



Roof Drainage *Not my problem...Maybe*

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Abstract

Many structural engineers are surprised to learn that the *International Building Code* (IBC, 2009) requires the roof structure to be engineered for standing water weight in the vicinity of the drains and scuppers regardless of roof slope. In addition, some low-slope roofs also require special attention for water weight and stiffness for safety against ponding failures or protection against accelerated roofing deterioration. With a mixture of overlapping design disciplines between the architect, plumbing consultant and structural engineer, proper roof drainage is often not fully addressed in building design and can lead to catastrophic collapse.

The author has been involved as an expert consultant in several roof collapses in California due to excessive rain water accumulation, and brings some lessons learned to the profession. This paper provides an overview of the various disciplines involved in transporting rainwater from roofs, and recommendations for engineers to comply with code requirements.

Introduction

The vast majority of commercial, retail, institutional, and multi-family residential buildings have low-sloped roof systems, providing an efficient use of building materials to enclose a specific volume of usable space. For aesthetic appeal, low-slope roofs, which are nearly dead flat, are typically surrounded with parapet walls to screen roof top equipment and to provide a constant visual elevation. However, parapets block the drainage of rainwater from freely flowing over the roof edge. In these cases the IBC requires a primary drainage system with a back-up secondary (emergency) drainage system in case the primary drain is blocked or excessive rainfall occurs.

A poorly functioning roof drainage system can affect structural safety and lead to a roof collapse. At water collection points, inadequate performance of roof drains and wall scuppers frequently cause excessive amounts of water to back up and lead to a partial roof collapses. While at first

glance this may appear to not involve the structural engineer, it is prudent for design engineers to recognize the responsibilities of the various design professionals involved in a project, and to not only properly address the related structural issues but to also be more aware of the non-structural issues that could still have the engineer named in a lawsuit.

Besides inadequate drains and scuppers, another roof drainage issue involving structural engineers is roofs having inadequate slope. For low-sloped roofs, the 2009 IBC states that ponding instability must be investigated if the roof slope is less than $\frac{1}{4}$ " drop per horizontal foot. Ponding instability is the progressive accumulation of rainwater and subsequent additional deflection of the roof structure, leading further to more water accumulation, and to an overload failure.

In addition to ponding instability, caution must be exercised in these low-sloped flat roofs to minimize pooling of water which can accelerate the deterioration of the roofing system. Manufacturer of roofing products have very specific terms in their warranties in regards to this.

Is There a Problem?

The expectation is that the roofing system will keep occupants dry and the building protected against adverse weather, but when these expectations are not met, costs, litigation, and life-safety concerns become a major concern. As reported by Patterson & Mehta (Patterson, 2010), roofing issues at one time or another have been

- #1 source of litigation in construction
- #1 source of litigation for architects
- #1 source of insurance losses
- #1 source of building maintenance cost

In addition to the costs and time involved to resolve these issues, catastrophic roof collapses (Figures 1 and 2) regularly occur every year due to rainfall, putting people's lives at risk in addition to the millions of dollars in property damage.



Figure 1: Excessive rainwater load causes a steel roof structure to collapse. (Source: Patterson, 2010)



Figure 2: Excessive rainwater load causes a wood roof structure to collapse. (Source: John Lawson)

Typically the causes of these roof collapses are not due to a structural deficiency but instead related to poor design of, poor execution of, and/or poor maintenance of the roof drainage system. The author has often observed undersized primary and secondary drainage system, missing or blocked overflow systems, and portions of roofs that are nearly dead flat causing ponding water. The problems are widespread enough that others have called for tighter regulations and an

education campaign to be put in place (Verhulst, 2010; Patterson, 2010; Jordan, 2005).

The National Roofing Contractors Association (NRCA) has concluded that ponding water can be detrimental to most roofing membranes leading to accelerated deterioration. NRCA's *Handbook of Accepted Roofing Knowledge* (NRCA, 1989) identifies the detrimental effects standing water can have on the roofing membrane assembly: deterioration of the roof's surface and membrane; debris accumulation, vegetation, fungal growth and resulting membrane damage; deck deflections possibly leading to structural problems; tensile splitting of water-weakened roofing felts; and voiding of manufacturers' warranties.

For most roofing membranes, manufacturer warranties exclude any damages where proper positive drainage is not provided, and these manufacturers routinely refer to the NRCA *Roofing Manual* for a definition of positive drainage (NRCA, 2011; Wilen, 2012). This manual states, "The criterion for judging proper slope for drainage is that there be no ponding water on the roof 48 hours after a rain during conditions conducive to drying." Some manufacturers have reduced this 48 hour time frame down to 24 hours for their warranties. In many cases building owners are surprised to discover that their roof deck assemblies fail to comply with the terms of the manufacturer's warranty for positive drainage, and coverage can be denied when a damage claim is made.

Whose Responsibility?

In 1998, a heavy rain passed over Orange County, California, dropping significant amounts of rain on the tops of warehouse and manufacturing buildings. In the middle of the night, a fire sprinkler alarm was triggered by the sudden flow of water up the 6" fire risers into the building's suspended fire sprinkler system piping. The nearest Orange County fire station responded to the alarm and was informed by a night employee that power was out and large amounts of water were entering the building from an unknown source. In the darkness outside, firefighters erected a ladder at an exterior wall and peered over the parapet at the north corner of the building finding deep pools of water still on top of the roof, blocked by leaves clogging the drain. Efforts to clear the drain with their pike poles were unsuccessful, but it was quickly discovered that a large amount of water was entering the building from a partial roof collapse at the southwestern side. This collapse ruptured the pressurized fire suppression piping, sounding the alarm, and spewing water into the large warehouse storing sensitive laptop computers and associate components.

It was an expensive claim for the tenant's insurance company, and resulted in costly litigation. Blame was directed at numerous parties.

Was the architect to blame? His design extended the concrete tilt-up panels above the roof line creating parapets that naturally obstructed the water flow. Instead of providing secondary (emergency) scupper penetrations in the parapets, the architect used a second roof drain pipe instead, which was much more prone to clogging despite its apparent code compliance (See Figures 3 and 4). It was determined that up to 18-inches of water depth was on the roof just prior to collapse.



Figure 3: Primary and secondary (overflow) drains with domed strainers are susceptible to clogging.
(Source: John Lawson)

Was the plumbing consultant to blame? His drainage pipe design placed protective domed strainers across the drain pipe entrances, effectively ensuring a clog will take place from the numerous medium sized leaves on the roof. Despite being code-complying strainers, it was argued that these certainly exacerbated the clogging.

Was the landscape architect to blame? His design placed large deciduous trees along the street at the upwind side of the building, which routinely deposited large quantities of leaves onto the roof in late autumn just prior to the rainy season.

Was the owner to blame? Regular care and maintenance of the roof drainage system is the responsibility of the building owner or tenant, including seeing that the roof is free of loose debris that could cause blockage to the roof drains.

Was the City to blame? During the preliminary development review process for the building, the City verbally indicated to the architect that no scuppers through the parapet walls would be tolerated where they would face the street for aesthetic reasons as a condition of approval. After the collapse, all neighboring buildings were observed to have only primary and secondary drains in lieu of street-facing scuppers, leaders or downspouts, accommodating the City's desires. Additionally, the City provided input as to the need to visually screen the building from the street with substantial tree vegetation, which ended up later providing the source of the leaves clogging the drains.

Was the contractor to blame? An inspection of the timber roof structure debris found that the collapse may have initiated at a suspicious knot in the 4x wood purlin. Was the wood beam defective or of an inferior lumber grade?

As you can see from the above discussion, there are a number of various theories that can lead to a difficult time assigning fault and settling this matter. But the structural engineer responsible for the roof structure design has not been mentioned here. In this case, the structural engineer likely observed that the roof slope was specified by the architect to be at least $\frac{1}{4}$ " per foot, justifying under the terms of the building code that no ponding analysis was necessary. And besides, what primarily led to the collapse was not associated with the structural design but instead a bundle of leaves clogging drains behind architectural parapets that would have been better served with flow-through scuppers. Never the less, an aggressive plaintiff attorney will seize upon any small amount of engineering fault in an effort to gather more money to settle the lawsuit. Despite not being the responsible party, the structural engineer needs to convince the other parties (or possibly a jury) that his roof structure design meets the building code requirements for rain load.

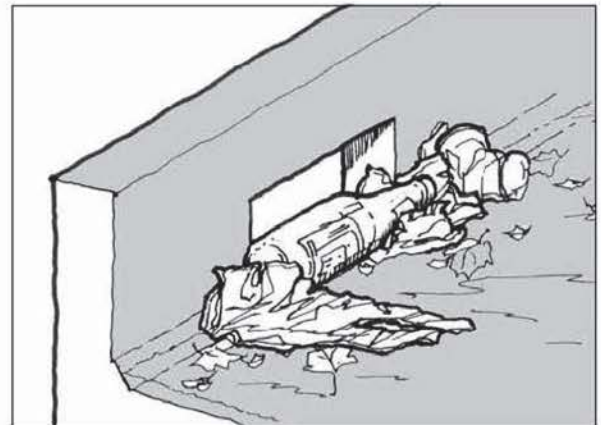
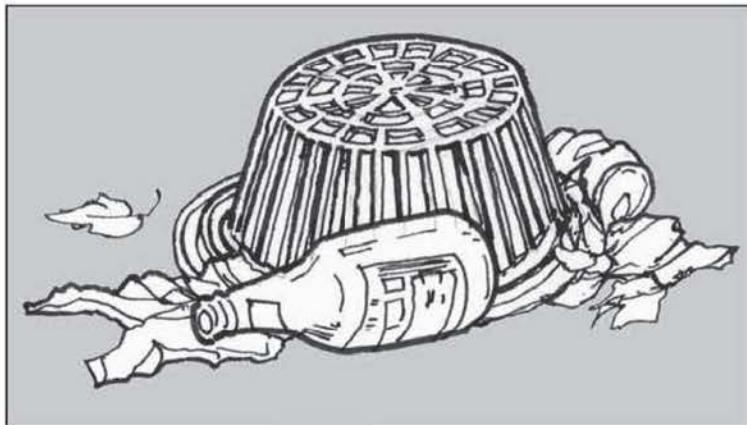


Figure 4: Drains with domed strainer (left) and scuppers through parapet walls (right) are susceptible to clogging. (Source: Patterson, 2010)

As rainwater flows down a roof slope to several collection points, water will naturally accumulate and rise to some height over the drain pipe or scupper inlet; and this height is the necessary head to create flow pressure. The more water head, the more water flow, but unfortunately also the more water weight on the roof, and this water weight can exceed the design roof live load over portions of the roof (See Figure 5). Whose responsible is it for addressing this water weight on the roof? The architect selecting the roof configuration? The plumbing consultant selecting the drain size? The structural engineer designing the roof structure?

With so many parties potentially involved in the success of the roof drainage system, it is necessary for the parties to agree on their respective areas of responsibility but more importantly communicate their intentions and resulting ramifications to the others involved. ASCE 7-05 *Minimum Design Load for Buildings and Other Structures* (ASCE, 2005) recognizes the importance of communication among the design professionals by stating in their commentary “Roof drainage is a structural, architectural, and mechanical (plumbing) issue... Design team coordination is particularly important when establishing rain loads.”

Traditionally, architects establish the building shape and size, including roof geometry and drainage slope. Roofing materials and cricket requirements are also selected by the architect. The architect also works with the plumbing consultant to determine the roof drainage style, number of roof drains, and the roof drain locations.

Armed with this knowledge, the plumbing consultant computes the required water flow and sizes the primary and secondary roof drainage and all associated piping as necessary to stay within the framework of the architectural

design. Primary drains are designed to transport all the water in a design storm event, and secondary drains provide an emergency backup system should the primary system fail. For this emergency system, the plumbing consultant is either designing a second redundant drain pipe system or a scupper hole through the parapet wall to relieve the roof from any overloads. The type of system necessary is usually at the direction of the architect (or as mandated by the city as in our Orange County story presented).

In this traditional division of responsibility, the structural engineer has had a passive role in the drainage design, simply designing the roof structure across a roof slope set forth by the architect. Historically from the structural engineer’s perspective, extra loads placed on the roof from roofing materials, mechanical, electrical, plumbing equipment, building facades, and even rain loads should be provided by the design professionals who are most familiar with the magnitude of those loads.

Various passages from the portions of the governing codes may imply responsibility for who is to address the rainwater loads. In the 2009 *International Plumbing Code* (IPC, 2009), Section 1101.11.1 states that for the primary roof drainage “The location and sizing of drains and gutters shall be coordinated with the structural design and pitch of the roof,” implying that the plumbing consultant has responsibility to consider how his design affects the structural engineer’s design. Similarly, the 2009 IPC states that the secondary (emergency) drainage system shall be designed “to prevent the depth of ponding water from exceeding that for which the roof was designed....” implying that the responsibility to not overload the roof structure falls on the plumbing consultant who is designing the drainage system, assuming that the structural engineer’s design load has been provided to him through some means. Furthermore, in Appendix D of the 2009 IPC, Section D4.0 for the design of rectangular scuppers states “The maximum allowable level of water on the roof should be obtained from the structural engineer,

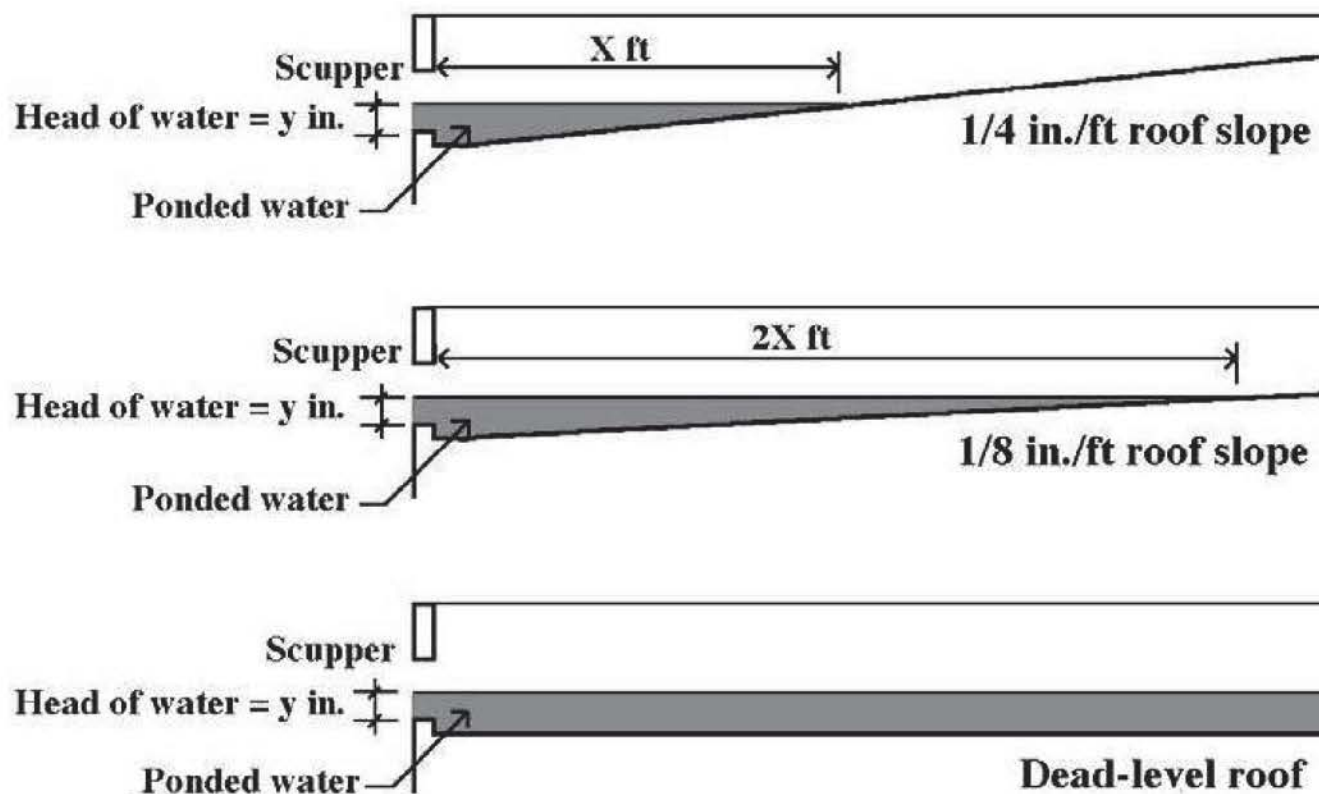


Figure 5: Ponded water weight behind a properly operating drainage scupper (Source: Patterson, 2010)

based on the design of the roof". Clearly this section places the burden on the drainage system designer who must obtain the allowable rainwater loading information from the structural engineer, but how often does this happen?!

As a licensed Structural Engineer in California for over 20 years, who has overseen over 100 million square feet of low-sloped roof structures with parapets in California, Nevada, Arizona, New Mexico and Oregon, I have never been asked to provide rainwater design loads to a project consultant. While my own experience is not a broad scientific survey, it does reflect my belief that much more communication is needed between the design consultants who impact roof drainage or are negatively affected by inadequate roof drainage.

This discussion has been quoting from the IPC, and it is seldom if ever reviewed in detail by a structural engineer; however, its companion 2009 IBC is. For the design of scuppers as secondary (emergency) roof drainage, Section 1503.4.2 of the 2009 IBC states that "When scuppers are used for secondary (emergency overflow) roof drainage, the quantity, size, location and inlet elevation of the scuppers shall be sized to prevent the depth of ponding water from exceeding that for

which the roof was designed..." Again, the burden seems to be placed upon the designer of the drainage system to live within the constraints of the roof structure design, assuming that the structural engineer's design load has been provided to him through some means.

Structural Engineer's Responsibility

With all the evidence above indicating the burden of protecting the roof structure from rainwater overload falls on those designing the drainage system, there are a several building code provisions that do place some responsibility for rain load design on the structural engineer. In the 2009 IBC Section 1611.1 and ASCE 7-05 Section 8.5, the building's rain load R specifically includes water weight that accumulates with some head height at the roof drainage collection points. The structural engineer is to combine this accumulating rain load R with other applicable loads as outlined in the various load combinations of IBC Section 1605. Besides water weight accumulating at the drainage collection points, the structural engineer is required to investigate the potential for ponding instability per IBC Section 1611.2 and repeated in ASCE 7-05 Section 8.6. Ponding instability begins as deflection under the water

weight progresses causing more and more water to be retained on very flat and flexible roof structures.

Water Accumulation at Drainage Collection Points

It is clear that the structural engineer must consider the weight of the rainwater that accumulates at the drainage points, as evident in the following 2009 IBC passage:

1611.1 Design rain loads. Each portion of the roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow. The design rainfall shall be based on the 100-year hourly rainfall rate indicated in Figure 1611.1 or on other rainfall rates determined from *approved* local weather data.

In order to eventually determine the accumulated water weight expected on the roof, the structural engineer needs to be given the anticipated depth of rainwater accumulating at the roof drain, or determine it himself. Often the structural engineer is working ahead of the plumbing consultant, and thus it may be beneficial to estimate this water weight or set a maximum limit for the plumbing consultant instead.

To estimate the rainwater weight on the roof, the rainfall intensity and design flow are needed. Similar to seismic design and wind design, the IBC now provides contour maps of 1-hour rainfall amounts associated with a 100-year return period storm, for the Western, Central and Eastern United States. From these maps, the amount of water required to flow through the roof drainage system can be calculated.

First introduced into the 2009 IBC (Figure 1611.1), these maps were developed by the National Weather Service but are based on fairly old data. For example, the Central and Eastern United States maps are based on rainfall maps published in 1977 (NWS, 1977), and the Western United States map is based on rainfall maps published over fifty years ago (Hershfield, 1961).

Recognizing the need to update these maps, the National Oceanographic and Atmospheric Administration (NOAA) is in the process of developing the NOAA Atlas 14 maps, providing better information with greater precision in full color. In 2004, these newer contour maps began to be released as they were completed for each state. Currently, states in the Southwest as well as states around the Ohio River Valley vicinity are updated and posted online at NOAA's National Weather Service website (http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_maps.html), and more are currently in progress. California's 100-year return

map which was recently released in 2011 is shown in Figure 6 and contains 60-minute rainfall rates that range from 0.5-inches to 3.5-inches. The electronic version of these maps can be significantly enlarged to view county lines, major highways, and topographic features all in an effort to assist the user in accurately locating a site of interest.

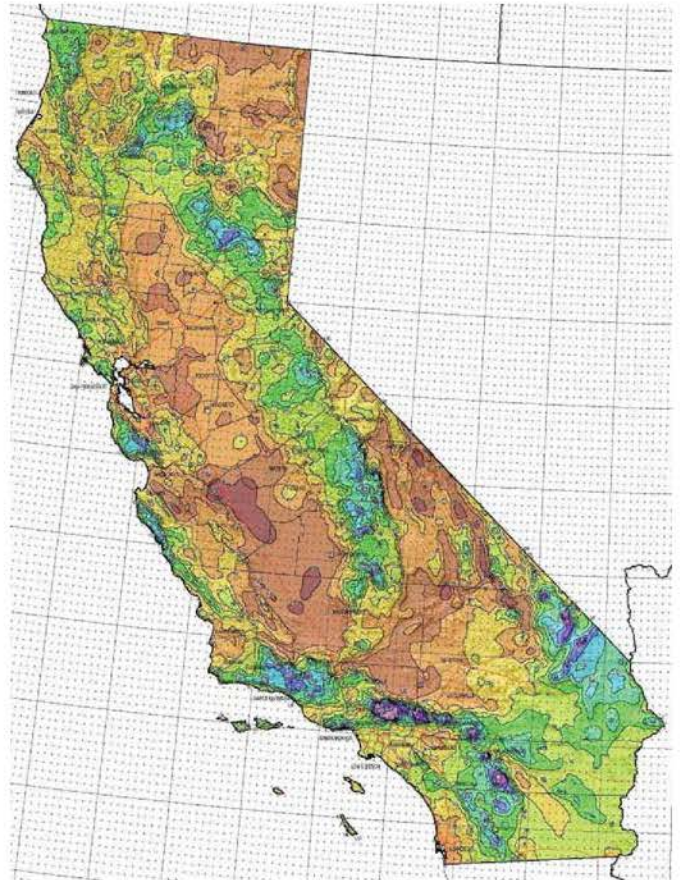


Figure 6: Graphic of 60-minute rainfall rates with a 100-year return period for California (Source: NOAA).

Probably more useful are NOAA's Atlas 14 interactive website maps where the user may point-and-click at any map location or manually enter a longitude and latitude to obtain very specific rainfall estimates at a precise location. The new Atlas 14 data is far more accurate than the maps currently in the IBC, and take into account much more rainfall history and the influence of regional topographic features. This interactive website is currently accessed at <http://hdsc.nws.noaa.gov/hdsc/pfds/index.html>. While this data is considered a self-proclaimed defacto national standard as it is released, the governing local jurisdiction should be consulted to determine which source is approved for use or whether the jurisdiction has their own adopted hourly rainfall intensity for design.

One source of concern is that 2009 IPC Appendix D also lists maximum rates of rainfall, but only for a small number of cities in each state based on the 1961 Data in U.S. Weather Bureau TP-40 (Hershfield, 1961). In California, only ten cities are listed in the IPC, yet NOAA's Atlas 14 map indicates dramatic variations are possible on a localized level due to topography. While it may be within the standard of care for a plumbing consultant to simply estimate or interpolate rainfall rates from these few cities in Appendix D, it may be prudent for plumbing consultants to consult the more recent Atlas 14 data or at the very least the 2009 IBC maps in Figure 1611.1 and select the worst case. It has been the author's experience in California that lower rainfall rates are often obtained from the more recent Atlas 14 data, except in mountainous regions.

As an illustrative example, the following information is for the Los Angeles area's precipitation depth PD for a 1-hour duration/100-year frequency rainfall:

2009 IBC Figure 1611.1 (1977 data):

1.5- to 2.5-inches Greater Los Angeles

2009 IPC Table D-1 (1961 data)

2.0-inches Los Angeles

NOAA Atlas 14 Interactive Maps (2011)

1.5-inches	Northridge
1.6-inches	Los Angeles City Hall
1.6-inches	Compton
1.7-inches	West Los Angeles
2.0-inches	Glendale
2.0-inches	Encino
2.6-inches	Topanga

As can be seen from the above example, a significant variation can occur in one single geographic region that is not captured in the current plumbing code.

After determining the precipitation depth for a one-hour design rainfall estimate, the design flow rates for the drains are simply the hourly precipitation depth multiplied by each drain's tributary collection area.

Required Drain Flow (Gallons per minute):

$$q = PD_{1\text{ hr}/100\text{ yr}} \times A_{\text{Trib}} \times \frac{1\text{ ft}}{12\text{ in}} \times \frac{7.48\text{ gal}}{\text{ft}^3} \times \frac{1\text{ hr}}{60\text{ min}}$$

where

q = drain flow in gallons per minute

$PD_{1\text{ hr}/100\text{ yr}}$ = Precipitation depth of one-hour duration rainfall occurring on average every 100 years in inches.

A_{Trib} = the tributary area of the roof surface projected on a horizontal plane that feed to the drainage of interest in square feet.

For an example warehouse building in Los Angeles ($PD = 2.0$ -inches per IPC) with parapets, a typical drainage collection point is fed by 18,000 square feet. Determine the required design drain flow:

$$q = 2.0 \frac{\text{in}}{\text{hr}} \times 18,000\text{ ft}^2 \times \frac{1\text{ ft}}{12\text{ in}} \times \frac{7.48\text{ gal}}{\text{ft}^3} \times \frac{1\text{ hr}}{60\text{ min}}$$

$$q = 374\text{ gallons per min}$$

Because there is a parapet, both the primary and secondary drainage systems must be designed independently for this water flow. These drainage systems may consist of vertical drain pipes with dome strainer caps, parapet holes acting as scuppers with or without attached downspouts, or a combination of each. Both systems must be sized appropriately to accommodate the design water flow. As required by IBC 1611.1, the structural engineer must design the roof structure "to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow."

Aggravating the accumulating water load on the roof is that the secondary drainage system inlet is often significantly higher than the roof surface causing water to rise before the secondary drainage provides relief. When a secondary roof drain pipe is used, the IPC requires it to be higher to mitigate clogging. For the secondary roof drain, Section 1101.11.2.2 of the 2009 IPC states:

The secondary roof drains shall be located not less than two (2) inches above the roof surface. The maximum height of the roof drains shall be a height to prevent the depth of ponding water from exceeding that for which the roof was designed....

While this is a requirement for roof drains acting as the secondary system, architects strongly desire to also raise the inlet elevation of scuppers used as emergency secondary systems. It is considered undesirable to have emergency scuppers often spilling water down and staining building sides in non-emergency situations. Thus, architects most often raise the

scupper inlets 2-inches above the roof as well, or even higher. The raising of the secondary drainage system inlet elevation directly contributes to ponding water design load at the low portions of the roof.

While a two-inch minimum rainwater depth seems inconsequential at slightly more than ten pounds per square foot, the additional head build-up necessary to achieve proper drainage flow can be substantial increase this load. Currently, primary and secondary drain pipes are required to have dome strainers extending a minimum of four inches high and having an inlet area of $1\frac{1}{2}$ times the conductor pipe area (IPC Sec. 1105.2). Considering the shape of these strainers, it is reasonable to assume that the equivalent inlet elevation is half of the four inches strainer height, or at two inches above the base. With the base also raised a minimum of two inches already for secondary drains and with the additional $1\frac{3}{4}$ " water head assumption of IPC Table 11-1 (Note 1) for design flow, logic follows that $2'' + 2'' + 1\frac{3}{4}'' = 5\frac{3}{4}''$ of water weight minimum is accumulating on the roof, or approximately 30 lbs/ft². If drains are intentionally oversized this 30 lbs/ft² load will reduce.

In the case of an overflow scupper set also with an inlet elevation of 2-inches above the roof, the necessary head to achieve design flow can be substantial more than the $1\frac{3}{4}''$ assumed for the drain pipe condition. Scupper flow rates through parapet walls are estimated using a channel type weir equation, where the water flow is bounded on three sides while open at the top. The height of the scupper opening should be at least two times the design head height as recommended by IPC Table D-2, Note 3. The weir flows in Table D-2 are based on the Francis Formula, presented here in a format consistent with our typical units:

$$q = 3.0(b - 0.2h)h^{3/2}$$

where

q = the flow rate in gallons per minute

b = the width (breadth) of the opening in inches

h = the head height of the water in inches

In our Los Angeles warehouse example, we computed a required design flow of 374 gallons per minute, and IBC Section 1611 requires the secondary scupper to be designed as if the primary drainage is fully blocked. Also, in this example the architect desires square scupper penetrations for aesthetic reasons. A plumbing consultant would compute the scupper size as follows.

Because of the desire for a square scupper penetration and because the height H should be twice the head height h we have the following relationship:

$$b = H = 2h$$

Substituting into the Francis Formula

$$q = 3.0(2h - 0.2h)h^{3/2}$$

$$q = 3.0(1.8h)h^{3/2}$$

$$q = 5.4h^{5/2}$$

With q given as 374 gallons per minute, the required head height to achieve this flow may be determined:

$$h = \left(\frac{q}{5.4}\right)^{2/5} = \left(\frac{374}{5.4}\right)^{2/5} = 5.45 \text{ inches}$$

and thus the required square scupper size is:

$$b = H = 2h \approx 11\text{-inches square}$$

An eleven inch square scupper will provide the necessary water flow to drain this portion of the roof should the primary drain become blocked during a design rain storm.

However, when determining the rainwater load on the roof, the 5.45" head height is added to the 2" inlet elevation above the roof to obtain a 7.45" water depth at the drain, or nearly 39 lbs/square foot of rain load. The concept of this total water weight accumulating near the drain is shown in Figure 5.

Rain load R and roof live load L_r are not combined in the IBC load combinations; however, in this warehouse example the rain load R clearly exceeds a basic roof live load of 20 psf, which is potentially even less at large tributary areas, and thus rain would likely govern the design of roof members covered with accumulating rainwater. In situations where the roof has sufficient slope, the effect of the high rainwater weight might be limited to only framing members in the immediate vicinity of the drainage point as shown in Figure 5.

In lieu of the square scupper shape, a wider scupper could be used to reduce the head height h and to reduce the rain load R on the roof. In order to reduce the rain load to around 20 psf, an approximate 4-inch water height would be the maximum allowed. With the scupper inlet at the typical 2-inch height above the roof surface, this leaves only 2-inches of head height available. Using the Francis Formula, the necessary scupper width b can be obtained.

$$q = 3.0(b - 0.2h)h^{3/2}$$

$$374 = 3.0(b - 0.2(2))2^{3/2}$$

$$b = \frac{374}{3(2)^{3/2}} + 0.2(2)$$

$$b = 44.5 \text{ in} \text{ Say } 45\text{-inches}$$

A rectangular scupper 45-inches wide and $2h = 4$ -inches tall does not seem very desirable architecturally or structurally in the parapet; however that is what would be necessary to limit the design rain load from exceeding 20 psf at the lowest point in the roof. This illustrates that it might not be realistic to expect the plumbing consultant to design his drainage system within the confines of the basic roof design live load.

Another approach would be to instruct the architect to limit the tributary drainage area for each drain to a specific number of square feet, but that might not be possible or desirable.

For a more proactive approach, the structural engineer could simply add some additional strength in the vicinity of all roof drains and scuppers to accommodate the higher rain load. It may be reasonable to set some upper limits, so it is the author's recommendation to design the roof framing in the drain vicinity for some maximum water depth, and communicate that maximum assumed depth to both the architect and plumbing consultant.

One suggestion is to design for a maximum water depth of 6-inches in the vicinity of the drainage low point, and to limit the scupper inlet height to 2-inches, thus providing a 4-inch head of water for flow. The 4-inch head achieves decent scupper flow rates, and more importantly accommodates the maximum heads listed in IPC Table D-2 for the plumbing consultant's reference, which contains scupper discharge rates in table-form derived from the Francis Formula.

Returning to our Los Angeles warehouse example with this suggestion, using a 4-inch water head h the necessary scupper width b is

$$b = \frac{374}{3(4)^{3/2}} + 0.2(4)$$

$$b = 16.4 \text{ in} \text{ Say } 17\text{-inches}$$

Alternatively, an 18-inch scupper width is obtained from IPC Table D-2. With the scupper height recommended to be equal to twice the head, the scupper size is 18" wide and 8" high. This is a more realistic scupper size than the 45" x 4" rectangular size for a 2-inch maximum head.

Following the 6-inch maximum water depth suggestion, the adequacy of the nearby framing members needs to be

investigated. The suggested 6-inch water depth results in a maximum 31.2 psf water load; however with a sloping roof, the water depth and load tapers down away from the drain point. While smaller framing members or decking adjacent to the drain might still have more than 20 psf of effective superimposed load, longer members extending upslope could have a water load effectively below their reduced design roof live load as shown in Figure 5.

Most often these low-sloped roof systems consist of either untopped metal decking or wood roof framing; and because these roofs are especially lightweight, excessive ponding water weight can quickly overwhelm their design capacity. But even with a blocked primary drain, the accumulated water weight in the vicinity of a functioning secondary drain or scupper only modestly overloads the roof in a localized region. Roof collapses from accumulating water weight at a drain are not very likely due to a structural design that omitted consideration for rain load. Instead, insufficient drainage design or drainage operation are the typical causes. Never the less, it is important for the structural engineer to consider the accumulating rain load in his design to avoid the appearance to a layperson (or juror!) that he contributed to the collapse in some way.

Investigating Potential for Ponding Instability

In low-sloped roof systems, decking, beams or joist may have some initial sag or deflection allowing water to pool or collect, causing more deflection, and thus more load to collect, and thus more deflection, and so forth. This progressive deflection and loading sequence of events may lead to a ponding instability failure where the water weight eventually overwhelms the roof structure strength. Sufficient roof slope and/or roof stiffness is necessary to prevent ponding instability. In rare situations, roof systems are installed dead flat, and water will pool to some depth or head before it can sufficiently flow through the drainage system. These dead-flat conditions are very susceptible to ponding instability, and must have significant strength and stiffness to preclude collapse failure.

With this in mind, the IBC requires a ponding investigation for roof slopes less than $\frac{1}{4}$ " per foot:

"1611.2 Ponding Instability. For roofs with a slope less than $\frac{1}{4}$ -inch per foot, the design calculations shall include verification of adequate stiffness to preclude progressive deflection in accordance with Section 8.4 of ASCE 7."

The $\frac{1}{4}$ " per foot roof slope magnitude is considered to be sufficiently steep to overcome long-term dead load deflections and construction tolerances which potentially

result in flat portions susceptible to ponding (ASCE, 2005, commentary).

On steel roof systems, a suitable analysis to determine if a roof has sufficient strength and stiffness is found in the *Specification for Structural Steel Buildings* (AISC, 2005). In its Appendix 2, a conservative procedure is provided to ensure two-way structural systems are sufficiently stiff to avoid ponding instability failure. A two-way system is an assembly of primary members (decking, joists, beams) and secondary members (beams, girders) in which both primary and secondary members are sufficiently flexible to have significant contribution to the overall ponding instability.

On two-way wood roof systems, the current *National Design Specification for Wood Construction* (AF&PA, 2005) does not have specific provisions for determining the adequacy of the roof's structural stiffness. However its predecessor, *Standard for Load and Resistance Factor Design for Engineered Wood Construction* (AF&PA, 1996) does provide a methodology to checking adequacy in Appendix A3.

In wood roof systems where either the primary or secondary members are relatively stiff compared with the other, a simplified one-way approach is useful as described in the *Timber Construction Manual* (AITC, 2005). This basic approach simply ensures that 1-inch of water weight does not lead to more than $\frac{1}{4}$ " of deflection, and the resulting design stresses are checked with a magnification factor provided in the text. Also the magnification factor addresses the effects of long-term creep and variation in modulus of elasticity for wood materials. These effects will be discussed more in depth later in this paper.

Probably the safest approach on low-sloped roof systems is to minimize the potential for any standing water on the roof surface. While the IBC states that a minimum slope of $\frac{1}{4}$ " per foot can satisfy the need to check ponding, the next section of the paper provides a more detailed approach to minimize standing water in certain structural systems.

Investigating Potential for Standing Water

Besides the instability issues that can be produced by ponding loads, the longevity of a roofing membrane is also significantly reduced by standing water. To minimize the possibility of standing water, sufficient roof slope and roof stiffness is necessary. Even with a $\frac{1}{4}$ " per foot roof slope, ponding water is possible if the roof structure is far too flexible. Likewise, it is possible to provide a $\frac{3}{16}$ " per foot roof slope of sufficient stiffness and avoid pooling of water.

In order to investigate this relationship, it is important to recognize where this issue is likely to first arise on a flat roof

with a shallow slope. As can be seen in Figure 7, flat roofs with shallow slopes are susceptible to the curvature of horizontal bending members. Beams with insufficient stiffness will deflect under dead loads and create flat spots or negative roof slopes that collect water at their lower end. Additionally, long-term creep, straightness tolerances, material variability, and assembly tolerances all must be considered when minimizing the potential for standing water.

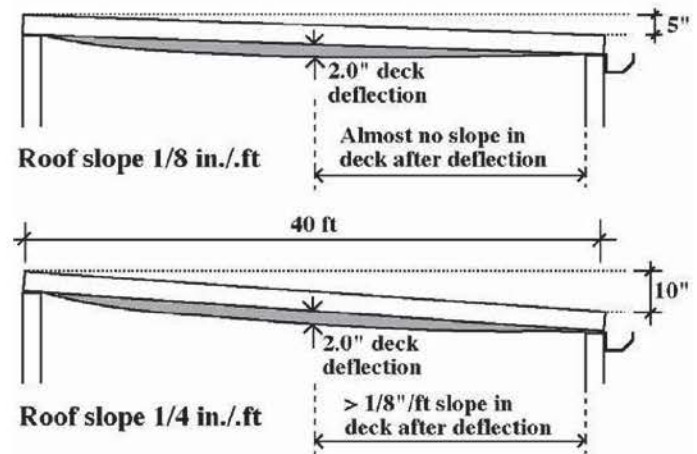


Figure 7: Deflected shapes may create insufficient slope at the lower end of bending members. (Source, Patterson, 2010)

Considering the relationship of roof slope and roof member stiffness, an expression can be derived to determine the necessary member stiffness for a given roof slope where the roof member's axis is parallel to the drainage direction. If the general roof slope is less than the localized end slope θ of the bending member, then water will pool there (See Figure 8). The IBC requirement for $\frac{1}{4}$ " per foot can be compared with the member's curvature from bending at the lower support.

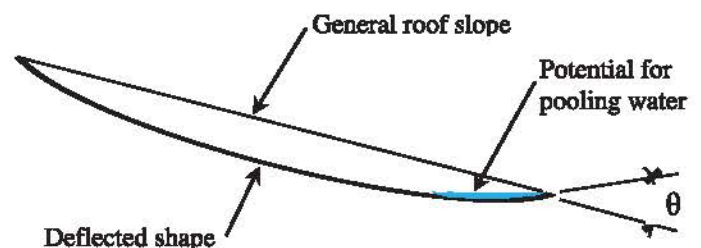


Figure 8: Excessive deflections cause water to pool.

An expression can be developed to compute the slope θ from curvature anywhere along a bending member of constant modulus of elasticity E and moment of inertia I . From elementary calculus (Beer, 2012), the relationship between a member's elastic curvature and its bending moment is expressed as:

$$EI \frac{dy}{dx} = \int_0^x M(x) dx$$

Where $M(x)$ is the expression for bending moment with respect to distance x along the beam, and more importantly dy/dx is the change in vertical beam location y with respect to horizontal location along the beam x . Stated another way, $dy/dx = \tan \theta$; however, the angles we are interested in are very small and thus the following expression may be used where θ is in radians:

$$\frac{dy}{dx} = \tan \theta \approx \theta(x)$$

Therefore:

$$EI \theta(x) = \int_0^x M(x) dx$$

For uniformly loaded simply supported beams, an expression for the bending moment in terms of distance x can be obtained from a free-body diagram:

$$M(x) = \frac{wL}{2}x - \frac{w}{2}x^2$$

Substituting into our slope equation and solving the integral:

$$EI \theta(x) = \int_0^x \left(\frac{wL}{2}x - \frac{w}{2}x^2 \right) dx$$

$$EI \theta(x) = \frac{wL}{4}x^2 - \frac{w}{6}x^3 + \text{Constant}$$

The constant can be solved with the help of the known boundary condition in which the slope is zero at the beam's mid-span. More specifically, $\theta = 0$ when $x = L/2$. Making this substitution

$$0 = \frac{wL}{4} \left(\frac{L}{2} \right)^2 - \frac{w}{6} \left(\frac{L}{2} \right)^3 + \text{Constant}$$

$$0 = \frac{wL^3}{16} - \frac{wL^3}{48} + \text{Constant}$$

$$0 = \frac{wL^3}{24} + \text{Constant}$$

$$\text{Constant} = -\frac{wL^3}{24}$$

Thus our complete slope equation for a uniformly loaded beam is

$$EI \theta(x) = \frac{wL}{4}x^2 - \frac{w}{6}x^3 - \frac{wL^3}{24}$$

Or more conveniently written as

$$\theta(x) = \frac{w}{24EI} (6Lx^2 - 4x^3 - L^3)$$

For a roof with the beam's axis parallel to the direction of general roof slope, we are most concerned with the beam's slope near the lower support where water may tend to pond. The beam's slope in radians at $x = L$ is

$$\theta(L) = \frac{wL^3}{24EI}$$

Water will begin to theoretically pond when the general overall roof slope is less than the local beam's slope from curvature at the lower support. IBC's minimum roof slope of 1/4" fall per horizontal foot may be written as follows for small angles

$$\theta \approx \frac{\Delta y}{\Delta x} = \frac{0.25"}{12"} = 0.0208 \text{ radians}$$

The beam is on the cusp of allowing water to pond when the beam's deflected shape contains a beam slope θ at the support equal to 0.0208 radians.

$$0.0208 = \frac{wL^3}{24EI}$$

or simplified further

$$\frac{wL^3}{EI} = 0.500$$

Typically, maximum allowed deflections are expressed as a ratio of the maximum vertical deflection Δy_{\max} divided by the total beam span L . For example, this dimensionless ratio is limited by the IBC in Table 1604.3 where limits of $L/120$, $L/180$, and $L/240$ are typical for roof systems supporting dead and roof live loads. In general, deflection limits of L/X are

specified and X increases as the need for stiffness increases. For a uniformly loaded, simply supported beam, the maximum estimated mid-span deflection is

$$\Delta y_{max} = \frac{5wL^4}{384EI}$$

And in general $\Delta y_{max} \leq L/X$ where as mentioned previously X is equal to 120, 180 or 240 typically for roof systems considering dead plus roof live loads. Substituting into the above equation

$$\frac{5wL^4}{384EI} \leq L/X$$

or

$$\frac{5}{384} \left(\frac{wL^3}{EI} \right) \leq 1/X$$

Substituting our previous expression derived for a ¼" per foot overall roof slope into this equation and solving for X , we obtain

$$\frac{5}{384} (0.500) \leq 1/X$$

$$X \geq \frac{384}{0.500(5)}$$

$$X \geq 154$$

What we have determined is that a beam's deflection with axis parallel to the roof slope cannot exceed $L/154$ when the overall roof slope is ¼" per foot to ensure no flat spots or bowl shaped depressions occur leading to pooling rain water. Evaluating the IBC load combinations, rain load R need not be combined with roof live load L_r and thus only the dead load deflection Δ_D must be smaller than $L/154$. This same approach can be used for other overall roof slopes besides ¼" per foot. Designers investigating the possibility of standing water on roofs with other overall slopes can use the following expression

$$X \geq \frac{38.4}{F}$$

Where F is the number of inches of fall per foot similarly as defined in IBC Section 1607.11.2.1.

Thus for any roof with a slope of F inches fall per horizontal foot, the dead load deflection Δ_D of bending members whose axes are parallel to the roof slope must be limited as follows to prevent a theoretical flat spot.

Equation 1:

$$\Delta_D \leq \frac{L}{38.4/F}$$

This expression has assumed ideal conditions in the evaluation of ponding potential. However, a number of other variables must also be considered.

Creep Effects on Ponding Potential

While steel roof members are relatively stable under long-term loading, wood and concrete members will creep downward under gravity with time. This creep increases the dead load deflection and thus increases the potential for the roof to pond water.

For wood construction, NDS Section 3.5.2 (AF&PA, 2005) provides a multiplier to the initial deflection from long-term loads, typically categorized as dead load deflection. This time dependent deformation (creep) factor K_{cr} is 1.5 for seasoned lumber and glued laminated timbers in dry conditions and 2.0 for unseasoned lumber or glued laminated timbers in wet service conditions. The vast majority of roof structures are designed assuming in-place seasoned lumber under dry service conditions, thus the dead load deflection is expected to creep an additional 50% over time. With this in mind, Equation 1 is modified to limit the *initial* dead load deflection Δ_{Di} when considering wood bending members.

Equation 2: (For dry-wood members)

$$\Delta_{Di} \leq \frac{L}{57.6/F}$$

Concrete construction has similar behavior, in which downward creep occurs over time, increasing the potential to pond water. The time dependent deformation (creep) factor for concrete is less predictable than for wood members and has historically been often underestimated (Gilbert, 1999). Designers who are sensitive to standing water on concrete roofs need to exercise engineering judgment and modify the Equation 1 as needed. While standing water on concrete roofs can negatively impact the roofing longevity, runaway ponding failure in concrete structures is normally unlikely due to the large self-weight of concrete structures compared with the pooling water weight.

Straightness Tolerances Affecting Ponding

Different framing members are subject to different fabrication tolerances that can exacerbate the potential for pooling water. For example, steel wide-flange beams may have some degree of curvature (inadvertent camber) as received from the mill. Steel mill straightness tolerances are specified in ASTM A6, and a maximum departure from a straight line is permissible up to 1/8" for every 10-feet of member length. Assuming a somewhat uniform curvature with maximum departure at mid-length, this is in essence an L/960 pseudo-deflected shape prior to installation. If the designer assumes that a beam with axis parallel to roof slope was installed with a worst case inadvertent camber in the downward direction, the following equation can be used to avoid a theoretical flat spot:

Equation 3: (For Steel Members)

$$\Delta_D \leq \frac{L}{38.4/F} - \frac{L}{960}$$

For cast-in-place concrete, the very nature of the construction requires greater tolerance to be allowed. *Specifications for Tolerances for Concrete Construction and Materials and Commentary* – ACI 117 (ACI, 2006) contains acceptable deviations of surfaces from a sloping plane, but unfortunately most of the limits are associated with floor systems and a lot more deviation is possible in a roof system unless limited specifically in the project specifications. As mentioned previously, ample engineering judgment is necessary when working with predicting ponding potential in concrete structures due to their varied behavior and construction tolerances. Never the less, the substantial self-weight of concrete structures compared with ponding water makes life safety concerns less of an issue.

For wood framing members, the milling tolerances and visual grading limits can be referenced in the visual grading rules; however, natural seasoning and moisture changes within the lumber will cause further changes to straightness. Fortunately, determining wood framing orientation in the field is often done, and seldom done with steel construction. Dimensional lumber can be requested to be installed with "crown up" indicating that any natural camber or "crown" shall be curved upwards. Having this flexibility removes the need to worry about straightness tolerances.

Glued-laminated timber beams and steel joists and joist-girders are usually cambered intentionally upward in roof systems and this assists in minimizing the potential for ponding water on low-sloped roofs, assuming the camber is not excessive. Heavily cambered beams and joists could

cause standing water at the upslope end of the member and should be investigated where standing water is a critical issue. Because this standing water is at the upslope end, it will not likely lead to ponding instability.

Material Stiffness Variation Affecting Ponding

A member's material stiffness is identified as the modulus of elasticity E or Young's Modulus. This material property is typically reported as an average value for computing estimated deflections. For a material with a wide statistical range for E , a value less than the average is justified for deflection critical applications.

Carbon steel at normal building temperatures has a very consistent E , and an adjustment is not necessary to capture lower portions of the acceptable range. On the other hand, the estimation of E for concrete is often difficult and a number of other issues associated with this material make estimating deflections problematic (Gilbert, 1999). As mentioned previously, ample engineering judgment is necessary when working with predicting ponding potential in concrete structures due to their varied material behavior. Never the less, the substantial self-weight of concrete structures compared with ponding water weight makes life safety concerns less of an issue.

Wood has a wide range for E even within a single grade of lumber, and should be accounted for when estimating deflections in structures sensitive to standing water. When considering ponding in wood structures, it is customary to use the lower fifth percentile modulus of elasticity, $E_{0.05}$, for computing member stiffness (AITC, 2005). With this statistical approach, there will be only a 5% chance that the actual material stiffness will be less than assumed. Computing $E_{0.05}$ is as follows based on the coefficient of variation for E (AF&PA, 2005).

$$E_{0.05} = E(1 - 1.645 \text{ COV}_E)$$

where

$E_{0.05}$ = the lower fifth percentile modulus of elasticity

E = the average modulus of elasticity design value

COV_E = the coefficient of variation for E

For visually graded sawn lumber, COV_E is 0.25 as found in the NDS Table F1 (AF&PA, 2005), and for typical glued-laminated timbers of at least six laminations, COV_E is 0.10.

Thus, for visually graded sawn lumber

$$E_{0.05} = E(1 - 1.645 (0.25))$$

$$E_{0.05} = 0.589E$$

In Equation 2 for Δ_{Di} we will substitute $E_{0.05}$ in place of E , to obtain a new equation for visually graded sawn lumber which accounts for overall roof slope, long-term creep and variation in material stiffness.

$$\Delta_{Di} \leq \frac{L}{\frac{E}{E_{0.05}} \frac{57.6}{F}} = \frac{L}{\frac{E}{0.589E} \frac{57.6}{F}}$$

We can simplify this equation further.

Equation 4: (For dry visually-graded sawn lumber)

$$\Delta_{Di} \leq \frac{L}{97.8/F}$$

Note that when wood framing members are spaced relatively close together, instead of acting individually the members begin to act collectively with load sharing (and stiffness sharing) between them. It is reasonable to assume the same guidelines apply as used for the repetitive-member factor found in the NDS Section 4.3.9 (AF&PA, 2005). Specifically, it is the author's belief that closely spaced repetitive members will behave more in line with the average E instead of $E_{0.05}$, assuming that the spacing is not more than 24-inches on center and are not less than three in number. Isolated purlins at 8-feet on center such as in a panelized roof system should be checked against $E_{0.05}$ for deflection sensitive roofs. Spacings in between are subject to more engineering judgment.

A similar equation is obtained for glued-laminated timber (six or more lams) considering variation in E , but is likely not applicable because sufficient camber is usually provided to offset the effects of dead load deflection and long-term creep.

Thus, for glued-laminated timbers without camber

$$E_{0.05} = E(1 - 1.645(0.10))$$

$$E_{0.05} = 0.836E$$

Equation 5: (For glued-laminated timbers without camber)

$$\Delta_{Di} \leq \frac{L}{68.9/F}$$

Other Issues Affecting Ponding Potential

For a roof with a shallow overall slope, this paper has suggested limiting the calculated initial deflection while

addressing overall roof slope, long-term deflection creep, straightness tolerances, and variations in the material's E . Because construction is an imperfect process, other variables can sabotage sometimes the best laid plans. The flatness of a roof system is also sensitive to framing connection fit-up tolerances, the layering of a built-up roofing membrane, and the uniformity of rigid insulation installation.

Controlling these issues is best in the hands of the contractor and subcontractors. NCRA's performance-based criteria of allowing water to stand up to 48-hours and remaining in compliance provides some tolerance to these other construction variables.

Load Duration Adjustment Factors for Wood

The design of wood framing considers the duration of loading. Wood has the unique ability to withstand higher loads for shorter time periods, and thus a stress adjustment factor C_D is provided in the NDS (AF&PA, 2005). However, there is no clear guidance on what is a proper C_D factor for rain load R .

C_D is based on the cumulative duration of the maximum load during the life of the structure. When water ponds due to a beam's deflection (Figure 8), significant amounts of water may stay there for weeks until evaporated. But when water accumulates at a drainage low spot waiting to flow out (Figure 5), the water is there for a brief time period. Thus for ponding of water at the mid-span of a deflected beam, C_D should be based on a longer duration than the accumulation of water at drainage low spot.

The use of an adjustment factor similar to a snow load, $C_D=1.15$ for 2 month duration, is suggested for midspan ponding and is likely conservative for most cases and should not raise concerns from reviewing agencies. This approach may be more justifiable than attempting to use a $C_D=1.25$ for a 7 day duration for ponding if significant water depths could occur.

For designing wood framing members adjacent to the drainage low spots, a larger C_D can be justified if necessary. Assuming a fully functioning secondary drainage system, the IBC's design criterion is a 1-hour duration rain storm which occurs on average every 100 years. Thus for a building with a 100-year or less expected lifespan, the maximum ponding load is a single event with a 1-hour duration.

To compute the load duration factor C_D for a 1-hour loading, we must revisit the "Madison Curve", first developed by the Forest Products Laboratory in Madison, Wisconsin. The Madison Curve has its basis in an empirically derived hyperbolic curve normalized at 7.5 minute (Wood, 1951).

Madison Curve Equation:

$$SL = \frac{108.4}{D^{0.04635}} + 18.3$$

where

SL = Strength Level compared with 7.5 minute loading (%)

D = Duration of loading in seconds

This load duration equation is more useful if we normalize the strength comparison around the 10-year duration time frame instead of 7.5 minutes, as is done in the NDS (AF&PA, 2005). Reworking the equation, the following expression is obtained:

$$C_D = \frac{SL(\text{duration } D)}{SL(10 \text{ years})}$$
$$= \frac{\frac{108.4}{D^{0.04635}} + 18.3}{\frac{108.4}{(10\text{yrs} \times 365\text{days} \times 24\text{hrs} \times 60\text{min} \times 60\text{sec})^{0.04635}} + 18.3}$$

This can be simplified to be more useful.

Equation 6:

$$C_D = \frac{1.747}{D^{0.04635}} + 0.2949$$

This equation derived here is in general agreement with the C_D load duration factors within NDS Table 2.3.2 and the graphed curve in NDS Appendix B used for allowable stress design. Because the design rain intensity is defined as a 1-hour duration with a reoccurrence interval of 100 years, and because the useful life of a structure is seldom greater than 100 years, an appropriate load duration factor C_D can be obtained from this equation for water that accumulates near a properly functioning secondary drain.

$$C_D = \frac{1.747}{(1 \times 60 \text{ min/hr} \times 60 \text{ sec/min})^{0.04635}} + 0.2949$$

or approximately, $C_D = 1.5$

Closing Remarks

The collapse of lightweight low-sloped roof structures occurs too often during rain storms. While the culprit most often appears to be an inadequate or clogged drainage system, the structural engineer can be pulled into the lawsuit simply to find more money to settle the damage claim. In states with a joint-and-several liability system such as California, in negligence cases such as these jurors or arbitrators only need

to be convinced that the standard of care was not met and that it contributed in some amount (even a small amount) to the damages. Even if everyone knows the primary cause of the roof collapse was primary and secondary drains clogged with leaves resulting in 18-inches of water on the roof, a roof beam that is overstressed by 10% under the normal head of the secondary drain could bring the structural engineer into the litigation.

To avoid landing into this kind of trouble, whether justified or not, structural engineers of low-sloped roof systems should pay attention to how the water is being transported off the roof and communicate with the architect and plumbing consultant if possible.

For the water accumulation at the drains, the structural engineer should either check the anticipated rainwater load on the roof structure here, or simply indicate on the plans (or in a letter) what the assumed maximum design rainwater depth at the drains is. Communicating this information to the drainage designer is a proactive approach that will assist a smooth, transparent design process. Waiting to check the drainage design towards the end of every job and potentially redesigning portions of the roof structure to make it work may not be desirable to expediting the job.

For ponding instability, the structural engineer should verify that the architect has provided a general $\frac{1}{4}$ " per foot slope. If a flatter slope has been specified, use the discussed AISC procedure for steel structures or AF&PA procedure for wood structures to verify sufficient stiffness is provided. Alternatively for primarily one-way wood structures, a $\frac{1}{2}$ " deflection limit for 5psf added load can be utilized. Roof members with intentional camber greatly mitigate ponding instability.

Where standing water is much more than a structural concern and the architect or owner has expressed that it is a very sensitive issue for roofing longevity, the roof structure may need to be made stiffer to theoretically remove the flat spots or to limit their pooling depths to allow for rapid drying. In roofs with steel and wood members without camber, a series of equations have been provided to assist in mitigating roof flat spots.

While designing low-sloped roof structures for large amounts of pooling water near drains has not yet become the standard of care for structural engineers in California, the provisions in the IBC, IPC and ASCE 7 clearly indicate that some consideration is needed.

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