New Structural Systems in Small-Diameter Round Timber

by

Aurimas Bukauskas

Submitted to the Department of Architecture
in Partial Fulfillment of the Requirements for the Degree of

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Abstract:

Trees, when used as structural elements in their natural, round form, are up to five times stronger than the largest piece of dimensioned lumber they could yield. Additionally, these whole-timbers have a lower effective embodied carbon than any other structural material. When combined into efficient structural configurations and joined using specially-engineered connections, whole-timber has the potential to replace entire steel and concrete structural systems in large-scale buildings, bridges, and infrastructure. Whole-timber may be the most appropriate structural solution for a low-carbon and fully renewable future in both developed temperate regions and the developing Global South. To reduce barriers to adoption, including project complexity and cost, a standardized "kit of parts" in whole-timber is proposed. This thesis proposes new designs for the first and most important element of this kit: a structurally independent column in whole-timber. A 20' compound column in whole-timber is prototyped at full-scale. New, simple calculation methods are developed for estimating the buckling capacity of tapered timbers. Based on conservative assumptions, the embodied carbon of whole-timber column systems is shown to be between 30% and 70% lower than conventional steel systems.

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1. Introduction

1.1. Why do efficient structures matter?

We are approaching a climate change tipping point. It is critical that we, as a civilization, reduce the amount of energy we use and switch to entirely renewable or recyclable materials.

Buildings consume 40% of energy in the United States (US EIA 2015). As building operation becomes more efficient, an increasing fraction of this energy is spent before buildings are even occupied. This is called embodied energy, or the energy required to harvest, process, and assemble the components of a building. 71% of the energy it takes to construct a building goes towards its structure (De Wolf 2014). It is critical, therefore, to reduce the embodied energy of our structural systems. We can do this by using materials with lower embodied energy, and using them in efficient structual configurations which achieve maximal structural capacity while using minimal material.

1.2. Timber as a Low-Energy, Renewable Structural Material:

Timber has been shown to have a lower effective embodied energy than steel and reinforced concrete, the most commonly used structural systems. (Hammond and Jones 2011) Additionally, using timber in structures temporarily sequesters atmospheric carbon in those structures (wood is approximately 50% carbon by weight.) When sustainably managed, timber is renewable, and at the end of life of a building, timber elements can be recycled, left to biodegrade, or be burned as biofuel. Thus, timber may be the most appropriate building material for a low-carbon, renewable future.

1.3. Whole-Timbers as Structure:

Research by Ron Wolfe at the US Forest Products Laboratory has demonstrated the exciting potential for trees, in their natural round form, to be used as structural elements. Wolfe showed that forest stands throughout the United States are overstocked with small-diameter trees, posing a major forest fire and disease risk (2000). This is because small-diameter trees (between 4 and 10 inches in diameter) have little market value. Currently, such small-diameter trees are processed into low-value products such as firewood or woodchips. Wolfe proposes the use of small-diameter trees in their whole, round form (henceforth referred to as whole-timber) as structural elements, showing that whole-timbers can be assigned load capacities up to 5 times greater than the largest piece of dimensioned lumber they could yield. Additionally, whole-timbers require much less energy to be processed into structural elements, as they do not need to be sawn or kiln-dried. Thus, whole-timber has an even lower embodied carbon than conventional timber in the form of dimensioned lumber, and can achieve equivalent structural performance with even less material.

This thesis proposes the use of whole-timbers as primary structural elements in large-scale structural systems with dramatically lower embodied energy and environmental impact than conventional steel, concrete, and dimensioned lumber structures. A standardized “kit of parts” in whole-timber is proposed as a solution to barriers including project complexity and cost. This thesis focuses on the design of the first and most important element of this kit: a structurally independent column in whole-timber.

2. Motivations

2.1. Why Whole-Timber?

Whole-timber has all of the positive attributes of timber in general, with some additional advantages. Compared to conventional dimensioned lumber products, whole-timber has:

1. Lower Embodied Energy
2. Greater Strength per Unit Weight
3. Reduced Environmental Impact / Low-Toxicity

Embodied Energy of Whole-Timber:

Whole-timber has a lower embodied energy than dimensioned lumber because it requires less processing. Whole-timber is not sawn, and does not need to be kiln-dried. The 2011 Inventory of Carbon and Energy database provides two embodied carbon coefficients for timber products: EC_{fos} and EC_{bio}. EC_{fos} corresponds to the fossil fuel energy consumed harvesting, transport, and production, while EC_{bio} corresponds to the carbon emmissions from burning timber offcuts as fuel for drying kilns.

There are no studies yet which quantify the embodied carbon of whole-timber specifically. However, we can conservatively approximate the embodied carbon of whole-timber by subtracting the EC_{bio} component from the embodied carbon coefficient for sawn softwood timber. As whole-timber produces no offcuts and does not need to kiln-dried, the EC_{bio} component can be subtracted. There is as of yet no rigorous way of accounting for the energy savings of not having to saw whole-timbers.

Table 2.1.1: Embodied Carbon Coefficients of Various Primary Structural Materials. Units: kg CO_2e / kg

	EC_{fos}	EC_{bio}	EC_{total}
Steel (General)	n/a	n/a	1.46
Glue-Laminated Timber	0.42	0.45	0.87
Sawn Softwood	0.2	0.39	0.59
Whole-Timber	0.2	n/a	0.2

Note: For all embodied carbon coefficients, CO_2e, meaning CO_2 equivalent, is used.

Greater Strength per Unit Weight:

Whole-timber is stronger than dimensioned lumber for three reasons:

1. Fiber Continuity
2. Sectional Properties
3. Taper

Fiber Continuity:

Sawing whole-timbers into prismatic members cuts through the grain of the tree, which naturally curves around knots and other imperfections (Fig. 2.1.1). This exposes the ends of the fibers that give a tree its strength, introducing local weaknesses in the resulting prismatic members which can reduce their strength by up to 2-3 times.

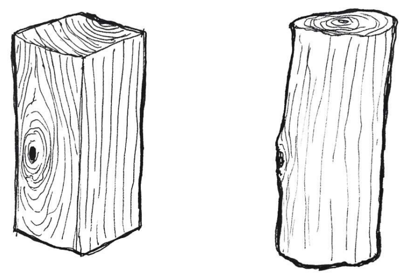


Fig. 2.1.1: Effect of knots and other imperfections in dimensioned lumber vs. whole-timber as shown by grain patterns.

Sectional Properties:

The cross-section of a round timber has an area 1.57 times greater, and a moment of inertia 2.35 times greater than the largest prismatic member that can be derived from it (Fig. 2.1.2). These directly correspond to an increased crushing capacity by a factor of 1.57 and an increased buckling capacity by a factor of at least 2.35.

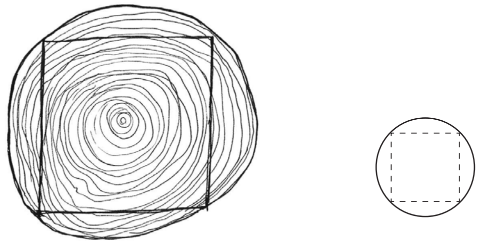


Fig. 2.1.2: Cross-section of a whole-timber with its largest inscribed square.

Taper:

All trees taper over their length. The size of any prismatic or uniform-section round member that can be derived from a tapered timber is limited by the smaller end of that timber (Fig. 2.1.3). The buckling capacity of a whole, tapered timber is significantly greater than the largest prismatic or uniform-section round timber that can be derived from it. A calculation method for computing the buckling capacity of tapered timbers is described in section 4.3. The buckling capacity of a naturally tapered timber with 3% taper, a base diameter of 6", and a length of 10' is 43% greater than that of the largest uniform-section round timber that it can yield.

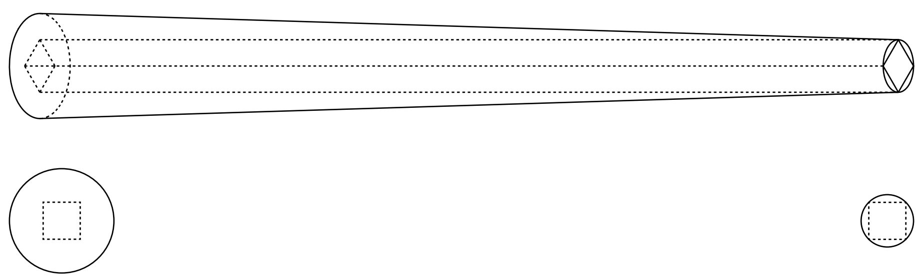


Fig. 2.1.3: Naturally tapered whole-timber with the largest prismatic member it could yield.

Reduced Environmental Impact / Low Toxicity:

Unlike glue-laminated timber, no adhesives or other additives are required to use whole-timber structurally. Additionally, as Wolfe describes, the logging industry currently has a strong preference for large-diameter trees, resulting in ecological stress for overstocked forests. Using small-diameter trees is beneficial to dangerously overstocked forests, and can help ensure the sustainability of forest resources in general into the future.

To process a whole-timber into a structural member, the timber must be felled, debarked, and any necessary connection cuts made. It should be noted that nearly all of the steps to produce whole-timber other than any long-distance transport can be accomplished by hand - that is, without the use of heavy machinery. This is an opportunity for potential energy savings and reduced reliance on fossil fuels. It also makes whole-timber an especially appropriate low-cost material for developing regions, where capital resources for industrial equipment are often prohibitive.

2.2. Why aren’t we building in Whole-Timber already?

The two main obstacles preventing wider adoption of whole-timber as a structural material are:

1. Complex and Costly Connections
2. Material-constrained Design Process

Connection Design in Whole-Timber:

It is difficult to design structural connections which can transfer load between multiple, round and irregular timbers meeting at a node. These connections often require custom design and fabrication for each structure.

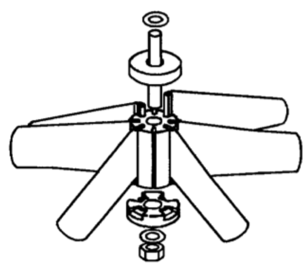


Fig. 2.1.4: Space truss connection in whole-timber (Wolfe 2000).

Material-Constrained Design Process:

Conventional structural design proceeds by establishing a geometry for a structure, determining the loading, and sizing the structural elements to meet the required loads. With whole-timber, sections and lengths are strictly limited, thus this design process must proceed in reverse. The available timber sections and lengths dictate the loads that can be resisted, which in turn dictate the geometry of the structure.

2.3 Why large-scale structures?

The ultimate goal of this research is to replace the maximum volume of steel, concrete, and glue-laminated structural material in the least time. Timber is already commonly used in residential and mid-scale construction in the form of dimensioned lumber, and glue-laminated timber is used in some large-scale structures. Generally, however, steel and concrete dominate as structural systems for large-scale structures. The improved strength to weight ratio of whole-timber makes it suitable for large-scale structures typically thought to be impossible in timber. By addressing the design of the largest and most abundant structures first, this research aims to maximize the volume of structural material replaced by whole-timber in the least time.

3. Built Precedents in Whole-Timber

Fig. 3.1.1: "Big Shed," interior view (Bennet, Self, and Randzio 2014)

Fig. 3.1.2: Flitch plate connection(Evans 2011)

Big Shed (2012):
Designers: Atelier 1, Buro Happold, Mitchell Taylor Workshop
Location: Hooke Park, Beaminster, Dorset, UK

The Big Shed (Fig. 3.1.1) uses bolted steel flitch plates (Fig. 3.1.2) to join untreated larch timbers into planar trusses for its primary structure. Flitch plates have the advantage of being able to resist both tension and compression, accomodate the dimensional irregularities of whole-timbers, and are relatively straightforward to design and assemble. However, they require a significant amount of steel, and the their load capacity is strongly limited by the bearing capacity of bolts within the timbers. Any penetrations in whole-timbers additionally reduce the capacity of connections by initiating fractures.

Fig. 3.1.3: Interior of workshop at Hooke Park (Bennet, Self, and Randzio 2014)

Fig. 3.1.4: Exterior of workshop at Hooke Park (Bennet, Self, and Randzio 2014)

The Workshop at Hooke Park (1989):
Designers: Richard Burton, Frei Otto, Buro Happold
Location: Hooke Park, Beaminster, Dorset, UK

The workshop at Hooke Park (Fig. 3.1.3) uses bent Norwegian Spruce thinnings shear-bolted together to acheive the structure for a lightweight membrane roof (Fig. 3.1.4). Shear bolt connections have the advantage of simplicity, low cost, and minimal steel, but have limited capacity because of their eccentricity, and because of the limited bearing capacity of bolts in whole-timbers.

Fig. 3.1.5: Space-truss in regularized round timbers at Doncaster Earth Center.

Doncaster Earth Centre Solar PV Canopy (1999):
Designers: Atelier 1, Feilden Clegg
Location: Doncaster, UK

The Doncaster Earth Centre photovoltaic canopy(Fig. 3.1.5) uses round, regularized larch timbers in a space-truss configuration, with steel node/flitch plate connections. While these connections allow for an efficient and expressive structural geometry in round timber, they are difficult to fabricate, non-repeating, and require large amounts of steel. They are also limited by the capacity of the bolts in the flitch plates.

3.2. The Whole-Timber Kit of Parts:

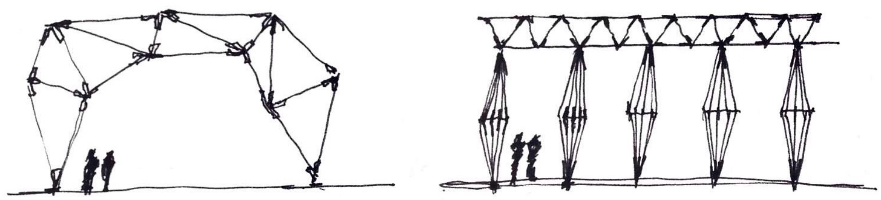


Fig 3.2.1: Custom whole-timber structure vs. whole-timber structure using modular, standardized elements.

The precedents shown demonstrate that large-scale structures are feasible in whole-timber. Each of these structures is a custom design, however, involving a unique connection design for the project, as well as a unique and continuous structural geometry. The cost, skill, and resources required to design, fabricate, and assemble such custom projects makes them inaccessible to a broader group of potential adopters.

The limiting factor for broader adoption of whole-timber as a structural system for large-scale structures is not any deficiency of the material itself, but the high cost and complexity associated with the design of whole-timber structures. This is due to the challenge of designing and fabricating connections for whole-timber elements, and the material-constrained design process specific to whole-timber.

The solution proposed by this thesis is a “kit of parts” in whole-timber: a set of structurally independent elements such as columns, beams, roof trusses, etc. with standardized connections and minimal steel (Fig. 3.2.1). Standardized geometries and connections can reduce design, fabrication, and installation costs by reducing project complexity. Designers interested in incorporating whole-timber into their structures can specify whole-timber structural elements just as they would specify standard steel or reinforced concrete elements today. This idea is inspired by the work of Whole-Trees, an architecture and structural firm based in Madison, Wisconsin which specializes in the design and fabrication of struturally independent whole-timber structural elements. (Gundersen 2014, 2015)

The Goal is Volume:

It is important to reiterate that the primary goal of this research is to increase the volume of this material being used in structural systems relative to conventional structural materials like steel, concrete, or glue-laminated timber. This may at first mean using whole-timber elements in only one part of structure - as the vertical-load bearing elements, for example. Structurally independent elements in whole-timber allow for the incremental adoption of whole-timber, thereby producing maximal embodied carbon offsets as quickly as possible.

4. Methods

4.1. Design Challenge: the Whole-Timber Column

The focus of this thesis is the design of the first and most important element in the whole-timber kit of parts: a structurally independent column in whole-timber.

The design criteria for a column in whole-timber are that it have:

1. Minimal steel
2. Maximum standardization
3. Maximum capacity

Multiple-Timber Columns

Steel, concrete, and glue-laminated timber columns can be sized according to their desired load capacity - that is, their sections can be increased or decreased based on the desired load. Whole timber is strictly limited by its section, and by its length. Thus, for cases when the desired capacity is greater than what is provided by the available whole-timber section, multiple elements must be used and connected to provide greater load capacity.

At short lengths and small loads, a single whole-timber is a practical solution. At greater lengths, multiple-timber solutions should be explored. For a tall column, the governing failure mode is generally buckling. Thus, any tall column design in whole-timber should have a geometry that maximizes its resistance to buckling.

Compression-Only Connections

Traditional timber structures such as covered bridges are a good reference for the design of minimal-steel whole-timber connections, because builders had to maximize capacity in timber-timber connections with minimal or no metal components. Tension connections in historic timber structures are complex, difficult to fabricate, and have reduced capacity. Compression connections, on the other hand, are easy to fabricate, and have high capacity.

In order to minimize steel and maximize capacity in the whole-timber column, I chose to use timbers only in compression, and any steel elements exclusively in tension, as cables or straps.

The Compound Column:

Based on these constraints, I arrived at a double-tetrahedral geometry (henceforth referred to as a compound column), in which six identical timbers are joined in three pairs, which meet the ends of the column, and spread to an equilateral triangle at its mid-height (Fig.4.1.2). Steel elements are used in tension to resist the outward spread of the column at its mid-height. This spread provides global buckling resistance.

Post-Tensioning:

Compression-only connections in the compound column are adequate under normal vertical loading, but when the column is being moved on site, or if it must resist uplift from a roof under wind-loading, these connections will be subjected to tension.

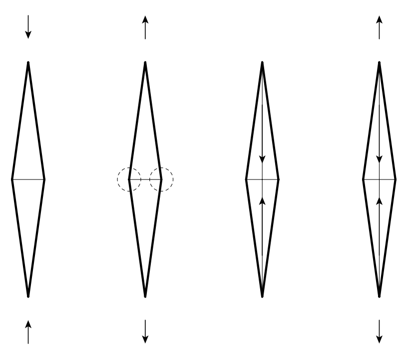


Fig. 4.1.1: Post-tensioning allows for compression-only connections

Post-tensioning allows additional compression to be introduced to the compound column, such that compression connections will never be in tension, even when the column is subjected to uplift or is being moved on site (Fig. 4.1.1). A central cable connected to bearing plates at each end of the compound column can be tensioned to add additional compressive load to the column as a whole.

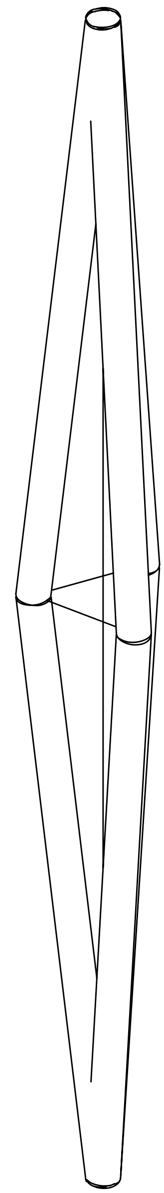


Fig 4.1.2: A "compound column" in whole-timber

Connection Design in the Compound Column:

The mid-height connection in the compound column (Fig. 4.1.3) must transfer compression between the two timbers, and lateral tension from the peripheral steel elements. In this design, steel straps are explored as a method of distributing this tensile load over the exterior of the timber, thereby minimizing penetrations which could limit the capacity of the connection.

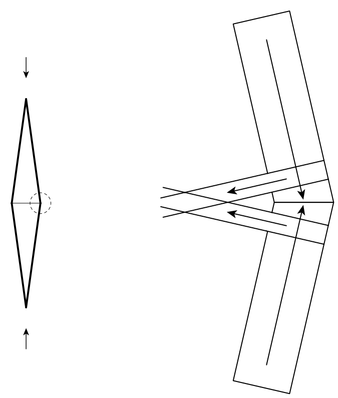


Fig. 4.1.3: Conceptual design for mid-height connection in compound column

The end-node connection of the compound column (Fig. 4.1.4) must provide a way for the centerlines of the timbers it connects to meet at a theoretical working point. This is to prevent the column from behaving as a mechanism, or developing bending moment at this connection. It must also transfer the tensile load from the central post-tensioning cable to each of the timbers it connects using a bearing plate at the end of column.

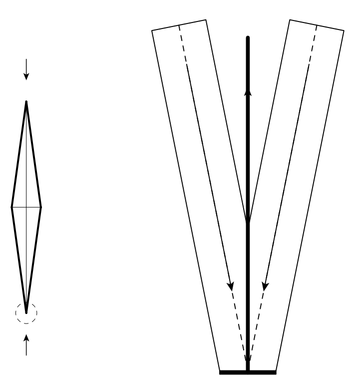


Fig. 4.1.4: Conceptual design for end-node connection in compound column

The Bundled Column:

An alternative multiple-timber column in whole-timber may be one in which timbers are “bundled” together in parallel using shear bolt connections (Fig. 4.1.5).

“Bundling” tapered timbers in even multiples provides a way to overcome some of the limitations posed by using single whole-timbers as columns.

A bundle of four tapered timbers may provide a practical, easily fabricated, and flexible column system which provides high and equal crushing capacity at each end.

The timbers could be arranged such that at least two faces of the column would be flat, providing an attachment for wall or envelope elements. Additionally, this system would be able to provide a large column grid spacing for applications where longer clear spans are needed.

The challenge with designing and fabricating a bundled column would be to develop a method for fastening the timbers together such that they could develop sufficient shear resistance so that they could benefit from each other in resisting buckling.

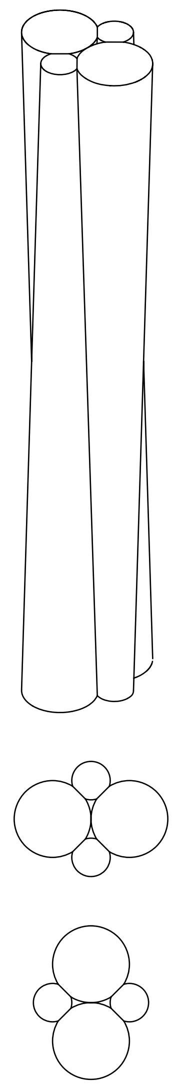


Fig. 4.1.5: “Bundled” column using 4 tapered timbers

4.2. Prototyping the Compound Column:

1/12, 1/4, and full-scale prototypes of the compound column were made (Fig.4.2.1, Fig. 4.2.2., Fig. 4.2.18). 1/2 and full-scale prototypes of compression-only connections were also made.

Fig. 4.2.1: 1/12-scale compound column prototype

Fig. 4.2.2: 1/4-scale compound column prototype

Structural Load Testing of Compression-Only Connection Prototypes

A compression-only connection prototype in whole-timber was made at 1/2-scale and tested to failure. This connection included a slot to contain a registration tab to help keep timbers aligned during construction. This connection failed at 68% the capacity of a clear section of whole-timber (28000 lbs./41000 lbs), suggesting that even this small penetration could iniate fracture and significantly reduce the capacity of the connection (Fig. 4.2.3, Fig. 4.2.4). Subsequent connection designed used no slot, relying on friction to keep the timbers aligned during assembly and while under load. Future connection designs may also incorporate wire lacing to prevent splitting.

Fig. 4.2.3: Initiation of splitting failure in compression-only connection.

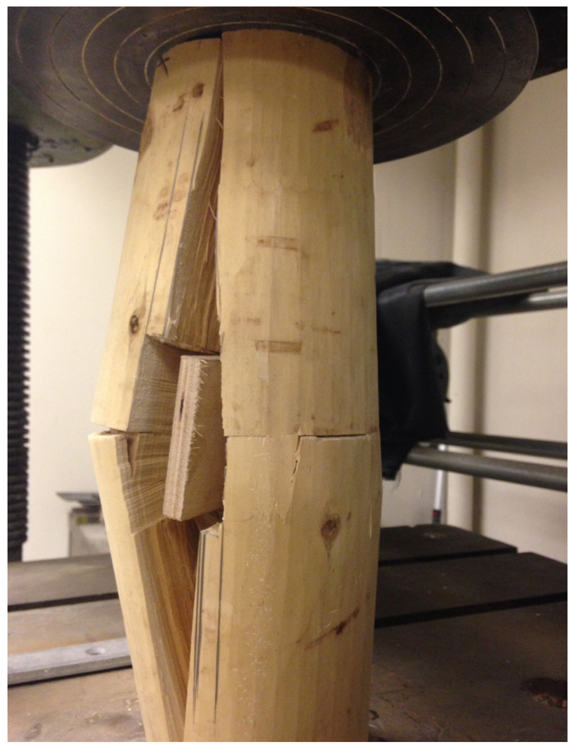


Fig. 4.2.4: Splitting failure in compression-only connection.

Structural Load Testing of Compound Column Prototypes

Preliminary load testing of the 1/12-scale model was done to observe its general behavior under load (Fig. 4.2.5). The column showed local buckling in one timber member, which eventually precipitated global buckling involving one pair of timbers.

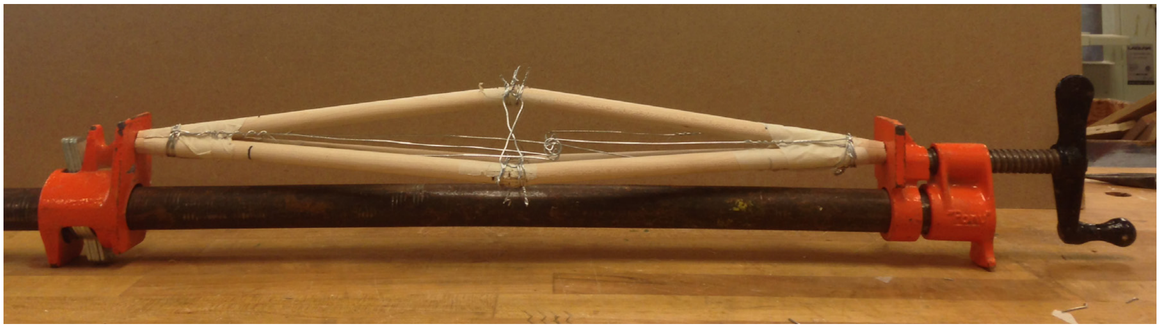


Fig. 4.2.5: Structural load testing setup for rapid-prototyping.

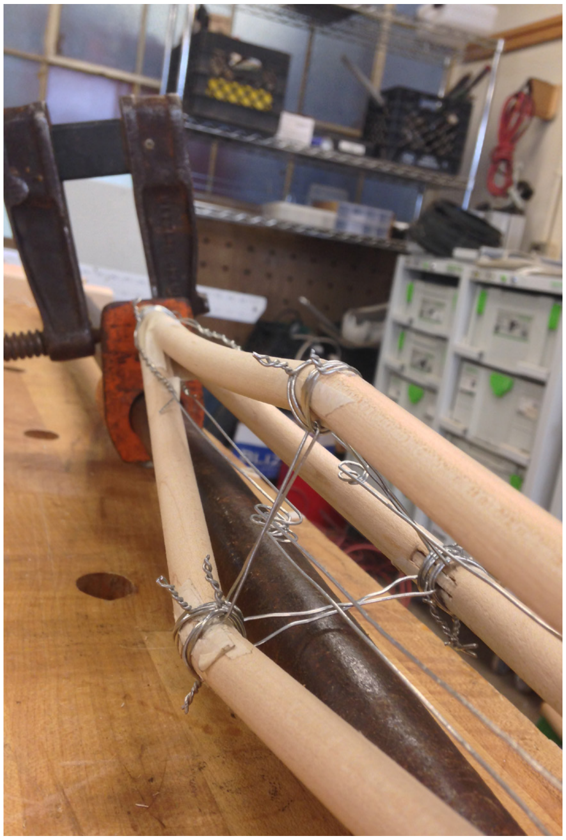


Fig. 4.2.6: Member buckling failure.

Fig. 4.2.7: Global buckling failure.

Full-Scale Prototyping

A prototype of the compound column was built at full-scale using 10' long 6" diameter red pine timbers (Fig. 4.2.8). A chainsaw was used to make the cross-cuts and beveled connections, and structurally rated steel hardware was used for the tension elements. Additional dimensioned lumber elements were added during assembly and set-up as an additional factor of safety in the event of a failure.

Each timber was first prepared with one beveled end and one angled cross-cut end (Fig. 4.2.9). Each timber in the compound column is identical, making this process straightforward. The larger end of each timber was oriented towards the ends of the compound column. A triangular template in dimensioned lumber was used to assemble each half of the full column in the form of a tripod (Fig.4.2.10). The two halves were then mated and aligned (Fig. 4.2.11) and strapping was added (Fig. 4.2.12) around the mid-section of the column. A central cable was added, and attached to steel bearing plates at each end of the structure, which were waterjetted from plate steel (Fig. 4.2.14). The central cable was tensioned to achieve stiffness and engage the lateral strapping. Finally, bracing was added, and the structure was lifted into vertical position using a crane (Fig. 4.2.17), and then secured against overturning using guy wires (Fig. 4.2.18).

Fig. 4.2.8: 8" diameter 10' debarked, untreated red pine poles.

Fig. 4.2.9. Beveling of ends of timbers.

Fig. 4.2.10: Joining beveled ends of timbers.

Fig. 4.2.11. Mating and aligning two halves of the compound column.

Fig. 4.2.12: Steel strapping at column mid-height.

Fig. 4.2.13: End-node connection without steel cap.

Fig. 4.2.14: Waterjetting steel cap for end-node.

Fig. 4.2.15: Steel end-node connection assembly.

Fig. 4.2.16: Compound column in horizontal position being before post-tensioning.

Fig. 4.2.17: Compound column being lifted by crane into standing position.

Fig. 4.2.18: Completed 20’ compound column prototype (with temporary dimensioned lumber bracing).

4.3. Estimating the Capacity of Tapered Timbers

All trees taper over their length. In order to reduce connection complexity, simplify calculations, and standardize appearance, whole tapered timbers are typically processed into constant-section timbers before being used structurally. (Fig. 4.3.1) This process dramatically reduces the buckling strength of these timbers, consumes energy, and wastes material.

Disregarding the additional negative effect of breaking fiber continuity on the exterior of the timber, regularizing a tapered timber into a constant-section timber theoretically does not reduce its capacity in crushing, as the tapered timber is already limited by the cross-sectional area of its smaller end. However, based on geometric effects alone, regularizing has a significant impact on buckling capacity.

Because there is as of yet no convenient way of estimating the strength of a tapered timber vs. the largest regularized timber that can be yielded from it, there is no way for structural engineers to take advantage of this natural strength when designing structures. The advantages of a uniform section for connection simplicity, appearance, and weight reduction thus typically outweigh the advantages of any potential strength or embodied energy savings from not regularizing.

The goal of this section is to demonstrate a simple method for computing the buckling capacity of tapered timbers, based on the work of Timoshenko and Gere (1961). This method should allow engineers to take advantage of the naturally high buckling capacity of whole, tapered timbers, while saving energy and material.

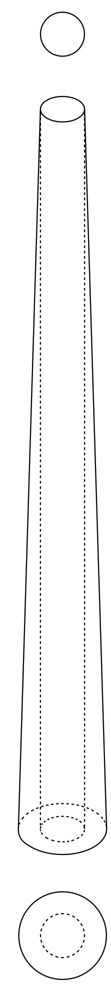


Figure 4.3.1: Naturally tapered whole timber with largest regularized timber that could be derived from it.

Elastic Buckling of a Column of Uniform Section:

Euler's buckling equation for long, slender columns of uniform section is:

Eq. 4.3.1:
$$P_{buckling} = \frac{\pi^2 EI}{(kL)^2},$$

Where E is the elastic modulus of the column, I is the moment of inertia of the column's uniform section, k is a factor to adjust for support conditions, and L is the length of the column.

Timoshenko and Gere's Method:

As taper in trees is roughly linear, a tapered timber can be reasonably approximated as a truncated cone. Timoshenko and Gere provide the following equation for computing the buckling capacity of a truncated cone (1961):

Eq. 4.3.2:
$$P_{buckling} = \frac{mEI_1}{L^2},$$

where m is a coefficient which corresponds to the ratio of the moment of inertia at the top of the column I_2 to the moment of inertia at the base I_1. Timoshenko and Gere used trial and error to determine the coefficient m, creating the following table:

Table 4.3.1: The factor *m* for given I_2/I_1 ratios. Adapted from (Timoshenko and Gere 1961).

I_2/I_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
m	4.808	6.02	6.84	7.48	8.008	8.464	8.868	9.232	9.564	π^2

Note: Timoshenko and Gere solved m *for a* k *= 2 column (pinned at base, unsupported at top). To adjust* m *to represent a* k *= 1 column (pinned/pinned), I multiplied their values of* m *by a factor of 4.*

Note: as I_2/I_1 *approaches 1,* m *approaches* π^2*, which describes the case of a column of uniform section.*

Finding a Continuous Function for the Factor *m*:

I used Microsoft Excel's trendline tool to generate a power function to fit this data. My goal was to find a simple function that was easy to remember that would allow engineers to estimate the factor *m* for an arbitrary I_2/I_1. The function generated was:

Eq. 4.3.3:
$$m = 9.9101 * (\frac{I_2}{I_1})^{0.3113}$$
$$R^2 = 0.9998$$

I noted that 9.9101 was remarkably close to π^2 and 0.3113 remarkably close to 1/3. These substitutions result in the following function for the factor m:

Eq. 4.3.4:
$$m = \pi^2(\frac{I_2}{I_1})^{1/3}$$
$$R^2 = 0.9995$$

The error between this function and the values computed by Timoshenko and Gere was less than 5% for I_2/I_1 from 0 to 1, and always conservative. The R^2 value of the function is only marginally lower than the original fit function produced by Excel.

Table 4.3.2: Values of m predicted by Eq. 4.3.4 vs. those computed by (Timoshenko and Gere 1961)

I_2/I_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
T&G 1961	4.808	6.02	6.84	7.48	8.008	8.464	8.868	9.232	9.564	π^2
Eq. 4.3.4	4.581	5.772	6.607	7.272	7.834	8.324	8.763	9.162	9.529	9.870
Error	-4.72%	-4.12%	-3.41%	-2.78%	-2.18%	-1.65%	-1.18%	-0.76%	-0.37%	0.00%

Because Eq. 4.3.4 is easy to remember, always conservative, and deviates less than 5% from the values provided by Timoshenko, it will be used for all subsequent tapered timber buckling calculations, as this is the equation likely to be used by engineers in the design process for tapered whole-timbers.

If the diameters at both ends of a given tapered timber are known, the equation for estimating the buckling capacity of that timber is:

Eq. 4.3.5:
$$P_{buckling} = (\frac{\phi_2^4}{\phi_1^4})^{1/3} * \frac{\pi^2 EI}{(kL)^2},$$

Where ϕ_1 is the diameter at the large end of the timber, and ϕ_2 is the diameter at the small-end of the timber.

Taper Coefficients:

For cases when timbers have not already been cut to length, a taper coefficient can be used to determine the diameter at a given length when the base diameter is known. Wolfe and Kluge (2005) provide a range of taper coefficients between 0.011 and 0.077 inches circumference / inch length for various softwood timber species. 0.03, or 3% taper, is chosen as a median value and used for all subsequent calculations in this thesis.

If the length of the timber and its diameter at the base ϕ_1 is known, its diameter at its small end ϕ_2 can be found by:

Eq. 4.3.6:
$$\phi_2 = \phi_1 - \frac{C_t L}{\pi},$$

where C_t is the taper coefficient and L is the length of the timber.

Table 4.3.3: Small-end diameters (in inches) for timbers of given base diameters and lengths with a taper coefficient of C_t = 0.03.

	5′	10′	15′	20′	25′	30′	35′	40′	45′	50′
4″	3.4	2.9	2.3	1.7	1.1	0.6				
6″	5.4	4.9	4.3	3.7	3.1	2.6	2.0	1.4	0.8	0.3
8″	7.4	6.9	6.3	5.7	5.1	4.6	4.0	3.4	2.8	2.3
10″	9.4	8.9	8.3	7.7	7.1	6.6	6.0	5.4	4.8	4.3
12″	11.4	10.9	10.3	9.7	9.1	8.6	8.0	7.4	6.8	6.3

For a timber of known base diameter, length, and taper coefficient, I_2/I_1 can be expressed as:

Eq. 4.3.7:
$$I_2/I_1 = \frac{(\phi_1 - \frac{C_t L}{\pi})^4}{\phi_1^4}$$

Estimating Strength Losses from Regularizing Tapered Timbers

This allows us to estimate the fraction of buckling capacity lost from regularizing a tapered timber to the largest uniform section that can be yielded from it for a range of base diameters and lengths (Table 4.3.5). It should be noted that very long timbers (greater than 25 feet) would likely be cut to shorter lengths before regularizing, as this process is limited by the size of lathes and the stability of the timber in the machine.

Table 4.3.4: Ultimate capacity in kips of tapered timbers with a taper coefficient C_t = 3%. Crushing failures denoted with superscript [c].

	5′	10′	15′	20′	25′	30′	35′	40′	45′	50′
4″	42.1	8.2	2.7	1.0	0.4	0.1				
6″	139[c]	49.3	18.5	8.6	4.4	2.3	1.2	0.6	0.2	
8″	260[c]	168	66.5	32.9	18.3	10.9	6.7	4.2	2.6	1.5
10″	419[c]	369[c]	174	89.2	51.5	32.0	20.8	13.9	9.5	6.5
12″	615[c]	555[c]	378	197	116	74.1	49.7	34.4	24.4	17.6

Table 4.3.5: Percent reduction in buckling capacity from regularizing tapered timbers to the largest uniform section that can be yielded from them. C_t = 3%.

	5′	10′	15′	20′	25′	30′	35′	40′	45′	50′
4″	34%	59%	78%	90%	97%	99%				
6″	n/a	43%	59%	72%	82%	90%	95%	98%	99%	
8″	n/a	34%	48%	59%	69%	78%	84%	90%	94%	97%
10″	n/a	16%	40%	50%	59%	67%	75%	81%	86%	90%
12″	n/a	n/a	34%	43%	52%	59%	66%	72%	78%	82%

4.4. Estimating Capacity and Embodied Carbon of Various Whole-Timber Columns

A major goal of this research is to make conservative and realistic estimates of the embodied carbon of whole-timber structural systems vs. conventional steel, concrete, and glue-laminated timber systems. To do this, the capacities and embodied carbon of each column type must be estimated. These are computed for columns of 1 through 4 stories in height. The total embodied carbon per square foot is then computed for each column system at every story height, given a floor loading of 150 PSF. The material properties assumed for these calculations are given in Table 4.4.1.

The previous section demonstrated a simple method for computing the buckling capacity of tapered timbers. This method will be used to compute the load capacities of various whole-timber columns in the following section.

The embodied carbon of building elements is found by multiplying their weight by the embodied carbon coefficient for their material. The density and embodied carbon coefficients of the materials used in these columns are listed in Table 4.4.1.

Steel elements are included in the total embodied carbon calculation for each column, but steel connectors which would attach each column to other elements in the structure are not included, as these would be roughly similar for each column type, and will depend strongly on the type of spanning system used.

For the purposes of this calculation, the diameters of whole-timber and glue-laminated columns is chosen as 6”,8”,10”, and 12” for story heights of 1,2,3, and 4, respectively. Corresponding W-sections are chosen such that the moment of inertia of each section is greater for each subsequent floor. See Table 4.4.2 for details on the W-sections chosen.

Table 4.4.1: Values used to estimate the capacity and embodied carbon of various columns. *ρ: Density; EC: Embodied Carbon; E: Elastic Modulus; σ: Allowable Compressive Stress; S: Safety Factor for Buckling.*

	lbs/in3	*kgCO2e / kg*	*10^6 psi*	*10^3 psi*	*dimensionless*
	ρ	*EC*	*E*	*σ*	*S*
Steel	0.28	1.46	29	25	3
Glue-lam.	0.025	0.87	1.5	1.5	3
Whole-Timber	0.03	0.2	1.5	1.0	3

The Single-Timber Column:

Estimating Capacity:

Estimating the capacity of a single-timber column, for the purposes of this calculation, is essentially the same as estimating the capacity of a single tapered timber.

The crushing capacity of a single-timber column is governed by the cross-sectional area at the small-end of the timber:

Eq. 4.4.1: $$P_{crushing} = \sigma A,$$

Where A is the cross-sectional area, and σ is the allowable stress in compression.

The ultimate buckling capacity of a single-timber column is computed as described in the previous section, using Eq. 4.3.5.

Estimating Embodied Carbon

The volume of material in the single-timber column can be found using the equation for the volume of a truncated cone:

Eq. 4.4.2: $$V = \frac{\pi L}{3}(r_1^2 + r_1 r_2 + r_2^2),$$

where r^1 and r^2 are the radii at the large and small end, respectively, and L is the length of the timber.

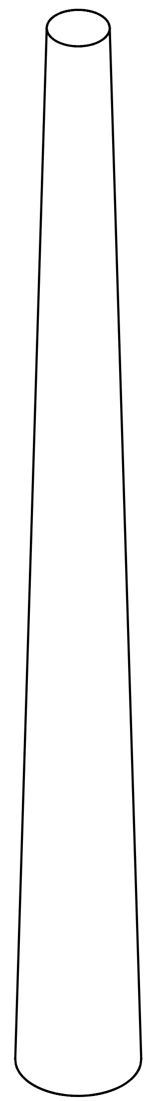


Fig. 4.4.1: The single-timber column.

The Bundled Column:

Estimating Capacity:

The crushing capacity of a bundled column (Fig. 4.4.2) is governed by the total cross-sectional area of the timbers at mid-height, assuming connections that can fully and evenly distribute load to all four timbers. This is computed using Eq. 4.4.1.

Accurately estimating the buckling capacity of a bundled column will require prototyping and structural load testing. The very conservative method used in this paper is to take the simple sum of the buckling capacities of the individual tapered timbers, using the method described in section 4.3. This method assumes no shear capacity in the connections between timbers.

Embodied Carbon:

The timber volume of the bundled timber is found using the same method as described for the single-timber column, using Eq. 4.4.2.

The weight of all steel connectors in the column is conservatively estimated at 10% the total timber weight of the column.

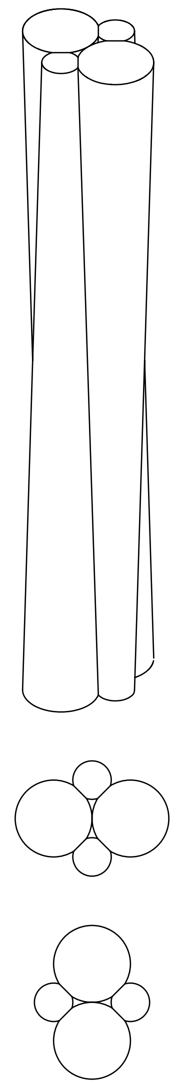


Fig. 4.4.2: “Bundled” column using 4 tapered timbers.

The Compound Column:

Estimating Capacity

In order to estimate the capacity of the compound column, we need to determine the forces in each of the members for a given external load and post-tensioning load. We can do this using graphic statics.

3D structures can be represented accurately in graphic statics using the method of projection. (Jasienski, Fivet, and Zastavni 2014)

We know the force in each timber will be identical by symmetry - the compound column is radially symetric about its centerline and symmetric about its mid-height plane.

Therefore if we can accurately represent the force in one timber using a graphic statics projection, we know the forces in all of them.

We can also use graphic statics to find the minimum required post-tensioning force in the central cable such that the compound column can be lifted from horizontal from one end (the most stressful lifting case).

Accounting for Taper

Every timber in the compound column is assumed to be tapered with a taper coefficient C_t = 3%. The calculation for the buckling capacity of each timber takes this into account, using the method described in section 4.2. The large ends of the timbers are oriented towards the ends of the compound column to provide additional crushing capacity there.

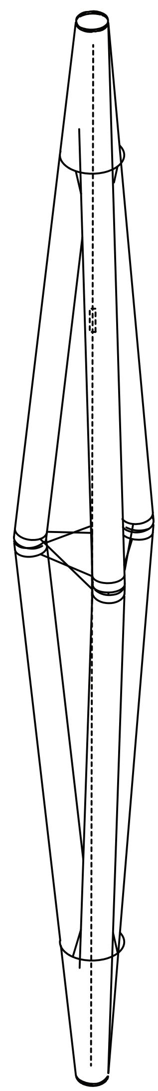


Fig. 4.4.3: “Bundled” column using 4 tapered timbers.

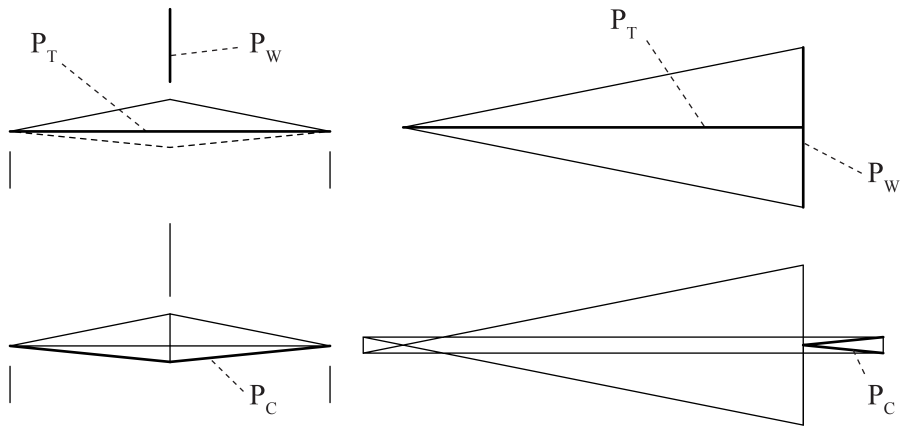


Figure 4.4.4: Graphic Statics projection of compound column in a horizontal orientation being lifted on site. P_W: self-weight of the compound column; P_T: minimum post-tensioning in central cable; P_C: compressive force in bottom timbers of compound column under sufficient post-tensioning.

Accounting for Global Buckling

Preliminary structural load testing suggests that the compound column has global buckling behavior which occurs as a result of instability caused by a single member buckling. Because this behavior is difficult to calculate for a wide range of column sizes, a conservative safety factor of 3 is assumed for member buckling such that such a failure would not be likely to occur. Further destructive testing and analysis must be done to better characterize the global buckling behavior of the compound column.

Finding the Minimum Initial Post-Tensioning

In the compound column, post-tensioning is used to keep the timber-timber connections in compression, and steel cable in tension under conditions of uplift and when the structure is being lifted during installation on site. The most stressful lifting case for the column is when it is being lifted from a horizontal orientation, as this could place one or more of the spokes of the column in tension.

A two-dimensional projection of the column in a horizontal orientation produces a minimum central cable tension of 2.5 times the self-weight of the compound column when supported at both ends (Fig. 4.4.4). A conservative safety factor of 3 is chosen to account for dynamics and for any imperfections in fabrication. Thus the tension in the central cable should be 7.5 times the self-weight of the column.

It is important to note that though this degree of post-tensioning is required during installation, it can be relaxed once the structure is completed. The remaining post-tensioning can be chosen such that the compound column can resist a desired uplift load.

Finding Member Forces in the Compound Column

A two-dimensional projection of the compound column under vertical loading and post-tensioning viewed from the side shows the forces in the timber members and cables.

For an uplift post-tensioning of 10% of dead load, the relationship between the force in one timber is about 3/8 the external load on the column. The force in each peripheral cable is about 1/10 the external load (Fig. 4.4.5).

Connection Capacity

Further destructive testing of connections in the compound column is necessary to more accurately determine their capacity. The crushing capacity of the mid-height and end-node connections is conservatively assigned a reduction of 50%.

Steel Cable Capacity

For the purposes of these calculations, steel elements are assumed to be sized to fail after any timber elements have failed. It is important to note that future iterations of the compound column may need to incorporate sacrificial steel elements designed to fail ductilely just under the predicted capacity of the column. This would prevent or slow what would otherwise might be a sudden failure in the timber in the column.

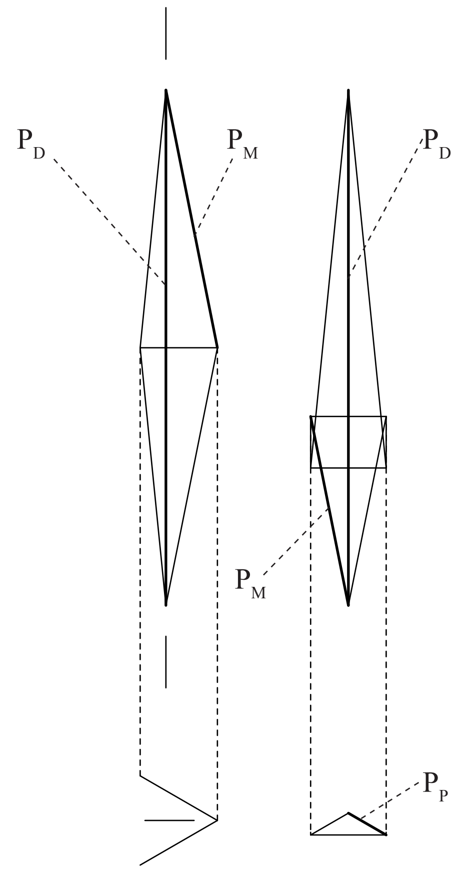


Figure 4.4.5: Graphic Statics projection of compound column under vertical loading and 10% post-tensioning. P_D: external dead load applied to column; P_M: force in individual timber member; P_P: force in peripheral cable.

Embodied Carbon:

The weight of timber in the compound column is computed by computing the weight of an individual tapered member (see Eq. 4.4.6) and multiplying by 6. The reduction in volume due to beveling and angled connections is not accounted for. The weight of steel in the column is conservatively assumed to be 10% of the total timber weight of the column, based on the amount of steel used in the full-scale prototyping of the structure.

Glue-Laminated Column

Estimating Capacity:

The capacity of a glue-laminated timber column (Fig 4.4.6) is found using standard crushing and buckling equations (Eq. 4.4.1, and Eq. 4.3.1, respectively).

Embodied Carbon:

The weight of the glue-laminated column is found using the formula for a volume of a cylinder:

Eq. 4.4.3: $$V = AL,$$

where A is the cross-sectional area, and L is the length of the column.

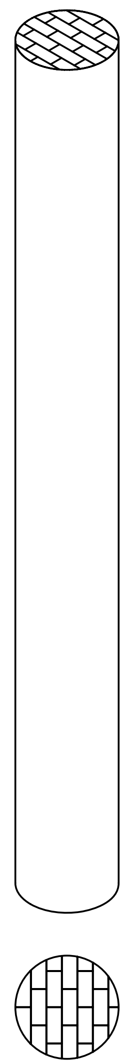


Fig. 4.4.6: Glue-laminated column.

Estimating Capacity of a Steel W-Section:

The American Institute of Steel Construction (AISC) provides tables listing the properties of various steel W-sections, including area, moment of inertia, and weight per linear foot (Table 4.4.2).

These values are used to compute the crushing and buckling capacity of W-section columns (Fig. 4.4.7) compared to whole-timber and glue-laminated columns, using Eq. 4.3.1. for crushing, and Eq. 4.2.1 for buckling.

Table 4.4.2: Moment of inertia *I*, area *A*, and weight per linear foot *w* of various W-sections.

	in^4	in^2	*lb/ft*
Section	*I*	*A*	*w*
W 14 x 43	45.2	12.6	43
W 14 x 53	57.7	15.6	53
W 14 x 68	121	20	68
W 15 x 82	148	24.1	82

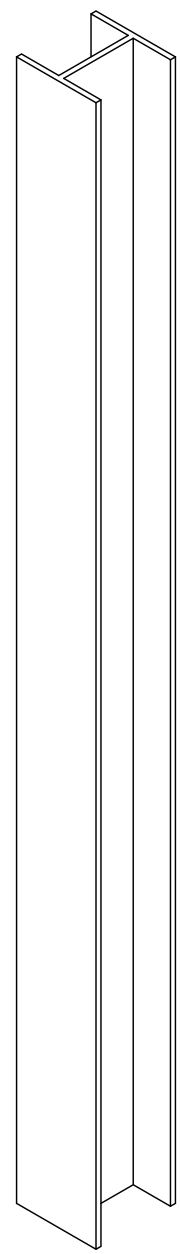


Fig. 4.4.7: Steel W-section column

4.5. Performance Metrics to Compare Whole-Timber Columns:

It is important to establish useful metrics to compare the performance of various whole-timber columns vs. conventional columns.

Strength / Weight (P/W):

Strength per unit weight is a good metric of the overall material efficiency of the column compared to other columns in the same material.

Strength per Carbon Emissions (P / EC):

This metric provides a basis for the comparison of the performance of individual columns in terms of their embodied energy vs. capacity.

Grid Spacing

If a column will be used to support a floor slab or roof, it is important to understand what the minimum required column spacing must be in order to support that load (Fig 4.5.1). The minimum required column grid spacing s in feet is found using the following equation:

Eq. 4.5.1:

$$s = \sqrt{\frac{P}{P_{sf}}},$$

where P is the maximum allowable load of the column in pounds-force_ and P_{sf} is the loading of the slab or roof in pounds per square foot. For the calculations in this paper, 150 PSF is assumed for floor loads, and 100 PSF is assumed for roof loads. Embodied carbon per foot is found by:

Eq. 4.5.2:

$$EC/ft^2 = \frac{EC}{s^2}$$

Carbon Emissions per Square Foot (EC / sf):

The most important comparative performance metric is the amount of embodied carbon emmitted per square foot of supported floor area. This value corresponds to the overall efficiency of the structural system in terms of its embodied carbon impact and should be considered the definitive metric for establish the performance of one structural system vs. another.

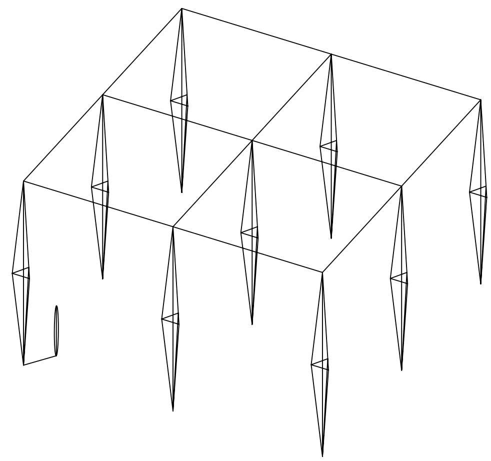


Fig 4.5.1: 20’ Compound columns arranged in a 15’ grid. 6’ person shown for scale.

Weight per Square Foot (W / sf):

This metric provides a measure of the contribution of a column grid system to the self-weight of the structure per unit floor area. This may be especially important in tall structures.

5. Results

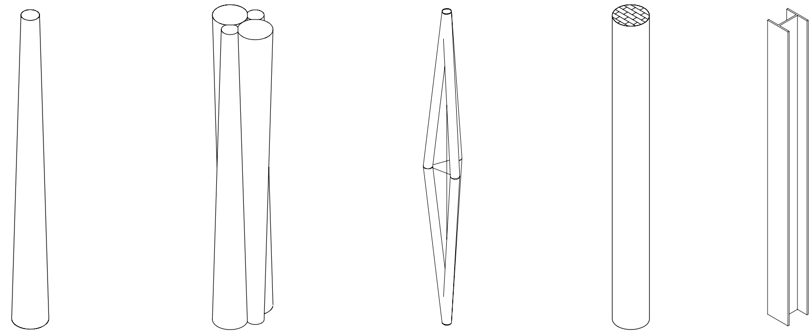
5.1 Performance of Various Whole-Timber Columns vs. Glue-Lam & Steel Columns

The results of the structural and embodied carbon calculations for whole-timber column systems of various story heights can be found in Table 5.1.2. This table includes values for the following quantities listed in Table 5.1.1.

Table 5.1.1. Performance metrics represented in Table 5.1.2

Symbol	Description	Units
P:	Maximum Load Capacity	10^3 lbs (kips)
P / W:	Strength to Weight Ratio	lbs / lbs
P / EC:	Load per lb of Carbon Emmissions	lbs / lb CO_2e
Spacing:	Minimum Column Grid Spacing (Square)	ft
EC / sf	Embodied Carbon Emmissions per Floor Area	lb CO_2e / ft^2
W / sf	Weight per Floor Area	lbs / ft^2

Table 5.1.2: Comparison of Whole-Timber Columns vs. Glue-Laminated and Steel Columns



	Single	Bundled	Compound	Glue-Lam	W-Section
1 Story	*ϕ = 6"*				*W14x43*
P	10.7	42.8	14.1^c	15.1	189^c
P / W	111	111	42	149	366
P / EC	556	321	209	171	251
Spacing	8	17	10	10	35
EC / sf	0.27	0.47	0.72	0.88	0.60
W / sf	1.3	1.48	3.9	1.0	0.41
2 Story	*ϕ = 8"*				*W14x53*
P	6.8	27.3	25.1^c	12.0	66
P / W	23	23	22	33	52
P / EC	113	65	64	38	36
Spacing	7	13	13	9	21
EC / sf	1.33	2.30	2.33	3.95	4.20
W / sf	6.6	7.31	7.4	4.5	2.87
3 Story	*ϕ = 10"*				*W14x68*
P	6.4	25.5	39.3^c	13.0	62
P / W	9.7	9.7	15.4	15.3	25
P / EC	49	28	45	18	17
Spacing	7	13	16	9	20
EC / sf	3.08	5.33	3.36	8.5	8.7
W / sf	15.4	16.9	10.7	9.8	5.94
4 Story	*ϕ = 12"*				*W14x82*
P	6.7	26.7	56.5^c	15.1	43
P / W	5.6	5.6	11.9	9.3	11
P / EC	28.0	16.2	34.3	10.7	7
Spacing	7	13	19	10	17
EC / sf	5.37	9.3	4.37	14.0	20.3
W / sf	26.8	29.5	13.9	16.1	13.9

Units: P: kips; P / W: lbs/lbs; P/EC: lbs/lbCO$_2$e; Spacing: ft; EC / sf: lbCO$_2$e/ft^2; W / sf: lbs/ft^2 ;
Note: crushing failures denoted with superscript ^{c}.

6. Conclusions:

6.1. Analysis of Results:

The calculation results demonstrate that, under conservative assumptions, whole-timber column systems, although generally much lower in gross load capacity than steel systems, have a lower embodied carbon per unit floor area than steel systems for all but one case (the compound column at a single story). The embodied carbon savings in the best case (the compound column at four stories) are as high as 68%.

The single-timber column has the lowest embodied carbon per floor area of all the structural systems at heights between one and three stories, while the compound column performs the best at 4 stories, and potentially at greater heights.

While the single-timber column may have the best performance in terms of embodied carbon, the maximum column grid spacing for a single-timber system for a 150 PSF floor load is only 7 to 8 feet. Systems such as the bundled timber may allow for a larger column spacing, possibly at the expense of greater embodied carbon per floor area.

The high performance of the compound column compared to other columns at 4 stories can be attributed to the fact that it is the only column which is still governed by crushing at this height. The compound column has excellent buckling resistance because its buckling failure is governed by the buckling of its individual members, which each only carry about 3/8 of the full column load and are approximately 1/2 of the full length of the columns.

6.2. Summary of Contributions:

The ultimate goal of this research is to increase the adoption of whole-timber structural systems. The intent of this thesis has been to demonstrate feasibility of novel, standardized whole-timber column designs, develop a method for designers to easily estimate the capacity of whole-timbers in their natural tapered form, and to quantify the embodied carbon savings of whole-timber column systems.

6.3. Potential Impact:

Though this thesis has thusfar focused on use of small-diameter whole-timber from overstocked forests in the United States, small-diameter whole-timber is abundant in most forested regions of the world, many of which suffer from the same issues of stand overcrowding. In tropical regions, bamboo has similar properties and poses similar design challenges to small-diameter whole-timber when used as a structural material.

The design of whole-timber structural systems is not timber species-specific. That is, the structural geometries developed in this paper for whole-timber columns can be applied to any timber or bamboo species. Connection designs, however, should be specific to their timber species, material, and economic context.

6.4. Future Work:

There are dozens of fascinating and important research questions to be explored in the development of whole-timber structural systems. These include, among many others:

1. Accurately quantiying the embodied carbon of whole-timber using a complete life-cycle analysis of existing whole-timber structures.

2. Developing other elements of the whole-timber kit of parts, including beams, roof trusses, towers, etc.

3. Quantifying the buckling capacity of swept or crooked timbers to increase the number of trees that can be used as structural elements in their whole form.

6.5. Closing Thoughts:

Current structural systems for large-scale structures are unsustainable. They use non-renewable, non-recyclable materials, the harvesting, processing, and assembly of which degrade the environment and driving irreversible climate change. These structures are not recyclable or biodegradable at end of life, and they are toxic to the humans that build and occupy them, as well as the environment.

The guiding philosophy behind the design of whole-timber structural systems in this thesis is the idea of plant-based structural systems. Some plants, such as trees, bamboo, and grasses, have inherent structural capacity in their whole form. Using them as such in structural systems on a large-scale not only saves energy and acheives structural performance, but does so without the introduction of non-biodegradable or toxic components, and with minimal or positive impact on the ecologies they are sourced from. At the end of life of such a structure, most or all of the structure can simply biodegrade, or be recycled as useful biomass.

This research has shown that the limit to the design of whole-plant based structures is not the materials themselves, but our collective imaginations, and our willingness to push the boundaries of what is possible in structural design.

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Appendix A: Speculative Designs for Large-Scale Whole-Timber Structures

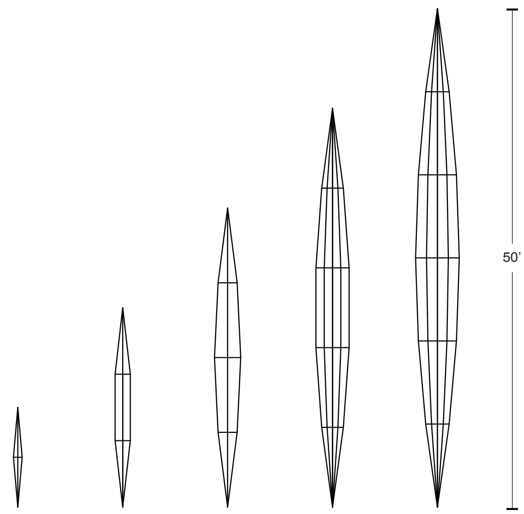

Fig. A.1: Large-scale compound columns in whole-timber

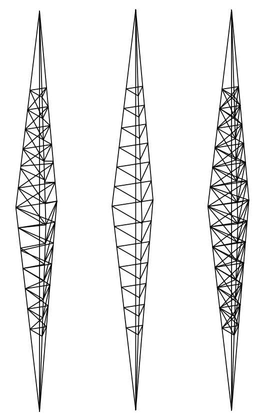

Fig. A.2: Bracing for compound columns

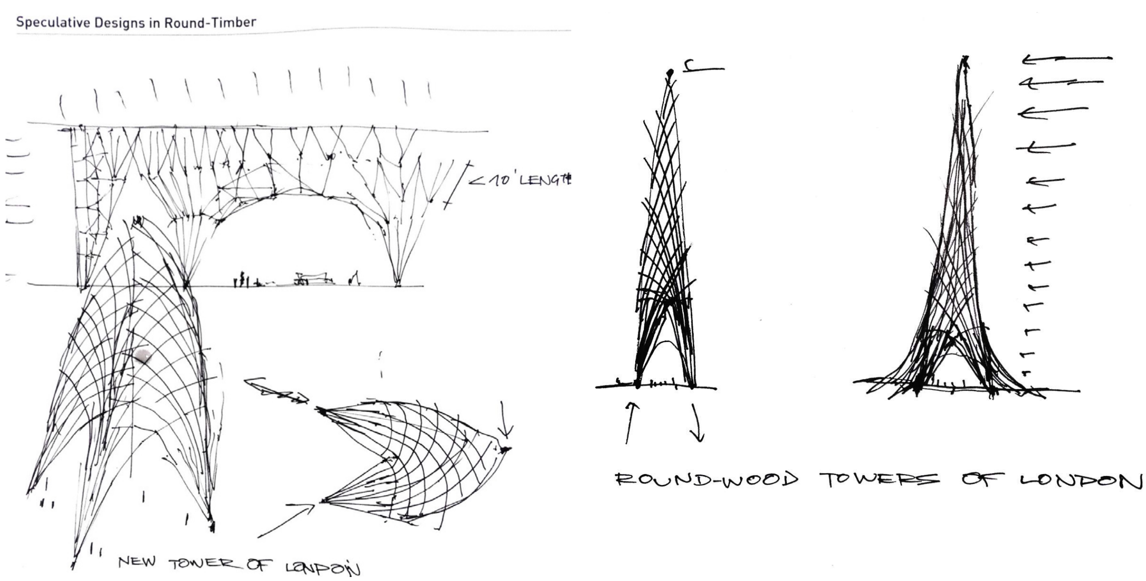

Fig. A.3: Large-scale towers and long-span roofs in whole-timber

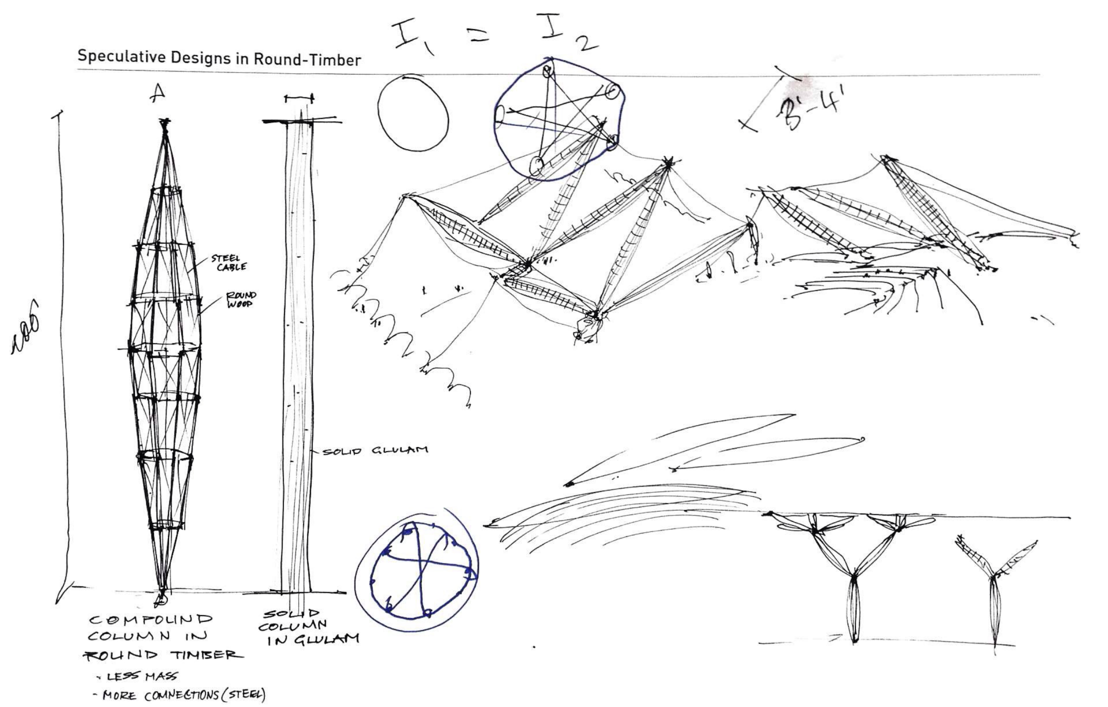

Fig. A.4: Large-scale membrane structures using large-scale compound columns.

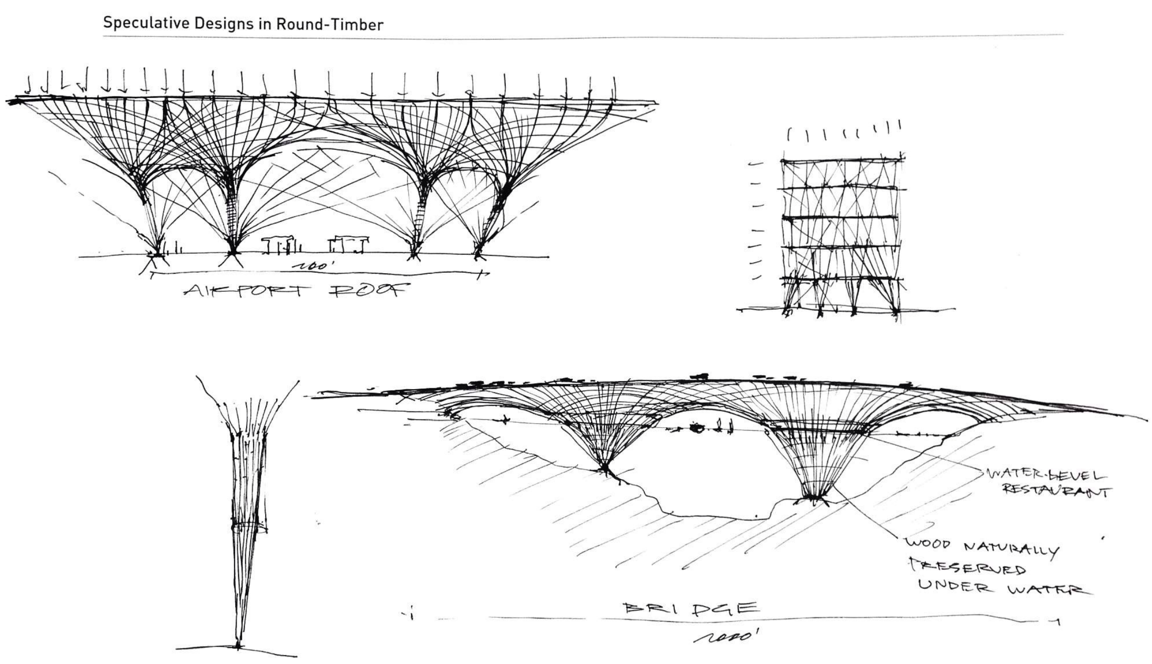

Fig. A.5: Long span roof, bridge, and office tower in whole-timber