PLANNING & DEVELOPMENT SERVICES ADVISORY

Post Frame (Pole Barn) Construction

Purpose

Post-frame (pole barn) construction was traditionally used for agricultural (farm) buildings, but its use has expanded rapidly in urban areas for non-agricultural applications such as office buildings, industrial facilities, and assembly buildings. The National Building Code (NBC), requires wood design to conform to CSA O86, *Engineering Design in Wood.* However, CSA O86, does not provide a complete design method for post-frame construction, particularly for roof diaphragm and shearwall systems without sheathing, as well as for embedded posts. Rather than requiring a formal alternative solution for each post-frame building, the sealed drawings shall include sufficient details to identify the design methods and criteria being utilized. This advisory and accompanying appendix provides guidance to support applicants and designers in ensuring compliance and structural integrity.

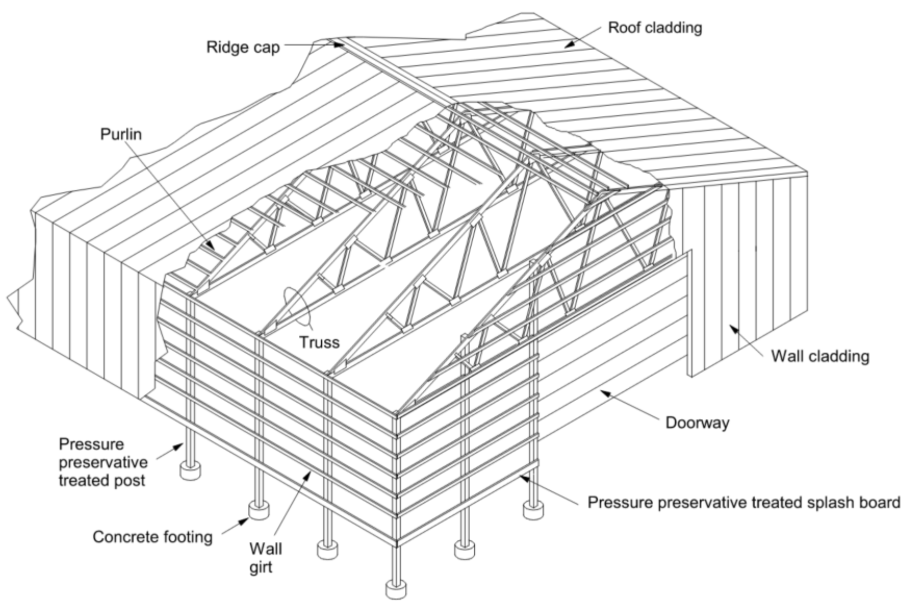

Figure 1: *Simplified diagram of a post-frame building. Some components such as permanent roof truss bracing and interior finishes are not shown.*

Lateral-Loads-Resisting System

Roof and ceiling diaphragms resist lateral (side-sway) forces by acting as large, stiff plates that transfer loads from wind or earthquake to wall posts and/or shearwalls.

Diaphragms made from sheathing materials such as plywood and OSB are well-documented, but less information is available on wood-framed, metal-clad diaphragms, which are widely used in the post-frame building industry.

This portion of the advisory provides some basic information about two lateral load resisting systems (see the following sections):

- Diaphragm-Shearwall Resisting Systems, and
- Post-Frame Resisting Systems.

Design Note: As part of the submitted designs, designers shall include details demonstrating one of the following approaches for resisting lateral loads:

- A) Where relying upon diaphragm-shearwall resisting systems: details showing that the diaphragms are sufficiently stiff (diaphragm-shearwall resisting system). See the Appendix for more information; OR
- B) Where the diaphragms are not sufficiently stiff: details confirming that the posts have been designed to provide lateral resistance for the building (post-frame resisting systems).

Diaphragm-Shearwall Resisting Systems

A shearwall is any interior or exterior wall that provides measurable racking resistance. Most of the load acting on a diaphragm is transferred to the foundation by shearwalls orientated parallel to the direction of the applied load.

Figure 2 illustrates how wind load applied to a sidewall is transferred through the roof diaphragm to the end shearwalls and an interior shearwall. Conversely, when wind load is applied to the end wall, the side walls serve as shearwalls, transferring the load from the roof diaphragm to the foundation system.

A typical metal-clad roofing system without sheathing beneath is considered a flexible roof diaphragm; therefore, the design of the posts requires the posts to contribute to resistance of lateral forces. However, when the diaphragm stiffness is significantly greater than the stiffness of interior post frames, the designer may assume that the diaphragm and shearwalls are infinitely stiff (see Appendix for more details on this topic). Under this assumption, 100% of the applied eave load is transferred by the diaphragm to shearwalls, and none of the applied eave load is resisted by the frames (posts). Since all eave load is assumed to be transferred to the shearwalls, no special analysis tools or design tables are required to determine load distribution between diaphragms and post-frames.

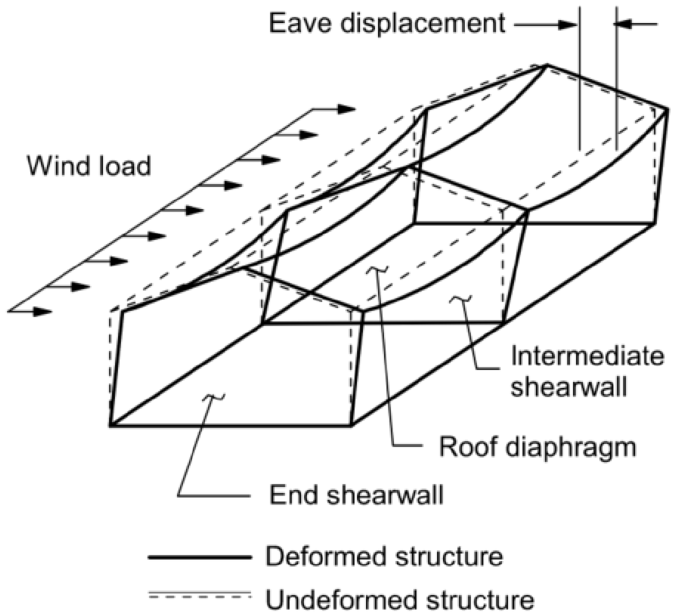

***Figure 2**: Example of diaphragm action in which the roof diaphragm transfers the load to three shearwalls – one interior and two exterior walls.*

Post-Frame Resisting Systems

When a load is applied perpendicular to a structure's sidewall, the posts may provide lateral resistance. The amount of load an individual post-frame transfers to the foundation depends on: (1) the in-plane shear stiffness of the diaphragm, and (2) the relative racking stiffness of the post-frame compared to other post-frames and shearwalls.

If a diaphragm is designed with high shear stiffness, diaphragm action is enhanced, allowing it to transfer loads from post-frames with low racking stiffness to shearwalls and post-frames with higher racking stiffness. However, if the shear stiffness of the diaphragm is relatively low then load transfer is minimal, and the structure behaves more like a series of independently acting post-frames.

Other Important Topics for Post-Frame (Pole Barn) Buildings

The remainder of this advisory goes through a variety of topics that are important to consider in the design of post-frame (pole barn) buildings. The designer shall ensure that each item specifically requested to be shown on the design drawing is included (these are indicated under each section where there is a '**Design Note**'). Additional details are available in the Appendix.

Light-gauge Steel Cladding Diaphragm Tables (CSSBI-B18-16)

As mentioned above, NBC and CSA O86 do not provide guidance for design of diaphragms and shearwalls that utilize steel cladding. However, the Canadian Sheet Steel Building Institute has a document titled *Light-gauge Steel Cladding Diaphragm Tables* (CSSBI-B18-16), which is often utilized in the design of such buildings. More details on this subject are provided in the Appendix.

Design Note: As part of the submitted designs, if steel cladding is being used to create a diaphragm, the sheet steel and fastening specifications must be clearly identified on the sealed drawings, and the drawings shall clearly reference the standard being followed for the design.

Shearwalls

A significant portion of the shear forces in roof and ceiling diaphragms is transferred to the building foundation through shearwalls. In many post-frame buildings, only the exterior walls (end walls and side walls) are available for this purpose. However, where present, interior partition walls can be designed to transfer additional shear. Designers shall account for impacts due to openings in walls. Designers shall also design for necessary hold-downs for shearwalls.

Design Note: As part of the submitted designs, ensure all shear walls are clearly identified. Also, ensure that hold-down devices are clearly identified, where required, in the sealed design drawings.

Posts

The function of the wood post is to carry axial and bending loads to the foundation.
Posts are typically embedded in the ground or attached to concrete piles. Any portion of a post that is embedded or exposed to weather must be pressure-treated with preservative chemicals to resist decay and insect damage.

Design Note: the submitted sealed drawings shall clearly identify/confirm the following (see the Appendix for further discussion on each topic):

- **Ke factor (effective length factor):** Use a minimum of 2.2 unless otherwise justified by engineering rationale.
- **P-Δ effects:** Show that these have been accounted for.
- **Truss to Post Connection Details:** Must be included on drawings.
- **Column construction/splice details:** Must be shown on drawings.
- **Wet service condition factor (Ksc):** Confirm its use for embedded posts.

Foundation and Soil Conditions

The design of foundations shall be based on a subsurface investigation (geotechnical report) conducted in accordance with Section 4.2 of National Building Code (NBC) Div. B. However, presumptive load-bearing values from **Table 1806.2** of the International Building Code (IBC) will be accepted, provided that the engineer's assumptions are justified and the engineer has sufficient local experience, as per NBC Clause 4.2.4.1.(1)(b). This table has also been provided here for easy reference:

REGINA

TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERIAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystaline bedrock	12,000	1,200	0.70	_
2. Sedimentary and foliated rock	4,000	400	0.35	_
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	_
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM, and GC)	2,000	150	0.25	_
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH, and CH)	1,500	100	_	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m

a. Coefficient to be multiplied by the dead load.

b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2

Design Note: If using Table 1806.2 from IBC for pole-building designs in Regina, Class 5 shall be used. If higher values are desired, a geotechnical investigation report shall be provided to validate. The drawings shall clearly identify the geotechnical parameters used, cite the source (e.g., IBC Table 1806.2 or a geotechnical report), and confirm that total foundation settlement does not exceed 25 mm.

Fire-resistance rating (UL Design No. V304)

Finding fire-rated wall assemblies for post-frame buildings can be a challenge, due to its unique construction. In 2015, the National Frame Building Association successfully designed and tested a load-bearing fire separation for post-frame construction that can achieve fire-resistance ratings of 1, 2 or 3 hours (UL Design No. BXUV.V304). This design uses wood columns, wood framing and multiple layers of 5/8" thick fire-rated gypsum board applied to each side of the wall face. This design permits the columns to be embedded in the ground or surface-mounted on masonry or concrete.

For exterior walls, the 5/8-inch thick fire-rated gypsum board is applied only to the interior face of the wall.

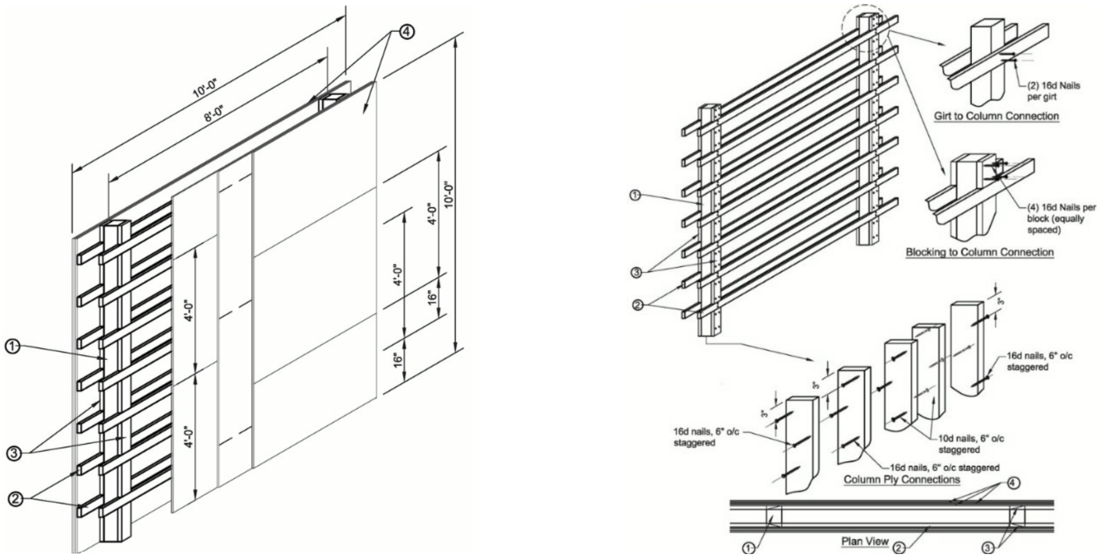

Figure 3: UL Design No. BXUV.V304 Installation

Roof Trusses

The designer should consider the following general topics for truss designs (see Appendix for more details on these topics):

- Chemical treatment impacts
- Wet Service Conditions:
  - Wet service conditions refer to environments where materials are exposed to moisture during service, such as in salt/sand storage facilities.
  - Note that G90 galvanization is omitted from consideration because it may not be suitable for use in wet service.
- Long-span trusses (clear spans larger than 24.4m (80'-0") and 30.5m (100'-0")) shall be designed for dry service conditions only.

<u>Design Note:</u> the sealed structural drawings for the building shall clearly identify the criteria to be followed by the truss designer.

Conclusion

This advisory serves to streamline the review process and promote safe, code-compliant post-frame construction. Designers are encouraged to refer to the Appendix and cited references to ensure a complete and accurate submission.

References

- Post Frame Building Design Manual (NFBA)
- Light-gauge Steel Cladding Diaphragm Tables (CSSBI-B18-16)
- International Building Code (IBC 2024)
- Truss Plate Institute of Canada (TPIC) Technical Bulletins
- Basic Fire Wall Provisions for Post-Frame Construction by Ronald L. Sutton
- UL Product iQ.

For more information on Building Permits, Building Safety or Zoning Information, please visit Regina.ca or contact Service Regina.

Post Frame (Pole Barn) Construction Appendix

Shearwalls

A significant portion of the shear forces in roof and ceiling diaphragms is transferred to the building foundation through shearwalls. In many post-frame buildings, only the exterior walls (end walls and side walls) are available for this purpose. However, where present, interior partition walls can be designed to transfer additional shear. Designers shall account for impacts due to openings in walls, and shall also design for necessary hold-downs for shearwalls.

1. End walls

End walls in post-frame buildings resist wind loads acting on them and also transmit roof shears (due to wind components parallel to the end wall) to the ground. In the diaphragm-shearwall design procedure, maximum roof shears occur at the end walls. The roof shear is transferred into the top truss chord or rafter of the end wall, through the end wall to the ground level, and finally to the ground via posts or posts connected to a concrete slab. In addition to shear forces, the end wall is also subject to overturning forces.

2. Wall Openings

Allowances must be made for openings in shearwalls. A common practice in post-frame construction is to place large doorways in the building end walls. These openings impact the design of shearwall segments and reduce their shear resistance.

3. Partitioning

Partitioning the building into structural segments is one method to reduce maximum roof and end wall shears. For example, if reinforcing an end wall with a large door is impractical, an alternative is to install a structural partition in the center of the building. This partition must meet the shear requirements imposed by the roof diaphragm. Buttresses, either inside or outside the walls, can also be used to reduce the effective length of the building, thereby decreasing maximum roof and end wall shears.

4. Overturning restraint

Design dead loads that exceed the weights of construction materials and permanent fixtures are generally permitted, except when checking building stability under wind loading. While using inflated design dead loads may result in conservative designs for gravity load conditions, it is not a conservative assumption when designing anchorage to counteract uplift, overturning and sliding due to wind loads. For wind uplift and overturning, the dead load used in design must not exceed the actual dead load of the construction. Therefore, it is important to properly design shearwall segments with hold-down devices if required.

Posts

The function of the wood post is to carry axial and bending loads to the foundation.
Posts are typically embedded in the ground or attached to concrete piles. Any portion of a post that is embedded or exposed to weather must be pressure-treated with preservative chemicals to resist decay and insect damage.

1. Design of Posts

Historically, some designers calculated the maximum post moment for embedded posts using the simple structural analogy of a propped cantilever (i.e. a fixed reaction at the post bottom and a pin reaction at the top). This analogy implicitly assumes that the roof diaphragm and shearwalls are infinitely stiff. While this model may be adequate for buildings with extremely stiff roof diaphragms and can provide conservative estimates of shear forces in the roof diaphragm, it may underestimate the maximum post moment in many post-frame buildings.

If posts are embedded, two bending moments generally need to be calculated: one at the groundline and another above ground.

For surface-attached posts, the bottom reaction can be modelled as a pin, and typically, only one bending moment is considered.

When a post is embedded in the soil, calculated post stiffness (and consequently, the calculated frame stiffness) is highly dependent on how the embedded portion is modelled. Traditionally, engineers have ignored soil properties and have modelled embedded posts using the analogs shown in **Figures 3(a)** and **3(b)**. A key deficiency of these analogs is that the pin supports used to fix the post below grade do not allow the post to displace naturally. For this reason, post-stiffness values predicted using the analogs should be applied with caution. Additionally, the analogs in **Figures 3(a)** and **3(b)** suggest a reduction in post stiffness as the embedment depth, d, increases (due to the length of the member increasing, and thus increasing the slenderness of the member). In reality, however, deeper embedment increases the stiffness of the post. For these reasons, **Figures 3(a)** and **3(b)** are not the most representative models of embedded posts, while **Figure 3(c**) is a much better model, as it accounts for soil and post interactions.

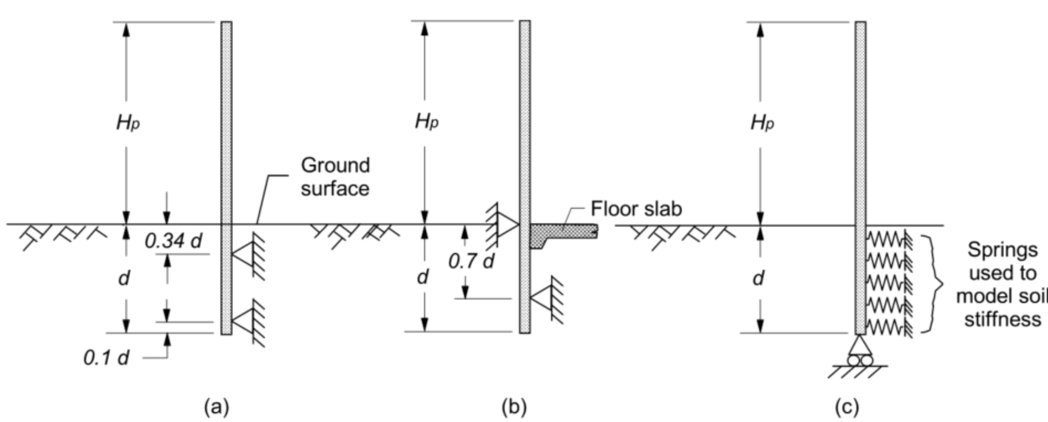

Figure 3: Structural analog traditionally used for (a) non-constrained and (b) constrained posts. (c) A more realistic non-constrained post analog that accounts for soil stiffness.

The effective length factor, Ke, shall be determined based on the degree of end restraint of the compression member. Lower values will be associated with more end fixity and less lateral translation while higher values will be associated with less end fixity and more lateral translation. The embedded post at the bottom is considered effectively held in position but only partially restrained against rotation, while the top of the post is considered partially held in position but not restrained against rotation. **In most cases, a Ke factor of at least 2.2 shall be used for embedded posts, as determined from CSA O86, unless an engineer justifies a lower value.**

2. Combined Stress Analysis

Forces involved in post design subject the posts to combined bending and compressive or tensile axial loads, and so shall be designed to satisfy the following appropriate interaction equation as per 6.5.9 of CSA O86. Additionally, it must consider the P-Δ effects for side-sway scenarios, if applicable.

3. Connections

The truss-to-post connection must be designed to resist both bearing and uplift forces. This connection should be modelled as a pin unless its moment-carrying capacity can be justified. Direct end-grain bearing is preferable and is often achieved by notching the post to receive the truss. When designing the truss-to-post connection for uplift, it is important to accurately estimate the weights of construction materials if any counteracting credit is to be taken.

For surface-attached posts, the bottom connection must be checked for maximum shear and uplift forces. For embedded posts attached to collars or footings, the connections must be properly designed to withstand gravity and uplift loads, and corrosion-resistant fasteners must be used.

4. Spliced Posts

Design provisions for spliced built-up compressive members were included in CSA O86-01 but removed in CSA O86-09, as they were deemed poor practice by the CSA O86 technical committee. Research indicates that non-reinforced butt-spliced columns have approximately 40% of the strength of non-spliced columns.

5. Wet Service Condition for Embedded Posts

Soil contact exposes wood members to significant moisture, particularly when the end grain is embedded in the soil. As a result, the service condition for such poles, piles, or columns is generally considered “wet”. Therefore, the service-condition factor, Ksc, should be selected for wet-service conditions from Table 6.10 of CSA O86 in the case of embedded wood posts. Additionally, they require a high level of preservative treatment in accordance with CSA O80.

Foundation

Post-frame building foundations include posts embedded in the ground or surface attached to a concrete foundation. Posts shall be designed to resist side-sway and overturning forces due to wind or seismic loads, as well as wind uplift, and gravity loads. Post-foundation design is an important aspect of post-frame building design that is not well-known in the structural engineering design community, and therefore the following detail below is dedicated to this subject.

1. Post Foundation Classification

Based on their depth, post foundations are classified as shallow foundations, which behave differently from deeper systems such as pilings. Specifically, post deformation below grade is relatively insignificant compared to the deformation of the surrounding soil. Soil deformation around a post is a three-dimensional phenomenon.

Figure 4 illustrates lines of constant soil pressure (in a horizontal soil plane) that develop when a post moves laterally. The greater the distance between two posts, the less influence one post has on the soil pressure near the other. For design purposes, individual embedded posts are considered isolated foundations when post spacing is six times the post width.

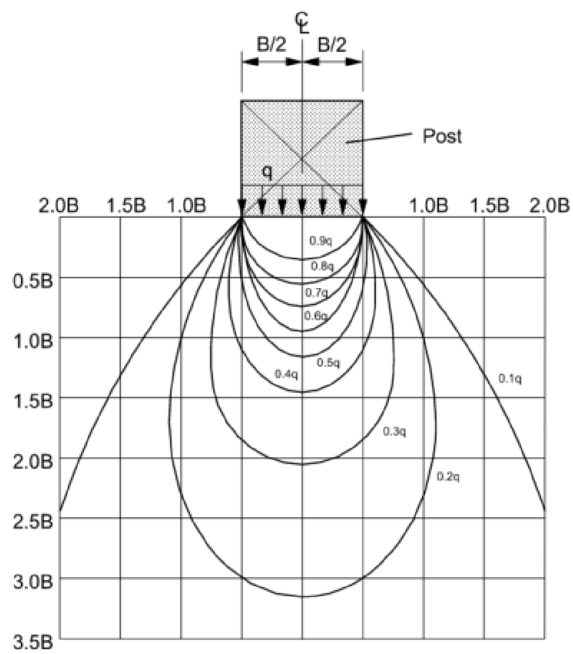

Figure 4: Constant Pressure Lines in a Horizontal Plane of Soil

2. Design Variables

Factors influencing the strength and stiffness of a post foundation include post constraint, soil properties, footing size, collar size, lateral loading, embedment depth, uplift loading, and frost heave considerations. Each of these subjects is discussed below.

a. Post Constraint

(1) Non-constrained Post:

The most basic type of post foundation consists of a post simply embedded in the ground, with no attachments or additional support (**Figure 5**). If the rotation and lateral displacement of the post are resisted solely by the soil, the post foundation is considered non-constrained.

(2) Constrained Post:

If a post bears on or is attached to an additional "immovable" structural element such that the lateral displacement at some point at or above the ground surface is essentially zero, the post foundation is considered constrained. An example of a constrained post foundation would be when the post is installed immediately adjacent to a concrete slab floor in the building (**Figure 6**).

(3) Varying Constraint:

It is important to note that a single post can be both constrained and non-constrained, depending on the load case. Using the previous example of a slab floor, and assuming that the post is not attached to the slab: if the wind loading causes the post to push against the slab, the post would be considered constrained. However, if the wind were blowing in the opposite direction, the post would not be supported by the slab, and thus, it would be analyzed as non-constrained for that load case.

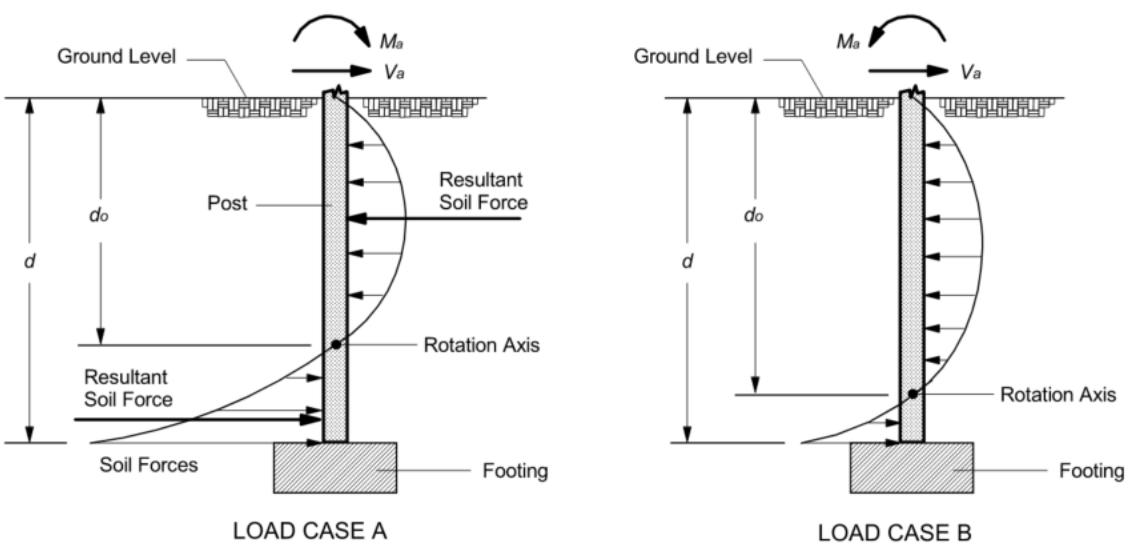

Figure 5. Free body diagrams of non-constrained post foundations.

- Load Case A: Groundline shear and moment both cause clockwise rotation of the embedded portion of the post.

- Load Case B: Groundline shear causes clockwise rotation, while the moment causes counterclockwise rotation of the embedded portion of the post.

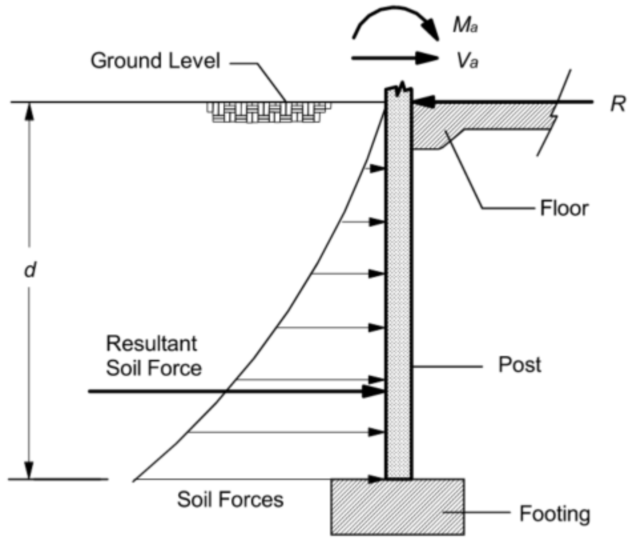

Figure 6. Free body diagram of a constrained post foundation.

b. Soil Properties

The ability of soil to handle loads transmitted by a post depends on such characteristics as particle size, size distribution, moisture content, density, and depth below grade. These soil characteristics control the allowable vertical and lateral soil pressures.

(1) Soil Moisture Content:
The effective shear strength of soil can be significantly reduced when it becomes saturated with water. To prevent soil saturation around posts, install rain gutters, and slope the finished grade away from the building. A minimum slope of 2% for a distance of at least 6 feet (2 m) from the building walls is recommended.

(2) Soil Density and Depth:
Allowable vertical and lateral soil pressures increase with soil density and depth. This is because soil confinement pressures rise as both of these factors increase.

(3) Presumptive Load-Bearing Values:
The design of foundations shall be based on a subsurface investigation (geotechnical report) conducted in accordance with Section 4.2 of National Building Code (NBC) Div. B. However, presumptive load-bearing values from **Table 1806.2** of the International Building Code (IBC) will be accepted, provided that the engineer's assumptions are justified and the engineer has sufficient local experience, as per NBC Clause 4.2.4.1.(1)(b). This table has also been provided below, for easy reference.

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3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.

b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

c. Footings

Typically, soil alone cannot adequately resist applied vertical loads when those loads are transferred solely through the post. Therefore, a post is set on a footing, which is placed in the hole before post installation. In post-frame construction, footings are usually poured concrete as depicted in **Figure 7**.

(1) Friction:
A footing is assumed to resist only vertical loads. The friction between the footing and the post is considered negligible when evaluating the post lateral load resistance. Similarly, the friction between the post (and/or collar) and the surrounding soil is assumed to be negligible when assessing the vertical load-carrying capacity of a given post foundation design.

d. Collars

When lateral soil pressures exceed allowable values, additional lateral surface area can be obtained by increasing post depth, or by adding a structural element called a collar. A collar is typically either concrete cast around the base of the post (and considered to be attached to the post) or built-up wood attached to the post. These structural elements are represented in **Figure 7**.

(1) Location:
The collar increases the lateral load resistance capability of the post foundation by increasing the bearing area in the region of the post where lateral soil capability is relatively high. Collars are typically not placed at the top of the post foundation (at the surface of the ground) due to the possibility of frost heave.

(2) Attachment:
Whether poured concrete or wood, the collar must be attached to the post in a manner sufficient to carry the structural loads involved. As with any wood structural element exposed directly to the soil, appropriate preservatives and fastener systems must be employed to maintain structural integrity over the design life of the building.

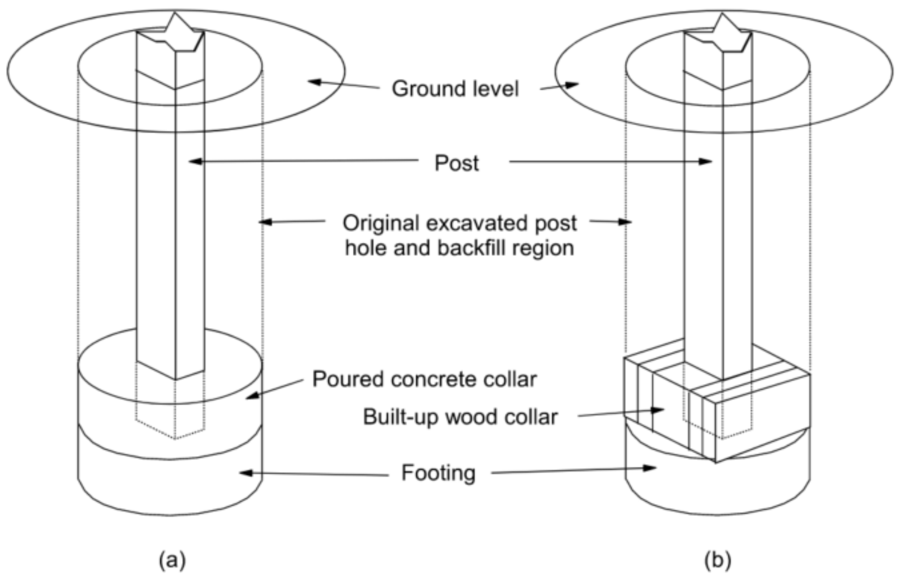


Figure 7. Examples of common post foundation elements with (a) a poured concrete collar, and (b) a built-up wood collar.

e. Lateral Loading

Bending moments and post shears cause lateral movement in the post foundation. Designers must ensure that this movement does not generate soil stresses that exceed allowable lateral soil pressures. If the allowable lateral soil pressure is exceeded, the designer must increase the lateral soil bearing area by adding a collar, increasing the embedment depth, d, and/or increasing the effective post width, b.

In most cases, the most economical way to increase bearing area is by increasing the post depth. For this reason, embedment depth, d, is typically the dependent variable in design equations. Occasionally a designer may add an extra laminate to the embedded portion of a laminated post to increase its effective width. More commonly, designers backfill all or a part of the hole with concrete, effectively creating a concrete collar.

f. Embedment Depth (1807.3.2 IBC)

(1) Non-Constrained:
The following formula shall be used to determine the required depth of embedment, d, for resisting lateral loads when:

- No lateral constraint is provided at the ground surface (e.g., no rigid floor or pavement)
- No lateral constraint is provided above the ground surface (e.g., no structural diaphragm).

$$d = 0.5A\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right)$$

Where:
A = (2.34xP)/(S1 x b).
b = Diameter of a round post/footing or the diagonal dimension of a square post/footing (ft or m).
d = Depth of embedment in earth in (ft or m), not exceeding 12 ft (3658 mm) for lateral pressure computations.
h = Distance from ground surface to the point of application of P (ft or m).
P = Applied lateral force (lbs or kN).
S1 = Allowable lateral soil-bearing pressure per Table 1806.2 of IBC, based on a depth of one-third of embedment depth (psf or kPa).

(2) Constrained:
The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or slab-on-ground.

$$d = \sqrt{\frac{4.25Ph}{S_3 b}}$$

Or alternatively, using the moment at grade:

$$d = \sqrt{\frac{4.25M_g}{S_3 b}}$$

Where:
- Mg = Moment in the post at grade, in foot-pounds (kN-m).
- S3 = Allowable lateral soil-bearing pressure as set in Table 1806.2 of IBC based on a depth equal to the depth of embedment in pounds per square foot (kPa).

g. Uplift Loading
If the net vertical force acting on a post is upward, the footing or a collar must be securely attached to the post. When the footing or a collar is attached, the post foundation cannot move upward without displacing a cone-shaped mass of soil.

The size of this displaced soil mass depends on several factors, including:
- Foundation depth
- Footing or collar size
- Soil density
- Soil internal friction angle

(1) Skin Friction:
An attached footing or collar is required to resist uplift forces, as skin friction between a post and backfill alone cannot be relied upon for uplift resistance.

(2) Concrete Backfill:
Concrete cast against undisturbed soil and mechanically fastened to the post enhances uplift resistance by contributing both the weight of concrete mass and the skin friction between the concrete and soil. However, this method is not recommended in soils highly susceptible to frost heave.

(3) Volume of Displaced Soil:
The volume of soil that must be displaced when a footing or collar is pushed upward depends on the shape of the foundation element. Figures 8 and 9 illustrate configurations for circular and rectangular foundation elements, respectively.

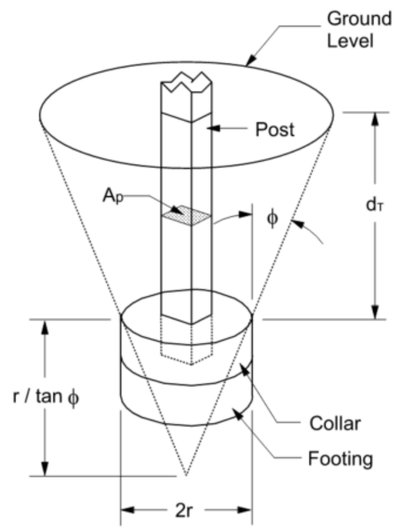


Figure 8. Schematic of relevant uplift resistance components for post foundation with an attached circular collar.

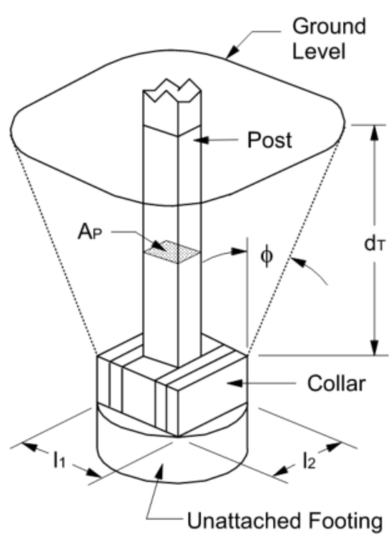


Figure 9. Schematic of relevant uplift resistance components for post foundation with an attached rectangular collar.

h. Frost Heave Considerations

Freezing temperatures in the soil lead to formation of ice lenses in the spaces between soil particles. Under the right conditions, these ice lenses continue to attract water and grow, causing an increase in soil volume. If this expansion occurs beneath a footing or alongside a foundation element with a rough surface, the foundation may be forced upward due to frost heave.

(1) Problems:

Frost heave can cause significant differential movement in a foundation, leading to:

- Cracking in building finishes.
- Increased stress in structural connections and components.
- Soil saturation upon thawing, reducing soil strength.
- Settlement of the foundation as excess water drains and effective soil stresses increases.

(2) Minimizing Frost Heave:

Frost heave can be mitigated through proper soil selection, adequate footing depth, and effective drainage.

1. Avoid Silts and Clays:

   Fine-grained soils, such as clays and silts, are highly susceptible to frost heave because water is drawn upward through fine pores acting as capillaries, and the high surface area of fine particles increases water adsorption.

2. Footing Depth:

   The most effective way to prevent frost heave is to place footing below the frost line, where freezing does not occur. In Regina, the frost line typically reaches a depth of around 2.0 m. Exceptions include footings placed on solid rock and floating foundations, which are reinforced to move as a monolithic unit as soil expands and contracts.

3. Water Drainage:

   Proper surface and subsurface drainage reduces frost heave risk. Drainage can be improved by installing rain gutters and sloping the finished grade away from the building, raising the building elevation above surrounding area, and using subsurface drainage system, such as coarse granular material below the maximum frost depth, to lower the water table and prevent water movement through the soil.

Roof Diaphragm and Ke Value for Post-Frame Structure

1. Rigid Diaphragm

A diaphragm is considered rigid when its midpoint displacement under lateral load is less than twice the average displacements at its ends. Rigid diaphragms distribute horizontal forces to vertical resisting elements in proportion to their relative rigidities.

Key characteristics:

- Assumes minimal deformation under load.
- Causes each vertical element to deflect equally.
- Capable of transferring torsional and shear forces.
- Includes reinforced concrete diaphragms, precast concrete diaphragms, and composite steel decks.

Rigid diaphragms undergo rigid body rotation, leading to additional shear forces in shear walls. Their design assumes that both the diaphragm and shear walls rotate together under lateral forces.

2. Flexible Diaphragm

A diaphragm is considered flexible when the maximum lateral deformation of the diaphragm is more than twice the average story drift of the associated story. This is determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the drift of adjoining vertical elements.

Key characteristics:

- Deflects significantly under lateral load.
- Distributes lateral forces based on tributary width rather than relative stiffness.
- Incapable of redistributing torsional and rotational forces.
- Includes diaphragms sheathed with plywood, wood decking, or metal decks without structural concrete topping slabs.

The classification of diaphragms as rigid or flexible is based on their relative flexibility, influencing load distribution and lateral force resistance.

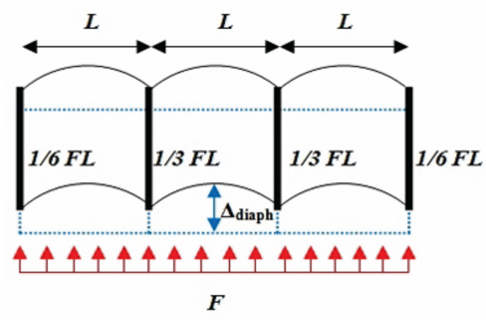
3. Determining Flexible vs. Rigid Diaphragm Behavior

The purpose of classifying diaphragms as flexible or rigid is to determine whether loads should be distributed based on tributary area (for flexible diaphragms) or relative stiffness (for rigid diaphragms).

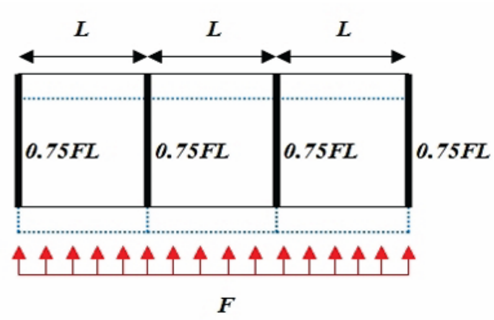
- **Flexible diaphragms** distribute seismic forces according to tributary area and simple beam analysis.
- **Rigid diaphragms** exhibit rotational or torsional behavior, leading to redistribution of shear forces to vertical force-resisting elements.

When evaluating diaphragm behavior:

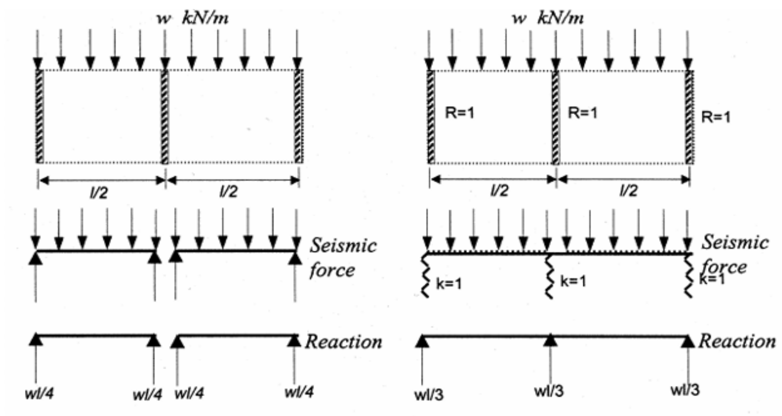
- If the mid-span deflection of the diaphragm exceeds twice the shear wall drift, it is considered **flexible**.
- If the diaphragm deflection is equal to or less than twice the shear wall drift, it is considered **rigid**.



Flexible Diaphragm



Rigid Diaphragm



Comparison between flexible and rigid diaphragm

American Society of Civil Engineers (ASCE) Standard stated:
For a more explicit criterion, the ASCE 7 standard, which is widely referenced in North America, provides guidance on this matter. According to ASCE 7 Section 12.3.1.3., a diaphragm can be considered flexible if its maximum in-plane deflection under seismic (or wind) loading is greater than two times the average storey drift of the adjoining vertical-resisting elements (e.g., shear walls). This means:

$$\boldsymbol{\delta} \text{ diaphragm} > 2 \times \boldsymbol{\Delta} \text{ shearwall}$$

Where:

- **δ** diaphragm = Maximum deflection of the diaphragm
- **Δ** shearwall = Average story drift of the adjacent shear walls

This definition is widely used in structural engineering to differentiate between **rigid** and **flexible diaphragms**, impacting how lateral forces are distributed to the vertical elements.

4. Sway-sensitive or Non-Sway?

CSA O86 doesn't specify a formula for determining whether the structure is sway-sensitive or non-sway; therefore, use Sentence 10.11.4 from ACI (American Concrete Institute) Building code as shown below.

10.11.4 — Columns and stories in structures shall be designated as non-sway or sway columns or stories. The design of columns in non-sway frames or stories shall be based on 10.12. The design of columns in sway frames or stories shall be based on 10.13.

10.11.4.1 — It shall be permitted to assume a column in a structure is non-sway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

10.11.4.2 — It also shall be permitted to assume a story within a structure is non-sway if:

$$Q = \frac{\Sigma P_u \Delta_o}{V_u \ell_c} \qquad (10\text{-}7)$$

is less than or equal to 0.05, where $\Sigma \boldsymbol{P_u}$ and $\boldsymbol{V_u}$ are the total vertical load and the story shear, respectively, in the story in question and Δ_o is the first-order relative deflection between the top and bottom of that story due to $\boldsymbol{V_u}$.

R10.11.4 — The moment magnifier design method requires the designer to distinguish between non-sway frames which are designed according to 10.12 and sway frames which are designed according to 10.13. Frequently this can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member may be assumed braced by inspection if it is located in a story in which the bracing elements (shearwalls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent by inspection, 10.11.4.1 and 10.11.4.2 give two possible ways of doing this. In 10.11.4.1, a story in a frame is said to be non-sway if the increase in the lateral load moments resulting from $\boldsymbol{P}\Delta$ effects does not exceed 5 percent of the first-order moments.[10.25] Section 10.11.4.2 gives an alternative method of determining this based on the stability index for a story $\boldsymbol{Q}$. In computing $\boldsymbol{Q}$, $\Sigma \boldsymbol{P_u}$ should correspond to the lateral loading case for which $\Sigma \boldsymbol{P_u}$ is greatest. It should be noted that a frame may contain both non-sway and sway stories. This test would not be suitable if $\boldsymbol{V_u}$ were zero.

Alternative Check for Sway Frames in Wood Design

1. **Compute Q using the concrete formula**:

$$Q = \frac{P_u \times \Delta}{V_u \times L_c}$$

   - If **Q > 0.05** → Frame is **sway-sensitive**.
   - If **Q ≤ 0.05** → Frame is **non-sway**.

Here are some general conditions of sway structure:

- Eccentric Loading (Unsymmetrical Loading):
  When loads are not applied directly at the center of the structure's members, they create bending moments that can cause lateral movement.

- Asymmetrical Loading:
  Uneven distribution of loads across the structure can lead to lateral displacement as different parts of the structure experience different forces.

- Different End Conditions of Columns:
  If columns have varying support conditions (e.g., fixed versus pinned), the structure may be more susceptible to sway.

- Non-Uniform Sections of Members:
  If columns or beams have different cross-sectional properties (e.g., different sizes or materials), they can deform differently under load, leading to lateral movement.

- Settlement of Supports:

  Uneven settlement of the foundation or supports can cause the structure to lean or sway.

1. Effective Length Factor, Ke

Table A.4 (Concluded)

Degree of end restraint of compression member	Effective-length factor, K_e	Symbol
Effectively held in position at both ends but not restrained against rotation	1.00	
Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position	1.20	
Effectively held in position and restrained against rotation at one end, and at the other partially restrained against rotation but not held in position	1.50	
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position	2.00	
Effectively held in position and restrained against rotation at one end but not held in position or restrained against rotation at the other end	2.00	

Note: *Effective length $L_e = K_eL$, where L is the distance between centres of lateral supports of the compression member in the plane in which buckling is being considered. At a base or cap detail, the distance shall be measured from the outer surface of the base or cap plate. The effective-length factor, K_e, shall not be less than what would be indicated by rational analysis. Where conditions of end restraint cannot be evaluated closely, a conservative value for K_e shall be used.*

1. **Partially Fixed at the Bottom**: When the bottom is partially fixed (moment resistance but still allowing some rotation and lateral movement), the column has some degree of restraint, but it is not fully rigid. This allows a **small rotation** at the groundline and some lateral movement. As a result, the column behaves like a **more flexible column** than a fully **fixed-bottom column**, increasing its **effective length**.
2. **Pinned at the Top**: The top being pinned with allowed lateral movement (side sway) does not provide restraint, so the column is free to sway at the top.
3. **Inflection Point**: The **inflection point** (the point where the bending moment is zero) in this case will indeed be **lower than the groundline** because the column's moment at the bottom is partially resisted but not fully fixed. The inflection point shifts downward due to the reduced resistance at the bottom, and this results in a **longer effective length** for the column compared to one that is fully fixed at the bottom.

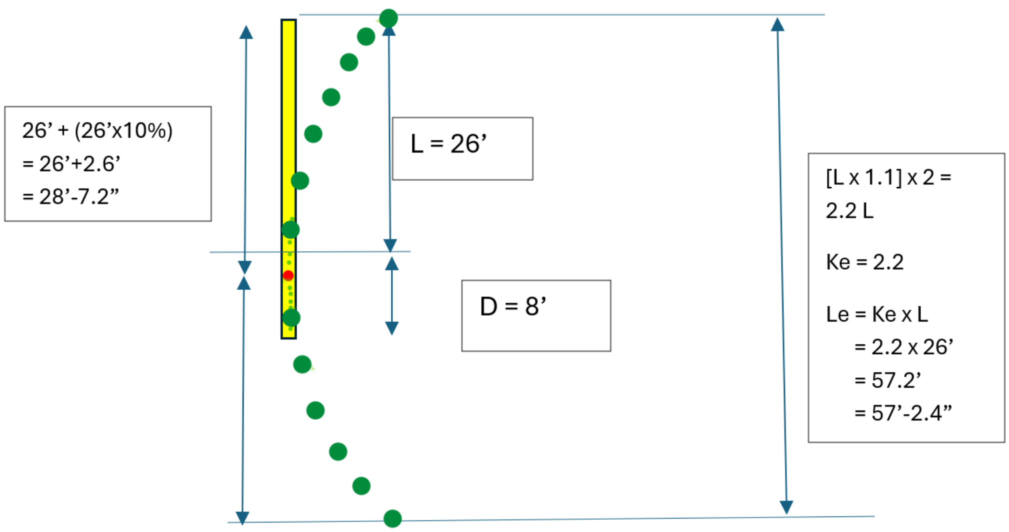
13.5.3. of CSA-O86 stated:

13.5.3 Effective length
When the finished pile projects above ground level and is not secured against buckling by bracing, the effective length shall be governed by the fixity conditions imposed on it by the structure it supports and by the nature of the ground into which it is driven. In firm soil, the lower point of contraflexure may be taken to be at a depth below ground level of about one-tenth of the exposed length. Where the top stratum is soft clay or silt, this point may be taken at about one-half the depth of penetration into this stratum, but not less than one-tenth of the exposed length of the pile. Where a pile is wholly embedded, its carrying capacity is not limited by its strength as a long column. However, where there is a stratum of very soft soils or peat, piles shall be designed in accordance with Clause 13.5.5.

**Contraflexure (inflection) refers to the point where the moment transitions from positive to negative or vice versa, and thus, the moment is zero at that point.

Compression resistance is reduced when a pile is not restrained against lateral buckling. The factored compressive resistance, *Pr*, of that portion of the pile in contact with air, water, or soft mud—including the portion which may be exposed as a result of scouring action—must be calculated considering the slenderness of the pile.

Figure 1: In firm soil: one-tenth of the exposed length



Unless justified otherwise, a Ke value of at least 2.2 shall be used for embedded posts as per CSA O86.

Comments from CWC

The first step in determining the compressive resistance of the unembedded portion is to determine the effective length of the member. The effective length depends on the type of restraint at the pile head and the soil conditions at the soil surface. In firm soil, the lower point of contraflexure may be taken at a depth below ground level of about one-tenth of the exposed length. Where the uppermost stratum is soft clay or silt, this point may be taken at about one-half of the depth of penetration into this stratum but not less than one-tenth of the exposed length of the pile (see Figure 2.5). Effective length is the product of the buckling length of the pile and the effective length factors, which depend on end restraint conditions (see Figure 2.6).

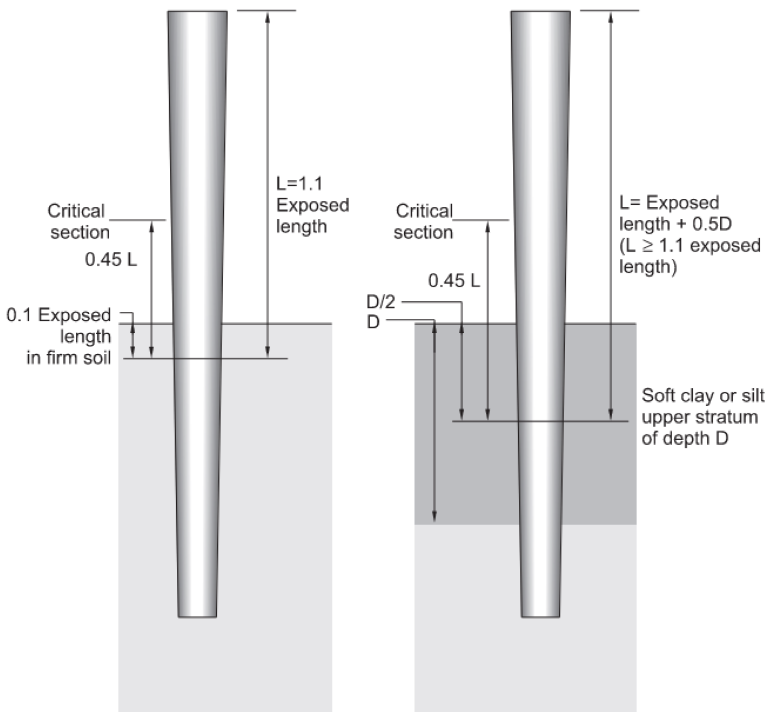


Figure 2.5
Method of determining the lower point of contraflexure in a wood pile

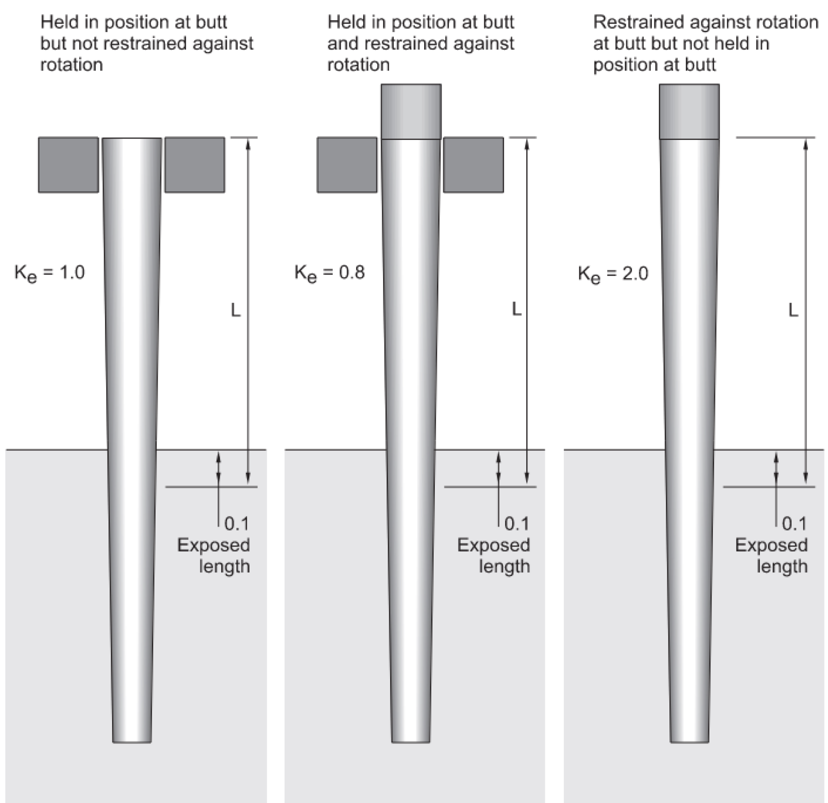


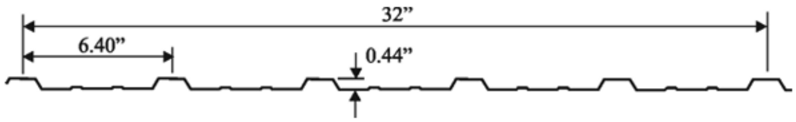
Figure 2.6
Effective length factor in a wood pile

References: CWC, ASCE 7, CSA-O86, ACI Building Code

Light-gauge Steel Cladding Diaphragm Tables (CSSBI-B18-16)

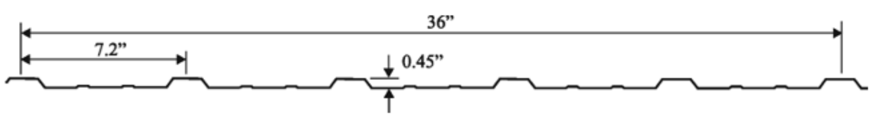
A design method is available to guide the selection of the sheet steel cladding and fasteners needed to create a lightweight steel cladding diaphragm. This design method is included in a publication of the Metal Construction Association titled “A Primer on Diaphragm Design”, First Edition 2004. Examples of cladding profiles and section properties are provided in the Appendix of this advisory. It is important to note that the cladding profiles shown are only representative of products available in Canada; products from specific manufacturers may differ. The manufacturer should be contacted for the design values applicable to their products. The factored shear resistance values provided in the tables incorporate the Limited States Design (LSD) resistance factors based on screwed connections and wind loading from Table D5 of CSA-S136-12 North American Specification for the Design of Cold-Formed Steel Structural Members.

Cladding Profiles and Section Properties



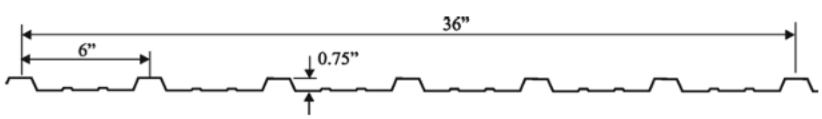
Profile: 7/16″ x 6.4″ x 32″

Design Thickness (in.)	Yield Stress (ksi)	Moment of Inertia (in^4/ft)
0.0180	33	0.0072
0.0277	33	0.0095
0.0300	33	0.0119



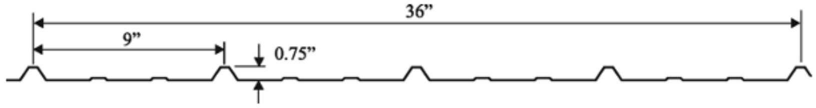
Profile: 7/16″ x 7.2″ x 36″

Design Thickness (in.)	Yield Stress (ksi)	Moment of Inertia (in^4/ft)
0.0120	33	0.0041
0.0150	33	0.0057
0.0180	33	0.0075



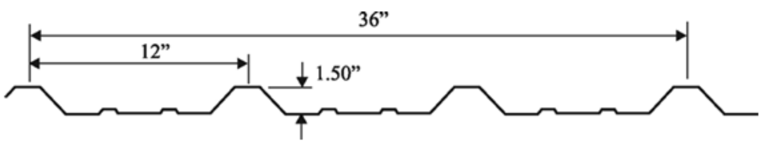
Profile: 3/4″ x 6″ x 36″

Design Thickness (in.)	Yield Stress (ksi)	Moment of Inertia (in^4/ft)
0.0135	80	0.0152
0.0180	33	0.0227
0.0240	33	0.0302



Profile: 3/4″ x 9″ x 36″

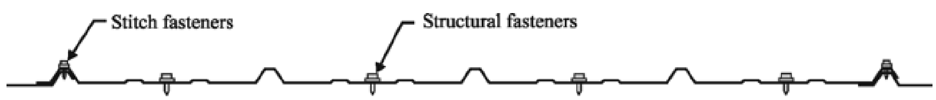
Design Thickness (in.)	Yield Stress (ksi)	Moment of Inertia (in^4/ft)
0.0135	80	0.0083
0.0180	33	0.0110
0.0240	33	0.0146



Profile: 1-1/2″ x 12″ x 36″

Design Thickness (in.)	Yield Stress (ksi)	Moment of Inertia (in^4/ft)
0.0135	80	0.0455
0.0180	33	0.0673
0.0240	33	0.0938

Fastener Patterns



Fasteners in the Flats

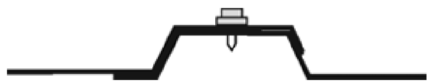
Strength and Flexibility of Frame Fasteners through Flats into Wood

t (in.)	F_y (ksi)	Steel Q_{fs} (kips)	Wood Q_{fw} (kips)	S_f (in/kip)
0.0120	33	0.271	0.886	0.0137
0.0135	80	0.492	0.886	0.0129
0.0150	33	0.378	0.886	0.0122
0.0180	33	0.497	0.886	0.0112
0.0240	33	0.765	0.886	0.00968
0.0277	33	0.949	0.886	0.00901
0.0300	33	1.033	0.886	0.00866

Fasteners in the Ribs

Strength and Flexibility of Frame Fasterns Through Ribs into Wood

t (in.)	Q_f (kips)	S_f (in/kip)
0.0120	0.263	0.0274
0.0135	0.314	0.0258
0.0150	0.368	0.0244
0.0180	0.483	0.0224
0.0240	0.744	0.0194
0.0277	0.923	0.0180
0.0300	1.005	0.0173



Strength and Flexibility of Side-Lap Stitch Fasteners

t (in)	F_y (ksi)	#12 screws Q_s (kips)	S_s (in/kip)
0.0120	33	0.195	0.0274
0.0135	80	0.233	0.0258
0.0150	33	0.273	0.0244
0.0180	33	0.358	0.0224
0.0240	33	0.552	0.0194
0.0277	33	0.684	0.0180
0.0300	33	0.745	0.0173

Note: Tables of Diaphragm Design Strength and Stiffness can be found at the following link: CSSBI-B18-16 Design Manual.

Roof Trusses

The following information is being provided, as they can impact the design criteria for roof trusses, especially for typical uses for pole barn buildings.

TPIC Technical Bulletin #4

1. **Strength Reducing Chemicals:**

These include organic acids, inorganic acids, oxidizing/reducing agents, alkaline solutions, or any chemical that has the net effect of degrading the strength of the connectors. While weak chemicals degrade connectors over long periods of time, stronger chemicals can quickly consume the connectors and lead to catastrophic failure. Moreover, the interaction between the reactants can be very complex; a protective coating that works for some chemicals may not work for others. As such, whenever trusses are exposed to this type of chemicals, it is recommended that the appropriate professionals be consulted in advance.

2. **Wet Service Condition**

CSA O86 defines wet service condition as any service condition other than dry. Dry service is a climatic condition in which the average equilibrium moisture content of solid wood over a year is 15% or less and does not exceed 19% (CSA 2019). As shown in the graph below, in order to get above 19% moisture in the lumber, the relative humidity in the air needs to reach about 80% or higher, depending on wood species and temperature.
Unfortunately, 80% is above the critical humidity level for the electrochemical oxidation of steel, which is around 60% (Roberge 2000). Beyond 60%, the rate of corrosion increases due to an increase in available moisture to facilitate the corrosion process. The service condition factors shall be applied to both the lumber and connector whenever there is wet service condition (CSA O86). Note that G90 galvanization is omitted from consideration because it may not be suitable for use in wet service.

3. **Salt Storage Facilities**

These are buildings used for the storage of bulk salt/sand mixture for roads/highways maintenance. It is reasonable to expect that the truss may be exposed to both the corrosive effect of salt and wet service condition even though the amount of moisture within the buildings may vary from time to time depending on the composition of the mixture and the design, ventilation performance, and geographic location of each building.

Guideline for Salt Storage Facilities	
Protection	**Comment**
G90-Duplex G185-Duplex Stainless Steel	Apply strength modification factors for wet service condition unless appropriate measures are taken to ensure dry service condition.

TPIC Technical Bulletin #10

Due to their higher-risk design, long span trusses should be looked at with special care. In fact, these trusses develop higher member and joint forces, higher bearing reactions and higher bracing forces to be distributed in the building. In addition to the actual usage loading, these trusses are often deformed a lot during handling, causing unaccounted and unwanted stress in the joints and in the members. Therefore, the designer should use special consideration when designing this type of truss.

Long span trusses are trusses that have a clear span (between bearings) larger than 24.4m (80'-0"), but less than 30.5m (100'-0").

1. **Design consideration for long span trusses**
    - Service condition: Long span trusses shall be used **only** in dry service condition.
    - Lumber: The minimum web size for long span trusses shall not be less than 38x89 (2x4), and long span trusses shall be manufactured using lumber of 19% or less moisture content.
    - Plates: The maximum grip JSI of connector plates shall be limited to 0.80.
    - Heels: The height of the bottom chord of a truss at the interior side of bearing of a heel joint shall be at least 75% of chord depth.