

LRFD Limit States and Load Combinations

Every LRFD bridge design check starts with the same inequality: the factored resistance must be at least the factored load effect. This course explains the reliability philosophy behind the AASHTO LRFD Bridge Design Specifications, the four groups of limit states (strength, service, extreme event and fatigue and fracture), the load factors in Table 3.4.1-1 and the permanent load factors in Table 3.4.1-2, the load modifiers for ductility, redundancy and operational importance, and a practical way to pick the governing combination for a member. Table values were checked against the current form of the tables reproduced in AASHTO's SHRP2 R19B implementation white paper, which shows the wind and fatigue revisions carried into the 8th (2017) and 9th (2020) editions. Example numbers are illustrative teaching values unless a source is named. Always check the edition and interim revisions your owner has adopted.

Learning objectives

- Explain the LRFD reliability philosophy
- Identify strength, service, extreme event and fatigue limit states
- Apply load factors and load modifiers
- Select governing combinations

The LRFD idea: reliability, not a single safety factor

Under allowable stress design, one factor of safety covered every source of uncertainty. Load factor design added factors on loads. LRFD goes further: it puts separate factors on each load and on resistance, and the factors were calibrated so that bridges designed to the specification reach a fairly uniform level of reliability across span lengths, materials and load types.

The basic requirement in Article 1.3.2.1 is that, for each limit state,

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

where Q_i is a force effect, γ_i a load factor, η_i a load modifier, R_n the nominal resistance, ϕ a resistance factor and R_r the factored resistance. The left side is the total factored force effect, written in Article 3.4.1 as $Q = \sum \eta_i \gamma_i Q_i$ (Eq. 3.4.1-1).

Where the factors came from

The strength limit state factors were calibrated in NCHRP Report 368, *Calibration of LRFD Bridge Design Code* (Nowak, 1999). The calibration treated loads and resistances as random variables, estimated the reliability index β of bridges designed under the older Standard Specifications, and chose load and resistance factors that would bring designs close to a target reliability. The MnDOT LRFD Bridge Design Manual sums this up: the load factors and combinations in Table 3.4.1-1 were calibrated to produce structures with more uniform reliability than Standard Specification designs.

Two practical consequences follow:

- Load factors reflect how uncertain each load is. Dead load of factory-made components (DC) is well known and gets a maximum factor of 1.25. Wearing surfaces and utilities (DW) are less certain and get 1.50. Vehicular live load in the basic strength combination gets 1.75.

- Resistance factors reflect how uncertain each resistance model is, and they are set in the material sections (Sections 5, 6, 7, 8 and so on). They only give the intended reliability when used with the load factors they were calibrated with. Mixing factors from different codes breaks that link.

Limit states

A limit state is a condition beyond which the bridge or a component no longer meets the requirement it was designed for. The specification groups them into four kinds (Article 1.3.2):

- Service limit states** restrict stress, deformation and crack width under regular service conditions.
- Fatigue and fracture limit states** restrict stress range under repeated truck passages and set toughness requirements.
- Strength limit states** ensure strength and stability, local and global, for statistically significant load combinations the bridge is expected to see in its design life.
- Extreme event limit states** ensure the structural survival of the bridge during a major earthquake, flood, vessel or vehicle collision, or ice event.

Service and fatigue checks protect durability and function. Strength checks protect against failure under heavy but expected loads. Extreme event checks accept that damage may occur, but aim to keep the bridge from collapsing.

Table 3.4.1-1: the load combinations

Table 3.4.1-1 lists each load combination as a row and each load as a column. The values below are the live, wind and fatigue factors in the current form of the table, as reproduced in AASHTO's SHRP2 R19B implementation white paper. Permanent loads use γ_p from Table 3.4.1-2 in the strength and extreme rows and 1.00 in the service rows.

Combination	DC, DW, EH, EV, ES...	LL, IM, CE, BR, PL, LS	WS	WL	TU
Strength I	γ_p	1.75	—	—	0.50/1.20
Strength II	γ_p	1.35	—	—	0.50/1.20
Strength III	γ_p	—	1.00	—	0.50/1.20
Strength IV	γ_p	—	—	—	0.50/1.20
Strength V	γ_p	1.35	1.00	1.00	0.50/1.20
Extreme Event I	1.00	γ_{EQ}	—	—	—
Extreme Event II	1.00	0.50	—	—	—
Service I	1.00	1.00	1.00	1.00	1.00/1.20
Service II	1.00	1.30	—	—	1.00/1.20
Service III	1.00	γ_{LL}	—	—	1.00/1.20
Service IV	1.00	—	1.00	—	1.00/1.20
Fatigue I (LL, IM, CE only)	—	1.75	—	—	—
Fatigue II (LL, IM, CE only)	—	0.80	—	—	—

Water load and stream pressure (WA) and friction (FR) take 1.00 in every row except fatigue. The extreme event loads (EQ, BL, IC, CT, CV) each take 1.00 in their row, and the table says to use only one of them at a time.

Changes engineers should know about

- **Wind.** Older editions used $WS = 1.40$ in Strength III, 0.40 in Strength V, 0.30 in Service I and 0.70 in Service IV. In the current table these are all 1.00 , because the wind provisions were revised so that the wind pressure itself depends on the combination's design wind speed. Mixing an old-edition factor with new-edition wind pressures gives wrong answers.
- **Fatigue.** The Fatigue I factor rose from 1.50 to 1.75 and the Fatigue II factor from 0.75 to 0.80 , as summarized in the Grubb presentation on the 8th and 9th edition Section 6 changes.
- **Service III.** The live load factor is now γ_{LL} from Table 3.4.1-4: 1.0 for prestressed components designed with the refined estimates of time-dependent losses in Article 5.9.5.4 together with elastic gain, and 0.8 for all other prestressed components. This came from SHRP2 R19B service limit state research.

Temperature, gradient and settlement

Uniform temperature (TU), creep (CR) and shrinkage (SH) carry two values, such as $0.50/1.20$. The larger value is used for deformations and the smaller for all other effects. Gradient (TG) and settlement (SE) use project-specific factors γ_{TG} and γ_{SE} .

Table 3.4.1-2: permanent load factors

Permanent loads in strength and extreme event combinations use γ_p , which has a maximum and a minimum value. The current values reproduced in the SHRP2 R19B white paper include:

Load	Maximum	Minimum
DC: Component and attachments	1.25	0.90
DC: Strength IV only	1.50	0.90
DD: Downdrag, piles, α Tomlinson method	1.4	0.25
DD: Downdrag, piles, λ method	1.05	0.30
DD: Downdrag, drilled shafts, O'Neill and Reese (1999)	1.25	0.35
DW: Wearing surfaces and utilities	1.50	0.65
EH: Active	1.50	0.90
EH: At-rest	1.35	0.90
EH: AEP for anchored walls	1.35	N/A
EL: Locked-in construction stresses	1.00	1.00
EV: Overall stability	1.00	N/A
EV: Retaining walls and abutments	1.35	1.00
EV: Rigid buried structure	1.30	0.90
EV: Rigid frames	1.35	0.90
ES: Earth surcharge	1.50	0.75

The table also gives EV factors for flexible buried structures by culvert type; check the table directly for those.

Using maximum and minimum values

Use whichever value produces the more critical effect. The MnDOT manual gives a clear culvert example: for moment in the top slab, use the maximum factor on vertical earth load and the minimum on lateral earth load; for moment in the walls, reverse them. It also warns against using more than one factor for the same load component. When checking uplift at an end support, for instance, use 0.90 or 1.25 on the dead load of all spans, not 0.90 on one span and 1.25 on the next.

Worked example (illustrative numbers)

The following are illustrative teaching values. An interior girder of a simple-span composite bridge has unfactored midspan moments of $M_{DC} = 1,400$ kip-ft, $M_{DW} = 220$ kip-ft and $M_{LL+IM} = 1,650$ kip-ft, with all load modifiers equal to 1.00.

- Strength I: $1.25(1,400) + 1.50(220) + 1.75(1,650) = 1,750 + 330 + 2,888 = 4,968$ kip-ft
- Strength IV: $1.50(1,400) + 1.50(220) = 2,100 + 330 = 2,430$ kip-ft
- Service II (steel girder yielding check): $1.00(1,400) + 1.00(220) + 1.30(1,650) = 3,765$ kip-ft

For this ordinary span, Strength I governs strength design by a wide margin. Strength IV only governs when dead load is very large compared with live load, as in long spans.

The combinations one by one

Knowing what each combination is for makes it much easier to decide which ones matter for a given member. The descriptions below follow Article 3.4.1, with practical notes from the MnDOT LRFD Bridge Design Manual (January 2019) where they help.

Strength combinations

- **Strength I** is the basic combination for normal vehicular use without wind. It governs flexure and shear in most girders, decks, and many substructure elements of short and medium span bridges. If you only remember one combination, this is it.
- **Strength II** covers use of the bridge by owner-specified special design vehicles, evaluation permit vehicles, or both, without wind. The live load factor is lower (1.35) because the permit vehicle weight is better known than random traffic. MnDOT notes that it does not typically use special design vehicles, so Strength II rarely applies to its designs; other owners specify permit design vehicles and Strength II then must be checked.
- **Strength III** is for the bridge exposed to the design wind, with no live load. The reasoning is that vehicles are not expected on a bridge during extreme wind. MnDOT describes its design wind as based on a 3-second gust speed of 115 mph. Strength III often governs tall piers, high-level bridges and lateral bracing.
- **Strength IV** relates to very high dead-to-live load ratios. It uses the DC factor of 1.50 and no live load. MnDOT points out that it applies only to superstructures and directs designers to the modified combination given in AASHTO Commentary C3.4.1, $1.4DC + 1.5DW + 1.45LL$; check whether your edition and owner use that form. Strength IV matters for long-span bridges where dead load dominates.
- **Strength V** is normal vehicular use with wind. MnDOT describes it as concurrent with a design wind based on a 3-second gust of 80 mph. It uses $LL = 1.35$ with $WS = 1.00$ and wind on live load $WL = 1.00$. It can govern piers and bracing where wind and live load act together.

Extreme event combinations

- **Extreme Event I** includes earthquake. The live load factor γ_{EQ} is set on a project-specific basis. Some owners fix it by policy; the NYSDOT LRFD Blue Pages (October 2024), for example, set $\gamma_{EQ} = 0.50$ for bridges with ADTT greater than 5,000 and 0.00 for all others. Seismic design itself is governed by Article 3.10 or by the AASHTO Guide Specifications for LRFD Seismic Bridge Design, which are covered in a separate course.
- **Extreme Event II** covers ice load, vessel collision, vehicle collision and certain hydraulic events, with live load reduced to 0.50. MnDOT uses it for barrier design, deck overhang design and pier design under its pier protection policy. Only one of the extreme event loads (EQ, BL, IC, CT, CV) is applied at a time.

Service combinations

- **Service I** is normal operational use with all loads at nominal values, plus wind. It is used for many checks: compressive stresses in prestressed concrete, deflection, crack control in reinforced concrete, and slope stability among others.
- **Service II** controls yielding of steel structures and slip of slip-critical connections under vehicular live load. Its live load factor is 1.30.
- **Service III** checks tension in prestressed concrete superstructures for crack control, and principal tension in the webs of segmental concrete girders. Its live load factor is γ_{LL} from Table 3.4.1-4 (0.8 or 1.0, depending on how losses were estimated).
- **Service IV** relates only to tension in prestressed concrete substructures, with the objective of crack control. It has wind but no live load.

Fatigue combinations

- **Fatigue I** is for infinite load-induced fatigue life. Its factor of 1.75 on the single fatigue truck represents the largest stress range the detail is expected to see; the detail must keep that stress range below its constant amplitude fatigue threshold.
- **Fatigue II** is for finite load-induced fatigue life. The factor of 0.80 represents an effective stress range from the spectrum of trucks, used with the number of cycles over the design life.

Fatigue combinations include only live load, dynamic load allowance and centrifugal force. Permanent loads do not change, so they do not cause a stress range. The fatigue truck, dynamic allowance for fatigue, and stress cycle counts are covered in the fatigue course in this catalog.

Construction

Construction stages need their own checks. Article 3.4.2 gives load factors for construction loads, and MnDOT reminds designers to check every appropriate combination at every stage. A high parapet abutment, for instance, may be backfilled before the beams are placed, which maximizes horizontal earth pressure with little vertical load, or the superstructure may be completed before backfilling. Both cases must be investigated.

Load modifiers: ductility, redundancy and importance

The load modifier η_i scales the factored load to reflect how the bridge is likely to behave and how important it is. It is a product of three factors defined in Articles 1.3.3 through 1.3.5:

- η_D , for ductility
- η_R , for redundancy
- η_I , for operational importance

When the maximum value of γ_i is appropriate, $\eta_i = \eta_D \eta_R \eta_I \geq 0.95$. When the minimum value of γ_i is appropriate, $\eta_i = 1/(\eta_D \eta_R \eta_I) \leq 1.00$. The inverse form ensures that a modifier meant to add safety does not reduce safety when a load is helping, for example dead load resisting uplift.

Ductility

A ductile structure gives warning before failure through large inelastic deformations, and it can redistribute load. For the strength limit state, the specification sets:

- $\eta_D \geq 1.05$ for nonductile components and connections
- $\eta_D = 1.00$ for conventional designs and details complying with the specifications
- $\eta_D \geq 0.95$ for components and connections for which additional ductility-enhancing measures have been specified beyond those required

For all other limit states, $\eta_D = 1.00$. MnDOT's policy table, as an owner example, uses 1.00 for steel structures, timber bridges and ductile concrete structures, and 1.05 for nonductile concrete structures.

Redundancy

Multiple load path and continuous structures should be used unless there are compelling reasons not to. For the strength limit state:

- $\eta_R \geq 1.05$ for nonredundant members
- $\eta_R = 1.00$ for conventional levels of redundancy
- $\eta_R \geq 0.95$ for exceptional levels of redundancy

For all other limit states, $\eta_R = 1.00$. MnDOT treats beam-type superstructures with four or more beams per span as redundant, which is a practical rule that many owners use in some form. A two-girder bridge, a single-box girder, or a single-column pier is commonly treated as nonredundant. Redundancy in the sense of load paths is related to, but not the same as, the federal fracture-critical member designation; owners may use other criteria, and system redundancy can be shown by analysis.

Operational importance

This factor applies only to the strength and extreme event limit states. Traditionally the specification set $\eta_I \geq 1.05$ for critical or essential bridges, 1.00 for typical bridges and ≥ 0.95 for relatively less important bridges. The 9th edition revised Article 1.3.5, and owners classify bridges using their own criteria, often tied to emergency routes, security, and the consequences of loss. Check the exact text of your adopted edition and the owner's classification. Owner examples:

- NYSDOT's 2024 Blue Pages replace the paragraph with $\eta_I = 1.05$ for critical bridges and 1.00 for essential and other bridges at the strength limit state, and 1.00 for all other limit states.
- MnDOT's table uses 0.90 for temporary bridges, 0.95 for ADT under 500, 1.00 for ADT from 500 to 40,000, and 1.05 for major river crossings, ADT over 40,000 on the bridge, or mainline interstate on the bridge. MnDOT applies it to

- superstructure design only, not to decks of deck-on-girder bridges, and only to new bridges.
- Caltrans' Bridge Design Practices chapter on specifications indicates that values of 1.0 are used for η_D , η_R and η_I until their application is studied further.

These differences show why the owner's policy must be checked. The same bridge could legitimately be designed with $\eta = 1.00$ in one state and $\eta = 1.05$ or higher in another.

Effect on the design

A modifier of 1.05 increases every factored load effect in the combination by 5 percent. For a nonredundant, nonductile, critical structure the product could reach about 1.16 ($1.05 \times 1.05 \times 1.05$). That is a significant increase, and it reflects the higher consequence of failure and lower warning that such a structure has. Load modifiers are not a substitute for good detailing; a designer should first try to provide ductility and redundancy, and use the modifier only when they cannot be provided.

The η limits of 0.95 and 1.00 apply to the product, not to each factor separately. Read the article text carefully when combining a factor below 1.00 with one above.

A workflow for finding the governing combination

A bridge has dozens of members and hundreds of design sections. Checking every combination at every section by hand is not practical, and design software does most of that work. The engineer still has to set the problem up correctly and judge whether the output makes sense. The workflow below is a practical way to organize the work. It is not a procedure taken from the specification. It is a way to make sure the specification's requirements are met.

Step 1: Confirm the governing documents

Identify the adopted edition of the AASHTO LRFD Bridge Design Specifications, any interim revisions, and the owner's manual or blue pages. As this course shows, the owner can change load modifiers, the γ_{EQ} value, whether Strength II applies, and the form of Strength IV. Write these decisions down in the design criteria before analysis starts. Changes found late in design are expensive.

Step 2: List the loads that act on the member

Not every load in Table 3.4.1-1 applies to every member. A deck slab sees DC, DW, LL and IM, and its overhang sees CT in Extreme Event II. A pier column may see DC, DW, LL, BR, WS, WL, TU, CE, FR, EQ, CV and WA. A buried culvert sees EV, EH, ES and LL, and DC from its own weight. Making a list keeps loads from being forgotten and keeps loads that do not apply from cluttering the work.

Step 3: Choose the combinations that can govern

Use the purpose of each combination to narrow the list. For a typical multi-girder superstructure, Strength I generally governs girder flexure and shear, Service I governs deflection and compressive stress in prestressed concrete, Service II governs permanent deformation of steel and slip-critical connections, Service III governs tension in prestressed girders, and the fatigue combinations govern steel details. Strength III, Strength V and Extreme Event combinations usually matter more for substructures, bracing and long or tall bridges. If a combination can be shown not to govern by inspection, record why. Do not drop a combination simply because it rarely governs.

Step 4: Apply maximum and minimum factors

For each permanent load, choose the maximum or minimum factor from Table 3.4.1-2 so as to produce the extreme force effect. Remember that the specification says each force effect is to be determined using the load factor that produces the extreme total, and that for the same combination you may need to run several cases. The MnDOT box culvert example illustrates this: the top slab moment uses the maximum factor on vertical earth load and the minimum on horizontal earth load, and the wall moment reverses the choice.

Step 5: Apply load modifiers

Apply the owner's η values for the strength (and, if applicable, extreme event) limit states. Use $\eta = \eta_D \eta_R \eta_I$ with maximum load factors and its inverse with minimum load factors.

Step 6: Envelope the results

For each design section, keep the maximum and minimum of each force effect from all cases. Live load envelopes come from moving the design vehicle and lane load over influence lines, and different positions control positive moment, negative moment and shear. Permanent and live effects are then combined using the factors for each case.

Step 7: Compare with factored resistance

Check that the factored load effect does not exceed the factored resistance ϕR_n at the strength and extreme event limit states, and that stresses, deformations and crack widths satisfy the service and fatigue requirements. Resistance factors ϕ come from the material sections (5 for concrete, 6 for steel, 8 for wood, 10 and 11 for foundations and walls). At service and fatigue limit states the resistance factor is usually 1.0, and at the extreme event limit state it is commonly 1.0 unless otherwise specified; confirm the value in the relevant material section.

Step 8: Review the results

Compare the results with a hand check of the largest effects. A simple-span dead load moment of $wL^2/8$ or an approximate live load moment from a table is often enough to catch an input error such as a missing lane load, a wrong unit, or a wrong distribution factor. A good habit is to ask which combination you expected to govern and investigate if the software says something different.

Using maximum and minimum factors correctly

The maximum and minimum factors of Table 3.4.1-2 cause more errors than almost anything else in this part of the specification. The rules are simple once stated clearly.

The principle

Uncertainty in a permanent load works in both directions. The actual dead load could be heavier or lighter than estimated. When a load adds to the effect being checked, a heavier-than-expected load is the dangerous case, so the maximum factor is used. When a load reduces the effect being checked, a lighter-than-expected load is the dangerous case, so the minimum factor is used.

One factor per load, per case

Within one load case, a given permanent load type carries one factor throughout the structure. MnDOT's manual uses uplift as the example: when investigating uplift at a bearing of a continuous beam, it is not correct to use the maximum factor on the DC in spans that produce uplift and the minimum factor on the DC in spans that resist it. For each case, one value of the factor is applied to the full load. The designer runs a case with the maximum factor and a case with the minimum factor and takes the more severe result.

Typical situations where minimum factors govern

- **Uplift at bearings**, in continuous girders with unequal spans or in curved and skewed bridges, where dead load holds the girder down.
- **Overturning and sliding of walls and abutments**, where vertical earth load and structure weight provide stability. Note the separate EV value for overall stability in the table.
- **Culvert design**, as in the MnDOT example, where vertical and horizontal earth pressures counteract each other in different elements.
- **Footing design**, where minimum vertical loads may control eccentricity and sliding even though maximum loads control bearing pressure.
- **Prestressed concrete at release or in negative moment regions**, where self-weight reduces the stress caused by prestress. (These are service and construction checks, and their factors and stress limits are defined in Section 5.)

Special entries in the table

A few rows of Table 3.4.1-2 deserve extra attention:

- **DC, Strength IV only (1.50/0.90)**. This higher maximum applies only to the Strength IV combination. It reflects the greater relative importance of dead load uncertainty when dead load dominates.
- **DD, downdrag**. The factors depend on the method used to calculate the downdrag on piles or shafts. Using the factor for the wrong method gives the wrong result.
- **EH, horizontal earth pressure**. Different factors apply to active, at-rest and AEP (apparent earth pressure for anchored walls).
- **EV, vertical earth pressure**. Different factors apply to overall stability, retaining walls and abutments, rigid buried structures, rigid frames, and other buried structure types. Check the full table for flexible buried structures, which are not repeated in this course.
- **EL**. Locked-in construction force effects, including those from secondary forces of post-tensioning, take 1.00 for both maximum and minimum in the strength limit states.

Illustrative example: an exterior bearing uplift check

All numbers in this example are illustrative, chosen to show the method. They are not from a real bridge or from the specification.

Consider a two-span continuous steel girder with a short end span. At the end bearing, the unfactored reactions are assumed to be:

- DC (girder, deck, barrier): +60 kips (downward)
- DW (future wearing surface): +8 kips

- LL+IM, maximum downward: +55 kips
- LL+IM, maximum upward (lanes in the long span only): -40 kips

Maximum downward reaction, Strength I

Use maximum factors: $1.25(60) + 1.50(8) + 1.75(55) = 75 + 12 + 96.25 = 183.25$ kips, which would be multiplied by η . With $\eta = 1.00$, the design reaction is about 183 kips.

Maximum uplift, Strength I

Use minimum factors on the permanent loads, because they resist uplift, and the full live load factor on the upward live load. DW is often left out entirely when it helps, because a future wearing surface may not be present; the minimum DW factor of 0.65 in the table applies when the load is assumed present, and owners vary on whether to include it. Here, taking the conservative approach of leaving DW out: $0.90(60) + 1.75(-40) = 54 - 70 = -16$ kips. The net result is uplift of 16 kips. With minimum factors governing, the load modifier is $1/(\eta_D \eta_R \eta_I)$, which is 1.00 when all factors are 1.00.

If maximum factors had been used on DC by mistake, the result would be $1.25(60) - 70 = +5$ kips, apparently no uplift. The error would hide a real design problem: the bearing would need a hold-down device or the span arrangement would need to be revised.

Service check of the bearing

Bearing design uses service loads for many checks, so the same reactions would be combined at Service I with factors of 1.00: $60 + 8 + 55 = 123$ kips downward. The point of the example is that the same member needs several combinations and both factor extremes to be fully checked.

Illustrative example: comparing combinations for a pier cap

Illustrative numbers only.

A multi-column pier cap has the following unfactored moments at the face of a column (kip-ft): DC 900, DW 120, LL+IM 700, WS on the structure transmitted to the cap 60, WL 25. The cap is assumed to be conventional, redundant and typical, so $\eta = 1.00$. Wind effects in the cap are small because the cap is mainly a gravity member, which is typical.

- Strength I: $1.25(900) + 1.50(120) + 1.75(700) = 1,125 + 180 + 1,225 = 2,530$ kip-ft
- Strength III: $1.25(900) + 1.50(120) + 1.00(60) = 1,125 + 180 + 60 = 1,365$ kip-ft
- Strength V: $1.25(900) + 1.50(120) + 1.35(700) + 1.00(60) + 1.00(25) = 1,125 + 180 + 945 + 60 + 25 = 2,335$ kip-ft
- Service I: $900 + 120 + 700 + 1.00(60) + 1.00(25) = 1,805$ kip-ft

Strength I governs the cap flexure, as expected. For the columns and the foundation, the picture changes: lateral wind and braking forces create column moments that may make Strength III or Strength V govern, especially for tall piers. This is why the combination must be chosen for each force effect and each element, not once for the whole bridge.

Notice also how the 9th-edition-style factors matter. Under the older table, Strength III used $WS = 1.40$ and Strength V used $WS = 0.40$, with the design wind defined differently. Mixing a current wind pressure with an old load factor, or the reverse, gives a wrong answer. Always pair the load and the factor from the same edition.

Common mistakes and a review checklist

Common mistakes

1. **Using an outdated table.** Older references list Fatigue I as 1.50 and Fatigue II as 0.75, and WS as 1.40 in Strength III. The current values are 1.75 and 0.80 for fatigue and 1.00 for WS in Strength III, Strength V, Service I and Service IV. Check the edition of every reference, design aid and spreadsheet.
2. **Applying the maximum factor everywhere.** This can hide uplift, overturning and sliding problems.
3. **Splitting one load between two factors in the same case.** Use one factor per permanent load type per case.
4. **Forgetting construction stages.** Article 3.4.2 load factors apply, and staged construction or backfill sequence can control.
5. **Applying load modifiers to the wrong limit states.** Ductility and redundancy modifiers apply only to the strength limit state; operational importance applies to strength and extreme event limit states. Owners may narrow this further, as MnDOT does.
6. **Combining two extreme events.** The extreme event loads are applied one at a time unless the owner requires otherwise.
7. **Using $\gamma_{LL} = 1.0$ at Service III without refined losses.** Table 3.4.1-4 ties the 1.0 value to refined losses and elastic gains; other prestressed components use 0.8.
8. **Ignoring the owner's γ_{EQ} .** Extreme Event I live load is project or owner specific.
9. **Mismatching factors with resistance.** Factored loads must be compared with factored resistance from the same edition and material section.

Review checklist

- Adopted edition, interims and owner manual identified and noted in the design criteria
- Load modifiers chosen and documented, with the reason for any value other than 1.00
- All loads acting on each element listed
- Every applicable combination checked or dismissed with a written reason
- Maximum and minimum factor cases run for each permanent load
- Construction stages checked with Article 3.4.2 factors
- Service III γ_{LL} consistent with the loss method used
- Fatigue factors at current values and fatigue truck used
- Software output compared with hand checks

Why the factors are what they are

LRFD load factors were calibrated, not chosen arbitrarily. NCHRP Report 368 (Nowak, 1999) documents the calibration of the original LRFD specification. The calibration used statistical models of dead load, live load and resistance, with live load based on truck survey data, and selected factors so that the reliability index for typical bridge components would be close to a target value. The commonly cited target reliability index for the strength limit state is $\beta = 3.5$.

The higher factor on DW (1.50) than on DC (1.25) reflects the greater variability of wearing surfaces and utilities: overlays can be placed thicker than shown, or added later. The live load factor of 1.75 reflects the uncertainty of heavy trucks over a 75-year design life. The lower live load factors in Strength II (1.35) and Strength V (1.35) reflect a better-defined permit vehicle and the lower chance of maximum live load at the same time as high wind.

The service and fatigue limit states were calibrated later. The SHRP2 R19B work documented calibration of the service limit states, and its results are reflected in the current factors for fatigue and some service combinations. Practitioners do not need to repeat any of this work, but understanding it helps explain why the factors should not be changed casually and why an owner's departures from the table are made by policy rather than by individual designers.

Which combinations usually matter, member by member

The table below summarizes where each combination tends to matter for common bridge elements. It is a planning guide from general practice, not a rule from the specification. Every project must still check the combinations its owner and the specification require.

Element	Combinations that commonly govern	Notes
Deck slab (interior)	Strength I, Service I	Service I for crack control of reinforcement. Live load is from the design truck or tandem wheel loads on strips.
Deck overhang	Strength I, Extreme Event II	Extreme Event II for vehicle collision with the barrier (CT). MnDOT lists overhang design under Extreme Event II.
Steel girders	Strength I, Service II, Fatigue I/II, construction	Constructibility of noncomposite sections during deck placement is often critical.
Prestressed concrete girders	Strength I, Service I, Service III, construction (release)	Service III tension and v_{LL} from Table 3.4.1-4.
Long-span superstructures	Strength I, Strength IV	Strength IV when dead load dominates.
Bearings	Service I, Strength I (max and min)	Uplift case with minimum permanent load factors.
Pier caps	Strength I, Service I	Gravity members.
Pier columns and footings	Strength I, III, V, Extreme Event I, II	Lateral loads from wind, braking, collision, ice, water and earthquake.
Abutments and walls	Strength I (max and min), Service I, Extreme Event I, construction	Earth pressure factors from Table 3.4.1-2; backfill sequence.
Box culverts	Strength I (max and min), Service I	Opposing vertical and horizontal earth pressures, as in the MnDOT example.
Barriers	Extreme Event II	Crash-tested systems, Section 13.

Deck and superstructure notes

Decks are designed for wheel loads, and their live load comes from the design truck or design tandem rather than the full HL-93 with lane load in the approximate strip method. Superstructure girders see the full HL-93 combination of truck or tandem with lane load, distributed by the live load distribution factors covered in the next two courses. The dynamic load allowance IM is part of the live load effect and is factored with it, except that it is not applied to the design lane load or pedestrian load.

Substructure notes

Substructures collect the largest number of loads. Braking force BR, centrifugal force CE, wind on structure WS and wind on live load WL, friction FR, uniform temperature TU, shrinkage SH and creep CR all produce forces in piers and abutments, along with water load WA and the extreme event loads. Temperature, shrinkage and creep have two factors in the table for

TU: the larger value is used for deformations and the smaller for other effects. Settlement SE uses an owner or project specific factor γ_{SE} . Because so many loads are involved, substructure design is where a systematic workflow pays off the most.

Foundations and geotechnical checks

Foundation design uses the same load combinations, with resistance factors from Section 10. Service limit states control settlement and lateral movement. Strength limit states control bearing resistance, sliding and structural capacity. The overall stability EV factor of 1.00 in Table 3.4.1-2 is used for global stability, where Section 11 also gives the resistance factor applied to the slope stability analysis. Coordinate closely with the geotechnical engineer so that the loads they use for settlement and capacity match the structural combinations.

Summary

The limit state approach asks a designer to think about how a member can fail or stop performing and to check each of those conditions with factors calibrated for it. Table 3.4.1-1 tells you which loads act together and with what factors. Table 3.4.1-2 gives the range of permanent load factors that must be enveloped. The load modifiers reflect ductility, redundancy and importance, and owner policy decides many of the details. With the current edition's values, a clear list of loads, both factor extremes, and a hand check of the results, a designer can work through the combinations with confidence.

Load abbreviations used in Article 3.3.2

The tables use short designations for each load. The ones referred to in this course are listed here for quick reference; see Article 3.3.2 of the adopted edition for the complete list and exact definitions.

Permanent loads

- **DC**: dead load of structural components and nonstructural attachments
- **DW**: dead load of wearing surfaces and utilities
- **DD**: downdrag force
- **EH**: horizontal earth pressure load
- **EV**: vertical pressure from dead load of earth fill
- **ES**: earth surcharge load
- **EL**: miscellaneous locked-in force effects from the construction process, including jacking apart of cantilevers in segmental construction and secondary forces from post-tensioning
- **PS**: secondary forces from post-tensioning (in later editions)
- **CR**: creep; **SH**: shrinkage

Transient loads

- **LL**: vehicular live load; **IM**: dynamic load allowance; **PL**: pedestrian live load; **LS**: live load surcharge
- **BR**: vehicular braking force; **CE**: vehicular centrifugal force
- **WS**: wind load on structure; **WL**: wind on live load
- **WA**: water load and stream pressure
- **FR**: friction load
- **TU**: uniform temperature; **TG**: temperature gradient; **SE**: settlement
- **EQ**: earthquake; **BL**: blast loading; **IC**: ice load; **CT**: vehicular collision force; **CV**: vessel collision force

Loads that share a row in Table 3.4.1-1 (for example LL, IM, CE, BR, PL and LS) take the same factor in a given combination. When a load such as TG or SE has a project-specific factor (γ_{TG} , γ_{SE}), the specification gives guidance in Article 3.4.1, and the owner may set values by policy. Record these decisions in the design criteria with the other choices described in this course.

Key takeaways

- LRFD requires $\sum \eta_i \gamma_i Q_i \leq \phi R_n$ for every applicable limit state; the strength factors were calibrated for uniform reliability (NCHRP Report 368).
- Four limit state groups: service, fatigue and fracture, strength, and extreme event.
- Current Table 3.4.1-1: Strength I LL 1.75; Strength II and V LL 1.35; WS 1.00 wherever wind appears; Service II LL 1.30; Service III LL γ_{LL} (0.8 or 1.0); Fatigue I 1.75 and Fatigue II 0.80.
- Table 3.4.1-2: DC 1.25/0.90 (1.50 in Strength IV), DW 1.50/0.65, EH active 1.50/0.90, at-rest 1.35/0.90; use the max or min that is more critical, one value per load component.
- Load modifiers: $\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ when the maximum γ governs, and $1/(\eta_D \eta_R \eta_I) \leq 1.00$ when the minimum γ governs.
- Do not mix factors across editions: the wind and fatigue factors changed with the 8th edition.

References

1. AASHTO, LRFD Bridge Design Specifications, 9th Edition, 2020
2. AASHTO, Status of Implementation of SHRP2 Specifications to the AASHTO LRFD: Recommendations from Service Limit State Design for Bridges (SHRP2 R19B white paper)
3. Nowak, A. S., Calibration of LRFD Bridge Design Code, NCHRP Report 368, Transportation Research Board, 1999
4. Minnesota Department of Transportation, LRFD Bridge Design Manual, Section 3, Loads and Load Factors, January 2019
5. Grubb, M., Updates to the AASHTO LRFD BDS Section 6, 8th Ed. (2017) and 9th Ed. (2020), presentation hosted by South Dakota DOT, 2019

Study text only; the final quiz is taken online. Verify all references against the edition your agency has adopted.