organizations such as the Southern Pine Inspection Bureau (SPIB).

29.1 ALLOWABLE STRESSES

Since timber structures are customarily designed on a working stress basis rather than on a strength basis, we face the problem of deciding what the allowable stresses should be for a given species and grade. It is not sufficient to say that a structure is built of coast region Douglas fir, instead we must define the quality of fir used, be it construction grade, dense construction grade, select structural, dense select structural, or some other grade. Furthermore, within each of these grades we may have a further subclassification of individual members, such as J&P (joist and plank), B&S (beam and stringer), and P&T (post and timber). This latter classification is used because some members have defects that are permissible in compression members (P&T) but are unsatisfactory in bending members (B&S).

The strength properties reported in Section 19.4 are for small clear specimens, relatively free of any defects, tested in short time loading in a dry condition. The allowable stresses in structures made of large, imperfect timber elements that are to be loaded for decades must be but a fraction of the ideal strength properties given in Chapter 19. We define reduction coefficients C_t, C_v, and C_d to account for duration of loading, variability of clear wood strength, and naturally occurring defects, respectively. The following consideration of these factors is adapted from Wood [1960]. It typifies the rationale most often used in deriving working stresses.

The effect of duration of loading is given in Figure 19.19. We observe that the long-time strength of timber (e.g., 10 years loading duration) is approximately half the short time strength, or $C_t = 1/2$. The second factor, variability of clear wood specimen strength, is illustrated in Figure 19.18*b* for Douglas fir. The mean value for modulus of rupture for this particular tree is 7250 psi. About 95% of the values exceed 75% of the mean value, or 5440 psi. Thus we might be willing to accept 5440 psi as a near-minimum value of clear specimen bending strength. If so, the appropriate reduction factor is $C_v = 3/4$.

Nearly all timber members used in structures contain some natural defects. The grading rules are based on the maximum severity of defect permissible within each grade. The severity of defect may be defined by the fractional reduction in strength it produces, with a 3/8 maximum reduction being typical. This means that a member with 5/8 the strength of a top-of-the-line specimen would be acceptable within a given grading classification, or $C_d = 5/8$.

Applying the three reduction factors to the mean strength of the material of Figure 19 18*b*, the expected value of the near minimum strength under long-time loads is

$$F = C_t C_r C_d (7250) = \frac{1}{2}\left(\frac{3}{4}\right)\left(\frac{5}{8}\right)(7250) = 1700 \quad \text{psi}$$

It is essential to realize that the reduced stress of 1700 psi still does not contain a real factor of safety. In fact, if we took a large sample of beams that had just barely met the grading minimum rules, about 5% of the sample should have a long time strength less than 1700 psi since this fraction has a short-time strength less than the 5440 psi associated with $C_r = 3/4$. Most of the specimens would be substantially stronger than this, however, because they would be closer to or above the mean strength and have less critical defects.

A basic allowable bending stress of 1500 psi might be reasonable for this timber, in which case the near-minimum factor of safety would be 1700/1500 = 1.13, and the *mean factor of safety* would be about

$$F.S. = \frac{C_t C_d (7250)}{1500} = \frac{1}{2}\left(\frac{4}{5}\right)\frac{(7250)}{1500} = 1.93$$

if we assume that the average reduction C_d due to defects is 4/5.

This rather simple approach to safety has been augmented by probability-based studies, such as the one done by Wood [1960]. Given the large number of variables influencing timber strength, and the fact that at least some of them have rather well-defined frequency distributions, it seems only a matter of time before probabilistic concepts will play a major role in helping to establish more rational ways of handling safety in timber structures.

Typical allowable stress levels for a dry medium grade Southern pine based on a 10-year load duration, variability as discussed above, and defects for the grade are:

Bending and tension parallel to grain	1750 psi
Compression parallel to grain	1450 psi
Compression perpendicular to grain	540 psi
Horizontal shear	100 psi

← style of line

These allowable stress levels should be compared with the proportional limit and ultimate strength values given in Figure 19.17.

Allowable stresses are customarily adjusted for load duration in accordance with Figure 19.19. Thus durations shorter than 10 years result in increases from 15% for a two-month loading (snow) to 100% for impact loading Finally, the allowable stress levels must be decreased if the timber does not remain dry.

There are two major classifications of timber members used to resist bending moment, solid sawn and glued laminated. The *solid sawn* beams are similar to the familiar 2 by 10s, 4 by 8s, and other dimensioned lumber available at lumber yards. A major disadvantage of solid-sawn sections is a rather low limit on size and length. This problem is overcome in the second type of flexural member—*glued-laminated structural timber*. A typical glued-laminated member is made up of many small boards, 3/4 in. to 1-1/2 in. in thickness, glued together with a strong adhesive (see Figure 29.2*a*). The process of glue laminating was perfected in the 1940s and has provided the timber industry with a new dimension for designing larger and better timber structures. Straight and curved members of almost unlimited size may be fabricated for long-span girders, arches, and rigid frames. Several types of glued-laminated members are illustrated in Figure 29.2. Laminated members are also shown in the Volume 1 figures listed at the beginning of this chapter.

The main advantage of glued-laminated timber is that it is made up of standard commercial sizes of thin boards. Small trees can be transformed into large structural elements by this process. The individual boards can be arranged to have the highest quality lumber in the regions of greatest stress, thereby permitting higher allowable stresses in glued-laminated lumber than in solid lumber. They can be easily bent in the fabrication process to form curved arches and frames (see photo at front of Chapter and Figure 29.2). In addition, camber may be controlled precisely and tapered sections are readily made. Striking architectural effects are possible with exposed glued-laminated members. Finally, a large glued-laminated member is highly resistant to fire because deterioration is minimal after the initial surface charring takes place. The main disadvantages of this product are associated with the special equipment and skills needed in making high quality members. Size and length limitations are often imposed by transportation difficulties; this may lead to rather large and expensive connections in the field.

In addition to solid-sawn and glued-laminated flexural members, we often use plywood and other products such as particle board to carry bending loads in floors, roofs, and walls. The design of these elements is normally not controlled by stresses but rather by stiffness; for example, 3/8 in. plywood might be strong enough for a house floor, but it would be too bouncy for the occupants. Plywood is also utilized as web material in built-up timber girders and as a primary components of stressed-skin panels, and as diaphragms for resisting in-plane shear. The design of the latter types of structures is treated by Gurfinkel [1973].

The behavior of timber beams loaded to failure was discussed in Section 19.4, where we saw that timber has a load-deformation response that is linear to about half the ultimate capacity in bending (Figure 19.16*b*). The design basis for timber beams is conventional bending theory, with stresses computed from $F = Mc/I$ and $v = VQ/Ib$. These expressions are used even though timber beams do not have their neutral axis of bending precisely at their centroid because of differences in response to tensile and compressive stresses. Also, normal linear behavior may be altered by such factors as long-time loading effects, changing moisture content, and imperfections in the timber. These effects are not severe, especially when considered in comparison with all the other possible variations in the properties of a given piece of timber.

The conventional bending theory expressions are adjusted to account for several special problems that arise in timber flexural members. We shall concentrate our discussion on these problems and illustrate some of them with two examples: a solid-sawn beam and a glued laminated member. Special consideration is needed for the following effects in both solid and glued laminated timber elements:

a. Effect of depth on ultimate bending capacity.
b. Effect of cross-sectional shape on ultimate bending capacity.
c. Horizontal shearing stresses.
d. Compression stresses perpendicular to the grain at supports and at concentrated load points.

Curved glued-laminated members are also subject to additional stresses due to: (a) bending of laminations in the fabrication process and (b) radial ~~to: (a) bending of laminations in the fabrication process and (b) radial~~ stresses, acting at right angles to the normal bending stresses that arise as the curved section is subjected to bending moment. Radial tension is particularly troublesome because of the low strength of timber in tension across the grain.

The effect of beam depth on bending strength was described in Section 19.4. The deeper the beam, the smaller the support given to critically stressed compressive fibers by adjacent, less highly stressed fibers. A typical method for accounting for this effect is to decrease the allowable bending stress by an empirically derived depth factor expressed as a function of beam depth d. Using a 12 in. deep beam for the base strength, the U.S. Forest Products Laboratory has derived an expression for rectangular beams from extensive experimental work:

$$C_d = 0.81\,\frac{d^2 + 143}{d^2 + 88} \leq 1 \qquad (29.1a)$$

C_d also used for defects in prev. pages.

where d is the depth in inches and C_d is a factor less than unity that is used to decrease the allowable stress in beams. More recent work by Bohannan [1966] based on statistical strength theory results in the following expression for the depth factor:

$$C_d = \left(\frac{12}{d}\right)^{1/9} \qquad (29.1b)$$

The allowable moment on a beam is then

$$M_{allow} = C_d F_b S$$

where F_b = normal allowable bending stress and S = section modulus. The depth factor is applied to beams that are more than 12 in. deep. Either expression gives reduction factors in the range of 0.8 to 1.0.

The effect of the support afforded to critically stressed compression fibers also applies to the bending strength of nonrectangular shapes such as those shown in Figure 29.3. In cases (*a*) and (*b*) the width of the beam at the extreme fiber is narrow, there is an abundance of adjacent supporting fibers because the section increases in width with increased distance from the upper fiber. On the other hand, the box beam of Figure 29.3*c* has thin flanges and relatively little beneficial support can be developed. In addition to the fiber support arguments, we can also compare the failure of a timber beam to the yielding of a steel section. There is little reserve strength in an I-section after critical stressing of the flanges (low ratio of plastic section modulus to elastic section modulus in a steel beam), but the solid shapes can transfer an increasing amount of load to the inner fibers. We expect the first beams in Figure 29.3 to be stronger than a rectangular section with the same section modulus, while the third should be weaker. This effect can be expressed in terms of a form factor C_f that accounts for the geometry of a cross section, using the rectangular section as a base.

Dietz [1942] presents a semiempirical evaluation of form factors for different beam shapes. Typical values used in U.S. design practice are $C_f = 1.18$ for a circular section and $C_f = 1.414$ for a square section with bending about one of the diagonal axes. The expression for box and I-beams is more complex and is given by Gurfinkel [1973]. In all cases, the factor is applied as a reduction to the normal allowable bending stress, or

$$\text{allowable } M = C_f F_b S \tag{29.3}$$

Simultaneous use of the depth and form factors gives

$$\text{allowable } M = C_d C_f F_b S \tag{29.4}$$

Shearing stresses in timber members take on added importance because of the relatively low shear strength of timber along the direction of its grain and because of the possibility of horizontal checks and cracks. Considering a rectangular section, we can compute a maximum nominal shear stress

$$v = \frac{3}{2}\frac{V}{A}$$

at the neutral axis of bending. The horizontal components of this shear stress tend to produce translation of the top half of the beam with respect to the bottom half. If horizontal checks and cracks develop along the member, there will be a concentration of horizontal shear stress at the tip of the checks and cracks with the possibility of a shearing failure at a low average shear stress.

Assume that a beam is partially cracked along the mid-depth as shown in Figure 29.4*a*. The cracks do not necessarily extend completely through the thickness of the beam. The resulting behavior is complex; part of the shear is carried by the entire section while some is carried by the upper and lower sections trying to act as independent beams because of the horizontal cracking. This phenomenon is known as *two-beam action.* We should realize that the beam could be completely cracked at mid depth, thus two separate beams, and that the maximum shear stress in each half of the beam would still be the same as that in an uncracked beam provided that each half carried $V/2$.

Partial two-beam action is beneficial because the total shearing force near the beam ends is resisted partially by the beam acting as a unit and partially by the combination of two beams. Experimental and theoretical studies at the Forest Products Laboratory have been used to establish methods for handling the division of shearing load between the two load-carrying mechanisms. These rather elaborate methods, which are needed only for beams with high shearing stresses, are beyond the scope of this book (see Gurfinkel [1973]). The effects of checking are not expected in glued-laminated members that are properly protected from adverse drying conditions, since the lumber is kiln dried before assembly.

Compression stresses perpendicular to the grain at supports and load points must be considered in design because timber has little stiffness and strength in perpendicular compression (see Section 19.5). Typical allowable stresses are about 400 psi for softwoods if the bearing lengths in the direction of the beam is 6 in. or more. For shorter bearing lengths at points some distance (at least several inches) away from the end of the beam (Figure 29.4*b*), a higher bearing stress may be used because the deformed fibers at the point of bearing will develop tension along the grain. This tension component helps resist the transverse load; it is most effective when the bearing length is very short. Some specifications allow an increase in bearing stress of about 75% for a 1/2 in. bearing length, with smaller increases for lengths between 1/2 in. and 6 in.

Curved laminated elements such as arches or the knee section of rigid frames may have significant internal residual stresses because of the bending stress imposed on each lamination as it is forced into the desired curvature. Large external forces are often required to hold a curved member in position until the glue cures (see Figure 29.2*a*). When the forces are released the member will spring back slightly but cannot straighten out because of the restraining shearing stresses set up in the glue lines.

The combination of residual lamination bending stresses and glue line shear stresses will reduce the bending strength of a curved member as compared with a straight member of the same cross section. Experimental data has been used to derive a reduction expression

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$$F_{ct} = F\left[1 - 2000\left(\frac{t}{R}\right)^2\right] \tag{29.5}$$

where F_{ct} is the allowable working stress in the curved laminations, F is the allowable working stress in a straight lamination, and t/R is the ratio of lamination thickness to radius of curvature.

The other problem introduced in curved laminated members is one of radial stresses induced perpendicular to the grain when the member is bent. Curved beam theory, which is beyond the scope of this book, is needed to evaluate these stresses. However, we should be able to visualize that a tensile radial stress will be induced if we try to straighten the member, while bending action that produces more curvature will produce compressive radial stress. The maximum intensity of radial stress occurs at the neutral axis and has the value

$$f_{\text{radial}} = \frac{3M}{2rdb} \tag{29.6}$$

for a rectangular section.

Expressing f_{radial} in terms of the extreme fiber bending stress $f = Mc/I = 6M/bd^2$ we have

$$f_{\text{radial}} = \frac{f}{4}\frac{d}{r} \tag{29.7}$$

Thus a laminated member with a depth of 2 ft and a radius of curvature of 16 ft has a peak radial stress equal to 1/32 times the peak bending stress. A large depth to radius of curvature ratio is needed to produce large radial stresses, but even a relatively low tensile radial stress may be critical because of the low tensile strength of wood perpendicular to the grain.

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Example 29.1 Rectangular Beam Strength. A surfaced 8 in. by 16 in. timber beam is to carry a permanent uniform loading on a 15 ft span (Figure 29.5*a*). For normal allowable stresses of $F_b = 1750$ psi bending and $F_v = 100$ psi shear, determine the total allowable load on the span.

The allowable stresses are decreased by 10% for permanent loading, or $F_b = 0.9(1750) = 1575$ psi and $F_v = 0.9(100) = 90$ psi. A depth factor reduction also applies to the bending stress as the beam is greater than 12 in. deep. This reduction (Equation 29.1*b*) is $C_d = (12/15.5)^{1/9} = 0.97$, and $F_b = 1575(0.97) = 1520$ psi $= 1.52$ ksi.

The bending capacity is $M = F_b S$ where $S =$ section modulus $= bh^2/6$. Since the dimensions of a surfaced timber are 1/2 in. less than the nominal dimensions,

$$S = \frac{7.5(15.5)^2}{6} = 300 \text{ in}^3$$

and

$$M = 1.52(300) = 456 \text{ in.-k} = 38.0 \text{ ft-k}$$

The maximum shear stress in a rectangular section (neglecting two-beam shearing action) is $v = \frac{3}{2}\frac{V}{A}$, or

$$V = \frac{2}{3}Av = \frac{2}{3}(7.5)(15.5)(0.090) = 7.0 \text{ k}$$

From equilibrium, $M_{max} = w_b L^2/8$ and $V_{max} = w_v L/2$. Solving these expressions for w_b and w_v, we get

$$w_b = 1.35 \text{ k/ft} \qquad \text{and} \qquad w_v = 0.933 \text{ k/ft}.$$

The load capacity of the beam is controlled by shear stress. Shear is often critical in timber beams, in contrast to steel beams where shear seldom governs.

Example 29.2 The curved portion of a 36 in. by 10 in. glued laminated (Figure 29.5*b*) has 1 in. laminations and a radius of 12 ft. Determine the allowable wind induced moment on the corner portion if the normal allowable bending stress is 2100 psi and the allowable horizontal shear is 90 psi.

Three factors are applied to the bending stress: depth effect, curving of laminations, and duration of load. From Equation 29.1*b*,

$$C_d = (12/36)^{1/9} = 0.885 \qquad \text{and} \qquad F_b = 0.885(2100) = 1860 \text{ psi}.$$

From Equation 29.5,

$$F_c = F\left[1 - 2000\left(\frac{1}{144}\right)^2\right] = 1860(0.903)$$

d/

$$= 1680 \text{ psi}$$

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This stress level is increased by $33\frac{1}{3}\%$ for the very short duration wind loading, or

$$F_b = 1.333(1680) = 2240 \text{ psi} = 2.24 \text{ ksi}$$

We compute the section modulus $S = \dfrac{10(36)^2}{6} = 2160 \text{ in}^3$, and

$$M = 2.24(2160) = 4850 \text{ in.-k}$$

The allowable stress in radial tension is often taken as 1/3 the allowable horizontal shear stress, or 1/3(90) = 30 psi. There is no increase in allowable radial stress for wind loading because of the severity of the effects of possible tensile failure due to radial stress. The maximum radial stress is at the neutral axis and is equal to

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$$f_{\text{radial}} = \frac{f}{4}\frac{d}{r}$$

or the equivalent allowable bending stress is determined from

$$30 = \frac{F_b}{4}\frac{12}{3}\;;\; F_b = 480 \text{ psi} = 0.48 \text{ ksi}$$

which is only a fraction of the allowable bending stress of 2.24 ksi as determined from the regular bending analysis. This example is a rather extreme case in that the radius of curvature is small for the depth of the member, and the bending action is computed for wind stresses; both of these factors tend to make the radial tension much more serious than the regular bending. The proper method of treating this situation is to reinforce the laminated member with steel rods that can absorb the radial tension forces. Some change in geometry, such as a more gentle curvature with larger radius, would also help the situation.

29.3 COMPRESSION MEMBERS

Compression members of timber occur commonly as small building columns, supporting members in industrial structures, and truss members. The simplest timber compressive form is the solid rectangle, made of sawn or glued-laminated timber. When the length is excessive for this form, the space column, made of two or more column sections separated by spacers, is used. The spaced column has an increased radius of gyration for a given cross-sectional area. These two column types are illustrated in Figure 29.6a. Special shapes for architectural purposes can be fabricated in glued-laminated timber.

The most important behavior topic for wood columns is stability. The basic stability theory presented in Chapter 20 may be used to predict the buckling load of long columns. The Euler buckling equation,

$$f'_{cr} = \frac{\pi^2 E}{(kL/r)^2} \tag{29.8}$$

is usually modified for rectangular timber sections by replacing L/r with L/d, where d is the dimension of the column normal to the axis of buckling. We can easily show that the radius of gyration of a rectangular section is given by $r = d/\sqrt{12}$. Replacing r by $d/\sqrt{12}$ in Equation 29.8, the expression for Euler buckling is

$$f'_{cr} = \frac{\pi^2 E}{[kL/(d\sqrt{12})]^2} = \frac{0.82E}{(kL/d)^2} \tag{29.9}$$

Long columns tested at the Forest Products Laboratory (Newlin and Gahagan [1930]) produced results in excellent agreement with Equation 29.9. Sections 11½ in. by 11½ in., 24 ft long, gave ratios of (test/theory) 0.979 for dry Southern yellow pine and 0.964 for dry Douglas fir. The value of E in Equation 29.9 is customarily determined for each column specimen by a bending test prior to the axial loading.

Shorter columns, 2 ft and 12 ft long, were also tested by Newlin and Gahagan. The 2 ft long dry pine specimens failed by crushing and local buckling of the internal fibers, as described in Section 19.4, at stresses of around 5.6 ksi. The 12 ft long columns exhibited both instability and crushing at failure stresses of about 5.2 ksi. The data indicate that, for column lengths up to about 12 times the least dimension of the column, there is little reduction of strength below the strength of very short specimens.

A more recent study by Parker [1964] on a short and intermediate length timber columns verified the applicability of the tangent modulus theory (Equation 20.14) for predicting buckling loads at stress levels above the proportional limit. Dry Southern pine and Douglas fir columns 3.6 in. square and with lengths of 18, 36, 54, and 72 in. were loaded concentrically. The results of the tests on the Southern pine columns are summarized in Figure 29.6b. The average values of test results at each L/d ratio produce the type of curve we would expect (compare with Figure 20.12).

Timber columns with purely axial load may be designed using the stress versus L/d relationship shown in Figure 29.7, where the Euler stress divided by a factor of safety is used for elastic buckling and a constant allowable stress (f_{parallel}) is used for inelastic buckling. A more refined approach is not sought here because the inherent variabilities in column geometry and strength are greater than the differences between the tangent modulus theory and the simplified approach.

F_c

Example 29.3 At this point in your study of stability you should have some "feeling" as to whether a given steel or concrete column is to be classed as short (inelastic action) or long (elastic buckling). With this example we shall provide insight into typical proportions for short and long timber columns.

Typical allowable compression stress parallel to the grain, denoted by F_C, ranges from 400 to 2150 psi for normal load durations with construction grade and better ranging from about 900 to 2150. The modulus of elasticity values for these better grades of timber vary from about 1,200,000 psi to 2,000,000 psi. We shall now establish a definition of short and long columns from these given timber properties.

?

Using a factor of safety of 2.75 against elastic buckling, Equation 29.9 is transformed into an allowable stress expression:

$$F_a = \frac{0.82E}{2.74(kL/d)^2} = \frac{0.3E}{(kL/d)^2}$$

This expression is plotted in Figure 29.8 for the upper and lower values of E (1,200,000 and 2,000,000 psi) along with the upper limits of stress ranging from 900 to 2150 psi. We see that elastic buckling governs for (kL/d) ratios above 13 to 20 for the lower E value, and for ratios above 16.7 to 28.5 for the higher value of E. Since high strength and high E tend to go together, the true curve where the correct E is associated with each allowable compressive stress F_C is represented approximately by the dashed line of Figure 29.8, and we see that elastic Euler buckling occurs for kL/d ratios in the range of about 16 to 20. For a 6 by 6 in. column, the transition range is 8 to 10 ft.

Accounting for tangent modulus effects would increase the transition point slightly.

The axial-load capacities of spaced columns may be determined by applying the principles discussed above. The only new aspects are the determination of appropriate effective lengths for the individual elements in a spaced column and the connection requirements for the spacer blocks. These refinements are discussed in Volume 4.

29.4 COMBINED BENDING AND AXIAL LOAD

Timber members in a frame, truss, or other structures may be subjected to various combinations of bending and axial force (either tensile or compressive). Combined bending and compression is discussed in Section 20.2; the nonlinear beam-column behavior presented there is fully applicable to timber columns

Relatively little experimental work has been conducted on timber beam-columns. One study performed in the Structural Models Laboratory at Cornell University on 1 in. square by 7 in. long model columns made of clear white pine is summarized in Figure 29.9a, where the axial compressive failure load P is plotted against the failure moment Pe for nine different eccentricities ranging from 0 to infinity. Two specimens were tested at each eccentricity; the value of e for each specimen includes the amplification effect. As with reinforced concrete beam-columns, the failure modes are compressive for low M/P ratios and tensile for high M/P. The transition from one type of failure behavior to the other is indicated in the figure.

A nondimensionalized version of this interaction diagram is shown in Figure 29.9b. A conservative representation of the actual interaction curve is a straight line connecting P/P_u and M/M_u, where P_u is the pure column capacity of the section, and M_u is the pure bending capacity. Values of combined loading falling below this straight line may be expressed in terms of stresses instead of forces and moments:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad (29.10)$$

F_a = allowable stress in longitudinal compression
$f_b = M/S$ = bending stress
F_b = allowable bending stress

This method of treating combined stresses essentially assigns part of the strength of the section to column action and the remainder to beam action. Equation 29.10 may be used for the preliminary design of timber beam-columns as well as for cases of combined bending and tension by setting F_a = allowable stress in tension.

A more accurate estimate of beam-column behavior must include the amplification effects that are present in any beam-column. The National Design Specification for Stress-Grade Lumber and its Fastenings [1971] provides an expression for rectangular members subjected to combined eccentric end loads and side loads:

$$\frac{f_a}{F_a} + \frac{\dfrac{M}{S} + f_a(6e/d)\beta}{F_b - \gamma f_a} \leq 1.0 \qquad (29.11)$$

where M = moment induced by side loads, e = eccentricity of the axial load, and β, γ = amplification factors that depend on the column slenderness ratio, L/b. For short columns, $\beta = 1$ and $\gamma = 0$; for long columns, $\beta = 1.25$ and $\gamma = 1$. The breakpoint between short and long columns is defined by $L/B = (0.3E/\text{allowable stress parallel to grain})^{0.5}$.

29.5 CONNECTIONS

The provision of connections has always been the most difficult problem met in timber structures. Overall economy is drastically affected by connection details. Substantial improvements in connectors and in understanding their behavior have evolved over the past several decades, and have contributed greatly to increased usage of timber as a major structural material. Connections cannot be left to the judgment of the field crews; instead, the connections for any timber structure other than residential or similar light construction are carefully designed by qualified engineers. As in steel and concrete structures, we must not forget that the performance and strength of an assemblage of structural elements can be no better than its connections.

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An initial exposure to timber connector design is often bewildering because we utilize a large number of different types of connectors, each of which will have different behavior depending upon the orientation of the load with the connector and the connector with the grain of the wood. In addition, the strength of any timber connector defies a rational, mathematical analysis.

Connectors include glue, nails, spikes, bolts, lag screws, pins, metal plates, and special patented timber connectors called split rings, shear plates, toothed rings, spike grids, and clamping plates. Each of the latter group of devices is used in conjunction with a bolt and is designed to spread the load transfer over a substantial area of timber, thus reducing the severe concentrated bearing stresses experienced around a bolt acting in simple shear. The split ring and shear plate connectors are illustrated in Figure 29.10. More detailed information on connection devices and on connection design is given by Gurfinkel [1973]. We shall restrict our attention here to bolts and split-ring connectors.

Bolted Connections

Bolted connections are popular because of their easy installation and relatively large load capacities. The main variables that influence the capacity of a bolted connection designed to transmit force from one timber member to another include: species of wood, grade of lumber, moisture condition of lumber, geometry of joint, and edge and end distances of bolts.

Given this number of variables, it is not surprising that bolted connection design is accomplished by resorting to tabulated values of bolt capacities. We should not use such tables, however, unless we have a sound understanding of how typical bolted connections actually behave when subjected to load.

We shall consider first the testing of a single 1/2 in. diameter bolt used to fasten two 2 in.-thick Southern pine side members to a thicker central member (Figure 29.11). The load is applied axially to the assemblage by pushing down on the outside members and up on the center member. Since the bolt hole is normally made slightly larger than the bolt diameter (either 1/32 or 1/16 in. larger), some slip occurs as the load is applied. After the bolt is in firm contact with the sides of the hole, the load-slip relationship is linear up to a load of about 2100 lbs. and a slip of 0.03 inches (Figure 29.11). Additional load produces some crushing action in the timber fibers adjacent to the interfaces of the three members. The load-slip curve continues to become more nonlinear as the timber crushes and the bolt bends. The maximum load is 9700 lbs at a slip of about 3/4 in. The condition of the timber members is illustrated at a "safe load" and at the maximum load in Figure 29.11.

A qualitative assessment of the effects of most of the variables mentioned above can be made on the basis of intuition. The stronger, denser woods carry higher loads, as will connections with larger bolts. The strength of the bolt may not affect the proportional limit of the connection unless the bolt is so weak that it begins to bend prior to crushing of the wood fibers; the longer the bolt remains straight the more uniform is the distribution of bearing stresses on the timber. A connection with thin side members or a thin central member fails at an earlier stage—there is a balanced condition of side member thickness and center member thickness at which all three members reach a critical state almost simultaneously.

If the bolt is placed too close to the end of a member we may find a completely different mode of failure in which the wood is split because of the wedging action of the bolt. Using 2 or 4 bolts will not necessarily double or quadruple the strength of the connection because there is no guarantee that each bolt will carry the same load, also, there may be some deleterious interaction of deformations if the bolts are too close together. Finally, if the connection geometry is changed from that in Figure 29.11 to one with a single member on each side of the joint, strength will be substantially reduced because of the reduction to a single shearing plane, the eccentricity of the forces in the members, the large amount of bolt bending action, and the early fiber crushing.

The second type of bolted connection to be considered is a four-bolt configuration with (a) two side members parallel to the center member, and (b) two side members perpendicular to the center member, as shown in Figure 29.12. The experimental results are taken from a study on Douglas fir members conducted by Doyle and Scholten [1963] The investigation included tests on one-bolt joints to assess the relative efficiency of the four-bolt connections. Specimen geometry and loading is given in Figure 29.12. Other variables studied included stage of seasoning of the lumber, bolt diameter (1/2, 3/4, and 1 in.), and snugness of bolt fit.

Four load-slip curves are shown in Figure 29.13*a*; loading is parallel to the grain in all members. The three solid curves give the average response for each bolt in four-bolt connections made with 1/2, 3/4, and 1 in. diameter bolts. The dashed line shows the load-slip response of a connection made with a single 3/4 in bolt. In all specimens the inelastic behavior included crushing of the wood adjacent to the bolts. There was some bolt bending and splitting of the wood along the bolt hole lines.

It is apparent that each bolt in the four-bolt connection behaved essentially like a single bolted connection. It is also apparent that changing from 1/2 in. bolts to 3/4 in. bolts provides a substantial increase in connection strength and stiffness, but that a further increase in bolt size above 3/4 in. does not increase either the ultimate capacity or the initial slope of the load-slip relation. This is because the connection behavior is controlled almost completely by deformation of the wood for all bolt sizes above a certain critical size The concept of matching connector (bolt) strength to the strength of the connected pieces is just as valid in timber connections as it is in metal structure connections, where we should not use one large bolt to connect two thin sheets of metal.

Bolt length is usually accounted for by considering the nondimensional ratio L/d, where L is the bearing length and d is the diameter of the bolt. An increase in L/d normally leads to a lower proportional limit for the connection because longer bolts bend easier, thus producing earlier nonuniform bearing stresses at the bolt-timber interface.

Figure 29.13*b* gives the response of the second type of connection, in which the bolt loads are perpendicular to the grain of the center member. The low perpendicular to grain bearing capacity of the wood under the bolts in the center member leads to relatively low values of proportional limit and load capacity in the connection. Furthermore, the bolt loads produce a splitting effect on the center member that must be resisted by the tensile capacity perpendicular to the grain. The low value of tensile capacity can lead to abrupt failure of this type of connection.

The final aspect of behavior to be considered is that of load distribution in a multiple bolt tension joint, where there are a number of bolts in line, parallel to the axis of loading. As in steel structures (Chapter 21), the outer bolts tend to carry a higher share of the load than the inner bolts. Cramer [1968] showed that the two outermost bolts on each side of a long connection carried about half the total load whenever 6 or more bolts were placed in line, and that there was relatively little redistribution of bolt loads after the proportional limit of the outer bolts had been exceeded. In design we account for this fact by using reduction factors that are a function of the number of connectors in line.

Split-Ring Connectors

The *split-ring connector* (Figure 29.10) is one of several important forms of the many types of mechanical shear developers used in heavy timber construction. A split-ring connector and the manner in which it transmits shearing forces in a joint are illustrated in Figure 29.14. The steel ring is placed in circular grooves cut in the surfaces of the pieces to be connected, with half the depth of the ring protruding into each member. The members are then held together with a bolt. External member forces are transmitted through the connection by bearing stresses distributed over the rather large surface area of the split ring instead of being transmitted by high bearing stresses on the bolt. The ring is split to facilitate better distribution of load both inside and outside the ring and to accommodate any dimensional changes in the timber.

The *shear connector* is another important type of mechanical connector. It consists of two metal pieces, set into rings in each of the pieces to be joined (Figures 29.10 and 29.14*b*). Thus load transfer from member to member is by shear in the bolt, but the transfer of load from the member into the bolt is on the large bearing surface around the ring of the connector.

The various mechanical connectors were established in Europe around 1910 and were introduced into U.S. practice about 20 years later. They have been widely used since the early 1940s. The more important advantages of mechanical connector joints include high efficiency, relative ease of fabrication and assembly, adaptability to prefabrication techniques, and a minimum of exposed metal that leads to better appearance and greater fire resistance than is found in conventional bolted joints.

The complex interaction of the many variables affecting the strength of split ring and shear connector joints precludes any rational analysis. As in the case of bolted connections, we must utilize the results of experimental programs to form a basis for designing these joints. Scholten [1944] has presented extensive data gathered in tests on split ring connectors at the U.S. Forest Products Laboratory. The data used in the following discussion are taken from his paper.

Prior to considering the influence of the major variables, we shall examine the behavior of a connection made with two 4-in. diameter split rings loaded parallel to the grain of air-dry wood (Figure 29.15*a*). As shown in Figure 29.16, there is an initial slip of about 0.02 in. that occurs as the connector and the timber surfaces are brought into solid bearing. The connection then exhibits a linear load-slip relation up to a proportional limit of about 2/3 the capacity of the connection. While the mode of failure varies depending on the size of the connector and thicknesses of timber, the first peak on the load-slip curve is often the load at which the circular core of wood enclosed by the connector fails in shear. This part of the failure mode often occurs with a rather loud noise, thereby providing a warning that the connection is in distress. After the shearing action, the joint continues to carry load and final failure consists of combined bolt bending and crushing of the wood fibers adjacent to the connectors. The maximum load for this particular connection was 40 kips at a slip of nearly 1/2 in.

The other basic joint configuration has the central member located perpendicular to the two outer members (Figure 29.15*b*). The performance of this connection is significantly below that of the parallel-to-grain loading case. The forces perpendicular to the grain of the transverse member produce splitting near the center of the connection and bulging below the split ring. Splitting extends in both directions, horizontally, from the connectors. The enclosed cores inside the ring ordinarily shear only in the upper half, above the horizontal split, while the intact lower half translates downward with the split portion of the member. Values of proportional limit for connections with bearing perpendicular to the grain are about 1/2 to 2/3 of the corresponding values for parallel-to-grain connectors.

The main variables to be considered in split ring connection design are: size of ring, species of wood, moisture content of wood, geometry of joint, and edge and end distances of timber members.

The effects of connector diameter and wood species are summarized in Figure 29 17 for $2\frac{1}{2}$, 4, 6, and 8 in. diameter connectors loaded parallel to the grain in a two-ring joint. Proportional limit loads are plotted against timber specific gravity for six different species. We see that the proportional limit P' for each connector size can be expressed as a linear function of specific gravity. The maximum capacities also follow this linear relation, for example, the maximum load for 4 in. diameter rings is $P_{max} =$ 76,000G. Connections in which the load is applied perpendicular to the grain of the transverse member (see Figure 29.15*b*) have the same general dependency upon specific gravity, but their behavior is much more erratic because of the more complex failure modes encountered in this type of joint.

Connections loaded at an angle between 0 and 90° are treated by utilizing the Hankinson formula (Equation 19.5) for basic strength of wood loaded in compression. For members inclined at 45°, we may utilize the fact that the perpendicular-to-grain strength Q is about 2/3 of the parallel-to-grain strength P, and we then compute the connection capacity to be

$$N = \frac{P\left(\frac{2}{3}P\right)}{P\left(\frac{1}{2}\right) + \frac{2}{3}P\left(\frac{1}{2}\right)} = \frac{4}{5}P \qquad (29.12)$$

or 80% of the parallel-to-grain strength.

Member thickness is important in that below a certain minimum thickness the failure mode in parallel-to-grain connections involves splitting as well as shearing of the enclosed circular core. The variation of proportional limit loads and maximum loads with thickness of center member in a three-member Southern yellow pine connection is shown in Figure 29.18. Side members are half the thickness of the center member with a $1\frac{1}{2}$ in. minimum thickness.

The optimum thickness for the 4 in. diameter ring is about 3 in. Joints with thicker members fail at a nearly fixed load by shearing inside the ring, while thinner members lead to splitting failures and reduced capacities. Allowable design loads for split ring joints are often tabulated for the optimum minimum thickness with reduction factors specified for thinner members.

Member width is crucial in joints with bearing perpendicular to the grain. An increase in width gives an increased proportional limit and increased maximum load because it helps to delay the splitting and bulging actions present in the perpendicular-to-bearing connection. Load values increase by about 10% for each inch of width above the minimum width ($4\frac{1}{2}$ in. for 4 in. ring) for widths up to at least the ring diameter plus 4 inches. On the other hand, the performance of parallel-to-grain connections is almost independent of specimen width.

The four parameters of end distance *e*, edge distance *a*, clear distance *c*, and spacing *s* are important factors in determining connection behavior. *End distance e* is crucial in a parallel-to-grain connection; the variation of capacity with *e* for a two connector (4 in. diameter) joint is shown in Figure 29.19. With end distances of less than 6 in. the failure modes involve shear along the projection of the connectors as well as splitting through the bolt holes for the lower values of *e*.

Likewise, the *edge distance a* is critical in members with perpendicular-to-grain connection forces. Test results are plotted in Figure 29.20. Note that the real need for the edge distance is on the side of the member in the direction of applied loading, and that the split rings may be placed off-center to provide an increased effective edge distance and higher strength.

25

Spacing s in parallel-to-grain connectors is important in that a connection with closely spaced rings will fail by shearing of the wood between connectors on the plane defined by the edge of the rings. The substantial reduction in proportional limit and capacity for 4 in. connectors spaced less than 9 in. is evident in Figure 29.21. Tests on these and other size connectors reveal that the minimum spacing needed to preserve the behavior of a single connector joint is about 3 in. plus 1.5 times the ring diameter.

Finally, we consider the influence of moisture on split-ring connection strength. Scholten [1944] found that the test results on green timber connections were lower than the results for similar connections in dry timber, just as the basic strength properties of green timber are lower than those of dry timber. An approximate empirical relationship is

$$\frac{P_{\text{green}}}{P_{\text{dry}}} = \frac{\text{parallel compression strength}_{(\text{green})}}{\text{parallel compression strength}_{(\text{dry})}}$$

The actual design of split ring and other mechanical connectors is done by using tabulated values with appropriate adjustment factors for moisture content, duration of loading, connector spacing, and actual values of timber thickness, end distances, and edge distances. The assignment of a suitable factor of safety is a crucial step in formulating the tables of allowable connector loads. Allowable working loads for split ring connectors are normally taken as about 1/4 of the ultimate capacity or 1/2 the proportional limit, whichever is less. Connections with perpendicular-to-grain bearing are rated from the proportional limit because of the greater variability in failure mode and ultimate capacity. Actual values of allowable connector loads, as specified by the National Design Specification [1971] for timber, will be given in Volume 4.

29.6 SUMMARY

The behavior of structural timber elements and connections is made complex primarily by the great differences in the strength and stiffness properties of timber as it is loaded at various angles to the grain and by different types of stress. In spite of these complexities, stresses are normally computed by conventional elastic analysis expressions and then adjusted to account for such factors as depth effect, form effect, curving of laminations, and duration of loading.

PROBLEMS

29.1 The basic allowable bending stress for a certain grade of timber is 2000 psi. What is the allowable stress if the timber is used in a 48 in. deep glued-laminated member and you are checking stresses produced by snow loading (2 months duration)?

29.2 For the depth range from 12 to 100 in., plot Equations 29.1*a* and 29.1*b* and compare the predicted depth reduction factors.

29.3 Describe the possible failure modes for glued-laminated timber beams, including sketches of load-deflection response.

29.4 You have a timber beam with a knot in one face. How do you position the beam in the structure to minimize the effects of the knot on strength?

29.5 Timber design specifications place limits on the ratio of lamination thickness to radius of curvature (t/R in Equation 29.5). Why are limits needed? Can you suggest reasonable values for these limits?

29.6 Laminations 3/4 in. thick are bent to a curvature of 75 in. in the corner section of a laminated timber frame. If the allowable working stress in a straight lamination is 2200 psi, what stress can be allowed in the lamination?

29.7 Two in. by 10 in. floor joists on 16 in. centers are often used in home construction. If the floor span is 12 ft, estimate the peak stress in the joists. Are there structural criteria other than stress limits that enter into floor design?

29.8 A timber beam, 8 in. wide, has an end reaction of 24 kips. How much end bearing distance is required if the allowable compression perpendicular to grain is 450 psi?

29.9 What is the allowable load capacity of a 5½ in. square Ponderosa pine column, with hinged ends, if the length is (a) 8 ft and (b) 16 ft.
use $E = 1{,}100{,}000$ psi, $F_c = 800$ psi allowable compression parallel to grain, and a factor of safety of 2.7 on the elastic Euler buckling expression.

29.10 Determine the adequacy of the timber beam column in Figure P29.10, using both Equations 29.10 and 29.11. Use Douglas fir, Select structural grade P&T, with $F_c = 1200$ psi, $E = 1{,}600{,}000$ psi, and $F_b = 1500$ psi. Use a factor of safety of 2.7 on the elastic Euler buckling expression.

29.11 Compare Equation 29.11 with the interaction equation used for steel beam columns and comment on the differences. Which one most closely represents true beam-column behavior over the entire range of slenderness ratios?

29.12 Find a timber structure in your community and examine the joint details. Sketch typical joints and describe their function.

29.13 For the connections depicted in Figure 29.17, how many 2½ in. connectors can be replaced by a single 6 in. connector? Comment on the relative load-slip characteristics of joints made with the two size connectors.

29.14 Why are we interested in the slip in a joint, such as in Figure 29.13?

29.15 It is proposed that a timber connection be made with a mixture of four 1 in. bolts and two 4 in. split ring connectors. Comment on this proposal, giving particular attention to how the load might be shared between the bolts and the split ring connectors at various load levels up to failure

29.16 Would the use of a high strength, tensioned bolt in a split ring connector joint (such as in Figure 29.16) improve the slip resistance and strength of the joint? Explain carefully.

29.17 How would you make a truly hinged connection at the base of a three hinged timber arch? Sketch the connection and describe it fully.

29.18 How would you make a moment-resistant splice between two timber beams? Can your connection also transmit shear?

29.19 A heavy laminated timber beam frames into a timber column. Devise a simple connection detail that will transmit the end reaction of the beam into the column without transmitting any significant bending moment.

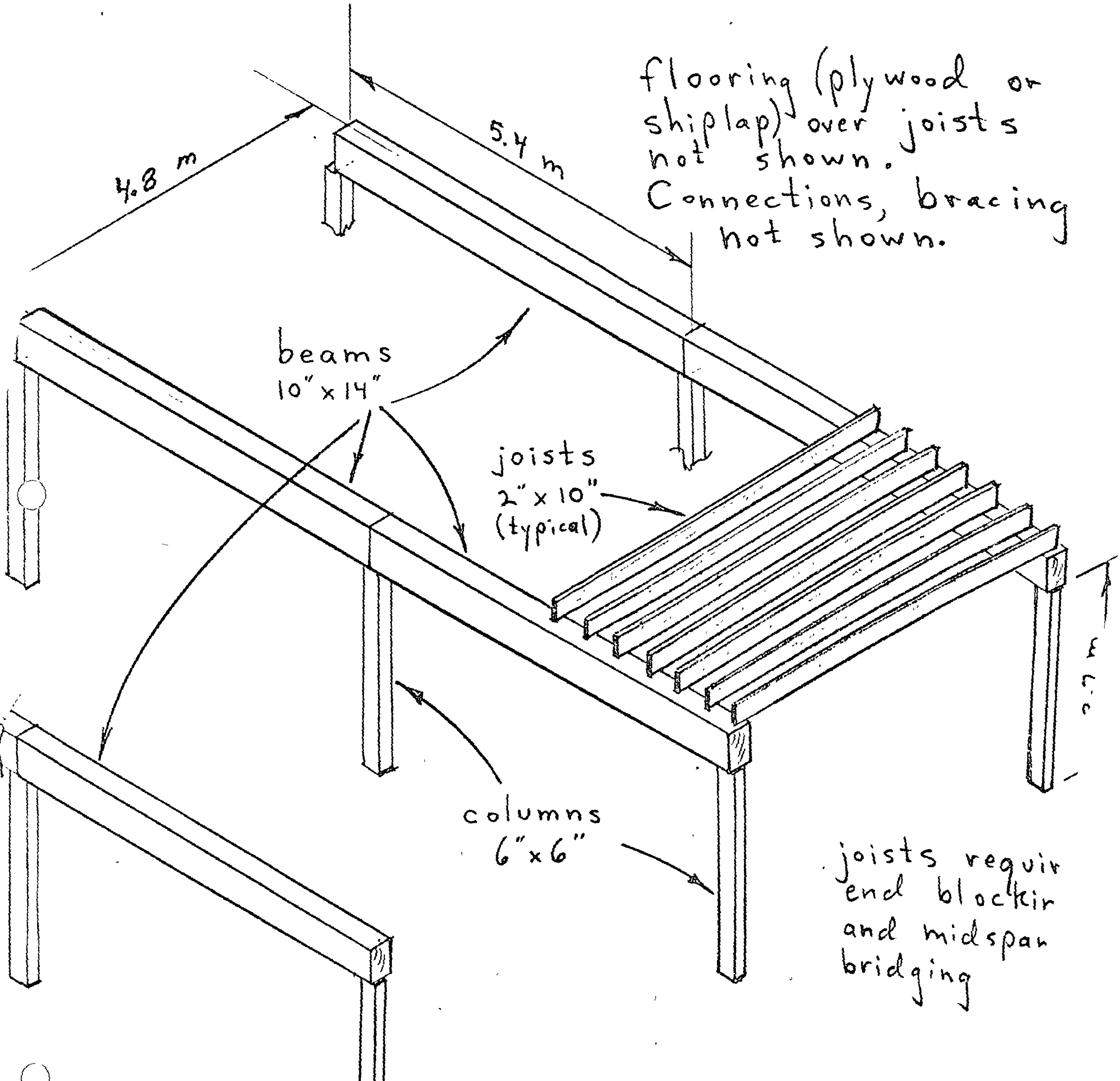


Fig. 1 Framing Plan
Beam & Joist F

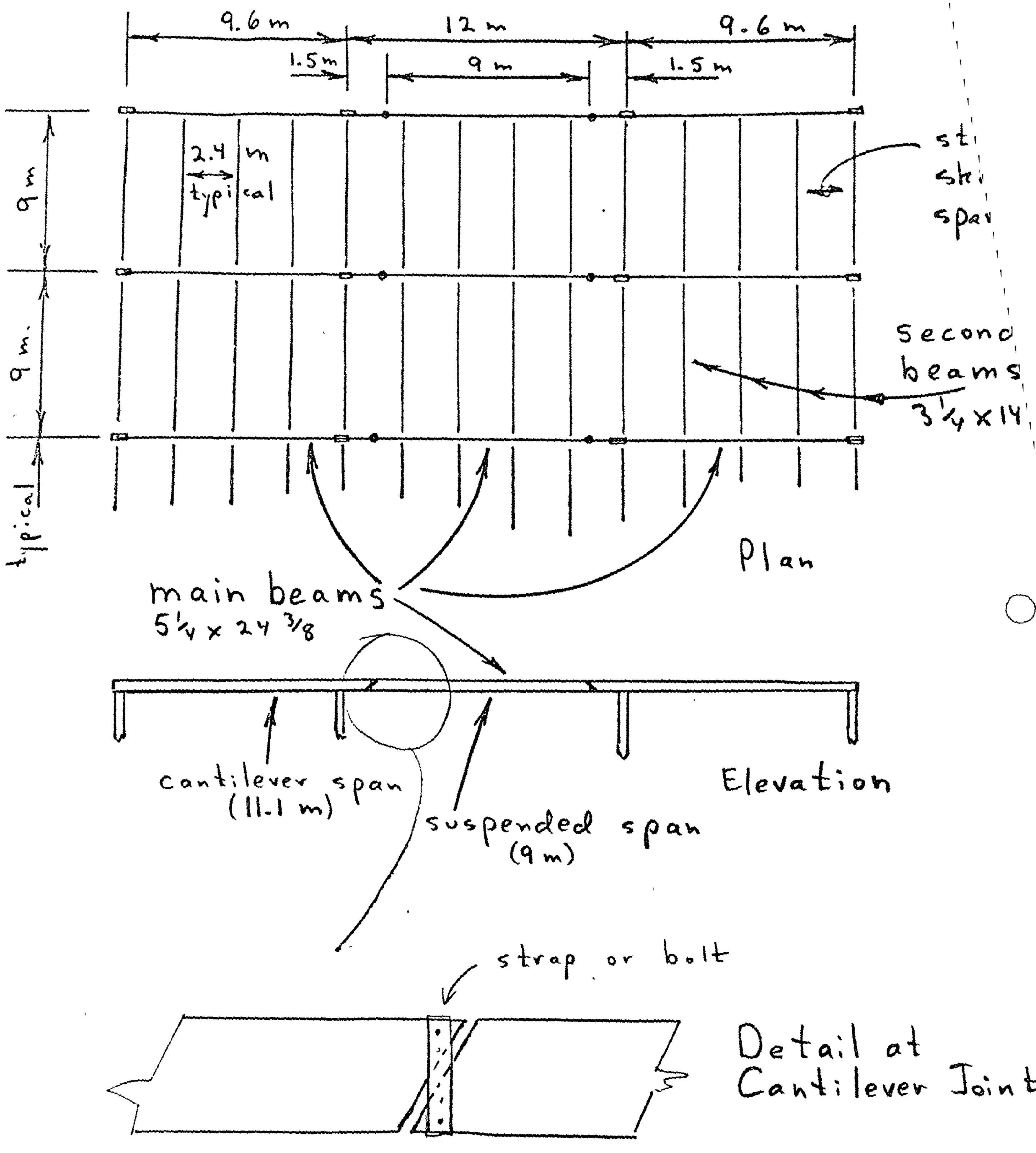
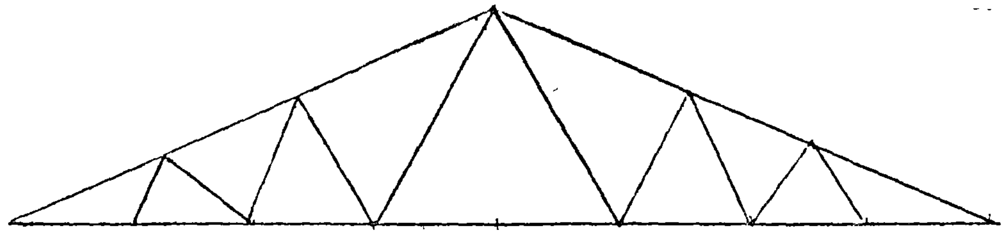
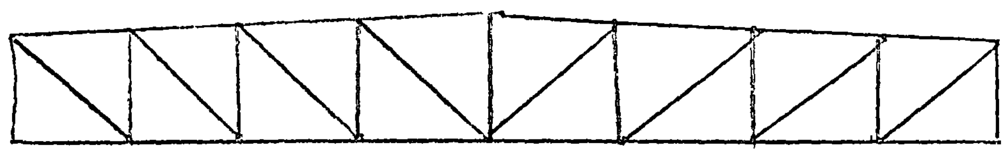


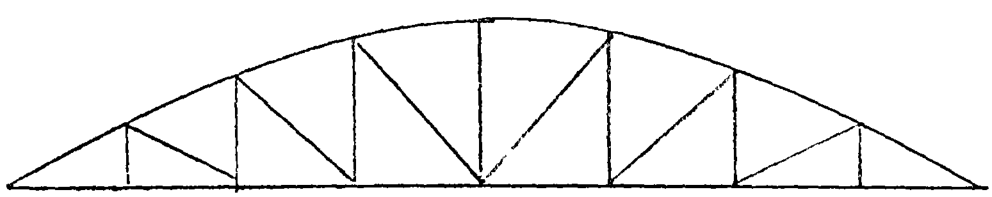
Fig 2 Framing Plan
Post



(a) Pitch truss

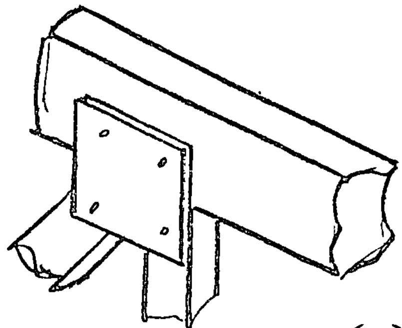


(b) Flat Pratt truss

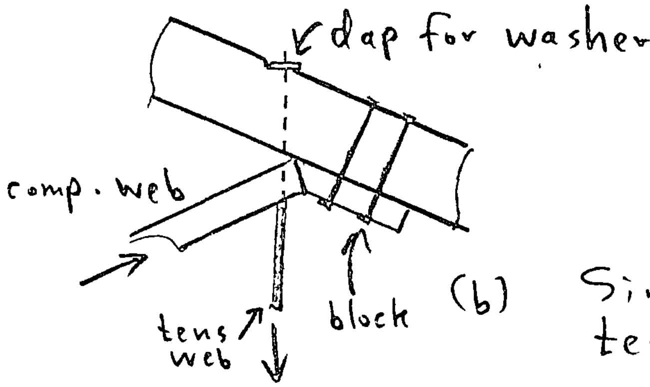


(c) Bowstring truss

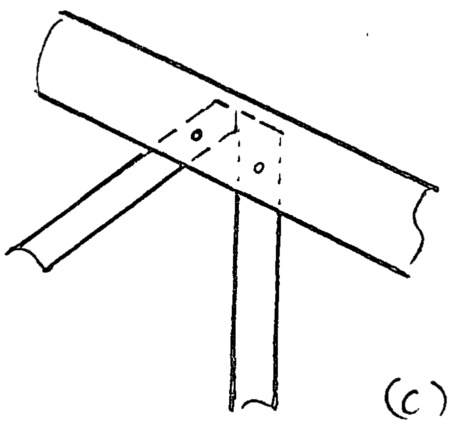
Fig. 3 Truss Types



(a) Single Chord with gussets

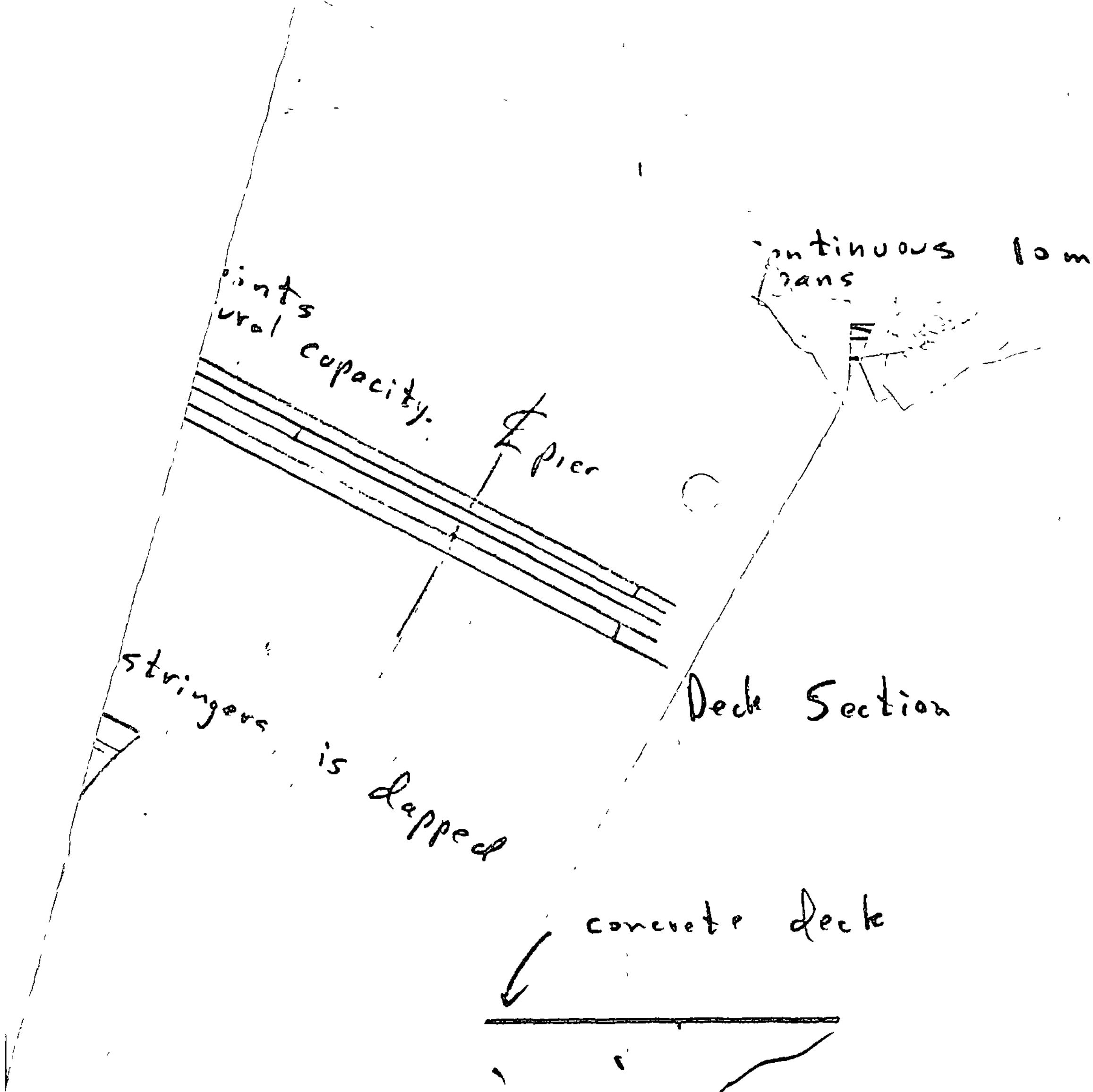


(b) Single chord with tension rods

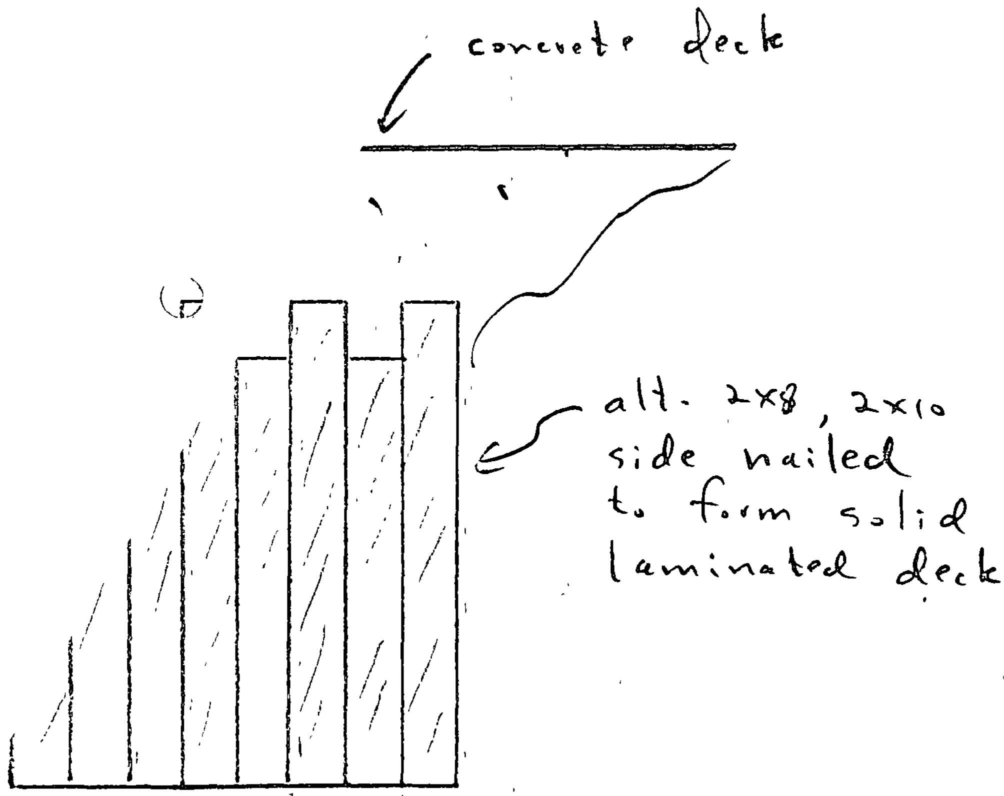


(c) Double spaced chord and single webs

Fig 4 Typical Joints



Deck Section



(b) Transverse Deck Section

Fig 1

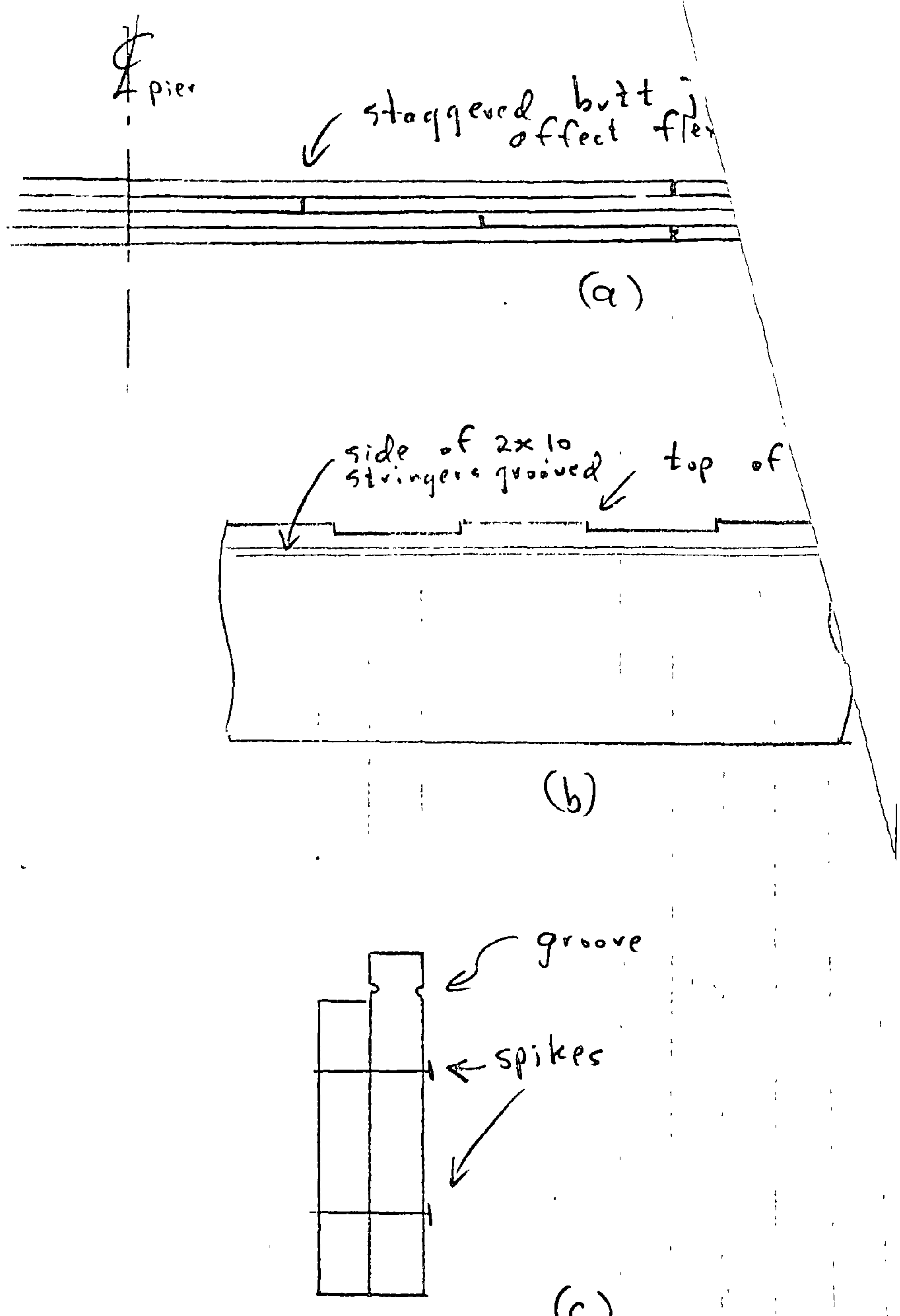


Fig 2 Details

L A M A D E R A E N L A V I V I E N D A

Francisco Robles
Ramón Echenique

1. EVOLUCION DE LA VIVIENDA DE MADERA

1.1 Factores que intervienen en la elección de la madera como material de construcción

¿Qué lleva al usuario a escoger la madera como material de construcción? No siempre son consideraciones de tipo económico o de disponibilidad las que determinan la elección de madera como medio para realizar vivienda. En efecto, se da el caso de regiones boscosas donde se prefiere la piedra y viceversa. Algo de esto es lo que sucede en México. Pueden ser factores decisivos la tradición, el afán de permanencia, la imitación de soluciones que han tenido éxito en otras regiones, la falta de una tecnología adecuada, e incluso, en algunas culturas, consideraciones religiosas. En México, una de las causas de rechazo de la madera como material para la vivienda es la tendencia a asociarla con viviendas de mala calidad (el jacal) y con estructuras provisionales (barracones, etc.)

1.2 Algunos antecedentes históricos

Son muy diversas las formas en que el hombre ha utilizado la madera para crear espacios habitables. Los ejemplos que se reseñan brevemente a continuación ilustran algunas de las modalidades que ha adoptado la vivienda de madera según el modo de vida del usuario y las condiciones climatológicas.

Fosas cubiertas y forradas de troncos.- En Rusia se han encontrado evidencias de viviendas de este tipo, que datan de la época paleolítica. Las dimensiones eran del orden de 3.5 x 13 m y el acceso consistía en una rampa.

Viviendas de postes y ramaje entrelazado.- Durante la época neolítica fueron comunes las viviendas de planta rectangular y techo empinado de doble vertiente cons truidas con postes clavados en el suelo como soportes verticales y vigas inclinadas para formar el techo. Con frecuencia se utilizaba una hilera interna de columnas para soportar la cumbrera. Los muros se formaban con postes de madera verticales, a veces de troncos partidos, clavados en el suelo, uno junto a otro, con entramados de rama cubiertos de lodo o con corteza de árboles. Para el techo parecía usarse zacate o algún recubrimiento de madera. En algunos casos las viviendas eran lo bastante grandes para alojar a varias familias. Se han encontrado restos de estructuras como las descritas en las cercanías de Colonia (4 000 a de J.C.) y en Würtemberg(1 100 a de J.C.). Sistemas semejantes se siguieron utilizando en los países escandinavos durante la época de los vikingos.

Palafitos.- Estas viviendas lacustres de la época neolítica se apoyaban sobre pilotes clavados en el fondo de los lagos. Los pisos probablemente estaban formados por troncos acostados unos al lado de otros.

Viviendas mexicanas de la época prehispánica.- Las viviendas de la época preclásica se parecían a las chozas actuales del Valle de México y consistían en una estructura a base de varas entrelazadas embarradas con arcilla, y una techumbre de paja.

En el norte del país se utilizaron viviendas comunales de forma cónica en diámetros hasta de 20 m. Otras variantes de vivienda en esta región fueron las casas alargadas de forma rectangular hechas con postes de madera y ramaje y combinaciones de adobe con pisos y techos de madera y paja. Estas últimas viviendas a veces tenían varios pisos.

En la cultura teotihuacana se utilizó la madera como refuerzo de muros y columnas de mampostería. Los techos se hacían con gruesas vigas de madera que sostenían un sistema de palos redondos y carrizo sobre el cual se colocaba una capa de tierra y un entortado.

Las viviendas de la época azteca eran parecidas a las de la época preclásica. Las de las clases acomodadas a veces eran de mampostería con techos de madera. Una aplicación interesante de la madera fue el empleo de estacas para cimentar.

Las casas de los mayas fueron muy parecidas a las de hoy todavía son comunes en Yucatán.

Casas de troncos horizontales ("log cabins").- Fueron comunes en el norte de Europa y posteriormente en Norteamérica.

Viviendas inglesas de la Edad Media.- Uno de los países donde con más éxito se ha utilizado la madera como material de construcción es Inglaterra. Son típicas las casas medievales con muros de construcción mixta de madera y mampostería - ("half-timber houses"). Los techos de armaduras de madera aserrada para las viviendas lujosas son notables. En la vivienda modesta fue frecuente el techo de zacate.

Viviendas de armazón de madera forrado de tablas ("frame house"). Este sistema es frecuente en algunos países de Europa y ha sido la modalidad de construcción de vivienda más común en Canadá y Estados Unidos.

Uso de la madera como material para vivienda en algunas culturas primitivas.- La variedad en las condiciones de la vida y en los tipos de maderas disponibles ha dado origen entre los pueblos de culturas primitivas a soluciones de notable diversidad. Compárense las casas de los malayos o los seminoles de Florida, las yaguas del Amazonas, con las chozas africanas, las viviendas portátiles de Mongolia (yurts) o de los nootkas del noroeste de Norteamérica y las estructuras de madera y tierra de los yakuts de Siberia.

2. SISTEMAS CONSTRUCTIVOS QUE SE HAN UTILIZADO EN MEXICO

La madera se ha usado tradicionalmente en México como material para construcción de viviendas. Sin embargo en los últimos años su nivel de aprovechamiento no ha alcanzado el de otros países productores de madera. Se describen aquí algunos usos tradicionales y algunas experiencias recientes.

2.1 Sistemas rudimentarios

En muchas regiones del país todavía es frecuente encontrar viviendas de tipo rudimentario (chozas y jacales) de gran variedad. Para el techo se usan diversas combinaciones de morillos o vigas con zacate, palma teja o lámina. El techo se apoya en unos casos sobre postes hincados en el suelo, y en otros, sobre muros de mampostería. Son ejemplos las viviendas de Veracruz, Yucatán, Estado de México, etc.

2.2 Techos de vigas

Se encuentran dos variantes. En una, las vigas soportan una tarima de tablas sobre la que se coloca tierra (techumbre de terrado). En la otra la techumbre se forma con ladrillo (bóveda plana o bóveda catalana).

2.3 La vivienda de madera michoacana

Michoacán es una de las regiones de México donde la vivienda de madera tiene mayor tradición. El sistema estructural es a base de postes, vigas y tablas combinados con un estilo peculiar de la región.

2.4 Viviendas prefabricadas de madera de triplay

Aunque todavía en escala muy reducida, se ha iniciado la producción de casas prefabricadas.

La empresa "Triplay y Maderas de Durango", por ejemplo, ha construido más de 100 casas en Mazatlán, Guadalajara y la Paz, a base de secciones prefabricadas de madera para los muros y el techo. El techo se recubre con lámina galvanizada. La superficie de las vivendas es del orden de 50-70 m^2 y su costo es aproximadamente \$ 500.00/m^2, incluyendo instalación eléctrica e hidráulica.

Indeco ha propuesto un sistema de muros prefabricados de elementos "sandwich" de triplay y poliuretano mediante el cual puede construirse una vivienda de cerca de 50 m^2 en tres días, con un costo de \$ 18,000.00. El sistema parece haberse usado en California y Nicaragua.

Se encuentra en estudio el empleo de armaduras prefabricadas de madera para techar, construidas con elementos "gang-nail" para las uniones.

2.5 Viviendas prefabricadas con láminas aglomerados

PROTINBOS ha promovido en Toluca el sistema VELOX, basado en una patente alemana, consistente en el empleo de láminas prefabricadas de aglomerados a base de desperdicios de madera y cemento, que se usan tanto para los muros como para el techo. Los techos se protegen con papel asfáltico e impermeabilizante. La superficie de las viviendas es 62 m^2 y su costo es \$ 450/m^2.

2.6 Residencias vacacionales

Un ejemplo de residencia vacacional es la casa octagonal construida por "Wood Products de México" en Valle de Bravo. El techo es de madera protegida con papel asfáltico y tejamanil de asbesto. La vivienda tiene una superficie de 200 m^2 y su costo fue de aproximadamente \$ 1,400/m^2.

También en Valle de Bravo, Arturo B. Avendaño, ha construido cabañas en A, de dos niveles, con una planta de 48 m^2. Los muros y techos son de madera aserrada y triplay. La techumbre está cubierta de papel asfáltico y tejamanil. El costo aproximado de una cabaña es \$ 120 000.

2.7 Conjunto habitacional popular en estudio por SAG

La SAG tiene en estudio una unidad habitacional de madera para el medio rural. El techo será de madera con lámina corrugada de cartón o lámina. La superficie de la unidad será 144 m^2 y se estima que su costo será de unos \$ 400/m^2.

3. EXPERIENCIAS EN OTROS PAISES

3.1 Los países escandinavos

Los países encandinavos son posiblemente los más adelantados en el empleo de sistemas de prefabricación para producir vivienda de madera.

Producen no solamente para el consumo interior sino también para la exportación. El país donde más extendido está el uso de la vivienda de madera es Suecia, donde los usuarios han mostrado en los últimos años una preferencia marcada por la vivienda aislada, que se presta a la construcción con madera, frente al multifamiliar de concreto. Los sistemas utilizados van desde el sistema de elementos "pre-cortados" (pre-cut), que se reduce a la preparación de paquetes de piezas cortadas a la medida, hasta sistemas espaciales. Existen multitud de variantes intermedias a base de distintos tipos y combinaciones de componentes modulares.

3.2 Estados Unidos, Canadá, Australia, Nueva Zelandia

En estos países, la madera es probablemente el material más utilizado para la construcción de vivienda. En los Estados Unidos cerca del 90% de las viviendas son de madera. Los datos correspondientes para Nueva Zelandia y Australia son semejantes. Todavía parece prodominar la construcción convencional aunque el uso de la prefabricación se está extendiendo mucho, Hay empresas que producen viviendas completas en sus plantas, desde donde las transportan en plataformas al lugar de destino. En los Estados Unidos ha adquerido gran popularidad el concepto de vivienda móvil ("mobile home"). El 25% de las viviendas unifamiliares que se construyen en la actualidad son de este tipo.

En los Estados Unidos, El Departamento de Agricultura ha elaborado un gran número de instructivos que facilitan la auto-construcción de viviendas rurales. Un tipo de estructura que ha despertado considerable interés en los últimos años es el "pole-building", apoyado sobre postes hincados en el terreno. Se han desarrollado también otros sistemas usando el concepto de "stress-skin" o a base de paneles "sandwich".

Sistemas modulares interesantes son el "Nu-Frame" y el "Unicom".

constructivos empleados en Nueva Zelandia

Los procedimientos y Australia son semejantes a los de los Estados Unidos, aunque en Nueva Zelandia y Australia es común usar teja o algún tipo de lámina para techar.

3.3 <u>Japón</u>

El Japón es otro país donde la madera es uno de los materiales más comúnmente utilizados en la construcción de vivienda, a pesar de que debe importar aproximadamente la mitad de la madera requerida.

3.4 <u>Sureste de Asia</u>

Es muy frecuente el uso de soluciones mixtas de muros de mampostería y armaduras de madera, éstas últimas prefabricadas en muchos casos. En general el uso de la madera para vivienda está más extendido que en Africa y Latinoamérica.

3.5 <u>India</u>

Existe un gran interés por promover el uso de la madera en la vivienda. La aplicación más usual es en casas de construcción mixta con muros de mampostería con techos de armadura. Se han desarrollado sistemas estandar de armaduras que permiten el aprovechamento de piezas de madera pequeñas.

3.6 <u>Africa</u>

En Nigeria existe una empresa (African Timber and Plywood) que ha desarrollado un sistema a base de componentes modulares de madera y triplay para muros y techos. El módulo básico es 3' y 4" (102 cm), de manera que el usuario puede plantear diversas soluciones sobre una retícula cuadrada de esta dimensión. Se proporcionan instructivos y asesoramiento a los usuarios que deseen construir ellos mismos su vivienda. En otras regiones el uso de la madera para vivienda está poco extendido.

3.7 <u>Latinoamérica</u>

<u>Brasil.</u> El sistema Campolar es un sistema a base de elementos verticales con

ranuras en las que deslizan las tablas o las ventanas o puertas. Los techos son de armadura. Se han empleado también sistemas a base de paneles de aglomerado (Brasilia).

Guayana Holandesa.- El costo de la vivienda de madera es la mitad del de la vivienda de mampostería. Existe una empresa que produce casas prefabricadas en que en los muros se emplea un tablero de partículas aglomeradas con resinas fenólicas (Fenboard).

Guyana.- La mayor parte de las viviendas es de madera. Se emplean los siguientes tipos de construcción.

a) Viviendas de postes redondos.- Son las más económicas y sulen construirlas los propios usuarios.

b) Casas de madera aserrada construidas convencionalmente.- Es el sistema más usual. En algunos casos se combinan muros de mampostería con techos de madera.

c) Casas prefabricadas.- Se empezaron a producir recientemente. Tienden a ser más baratas que las construidas convencionalmente, pero menos satisfactorias estéticamente.

Panamá.- Se encuentra en bloques de concreto estudio un sistema a base de piezas redondas, carrizo y techo de lámina que parece apropiado para programas de auto ayuda.

Chile.- Es el país de America del Sur donde es más común el uso de la madera en la construcción de viviendas. Se emplea tanto en las zonas rurales como en las ciudades. Se utilizan los sistemas convencionales y los basados en prefabricación.

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De una manera general puede afirmarse que en los países latinoamericanos, no obstante las aparentemente abundantes reservas forestales, el uso de la madera en la vivienda está limitado a la vivienda rural rudimentaria o al jacal de las ciudades pérdidas, por un lado, y a la construcción de un mínimo reducido de residencias de lujo. Salvo algunas excepciones, como Guyana, apenas se emplea en la construcción de viviendas modernas de costo bajo y medio.

4. RESISTENCIA A LOS INCENDIOS

La madera es un material combustible, y, por lo tanto, se considera como un material peligroso en caso de incendio. Sin embargo la naturaleza y grado de peligrosidad dependen mucho de la forma cómo se usa. Hasta la fecha no existe ningún tratamiento que transforme la madera en un material incombustible, pero experiencias e investigaciones en muchos países, han demostrado que bajo condiciones muy bien definidas la madera se puede utilizar en forma económica y segura como material en la construcción de viviendas.

Es muy importante tener en mente que los contenidos usuales de las viviendas (camas, mesas, sillas, cortina etc.) son, combustible suficiente para crear incendios serios sin importar que las paredes y techos sean de materiales combustibles o no.

En una vivienda la madera puede funcionar como: 1) sistema estructural, 2) protección y 3) decoración. Para cada caso el peligro de incendio asociado es diferente, lo mismo que las medidas de protección que se pueden utilizar.

4.1 Sistema estructural

La estabilidad estructural en un incendio depende del grado de resistencia al fuego que ofrezca y esto no es una propiedad del material, pero del elemento constructivo que puede estar hecho de cualquier material apropiado o combinación de materiales.

El propósito de resistencia al fuego es primeramente el de contener el incendio al lugar de origen y en caso de que esto falle el de asegurar que la vivienda no se derrumbe antes de que el incendio se consuma.

Los principales elementos estructurales de una vivienda que requieren resistencia al fuego son las vigas y columnas, para estabilidad estructural; las paredes y pisos para estabilidad estructural y/o integridad para prevenir el paso de gases y flamas a otras porciones de la vivienda y/o aislamiento térmico para impedir que otras partes o viviendas se prendan a causa del calor conducido.

En general la madera se consume a razón de 38 mm/h en condiciones de incendio, por lo que columnas y vigas de secciones grandes, resisten muy adecuadamente las cargas aún en incendios severos. Cuando las superficies de un madero se han prendido la combustión procede perpendicularmente a la superficie, se forma una capa de carbón, que actúa como aislante e impide la penetración de oxígeno a la porción central de la pieza.

4.2 Sistema de protección

Los techos y cubiertas exteriores, de muros forman para las viviendas los principales sistemas de protección del medio ambiente. El techo debe además tener la capacidad de impedir la penetración de fuego de una vivienda contigua o el que se prenda a causa de radiación de calor o de brasas volantes. Si se usa madera en estos casos, el techo se puede cubrir con material incobustible y los muros externos deben estar a cierta distancia del muro de la vivienda contigua para evitar la trasmisión del incendio, o deben estar protegidos mediante cubiertas aislantes o de material incombustible.

4.3 Sistemas de decoración

El problema principal en este caso es el de asegurar que la madera tenga resistencia a ser prendida y que no ayude a trasmitir el fuego. Por lo regular, la madera aserrada, contrachapada, los tableros de partículas y fibras son de secciones delgadas que los hacen muy susceptibles a prenderse y a trasmitir el fuego de un sitio a otro. Sin embargo son muy fáciles de proteger aplicando retardantes de fuego mediante los cuales la madera puede utilizarse reduciendo su peligrosidad inherente.

Los retardantes de fuego retardan, no controlan la ignición de la pieza y limitan las flamas en su superficie. En caso de incendios severos los retardantes de fuego son inefectivos.

En resumen, se puede decir que aunque la madera es un material combustible esta no representa demasiado riesgo en caso de incendio si en su uso en la vivienda se sigue una serie de consideraciones y recomendaciones.

5. RESISTENCIA A LOS SISMOS

Es bien conocido el hecho de que estructuras de madera se han comportado muy satisfactoriamente bajo fuerzas sísmicas. Son de interés los casos de Japón y de Califomia, regiones sísmica, donde una gran parte de las viviendas son de madera. La madera es un material con gran capacidad de absorber vibraciones y fuego de corta duración. Además su flexibilidad y ligereza la hacen un material ideal para construir viviendas en zonas sísmicas. Es interesante mencionar que en la escala modificada de intensidad de temblores de Mercalli, el desplome de edificios de madera bien construidos precede al grupo 12 que corresponde a la destrucción total. Las estructuras de madera son fáciles de rigidizar y de evitar deformaciones angulares de los muros.

Los ejemplos que se muestran, en las diapositivas ponen muy bien en evidencia las grandes ventajas que ofrece la madera, en pequeñas estructuras, a causa de su capacidad de absorción de energía, flexibilidad y ligereza.

6. RESISTENCIA AL ATAQUE DE ORGANISMOS

Aunque en pláticas anteriores se tocó el tema de durabilidad de la madera, ahora que estamos comentando sobre las viviendas, vale la pena recordar ciertos casos sobre hongos e insectos.

Es evidente que las condiciones ambientales en el país no son las mismas para todas las regiones; por lo mismo el riesgo de que la madera usada en viviendas sea atacada por hongos e insectos también es variable, según la región de que se trate. Las zonas desérticas o permanentemente frías ofrecen los riesgos más bajos mientras las cálido-húmedas, los más altos.

Para el caso de viviendas de madera los hongos causantes de pudrición y las termitas son organismos que tenderían a diseñar la estructura. Si recordamos nuevamente que para que los hongos destructores de madera se establezcan y desarrollen necesitan una serie de condiciones especiales de temperatura, humedad, alimentación, etc., y que la humedad es un factor crítico y relativamente fácil de controlar para madera que no está en contacto directo con el suelo o completamente expuesta a la intemperie, podemos decir que si se tiene el cuidado de detallar la construcción de la vivienda de tal manera que la madera que se utiliza siempre se mantenga seca, se evitara su deterioro por hongos, en forma muy fácil.

Para la madera que esté en contacto con el suelo, o expuesta a humedecerse por la lluvia, se le puede aplicar algun tratamiento con preservadores. El tipo, forma de aplicación y penetración y retención requeridos, será de acuerdo con el riesgo de que la madera esté sujeta al ataque por hongos y termitas. En esta forma se protege en forma efectiva a la madera. También en algunos casos se podrá utilizar madera sin tratar de especies, de gran durabilidad natural.

Por lo anterior podemos decir que aunque la madera es un material orgánico susceptible de ser atacado por organismos, si esta se usa en viviendas, mediánte detalles constructivos y aplicación de preservadores cuando el caso lo amerite, es posible garantizar la integridad y vida de la vivienda construida con elementos de madera.

7. COSTOS

La comparación de costos de sistemas constructivos diferentes es siempre difícil. Un análisis de costo superficial, basado en los precios que rigen en el mercado, puede fácilmente llevar a conclusiones equivocadas. En el caso de viviendas de -

madera, debe considerarse la influencia que puede tener el aumento en el volumen de producción con el aumento de demanda de vivienda de madera, el efecto que puede tener la normalización de elementos y componentes, y la influencia de la rapidez de construcción, que afecta importantemente al costo de financiamiento.

Existen otras ventajas de la madera, de naturaleza indirecta, que son difíciles de valuar, tales como el ahorro de energéticos, la disminución en el costo de financiamiento, el bajo costo de transporte y plantas, y otras.

Indudablemente son también significativos el costo de seguros y la durabilidad. Koenigsberger da algunas orientaciones sobre costos comparativos que son aplicables en regiones cálido-húmedas donde abunda la madera y predominan los techos inclinados. Indica que los cimientos de estructuras de madera son más económicos que para estructuras de mampostería. Considera también que tanto los pisos como los techos de madera son más económicos que los de otros materiales. Sin embargo opina que los muros de bloque o ladrillo son económicamente preferibles a los de armazón de madera forrado.

Algunos datos comparativos han sido compilados recientemenre por el Ing. Joel Octavio Quiñones. Según el Ing. Quiñones un techo de concreto cuesta \$ 92/m^2 mientras que el costo de la solución de madera equivalente es de solo \$ 79. El costo de cimentación puede disminuir de \$33.35, para una solución en mampostería, a \$ 21/m^2, cuando la estructura es de madera. Estos datos no deben tomarse sino como indicio del interés de la madera como material estructural.

Se menciona en la actualidad como un costo razonable el de unos \$ 450/m^2. (Los costos en Asia oscilan entre \$ 90 y \$800/m^2.) Esto significa que una vivienda de 60 m^2 costaría \$ 22,000.00, valor que no está al alcance de un sector muy importante de la población. Deben buscarse, por lo tanto, soluciones aun más económicas.

8. EL FUTURO DE LA VIVIENDA DE MADERA EN MEXICO

8.1 Sugerencias sobre las características de programas tendientes a fomentar el uso de la madera en la vivienda.

La actitud tanto del usuario de vivienda como de las empresas e instituciones que la construyen, financían y aseguran, es poco favorable al empleo de la madera. Sin embargo parece claro que, dadas las característucas económicas del país, el aprovechamiento de los recursos forestales para la solución del problema habitacional presenta perspectivas prometedoras. ¿Qué puede hacerse, entonces, para fomentar el uso de la madera en la vivienda? ¿Cuáles son las condiciones más apropiadas para el desarrollo de la vivienda de madera? ¿Cuáles son las aplicaciones más convenientes?

No es este el lugar para analizar con detalle las políticas y medidas que pueden contribuir a propiciar el uso de la madera. Es evidente que es necesario modificar los reglamentos de construcción, revisar la política de las compañías de seguros, introducir programas de estudio adecuados en los centros de enseñanaza tecnológica media y superior, estimular la investigación sobre tecnología de la madera, y promover el interés de los arquitectos en la madera como material estructural.

Se ofrecen a continuación algunas consideraciones sobre políticas a seguir en el desarrollo de programas tendientes a fomentar un mayor uso de la madera como material para la construcción de viviendas.

Existen ciertas restricciones que deben tenerse en cuenta al plantear el problema:

a) Los recursos forestales parecen adecuados, pero se carece de información sobre las propiedades de la madera.

b) Son escasos los técnicos de niveles medio y superior familiarizados con la madera.

c) La tecnología del tratamiento de la madera se encuentra en un estado rudimentario.

d) El capital disponible es escaso.

e) Abunda la mano de obra no calificada.

f) Existe un rechazo bastante extendido al uso de la madera.

g) Es escasa la capacidad administrativa.

Un examen de estas restricciones indica la conveniencia de empezar con soluciones mixtas, que combinen los materiales tradicionalmente aceptados con la madera. Parece claro que no puede esperarse un cambio repentino en la actitud del público hacia la vivienda de madera. Debe procederse gradualmente.

Los proyectos deben ser sencillos, sin refinamientos constructivos, de manera que no se requiera un excesivo control de calidad. Debe preferirse el clavo a la cola. Deben preverse holguras amplias y deben buscarse soluciones que admitan variaciones dimensionales importantes. La prefabricación debe introducirse gradualmente a medida que se desarrolla la técnica local. La instalación inmediata de fábricas altamente mecanizadas no parece aconsejable. Debe restringirse al mínimo la importación de tecnología. Puede ser útil el establecimiento de sistemas de coordinación modular que propicien la producción de elementos modulares sencillos que puedan utilizarse tanto en soluciones mixtas como en soluciones completas. Los sistemas a base de prefabricación deben dar preferencia a sistemas de elementos relativamente pequeños que puedan montarse sin equipo.

8.2 Formas convenientes de usar la madera en la vivienda

Se presenta a continuación un posible esquema de la evolución del uso de la madera en la vivienda y se hacen sugerencias sobre algunas aplicaciones que parecen ser de interés.

Una solución mixta que probablemente tendría fácil aceptación es la combinación de muros de mampostería con techos de armaduras de madera, por lo que podría constituir el punto de partida de un programa de popularización de la madera. La transición del tipo de vivienda usual a este tipo mixto no es violenta. Las ventajas del techo de madera son evidentes: ligereza, eliminación de la cimbra, rapidez, sencillez constructiva. Se presta tanto a sistemas de prefabricación como a sistemas convencionales de construcción. Desde el punto de vista de protección contra el clima puede ser ventajoso. El desarrollo de métodos económicos para construcción de armaduras parece un campo de investigación interesante. Existen dos opciones: la prefabricación en serie con un alto grado de mecanización o el desarrollo de sistemas basados en el empleo racional de mano de obra. En ambos casos parece aconsejable establecer sistemas estandarizados basados en coordinación modular. Puede ser interesante la elaboración de instructivos que faciliten la auto-construcción. Un aspecto importantes es el de las uniones. Deben desarrollarse sistemas que no impliquen pago de patentes. Parecen de interés las que se realizan con placas de triplay clavadas o pegadas. Sistemas que permitan el empleo de piezas pequeñas pueden ser convenientes.

Otra solución mixta podría basarse en el uso de vigas estandar prefabricadas de madera y triplay. Para que este tipo de solución resulte económica deben desarrollarse tipos de triplay más baratos que los que existen en el mercado, cosa que es posible en la opinión de técnicos como el Dr. Blomquist. Una de las razones del alto costo del triplay es el precio de las resinas fenólicas usadas en su fabricación, que son un producto de la industria petroquímica.

Tres aspectos de techos de madera como los descritos que requieren estudio son el aislamiento térmico, el tipo de revestimiento conveniente (tejamanil, lámina, papel asfáltico, etc.), y la forma de protegerlos contra el riesgo de incendios.

Un segundo paso en la popularización de la madera en la vivienda podría ser la introducción de muros divisorios a base de paneles. Existe ya en el mercado una diversidad de materiales derivados de la madera apropiados para la fabricación de paneles: aglomerados de diversos tipos, tableros de fibra, triplay, etc. Los paneles puede usarse no sólo para formar muros divisorios sino también para recubrimiento de techos y para formar plafones. En relación con el uso de estos materiales es necesario desarrollar técnicas para pegar los paneles a las armazones que los soporten. Es evidente el interés de producir componentes estandar para muros divisorios.

En construcciones mixtas de madera y mampostería puede pensarse en utilizar madera no sólo para los techos sino también para los sistemas de piso. De hecho, este sistema constructivo se ha usado tradicionalmente en México.

El paso siguiente podría ser el desarrollo de elementos verticales de carga. Estos pueden ser de armazón de madera cubierto de tablas o paneles construidos en forma convencional, o de componentes prefabricados. Los muros de carga presenten mayores problemas constructivos que los divisorios. Deben buscarse sistemas estructurales que resistan eficientemente tanto las fuerzas horizontales como las verticales. Un recurso es el de hacer que los materiales de revestimiento funcionen conjuntamente con las armazones que los soportan (stress-skin").

El último paso en la introducción de la casa de madera como una solución viable al problema habitacional sería la construcción de casas completas, sea por sistemas convencionales o sea recurriendo a la prefabricación.

Un tipo de estructura completa de madera cuyas posibilidades valdría la pena explorar es el apoyado sobre postes hincados en el suelo ("pole-building"). Presenta la ventaja de prácticamente eliminar la cimentación. Además, permite levantar el piso sobre el suelo a un costo bajo, lo que implicaría de por sí una gran ventaja sobre muchos de los sistemas de construccion rural usuales en la actualidad, sin que suponga cambios radicales en los métodos aceptados.

Uno de los sistemas constructivos más extendidos en México es el basado en muros de adobe. Desgraciadamente la falta de resistencia del adobe a las acciones sísmicas es notoria. Puede ser interesante investigar la posibilidad de reforzar los muros de adobe con un sistema rigidizante de madera.

Una aplicación de la madera de indudable interés es en los programas de renovacion o regeneración de vivienda.

8.3 Medidas que pueden contribuir a fomentar el uso de la madera en la vivienda

Independientemente de los programas nacionales para el fomento del uso de la madera puede ser útil favorecer toda clase de ensayes piloto en pequeña escala.

Se ha sugerido la conveniencia de promover la vivienda de madera entre los sectores de ingreso medio. La aceptación por estos sectores contribuiría a popularizar el uso de la madera.

Puede ser provechosa la acción de organismos oficiales como las Secretarías de Agricultura, Obras Públicas, Recursos Hidráulicos y la Comisión Constructora de la Secretaría de Salubridad, que podrían promover el empleo de viviendas de madera para sus empleados en estaciones experimentales, programas de desarrollo de la comunidad, campamentos permanentes. etc.

De gran interés puede ser la promoción de programas de auto-construcción en que se involucre la participación de los usuarios.

El uso de madera en zonas urbanas densamente pobladas, salvo posiblemente en los techos de sistemas mixtos, no parece que tenga muchas perspectivas de éxito inmediato. Los primeros intentos del uso de madera en gran escala deben hacerse en las regiones rurales y en particular en poblaciones de reciente formación. ("Lázaro Cárdenas", en las Truchas).

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USOS ESTRUCTURALES DE LA MADERA

TEMA DE PUENTES DE MADERA

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GLUED LAMINATED TIMBER BRIDGES--REALITY OR FANTASY[1]

Billy Bohannan[2]

Gentlemen: I sincerely feel it is an honor to address this group. Your Institute and the Laboratory have had one of the better relationships throughout the years. If we look back at the things we have accomplished together, we should feel exceedingly proud. However, my personal relationship with your industry has been more like a stormy romance--we have had our ups and downs. For this reason, I am awed at being permitted this captive attention. Whether our relationship will be up or down in about 20 minutes remains to be seen. It is like the man said, "I've got some good things to tell you and also some bad; which do you want first?"

I intend to reminisce a little about past and current research on timber bridges. I will give you some factual experiences with research bridges that the Forest Service has built. I will also give you some opinions of some of your consumers throughout the United States who should, may, or have already used glued-laminated timber in bridges. I may even throw in a few of my own opinions just to confuse the issue.

Just so we are all talking about the same thing, what is a timber bridge? If there is a timber bridge market, what type bridges are we talking about? The Keystone Wye (figure 1) is a beautiful structure. You all know about it, but if you have not actually seen the bridge, you cannot really appreciate how nice a structure it is. However, even though the Keystone Wye proves that glu-lam timber can be used in such structures, I would not predict a large market for this particular design. Rather, the places glu-lam arch bridges will be used are like these two examples: foot bridges (figure 2) and pedestrian crosswalks (figure 3). These curved shapes fit in well in many settings and should be promoted for such use. But to promote glued wood arches for major highway structures is questionable.

The major market for glued-laminated timber bridges is the plain, simple, functional bridge with longitudinal stringers and transverse wood deck (figure 4). Such bridges are used in all but the primary highway systems. Another potential market is railroad bridges (figure 5). Many railroad bridge structures are transverse crossties on longitudinal girders that are supported on pile bents. Glu-lam timbers should be used for pile caps, girders, and crossties.

The potential market for straight girder bridges is for spans from approximately 20 to 60 feet. However, wood bridges are used for much

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1. For presentation at Annual Meeting of AITC, Scottsdale, Ariz., March 13-16, 1972.
2. Engineer, U.S. Forest Products Laboratory, Madison, Wis.

longer spans. Long-span wood bridges are more popular in Canada. For instance, this 125-foot king-post truss bridge is a rather striking structure (figure 6). The Forest Service in British Columbia has used such bridges to span 160 feet, and have one planned which will span over 180 feet. The B.C. Forest Service also uses a straight girder with an I-section for long spans. We have one of their designs for an 86-inch-deep girder designed to span 151 feet.

The use of glu-lam timber in bridges dates back more than 30 years. I do not know which glu-lam bridge is the oldest. In Madison, Wis., a glu-lam arch foot bridge was first built in 1940. Unfortunately, it had to be rebuilt in 1951--untreated southern pine glued with casein glue, need I say more? One railroad in Texas used 11 glu-lam preservatively treated girders in a bridge in 1944. It was reported within the last year that these girders were still giving good service.

Bridge Market

One question that I anticipated was "how large a bridge market are we considering?" Such information is not readily available. The most complete figures that I could find were given in the Highway User Magazine, February 1971. This publication showed the total number of bridges in the United States to be over half a million (563,000). Of these, about 80 percent are in the state secondary, county secondary, rural roads, and city streets. These bridge classes imply the shorter span structures, for which timbers could be used. Most of the 80 percent on secondary roads were built prior to 1935. With the current movement toward highway bridge safety, many of these bridges will undergo inspection, and needless to say, many will be replaced.

Within the Forest Service road system, there are many thousand bridges. The estimated number of new and replacement bridges is 200 to 250 annually.

Somewhere I read that there are 1,700 miles of timber bridges within the railroad network. I could not verify this figure, however.

Based on this very limited survey, there is a timber bridge market, which I assume is extremely interesting to you. Now, what about our involvement?

Forest Service Timber Bridge Research

About 10 years ago the Forest Products Laboratory, in cooperation with the Division of Engineering of the U.S. Forest Service, initiated a timber bridge research program. The initial research evaluated the maximum bending moments and shear loads that should be used in the design of the longitudinal stringers. In the study, three typical single-span timber bridges were assembled and load tested under laboratory conditions, and five single-span Forest Service bridges in three National Forest areas were load tested in place. The conclusions were that the criteria for maximum bending moments of stringers were satisfactory. However, the design criteria for shear were very conservative. The AASHO Standard Specifications for Highway Bridges were changed to reflect these findings.

Since 1965 research has continued along two main directions, one pathological and one engineering.

In a decay hazard survey, moisture probes permanently placed in several Forest Service bridges have consistently shown hazardously high moisture content areas in the upper laminations of some glu-lam bridge stringers. To study possible factors that contributed to the high moisture content and potential decay hazard aspects, a deck was removed from a typical single lane Forest Service bridge that was 12 years old. Definite advanced decay was found in one stringer and other questionable areas of incipient decay were observed. The suspected reasons for the decay were insufficient penetration of preservatives due to lack of incising before treatment, and water entry into top laminations through deck fastener holes and checks in the wood. In both the fastener holes and checks, the moisture penetrated through the preservative.

Two approaches were proposed to meet the decay hazard: (1) a better preservative treatment of top laminations and (2) a bridge deck that would provide an effective roof over the complete bridge structure.

Nearly 100 percent treatment of the top two or three laminations is possible with a dual preservative treatment. Prior to manufacture, the top two or three critical laminations could be treated with a pentachlorophenol in volatile petroleum solvent. After manufacture, the complete stringer could be treated with a creosote petroleum solution. The stringers in one Forest Service experimental bridge have been given this dual preservative treatment. The same thing might be accomplished without dual treatment by using all-sapwood top laminations of an easily treated species.

A deck that will provide an effective roof over the complete bridge structure is also a reality. The most common timber deck is a nailed-laminated assembly of nominal 2-inch dimension lumber fastened with through-nailing of the laminations and toenailing of the laminations to the stringer. This system dates back 40 or more years and was basically satisfactory when used on solid-sawn stringers spaced close together. In this construction, the decking deflected very little between stringers so service performance was acceptable. However, with the advent of glued-laminated timbers the spacing of stringers has more than doubled although the decking system remained the same. This has magnified the necessary structural requirements of the decking.

On wide stringer spacings, the bending strength of the decking is still adequate, but deflection increased significantly. These deflections caused a working within the fastener system, resulting in an overall loosening of the deck. This loosening is exaggerated by shrinking and swelling of deck members through repeated wettings. As a result, for certain designs of bridges using glued-laminated stringers on wide spacings, the deck becomes excessively loose. In turn the loose deck not only undermines confidence in its structural preformance, but ineffectively roofs the bridge structure.

A decking system that will support a waterproof covering would effectively roof the deck structure and minimize the possible decay problems. But this requires a relatively stiff wood slab deck. Such a deck has been available for 30 or more years in the form of vertical glued-laminated wood slabs. The same 2-inch dimension material as now used in the nailed-laminated deck can be glued into a flat slab. The resulting panel will be stiff and, when covered with a sealant coat, will resist penetration of surface water.

Current AASHO Standard Specifications for Highway Bridges do not cover design of vertical glued-laminated wood bridge decking. However, a study, now just completed at the Laboratory, was undertaken to provide such design criteria. The experimental evaluations of a paneled deck system were supplemented with an exact series solution to develop design criteria for glued-laminated wood deck systems.

The expected practical width of glu-lam panels is 4 feet or less. Thus, at 4-foot intervals or less along the lengths of the bridge surface, there will be a joint in the decking. Such joints will permit relative deflections of adjacent panel edges, thus reducing the overall effectiveness of the glued slate system. However, design criteria have been developed for an effective shear-moment connector device to minimize this relative movement and provide some continuity of the load distribution properties of the deck.

Both criteria for the glued-laminated deck and for the shear-moment connector system will soon be published.

Experimental Field Bridges

To further evaluate the performance of glu-lam panels, experimental test decks are being used on selected Forest Service bridges. To date, six such bridges are in service and three additional bridges have been designed. We are planning three or four more later this spring. For this discussion the exact descriptions and locations of the bridges are not important. Such information is available from the Laboratory for those who are interested. The thing that is important to this discussion is our experiences in building these bridges.

One thing that naturally came up with each bridge was the cost of the glu-lam deck panels. For the most part, these were simple, straight panels with a minimum of shop fabrication. Yet the quotations on the delivered cost of treated deck panels varied widely, ranging from $350 to $725 per thousand board feet. To show you the variation, quotations were $350, $386, $418, $500, $615, and $725 per thousand board feet.

In all fairness the $725 per thousand board feet panels did require considerable plant fabrication. These panels were installed on a steel pony truss bridge. A total of 18 panels were needed and six of the 18 required special cutouts to fit over the steel superstructure. It was realized that these special cutouts would be reflected directly in the cost of the panels. Therefore, the high initial cost did not greatly disturb the Forest Service engineer. However, during installation of the panels, it was discovered that each panel was furnished exactly one-half inch too wide. The accumulated extra width in the 71-foot-long bridge resulted in the special cutout sections not fitting the bridge. Such experiences tend to discourage further use of glu-lam decking, at least on that forest.

One of our most reoccurring experiences has been that contractors are not familiar with erection of glu-lam timbers. One Forest Service engineer indicated that bid prices for the treated glu-lam material were reasonable whenever only manufacture and shipping were involved. However, when the bids included erection inplace, prices were very inflated.

I have discussed this individually with numerous laminators and was lead to believe that some laminators would both furnish material and arrange for bridge erection. This was encouraging, and bid invitations for the next two experimental bridges were sent to five laminators in addition to other contractors. Two bridges were included in the same bid package. They were small bridges, slightly in excess of 20-foot spans, which were being built on the same Forest Service road. From previous experiences, the Forest Service engineer was reluctant to bid both materials and erection cost in the same package. I assured him, however, that I had discussed this with the laminators and they would give a fair bid for both material and erection. The day of the bid opening, I received a phone call from a rather disturbed engineer. Not one of the five laminators submitted a bid. Needless to say, I called the principals of five laminating plants. Some of you may be here today. I won't relate the reasons given, but I will say that one Forest Service engineer does not trust you <u>or</u> me now.

The low bid that we received for the material inplace for the glued-laminated deck panels was $2,500 per thousand board feet. The next low bid was more than <u>double</u> that figure. Now these were plain, simple, straight longitudinal girders and transverse deck, nothing fancy. There was no pier work involved. The bridges were being placed on existing piers of old bridges. I am sure a crew of three people could have built both bridges in less than 1 week.

We have since rebid the bridges for material only. The second bid invitation was sent to only two laminators in addition to other material suppliers. Again upon bid opening day, I received another phone call; the laminators didn't bid. This time instead of calling principals of laminating plants I called the sales representative at each plant location. One did not bid because the Forest Service specified 8-week delivery and he had a 10-week backorder in his plant. The other laminator had bid through a treating company. For some reason or other the additional exercise of going through a treating plant resulted in an extra $100 per thousand board foot cost.

The accepted bid price of the panels was $650 per thousand board feet. This was for the glued-laminated panels, pentachlorophenol treated, and delivered to the job site. For some reason we also got some very inflated costs for the auxiliary solid-sawn members that were required with the bid. These were the handrails, wheelguards, and blocking. For instance, the blocks between the wheelguards and the deck are 4 by 6 by 12 inches in size. These were bid at $13 each. I think that figures out to be $6,500 per thousand board feet.

Candid Comments

To determine if our experiences with our experimental bridges were biased in any way, I corresponded with between 30 and 40 engineers to learn their experiences in the use of glued-laminated timbers. These engineers included Forest Service engineers, railroad bridge engineers, American Wood Preservers Institute district engineers, and several laminators. Some of these contacts requested that I not connect comments to them, and for this reason I will not provide any sources of information. But these are honest candid comments, and I think you'll be interested in what the consumers of your products are saying.

The Economics of Glu-Lam Timbers

Why not use glu-lam timbers? An almost universal comment, usually without explanation, is that "Glu-lam timbers cost too much for bridges." One indicator is the pile cap, a much-used timber in the railroad industry. This cap is 14 inches square and 14 feet long. One railroad indicated that on two occasions they considered switching to glu-lam timber caps. Once the cost of glu-lam timber caps was three times that of solid-sawn caps, and on the other occasion it was four times.

In addition to solid-sawn timbers, glu-lam must compete with two other materials for the bridge market--steel and concrete. Hopeful to you is the indication that steel will cost more than glu-lam, maybe in a ratio of 2 to 1. However, the ratio of cost of timber bridges to prestressed concrete bridges is reported to be about 5 to 3.

Solid-sawn timbers are still competing in spans up to about 35 feet. One forest is using 10- by 24-inch girders, 35 feet long. These are costing $340 per thousand board feet, preservatively treated and delivered.

It is difficult to make a direct comparison between cost of solid-sawn timber and glu-lam timbers because of the differences in the properties of the two materials. The glu-lam timbers have higher strength properties, thus smaller sections can be used. On the other hand, a solid-sawn 10 by 24 contains 20 board feet per running foot. A 10 by 24 glu-lam contains 27 board feet per running foot. Thus part of the economics of increased properties are offset by the increased number of board feet.

To sum up this whole question of economics, why do Canadians build large glu-lam timber bridges? The reason they give is that "glu-lam bridges are the cheapest." Yet in the United States consumers aren't convinced.

Competition from Concrete

The major competitor of timber for bridges is concrete, both reinforced and prestressed. Both forms adapt well to short span bridges. Prestressed concrete is being used for a large percentage of the new and replacement bridges in many areas of the country. One engineer indicated an 85 to 95 percent use of concrete. Some railroads indicated up to a mile and a half of timber trestles were being replaced each year with concrete.

Lack of Information

In many instances, the lack of information was given as the reason for not using glued-laminated timbers. In a conversation with one engineer, I asked if he had considered using glued-laminated timbers for bridges. His reply was, "Yes, I have considered it, but I understand it's not commercially available yet."

Another comment was that the information is available, but it is so fragmented that it would require a considerable amount of time to collect it all and put it into some usable fashion. This engineer suggested a design manual that would cover design criteria, design specifications, and material properties and typical bridge designs. The Canadian Institute of Timber Construction published a manual, Modern Timber Bridges, which basically covers this needed information. Your Institute may wish to consider such a manual.

Tom Brassell and I are members of the American Railway Engineering Association Timber Bridge Committee. This group of engineers is quite eager for information concerning glu-lam timbers. They asked me if we would prepare a special feature on the reliability of glu-lam timbers for the annual meeting of AREA.

We accepted and selected a distinguished panel of three representatives of laminating companies, Tom Williamson of AITC staff, and myself serving as the moderator. We presented a 20-minute show-and-tell, then opened the meeting to a panel discussion. This was the most enthusiastic audience I had ever appeared before. After about 50 minutes we had to stop the panel discussion with a dozen people standing in the audience with their hands raised for another question.

The most direct indication of the success of the panel discussion came immediately after we closed. Before we left the stage, a member of the Building and Bridge Association of the railroad industry came on stage and asked if we would put on the same show for their annual meeting later in the year. The same type meeting was presented again with even greater success. This indicated, certainly, that people are willing to learn about the use of glued-laminated timber.

Promotion of Glu-Lam Timbers

One discouragingly candid comment that kept reoccurring was that your industry is doing nothing to promote and sell glu-lam timbers. One comment along this line was that contacts made with laminators have been very unproductive. Many people consider the advancement of concrete in the bridge market was through promotion rather than material performance. Also, the cost stigma that is attached to glu-lam timbers can only be overcome through education.

This is not only education of the designers of glu-lam timbers, but also education of contractors that erect the timbers.

It is the opinion of many that if your industry wants the timber bridge market, you must be prepared to wage an extensive campaign in favor of timber. This campaign must cover top management right on down through to the person building the bridge. The campaign must be with engineers talking to engineers, not salesmen talking to engineers.

Some or all of your companies do have brochures out on timber bridges. I have seen some of them. One in particular I thought was quite impressive. This particular one implies engineering services, i.e., the customer furnishes bridge span and design load, and the laminator will furnish the bridge design with accurate material cost. This to me seems like a very attractive package if erection cost could be included.

Prefabricated Bridge Package

Well, gentlemen, those are the candid comments for what they are worth. Just a dream or two of mine, then I will quit. Why not a prefabricated bridge package? With glu-lam girders and glu-lam deck, nearly every piece of a bridge can be manufactured and fabricated in your plants. A truck with a self-contained hydraulic boom such as used by many loggers will transport the panels to the bridge site and also erect the bridge.

A systems approach will produce nearly instant bridges with minimum of onsite labor. If we dream a little further, a modular, shop-assembled bridge is possible. Maybe one day I will give you my thoughts on this.

In Conclusion

Glu-lam timber bridges, are they a reality or fantasy? I hope these factual experiences and the honest candid comments will help you decide. Timber bridges are certainly not an obsolete type of structure. I feel they are still an economic structure for many secondary road systems. Properly designed, they can give many years of low maintenance service. Current Laboratory evaluations and field bridge studies indicated that better overall performance and an extended service life of timber bridges are possible through the use of glu-lam panel decking. This combined with glu-lam girders essentially makes a prefabricated bridge package. It is possible through a systems approach to erect this package with a minimum of onsite labor.

What is needed to make glu-lam timber bridges a reality?

1. Sell your product.
2. Sell top management that wood is good.
3. Provide an information package that is readily usable by the design engineer.
4. Educate contractors in the use of your product.

TIMBER BRIDGES GO MOD

by roger tuomi
and billy bohannan

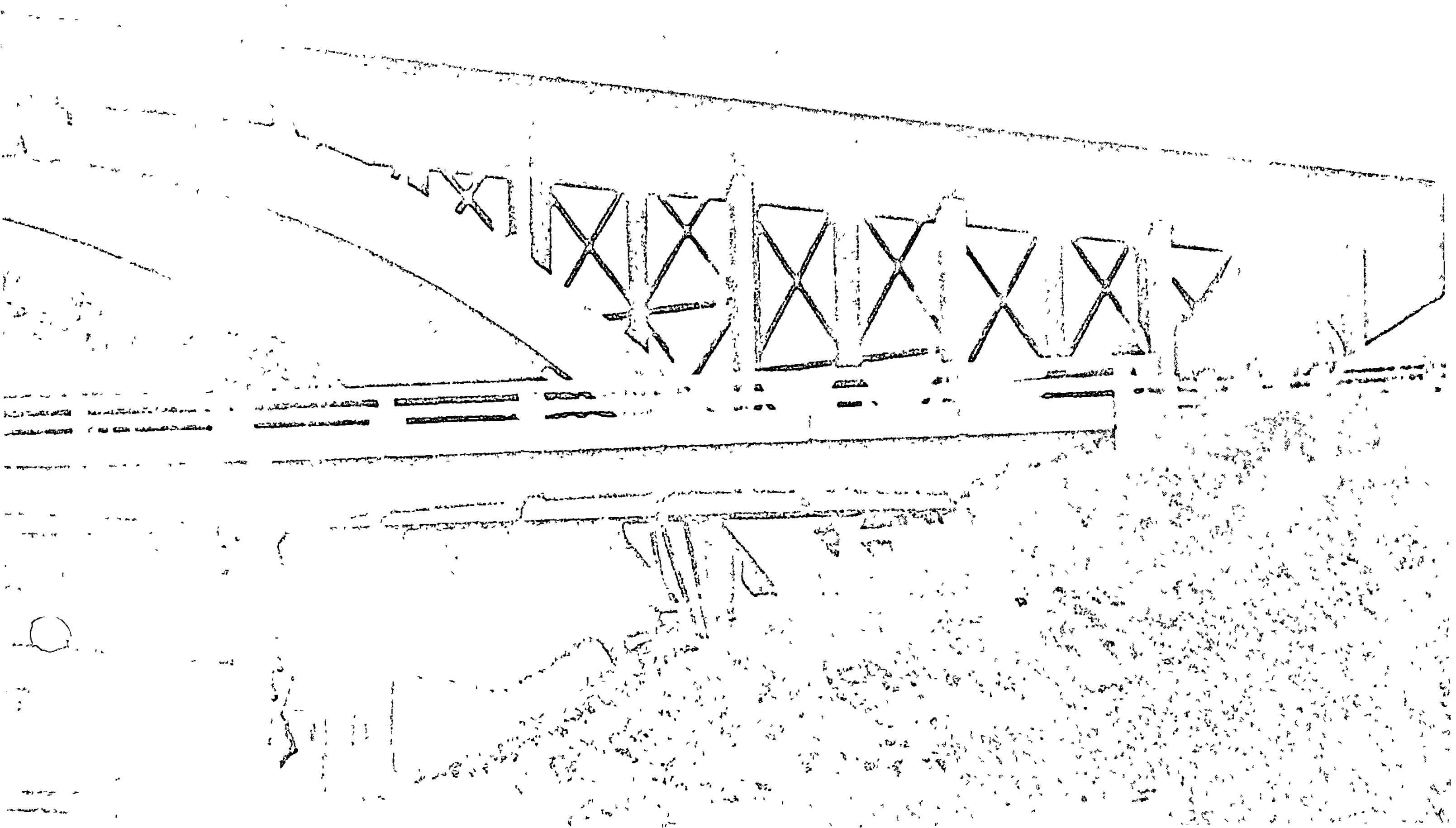
Who says timber bridges are dead? Should anyone be so rash, he'll encounter convincing rebuttal from Tuomi and Bohannan, engineers headquartered at the Forest Products Laboratory. The Lab, an arm of the USDA Forest Service, is maintained at Madison, Wisconsin, in cooperation with the University of Wisconsin.

The following article is based on a technical paper presented by Bohannan at the 1971 Meeting of the Structural Engineering Division, American Society of Civil Engineers. As you'll see, FPL research is pointing out new directions for timber construction.

The versatility and durability of wood make it an important engineering material for bridge construction, but its potential for such use is not generally recognized. Some people may even believe that timber bridges belong to the past. Far from being obsolete, timber bridges are widely used in secondary road systems and on 187 million acres of National Forest land. Timber construction also is used on more than 1,500 miles of railroad line in the United States and, to a limited extent, in major bridges for primary road systems. Timber bridges, properly designed and properly treated with preservative, will give many years of low-maintenance service.

Timber bridges come in a variety of shapes and sizes. Covered timber bridges, a part of our national heritage, are good examples of service longevity. One such bridge, 460 feet long, was built in 1866 and is still in service. Truss bridges are a popular design. One somewhat unusual truss bridge is located near Sumner, Wash. (Fig. 1).

Following the development of glued-laminated construction, two- and three-hinged timber arches have been used to support long, clear-span bridges. The Keystone Wye interchange near Mount Rushmore (Fig. 2) is a recent example of an arch bridge. The top bridge of the three-level interchange spans 290 feet.



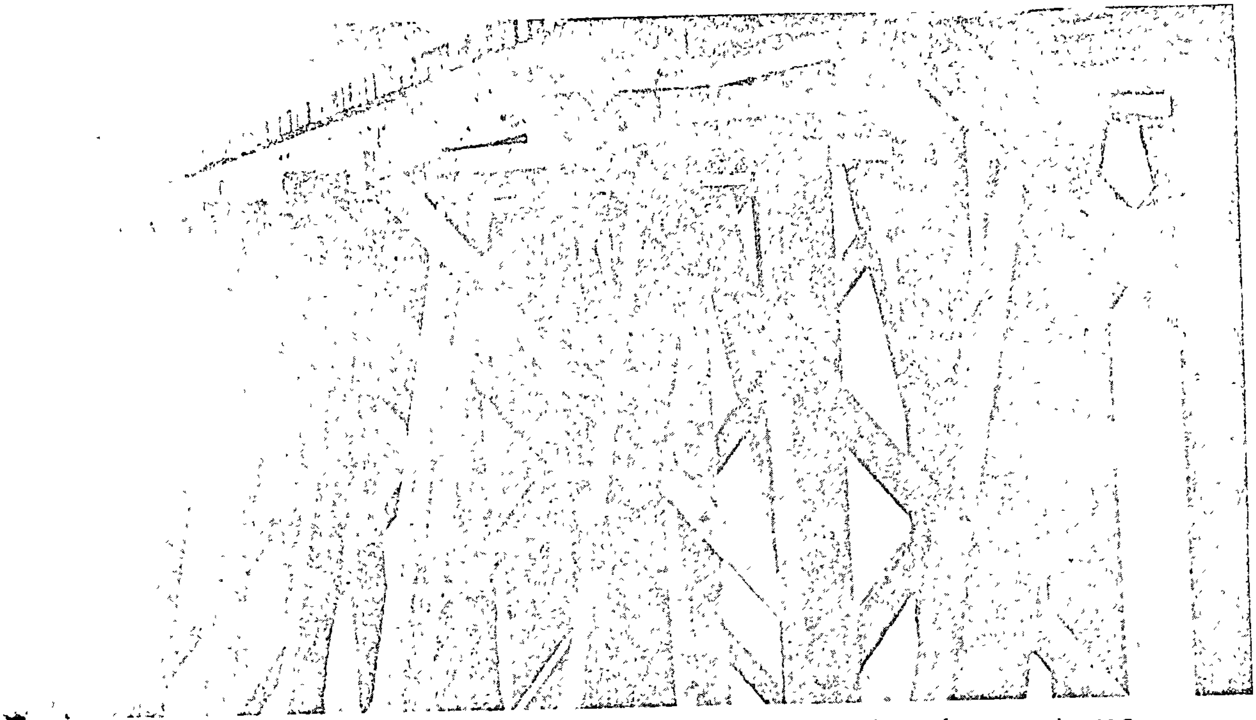


Figure 5.--Timber trestle bridges are common railroad structures. (M 140 028-17)

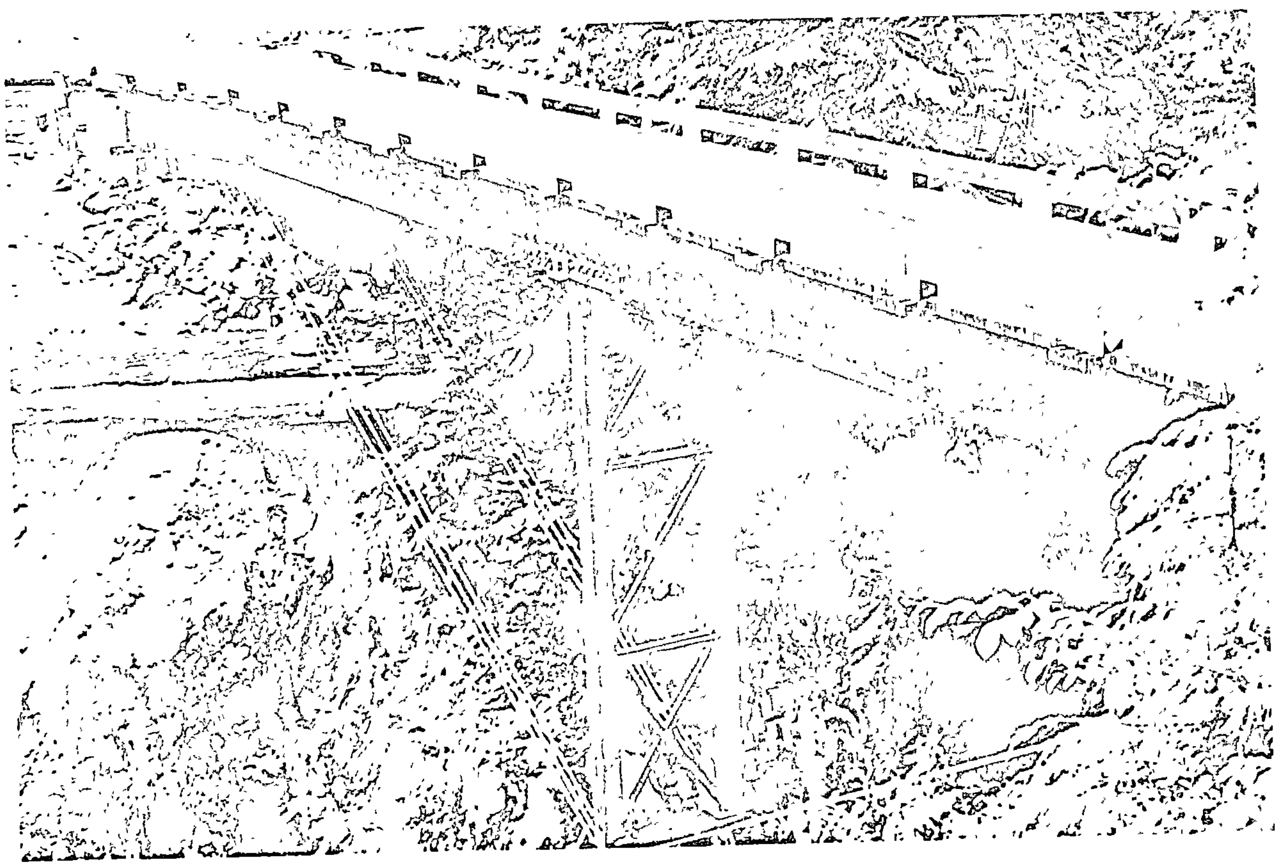


Figure 6.--A 125-foot king-post truss bridge. (M 140 028-19)

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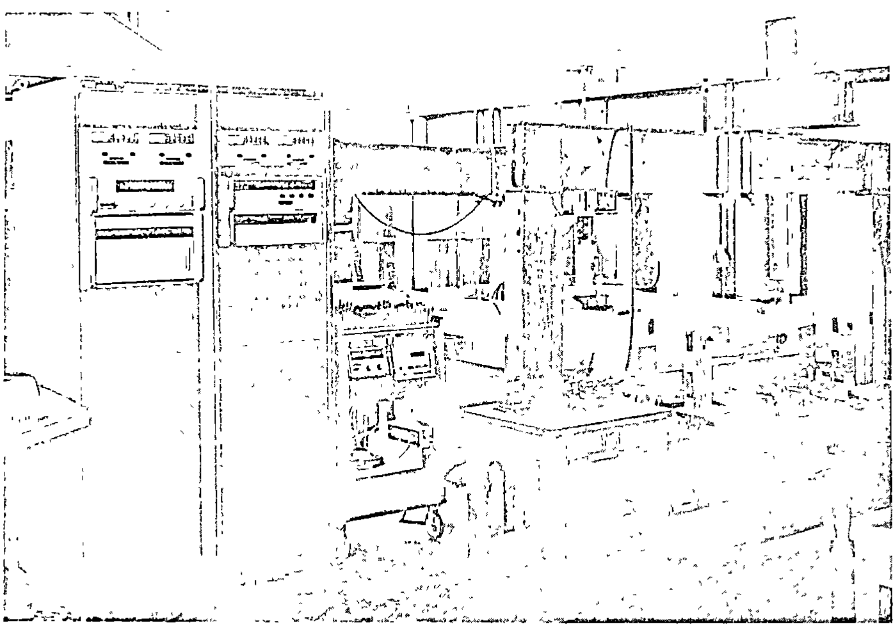

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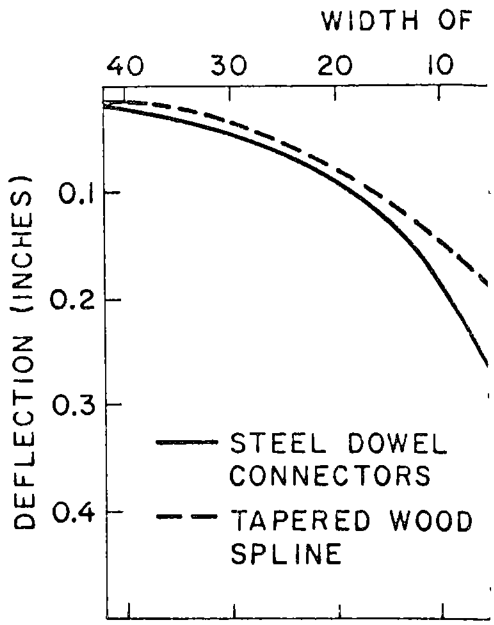

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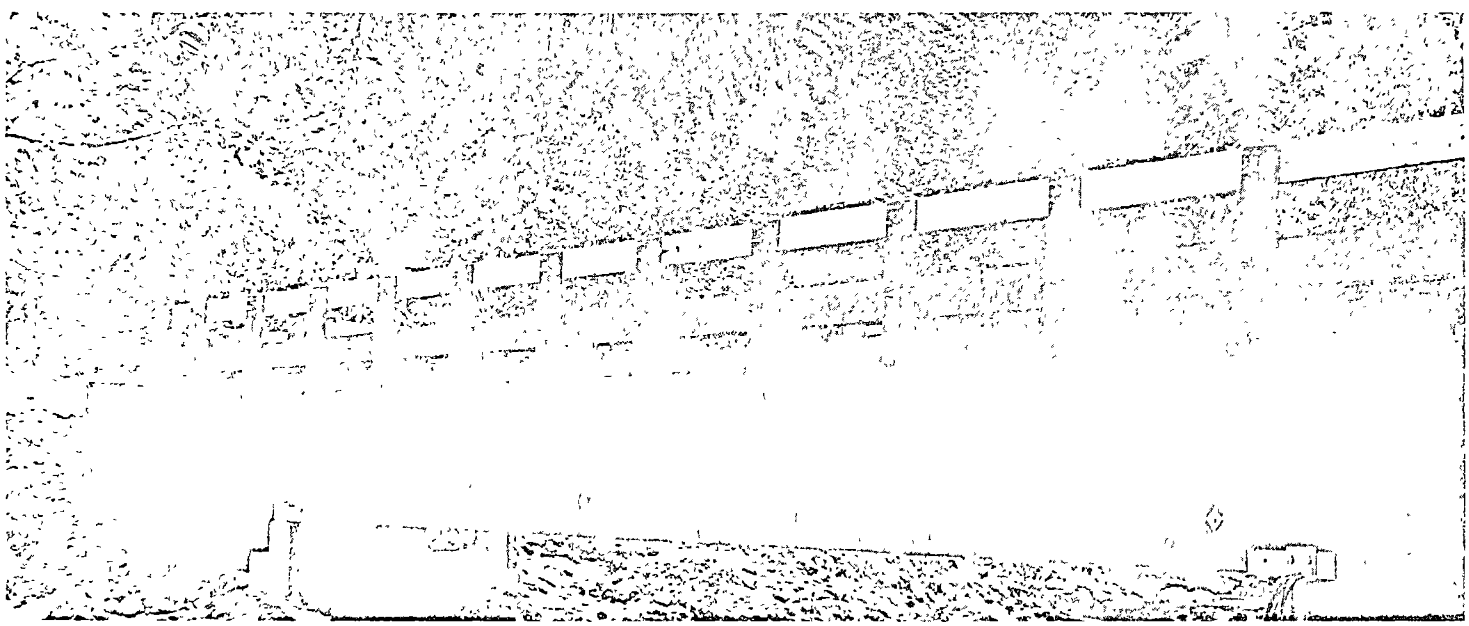
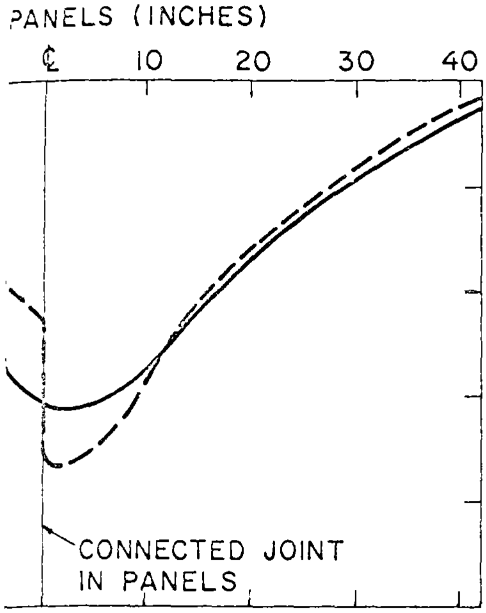


The Forest Products Laboratory and the Forest Service Division of Engineering have a continuing research program on timber bridges. This program covers evaluations of existing bridges, laboratory evaluations of new or revised design criteria, and studies of experimental bridges in the field. The objectives are to develop new design methods, better methods of preservative treatment of bridge members, improved deck systems, and improved methods to provide a watertight seal and wearing surface for the deck.

Field observations have shown that the bridge deck is the critical component with regard to deterioration. The most common timber bridge deck is an assembly that is constructed on site by nailing pieces of dimension lumber together and toe-nailing each lamination to the stringers. Shrinking and swelling of deck members due to changes in moisture content coupled with load vibrations tend to loosen the deck elements, allowing water to penetrate into both the wood deck and stringers. Research has indicated the possibility of glue-laminating dimension lumber into panel sections, making the deck more resistant to the penetration of water.

Glued panels are also far stiffer and have higher load capacities than equivalent thicknesses of nailed assemblies. A further advantage in glued-laminated construction is that most of the holes for securing the deck panels to the stringers can be drilled before preservative treatment, thus insuring effective preservative penetration. Figure 3 shows tests being conducted to develop design parameters for glued-laminated decks.

For erection purposes the practical width of glued-laminated panels is four feet or less. Therefore, it is necessary to devise connectors to transfer loads across the joints between panels. Seven joint-connector systems were evaluated in the FPL laboratory, and steel dowels proved to be the most effective. Figure 4 shows the effectiveness of steel



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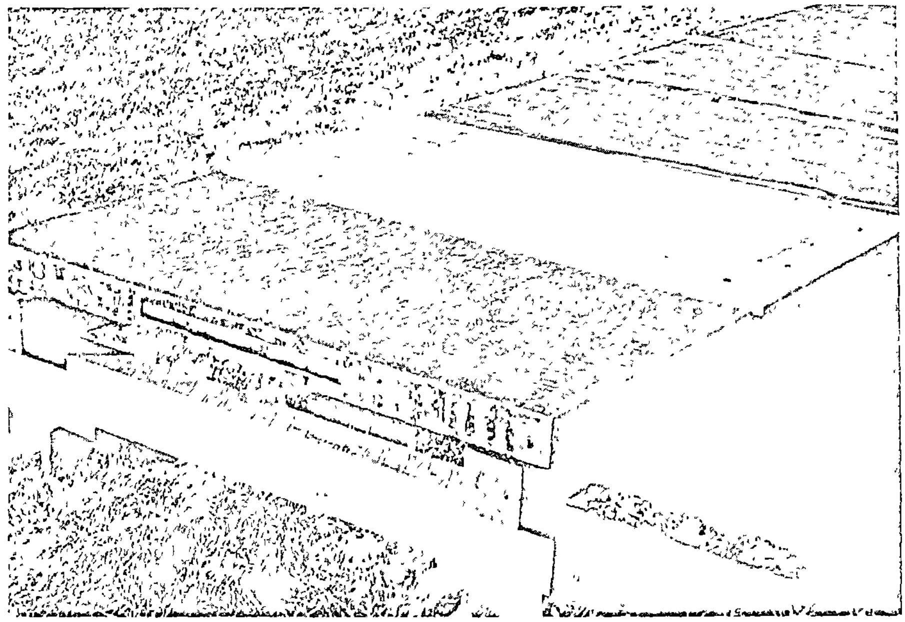
dowels compared to tapered, wood-spline connectors.

One potential problem in using wide panels for a deck system is dimensional stability, because bridge decks are subjected to numerous wetting and drying cycles that result in alternate swelling and shrinking. We hypothesized that pressure treating deck panels with creosote would greatly retard rapid changes in moisture content. To test this hypothesis six creosote-treated panels and two untreated panels were nailed to simulated bridge stringers (Fig. 5) and were exposed outdoors at Madison, Wis. After nearly 3 years of exposure, the maximum absolute change in



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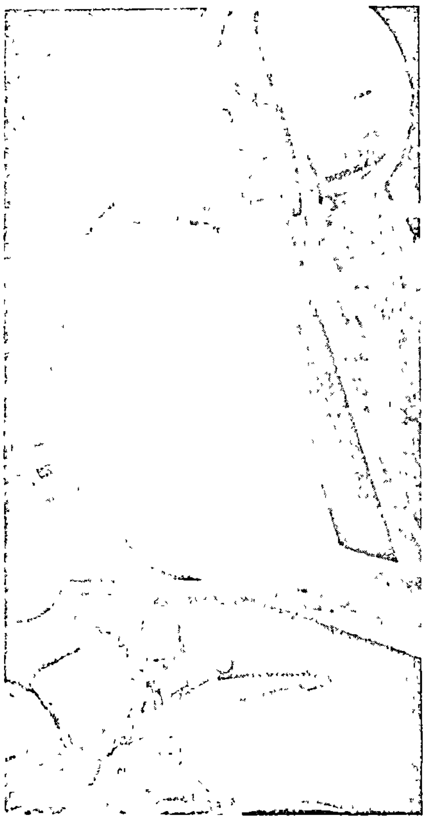

Preservative treating one of the few holes that required field drilling on the Au Train Bridge.

11

dimension for treated panels was about 0 4 percent while untreated panels changed by more than 1.0 percent.

To further evaluate the performance of glued-laminated panels, experimental test decks have been applied to six Forest Service bridges throughout the United States. One of these is the Smith Creek Bridge in the Gifford Pinchot National Forest in Washington which was designed to include several experimental features (Fig. 6). Selected stringer laminations received dual preservative treatment and the installation provides for a comparison of nailed-laminated and glued-laminated decks (Fig. 7).

The top three laminations in the glued-laminated stringers were given a dual preservative treatment. Before manufacture, they were treated with pentachlorophenol, and after manufacture they were treated again with a creosote/petroleum solution. The dual treatments resulted in almost 100 percent penetration of the preservatives, virtually eliminating future decay possibilities.

A new design concept was utilized in the recently constructed Au Train Bridge in the Hiawatha National Forest in Upper Michigan. This bridge is unique in that the glued-laminated deck panels are laid lengthwise to the roadway and no stringers are required under the deck. The bridge is 26 feet wide and 58 feet long. The panels span between pile supports located approximately 19 feet apart. Construction of the entire superstructure, excluding wheel guards and guard rails, was completed by Forest Service personnel in only 2½ days under adverse weather conditions. The various operations and construction sequences are illustrated (Figs 8-14).

Timber bridges are certainly economic structures well suited for many road systems. They give many years of low-maintenance service when properly designed and properly pressure treated with preservative. Advanced design concepts can and will contribute to making the timber bridge an even more efficient structure in the future.

The new dual lane Au Train Bridge.

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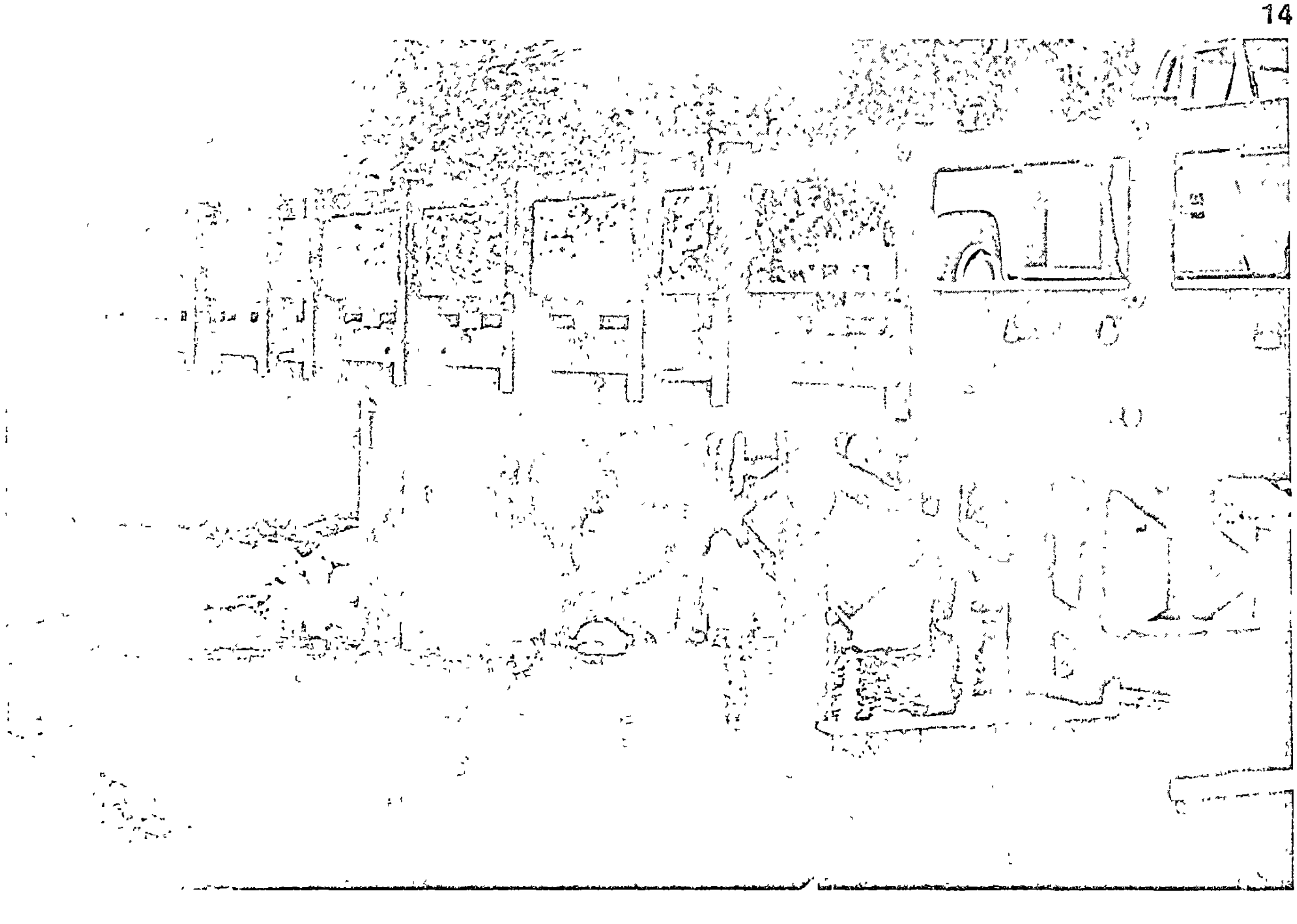
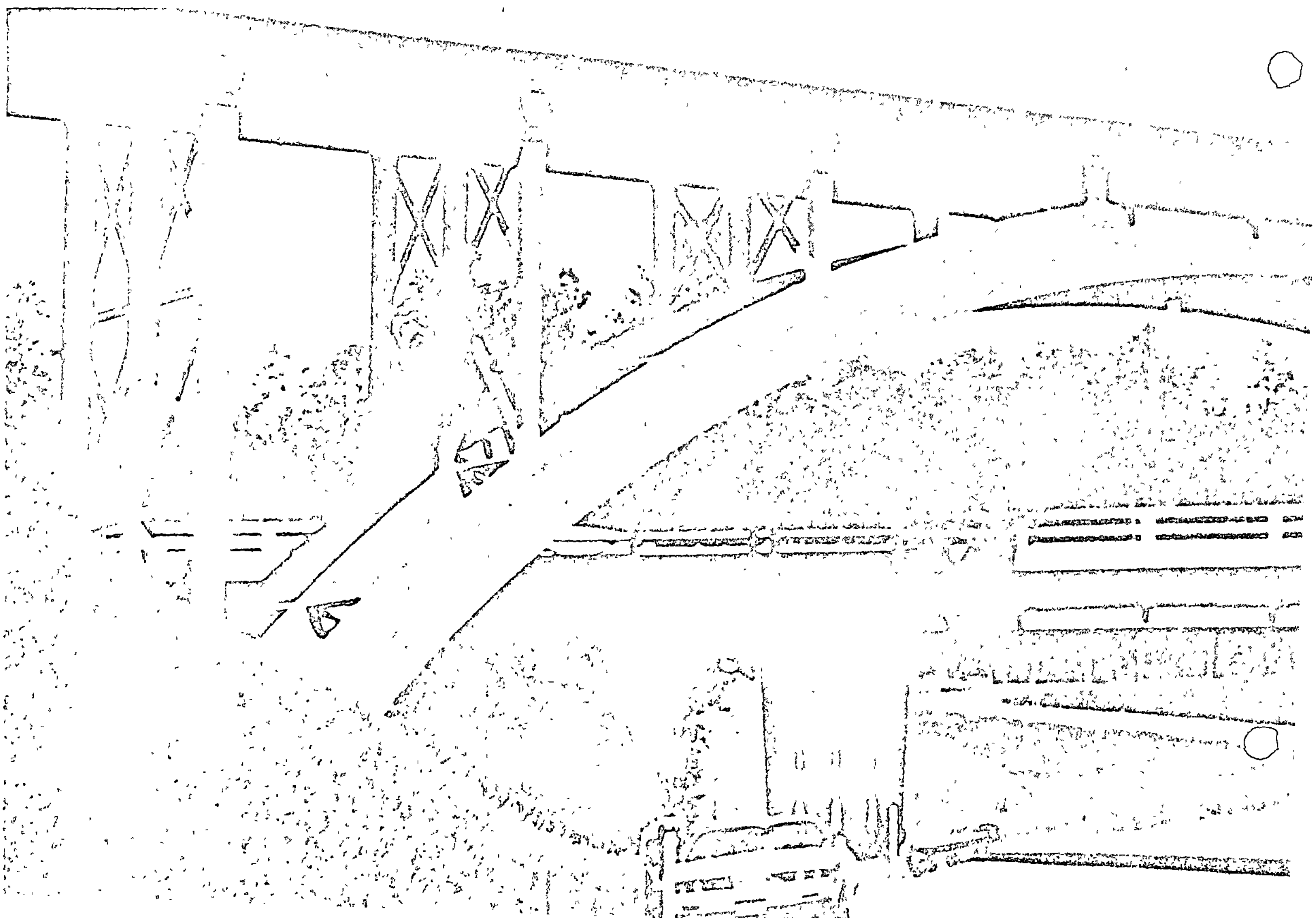


1 An unusual timber-truss bridge that supports heavy logging truck traffic.

2 Glued-laminated arches spanning 155 feet support the upper roadway of the Keystone Wye interchange.



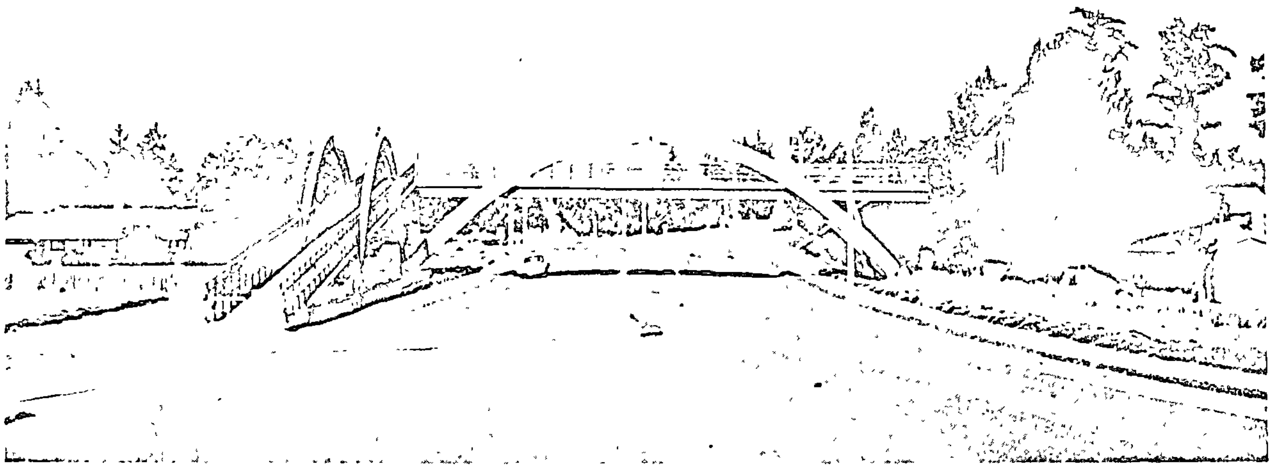


Figure 3.--A pedestrian crosswalk of glued-laminated arches. (M 140 028-22)

Figure 4.--Smith Creek bridge in the Gifford Pinchot National Forest. (M 138 865)

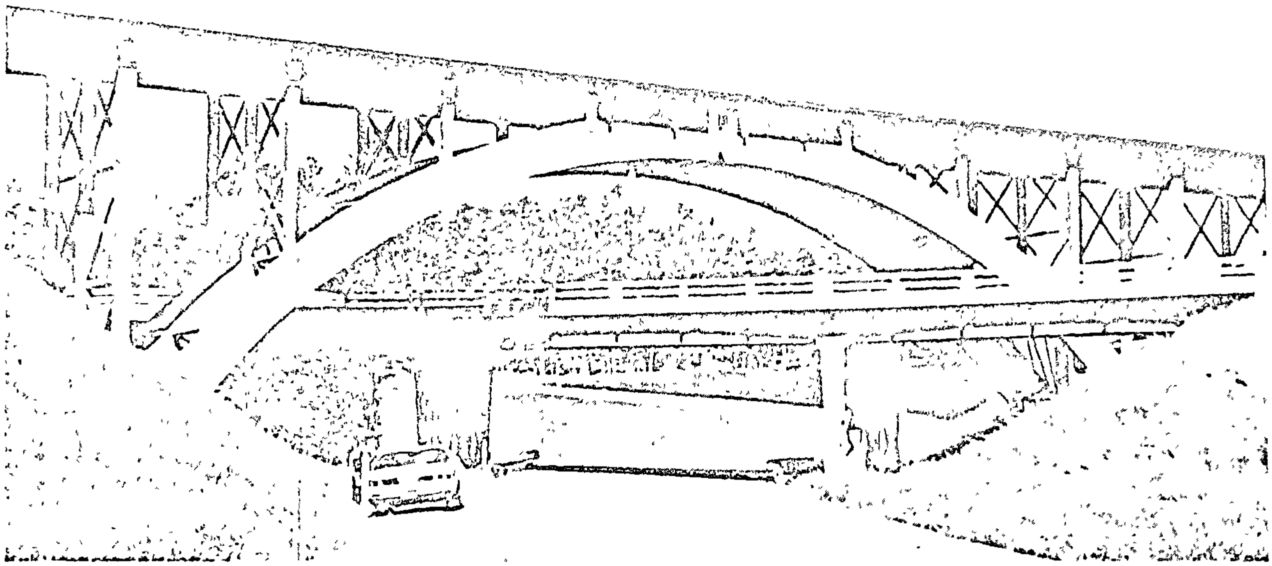


Figure 1.--Glued-laminated arches span 155 feet and support the upper roadway of the Keystone Wye Interchange in South Dakota. (M 138 863)

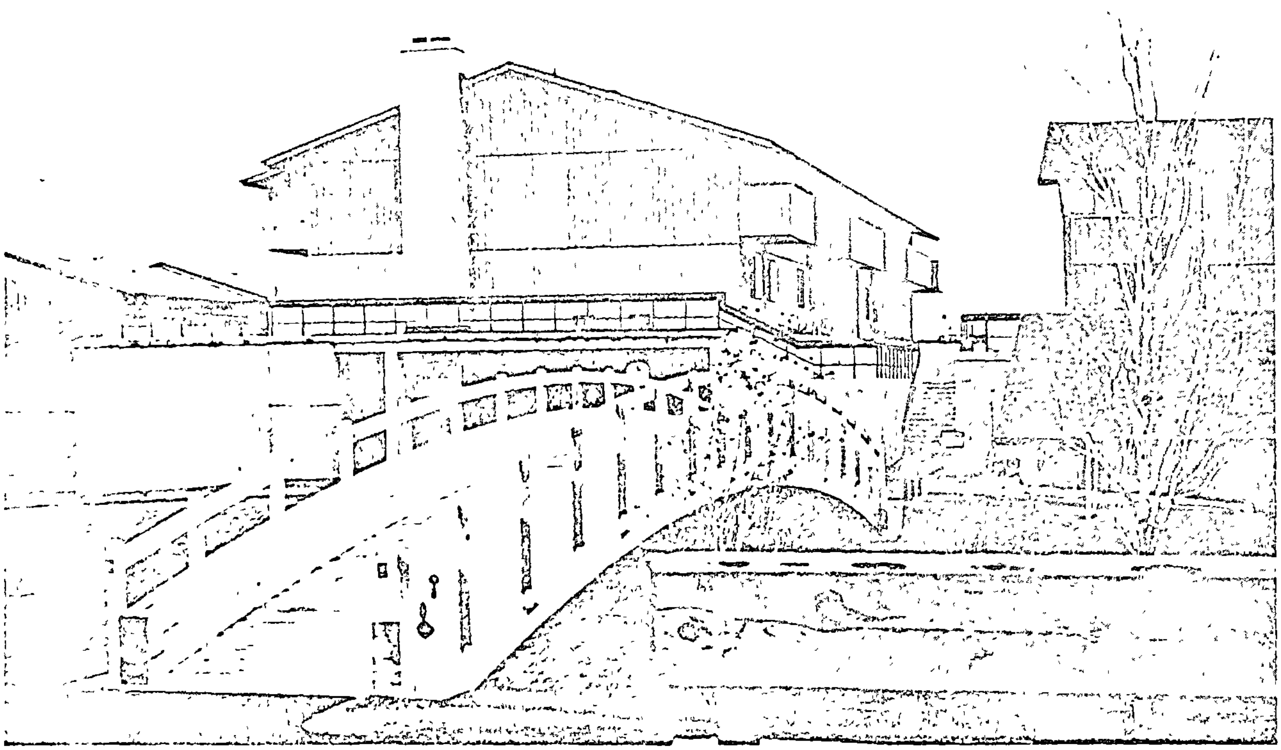


Figure 2.--A foot bridge of glued-laminated arches. (M 140 028-20)

Driving preservative-treated piles to support the new Au Train Bridge in the Hiawatha National Forest.

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Field treating ends of piles that have been cut to finished elevation during construction of the Au Train Bridge.

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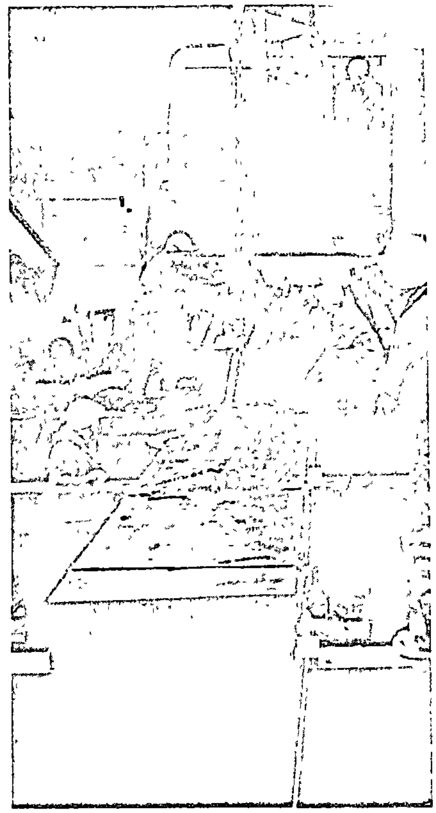

Placing the first glued-laminated deck panel to establish centerline of the Au Train Bridge.

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Steel dowels connecting Au Train Bridge deck panels.

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Pulling Au Train Bridge panels into position under extreme weather conditions.

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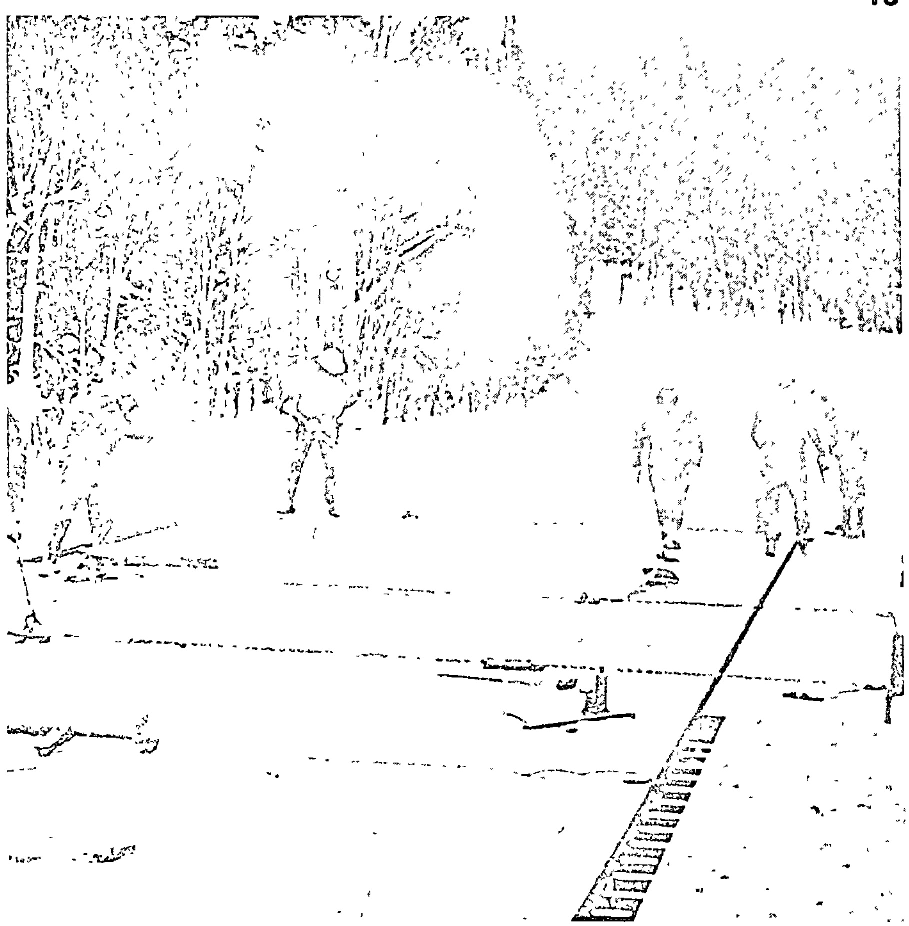

B R I D G E S

Robert Sexsmith
Visiting Professor
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Timber bridges can be very attractive in the short to medium span range, up to about 30 mts. In the U.S. and Canada there are about 1700 miles of timber bridges and trestles in service in the railway system. We shall discuss bridges in general, and then focus on the features of a composite timber concrete system that has particular advantages as a permanent short span structure.

A very simple timber bridge involves longitudinal girders made of laminated timber. Transverse floor beams span between girders. We may call this an "open a grid system". The structural action is simple. Span length is limited by the size of available laminated girders. Width is not really limited, but economy is best when sawn timber floor beams are used, possible up to two lanes. Analysis for flexure, shear, bearing and deflection is required. The real problems arise in the deck. On lightly travelled roadways a timber plank deck may be used.
Longitudinal planks span the floor beams. If girder spacings are small, cross beams are eliminated and transverse planking is used or a vertically laminated deck system with nail laminating. Problems occur in these system because of working of the joints and deck wear. Decay of the upper part of the stringers also occurs. Attempts at making a waterproof deck by asphalt paving usually fail because the asphalt eventually cracks.

Timbers must be well preserved. Often the top few laminations are treated with pentachlorophenol prior to laminating. The entire girder is then creosoted after laminating. Incising should be used for good penetration of creosote in many lumber species.
Some extensive research on bridge decks has been carried out. One possible deck element that has been tested in a number of field installations is the glued vertically laminated panel. 2" lumber is glued in the factory to form long panels with widths of about 1 or 2 m. These are rigid, fast to erect, and when properly treated will protect the stringers below from rot.

The advantage of these systems is very fast erection by local labor and low first cost. They cannot be used on heavly travelled roads because of deck wear problems. For heavy traffic, a concrete slab can be used instead of a wood deck.
To avoid costly formwork, and have a rigid system, the idea of the laminated timber deck is still attractive. A composite concrete - laminated timber deck has the advantages of both materials and no formwork.
A permanent 4 lane bridge on main the highway joining Seattle, USA and Vancouver, Canada, was constructed in 1962 using the concrete-timber composite deck. This highway receives heavy traffic. The structure is described here as an example of the kind of permanent bridge that could be used on major Mexican highways.
The following description represents the main idea of the structure, not an exact copy of the actual details and sizes used.

The substructure consists of timber piles with timber pile caps. There were about 10 spans of 9 metres each. When pile caps are in place, the laminated deck erection is begun. Only hand labor is required to place the 2 x 8 and 2 x 10 stringers. They are side nailed to each other, starting at one edge and working across. Butt joints are staggered, so that with 6 metre pieces, 9 metre continous spans are achieved.

When the laminated timber stringers are in place, a solid working deck is available. Steel reinforcement for shrinkage and for negative moment at supports is placed, and the concrete deck is poured. No formwork is needed escept at the sides and curb.

Fig. 1 shows a longitudinal and transverse section of the deck. Fig. 2 shows a typical stagger pattern for the butt joints. This must be planned as part of the design. The pattern shown results in section properties 2/3 of those for a solid deck of the same geometry because of the butt joints. The staggering of butt joints permits use of lumber shorter than the span and results in continuity over the support piers. To achieve shear transfer on the timber-concrete interface the top surfaces of the 2 x 8 and 2 x 10 stringers are dapped (about 1/2" deep, 3" long daps). This is done by the fabricator , not in the field. The 2 x 10's are also grooved to provide resistance to separation of concrete and timber. These details are shown in Fig. 2.

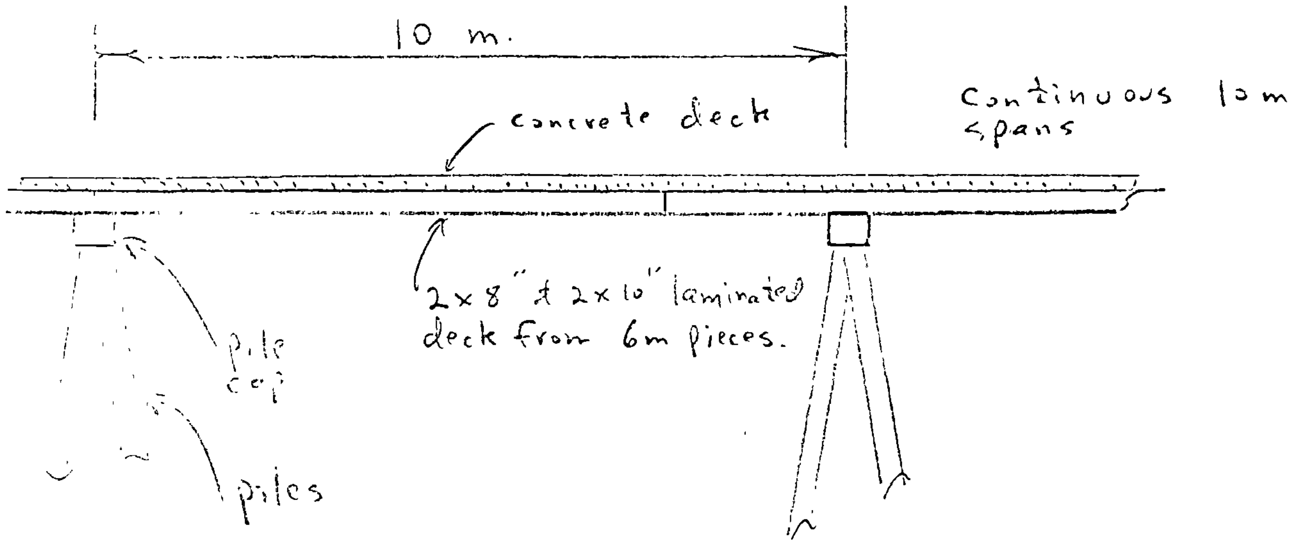
Spikes are about 4" galvanized and about 2 every meter are used. They pass through 2 laminations and partially into the third.

Design of the deck follows the usual considerations for composite construction. Use a fraction of the wood due to the joints (2/3 in this case). A transformed section is formed, and flexure formulae give concrete and timber stresses. Dead load is on the timber alone. Live load is on the transformed section.

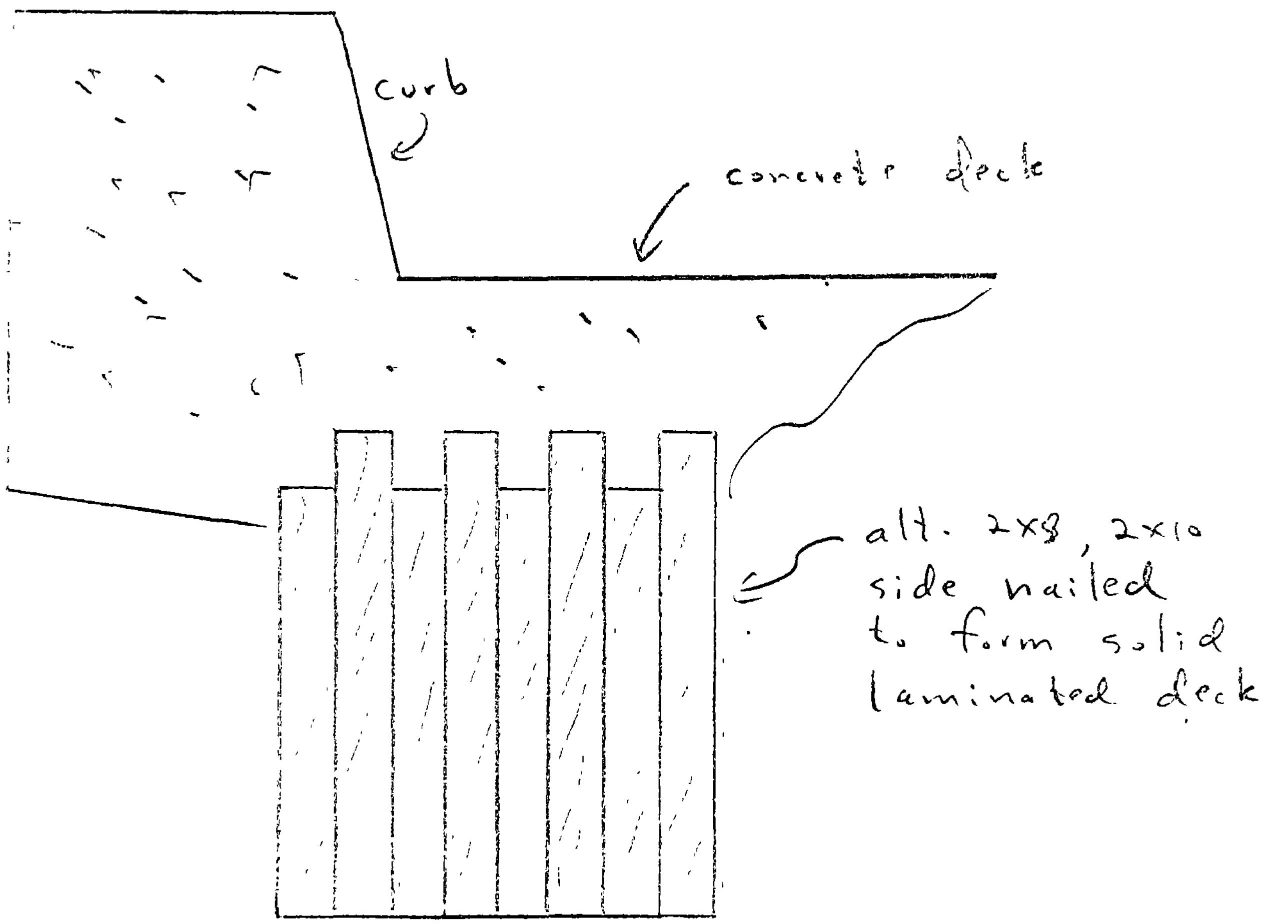
The daps will have to carry horizontal shear at the inter face.

The timber is preserved by pressure treatment with coal tar creo-gote after daps and grooves are formed.

The resulting deck is lighter than a concrete deck and concrete stringer system. It avoids field formwork. Its longevity is the same as that of a concrete bridge.



(a) Longitudinal Deck Section



(b) Transverse Deck Section

Fig 1

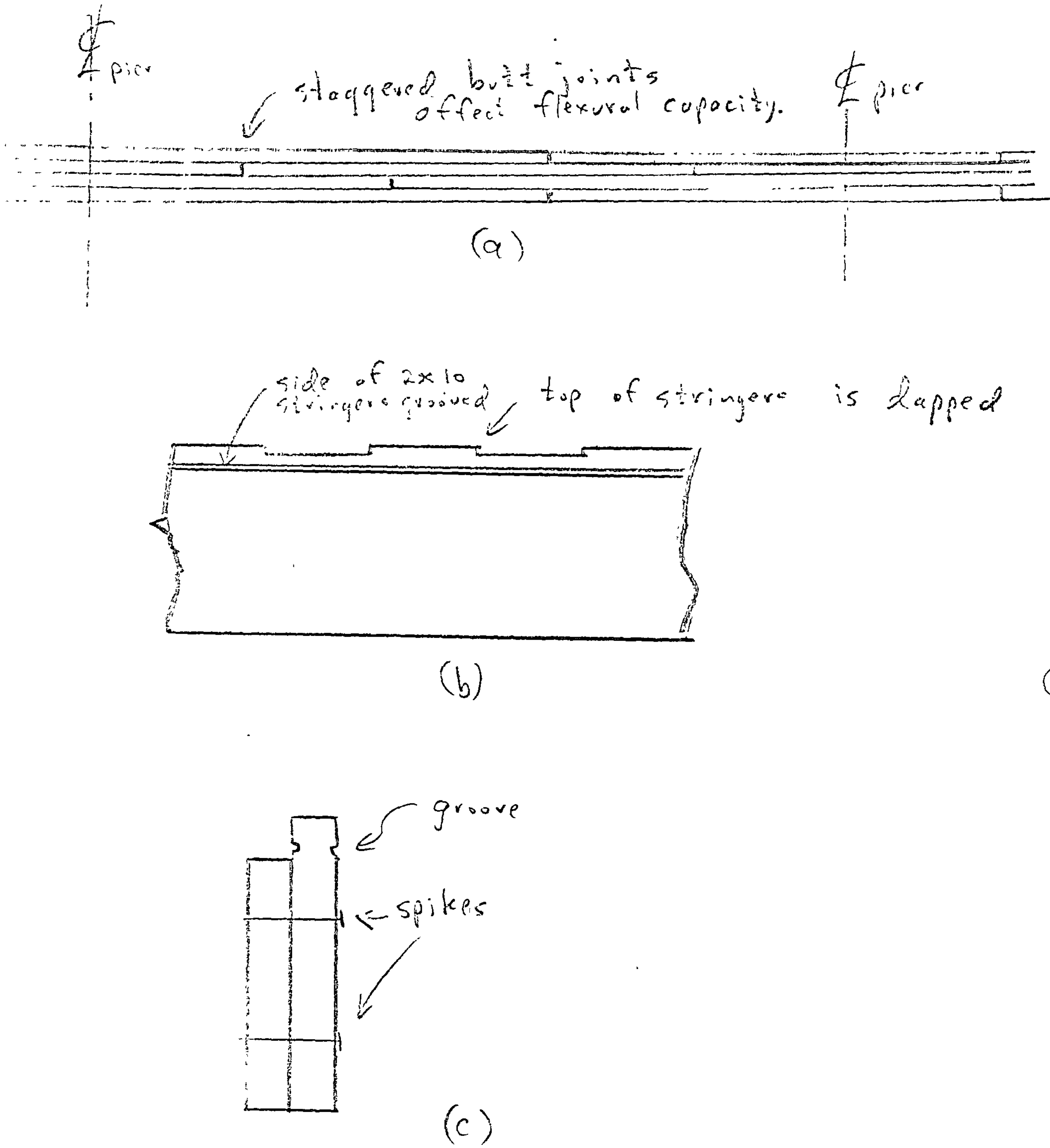


Fig 2 Details

R E F E R E N C E S

Timber Design and Construction

Timber Engineering Company, "Timber Design and Construction Handbook", McGraw Hill, New York.

Scofield and Obrien, "Modern Timber Engineering", Southern Pine Association (Southern Forest Products Assn), New Orleans, 1963.

Bohannan, "Alued Laminated Timber Bridges-Reality or Fantasy", Annual Meeting AITC, Arizona, 1972. (Reprinted with these notes).

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Southern Forest Products Assn
P.O.Box 52468
New Orleans, La 7015.

American Institute of Timber Construction
333 W. Hampden Ave,
Englewood, Colo. 80110

American Plywood Association
1119 A 5t.
Tacoma, Wash. 98401

A POST TENSIONED TIMBER SPATIAL STRUCTURE

Dr. R.F. Hooley, P.Eng.*

SUMMARY

The structure designed herein provides a column free area of 65 m by 305 m for the storage of potash. Twenty-four designs were done by computer to arrive at the economic solution of fifty-one parabolic glued laminated timber arches. Joists and plywood span between arches. Model and computer studies showed that the deep narrow arch section would fail by lateral torsional buckling at less than design load unless the roof deck was moment connected to the arch. This was achieved by passing ten cables 305 m long through holes in the arch. When these were post tensioned the resulting compression between arch and joist provided a moment connection so that when the rib tended to buckle laterally, the joists would have to rotate and so increase the critical load.

Wind on the flat end walls of the building was carried in part by bracing between arches and in part by the nailed plywood sheathing. The structure was analysed as an orthotropic shell with a low shear modulus generated by the nailed plywood roof. A comparison of the shell stiffness so found with the stiffness of the arch bracing gave the distribution of wind load to each component.

INTRODUCTION

The structure described herein had to provide a column free area of 65 m by 305 m with 19 m vertical clearance for the dry, unheated bulk storage of potash in Saskatchewan, Canada. The structure finally chosen for the job consisted of fifty-one parabolic three hinged glued laminated arches as shown in Fig. 1(a). A horizontal tie 6 m down from the crown not only stiffened the arch but also provided a platform for conveyors. Timber joists 4.1 cm by 24 cm spaced at 0.6 m span between arches. Plywood sheathing 1.59 cm thick is fastened to the joists with a waterproof membrane on top. The whole structure was nailed together and is shown during erection in Photo 1.

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Vancouver, B.C., Canada.

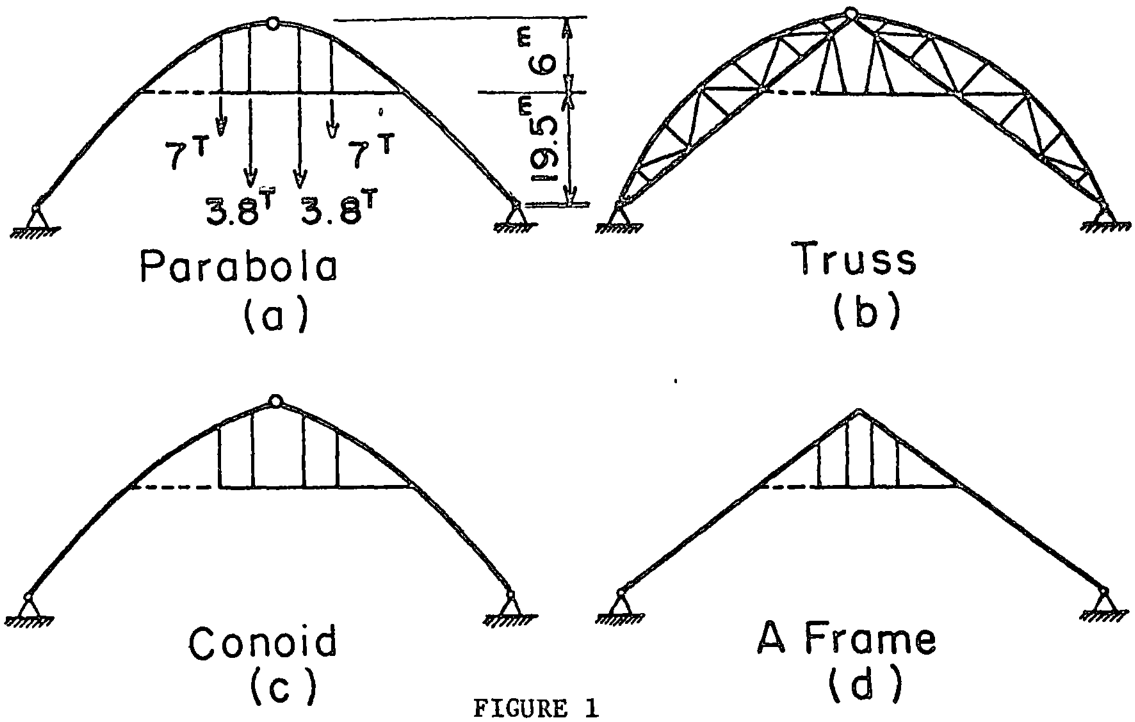


FIGURE 1

PRELIMINARY DESIGN

The National Building Code of Canada requires that structures of this type carry various combinations of dead load, wind, snow, and conveying machinery with the more improbable combinations having a higher allowable stress.

Analysis showed that the following loads governed design:

Dead Load	.0870 T/m^2
Snow Load	.101 T/m^2
Conveyor Dead Load	3.85 T/side (Fig. 1a)
Conveyor Traveller	7.06 T/side (Fig. 1a)

The governing snow load covered only half the structure and increased linearly from zero at the crown to maximum at the support with an average of .101 T/m. This was reduced near the supports because of the steep roof slope. The conveyor dead load of 3.85 T/side consisted of the dead weight of the conveyors, their load, and various walkways. The conveyor traveller however was like a travelling crane and each arch had to be designed for 7.06 T/side independent of the arch spacing. If the arch spacing were doubled, then all loads except the traveller would double. Hence, fewer arches at a greater spacing could be more economical. With this in mind then, the arches were designed for spacings of 6.1 m, 9.8 m, and 12.2 m.

The horizontal part of the tie shown dotted in Fig. 1 could either be omitted to produce a real three hinged arch or included to produce an indeterminate structure. The structure was designed both ways in search of an economic solution.

Since the parabola might not be the shape for minimum cost, the other three shapes of Fig. 1 were also designed with the same criteria as the parabola. These include inclined trusses, the conoidal shape and the A frame of Figs. 1(b), (c), (d). These four structures designed with or without a tie, and for three spacings then give twenty-four economic comparisons. Table 1 gives the total volume of material required per unit of area for the arches alone; that is the average thickness of material required over the plan area.

TABLE I

MEAN THICKNESS			
TYPE	SPACING meters	WITH TIE cm.	NO TIE cm.
Parabola	6.1	5.0	5.3
	9.8	4.3	4.6
	12.2	3.7	4.0
Truss	6.1	4.1	4.0
	9.8	3.0	3.0
	12.2	2.7	2.6
Conoid	6.1	6.8	6.6
	9.8	5.1	5.0
	12.2	5.0	4.9
A Frame	6.1	6.8	6.9
	9.8	5.1	5.2
	12.2	4.9	4.6

As expected, the arches required less material as the spacing was increased. This was due, in part to the constant traveller load and in part to the increased dimensions giving a greater efficiency for in plane buckling. However, when the increased cost of the joists between arches was included the smaller spacing of 6.1 m became in all cases the most economical. Although the truss system required the least volume of material, the multitude of joints required raised its total cost well above the other three. The savings in manufacturing the straight A frame were not enough to offset the extra material required so the choice reduced to the parabola or conoid. Of these two, Table 1 shows that the parabola at 6.1 m spacing, with a tie was the most economic structure. The above twenty-four designs were all done on the UNIVAC 1108 with the program DEGLU at a total cost of fifty dollars machine rental plus seven man days for data preparation. This program not only carries out a stress analysis but also designs each member for axial plus bending and shear for specified load combinations. At the end, it automatically calculates member dead loads and properties and repeats the analysis-design process for a prescribed number of iterations.

SIZE OF ARCH RIB

The governing load on the parabolic shape gave a design moment of 41.6 T-m with an axial thrust of 32.6T. A section must then be chosen to satisfy the interaction formula

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leqslant 1$$

where f_a = actual axial stress

F_a = allowable axial stress

f_b = actual bending stress

F_b = allowable bending stress

For the Douglas Fir used here the allowable bending stress was 194 Kg/cm^2 when duration of load for snow was included and the allowable axial was one-third of the critical axial buckling stress based on a modulus of elasticity of E = 126000 Kg/cm^2. A rectangular section 17.8 cm wide and 122 cm deep satisfies this condition as follows:

$$f_a = 32000\ Kg/17.8\ cm \times 122\ cm = 15\ Kg/cm^2$$

$$f_b = 41.6\ T\text{-}m/17.8\ cm \times 122\ cm \times 122\ cm = 95\ Kg/cm^2$$

$$F_b = 194\ Kg/cm^2$$

$$F_a = \frac{1}{3}\ \frac{\pi^2\ 126000\ Kg/cm^2}{12\,(43\,m/1.22\,m)^2} = 28\ Kg/cm^2$$

where the effective length for in plane buckling is taken as one-half the arc length. The interaction formula then yields:

$$\frac{15}{28} + \frac{95}{194} = 1.02$$

Cross sections of 22.8 x 110 and 28 x 103 would also satisfy the interaction criteria in the same manner. The increase in arch material of 17% and 33% respectively for these two sections when compared to the 17.8 x 122 made it economically desirable to use the deep narrow section.

LATERAL BUCKLING OF ARCH

At this point the design has considered the in plane buckling of the arch but not the lateral-torsional buckling. The extreme depth to width ratio of 122/17.8 = 6.9 requires a full analysis of this instability. This analysis was carried out by building a structure stiffness matrix K for the arch by adding together the member stiffness matrices k of twenty short straight bars so arranged to form a polygon along the real parabola. Since each of k contained lateral buckling stability functions, a plot of the determinate of K against the level of the external load gave the critical load at the location of zero determinate.

The basic member matrix k in member coordinates is given as follows:

$$k = \begin{array}{c} \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \end{array} \begin{array}{c} \begin{array}{cccccc} 1 & 2 & 3 & 4 & 5 & 6 \end{array} \\ \left[\begin{array}{cccccc}
\frac{12EIS_1}{\ell^3} & \frac{M}{\ell} - \frac{V}{2} & -\frac{6EIS_2}{\ell^2} & -\frac{12EIS_1}{\ell^3} & -\frac{M}{\ell} - \frac{V}{2} & -\frac{6EIS_2}{\ell^2} \\
\frac{M}{\ell} - \frac{V}{2} & \frac{GJ\chi}{\ell} & -\frac{V\ell}{6} & -\frac{M}{\ell} + \frac{V}{2} & -\frac{GJ\chi}{\ell} & \frac{V\ell}{6} \\
-\frac{6EIS_2}{\ell^2} & -M + \frac{V\ell}{3} & \frac{4EIS_3}{\ell} & \frac{6EIS_2}{\ell^2} & \frac{V\ell}{6} & \frac{2EIS_4}{\ell} \\
-\frac{12EIS_1}{\ell^3} & -\frac{M}{\ell} + \frac{V}{2} & \frac{6EIS_2}{\ell^2} & \frac{12EIS_1}{\ell^3} & \frac{M}{\ell} + \frac{V}{2} & \frac{6EIS_2}{\ell^2} \\
-\frac{M}{\ell} - \frac{V}{2} & -\frac{GJ\chi}{\ell} & \frac{V\ell}{6} & \frac{M}{\ell} + \frac{V}{2} & \frac{GJ\chi}{\ell} & -\frac{V\ell}{6} \\
-\frac{6EIS_2}{\ell^2} & \frac{V\ell}{6} & \frac{2EIS_4}{\ell} & \frac{6EIS_2}{\ell^2} & M + \frac{V\ell}{3} & \frac{4EIS_3}{\ell}
\end{array}\right] \end{array}$$

where

ℓ = member length

b = member width

d = member depth

$J = b^3 d(1 - .63\, b/d)/3$ = St. Venant's torsion constant

$I = b^3 d/12$ = weak axis moment of inertia

E = Young's Modulus of Elasticity

G = Shear Modulus of Elasticity = $E/15$ for Douglas Fir

$S_1 = 1 + P\ell^2/10EI$

$S_2 = 1 + P\ell^2/60EI$

$S_3 = 1 + P\ell^2/30EI$

$S_4 = 1 - P\ell^2/60EI$

$\chi = 1 + Pd^2/12JG$

The sign convention for the six degrees of freedom of k is given in Fig. 2

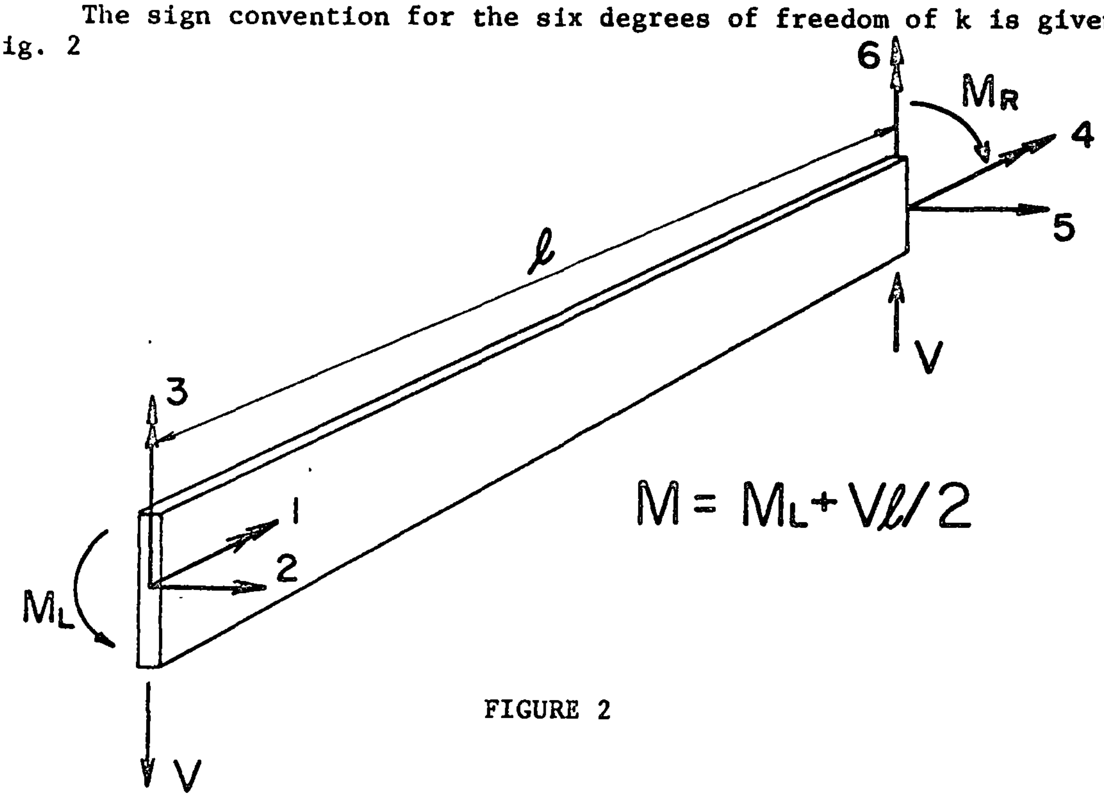


FIGURE 2

The terms such as $M/l - V/2$ * provide for lateral torsional buckling due to shear and moment while the terms S_i ** include the effect of axial load. Wagner buckling is contained in the factor χ **.

Since the deck of the structure described herein prevents lateral movement of the top edge of the arch rib it is necessary to modify k by transforming the degrees of freedom from the gravity axis to the top and setting the displacement one and four equal to zero. Transformation of this four by four matrix into the global coordinates of the arch and inclusion of the moment generated by concentrated external jointloads acting eccentric to the gravity axis finally yields the member matrix k**.

Computer calculations with this matrix on the 17.8 x 122 cross section showed that for half the arch loaded buckling occurred at 112 Kg/cm^2 bending stress while for the whole arch loaded buckling occurred at 22.5 Kg/cm^2 axial stress. A model of this arch was constructed of Douglas Fir to a scale of 1:18 and test loaded with several load combinations. The critical stresses as found by Southwell plots agreed with the theoretical calculations to within ten percent.

With the critical stresses now confirmed it is noted that since they are well below the elastic limit of the material, the commonly used factor of safety of three for timber buckling can be used to give allowable stresses as

$$F_b = 112 \text{ Kg/cm}^2/3 = 37.4 \text{ Kg/cm}^2$$

$$F_a = 22 \text{ Kg/cm}^2/3 = 7.5 \text{ Kg/cm}^2$$

The interaction formula now yields

$$\frac{15}{7.5} + \frac{95}{37.4} = 4.5$$

a value well above failure at design load.

The same analysis for the other two cross sections show

$$\frac{12.9}{13.7} + \frac{89.3}{66} = 2.3 \qquad 22.8 \times 110 \text{ cm}$$

$$\frac{11.3}{20} + \frac{84.5}{107} = 1.36 \qquad 28 \times 103 \text{ cm}$$

* Zavitz, B.A. "A Stiffness Matrix for Twist Bend Buckling of Narrow Rectangular Sections" M.A.Sc. Thesis, U.B.C., 1968.

** Charlwood, R.G. "Lateral Stability of Glulam Arches" M.A.Sc. Thesis, U.B.C., 1968.

In the last case the allowable axial stress from lateral buckling was 315 psi. Since this was greater than the allowable in plane buckling stress of 284 psi the smaller of the two was used.

For the support conditions encountered in this arch where the top edge is held in line, the National Building Code of Canada limits the depth to width ratio to between four and five for members under combined axial and bending. Even though the two wider sections comply with this rule, the interaction formula shows they have insufficient factor of safety. A study of the actual support of the top edge as shown in Fig. 3 indicates that it is not only held in line, but also partially restrained against torsional rotation.

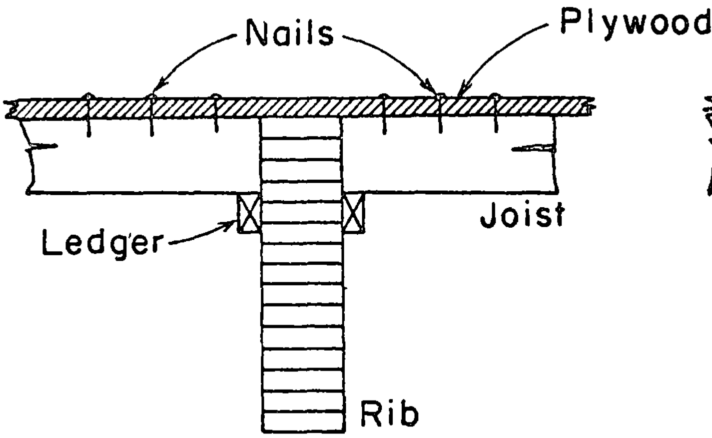


FIGURE 3

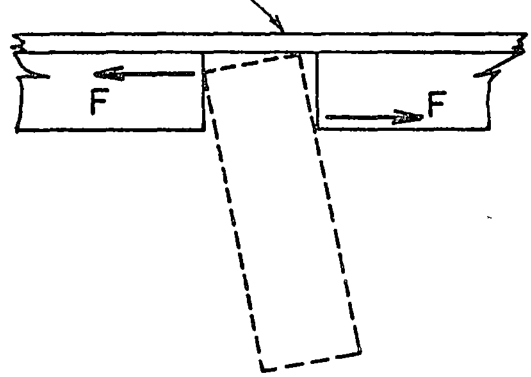


FIGURE 4

As the arch tends to buckle in torsion it will bear on the upper and lower edges of adjacent joists as shown in Fig. 4. The additional energy required to bend the joists will increase the buckling load so that the two wider sections are probably safe. It was decided though that this type of joint was too weak to restrain the 17.8 cm x 122 cm section which was overstressed by a factor of 4.5. The forces F of Fig. 4 induced by buckling would place the plywood deck in tension and the nails in shear. Splices in the plywood could occur near this joint and render it useless. Nail slip and local crushing at the points of bearing of the forces F increase the joint flexibility. Finally, shrinkage of the timber will produce a gap between the arch and joists so that no torsional restraint is generated until a finite rotation has taken place. A stiff moment connection was necessary between the rib and joists but the construction of 7000 such individual connections was uneconomic. At this stage it was decided instead to post tension the structure with 1.27 cm diameter neoprene covered steel cables. Ten of these cables, each 305 m in length, spaced equidistant along the rib ran the full length of the building. They passed through holes drilled in the rib as shown in Fig. 5 so that when they were tensioned to 9 T each, a uniform bearing compression of 6.5 Kg/cm^2 was induced between rib and joist. Photo 1 shows some of the anchor points for the cables and the splayed members required to transfer the tension uniformly to the joists. The torsional moment which can be transferred to the rib by a reduction in this bearing compression is .026 T-m per joist. The total of such moments over 15 m of the arch length is about 3% of the total bending moment in the rib. Since this is the same order of magnitude as the moment generated by a 4 cm initial lateral eccentricity over a 15 m length, the joint so formed was judged to be sufficiently strong and stiff.

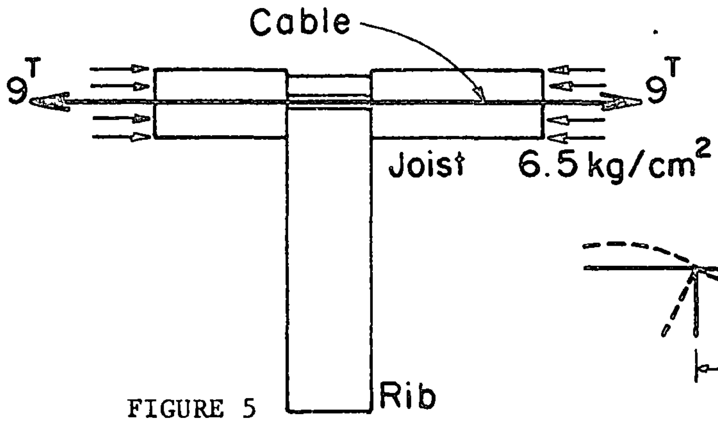


FIGURE 5

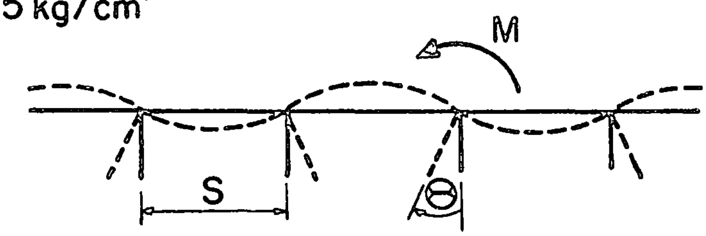


FIGURE 6

If the arch were to buckle laterally with the stiff joint it would bend the joists as shown in Fig. 6. A torsional rotation would then generate a restraining couple $\bar{M} = \bar{K}\theta$ where $\bar{K} = E\bar{b}\bar{d}^3/3Ss$

Herein

E = 126,000 Kg/cm^2

$\bar{b}$ = joist width = 4.1 cm

$\bar{d}$ = joist depth = 24 cm

S = arch spacing = 6.1 m

s = joist spacing = .61 m

so that $\bar{K}$ = 66 T/radian. A stiffness analysis could now be carried out to find the critical buckling stress with this torsional spring in the system. It was only necessary to multiply the spring constant by the member length, transform to global coordinates, and add into the appropriate diagonal terms of the structure matrix. Numerical calculations then showed that for half the arch loaded the critical bending stress increased by a factor of 7.2 to 810 Kg/cm^2. Since this is beyond the average ultimate stress for the material, the full allowable bending stress may be used with no reduction for lateral buckling. Further calculations with the arch fully loaded showed that the critical axial stress increased by a factor of 16.5 to 372 Kg/cm^2. Since the critical stress for in plane axial buckling is only 85 Kg/cm^2 the smaller figure governs and lateral buckling is again no problem. The initial design check of the 17.8 cm x 122 cm is then satisfactory with the interaction formula yielding 1.02 as shown previously. The additional cost of the cables was a small fraction of the material saved in using a deep, narrow cross section.

WIND BRACING

Wind acting normal to the plane of the arches created a positive pressure on one flat end wall and a negative pressure on the other. This generated a force of 108 T on each end wall, both forces acting in the same direction. Vertical beams in the end wall distributed half of this directly to the foundation but the other half placed an axial edge load of 54 T on each end of the parabolic cylinder. It is normal in structures of this kind to provide braced bays both to aid in erection and to carry this axial wind load. Each braced bay consists of eleven panels of struts and diagonals between two adjacent archs to form a space frame. Analysis of the five braced bays of this structure with a space frame stiffness program showed that they were strong enough to resist the wind load but that they deflected 28 cm normal to the plane of the arch at the crown. This 28 cm deflection over the half arc length of rib of 43 m means an average relative slip between adjacent 2.4 m sheets of plywood of 1.6 cm. Either the nails in the plywood sheathing give way to yield a leaky structure or the nails and plywood carry the wind load instead of the braced bays. It is the authors opinion that many reticulated shells leak for this reason. To overcome this, a more precise analysis is required which considers the stiffness of the shell under axial loads.

Fig. 7 shows the shell under its wind load where x and y are the coordinates of an element under the membrane loads N_x, N_y and N_{xy}. Axial and tangential displacements are U and V. Since it is assumed that the load is distributed by membrane action only, the necessary elastic constants of this orthotropic sheet are defined as follows:

$$N_x = D_x \epsilon_x$$
$$N_y = D_y \epsilon_y$$
$$N_{xy} = D_{xy} \epsilon_{xy}$$

The axial stiffness D_x will be the joist area per unit length times its elastic modulus with the plywood neglected because of its relatively flexible nailing. This yields

$$D_x = 24 \text{ cm} \times 4.1 \text{ cm} \times 126000 \frac{\text{kg}}{\text{cm}^2} / 61 \text{ cm} = 20300 \text{ T/m}$$

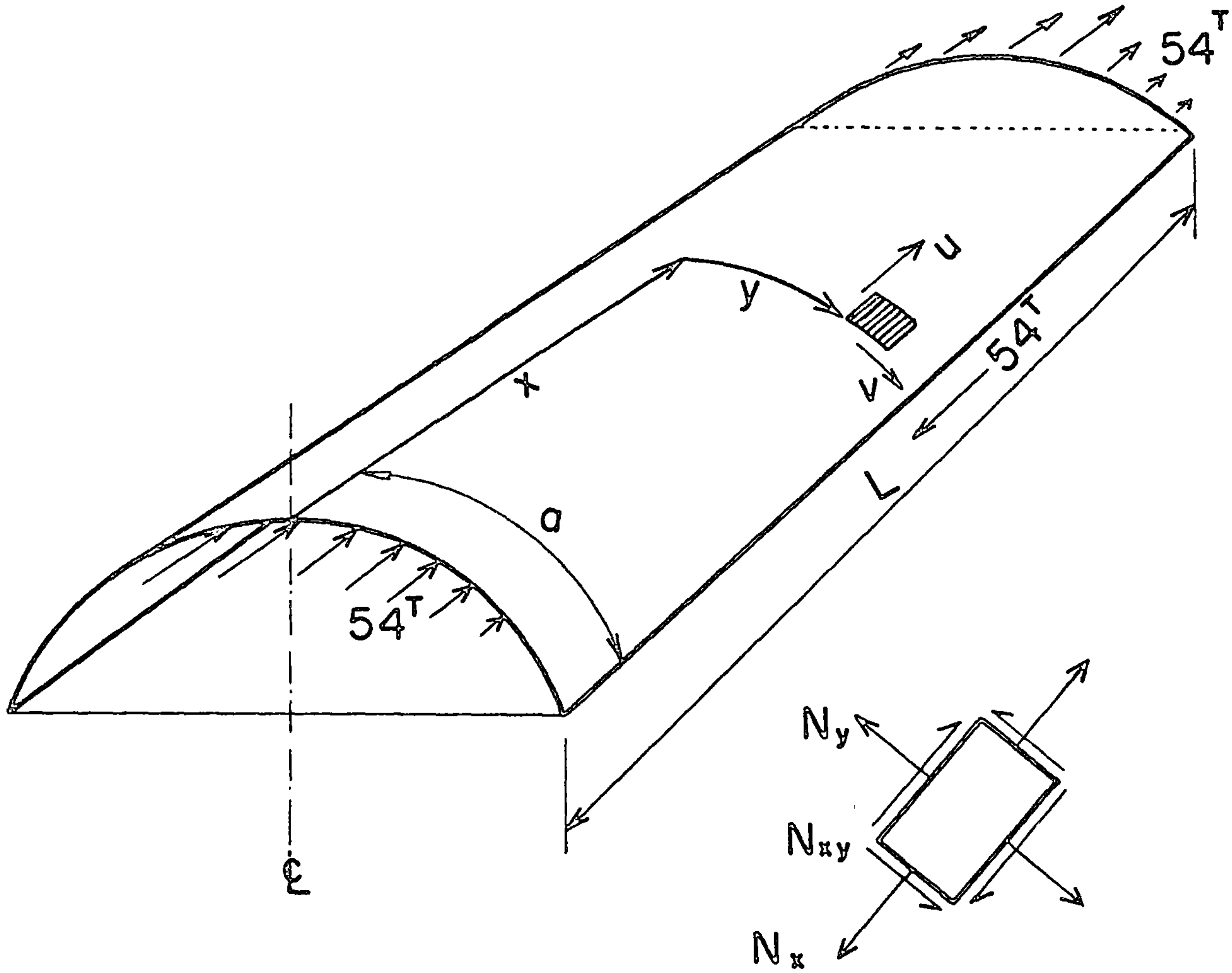


FIGURE 7

Since these joists bear in the flexible perpendicular to grain direction against the side of the rib, experiments were carried out to show that this value of D_x should be reduced by 55%. Thus

$$D_x = .45\,(20300) = 9100 \quad T/m$$

The stiffness D_y will be the arch rib area per unit length times its modulus or

$$D_y = 17.8\,cm \times 122\,cm \times 126000\,Kg/cm^2 / 6.1\,m = 43500\,T/m$$

The shear stiffness D_{xy} will be the shear modulus of the plywood times its thickness or

$$D_{xy} = 126000\,Kg/cm^2 \times 1.59\,cm / 15 = 1300\,T/m$$

The shear flow N_{xy} is transmitted between adjacent sheets of plywood by joists and nails in single shear as shown in Fig. 8. The concentrated shear force per joist is carried by a cluster of several nails at the edge of the plywood. Experiments with this system showed that, at the shear flows expected, nail slip would reduce this stiffness by 80%. Thus

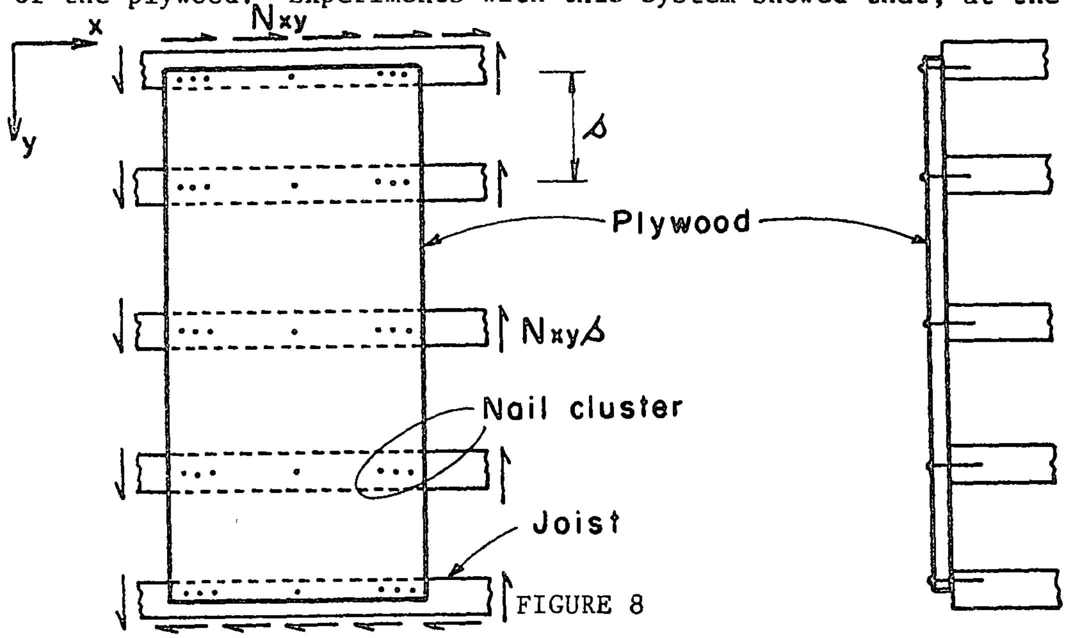


FIGURE 8

$$D_{xy} = 0.20 \times 1300 \text{ T/m} = 260 \text{ T/m}$$

With the relatively large D_y and the small loads expected in this direction it was assumed that the displacement v was negligible. Thus the strains of interest are

$$\epsilon_x = \partial u / \partial x \qquad \epsilon_y = -\partial u / \partial y$$

Finally, equilibrium of the element yields

$$\partial N_x / \partial x - \partial N_{xy} / \partial y = 0$$

The governing differential equation becomes

$$D_x \, \partial^2 u / \partial x^2 + D_{xy} \, \partial^2 u / \partial y^2 = 0$$

and its solution is

$$u = \frac{F}{4\sqrt{D_x D_{xy}}} \left[\coth\left(\frac{\gamma L}{2}\right) \cosh(\gamma x) - \sinh(\gamma x) \right] \cos\left(\frac{\pi y}{2a}\right)$$

$$N_{xy} = \frac{+\pi F}{8a} \sqrt{\frac{D_{xy}}{D_x}} \left[\coth\left(\frac{\gamma L}{2}\right) \cosh(\gamma x) - \sinh(\gamma x) \right] \sin\left(\frac{\pi y}{2a}\right)$$

where $\gamma = \frac{\pi}{2a} \sqrt{\frac{D_{xy}}{D_x}}$

$L =$ total length of structure $= 305$ m

$a =$ half the arc length of arch $= 43$ m

$F =$ total axial load $= 109$ T

The boundary conditions used herein are:

(1) @ $y = 0$ $\quad \partial U/\partial y = 0$

(2) @ $y = a$ $\quad U = 0$

(3) @ $x = L/2$ $\quad \partial U/\partial x = 0$

(4) @ $x = 0$ $\quad \int_0^a N_x \, dy = -F/4$

This last condition assumes that the load is distributed as a cosine function along the edge. This is reasonable since the parabolic arch shape distributes the edge load in a parabolic manner.

Numerical calculations based on these formulas show that the maximum is of course along the supported edge $y = a$ and varies from .227 T/m at the ends to .154 T/m at the midpoint. Thus the maximum is only 28% larger than the average of 108T/2 x 305 m or .178 T/m. The maximum value of U occurs at the origin and equals 2.4 cm.

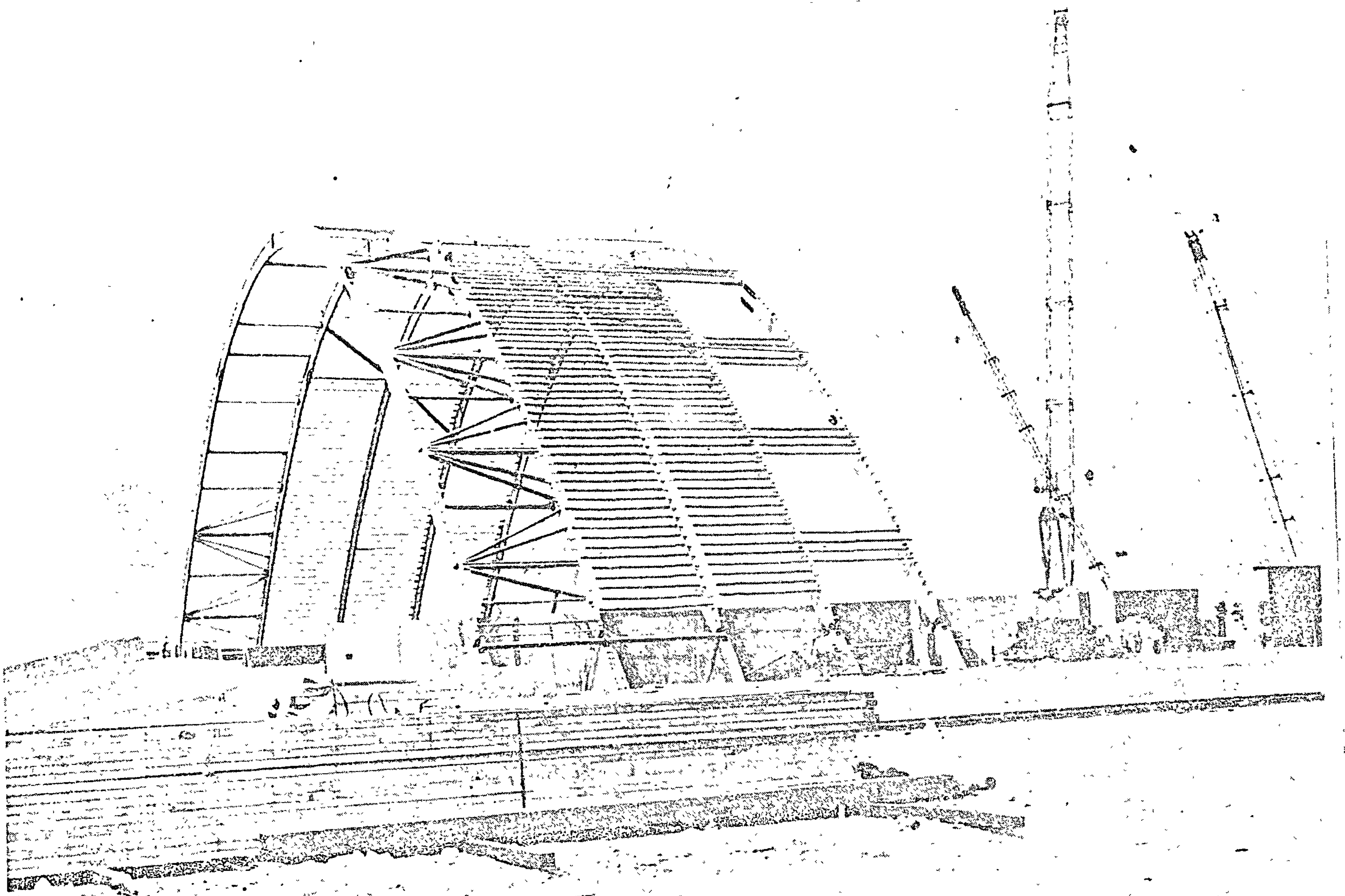
These figures are based on the assumption that the shell carries all the wind load. Since the braced bays and shell deflect 28 cm and 2.4 cm respectively under the same load, then the shell will carry a fraction 1/(1 + 2.4/28) = .92 of the total load. The maximum N_{xy} will become 0.92(.227 T/m) = .208 T/m. The nail clusters of Fig. 8 then must carry a load of .208 T/m(.61 m) = .127 T. This required four nails per cluster with a 32 Kg allowable load per nail. Since it was found impractical to drive these four nails without splitting the joists it was decided to stiffen the braced bays so they would attract more load. This was done by placing four additional struts in the structure. Each strut ran from the third point of the braced bay to the foundation by spiralling between five arches. Photo 1 shows one such strut partially in place on the far left side of the building. A space frame analysis of these five arches, the strut, and the braced bay showed a maximum axial deflection of 4.3 cm under 108 T wind instead of 28 cm with no strut. The fraction of load now carried by the shell becomes 1/(1 + 2.4/4.3) = 0.64 so that a maximum of three nails per cluster was required. This joint was practical to build, had sufficient strength, and generated a building stiff enough to prevent the waterproof membrane from failing.

CONCLUSIONS

This structure was completed in 1967 and in the succeeding six years has withstood temperature changes from -40°C to +40°C and wind velocities of 120 km/hr without discernible problems.

SYMBOL TABLE

a	= one half arch arc length
b	= arch width
$\bar{b}$	= purlin width
d	= arch depth
$\bar{d}$	= purlin depth
D_x, D_y, D_{xy}	= shell membrane stiffnesses in respective directions
E	= Young's Modulus of Elasticity
F	= wind force on building ends
f_a, f_b	= actual axial and bending stress
F_a, F_b	= allowable axial and bending stress
G	= shear modulus of Elasticity
I	= weak moment of inertia of arch
J	= St. Venants Torsional constant
K	= structure stiffness matrix
k	= member stiffness matrix
$\bar{K}$	= purlin moment stiffness
ℓ	= member length
L	= building length
M	= moment at centre of arch member
M_L, M_R	= moment at ends of arch member
$\bar{M}$	= purlin moment
N_x, N_y, N_{xy}	= shell membrane stresses in respective directions
P	= axial force in arch member, tension positive
s	= purlin spacing
S	= member spacing
S_i	= stability functions
u	= shell displacement in x direction
v	= shell displacement in y direction
V	= shear force in arch member
x	= coordinate along shell length
y	= coordinate across shell
$\epsilon_x, \epsilon_y, \epsilon_{xy}$	= shell membrane strains along respective directions
λ	= constant in shell differential equation = $\pi\sqrt{D_{xy}/D_x}/2a$
θ	= angle of rotation of arch
χ	= Wagner torsional stability term



The New Role of Timber in Large Building Projects

By L. E. GOWER, P.Eng.

On the frozen shores of Ungava Bay in Northern Quebec, one of the world's largest timber buildings provides storage for over 9,000,000 cubic feet of asbestos fibre, awaiting summer ocean shipment

At Cassiar, in Northern British Columbia, asbestos fibre is processed in a seven-storey mill building constructed of timber.

Ninety miles from Prince George, a massive logging bridge spans the turbulent McGregor River. This bridge, too, uses timber for most of its structural components.

Our consulting engineering firm of Gower, Yeung and Associates is very pleased to have had the opportunity of serving as design consultants for these three unique structures, in addition to several other timber projects, equally interesting and impressive.

Why Timber?

The use of timber in these types of structures is relatively rare. If, as is apparent from the success of these projects, laminated timber deserves an important place in large industrial construction, why do so few designers select this material? The answer is fairly simple. The structural use and acceptance of laminated timber is relatively new. Most designers have not been trained in its use and feel uncomfortable when attempting to select structural shapes and to detail connections. Laminated timber, when used to good advantage, is usually cheaper than steel, but many circumstances could alter this situation. If the designer has limited timber engineering expertise and fails to find an economical design solution, preliminary cost studies might well suggest that steel would be the more suitable construction material. Unless a marked saving can be seen, it is not likely that the designer will select an unfamiliar structural material.

A second, and largely unfounded concern, is that a timber structure will not be as fire-safe as a steel building. Insurance premiums may be more expensive, indicating a similar concern by insurance underwriters.

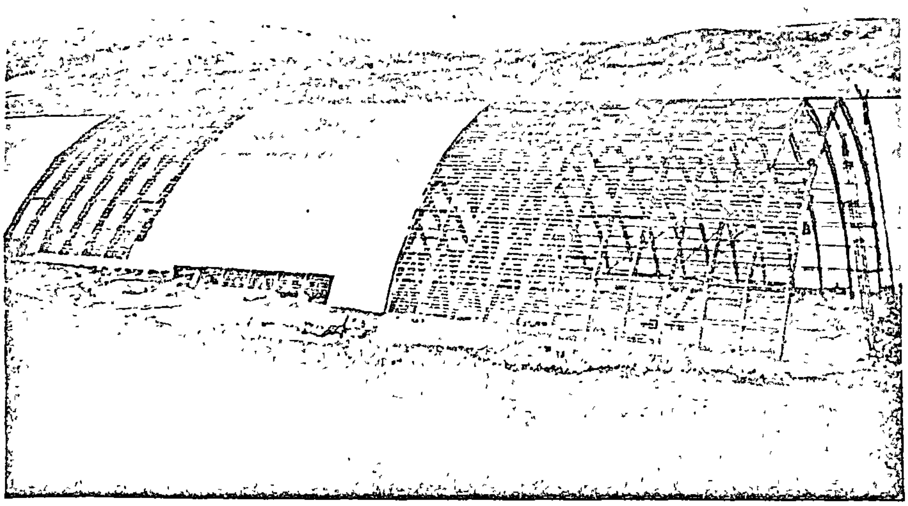


Fig 1 Part of the asbestos storage building built recently at Ungava, Quebec (sloping end wall segments not yet erected)

The Author

Mr Gower is president of Gower, Yeung & Associates Ltd, New Westminster, B C, a consulting engineering firm specializing in the structural design of buildings and bridges

Mr Gower was formerly Chief Engineer for Amfab Products Ltd, a prominent Canadian timber laminating firm, which has been responsible for the design and supply of many of the large industrial timber buildings throughout Canada These have included mills for the forest and mining industries and large warehouses and storage facilities, particularly used for potash storage

L.E Gower has been very active on various timber engineering and code committees, and has been one of those responsible for the present standards of manufacture and design of laminated timber in Canada

What, then, are the attractions of timber that led us to select it for such important structures as those mentioned?

There are many factors which influenced us in our decisions:

(a) Previous experience in industry permitted us to become thoroughly familiar with all facets of timber design and construction. We thus could take a more objective look at innovative timber concepts.

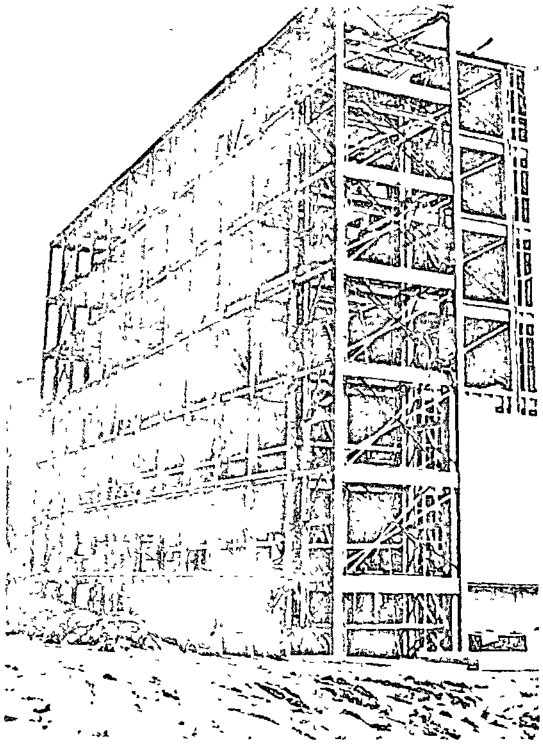


Fig 2 The Cassiar Asbestos mill building, Cassiar, B C

(b) We have learned that the best use of timber usually involves extensive use of steel as well. Actual, the term, "timber"' construction is a misnomer. It should be, "steel and timber" construction. The main choice is in the relative amounts of each of these materials. In our structural designs, we select whichever material does the job best, after all aspects of cost and performance have been considered. Some designs employ relatively large amounts of steel.

What characteristics usually influence the choice of materials? At the usual pricing levels, timber proves to be more economical than steel in most bending applications, with a possible exception of mass-produced steel bar joints. In tension members, steel will often prove to be more suitable, even though a strict comparison of the material in the main cross-section seems to favour timber. It is the cumbersome and rather expensive end connections which usually make timber less desirable as a tension member. One should also consider the relative strength reliability of each material. In small timber cross-sections, the consequence of errors in grading can be more serious than in large sections. My own preference is to use steel rather than timber for most highly stressed tension members. Compression members usually can be shown to be cheaper in timber, although frequently steel might be selected for other considerations. It might well be that the ease of designing the end connections and the better resistance of steel to abuse for traffic will weight the choice in favour of the steel.

(c) One of the most serious, but commonly overlooked factors influencing the total cost of a structure, may be that of erection. Not only does one have to consider the availability of materials, ease of installation and onsite fabrication, but even such subtleties as possible union conflicts and restrictive contract regulations that impose unrealistic employment demands.

In almost all of these considerations, wood usually proves to be the preferable material. Experienced erectors find that timber structures usually are very simple to erect and seldom lead to union jurisdiction problems.

Some Interesting Timber Structures

The immense storage building in Northern Quebec (Figure 1) was a most interesting project with several unique problems to be solved. The purpose of the building is to store asbestos fibre manufactured throughout the year but able to be shipped only during an ice-free period of four to five months The need to accommodate a year's supply of fibre required a clearspan building 300' wide and 750' long. Future plans are to increase the length by another 300' or 400'. To help visualize the size of the present building, picture five American football fields placed side by side. This building would cover these five fields. At least from the point of view of timber content, this building is likely to be the largest timber building in the world. There are several similar buildings in Saskatchewan, storing potash, which cover about the same area but they are narrower and longer and do not have the massive members required for the Ungava project.

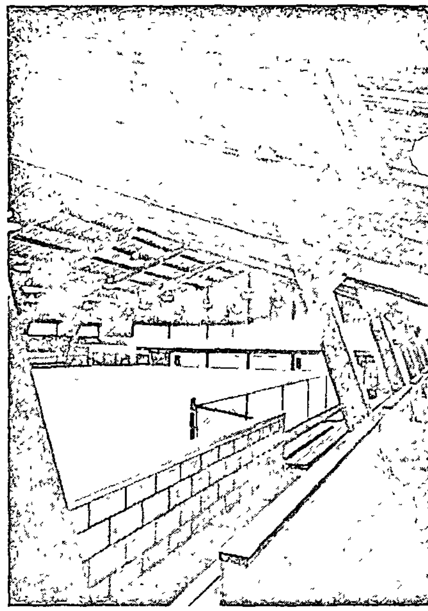


Fig. 3 Victoria Park Arena

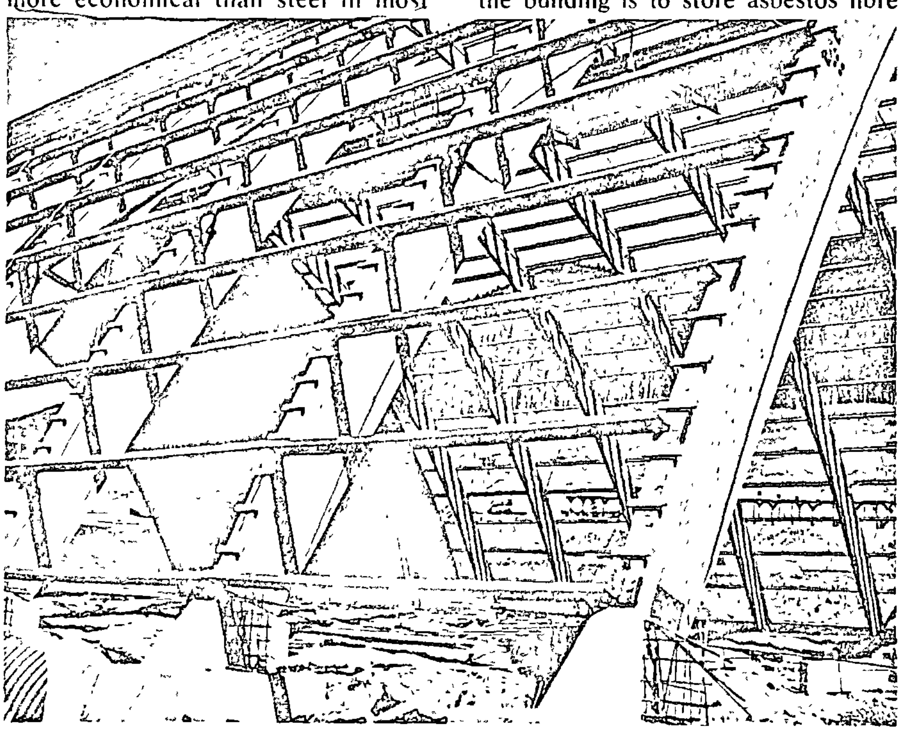


Fig 4 Part of the coal storage building, Fording Coal Company near Fernie, B.C.

The arches for this building are constructed of laminated timber in an "I" section They measure approximately 6' deep. Shipping restrictions limited the length of members to a maximum of 80'. Since the individual half-arches measured up to 224', two splices were necessary in each length To permit practical sizes for the splice members, the arch segments were deepened at the splices, which seriously complicated the geometry of the structure. The irregular rocky terrain proved to be costly to grade level, so it was necessary to accommodate the building to a sloping floor surface and perimeter walls with varying top elevations. This wall irregularity and the sloping end roof sections (not visible in picture) created a nightmare of goometrical complications and it is very much to the credit of the detailers and the fabricators that the building went together almost without a hitch. The building had to be erected under most adverse weather conditions. Frequent wind and snow storms made erection hazardous and resulted in steady labour turnover. It was impossible to maintain a capable, experienced labour force, but due to competent supervision, the building was completed successfully and on schedule. Because of the size of the project and the short

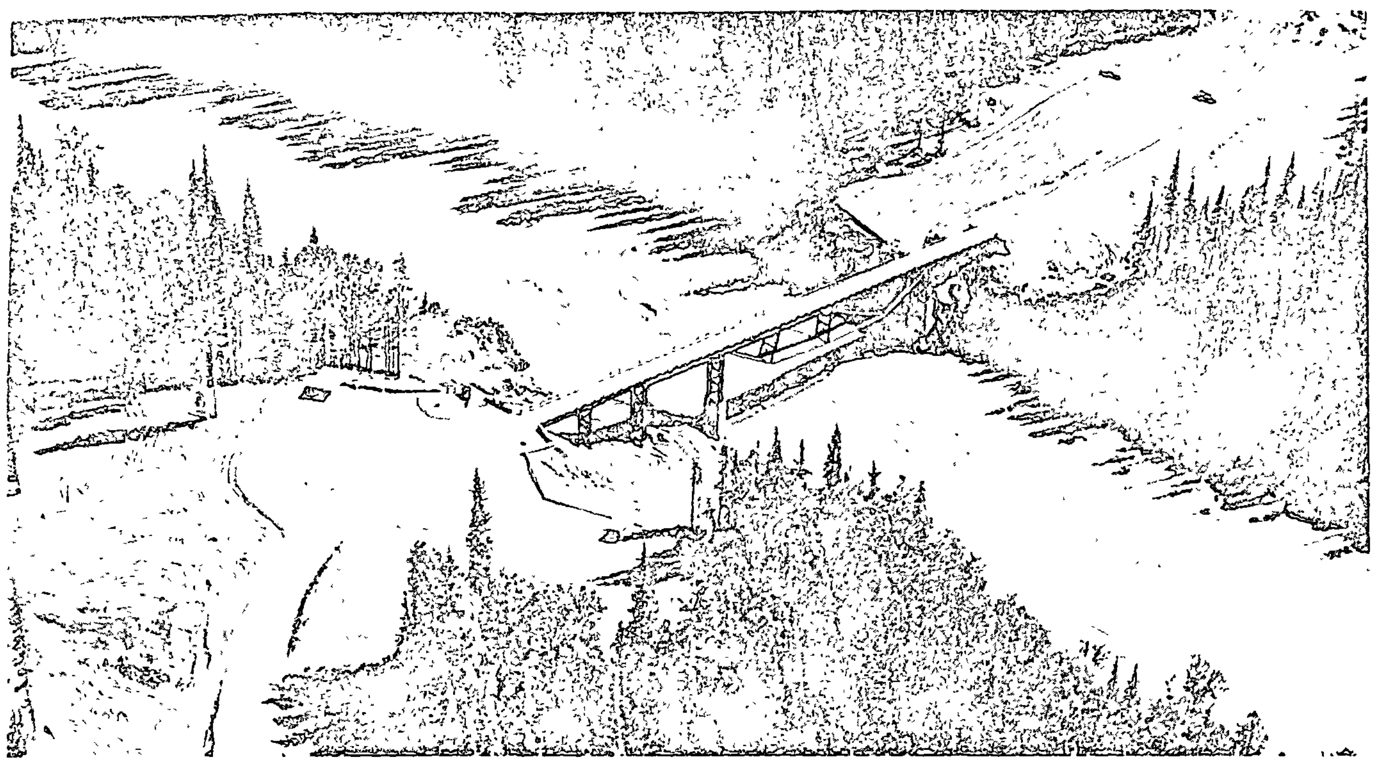


Fig 5 Nass River bridge near Stewart, B.C

shipping season, a great deal of planning was necessary in the scheduling and expediting of the various components. The laminated timber components were manufactured at three different laminating plants in Canada. The timber joists were obtained from several sources, including the eastern United States. Steel connections were manufactured partly in the west and partly in the east. Patterns and test fitting of connections were used to ensure accurate fabrication.

The selection of the materials for this building by the owners and the general contractor, was based strictly on competitive bidding. All materials, including the steel cladding, were chosen because of their availability and low in-place price. No doubt the remoteness of the site and the anticipated shipping and erection problems helped to increase the attractiveness of laminated timber.

Another building for which timber was selected for almost all of the components, including the wall cladding and the floor decking, because of the same problems of remoteness, was the asbestos mill building at Cassiar, British Columbia (Figure 2). Since this is one of the few high-rise timber buildings in existence, it has attracted considerable attention and has been written up in several engineering papers and reports.

The selection of timber for this building was due to its availability, competitive cost, its ease of erection, and its versatility in accommodating changes in the machinery layout. The structure consisted of continuous laminated timber columns extending to the roof, on a 12' x 18' grid. Laminated timber beams spanned the 18' direction between columns and sawn struts between columns in the 12' direction completed the grid structure. Nail laminated decking, covered with hardwood surface flooring, was used for all floors. Complete versatility was afforded by this arrangement as machinery could be placed anywhere it was desired, holes could be readily cut in the floor and beams could be removed or replaced.

Though the results were very satisfactory and timber has proven its suitability for this type of project, it should be recognized that wood has a shortcoming which created a problem with this type of building. Wood is comparatively light and elastic. For this reason, heavily vibrating equipment, such as was used in this asbestos mill, has a greater tendency to shake this wooden building than if it had been built of steel or concrete. It was necessary to find as many locations for bracing as possible, to dampen out the expected vibration.

An interesting combination of laminated timber and concrete is illustrated by the arena shown in Figure 3. This is one of about twenty ice hockey arenas which have been structural frames consisting of reinforced concrete traingular shaped lower legs attached to tapered upper legs made of laminated timber. These upper legs are re-sawn from parallel edged billets to produce a straight upper edge and a subtly curved lower edge to suit the changing bending moments. The response to this type of structure has been excellent. Besides being aesthetically pleasing and economical, the combination of timber and concrete has some very practical advantages. Architects like the idea of the durability of masonry block walls and concrete frames at the floor levels, while the use of timber over the ice areas avoids the problem of condensation and dripping.

Another example of an interesting combination of materials to suit the special requirements of a particular job is the large coal storage building owned by the Fording Coal Company, shown in Figure 4. This building consists of laminated timber arches spanning 192' at 24' spacing. Steel joists and steel cladding were selected because of the fire hazard from the coal dust. The large timber arches, having a better fire safety rating than unprotected seel, combine very well with steel joists and cladding to reduce the risk of fire loss.

Long span logging bridges constructed recently show how laminated timber and steel can be used together to solve transportation and erection problems in a very practical and economical manner. Several types of timber and steel bridges have been developed, one of the best being laminated timber girders with underslung steel trussing. The advantage of this type of system is that the girders can be made large enough to be erected as simple span girders carrying their own weight. The size of girder required for erection is usually close to the requirements for shear and being in the finished structure. Steel trussing is developed to suit clearance restrictions and to suit the section size selected. The structures are indeterminate and must be analyzed by a computer program. A great deal of care and experience is required to devise suitable connection details. Figure 5 shows a good example of this type of trussed girder bridge. It has a clear span of 186' and was built in 1971 across the Nass River to open up road access from the interior to Stewart, B.C. on the Pacific Ocean.

Fire Code and Insurance Problems

The efforts by designers to combine timber with other materials in quality buildings are often frustrated by unrealistic code and insurance rating requirements. In addition, people are inclined to think of buildings as consisting of only one structural material. Thus, fire underwriters class buildings as steel buildings, timber buildings and concrete buildings. Each of these materials has outstanding advantages and disadvantages, that designers should have more freedom to make use of. The criteria should be cost, availability, ease of erection, successful performance, durability and fire safety. Fire safety does not necessarily imply incombustible materials. Timber members of sufficient size to rate as "heavy timber" are usually much more fire resistant and give a much better performance in a fire than unprotected steel members.

The National Building Code of Canada has come a long way towards permitting a reasonable use of timber in large buildings, particularly when a building is sprinklered and accessible from several sides for fire fighting. Of course, the N.B.C. is not mandatory, unless it has been adopted by the body having building code jurisdiction in the location of the proposed building structure. In addition, many engineering structures such as wharves, conveyor support systems and logging bridges have no applicable codes to govern their design. It remains with the designer, in consultation with his client, to determine the best type of construction for his project. Thus, many large structures, especially in outlying areas, have been constructed of timber, when at first consideration it would appear that incombustible materials should have been used.

Avoiding Troubles with Timber

Wood is so familiar and has been used for so long that one might feel overly confident in its use. In actual fact, it is one of the most difficult materials to use well. It has many unique characteristics which one is more likely to appreciate if he has actually worked with wood. The need to have a "feeling for wood" or actually to "think in wood" probably requires complete familiarity with its properties, which can only be obtained by woodworking experience.

Some very serious mistakes have been made in wood design, and not only by inexperienced designers. The two most important characteristics of wood necessitating careful design consideration, are its weakness in tension perpendicular to the grain, and its shrinkage across the grain during drying. Most difficulties encountered with wood design and most actual unexpected failures are traceable to one or both of these characteristics.

Space limitations prevent any in-depth discussion of these design problems. Possibly, the following list of cautionary hints will help the designer to avoid the more obvious errors.

(a) Wherever possible, use details that avoid tension stresses perpendicular to the grain in a timber member. Typical examples are, notches in the bottom of beams, truss webs attached independently to timber chords instead of to each other, and secondary beam hangers attached close to the bottom of the carrying girders.

(b) Avoid restraining a large timber member in such a way that shrinkage from further drying will change the geometry of the structure or start a split that could lead to progressive failure of the member.

(c) Do not try to be too innovative in the design of timber structures, unless you are highly experienced. The simplest type of details is usally the best. If in doubt, seek expert advice.

Timber has proven through many years of successful performance that it has earned an important place in construction. However, it must be used with care and skill. Much remains to be learned about timber because of the complex nature of its structure and the variability of its growth characteristics. Fortunately, excellent research facilities exist in Canada to actively pursue the necessary investigations. Research information which they obtain is published and made available to the public. Designers should keep informed of the latest research developments. They should also check on current trade practices, material availability and cost and code restrictions, because all of these are subject to change. In spite of these problems, wood is a most interesting and challenging construction material, and to those who are familiar with its characteristics and can use it with confidence, it has a special appeal which is difficult to explain and certainly cannot be matched by any other construction material.

U.S. DEPARTMENT OF AGRICULTURE • FOREST SERVICE
FOREST PRODUCTS LABORATORY • MADISON, WIS.
In Cooperation with the University of Wisconsin

U.S.D.A. FOREST SERVICE
RESEARCH NOTE
FPL-0124
AUGUST 1968

WOOD FINISHING:

WATER-REPELLENT PRESERVATIVES

REM

By

Forest Products Laboratory, Forest Service
U.S. Department of Agriculture

Homeowners can avoid many exterior wood finishing problems by treatment with a water-repellent preservative solution (WRP). This treatment guards against damage to the wood and paint caused by water and by decay and stain fungi (mildew). WRP treatment of wood is recommended both before painting and also as a natural finish for exterior wood.

A WRP should be applied to all exterior wood that is normally painted. In a new house, use lumber treated by the manufacturer if possible, and treat cut ends on the job by brushing or dipping. If untreated lumber is used, treat all exterior surfaces.

In areas where decay is a serious problem, or when wood will be in contact with the ground (a fencepost, for example), some woods may need more protection than that afforded by a WRP.[1] In such cases, wood that has received a pressure treatment with preservative by commercial methods should be used.

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[1] A good classification of woods according to decay resistance appears in U.S. Forest Serv. Res. Note FPL-0153, Forest Products Laboratory, Madison, Wis.

What Does a WRP Do?

A WRP is a solution that gives wood the ability to repel water, such as rain and dew. It can do this because it contains waxlike material. By repelling the water, it fights decay and stain by denying fungi that cause these conditions the moisture they need to live. Wood surfaces that remain free of mildew have an attractive natural-finished appearance. A WRP also reduces water damage to the wood, such as the excessive swelling and shrinking that lead to cracking and warping. In addition, a WRP protects paint from the blistering, cracking, and peeling that often occur when excessive outside water penetrates the wood.

A WRP also contains a fungicide that kills any surface mildew living on the wood. This fungicide is usually penta (pentachlorophenol). Other components of the solution are a resin to improve paintability and to reduce blooming or crystallizing of fungicide on the surface.

There is a further benefit from a WRP treatment of exterior wood species such as redwood and western redcedar that contain colored water-soluble extractives. When water soaks into these woods through the paint and then dries out again, the colored substances are sometimes left on the paint surface. WRP treatment will effectively reduce this type of paint discoloration.

Safety First

In mixing and applying WRP, care should be exercised. The safest place to do the mixing is outdoors. The solution is a volatile flammable mixture. Don't breathe its vapors or expose them to flame or sparks. It is wise to wear protective clothing on the hands and arms and to take care that the solution is not splashed in the eyes or on the face.

Applying WRP to Wood Before Painting

Applying WRP solution to the surface of the wood with a brush, or by dipping, is an effective treatment for siding and exterior millwork (doors, window sash, door and window frames, sills, moldings, fascia), for wood fencing, and for lawn furniture.

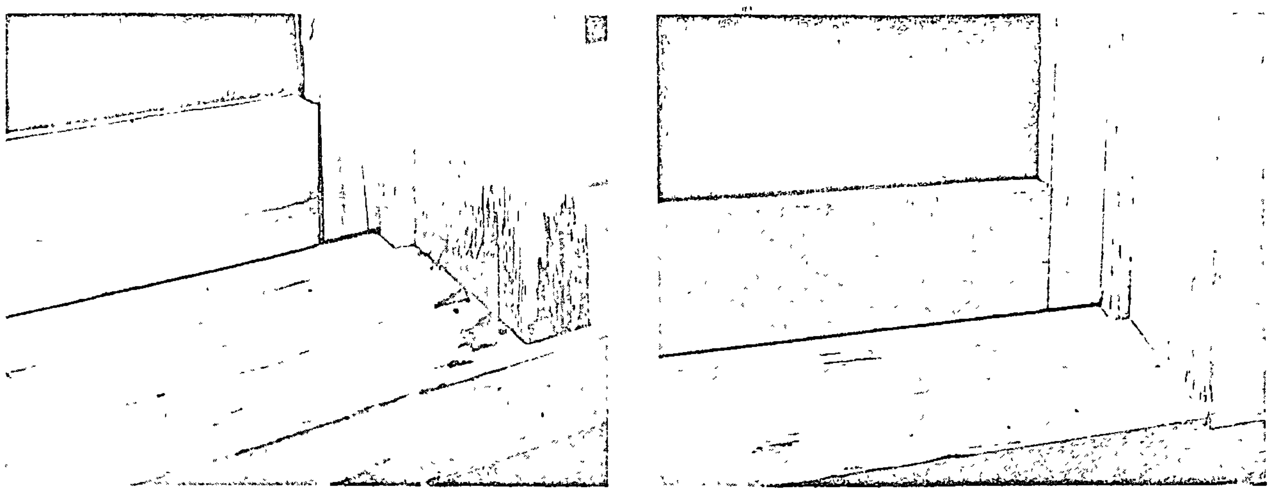

Figure 1.--Experimental window sash and frames after severe rain and weathering. Left - The sash and frame were not treated before painting. Rain that entered the joints caused extensive paint peeling and excessive swelling of the wood. It has also caused cracking of the putty and serious decay in the sash, which must now be replaced. Right - Before painting, the sash was dipped for 3 minutes in WRP. The solution was brushed on the frame. The wood is in good condition, and the paint has weathered normally to the point where repainting is needed.

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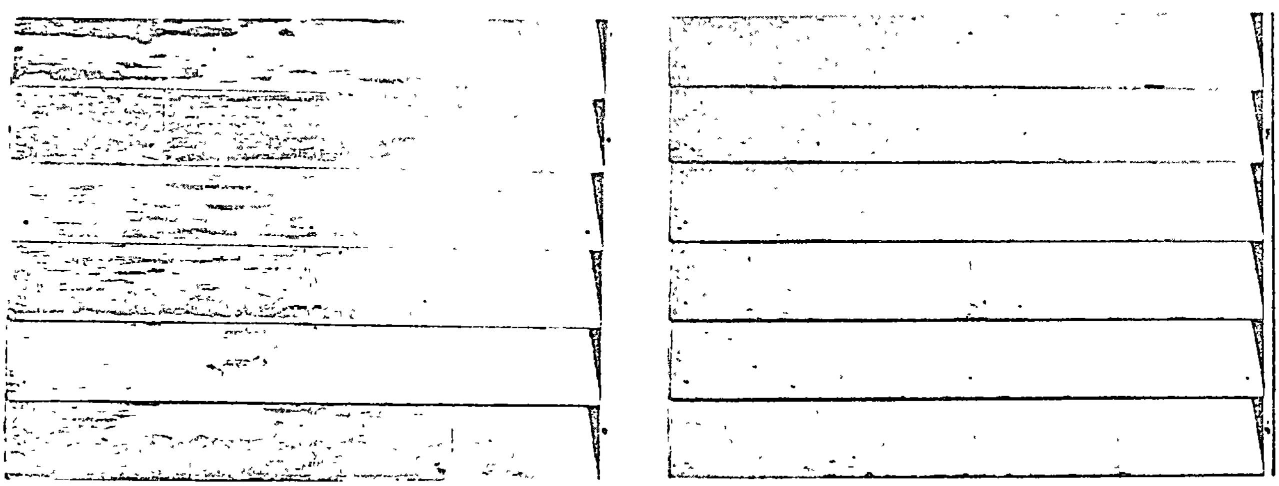

Figure 2.--Experimental panels of bevel siding after severe rain and weathering. Left - The siding was not treated before painting. Rain that entered the butt and lap joints has caused serious paint peeling. Right - Before painting, the siding was dipped for 10 seconds in WRP. The paint has weathered normally to the point where repainting is needed.

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The following steps are suggested for application to new wood:

(1) If treated siding or millwork is purchased, brush or dip-treat only the freshly cut surfaces.

(2) Wood which has not been factory treated can be treated by either dipping, brushing, or spraying. Care should be taken to treat the ends of boards and joints between boards. Open joints should be calked after treating and priming.

(3) Allow freshly treated wood to dry. If applied with a brush or spray, allow 2 days of favorable drying weather before painting. If dipped for 10 seconds, allow 1 week of favorable drying weather before painting. If enough time is not allowed for most of the solvent to dry from the wood, the paint applied over it may be slow to dry, or it may discolor or dry with a rough surface that looks like alligator leather.

When applying WRP to previously painted wood, remove the loose paint, brush the WRP into the joints only, and wipe off excess solution from the paint surfaces with a rag. Allow 2 days of favorable drying weather before repainting.

Whether treatment is to new wood or previously painted wood, particular care should be taken to apply the solution well at the ends of boards and joints between boards. Some homeowners do not realize that water will climb up the back of bevel siding from the lap joints. It does this by capillary flow. WRP applied to lap joints of the siding does a good job of preventing capillary flow. Accordingly, places that should be treated well include the butt and lap joints of horizontal siding, edges and top and bottom ends of vertical siding, and the edges and corner joints in window sash, sills, window frames, doors, and door frames. Often bottoms of doors and window sash are overlooked. These are areas where water can penetrate deeply and cause extensive damage if not treated. Treatment with WRP will eliminate many problems later.

The effectiveness of a light treatment with WRP has been confirmed in studies conducted at the Forest Products Laboratory. Figures 1 and 2 show the difference between WRP-treated and untreated bevel siding, window sash, and frames after exposure to severe weather conditions. The treated siding was dipped for 10 seconds, and the treated window sash for 3 minutes. This is the degree of treatment usually given by manufacturers. The treated window frame and sill in figure 1 were brushed with the solution. This is an easy treatment for any homeowner or painter. Treating by dipping, however, can be expected to be more thorough and effective than brush application.

As the illustrations show, the WRP treatment effectively reduced cracking of the putty and separation of the wood. The paint on the treated wood was protected from early peeling failure. The WRP treatment also prevented stain and decay damage. These results leave no doubt that WRP effectively improves the performance of both wood and paint.

Identifying Outside Water Damage to Paint

Paint damage that is the result of outside water, such as rain, heavy dew, or sprinkler water, is identified by observing when and where it occurs. It usually appears either as blistered, cracked, or peeled paint (paint failure) around the joints and ends of boards, or as discoloration by water-soluble substances from the wood at these same places.

Paint failure caused by outside water (rain and dew) can be distinguished from failures produced by other causes in four ways:

(1) Failure caused by the penetration of rainwater occurs only during seasons when it rains.
(2) The failure is usually most severe on the sides of the building that face the prevailing winds and rains.
(3) Failure caused by rainwater occurs only on wood that can be wetted by rain.
(4) Paint failure caused by rainwater may occur on unheated as well as on heated buildings.

Where rain can wet the wood, it may not be possible to distinguish between damage caused by rain and damage that is the result of heavy dew. Dew damage does occur, however, in areas that are sheltered from rain. Fortunately, a WRP can protect wood from both rain and dew.

If there is any doubt whether damage is from rain or dew, apply the treatment before repainting. It can do no harm, and it may do a great deal of good. Decayed wood should be replaced before repainting.

Using WRP as an Exterior Natural Finish

The color and appearance of weathered wood can be affected, to a marked degree, by mildew. In most parts of the country, mildew grows on the wood surface and gives it a dark gray, blotchy, and unsightly appearance. In contrast, weathered wood in very dry climates or in coastal regions where salt atmospheres may inhibit the growth of mildew, has a clean, silvery appearance.

The color of weathered wood is influenced to a lesser degree by highly colored wood extractives in such woods as western redcedar and redwood. These extractives gradually diffuse to the surface and produce a dark-brown color. This color may persist in protected areas not exposed to the sun and where extractives are not removed from the wood surface by the washing of rain.

A clean golden-tan color can be achieved in the weathering of wood by treating the surface to retard the accumulation of wood extractives and mildew on the surface. The treatment, originally recommended by the California Redwood Association, consists of applying a WRP to the wood surface. This method of finishing also is recommended for the popular siding species and for the natural finishing of exterior plywood, brushed plywood, and low grades of lumber that do not hold paint well. The treatment also reduces warping and cracking and prevents water staining at edges and ends of wood siding.

The first application of the WRP is usually short-lived. When the wood surface starts to show blotchy discoloration caused by extractives and mildew, clean it by mild scrubbing with a detergent or trisodium-phosphate solution. Then re-treat with another liberal brush application of water-repellent preservative solution.

Frequently it is necessary to clean and re-treat smoothly planed wood surfaces after the first year of exposure. After the cleaning and re-treating, the treatment should last much longer and need be refinished only when the surface starts to show an uneven discoloration pattern or small black spots indicating the start of mildew. The treatment will be more durable on weathered or rough-sawn surfaces because they adsorb a greater quantity of solution than a smooth surface.

Pigments in the form of colors in oil and tinting colors can also be added to the WRP solution to give a desired color effect and improve durability. A quantity of 4 to 6 fluid ounces of colors per gallon of solution is usually adequate. Pigmented WRP should be applied to the full length of a course of siding without stopping to avoid the formation of lap marks. Lap marks would also be minimized by applying two coats.

Penetrating pigmented stains such as described in U.S. Forest Service Res. Note, FPL-046 "Forest Products Laboratory Natural Finish," are considered more durable than the WRP type finish and can always be applied to wood previously finished with the WRP.

Nails for Natural Finishes

When wood weathers naturally, it is important to use nails that are highly resistant to rusting. Iron nails rust rapidly and produce a severe brown or black discoloration around the nail. Aluminum nails and galvanized nails to a lesser extent, are corrosion-resistant and prevent such difficulties.

A Typical WRP Solution

WRP solutions are widely made and distributed and are available in most paint and lumber stores. The following is a simple formula for a water-repellent preservative that will serve effectively both as a pretreatment of wood for painting and as a natural-type exterior finish for wood.

	Approximate quantity	
Ingredients	For 1 gallon	For 5 gallons
Penta concentrate, 10:1	1-3/4 cups	2 quarts
Boiled linseed oil	1-1/2 cups	1-3/4 quarts
Paraffin wax	1 oz. (1/16 lb.)	5 oz. (1/3 lb.)
Solvent (turpentine, mineral spirits, or paint thinner)	3 quarts	4 gallons

(Additional solvent may be added to make a full gallon or 5-gallon measure, if desired.)

Mixing the Ingredients

Melt the paraffin wax in the top unit of a double boiler or some other container heated by hot water. Don't use a direct flame--the paraffin wax will ignite. The solvent should be at room temperature (60° to 80° F.) before mixing. While vigorously stirring the solvent, slowly pour in the melted paraffin. After the paraffin wax and solvent are mixed, add--in order--linseed oil and penta concentrate. Stir until the mixture is uniform.

The ingredients will separate if the solution is at low or freezing temperatures. If this happens, reheat the solution to room temperature and stir to redissolve the ingredients.

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USOS ESTRUCTURAS DE LA MADERA

LA INDUSTRIA DE LA MADERA EN MEXICO

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LA INDUSTRIA DE LA MADERA EN MEXICO *

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I. INTRODUCCION

Pese a contar con bosques que admiten una producción mayor a la actual, en los últimos años, la producción forestal ha tenido un incremento anual tan solo de 1.1%. Las consecuencias han sido muy graves: destrucción del recurso, estancamiento de la producción, cuantiosas y crecientes importaciones, limitado desarrollo rural y la existencia de una industria poco eficiente, con costos altos y precios de la madera y de los -- productos.

En el futuro se demandarán mayores niveles de eficiencia y productividad, en la medida en que se fomenten los aprovechamientos y se incremente la producción, se busque sustituir importaciones o exportar, conquistar mercados internos (perdidos por precios altos o no explorados, vgr., vivienda). Así también se reclamarán formas distintas de presentación de los productos.

La magnitud y características de nuestros recursos forestales y la existencia de estos mercados, todavía nos permite ser optimistas al respecto y pensar que es factible, la creación de una seria industria forestal y un desarrollo armónico e integral de las zonas forestales.

El papel del Estado es definitivo para estimular la formación de este tipo de industria y desarrollo ó para restringirlos.

1.

1.1 RELACION ENTRE EL VOLUMEN CONCESIONADO Y EL VOLUMEN PRODUCIDO.

CUADRO No 1 RELACION ENTRE EL VOLUMEN CONCESIONADO Y EL PRODUCIDO EN 1972

ESTADO	VOLUMEN CONCESIONADO [illegible]			VOLUMEN PRODUCIDO [illegible]			[illegible]		
	CONIFERAS	PRECIOSAS	CORRIENTES	CONIFERAS	PRECIOSAS	CORRIENTES	CONIFERAS	PRECIOSAS	CORRIENTES
1. CHIHUAHUA	[illegible]		[illegible]	1,516,500		800	91		1
2. DURANGO	[illegible]		54,000	690,500		4,900	87		9
3. MICHOACAN	694,000	100	21,000	590,300	100	20,500	85	100	93
4. OAXACA	538,000	100	92,000	353,500	100	20,750	65	100	22
5. JALISCO	431,000	5,700	130,000	424,300	100	119,700	[illegible]	2	92
6. CHIAPAS	425,000	48,000	135,000	258,900	29,100	10,500	61	61	8
7. GUERRERO	435,000	900	85,000	179,100	100	21,300	41	10	22
8. MEXICO	235,000		41,000	203,000		40,400	88		98
9. QUINTANA ROO		31,000	33,000		30,400	32,500		99	93
10. CAMPECHE		48,000	30,000		47,500	16,900		99	56
11. VERACRUZ	27,000	2,100	54,000	12,800	2,000	53,500	47	95	99
12. SAN LUIS POTOSI	14,000	900	213,000	2,300	200	212,700	16	22	100
13. COAHUILA	2,500		1,500	800		1,500	29		100
14. PUEBLA	130,000	170	15,000	129,600	100	14,800	100	59	99
15. TABASCO		12,000	45,000		10,300	44,300		84	100
16. TAMAULIPAS	57,000	1,900	79,000	12,500	600	63,600	22	32	80
17. NUEVO LEON	25,000		10,000	21,600		6,500	86		65
18. ZACATECAS	63,000		11,000	25,300		5,000	40		45
19. SONORA	74,000		20,000	25,000		16,300	33		81
20. NAYARIT	47,000	300	27,000	7,500		16,100	16	0	60
21. GUANAJUATO	3,700		35,000	3,700		34,500	100		99
22. D. F.	32,000		100	31,500		100	99		100
23. MORELOS	80,000		4,500	32,300		400	41		9
24. YUCATAN		700	11,500		700	11,500		100	100
25. SINALOA	4,900	200	4,900	3,100	200	2,100	63	100	43
26. QUERETARO			4,900			400			8
27. B.CALIFORNIA NTE.	6,400		600	5,400		600	84		100
28. COLIMA	2,900	200	1,400	2,000	100	1,400	69	50	100
29. TLAXCALA	7,500		200	7,300			97		0
30. HIDALGO	3,500		100	400			11		0
31. B.CALIFORNIA SUR			300			300			100
32. AGUASCALIENTES	100		100	100			100		0
SUMA	6,289,800	152,700	1,217,700	4,777,100	121,500	774,200	76	80	64
Equivalente a al rollo total.	9,000,000	191,000	2,705,000						

FUENTE: SAG. 1972. ANUARIO FORESTAL.
INFORMES DE LA DIRECCION GENERAL DE APROVECHAMIENTOS Y DE LAS DELEGACIONES FORESTALES.

Del cuadro se deduce que en 1972, de las coníferas se aprovechó el 76% del volumen concesionado, el 80% de los volúmenes de preciosas y el 64% del volúmen de corrientes.

1.2

CUADRO No 2 VOLUMENES POTENCIALES DE CORTA ANUAL Y COMPARACION CON LOS VOLUMENES ACTUALES DE PRODUCCION

ESTADO	CONIFERAS: Volumen potencial de corta anual persistente (Próximos 5 años) m3 rollo aprovechable *	CONIFERAS: Volumen de producción 1972 m3 r. aprov.	CONIFERAS: Diferencia m3 rollo aprov.	PRECIOSAS: Volumen potencial de corta anual persistente (Próximos 5 años) m3 rollo aprovechable	PRECIOSAS: Volumen de producción 1972 m3 r. aprov.	PRECIOSAS: Diferencia m3 rollo aprov.	CORRIENTES: Volumen potencial de corta anual persistente (Próximos 5 años) m3 rollo aprovechable	CORRIENTES: Volumen de producción 1972 m3 r. aprov.	CORRIENTES: Diferencia m3 rollo aprov.
1.- [illegible]	[illegible]	100	200				1,000		1,000
2.- [illegible]	[illegible]	3,100	23,600				1,000	600	[illegible]
3.- [illegible]	[illegible]		200				1,000	[illegible]	[illegible]
4.- [illegible]				16,000	47,600	- 31,600	100,000	[illegible]	[illegible]
5.- COAHUILA	31,000	800	30,200				1,000	[illegible]	300
6.- [illegible]	200	2,000	- 1,800	800	100	700	4,000	1,400	[illegible]
7.- [illegible]	[illegible]	[illegible]	266,100	40,000	29,100	10,900	100,000	[illegible]	[illegible]
8.- [illegible]	[illegible]	[illegible]	[illegible]				90,000	[illegible]	[illegible]
9.- [illegible]	21,000	31,000	- 10,000				500	150	[illegible]
10.- [illegible]	2,100,000	[illegible]	1,210,500				110,000	[illegible]	[illegible]
11.- [illegible]	3,000	3,700	- 700				27,000	[illegible]	- 17,500
12.- GUERRERO	1,000,000	172,100	820,000	1,600	100	1,500	225,000	23,100	[illegible]
13.- [illegible]	100,000	400	99,600	800		800	31,000		31,000
14.- JALISCO	750,000	424,300	325,700	1,600	100	1,500	200,000	119,300	80,700
15.- MEXICO	400,000	[illegible]	192,000				90,000	40,400	[illegible]
16.- MICHOACAN	[illegible]	290,300	252,700	1,600	100	1,500	100,000	70,500	[illegible]
17.- MORELOS	42,000	32,700	9,300				22,000	400	21,600
18.- NAYARIT	130,000	7,600	122,400	1,600		1,600	67,000	[illegible]	[illegible]
19.- [illegible]	42,000	21,100	23,600				1,000	[illegible]	- [illegible]
20.- [illegible]	[illegible]	[illegible]	[illegible]	2,400	100	2,300	102,000	[illegible]	[illegible]
21.- [illegible]	[illegible]	129,600	[illegible]	500	100	400	67,000	[illegible]	[illegible]
22.- [illegible]	17,000		17,000				7,000	400	6,600
23.- [illegible]				36,000	30,400	5,600	[illegible]	[illegible]	[illegible]
24.- [illegible]	14,000	2,300	11,700	200	200		[illegible]	[illegible]	- [illegible]
25.- [illegible]	[illegible]	[illegible]	[illegible]	200	200		2,000	2,100	- 100
26.- [illegible]	[illegible]	20,000	56,000				12,000	[illegible]	- [illegible]
27.- [illegible]				3,200	10,100	- 6,900	45,000	[illegible]	[illegible]
28.- [illegible]	70,000	12,500	57,500	600	600		4,000	[illegible]	[illegible]
29.- TLAXCALA	10,000	7,300	2,700				2,000		[illegible]
30.- [illegible]	100,000	12,800	87,200	2,400	2,000	400	53,000	[illegible]	[illegible]
31.- [illegible]				4,000	700	3,300	20,000	11,500	[illegible]
32.- [illegible]	[illegible]	[illegible]	23,700				2,000	[illegible]	[illegible]
S U M A	9,910,700	4,777,100	+5,153,600	133,500	121,500	+12,000	2,101,200	774,200	+1.427,000
Equivalente en m3 rollo total	14,200,000	6,800,000	+7,400,000	267,000	243,000	+24,000	4,800,000	1,700,000	+3,100,000

FUENTES: INVENTARIO NACIONAL FORESTAL
INFORMES DE DELEGACIONES FORESTALES

* Mediante Atención al Bosque Estos Valores Deben Aumentar,

1.3 DEMANDA DE PRODUCTOS FORESTALES Y CELULOSICOS

CUADRO No. 3 DEMANDA ESTIMADA DE ALGUNOS PRODUCTOS FORESTALES Y CELULOSICOS EN MEXICO, 1975, 1980 y 1985.

PRODUCTO	CONCEPTO	1970	1975	1980	1985
ASERRADOS (miles de m3)	PRODUCCION IMPORTACION EXPORTACION CONSUMO-DEMANDA *	1641 67 1 1707	2040-2260	2500-2500	3100-3200
TRIPLAY (miles de m3)	PRODUCCION IMPORTACION EXPORTACION CONSUMO-DEMANDA	99 6 2 103	150-170	235 - 255	350-380
TABLEROS DE FIBRA Y TABLEROS DE AGLOMERADOS. (m3)	PRODUCCION IMPORTACION EXPORTACION CONSUMO-DEMANDA	81,500 310 1340 80,470	150,000	285,000	530 000
CELULOSA (Química de madera pasta mecánica y química de Bagazo de Caña, etc.)	PRODUCCION IMPORTACION CONSUMO-DEMANDA	473 125 598	700	850	1,600,000
PAPEL PARA CAJAS (LINER) (miles de ton.)	PRODUCCION CONSUMO-DEMANDA	198 198	305	470	720
PAPEL PARA CAJAS (CORRUGADO MEDIO) (miles de ton.)	PRODUCCION CONSUMO-DEMANDA	85 95	130	200	310
PAPEL PARA SACOS, BOLSAS Y ENVOLTURA (miles de ton.)	PRODUCCION CONSUMO-DEMANDA	198 198	270	370	490
OTROS PAPELES (Impresión, Escribir, Cartoncillos, Cartulinas, Higiénico, Facial y otros. (miles de ton.)	PRODUCCION IMPORTACION CONSUMO-DEMANDA	375 21 396	540	720	970
PAPEL PARA PERIODICO (miles de ton.)	PRODUCCION IMPORTACION CONSUMO-DEMANDA	40 241 281	285	380	510
TOTAL PAPELES	PRODUCCION IMPORTACION CONSUMO-DEMANDA	896 262 1158	1530	2140	3000
DEMANDA TOTAL PRODUCTOS FORESTALES Y CELULOSICOS. (en millones de m3 rollo aprovechable de madera). **	--	7.0	8.5-9.0	11.0-12.0	14.0-15.0

* CONSUMO APARENTE 1970

DEMANDA ESTIMADA 1975,1980,1985

** (bajo las siguientes consideraciones; manteniendo las proporciones actuales de madera, bagazo, pajas y desperdicio; en el caso del papel periódico, la producción futura proveniente de madera y bagazo en iguales proporciones y la producción de papel consumiendo celulosa nacional, desperdicio de papel nacional, celulosa de importación y desperdicio de papel de importación, en las mismas proporciones que en 1972.

FUENTES. - MEMORIAS ESTADISTICAS, [illegible], ASOCIACION NACIONAL DE FABRICANTES DE [illegible] DE MADERA 1973.

[illegible] DE LA [illegible] PARA EL DESARROLLO FORESTAL.

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1.4 SATISFACCION DE LA DEMANDA FUTURA

Existen varios caminos para buscar satisfacer las demandas futuras, a saber:

A). Captando los volúmenes desperdiciados en el monte y en los aserraderos de los aprovechamientos actuales, como materia prima celulósica. - Para 1973 se estima que se desperdició un volumen de coníferas con posibilidades de ser utilizado de 500,000-1,000,000 m3 aprovechable.

B). Aprovechando el total de los potenciales de corta en las diversas regiones del país.

En el cuadro No. 2 se muestran los volúmenes potenciales de corta anual para las coníferas de las áreas comerciales, y para preciosas y corrientes y se comparan con los volúmenes de producción de 1972, en los mismos grupos botánicos y en la misma unidad (m3 rollo aprovechable). Se deduce que hay un volumen adicional de 5.1 millones de m3 rollo aprovechables de coníferas, 12,000 m3 rollo aprovechable de preciosas y de existir mercados, 1.4 millones de m3 rollo aprovechable de corrientes.

Para las coníferas existen recursos para aumentar en un 100% la producción de 1972, las entidades que pueden adicionar más volumen son: Durango, Chihuahua, Guerrero, Oaxaca, Jalisco, Chiapas, Michoacán, México y Puebla. Para las "preciosas" se dispone de las selvas para mantener la producción actual, siendo Chiapas, Quintana Roo y Campeche las entidades más importantes. Para las corrientes y de existir mercados podría aumentar en un 200% la producción actual, siendo Guerrero, Durango, Chiapas, Michoacán, Oaxaca y Quintana Roo las entidades que pueden agregar mayor volumen.

C). Aumentando el crecimiento de los bosques naturales a través de labores de protección y mejoramiento. A partir de 1975 se podrá esperar incrementos en un plazo de 10-15 años, de tal suerte que sería hasta 1990 cuando podría obtenerse un aumento por este concepto. Conservadoramente y dependiendo del establecimiento de servicios técnico-forestales en las concesiones, se podría disponer de un volumen adicional mínimo de coníferas de 1.5 millones de m3 rollo aprovechable después de 1990.

D). Aumentando el volumen por medio de plantaciones forestales. Partiendo de una plantación de 2000 plantas por hectárea y un turno de 10-15 años -

en zonas tropicales y de 15-20 años en zonas templadas, se podría obtener un crecimiento anual del orden de 10-15 m3 rollo por hectárea equivalente a una producción total de 150-225 m3 rollo por hectárea. Con una plantación de 10,000 hectáreas anuales en 15 años se tendrían 150,000 hectáreas y una producción anual a partir del quinceavo año de 1.2-1.8 millones de m3 rollo aprovechable. Existen varias áreas ya detectadas con posibilidades:

- En zonas tropicales con buenas condiciones para especies de rápido crecimiento vgr. Cuenca del Papaloapan, Veracruz, Oaxaca, Tamaulipas, Chiapas, Nayarit.
- En zonas templadas, en terrenos originalmente forestales que actualmente soportan una agricultura o ganadería marginal vgr., Estado de México, Jalisco, Michoacán, Guerrero, Puebla, Chiapas, Nayarit, etc.
- En zonas templadas, en terrenos boscosos improductivos en donde se corte a matarrasa ó se desmonte y se hagan plantaciones con especies nativas o introducidas vgr. México, Michoacán, Jalisco, Oaxaca, Chiapas, Guerrero, etc.

E). Con el empleo más amplio del bagazo de la caña de azúcar, en aquéllos productos que más con-- venga y mezclado con fibra larga de coníferas en las proporciones más económicas. Veracruz y Sinaloa son las áreas más productoras de bagazo.

F). Con la utilización del henequén para producir vgr., papel moneda, y también a través de plantaciones de bambú para la producción de material celulósico, y el uso de palmas, etc.

G). Por medio de importaciones en aquéllos renglones que más convenga.

Resumiendo, para 1975 se tendrá una demanda equivalente de alrededor de 9 millones de m3 rollo aprovechable de madera, estimándose que habrá disponible en total 10.5 millones m3 rollo aprovechable de coníferas más 0.13 millones de m3 rollo aprovechable - de preciosas y 2.2. millones m3 rollo aprovechable de corrientes. En 1985 habrá una demanda de aproximadamente 14-15 millones de m3 rollo aprovechable, estimándose que podrá haber 14 millones m3 rollo aprovechable de coníferas, más 0.07 millones m3 rollo aprove-- chable de preciosas y 2.5 millones m3 rollo aprovechable de corrientes. Después de 1985 se podría disponer de un volumen adicional a través de las plantaciones comerciales. El cultivo de los bosques, las plantaciones comerciales forestales, la utilización de desperdicios, la integración industrial y el empleo de fibras cortas de bagazo de caña y otras serán requisitos básicos para el futuro

Los cuadros de demanda se ven ajustados con datos actuales y en base al ritmo de producción sin considerar nuevos proyectos, de la forma siguiente: Balance entre la demanda y la producción futura.

.MADERAS ASERRADAS:

AÑO	Demanda Mills.M3	Oferta Mills.M3	Balance Mills. M3
1975	2.418	1.867	0.551
1980	3.045	2.168	0.877
1985	3.836	2.469	1.368

.TRIPLAY	Miles M3	Miles M3	Miles M3
1975	159.7	124.3	85.4
1980	239.7	153.7	86.0
1985	313.6	183.1	130.5

.CELULOSA	T. M.	T. M.	T. M.
1975	606,704	578,656	72,100
1980	876,443	706,815	172,100
1985	1,277,986	834,973	354,400

2. SITUACION ACTUAL DE LA ACTIVIDAD FORESTAL

2.1. Aprovechamientos e Industrialización.

La estructura actual de las concesiones en base a los volúmenes concesionados de pino es la siguiente: - 30% son Unidades Industriales (vigencia de 25 a 50 años) 27% Unidades de Ordenación (vigencia de 10 años) y 43% - permisos precarios (vigencia de 3 a 10 años). Hasta la - fecha las empresas particulares, predominan en el aprovechamiento e industrialización de los bosques.

Las empresas ejidales junto con las empresas -- descentralizadas o del Estado aprechan alrededor del 25% del volúmen concesionado de pino. El Estado produce aproximadamente un 20% de la producción de triplay y un 30%- de la celulosa a partir de madera.

Juzgadas las formas actuales de aprovechamiento, en términos de eficiencia industrial, desarrollo rural y atención al bosque, la mayoría adolece de defectos, no - existiendo por lo general una atención equilibrada a esos tres elementos.

En base a las posibilidades socioeconómicas de- los bosques y a los resultados obtenidos en el pasado, el Estado ha tomado la decisión de intervenir directamente- en el proceso de desarrollo de las zonas forestales y -- así recientemente se han impulsado tres organismos des-- centralizados, dos federales "Forestal Vicente Guerrero" en Guerrero y Profortara en Chihuahua, y uno Estatal - - Aprofon en Nayarit, que junto con Proformex en Durango,-

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Protinbos en el Estado de México, Atenquique en Jalisco, Acuitzio y Villa Madero en Michoacán, pueden llegar a producir a -corto plazo casi 6 millones de m3 r anuales o sea casi el 70% de la producción actual.

Hasta la fecha las empresas particulares, predominan en el aprovechamiento e industrialización de los bosques. Las empresas ejidales junto con las empresas descentralizadas-o del Estado aprovechan alrededor del 25% del volúmen concesionado de pino. El Estado produce aproximadamente un 20% de la -producción de triplay un 30% de la celulosa a partir de madera.

Actualmente existen alrededor de 600 aserraderos, 30% de los cuales con una capacidad diaria/turno de 1,000- - -5,000 pies tabla, 50% entre 5,000-10,000 pies tabla, 15% entre 10,000-20,000 p.t. y un 5% de 20,000-30,000 pies tabla o más.

A fines de mayo había 18 fábricas de triplay; 2 -en Chihuahua, 4 en Durango, 2 en el Estado de México, 1 en Hidalgo, 1 en Veracruz, 1 en Chiapas, 2 en Yucatán, 1 en Campe--che, 1 en Quintana Roo y 1 en Oaxaca; 2 de ellas consumen de -40,000-60,000 m3 rollo por año; 4, de 15,000-40,000 m3 rollo -aprovechable por año y el resto menos de 15,000 m3 rollo aprovechable por año. De chapa hay 2 fábricas, una en el Estado de México (Cuautitlán) y otra en Campeche (Zon-Laguna), ambas consumen maderas tropicales en un volúmen inferior a 7,000 m3 rollo aprovechable por año.

Hay básicamente cuatro fábricas de tableros aglomerados; una en San Luis Potosí, una en Guerrero, una en Oaxaca y otra en Yucatán. Hay dos fábricas de tableros duros, una-en Durango y otra en San Luis Potosí con un consumo anual de -alrededor de 130,000 m3 rollo aprovechable. Hay aproximadamen-

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te 12 fábricas de caja clavada o alambrada; 11 instalaciones que fabrican molduras, 6 impregnadores, un sin número de operaciones que se dedican a hacer muebles y 33 plantas de destilación de resina de pino; 23 en Michoacán, 6 en Jalisco, una en Oaxaca, una en Guerrero y dos en México.

En la industria de la celulosa y papel hay 62 fábrica de las cuales 7 consumen madera; de estas una fábrica es la de Tuxtepec que elabora papel periódico y papel para libros de texto y revistas con producción anual de 45,000 toneladas por año; celulosa de Chihuahua produce más o menos 100,000 toneladas de celulosa al año; Atenquique 60,000 toneladas de productos Kraft por año, Loreto y Peña Pobre y San Rafael elaboran varios artículos en una cantidad de más o menos 85,000 toneladas por año; Cartón y Papel de México que fábrica varios productos y finalmente Celulosa del Pacífico que está empezando a producir papel corrugado.

El papel periódico procede en un 100% de madera, de la materia prima consumida para la producción de celulosa aproximadamente el 68% procede de madera, 24% de bagazo de caña y el 8% de residuos vegetales. De papel para empaque y otros papeles y cartones casi el 75% de la materia prima utilizada procede de madera o de desperdicios que originalmente fueron madera.

Veracruz y Sinaloa son las entidades que producen más caña de azúcar en el país. En la actualidad se utiliza una parte de la producción total del bagazo de la caña, en la fabricación de pulpa blanqueada y sin blanquear que se emplea en el papel escritura, papel celofán, papel bond y pañuelos, toallas y papeles sanitarios, etc. Estos productos se obtienen mezclando bagazo de caña con celulosa de fibra larga-y pasta mecánica de coníferas. 15

La calidad de los bosques se ha reducido considerablemente por la falta de un plan de regularización o se ha desperdiciado por un mal ajuste entre la calidad del arbolado y el tipo de industria. El 60% de los volúmenes autorizados en empresas permanentes se destina a productos celulósicos, aprovechándose partes del árbol de especificaciones apropiadas para productos de mayor redituabilidad como chapa, triplay y aserrados. Por otra parte, se desperdician considerables volúmenes de puntas de árboles y materiales de aserradero que podrían usarse vgr. para celulosa y papel. Las empresas papeleras aprovechan el 70% del volúmen total del árbol, las empresas de aserrío de pino el 60% y las de aserrío de especies tropicales el 45%. La no integración del aprovechamiento forestal representa un costo social elevado. (ver cuadro siguiente).

2.2. ABUSO DEL BOSQUE

Tan solo en el 20% del área de los bosques productivos de pino (en total 13 millones de Ha.) se dispone de servicios técnicos en donde la protección, mejoramiento y control son más o menos permanentes; el 90% del área de los bosques sobre-explotados de pino (en total 2.0 millones de Ha.) está abandonada y casi el 100% de los terrenos forestales de la zona templada sin bosques posibles de recuperar (1.5 millones de Ha.) está ignorado.

La limitada acción del Estado en la planeación, promoción y desarrollo de las zonas forestales, la falta de actividades, la alta densidad de población la muy reducida participación de los titulares de derecho de los bosques en su ---

aprovechamiento o industrialización, la demanda de madera, el carácter temporal de los permisos y el estado de veda, han sido las causas más importantes del abuso y la destrucción del recurso forestal, expresados en desmontes, pastoreo, incendios, cortas clandestinas, etc., y cuyos efectos se han traducido en el subdesarrollo de las zonas forestales y sus pobladores, en la desaparición de especies valiosas y sustitución por otras menos útiles, en la erosión de las tierras, en los azolves de las presas y en la reducción de los recursos boscosos.

Todavía en 14 entidades del país hay áreas forestales vedadas: Baja California Norte, Chihuahua, Sonora, Sinaloa, Hidalgo,Hidalgo, Puebla, Veracruz, Aguascalientes, Querétaro, Guanajuato, Colima, Jalisco, Morelos y el Distrito Federal.

Juzgadas las formas actuales de aprovechamiento en términos de eficiencia industrial, desarrollo rural y atención al bosque, la mayoría adolece de defectos: no existe por lo general una atención equilibrada a esos tes elementos.

2.3. PARTICIPACION DEL SECTOR CAMPESINO

En el pasado, los poseedores de los bosques han sido generalmente rentistas en su aprovechamiento y muchas veces ni en la renta misma han participado. Además no se ha creado una organización que haya permitido canalizar en forma eficiente sus derechos de monte hacia actividades productivas, ni los servicios del Estado se han orientado adecuadamente para fomentar el desarrollo rural en las zonas forestales. Su participación se ha prestado a que otros grupos se beneficien o bien que se favorezca tan sólo a los líderes y representantes.

CUADRO No. ESTRUCTURA DE LAS CONCESIONES Y SU CLASIFICACION DE ACUERDO CON EL DESTINO DE LA MADERA (1970)

TIPO DE CONCESIONES	VOLUMEN CONCESIONADO m3 r.t./año.	EMPRESAS CLASIFICADAS POR DESTINO MATERIA PRIMA +				
		A.T.P.	T.A.	A. o AP.	P.	TOTAL EMPRESAS
		No.	No.	No.	No.	No.
UNIDADES INDUSTRIALES						
. CONIFERAS	> 400,000	1				1
	200,000 - 300,000			1	1	2
	100,000 - 150,000		1		2	3
	50,000 - 80,000		2	1	2	5
. PRECIOSAS	9,000 - 15,000		2			2
	SUMA	1	5	2	5	13
UNIDADES DE ORDENACION	> 400,000					
. CONIFERAS	200,000 - 400,000	1	2			3
	100,000 - 150,000		2			2
	50,000 - 80,000		2	5		7
. CONIFERAS Y PRECIOSAS	10,000 - 40,000	4	4	16		24
	SUMA	5	10	21		36
PERMISOS PRECARIOS						
. CONIFERAS	> 50,000	2	4	35		41
. CONIFERAS Y PRECIOSAS	5,000 - 40,000	2	15	383		400
	SUMA	4	19	418		441
	TOTAL	10	34	441	5	490

+ A= Aserrío
T= Triplay o tableros.
P= Papel.

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3. PROGRAMA NACIONAL DEL DESARROLLO FORESTAL

El Estado ha tomado plena conciencia de su responsabilidad y ha elaborado a través de la Subsecretaría Forestal y de la Fauna y con la colaboración de Nacional Financiera; un instrumento de estudio, diagnóstico, planeación, promoción e impulso regional como lo es el Programa Nacional del Desarrollo Forestal; medio y no solución pero punto de partida para encauzar el crecimiento del sector, para correlacionar los recursos disponibles con los mercados nacionales y mundiales y las necesidades de crecimiento económico y desarrollo social del país y para coordinar esfuerzos. Este programa se basa en estudios de gran visión, regionales de prefactibilidad y regionales de preinversión y en otras acciones fundamentales y se aprovecha el Fondo de Estudios de Preinversión de Nacional Financiera para la ejecución de los estudios. El Programa no termina a la conclusión de los estudios, en realidad es un sistema continuo de planeación, para tomar decisiones, un medio para coadyuvar en la implementación de los proyectos y en el control de su avance. Se busca no tan solo mejorar el dignóstico de la situación actual sino darle solución a los múltiples problemas y corregir progresivamente los errores, además se persigue definir las políticas en diversas áreas, atraer nuevas inversiones y dar elementos de juicio para la toma de decisiones más racionales en campos tan vitales como organización, participación y funciones de los sectores, procedencia de las inversiones y distribución de los beneficios.

Por acuerdo presidencial publicado en el Diario Oficial del 14 de marzo de 1973, se creó la Dirección General para el Desarrollo Forestal dependiente de la Subsecretaría Forestal y de la Fauna, la cual se encargará de la conducción de estos trabajos.

En general, la Subsecretaría Forestal y de la Fauna llevará a cabo los estudios dentro del marco de los principios de la política forestal y en base a los guiones generales y términos de referencia (ver contenido en el Apéndice) aprobados por la propia Subsecretaría, el Comité Técnico y de Distribución de Fondos del Fondo de Estudios de Preinversión de Nacional Financiera y el Comité Asesor Forestal designado por el C. Presidente de la República con representantes del DAAC, SAG, CNC, CNIDS, CNIP, CONCAMIN y Directores de Organismos Forestales Federales. También se basará en los guiones específicos que se vayan elaborando para cada proyecto.

El Programa Nacional del Desarrollo Forestal ha sido aprobado y apoyado por todos los sectores concurrentes y los que forman el Comité Asesor Forestal.

ORGANIZACION TECNICA, ECONOMICA Y SOCIAL 20
DE LAS REGIONES FORESTALES

Las acciones en esta área forman un todo, están relacionadas entre sí y tienden a propiciar que en cada región forestal del país se vaya estableciendo la organización técnica, económica y social que garantice el cultivo del bosque, la superación del sector campesino, el desarrollo rural en las zonas forestales, el mejoramiento de la industria establecida y la incorporación de nuevas unidades de producción con bases sólidas de rentabilidad y eficiencia. En la actualidad existen diversas alternativas de asociación entre poseedores de los bosques e industriales, que son:

- Ejido, comunidad o pequeños propietarios trabajando por sí solos.
- Ejido y/o comunidad y/o pequeños propietarios con organismos descentralizados o industriales particulares o con empresas del Estado.
- Ejido y/o comunidad y/o pequeños propietarios con Estado e industriales particulares.

No se pretende una sola forma de organización ni de asociación, ya que las condiciones particulares de cada zona determinarán la fórmula más conveniente, pero en todos los casos se buscará que tengan elementos comunes: racionalidad, bases económicas y sociológicas firmes y moralidad. Interesa aumentar la producción, siempre y cuando cada m3 que se agregue, sea producido con niveles de eficiencia, calidad y justicia social diferentes a los que hasta ahora hemos tenido y también que paralelamente a ese aumento de producción se promueva el uso de otros recursos de las zonas forestales a través de un desarrollo regional integrado.

3.2. EL IMPULSO A NUEVOS PROYECTOS 21

Comprende estudios regionales de preinversión y de prefactibilidad para estudiar posibilidades de desarrollo silvícola industrial.

Los estudios de preinversión incluye, entre otros a los siguientes proyectos: Proformex, en Durango, Vicente Guerrero, en Guerrero, en proceso y a través del Organismo, Sur de Chihuahua, en Chihuahua. Los estudios de prefactibilidad comprende desarrollos para nuevas industrias que permitan una integración industrial regional o el aprovechamiento de recursos forestales no utilizados tanto en entidades no vedadas como vedadas:

Entidades no vedadas: Durango-El Salto, Zona Lacandona, en Chiapas, Oaxaca-Tuxtepec, y México. Entidades vedadas: Chihuahua, Michoacán, Jalisco-Atenquique, Veracruz, Puebla, Hidalgo, Sinaloa, Colima, Morelos, Sonora y Baja California Norte.

Comprende también la consideración y apoyo a proyectos ya iniciados o terminados, que afectan la relación de oferta-demanda, tales como los de: Aprovechamientos Forestales de Nayarit, Celulosa de Michoacán, Celulosa del Pacífico en Guerrero, Fábricas de triplay en Comitán, Chis., Forestal Vicente Guerrero, Maderas Moldeadas (Werzalith), en Dgo., Bonampak en Chiapas, Integración Industrial de Atenquique en Jalisco, Papel periódico y otros papeles a partir de bagazo de la caña en Veracruz, Sinaloa, etc.

Los estudios regionales comprenden revisiones de estudios previos, actualización de la información, evaluación de alternativas bajo criterios de costo/beneficio, etc. Las áreas principales de los estudios son: Recursos Forestales, Abastecimiento de Trocería y Leñas, Mercados, Transformación, Aspectos Económicos y Desarrollo

Aspecto fundamental en los proyectos será el dar respuesta entre otras a las siguientes interrogantes:

¿ Qué funciones y responsabilidades va a tener cada grupo; el organismo, los propietarios de los montes, los industriales particulares y los contratistas en el abastecimiento de trocería y leña, en los caminos, en la industrialización, etc.?

¿ Quién será responsable de la implementación y operación del proyecto y de la coordinación integral del mismo?

¿ Quién va a captar los créditos?

¿ Como se asegurará que a los puestos de responsabilidad lleguen gentes idóneas que aseguren el éxito del proyecto?

¿ Qué mecanismo será necesario establecer para la fijación de precios de venta, etc.?

3.3. EL MEJORAMIENTO Y DEPURACION DE LA INDUSTRIA ACTUAL.

A través de la revisión de la industria existente, se persiguen acciones inmediatas tendientes a: Mejorar las técnicas de producción, organización y comercialización para aumentar la productividad, reducir los costos y aprovechar la capacidad instalada. Capacitar al personal. Plantear medios de integración industrial, intercambio o venta de materias primas a nivel regional, y posibilidad de agrupación de áreas, concesiones y servicios. Aprovechar mejor las diversas calidades y partes del arbolado, buscando reducir el desperdicio con posibilidades de uso industrial, utilizar más ampliamente el oyamel, el encino y otras especies hojosas y buscar la mejor combinación de la madera con el bagazo de la caña, el bambú, henequén, palmas, etc., Establecer secadoras. Reducirla contaminación ambiental. Crear conciencia.

Reducir las causas por las cuales no se ejercen los volúmenes concesionados y en general promover medidas concretas para la consolidación y racionalización de la industria y para las posibles ampliaciones, buscando depurar la lista de concesionarios y aprovechar las capacidades de los industriales serios.

La industria que se revisará está localizada básicamente en las entidades más significativas y que contribuyen más en la producción forestal: Chihuahua, Durango, Michoacán, Oaxaca, Jalisco, Chiapas, Guerrero, México, Campeche y Quintana Roo.

3.4. EL DESARROLLO RURAL Y LA SUPERACION DE LOS NIVELES DE VIDA DE LOS PROPIETARIOS Y POSEEDORES DE LOS BOSQUES.

En el desarrollo agrícola del país y en su estructura agraria se han proyectado dos sectores: el sector agrícola moderno y comercializado formado en su mayoría por particulares y el sector tradicional campesino. Al primero, se han canalizado la mayoría de los beneficios directos o indirectos de la inversión del sector público: obras hidráulicas para riego, caminos, créditos, investigación, etc., mientras que la carencia de estos elementos es evidente en el otro sector(campesino)y dentro de éste, los habitantes de las zonas forestales, que constituyen una parte muy importante del sector rural, han sido los mas olvidados.

Se estudiarán posibilidades para generar empleos e ingresos adicionales a través de desarrollos en actividades agro-industriales, cinegéticas, turístico-recreativo, uso múltiple de los terrenos forestales, mineras y plantaciones.

Se busca también establecer en las organizaciones y concesiones forestales, un servicio de desarrollo rural que permita orientar y aplicar las utilidades y los derechos de monte que se capten y los servicios del Estado, y también considere las experiencias - sociales y técnicas del sector campesino.

Las experiencias de Aprofon, Forestal Vicente Guerrero, Profortara, Profortarah, Proformex, Atenquique y FONAFE, en cuanto a desarrollo rural, serán valiosas evidencias para orientar esta importantísima acción. La Subsecretaría Forestal y de la Fauna promoverá acciones de desarrollo rural en todas las demás zonas forestales del país.

3.5. EL CULTIVO DEL BOSQUE

Como acción inmediata de la Subsecretaría Forestal y de - la Fauna (SFF), asumirá la responsabilidad total de los Servicios - Técnicos Forestales tal como lo señala la Ley Forestal. En 1972 se incorporaron únicamente los Servicos Técnicos de la Unidad de Atenquique, debido a la tramitación para que el personal incorporado -- pertenezca al ISSSTE y disfrute de sus beneficios y prestaciones. - En 1973 se han incorporado Tutuaca, en Chih. Empresas Ejidales de El Salto, en Dgo. y Acuitzio-Villa Madero, en Mich. y se continuarán - estas acciones con el resto de las unidades industriales, los organismos descentralizados y las empresas ejidales.

Para cubrir el déficit actual de madera y el que se preve a mediano plazo, la SFF fomentará el cultivo del bosque y como tarea complementaria promoverá plantaciones forestales con fines comerciales. Se ha estimado que en los próximos 20-25 años deben plantarse alrededor de 10-30 mil Has. anuales, independientemente de los programas de protección de cuencas y saneamiento ambiental. Los Estados de México, Michoacán, Hidalgo, Puebla y Veracruz presentan grandes atractivos para llevar a cabo proyectos autopagables y rentables sobre plantaciones forestales.

La Subsecretaría Forestal y de la Fauna impulsará métodos -- sílvicolas vgr. matarrasa con plantaciones que permitan aumentar la productividad del bosque y obtener mayores beneficios económico-sociales.

La Subsecretaría Forestal estudiará los reglamentos y circulares vigentes en la materia para decidir cuales son operantes, - si deben modificarse y en que medida, o si deben suprimirse.

3.6. EL LEVANTAMIENTO DE VEDAS FORESTALES EN 15 ENTIDADES.

La Subsecretaría Forestal y de la Fauna en coordinación - con los Gobiernos de los Estados, levantará las vedas forestales buscando establecer la organización técnica, económica y social que garantice el desarrollo rural, la industrialización y el cultivo del bosque. Las entidades se pueden dividir en tres grupos en base a los recursos comprendidos en zona vedada: Primer - grupo: Michoacán, Jalisco y Chihuahua. Segundo grupo: Puebla, -- Veracruz, Hidalgo y Baja California Norte. Tercer grupo: Sinaloa, Sonora, Colima, Morelos, Distrito Federal, Querétaro, Guanajuato y Aguascalientes.

3.7. LA COORDINACION INTERGUBERNAMENTAL E INTERSECTORIAL.

El estudio de la parte institucional del Programa Nacional del Desarrollo Forestal, permitirá definir el area de trabajo de cada una de las dependencias y sectores, sus relaciones y correlaciones, las funciones que corresponden a cada grupo y establecer los mecanismos de coordinación entre las diferente dependencias del Ejecutivo, para que sus acciones en materia forestal queden ligadas a directrices generales y se mejore la eficiencia de la acción del sector público.

Existen varios organismos forestales descentralizados federales y estatales a saber: Aprovechamientos Forestales de Campeche, Comisión para los Aprovechamientos Forestales del Estado de Oaxaca, Aprovechamientos Forestales de Nayarit, Forestal Vicente Guerrero, en Guerrero, Productos Forestales Mexicanos, en Durango, Productos Forestales de la Tarahumara, en Chihuahua y Protectora e Industrializadora de Bosques, en el Estado de México. Como se han creado en distintos años y bajo diversas promociones y enfoques, resulta fundamental que la Subsecretaría Forestal y de la Fauna, respetando las características propias de sus zonas de influencia, promueva uniformar sus políticas y objetivos generales, para que como verdaderas unidades de desarrollo social sean congruentes con los objetivos de la política forestal y del Programa Nacional del Desarrollo Forestal. Además, se deberán establecer acciones conjuntas para resolver problemas comunes; vgr. capacitación de personal, organización, etc. La coordinación de los organismos por la Subsecretaría Forestal será una de las tareas importantes en el futuro.

Para evitar los problemas que plantean la fijación de los precios de las materias primas, (trocería, leña, tabla, triplay, etc.); la determinación en la participación de las utilidades, los derechos de monte, etc., se considera básico la integración inmediata de una Comisión que determine las bases conforme a las cuales se fijen los valores, de acuerdo con criterios económicos y sociales uniformes.

Esta Comisión, cuya política será congruente con los objetivos del Programa Nacional de Desarrollo Forestal, se integrará con representantes de la SAG, DAAC, CNC y del Sector industrial privado. Podrá invitarse a formar parte de ella a otros grupos de interés y, de acuerdo con lo establecido en la Ley Forestal, intervendrá el Banco de México, S. A.

La Comisión asesorará a los interesados en casos concretos, y de pedirlo uno de ellos, actuará como árbitro; sus decisiones obligarán a las partes en conflicto.

La coordinacion con dependencias federales, Gobiernos de los estados, organismos de desarrollo, etc., relacionados con el sector es tarea básica. Se contemplan varios medios de comunicación entre los sectores a través de: Comité Asesor Forestal, Comisión de Precios de Materias Primas, Participación de Utilidades y Derechos de Monte, Consejos de Administración de los Organismos Forestales Descentralizados y Comité Directivo del Programa Nacional del Desarrollo Forestal.

La clarificación de la tenencia de la tierra es un problema que demanda especial atención. La definición de quiénes son propietarios o legítimos poseedores con derecho a disposición de los bosques, es por fuerza lenta, por lo que es imperativo establecer los mecanismos legales adecuados, para que, sin perjuicio de los legítimos intereses de propietarios o poseedores, pueda aprovecharse en beneficio general la riqueza forestal de aquéllos predios cuyo dominio no esté plenamente clarificado o sea francamente indefinido.

En aquéllos casos no previstos por las actuales leyes en que por cualquier causa no se logre obtener el consentimiento de los propietarios o poseedores legítimos, pueda llevarse a cabo el aprovechamiento de los bosques depositando los derechos de monte o fondos comunes en el Banco de México, S.A. o en el Fondo Nacional de Fomento Ejidal, según se trate de conflictos en los que intervengan pequeños propietarios o unicamente ejidos o comunidades.

La coordinación con el Programa Nacional de Desmontes es importante para el uso apropiado de los terrenos forestales y para la utilización de las maderas tropicales y templadas.

3.8. LA FORMACION DE PERSONAL A TODOS NIVELES. (obreros, auxiliares, técnicos, profesionales y administradores.)

Para preparar el personal necesario que demande el desarrollo forestal y elevar la productividad y eficiencia del sector en todas sus áreas (cultivo del bosque, abastecimiento de -- trocería y leñas, industrialización y actividades conexas) y para hacer efectiva la participación del sector campesino en el -- aprovechamiento e industrialización se tienen ya algunas acciones que se empezarán a llevar a cabo durante 197

A) El Departamento de Asuntos Agrarios y Colonización en coordinación con la Subsecretaría Forestal y de la Fauna y -- con los Organismos Descentralizados empezará a desarrollar pro-- gramas y cursos de capacitación para incrementar la capacidad, - mejorar la organización y obtener una mayor preparación administrativa, técnica y comercial de los poseedores de los bosques.

B) La Subsecretaría Forestal y de la Fauna implantará el Programa Nacional de Desarrollo de Abastecimiento de Trocería y Leñas, dentro del cual se establecerán centros de entrenamiento para obreros en operaciones manuales y mecanizadas de corte y arrime de trocería y elaboración de leñas.

C) La Subsecretaría Forestal y de la Fauna promoverá - en coordinación con la Secretaría de Educación Pública el funcionamiento de un mínimo de once Secundarias Técnicas Forestales en Chihuahua, Durango, Jalisco, Michoacán, México, Guerrero, Vera--

cruz, Oaxaca, Chiapas, Baja California y Sonora; elevará a nivel de técnicos la Escuela de Guardas Forestales de Uruapan, a igual nivel, promoverá este tipo de cursos en Chihuahua, Durango, Guerrero y Veracruz; a nivel profesional apoyará la transformación de la Unidad de Bosques de la Escuela Nacional de Agricultura en Facultad o Escuela de Ingeniería Forestal para que ofrezca preparación en todas las áreas de la rama forestal y a nivel posgrado cree la maestría forestal.

Independientemente de las acciones anteriores se estudiarán formas efectivas para llevar a cabo la tarea de formación de personal a todos niveles.

3.9. LA ATRACCION DE INVERSIONES

Atraer inversiones hacia la industria forestal por medio de proyectos rentables y viables socialmente.

Parte vital de una nueva política forestal debe ser el establecimiento de instituciones o procedimientos de captación de recursos, a fin de establecer sistemas institucionales de crédito para la actividad forestal.

Parte importante es la programación de la inversión del sector público forestal, los diversos estudios y acciones darán a la Subsecretaría Forestal y de la Fauna mas elementos para justificar las futuras inversiones.

3.10. PRIORIDAD DE LAS REGIONES EN RELACION A LA EJECUCION DE LOS ESTUDIOS Y ACCIONES DEL PROGRAMA NACIONAL DEL DESARROLLO FORESTAL.

ACCION	PRIORIDAD I	PRIORIDAD II	PRIORIDAD III
ORGANIZACION TECNICA, ECONOMICA Y SOCIAL DE LAS REGIONES FORESTALES.			
. IMPULSO A NUEVOS PROYECTOS.	PROFORMEX, DGO. VICENTE GRO, GRO. SUR DE CHIH. MICHOACAN JALISCO CON ATENQUIQUE	APROFOM, NAY. CHIAPAS CON ZONA LACANDONA OAXACA CON TUXTEPEC. PROTINBOS, MEX. B. CFA. NTE.	PUEBLA VERACRUZ HIDALGO SINALOA SONORA COLIMA MORELOS OTRAS ENTIDADES
. MEJORAMIENTO Y DEPURACION DE LA INDUSTRIA ACTUAL	CHIHUAHUA DURANGO MICHOACAN	GUERRERO JALISCO OAXACA CHIAPAS MEXICO SURESTE	OTRAS ENTIDADES
. DESARROLLO RURAL Y SUPERACION DE LOS NIVELES DE VIDA DE LOS PROPIETARIOS DE LOS BOSQUES.	CHIHUAHUA DURANGO MICHOACAN MEXICO D. F.	GUERRERO NAYARIT JALISCO OAXACA CHIAPAS SURESTE	" OTRAS ENTIDADES
. CULTIVO DEL BOSQUE	CHIHUAHUA DURANGO MICHOACAN MEXICO D. F.	GUERRERO NAYARIT OAXACA JALISCO CHIAPAS SURESTE	" OTRAS ENTIDADES
LEVANTAMIENTO DE VEDAS FORESTALES.	MICHOACAN JALISCO CHIHUAHUA	B. CFA. NTE. SONORA SINALOA	PUEBLA VERACRUZ HIDALGO COLIMA MORELOS D. F. QUERETARO GUANAJUATO AGUASCALIENTES

4. EVALUACION PRELIMINAR DEL P. N. D. F.

Se ha logrado en varios de los proyectos en marcha la participación directa de la Subsecretaría Forestal y de la Fauna, a través de la Dirección General para el Desarrollo Forestal.

4.1. Disponibilidad de nuevos volúmenes de productos maderables:

Proyecto	Aserrío M3/año	Triplay M3/año	Celulosa Ton/año	Total M3/año rollo
1. PROFORMEX, DGO.	250,000	60,000	170,000	1,500,000
2. ATENQUIQUE, JAL.	75,000	15,000	15,000	240,000
3. PROTINBOS, MEX.	100,000	15,000	70,000	510,000
4. MICHOACAN	100,000	15,000	50,000	430,000
5. NAYARIT	50,000	5,000	-	110,000
6. SUR DE CHIH.	-	-	120,000	500,000
7. PUEBLA				
MORELOS				
HIDALGO				
VERACRUZ	-	-	-	200,000
8. CHIAPAS	50,000	50,000 Aglom.	40,000	310,000
9. F.VCTE.GRO, GRO.	-			500,000
	625,000	110,000 (menos Agl).	465,000	4,300,000

4.2 Otras Promociones Importantes:

. Educación.

- Programa nacional de desarrollo de abastecimiento de trocería y leñas.
- Creación del Departamento de Educación y Capacitación del INIF.
- Creación de la Escuela Superior de Tecnología de la Madera en Morelia, Mich.

. Plantaciones Comerciales:

- Programa de 3,000 Ha/año, Tuxtepec, Ver.
- " " " " " , Protinbos, Méx.
- " " " " " , Profomich. Mich.
- Creación del Fideicomiso Nacional para plantaciones comerciales.

. Fomento de la madera en la construcción

- Promover su uso básicamente en la vivienda
- Coordinar esfuerzos de los diferentes grupos interesados.
- Organización del grupo interno de trabajo

. Recreación

- Desarrollo de Parques Nacionales.
- Formación grupo de trabajo.
- 1974 inicio de inversiones Parque Miguel Hidalgo

. Levantamiento de vedas forestales.

La SFF en coordinación con los Gobiernos de los Estados, levantará las vedas forestales previo estudio social, económico y técnico.

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Estos estudios están contemplados en las acciones de mejoramiento en la industria actual e impulso a nuevos proyectos.

Grupo 1:	Grupo 2:	Grupo 3:
Chihuahua	Veracruz	Morelos
Michoacán	Hidalgo	D. F.
Jalisco	B.Calif.Nte.	Querétaro
Puebla	Colima	Guanajuato
Sinaloa		Aguascalientes
Sonora		

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