COMPARISON OF THE AMERICAN CODE (AASHTO) AND THE SPANISH CODE (IAP AND EHE) FOR THE CALCULATIONS OF A BRIDGE DECK.

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Comparison of AASHTO and IAP/EHE for the calculations of a bridge deck

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Introduction

When you approach a comparison of two codes from to different countries you expect to find very big differences, more over when for starters the unit systems are different.

The first thing you realize when you approach the codes, Spanish and American, is that the American code is unique for roadways. In this code you find from the loads to consider to the calculations of limit states for every material you can use. The Spanish legislation is quite different; it has a code for steel and one for concrete. This two are to be used in any structure you find. For example when designing a concrete bridge in America you only need AASHTO but in Spain you need EHE for the concrete considerations and IAP for the loads on a bridge. In Spain if we wanted to do a steel bridge we will use IAP for the loads again but EAE for the steel consideration. On top of this, in the USA the design parameters can change from state to state, but always comply with AASHTO.

The second thing noticed is that the two codes consider ultimate and service states, their approaches may in some cases vary, but they are all based in the same theory. They have different division within the ultimate and the service states, but their goal is the same: combine loads in different manners so different types of failure can be checked.

Finally one thing that differs a lot from one design method to another is the use of standards. The American code and the departments of transportation use a lot of standards, pre-design elements. The Spanish code doesn’t consider these pre-design elements; normally each element is design for each construction, talking about concrete.
Reading Guidelines

First of all, the units will be supposed international units if they are not specified. For a general idea of the units, see following table with some random values:

<table>
<thead>
<tr>
<th></th>
<th>Imperial Units</th>
<th>International Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length</strong></td>
<td>3 feet</td>
<td>91.44 cm</td>
</tr>
<tr>
<td><strong>Pressure</strong></td>
<td>3600 psi</td>
<td>24.82 N/mm²</td>
</tr>
<tr>
<td><strong>Area</strong></td>
<td>3 in²</td>
<td>2000 mm²</td>
</tr>
</tbody>
</table>

Please note that to display decimal numbers a dot is used between the whole number and its decimals. A coma may be used to separate thousands units, mainly when referring to AASHTO tables.

The work presented is organized by the theoretical comparison of the design loads, ultimate states and service states. In each of these theoretical chapters you will find small examples to illustrate the differences between the codes. Finally the design of a deck and prestressed beams will be conducted with both codes.

The calculations have been done with the support of SAP2000, Microsoft Excel and Visual Basic.
Loads

The base of the design of bridges is the definition of the loads that will be used for the calculations of the different elements of the bridge. This may be the part where the different codes may differ the most. Both codes classify the loads more or less with the same philosophy: dead loads, variable loads and accidental loads.

Dead Loads

AASHTO

In the American code divides the dead loads into two big groups, one with the dead loads and another with the earth loads. The first one includes the self-weight of the structure, all utilities attached, earth cover, wearing surface, future overlays and planned widenings. For the unit weight of each element a table is given if there is no more precise information. See table 1 for the numeric values of the most used materials. The second group includes all type of earth loads, earth pressure, earth surcharge and downdrag loads.

IAP

The Spanish code divides the dead loads into two groups: the ones with permanent value and the one with variable value.

The first group, the one with permanent value, includes the self-weight and all the loads that rely on the structure, such as wearing surfaces, utilities and earth covers. For all this loads a table is given with values in case no accurate information is at disposal.

The second group, the one with variable value, includes:

- Actions that are induced to the structure before the structure is in use, such as prestressed concrete.
- Reological actions
- Effects due to settlements
- Earth Pressure
- Friction sliding bearings

Pavement

Asphalt pavement is one of the loads considered in the dead loads with permanent value. The Spanish code dictates that the maximum thickness is 10 cm; to determine the characteristic value of the asphalt we can use the value shown in table 1.

The IAP has two values for the pavement loads:

- Inferior value: the theoretical one given in the construction specifications.
- Superior value: a 50% increase of the theoretical values.
Comparison

We can see both codes separate the dead loads in similar ways. In the first group, both codes consider the same, self-weight and any load that relies in the structure, with the following values for the most used materials:

<table>
<thead>
<tr>
<th>Material</th>
<th>AASTHO IS (kN/m³)</th>
<th>AASTHO US (kcf)</th>
<th>IAP IS (kN/m³)</th>
<th>IAP US (kcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum</td>
<td>27.5</td>
<td>0.175</td>
<td>27.0</td>
<td>0.172</td>
</tr>
<tr>
<td>Bituminous Wearing Surface</td>
<td>22.0</td>
<td>0.140</td>
<td>23.0</td>
<td>0.146</td>
</tr>
<tr>
<td>Cast Iron</td>
<td>70.7</td>
<td>0.450</td>
<td>72.5</td>
<td>0.462</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal weigh (f′c &lt; 5 ksi)</td>
<td>22.8</td>
<td>0.145</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal weigh (5 ksi &lt; f′c &lt; 15 ksi)</td>
<td>22 + 0.157f′c</td>
<td>0.140 + 0.001f′c</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No reinforcement</td>
<td>23.0 to 24.0</td>
<td>0.146 to 0.153</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforced or prestressed</td>
<td>25.0</td>
<td>0.159</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rolled Gravel, Macadam or Ballast</td>
<td>22.0</td>
<td>0.140</td>
<td>20.0</td>
<td>0.127</td>
</tr>
<tr>
<td>Steel</td>
<td>77.0</td>
<td>0.490</td>
<td>78.5</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Table 1. Self-weight typical values.

We can see that the value do not differ a lot from one code to the other. In concrete we can see that the typical value are considered in different ways, but the two have more or less the same value.

We can anticipate that the self-weight will not differ much from one code to another.

The second group of loads considered is not quite the same. In the Spanish code these are loads that always rely on the structure but do not have the same value in the lifespan of the structure. In AASHTO these loads are mainly earth loads.

The American code has no special considerations for the pavement as the Spanish one. The latter considers a possible increase of the pavement.
Live Loads: Traffic loads

AASTHO

Design lanes must have a width of \( w/12 \) (ft), with \( w \) being the width of the bridge. If the roadway width is from 20 to 24 ft, two design lanes shall be considered, each equal to one-half of the roadway width. The live loads considered are a combination of the design truck or tandem and the design lane load.

Design truck

The weights and spacing of axles and wheels for the design truck are shown in the following figure. The spacing between axles (distance \( Y \) in the figure) shall be varied between 14 ft and 30 ft to produce extreme force effects.

![](image)

Figure 1. Design truck. AASHTO.

Design tandem

The design tandem shall consist of a pair of 25 kip axles spaced 4 ft and 6 ft transversally.

Multiple presence factors

In ASSHTO when more than one line is considered factors for the trucks and tandem have to be consider. For example if 3 lanes are considered we have to multiply each tandem by the factor of 3 lanes, the same for the trucks and see which gives us a worst case. These factors are the following:

<table>
<thead>
<tr>
<th>Number of loaded lanes</th>
<th>Multiple presence factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>0.85</td>
</tr>
<tr>
<td>&gt; 3</td>
<td>0.65</td>
</tr>
</tbody>
</table>
Design lane load

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. In the transverse direction it will be distributed over 10 ft width.

The extreme force effect shall be taken as the larger of the truck or tandem both combined with the design load lane.
IAP

The design width of the lanes will follow the following criteria, being \( w \) the width in meters:

- When \( w < 5.4 \) m, one lane of 3 m and the rest will be considered shoulder.
- When \( 5.4 \leq w < 6 \) m, two lanes with a width of \( w/2 \).
- When \( w \leq 6 \) m, the number of lanes will be giving by rounding down \( w/3 \), the rest of the width that is not a lane will be considered shoulder.

**Design truck**

Three design trucks are considered, each of this truck has the same load in both axes and this axes are separated by 1.2 m. In the same axe the distance between loads is 2 m, transversally. The three trucks considered have 300 kN, 200 kN and 100 kN loads in each axe.

**Design lane load**

Two loads will be considered: one of 2.5 kN/m\(^2\) applied in the whole deck and one of 9 kN/m\(^2\) applied only in one of the lanes. The larger load will be applied in the lane with the worst response. The larger truck will go in the lane that produces the worst response (the one with the larger distributed load) and so on with the other two.

We can see the combination of distributed load and trucks in the following table.

<table>
<thead>
<tr>
<th>Situation</th>
<th>Truck (kN)</th>
<th>Distributed Load (kN/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane 1</td>
<td>2 \cdot 300</td>
<td>9</td>
</tr>
<tr>
<td>Lane 2</td>
<td>2 \cdot 200</td>
<td>2.5</td>
</tr>
<tr>
<td>Lane 3</td>
<td>2 \cdot 100</td>
<td>2.5</td>
</tr>
<tr>
<td>Other lanes</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>Shoulders</td>
<td>0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 2. Traffic loads IAP.
Comparison

The first thing that makes a difference between the two codes is the width of the design lanes: in AASTHO is 3.66 m (12 ft) and in IAP is 3 m (9.84 ft). This may lead that in some bridges in the Spanish code have more lanes that with the American code.

Trucks

Secondly, the trucks are completely different. AASHTO considers two types of trucks that are applied separately and IAP considers the same truck with different loads that are applied at the same time in different lanes. The trucks differ also in the loads and the distance between axles.

<table>
<thead>
<tr>
<th></th>
<th>Distance between axles.</th>
<th>Distance between axles.</th>
<th>Loads of each axe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transversal</td>
<td>Longitudinal</td>
<td></td>
</tr>
<tr>
<td>AASHTO Truck</td>
<td>1.8 m (6 ft)</td>
<td>4.3 m (14 ft)</td>
<td>35.6 kN (8 kip)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.3 – 9.1 m (14 -30 ft)</td>
<td>2 ⋅ 142.3 kN (2 ⋅ 32 kip)</td>
</tr>
<tr>
<td>AASHTO Tandem</td>
<td>1.8 m (6 ft)</td>
<td>1.23 m (4 ft)</td>
<td>111.2 kN (25 kip)</td>
</tr>
<tr>
<td>IAP Truck</td>
<td>2 m (6.6 ft)</td>
<td>1.2 m (3.94 ft)</td>
<td>2 ⋅ 100 / 200 / 300 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(2 ⋅ 22.5 / 45 / 67.4 kip)</td>
</tr>
</tbody>
</table>

Table 3. Comparison of design truck.

Lane loads

While the American code considers only one type of load for the whole lanes applied only in 10 ft out of the 12 ft of the lane. The load applied by AASHTO is 3.06 kN/m² (0.64 klf / 10 ft). In the other hand the IAP considers to types of loads, one for the lane with the worst response and one for the other lanes: 9 kN/m² (0.19 ksf) and 2.5 kN/m² (0.05 ksf) respectively.

We can see that the differences in the traffic loads are bigger that in the dead loads. Clearly this can lead us to completely different results, but we cannot say witch of the two will lead to more conservative results. It may happen that the most conservative changes from bridge to bridge.
Combination Factors

AASHTO

The total factored force effect shall be taken as:

\[ Q = \sum \eta_i \gamma_i Q_i \]  

\( \eta_i \) = load modifier
\( Q_i \) = force effects
\( \gamma_i \) = load factor

The American code specifies the following limit states for the applicable combination of factored extreme force effects:

- **Strength I**: Basic load combination relating to the normal vehicular use of the bridge without wind.
- **Strength II**: Load combination relating to the use of the bridge by owner specified special design vehicle, evaluation permit vehicles, or both without bridge.
- **Strength III**: Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.
- **Strength IV**: Load combination relating to very high dead load to live load force effect ratios.
- **Strength V**: Load combination relating to normal vehicular use if the bridge with wind of 55 mph velocity.
- **Extreme Event I**: Load combination including earthquake.
- **Extreme Event II**: Load combination relating to ice load, collision by vessels and vehicle, check floods and certain hydraulic events with reduced live load other than that which is part of the vehicular collision load.
- **Service I**: Load combination relation to the normal operational use of the bridge with a 55 mph wind and all the loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for investigation of slope stability.
- **Service II**: Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live loads.
- **Service III**: Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.
- **Service IV**: Load combination relating only to tension in prestressed concrete columns with the objective of crack control.
- **Fatigue I**: Fatigue and fracture load combination related to infinite load-induced fatigue life.
• **Fatigue II**: Fatigue and fracture load combination related to finite load-induced fatigue life.

_Load modifier (Article 1.3.2.)_

The load modifiers depend on the values of the load factors. If a maximum value of $\gamma_i$ is considered:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$$

(2)

If a minimum value of $\gamma_i$ is considered:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$$

(3)

$\eta_D$ = factor related to ductility. For the **strength limit state**:

- $\eta_D \geq 1.05$ for non-ductile components
- $\eta_D = 1.00$ for conventional designs and details complying with these Specifications
- $\eta_D \leq 0.95$ for components and connections for which additional ductility-enhancing measures are needed.

For all **other limit states** $\eta_D = 1.00$

$\eta_R$ = factor related to redundancy. For the **strength limit state**:

- $\eta_R \geq 1.05$ for non-redundant members
- $\eta_R = 1.00$ for conventional levels of redundancy.
- $\eta_R \leq 0.95$ for exceptional levels of redundancy beyond girder continuity and a torsionally-closed cross section.

For all **other limit states** $\eta_R = 1.00$

$\eta_I$ = factor related to operational classification. For the **strength limit state**:

- $\eta_I \geq 1.05$ for critical or essential bridges
- $\eta_I = 1.00$ for typical bridges.
- $\eta_I \geq 0.95$ for relatively less important bridges

For all **other limit states** $\eta_I = 1.00$

_Load Factors_

The load factors for various loads comprising a design load combination shall be taken as specified in the following table. The factors shall be selected to produce the total extreme factored force effect. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. In the following table we can see some of the factors used for some of the limit states.
表 4. 荷载因数。AASHTO.

<table>
<thead>
<tr>
<th>荷载组合限界状态</th>
<th>DC</th>
<th>DW</th>
<th>LL</th>
<th>LS</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC: 组件和附件</td>
<td>γ_p</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>服务 I</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>服务 III</td>
<td>1.0</td>
<td></td>
<td>0.80</td>
<td></td>
</tr>
</tbody>
</table>

Where,
- DC  死载为结构组件和非结构附件的重量
- DW  死载为表面和公用设施的重量
- PS  后张力的次级力
- LL  车辆活载
- LS  活载荷载

对于 γ_p，我们应该使用以下表格：

表 5. 根据 AASHTO 的负载因数

<table>
<thead>
<tr>
<th>荷载类型</th>
<th>最大值</th>
<th>最小值</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC: 组件和附件</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>DW</td>
<td>1.50</td>
<td>0.65</td>
</tr>
</tbody>
</table>

对于后张力效果，我们应该取 γ_p = 1.0 对于所有混凝土支撑结构和支撑结构，以及对于钢支撑结构。我们应该使用 γ_p = 0.5 如果使用 L_k 和 1.0 如果使用 L_{effective}。
The Spanish code uses two types of combination factors, one set of coefficients to obtain the calculation value of the loads and another set of coefficients to take into account the possibility of more than one load at the same time.

**Combination factors**

There are three types of combination factors for the live loads ($Q_k$):

- **Combination value** $\psi_0 Q_k$: this will be the value of the load when more than one live load is considered at the same time. This value will be used in when checking ultimate limit state in a persistent or transitory situation. It will be also use to check service limit state of irreversible states.
- **Frequent value** $\psi_1 Q_k$: this will be the value of the load that is exceeded in a short term of time respect to the lifespan of the bridge. This value will be used when checking accidental ultimate limit states and reversible service limit state.
- **Quasi-permanent value** $\psi_2 Q_k$: this will be the value of the action that is surpassed a great part of the lifespan of the bridge. This value will be used to check accidental ultimate limit states, reversible service limit state and the evaluation of differed effects.

The combination factors vary also in the type of load that they are applied to. In the following table the factors for the live loads are shown:

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>$\psi_0$</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Live Load</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical loads</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trucks</td>
<td>0.75</td>
<td>0.75</td>
<td>0</td>
</tr>
<tr>
<td>Distributed load</td>
<td>0.4</td>
<td>0.4</td>
<td>0/0.2$^{(1)}$</td>
</tr>
<tr>
<td>Sidewalk load</td>
<td>0.4</td>
<td>0.4</td>
<td>0</td>
</tr>
</tbody>
</table>

*(1) It will be considered 0 for all calculation except when seismic considerations are taken.*

**Calculation value for Ultimate Limit State**

In the ultimate limit state the Spanish code considers 3 situations: equilibrium, resistance and fatigue.

For equilibrium consideration two possible effects will be used for each type of load: one that stabilizes the loads and one that destabilizes. For equilibrium the following coefficients will be used:

\[
\begin{align*}
\psi_0 & = 0.75 \\
\psi_1 & = 0.75 \\
\psi_2 & = 0
\end{align*}
\]
<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Effect</th>
<th>Stabilizer</th>
<th>Destabilizer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent loads</td>
<td>Self weight</td>
<td>0.9</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Dead Loads</td>
<td>0.9</td>
<td>1.1</td>
</tr>
<tr>
<td>Variable Loads</td>
<td>Traffic</td>
<td>0</td>
<td>1.35</td>
</tr>
</tbody>
</table>

*Table 7. Factors for equilibrium ultimate limit state.*

For resistance we have also two types of coefficients, pro and adverse factors:

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Effect</th>
<th>Pro</th>
<th>Adverse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent loads</td>
<td>Self weight</td>
<td>1.0</td>
<td>1.35</td>
</tr>
<tr>
<td>of constant value</td>
<td>Dead Loads</td>
<td>1.0</td>
<td>1.35</td>
</tr>
<tr>
<td>Permanent loads</td>
<td>Prestressed Forces</td>
<td>1.0</td>
<td>1.35</td>
</tr>
<tr>
<td>with no constant</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>value</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Live Loads</td>
<td>Traffic</td>
<td>0</td>
<td>1.35</td>
</tr>
</tbody>
</table>

*Table 8. Factors for resistance ultimate limit state.*

As for the fatigue of the materials, as it is completely dependent on the material used, the considerations are in the material codes. For concrete it will be EHE and for steel EAE.

For pavement loads as we have to consider two values, the inferior value will be multiplied with $\gamma_G = 1.0$ and the superior one with $\gamma_G = 1.35$.

Then, the final combination of loads may be written as:

$$\sum_{j=1}^n \gamma_G jG_j + \sum_{m=1}^m \gamma_G mG^*_m + \gamma Q_1 + \sum_{i>1}^i \psi_0, iQ_j Q_j$$  \hspace{1cm} (4)

Where,

- $G$ characteristic value of permanent loads
- $G^*$ characteristic value of permanent loads with no constant value.
- $Q_1$ characteristic value of the dominant live load
- $Q_j$ characteristic value of all other live load considered
- $\Psi_0$ coefficient of combination. [Table 5]
- $\gamma_G, \gamma_Q$ coefficients in tables 6 and 7
Calculation value for Service Limit State

For the service limit state only one set of coefficients is considered. It also has its pro and adverse coefficients.

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Effect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pro</td>
</tr>
<tr>
<td>Permanent loads of constant value</td>
<td>Self weight 1.0</td>
</tr>
<tr>
<td></td>
<td>Dead Loads   1.0</td>
</tr>
<tr>
<td>Permanent loads with no constant value</td>
<td>Prestressed Forces</td>
</tr>
<tr>
<td>Live Loads</td>
<td>Traffic</td>
</tr>
</tbody>
</table>

Table 9. Factors for service limit state.

In the service limit state we have three possible combinations of actions:

- Characteristic combination:

\[
\sum_{j=1}^{1} G_{G,j} G_j + \sum_{m \geq 1} G_{G,m} G^*_m + Q_{Q,1} Q_1 + \sum_{i>1} \Psi_0,1 Q_{Q,j} Q_j
\]

(5)

This combination is used to check the service limit state of irreversible actions.

- Frequent combination:

\[
\sum_{j=1}^{1} G_{G,j} G_j + \sum_{m \geq 1} G_{G,m} G^*_m + Q_{Q,1} Q_1 + \sum_{i>1} \Psi_2,1 Q_{Q,j} Q_j
\]

(6)

This one is used to check the reversible service limit state.

- Quasi-permanent combination:

\[
\sum_{j=1}^{1} G_{G,j} G_j + \sum_{m \geq 1} G_{G,m} G^*_m + \sum_{i \geq 1} \Psi_2,1 Q_{Q,j} Q_j
\]

(7)

This combination is used to check certain reversible service limit state and to evaluate differed in time effects.

Where,

- \(G\)  characteristic value of permanent loads
- \(G^*\)  characteristic value of permanent loads with no constant value.
- \(Q_1\)  characteristic value of the dominant live load
- \(Q_j\)  characteristic value of all other love load considered
- \(\Psi_0, \Psi_1, \Psi_2\)  coefficients of combination. [Table 5]
- \(G_0, Q_0\)  coefficients in table 8
Ultimate Limit States

EHE

The main concept of the Spanish code is that the structural capacity should be higher than the structural need. In the Spanish code the structural need for bridges is determined by another code IAP, which stands for code of the actions to be considered in roadway bridges.

Equilibrium Limit State (Art. 41)

This limit state obliges to check that under the worst load case the basics of equilibrium of structures are not violated. It also mentions that the check must be done for the final configuration and for all construction cases that may happen.

Strength Limit State

In the EHE the factors are applied to the materials, for high level of quality control:

- Concrete $\gamma_c = 1.4$
- Steel $\gamma_s = 1.1$

Then the strength of the materials in the calculations will be:

- Concrete: $f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{f_{ck}}{1.4}$
- Steel: $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{f_{yk}}{1.1}$

The characteristic strengths of the materials vary for code to code, but this variation is small. This variation is only due unit changes in order to have round numbers.

Once the factors are applied to the materials the calculations of the capacity are done without any other factor. We will compare the moment capacity to the ultimate moment computed before.

Concrete Strength

The stress-strain curve used in the EHE can be the parabolic one, figure 1:

$$\sigma_c = f_{cd} \left[ 1 - \left( 1 - \frac{\varepsilon_c}{\varepsilon_{c0}} \right)^n \right] \text{ if } 0 \leq \varepsilon_c \leq \varepsilon_{c0} \quad (8)$$

$$\sigma_c = f_{cd} \quad \text{ if } \varepsilon_{c0} \leq \varepsilon_c \leq \varepsilon_{cu} \quad (9)$$

![Figure 2. Strain-stress diagram used in EHE](image)
Where,

- \( \varepsilon_c \) is the relative deformation of the concrete
- \( \varepsilon_{c0} \) is the deformation by simple compression break:
  \[
  \varepsilon_{c0} = 0.002 \quad \text{if } f_{ck} \leq 50 \frac{N}{mm^2}
  \]
  \[
  \varepsilon_{c0} = 0.002 + 0.000085(f_{ck} - 50)^{0.5} \quad \text{if } f_{ck} > 50 \frac{N}{mm^2}
  \]
- \( \varepsilon_{cu} \) is the ultimate deformation
  \[
  \varepsilon_{cu} = 0.0035 \quad \text{if } f_{ck} \leq 50 \frac{N}{mm^2}
  \]
  \[
  \varepsilon_{c0} = 0.0026 + 0.0144 \left(\frac{100-f_{ck}}{100}\right)^4 \quad \text{if } f_{ck} > 50 \frac{N}{mm^2}
  \]
- \( n \) defines the grade of the parabola considered
  \[
  n = 2 \quad \text{if } f_{ck} \leq 50 \frac{N}{mm^2}
  \]
  \[
  n = 1.4 + 9.6 \left[\frac{100-f_{ck}}{100}\right]^4 \quad \text{if } f_{ck} > 50 \frac{N}{mm^2}
  \]

For the average tensile strength, \( f_{ct,m} \) of concrete for the cracking control:

\[
 f_{ct,m} = 0.3 f_{ck}^{2/3} \quad \text{if } f_{ck} \leq 50 \frac{N}{mm^2}
\]
\[
 f_{ct,m} = 0.58 f_{ck}^{1/2} \quad \text{if } f_{ck} > 50 \frac{N}{mm^2}
\]

If lab results are not possible, for the characteristic tensile strength it should be used:

\[
 f_{ct,k} = 0.7 \cdot f_{ct,m}
\]

The average resistance to flexotraction:

\[
 f_{ct,m,ft} = max \left\{ \left(1.6 - \frac{h}{1000}\right)f_{ct,m}, f_{ct,m} \right\}
\]
Where \( h \) is the depth of the element in mm.

Finally the concrete elasticity modules is computed as:

\[
 E_{cm} = 8500 \sqrt{f_{cm}}, \text{ where } f_{cm} = f_{ck} + 8
\]

For instantaneous loads or rapidly varying, it should be taken:

\[
 E_c = \beta E_{cm}, \text{ where } \beta_E = 1.3 - \frac{f_{ck}}{400} \leq 1.175
\]
**Steel Strength**

For steel the diagram stress-strain used in EHE has the following shape:

![Stress-Strain diagram for reinforcing steel EHE.](image)

The Spanish code uses an elasticity modulus of 200.000 N/mm² and a maximum deformation of \( \varepsilon_{\text{max}} = 0.01 \). For general uses it is sufficient to use a straight line once the elastic limit is reached.

**Prestressed Steel Strands**

For the prestressed steel the EHE establishes the following diagram, with a first straight line and a second curved part. The straight line follows a slope of \( E_p = 190.000 \) N/mm². The curved part follows the next equation:

\[
\varepsilon_p = \frac{\sigma_p}{E_p} + 0.823 \left( \frac{\sigma_p}{f_{pk}} - 0.7 \right)^5 \quad \text{for} \quad \sigma_p \geq 0.7f_{pk}
\]

![Stress-strain diagram for prestressed steel. EHE.](image)
**Capacity of the Section**

As mentioned before the structural capacity must be higher than the structural needs. For this 5 deformation domains are established, depending with fibers are compressed, tensioned, if the steel has reached the yielding point... These domain can be represented in the following figure:

![Deformation domains defined in EHE](image)

**Figure 5. Deformation domains defined in EHE**

Where the deformations are the ones established in previous points and \( d \) is the distance between the tensile reinforcement or less compressed and the most compressed one.

- Domain 1: when all the section is in traction
- Domain 2: when the concrete does not reach breakage by flexure
- Domain 3: when the section is in flexion and the concrete reaches breakage by flexure at the top, \( \varepsilon_{cu} \).
- Domain 4: when the section is in flexion and the most tensioned steel fiber is between 0 and the yielding point
- Domain 4a: when all the steel is in compression but there is a little portion of concrete in tension
- Domain 5: when all the section is in compression

With these domains defined we can write the equilibrium equations.

**Design**

To illustrate the procedure and calculus applied in the EHE it will be done with an example later on.
**AASHTO**

**Strength Limit State**

In the American code the basics say that the factored load considered should be smaller that the structural capacity times the $\phi$ factor. This factor varies depending on the type of load studied:

- For tension-controlled reinforced concrete sections (Article 5.7.2.1): 0.90
- For tension-controlled prestressed concrete sections (Article 5.7.2.1): 1.00
- For shear and torsion:
  - Normal weight concrete 0.90
  - Lightweight concrete 0.80
- For compression-controlled sections with spirals or ties: (Article 5.7.2.1, except as specified in Articles 5.10.11.3 and 5.10.11.4.1b for Seismic Zones 2, 3, and 4 at the extreme event limit state) 0.75
- For bearing on concrete 0.70
- For compression in strut-and-tie models 0.70

For the transition between the tension controlled and compression controlled sections the $\phi$ factor may be computed for prestressed members as:

$$0.75 \leq \phi = 0.583 + 0.25 \left( \frac{d_t}{c} - 1 \right) \leq 1.0$$

and for nonprestressed members as:

$$0.75 \leq \phi = 0.65 + 0.15 \left( \frac{d_t}{c} - 1 \right) \leq 0.9$$

This may be represented in the following figure:

![Figure 6. $\phi$ factor transition diagram. AASHTO.](image)

**Concrete Strength**

In the American code the stress-strain diagram is not defined, but the parabolic one it is common used. The ultimate deformation considered must be $\varepsilon_{cu}$. 
For normal-weight concrete with specified compressive strengths up to 10 ksi, the direct tensile strength may be estimated as:

\[ f_r = 0.23 \cdot \sqrt{f'_c} \text{ (ksi)} \]

The Elasticity modulus for concretes with unit weights between 0.09 and 0.155 kcf and compressive strengths up to 15.0 ksi:

\[ E_c = 33000 \cdot K_1 \cdot w_c^{1.5} \sqrt{f'_c} \]

Where,

- \( K_1 \) Correction factor for source aggregate.
- \( w_c \) unit weight of concrete (kcf)
- \( f'_c \) compressive strength of concrete (ksi)

For normal weight concrete with \( w_c = 0.145 \) kcf may be taken as:

\[ E_c = 1820\sqrt{f'_c} \]

**Steel Strength**

The modulus of elasticity for the reinforcement shall be assumed as 29000 ksi. For prestressed steel if no more precise data is available should be taken as:

- For strands \( E_p = 28500 \) ksi
- For bars \( E_p = 30000 \) ksi

**Capacity of the Section**

In the American code the section equilibrium is based in the following deformation diagram:

![Figure 7. Equilibrium of the cross-section. AASHTO](image)

Where the ultimate deformation of concrete is \( \varepsilon_{cu} = 0.003 \) and for steel the strain limit for a compression controlled section is \( \varepsilon_s = 0.002 \). For the tension controlled section the strain limit for the steel may be set at \( \varepsilon_s = 0.005 \).

**Design**

To illustrate the procedure and calculus applied in the AASHTO it will be done with an example later on.
Comparison

One of the main differences between the EHE and AASHTO is that the Spanish codes applies a factor to the strength of the material and then computes the capacity of the section without other factors. In the other hand, AASHTO with values of strength of the materials you compute the capacity of the section and then applied the resistance factor, commonly known as the \( \phi \) factor.

In concrete compression strength there is a minor difference in the control of \( f_c \). In the Spanish code when we say that we are using a 30 N/mm\(^2\) concrete this value is a characteristic strength based on a statistical analysis of the cylinders taken in site. In the American code went said that a 4 ksi concrete is used all the strength results should be above this.

For purpose of the study we will consider both codes use the same strength value.

The elasticity modulus for concrete varies lightly for one code to another. If we consider the same compression strength for each code: \( f'_c = 4 \text{ ksi} = f_{ck} = 27.58 \frac{N}{mm^2} \), then the elasticity modulus are:

\[
\begin{align*}
\text{AASHTO: } E_c &= 1820 \sqrt{f'_c} = 25097 \frac{N}{mm^2} = 3640 \text{ ksi} \\
\text{EHE: } E_c &= 8500^2 \sqrt{f_{ck}} + 8 = 27956 \frac{N}{mm^2} = 4055 \text{ ksi}
\end{align*}
\]

The elasticity modulus of the reinforcement:

\[
\begin{align*}
\text{AASHTO: } E_s &= 199947 \frac{N}{mm^2} = 29000 \text{ ksi} \\
\text{EHE: } E_s &= 200000 \frac{N}{mm^2} = 29008 \text{ ksi}
\end{align*}
\]

There is almost no variation, for comparison purposes we will use \( E_s = 200000 \frac{N}{mm^2} \).

For the prestressed strands, the elasticity moduli are:

\[
\begin{align*}
\text{AASHTO: } E_p &= 196501 \frac{N}{mm^2} = 28500 \text{ ksi} \\
\text{EHE: } E_p &= 190000 \frac{N}{mm^2} = 27557 \text{ ksi}
\end{align*}
\]

Again for purposes of comparison we will use \( E_p = 190000 \frac{N}{mm^2} \).

Comparison of the Strength Ultimate Limit State, Flexure. (No prestressed)

We have above mentioned that the comparison of the calculation procedure would be done with an example. We are going to use a T beam with a flange of 2m long and 0.2m depth. The total depth of the beam is 1.2m and the web is 0.3m wide. From now on we will use the international unit system if not specified otherwise.

We will use a concrete with \( f_{ck} = 35 \frac{N}{mm^2} \) and a reinforcement of \( f_{ck} = 420 \frac{N}{mm^2} \).
We will use 3 layers of reinforcement with the following areas and distance to bottom:

\[ d_1 = 70\text{mm}, \quad d_2 = 300\text{mm}, \quad d_3 = 600\text{mm} \]

\[ A_{s1} = 490\text{mm}^2, \quad A_{s2} = 490\text{mm}^2, \quad A_{s3} = 400\text{mm}^2 \]

Figure 8. Cross Section used. All dimension in mm.

**EHE**

We will assume an intense quality control:

- Concrete: \( f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{35}{1.4} = 25 \frac{N}{\text{mm}^2} \)
- Steel: \( f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.1} = 363.63 \frac{N}{\text{mm}^2} \)

In order to obtain the resisting moment of the section, for the area of steel given we have computed for each possible deformation, as shown in figure 4, until we find the one that requires no axial force for the equilibrium. This means going thru every domain, varying the limit strain of the concrete and the steel.

For the example above, we would obtain the following interaction diagram:

Figure 9. Interaction diagram for EHE, in m ton (metric).
For the American code the idea of how to get capacity of the section is slightly different. Leaving apart the fact of the safety factors applied to the moment and not the material, the American code fixes the deformation of the concrete and moves only the deformation of the steel. This gives us a different interaction diagram:

![Interaction diagram for AASHTO, in m ton (metric).](image)

The pointed part of the diagram is due to the change of the $f_i$ factored, from a tensioned controlled section to a compression-controlled section. This means that when the section is tensioned controlled the $f_i$ factor is 0.9 and when the section is compression controlled the $f_i$ factor changes to 0.75. The change is not direct, it goes thru a phase where it evolves from one values to the other.
In order to better compare the two codes we will compute as shown above the capacity of this cross section with different areas of rebar. If we do this we obtain the following table:

<table>
<thead>
<tr>
<th>Φ AASHTO</th>
<th>Area rebar at the bottom layer (cm²)</th>
<th>φ Mr AASHTO</th>
<th>Mr EHE</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.900</td>
<td>4.9</td>
<td>451.75</td>
<td>448.81</td>
</tr>
<tr>
<td>0.900</td>
<td>10</td>
<td>663.16</td>
<td>656.63</td>
</tr>
<tr>
<td>0.900</td>
<td>15</td>
<td>866.91</td>
<td>859.81</td>
</tr>
<tr>
<td>0.900</td>
<td>20</td>
<td>1069.78</td>
<td>1062.42</td>
</tr>
<tr>
<td>0.900</td>
<td>25</td>
<td>1272.65</td>
<td>1264.22</td>
</tr>
<tr>
<td>0.900</td>
<td>30</td>
<td>1475.42</td>
<td>1465.35</td>
</tr>
<tr>
<td>0.900</td>
<td>35</td>
<td>1678.29</td>
<td>1666.02</td>
</tr>
<tr>
<td>0.900</td>
<td>40</td>
<td>1881.17</td>
<td>1865.36</td>
</tr>
<tr>
<td>0.900</td>
<td>50</td>
<td>2283.00</td>
<td>2263.36</td>
</tr>
<tr>
<td>0.900</td>
<td>75</td>
<td>3278.74</td>
<td>3248.10</td>
</tr>
<tr>
<td>0.900</td>
<td>100</td>
<td>4257.54</td>
<td>4219.00</td>
</tr>
<tr>
<td>0.900</td>
<td>125</td>
<td>5222.50</td>
<td>5176.84</td>
</tr>
<tr>
<td>0.900</td>
<td>150</td>
<td>6174.15</td>
<td>6119.82</td>
</tr>
<tr>
<td>0.900</td>
<td>175</td>
<td>7111.62</td>
<td>7048.64</td>
</tr>
<tr>
<td>0.900</td>
<td>200</td>
<td>8034.39</td>
<td>7966.12</td>
</tr>
<tr>
<td>0.900</td>
<td>300</td>
<td>11599.02</td>
<td>11140.38</td>
</tr>
<tr>
<td>0.776</td>
<td>400</td>
<td>12131.61</td>
<td>11833.03</td>
</tr>
<tr>
<td>0.750</td>
<td>500</td>
<td>12114.73</td>
<td>11972.91</td>
</tr>
</tbody>
</table>

Table 10. Capacities in AASHTO and EHE for different areas of rebar. (kN m)

![Diagram](image)

Figure 11. Representation of Capacities for different areas of rebar.

We can see that the two different approaches give similar results. Results vary when increasing the rebar, mainly due to the change of the fi factor.
**Service Limit States**

**Cracking service limit state**

**EHE**

The main thing checked by this limit state is that the characteristic crack is smaller than the maximum allowed, under the service combination of loads:

\[ w_k \leq w_{max} \]

The maximum crack allowed depends on the type of exposure:

<table>
<thead>
<tr>
<th>Class of exposure</th>
<th>Reinforced Steel</th>
<th>Prestressed Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>IIa, IIb, H</td>
<td>0.3</td>
<td>0.2</td>
</tr>
<tr>
<td>IIIa, IIIb, IV, F, Qa</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>IIIc, Qb, Qc</td>
<td>0.1</td>
<td>Decompression</td>
</tr>
</tbody>
</table>

*Table 11. Maximum cracks allowed by EHE*

When checking for reinforced steel we will use the quasi-permanent combination of loads and for prestressed we will use the frequent combination of loads.

Two main types of cracks are considered: the ones caused by compression and the ones caused by traction. The Spanish code mentions cracks by shear and torsion but says that in general if the ultimate limit states of shear and torsion are met, there should be no problem with the cracks caused by these stresses.

For cracks caused by compression the EHE makes the following restriction for the worst combination of service loads:

\[ \sigma_c \leq 0.60 \cdot f_{ck,j} \]

Where,

- \( \sigma_c \) is the stress of compression being checked
- \( f_{ck,j} \) is the characteristic resistance of the concrete at \( j \) days.
The stress of compression is computed from the following equilibrium for a T beam:

\[
\sigma_c = \frac{Mx}{I_f}, \text{ where } M \text{ is the moment considered depending on the reinforcement and } I_f \text{ inertia of the cracked section.}
\]

The Spanish code says that tensile cracks only have to be checked if the stress of the most tensioned fiber is larger than the resistance of the concrete to flexotraction. This resistance is considered in Art. 39.1 and has been previously mentioned in ULS:

\[
f_{ct,m,fi} = \max \left\{ \left(1.6 - \frac{h}{1000}\right)f_{ct,m} ; f_{ct,m} \right\} \text{ Where } h \text{ is the depth of the element in mm.}
\]

Where,

\[
f_{ct,m} = \begin{cases} 0.3f_{ck}^{2/3} & \text{if } f_{ck} \leq 50 \frac{N}{\text{mm}^2} \\ 0.58f_{ck}^{1/2} & \text{if } f_{ck} > 50 \frac{N}{\text{mm}^2} \end{cases}
\]

If the section has cracks by tensile stresses we will compute this with the following:

\[
w_k = \beta s_m \varepsilon_{sm}
\]

\(\beta\) relates the average value with the characteristic depending on the type of load

\(s_m\) is the average separation between cracks

\(\varepsilon_{sm}\) is the average strain of the reinforcement

**AASHTO**

The American code specifies that cracking should be check if the tension on the cross section exceeds 80% of the modulus of rupture. This modulus of rupture, \(f_r\), is defined as:

For normal weight concrete

Used to calculate the cracking moment of a member \(\text{(Art 5.8.3.4.3)}\)

\[0.2\sqrt{f_c^r} \]

Otherwise

\[0.24\sqrt{f_c^r} \]
AASHTO limits cracking by limiting the distance between reinforcement steel in the layer closest to the tension face. This distance shall satisfy:

\[ s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \]  \hspace{1cm} (10)

in which \( \beta_s = 1 + \frac{d_c}{0.7(h-d_c)} \)

Where,

- \( \gamma_e \): exposure factor
- \( d_c \): thickness of the concrete cover from extreme tension fiber to center of flexural reinforcement (in)
- \( f_{ss} \): tensile stress in steel reinforcement at service limit state (ksi)
- \( h \): overall thickness or depth of the element (in)

This equation is based on a physical crack model (Frosch, 2001). The equation is based on an assumed crack width of 0.017 in with a class I exposure. The crack width is directly proportional to the \( \gamma_e \). The code doesn’t directly limit the crack width but it is indirectly controlled by this factor. For example a \( \gamma_e \) factor of 0.5 will result in an approximate crack width of 0.0085 in.

**Comparison**

The approach of the cracking control is conceptually different. The Spanish code approaches the idea by putting a limit to the theoretical characteristic crack that will appear in an element. While the American code limits the separation between longitudinal reinforcement, this will limit the width of the crack. While the concept is different both codes limit directly or indirectly the width of the crack.

**Example**

To better illustrate the differences between the two codes a simple example will be evaluated.

The section studied will be a T section with the following dimensions:

![Figure 13. T Section for Comparison Example](image-url)
We will use the following materials for both codes:

\[ f_{ck} = 35 \text{ MPa} \]
\[ E_c = 30 \text{ MPa} \]
\[ f_s = 420 \text{ MPa} \]

In order to compare the cracking calculations of both codes, we are going to calculate for both the crack width for bending moments above the cracking moment of the section. We will consider the tensile strength 0 for concrete for the different bending moments.

For the stresses calculations of steel, the following bending moment – curvature diagram will be used:

![Bending Moment vs Curvature](image)

**Figure 14. Bending Moment vs. Curvature**

**EHE**

Tensile strength: \( f_{ct,m} = 0.3 f_{ck}^{2/3} = 3.2 \text{MPa} \). The cracking bending moment is computed in the Spanish code as:

\[ M_{crack} = \frac{I}{y_{cdg}} f_{ct,m} = 215.6 \text{ kN} \text{ m} \]

We know that: \( w_k = \beta s_m \varepsilon_{sm} \), with \( \beta = 1.7 \), then we have to compute \( s_m \) and \( \varepsilon_{sm} \), which are the average separation between cracks and the average deformation of the reinforcement.

\[ s_m = 2c + 0.2s + 0.4k_1 \frac{\varphi A_{c,eff}}{A_s} \]
Where,

\[ \begin{align*}
  &c \quad \text{Cover (mm)} \\
  &s \quad \text{distance between longitudinal bars (mm)} \\
  &k_1 = \frac{\varepsilon_1 + \varepsilon_2}{8 \varepsilon_1} \text{, where the deformations are defined in the following figure 15.}
\end{align*} \]

\[ \begin{align*}
  &A_{c,\text{eff}} \quad \text{Area of concrete as defined in figure 16 (mm}^2) \\
  &A_s \quad \text{Area of the reinforcement in } A_{c,\text{eff}} \text{ (mm}^2) \\
\end{align*} \]

\[ \begin{align*}
  &\varepsilon_{sm} = \frac{\sigma_s}{E_s} \left[ 1 - k_2 \left( \frac{\sigma_{sr}}{\sigma_s} \right)^2 \right] \geq 0.4 \frac{\sigma_s}{E_s}
\end{align*} \]

Where

\[ \begin{align*}
  &\sigma_s \quad \text{Stress in the reinforcement under cracked section} \\
  &\sigma_{sr} \quad \text{Stress in the reinforcement under cracked section at the cracking time} \\
  &k_2 \quad \text{Coefficient shown in figure 15. Depending on the stress situation of the beam}
\end{align*} \]
Computing we obtain a deformation of $\varepsilon_{sm} = 0.001$. We can now obtain the characteristic with of the crack, $w_k = 0.203$ mm.

The above calculations are done for a bending moment of 318.5 kN m, which gives a $f_{ss}$ of 374 kN m.

If we redo this process for several bending moments above the cracking moment we obtain the following cracked widths:

<table>
<thead>
<tr>
<th>Bending Moment (kN m)</th>
<th>$F_{ss}$ (MPa)</th>
<th>Crack Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>245</td>
<td>288</td>
<td>0.066</td>
</tr>
<tr>
<td>270</td>
<td>316</td>
<td>0.113</td>
</tr>
<tr>
<td>294</td>
<td>345</td>
<td>0.159</td>
</tr>
<tr>
<td>318.5</td>
<td>374</td>
<td>0.203</td>
</tr>
</tbody>
</table>

**Table 12. Results EHE.**

**AASHTO**

The American code limits the separation between longitudinal reinforcement as:

$$ s \leq \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c \text{ with } \beta_s = 1 + \frac{d_c}{0.7(h-d_c)} $$

Where,

- $\gamma_e$ exposure factor, 0.5 implies a crack width of 0.2159 mm (0.0085 in)
- $d_c$ thickness of the concrete cover
- $f_{ss}$ tensile stress in the reinforcement steel
- $h$ overall thickness

If we want to compare cracked width of both codes we will have to rearrange he equations as:

$$ \gamma_e = \frac{(s + 2d_c)\beta sf_{ss}}{700} $$

<table>
<thead>
<tr>
<th>Bending Moment (kN m)</th>
<th>$F_{ss}$ (MPa)</th>
<th>$\gamma_e$</th>
<th>Crack Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>245</td>
<td>288</td>
<td>0.26</td>
<td>0.11</td>
</tr>
<tr>
<td>270</td>
<td>316</td>
<td>0.28</td>
<td>0.12</td>
</tr>
<tr>
<td>294</td>
<td>345</td>
<td>0.32</td>
<td>0.14</td>
</tr>
<tr>
<td>318.5</td>
<td>374</td>
<td>0.34</td>
<td>0.17</td>
</tr>
</tbody>
</table>
To obtain the crack width we have extrapolated from $\gamma_e = 0.5$ implies a crack width of 0.2159 mm

**Deformation Service Limit State**

**EHE**

The maximum values for the deformations depend on the structure studied and the purpose of it. Normally, the maximums are a relationship between the depth and the span of the element.

For bridges the Spanish code limits the difference between the initial deformation and the final deformation. The initial deformation is what the EHE mentions as counter deformation, the one due to the prestressed actions in a short time study. The final deformation is the one taking into account the retraction and creep of the concrete. In no case the increment of the deformations must be higher than:

<table>
<thead>
<tr>
<th></th>
<th>Highways</th>
<th>Roads of fast traffic</th>
<th>Roads of slow traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isotactic bridges of one span</td>
<td>L/1750</td>
<td>L/1000</td>
<td>L/700</td>
</tr>
<tr>
<td>Isotactic bridges of more than one span</td>
<td>L/3500</td>
<td>L/2000</td>
<td>L/1400</td>
</tr>
<tr>
<td>Continuous bridges</td>
<td>L/1400</td>
<td>L/750</td>
<td>L/500</td>
</tr>
</tbody>
</table>

The Spanish code considers that the total deflection is the sum of the instantaneous and the differed deflection.

For the calculation of the instantaneous deflection under flexion if the section is not cracked the calculation of the deflection will be done by traditional methods. If the section is cracked the EHE gives a method to calculate the new inertia. The main idea behind the method is to average the inertia of the cracked sections and the non-cracked sections, this gives us an average inertia of the cross section. The formulation is based on the average inertia of Branson.

For the differed deflections the EHE has simplified method based on ACI-318. This method can be used unless it noted otherwise. This method consist in multiplying the instantaneous deflection by a factor to obtain the differed deflection:

$$\gamma_{\text{diff}} = \lambda \cdot \gamma_{\text{inst}}$$

$$\lambda = \frac{\xi}{1 + 50\rho'},$$

where $\rho' = \frac{A_s}{b_0d'}$, being $A_s$ the area of steel in compression, $d$ the depth of the section and $b_0$ the width of the depth.

The coefficient $\xi$ depends on the time where we want to calculate the deflection:

- 5 year or more: $\lambda = 2.0$
AASHTO

The Code considers that the total deformation of an element must be taken as the instantaneous deflection and the long-term deflection.

The deflections must be computed under dead load, live load, prestressing, erection loads, concrete creep and shrinkage, and steel relaxation.

The code allows computing the deflection either with the gross moment of inertia, $I_g$, or an effective moment of inertia, $I_e$, which considers the cracked section.

Unless a more exact determination is made, the long-term deflection may be taken as the instantaneous deflection multiplied by the following factors:

Based on $I_g$: 4.0
Based on $I_e$: $3.0 - \frac{1.2A'_s}{A_s} \geq 1.6$, $A'_s$ area of compression reinforcement, $A_s$ area of nonprestressed tension reinforcement (in$^2$)

Comparison

Both codes approach deformation in a similar way; they both are based in the same concept and compute the deflection considering the cracked section.
**Prestressed**

We will analyze the considerations that both codes have for the prestressed design of beams. We will mention the different limits of the concrete and the strands allowed by both codes in order to apply them to our example later on.

We will also study the types of losses and its calculations for both codes.

**AASHTO (Chapter 5.9.)**

**Limit Stresses**

The stress limits for prestressing tendons:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Stress-Relieved Strand and Plain High-Strength Bars</th>
<th>Low Relaxation Strand</th>
<th>Deformed High-Strength Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediately prior to transfer (f_{pe})</td>
<td>(0.70 f_{pu})</td>
<td>(0.75 f_{pu})</td>
<td>___</td>
</tr>
<tr>
<td>At service limit state after all losses (f_{pc})</td>
<td>(0.80 f_{py})</td>
<td>(0.80 f_{py})</td>
<td>(0.80 f_{py})</td>
</tr>
</tbody>
</table>

When it comes to limits for the concrete the American code considers two situations before losses and after losses.

For the first situation when it comes to compressive stresses the limit is \(0.6 f_{ci}'\), where \(f_{ci}'\) is the compressive strength at time of application of the prestressed force.

For tensile stresses before losses there is a wide range of stresses admitted that depend on the type of bridge and the point studied:

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Location</th>
<th>Stress Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Other Than Segmentally Constructed Bridges</td>
<td>In precompressed tensile zone without bonded reinforcement</td>
<td>(0.0948/\sqrt{f_{pc}} \leq 0.2) (ksi)</td>
</tr>
<tr>
<td>Segmentally Constructed Bridges</td>
<td>In areas other than the precompressed tensile zone and without bonded reinforcement</td>
<td>(0.24/\sqrt{f_{pc}}) (ksi)</td>
</tr>
<tr>
<td>Segmentally Constructed Bridges</td>
<td>In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5(f_{pc}), not to exceed 30 ksi.</td>
<td>(0.158/\sqrt{f_{pc}}) (ksi)</td>
</tr>
<tr>
<td>Segmentally Constructed Bridges</td>
<td>For handling stresses in prestressed piles</td>
<td>No tension</td>
</tr>
<tr>
<td>Transverse Stresses through Joints</td>
<td>Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of 0.5(f_{pc}) with internal tendons or external tendons</td>
<td>(0.0948/\sqrt{f_{pc}}) maximum tension (ksi)</td>
</tr>
<tr>
<td>Transverse Stresses through Joints</td>
<td>Joints without the minimum bonded auxiliary reinforcement through the joints</td>
<td>No tension</td>
</tr>
<tr>
<td>Stresses in Other Areas</td>
<td>For areas without bonded nonprestressed reinforcement</td>
<td>No tension</td>
</tr>
<tr>
<td>Stresses in Other Areas</td>
<td>In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5(f_{pc}), not to exceed 30 ksi.</td>
<td>(0.19/\sqrt{f_{pc}}) (ksi)</td>
</tr>
<tr>
<td>Principal Tensile Stress at Neutral Axis in Web</td>
<td>All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures</td>
<td>(0.110/\sqrt{f_{pc}}) (ksi)</td>
</tr>
</tbody>
</table>

**Table 14. Tensile Stresses**
For stresses at service limit state after losses the compressive stresses have to be studied with the load combination of service I and the tensile stresses have to be studied with service III. Each of this limits have different limits depending on the type of bridge and the location studied:

<table>
<thead>
<tr>
<th>Location</th>
<th>Stress Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads</td>
<td>0.45$\phi_w'$(ksi)</td>
</tr>
<tr>
<td>In segmentally constructed bridges due to the sum of effective prestress and permanent loads</td>
<td>0.45$\phi_w'$(ksi)</td>
</tr>
<tr>
<td>Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling</td>
<td>0.60 $\phi_w' f'_c$(ksi)</td>
</tr>
</tbody>
</table>

Table 15. Compressive Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Location</th>
<th>Stress Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Other Than Segmentally Constructed Bridges</td>
<td>Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections</td>
<td>0.19$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions</td>
<td>0.19$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions</td>
<td>0.0948$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>For components with unbonded prestressing tendons</td>
<td>No tension</td>
</tr>
<tr>
<td>Segmentally Constructed Bridges</td>
<td>Longitudinal Stresses through Joints in the Precompressed Tensile Zone</td>
<td>0.0948$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 $f_y$; internal tendons or external tendons</td>
<td>0.0948$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>Joints without the minimum bonded auxiliary reinforcement through joints</td>
<td>No tension</td>
</tr>
<tr>
<td></td>
<td>Transverse Stresses through Joints</td>
<td>0.0948$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>Tension in the transverse direction in precompressed tensile zone</td>
<td>0.0948$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td>Stresses in Other Areas</td>
<td>For areas without bonded reinforcement</td>
<td>No tension</td>
</tr>
<tr>
<td></td>
<td>In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 $f_y$, not to exceed 30 ksi</td>
<td>0.19$\sqrt{f'_c}$ (ksi)</td>
</tr>
<tr>
<td>Principal Tensile Stress at Neutral Axis in Web</td>
<td>All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures</td>
<td>0.110$\sqrt{f'_c}$ (ksi)</td>
</tr>
</tbody>
</table>

Table 16. Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components
**Losses**

In pretensioned members the American code considers the following:

\[ \Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \]

Where,

\( \Delta f_{pT} \)  total loss (ksi)
\( \Delta f_{pES} \)  sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)
\( \Delta f_{pLT} \)  losses due to long-term shrinkage and creep of concrete, and relaxation of steel (ksi)

The losses from elastic shortening for pretensioned members shall be:

\[ \Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cpg} \]

Where,

\( f_{cpg} \)  The concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).
\( E_p \)  Modulus of elasticity of prestressing steel (ksi).
\( E_{ct} \)  Modulus of elasticity of concrete at transfer or time of load application

The total elastic loss or gain may be taken as the sum of the effects of prestressed and applied loads.

The time dependent losses have two ways of being calculated within AASHTO, the approximated way and the refined way. The approximate way takes into account the relative humidity and the relaxation of the strands:

\[ \Delta f_{pLT} = 10 \frac{f_{pi} A_{ps}}{A_s} \gamma_h \gamma_{st} + 12 \gamma_h \gamma_{st} + \Delta f_{pR} \]

Where,

\( f_{pi} \)  prestressing steel stress immediately prior to transfer (ksi)
\( \gamma_h \)  correction factor for relative humidity of ambient air, \( \gamma_h = 1.7 - 0.01H \)
\( H \)  the average annual ambient relative humidity (%)
\( \gamma_{st} \)  correction factor for specified concrete strength at time of prestress transfer to concrete member, \( \gamma_{st} = \frac{5}{1+f_{c}\gamma_{tr}} \)
\( \Delta f_{pR} \)  an estimate relaxation loss taken as 2.4 ksi for low relaxation strand, 10 ksi for stress relieved strands, and in accordance with manufacturers recommendation for other types of strand (ksi)
**EHE (Art. 20)**

**Limits Stresses**

In the Spanish code the limits for the stress in the tendons will never be higher than the lower of the following:

\[ \sigma_{p0} = 0.70 \, f_{p \text{max}k} \quad \text{or} \quad \sigma_{p0} = 0.85 \, f_{pk} \]

Where \( f_{p \text{max}k} \) is the maximum characteristic unit charge and \( f_{pk} \) is the characteristic elastic limit.

Stresses in concrete have 4 restrictions:

- At the time of posttension the most tensioned fiber cannot exceed \( 0.6f_{c,j} \) (\( j \) being the days at with the posttension occurs). The most tensioned fiber cannot exceed \( 0.6f_{ctm,fl,j} \).
- At 28 days the most tensioned fiber can not exceed the \( f_{ct,m} \) and the most compressed fiber can not exceed \( f_{ck} \).

**Losses**

In pretensioned members the losses are dived in two groups: instantaneous and differed. The instantaneous:

\[ \Delta P_t = \Delta P_1 + \Delta P_2 + \Delta P_3 \]

\( \Delta P_1 \) losses due to friction computed as:

\[ \Delta P_1 = P_0 \left[ 1 - e^{-(\mu \alpha + \xi \eta)} \right] \]

This equation considers a friction coefficient of the curve (\( \mu \)), angular variations (\( \alpha \)), a friction coefficient of the parasite (\( K \)) and the distance between the section studied and the active anchor. All this coefficients may be found in table 20.2.2.1.1.a of EHE.

\( \Delta P_2 \) losses due to wedge penetration

\[ \Delta P_2 = \frac{a}{L} E_p A_p \]

This formula takes into account the distance of the anchor (\( a \)) the total length of the tendon (\( L \)), modulus of deformation of the prestressed (\( E_p \)) and the active area of the prestressed (\( A_p \)).

\( \Delta P_3 \) losses due to elastic shortening

\[ \Delta P_3 = \sigma_{cp} \frac{n-1}{2n} A_p E_p \frac{E_{c,j}}{E_{c,j}} \]

This loss is computed with the compression stress of the tendon (\( \sigma_{cp} = P_0 - P_1 - P_2 \)) and the modulus of deformation of the concrete at \( j \) days (\( E_{c,j} \)).

The differed losses are the ones that occur in long-term considerations. The shortening of the concrete due to shrinkage and creep and the relaxation of the steel
mainly produce these losses. These are computed as indicated in the art 20.2.2.2 of EHE.

**Comparison**

The general philosophy of the calculation of losses is almost the same. Losses due to friction, wedge penetration and elastic shortening are almost equal in both codes. Long-term losses are computed slightly different but provide the same results.
Design of a deck and prestressed beams

General Considerations

In AASHTO we will study strength I, service I and service III because these are the main combination loads to design a bridge and for the purpose of comparing both codes it will be sufficient.

In the Spanish code we design with the service limit state and we will check the resistance ultimate limit state, for the same reasons as in the American code we will only check this states because the purpose is to compare codes and not design a bridge.

In both codes if we wanted to design to full detail more limit states should be taken into account.
Bases of the Model

In order to compare with an example all the previous studies and see where the main differences are we will consider a span of 30 m (98.43 ft) of a set of 5 simply supported beams separated by 3.50 m (11.48 ft) and the extreme beams have an overhang of 1.5 m (4.92 ft). On top of the beams there is a deck of 0.20 m (8 in). The beams and slab have the following geometry:

![Figure 17. X-section of the beam and slab. (in)](image)

For each code there are slightly different compression resistances, modulus of elasticity, etc. We can see the ones used to proceed with the calculations in the following tables.
**AASHTO**

| Weight of concrete $f_c = 4$ksi | 0.145 kcf | 22.78 kN/m³ |
| Weight of concrete $f_c = 8$ksi | 0.148 kcf | 23.25 kN/m³ |

**Material Properties**

| Strength of Reinforcing Steel | 60.00 ksi | 413.69 N/mm² |
| Strength of Concrete | 4.00 ksi | 27.58 N/mm² |
| Strength of Concrete | 8.00 ksi | 55.16 N/mm² |
| Modulus of Elasticity - Steel | 29000.00 ksi | 199947.96 N/mm² |
| Modulus of Elasticity - Concrete 4ksi | 3644.15 ksi | 25125.51 N/mm² |
| Modulus of Elasticity - Concrete 8.5ksi | 5314.37 ksi | 36641.27 N/mm² |
| Modular Ratio n 4 ksi | 7.96 | 7.96 |
| Modular Ratio n 8 ksi | 5.46 | 5.46 |

Table 17. Materials constants for AASHTO, in both US and IS

**EHE/IAP**

| Weight of reinforced concrete | 25.00 kN/m³ | 0.159 kcf |

**Material Properties**

| Strength of Reinforcing Steel | 400.00 N/mm² | 58.02 ksi |
| Strength of Concrete | 25.00 N/mm² | 3.63 ksi |
| Strength of Concrete | 55.00 N/mm² | 7.98 ksi |
| Modulus of Elasticity - Steel | 200000.00 N/mm² | 29007.55 ksi |
| Modulus of Elasticity - Concrete 25 | 24854.15 N/mm² | 3604.79 ksi |
| Modulus of Elasticity - Concrete 59 | 32325.10 N/mm² | 4688.36 ksi |
| Modular Ratio n 25 | 8.05 | 8.05 |
| Modular Ratio n 55 | 6.19 | 6.19 |

Table 18. Materials constants for EHE and IAP, in both IS and US

The differences are not much, so we will use the same values for both models. The values used in the models are:

**Model**

| Weight of concrete | 25.00 kN/m³ | 0.159 kcf |

**Material Properties**

| Strength of Reinforcing Steel | 400.00 N/mm² | 58.02 ksi |
| Strength of Concrete | 27.58 N/mm² | 4.00 ksi |
| Strength of Concrete | 55.16 N/mm² | 8.00 ksi |
| Modulus of Elasticity - Steel | 199947.96 N/mm² | 29000.00 ksi |
| Modulus of Elasticity - Concrete 27 | 25096.92 N/mm² | 3640.00 ksi |
| Modulus of Elasticity - Concrete 59 | 34 523 N/mm² | 5147.74 ksi |
| Modular Ratio n 25 | 7.97 | 7.97 |
| Modular Ratio n 55 | 5.63 | 5.63 |
**Dead Loads**

In both codes we will consider the dead load of the structure, with a weight of 25 kN/m³ (0.159 kcf). This gives us the following diagram of bending moments for the central beam, with no amplification coefficients considered:

![Bending Moment Diagram](image)

**Figure 18.** Bending moments of the self-weight of the structure for several beams.

Beam 1, the one on the edge is the one with more moment because of the overhang and the concrete barrier at the ends. The graph shows the dead load of the beams and the deck.

Beams 4 and 5 are symmetrical to beams 2 and 1 respectively.

**Pavement**

In both models we will consider the load of the pavement. This pavement will be of 5 cm (1.97 in) and will have the weight that each codes considers.

**IAP**

The Spanish code considers that a bituminous pavement has a weight of 23 kN/m³ (0.146 kcf). The IAP also contemplates two values: the theoretical one (5 cm) and one with a 50% increase (7.5 cm).

**AASHTO**

The American code does not consider any increase on the pavement and uses a weight of 0.14 kcf (22 kN/m³).
Comparison

If we plot the moments for beam 1, the one in the edge, for both codes with no amplification factors we see the following. The Spanish code considers two possible pavements.

The small difference between AASTHO and the inferior value of IAP is given by the slightly difference between the weight considered in the codes.

Figure 19. Comparison of bending moments caused by the pavement.
Live Loads: Traffic loads

AASTHO

The number of design lanes is given by dividing the width of the deck in feet by 12, $\frac{55}{12} = 4.58$, giving us 4 design lanes. We can see in the figure (20) the result:

![Figure 20. Lanes in AASHTO](image)

*Design truck*

The design truck will be the one according to the code specifications.

*Design tandem*

The design tandem shall consist of a pair of 25 kip axles spaced 4 ft and 6 ft transversally.

*Multiple presence factors*

We have four design lanes, so we will multiply our truck and tandem loads by 0.65.

*Design lane load*

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. In the transverse direction it will be distributed over 10 ft width.

The extreme force effect shall be taken as the larger of the truck or tandem both combined with the design load lane.

We will compute all possible combinations. We will take the worst-case scenario, the one that gives us bigger stresses.

*IAP*

We can see in figure (21) that the Spanish code gives us the following number of design lanes: $\frac{17}{3} = 5.67$, this means 5 design lanes.
Design truck

The model has 5 lanes so there will be set of possible combinations where the design trucks can be considered. It will only be shown the worst of all scenarios possible.

Design lane load

The same thing as in the design trucks, having 5 design lanes a set of combinations is possible. All combinations of trucks and lane loads will be considered, but only the worst-case scenario will be shown.

Comparison

We can see that the main difference in this example is that the American code will consider 4 lanes and the Spanish one will consider 5 lanes. This may lead to significant differences.
Combination of Loads

It has been shown that we will consider the dead load of the structure and the traffic loads that both codes consider. We will consider the load combination cases noted before.

**AASHTO**

The total factored force effect shall be taken as:

\[ Q = \sum \eta_i \gamma_i Q_i \]  

\( \eta_i = \) load modifier  
\( Q_i = \) force effects  
\( \gamma_i = \) load factor

As explained before we will consider the following load cases:

- **Strength I**: Basic load combination relating to the normal vehicular use of the bridge without wind.
- **Service I**: Load combination relation to the normal operational use of the bridge.
- **Service III**: Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

*Load modifier (Article 1.3.2.)*

We will consider that we are dealing with a conventional bridge that complies with the specifications and with a conventional level of redundancy. This will give a load modifier of \( \eta = 1.0 \) for the limit states considered.

**Load Factors**

The total load and position of the loads to be considered has different factors for every type of load and combination. We will use the following notation:

- **DC**: Dead load of the structural components and nonstructural attachments  
- **DW**: Dead load of the wearing surface and utilities  
- **PS**: Secondary forces from post-tensioning  
- **LL**: Vehicular live load  
- **LS**: Live load surcharge

**Strength I**

For this combination of load we have a maximum and a minimum load to consider.

\[ Q_{max} = \eta \cdot 1.25 \cdot DC + \eta \cdot 1.50 \cdot DW + \eta \cdot 1.0 \cdot PS + \eta \cdot 1.75 \cdot LL \]  

\[ Q_{min} = \eta \cdot 0.9 \cdot DC + \eta \cdot 0.65 \cdot DW + \eta \cdot 1.0 \cdot PS + \eta \cdot 1.75 \cdot LL \]
Service I

\[ Q = \eta \cdot 1.0 \cdot (DC + PS + DW) + \eta \cdot 1.00 \cdot LL \]  

(14)

Service III

\[ Q = \eta \cdot 1.0 \cdot (DC + PS + DW) + \eta \cdot 0.8 \cdot LL \]  

(15)
The load combinations of the Spanish code we will use are:

- Permanent resistance ultimate limit state
- Frequent service limit state

As noted for the American code, we will consider the dead load of the structure, the traffic loads and the prestressed forces. The following notation will be used:

- \( G \) characteristic value of permanent loads (Dead Load)
- \( G^* \) characteristic value of permanent loads with no constant value. (Prestressed forces)
- \( Q_1 \) characteristic value of the dominant live load (Distributed traffic load)
- \( Q_j \) characteristic value of all other live load considered (Trucks)
- \( \Psi_0 \) coefficient of combination.
- \( \gamma_G, \gamma_Q \) coefficients

**Calculation value for Ultimate Limit State**

We will consider two states, the pro and adverse, giving us these two possible combinations:

**Maximum Load**

\[
M_{\text{max}} = 1.35 \cdot G + 1.35 \cdot G^* + 1.35 \cdot Q_1 + \sum_{l>1} 0.75 \cdot 1.35 \cdot Q_j
\]  

(16)

**Minimum Load**

\[
M_{\text{min}} = 1.0 \cdot G + 1.0 \cdot G^*
\]  

(17)

**Calculation value for Service Limit State**

- Frequent combination:

**Maximum Loads**

\[
M_{\text{max}} = 1.0 \cdot G + 1.0 \cdot G^* + 1.0 \cdot 0.4 \cdot Q_1
\]  

(18)

**Minimum Loads**

\[
M_{\text{min}} = 1.0 \cdot G + 0.9 \cdot G^*
\]  

(19)

- Quasi-permanent combination:

**Maximum Loads**

\[
M_{\text{max}} = 1.0 \cdot G + 1.0 \cdot G^* + 1.0 \cdot 0 \cdot Q_1
\]  

(20)

**Minimum Loads**

\[
M_{\text{min}} = 1.0 \cdot G + 0.9 \cdot G^*
\]  

(21)
**Ultimate Moments**

With the load cases defined for both codes and with the combination factors set we can proceed to input all this information in the model of SAP2000. This model will give us the maximum stresses of each case. We will use these stresses, moments and shears to design and check the limit states previously noted. Following we have the results for the maximum moments obtained in each combination.

**ASSHTO**

*Strength 1*

Considering both trucks and tandem we obtain the following maximum moments for this combination of loads:

\[
M_{\text{strength,max}} = k\text{-ft} \quad \text{Trucks}
\]
\[
M_{\text{strength,max}} = k\text{-ft} \quad \text{Tandems}
\]

Then we will take the maximum of the two for design purposes, this moment occurs at the mid-span of the first beam. In the following figure we can see the envelopes for the trucks and for the tandems.

![Envelope of maximum moments strength 1](image)

The minimum combination gives the following maximum moments:

\[
M_{\text{strength,min}} = k\text{-ft} \quad \text{Trucks}
\]
\[
M_{\text{strength,min}} = k\text{-ft} \quad \text{Tandems}
\]

The truck combination gives again the worst case and at the same place than the case before.
Service I & III

Proceeding the same way as in the strength combination, we have obtain that the worst case scenario for both combinations is given by the tandem combination at the mid-span of the first beam.

\[ M_{\text{service I}} = 5104.01 \text{ k-ft} \]
\[ M_{\text{service III}} = 4732.78 \text{ k-ft} \]

Figure 23. Envelopes of maximum bending moments for service I and III
IAP

**Ultimate Limit State**

After considering all the possible combinations for the ultimate limit state we have that the worst-case scenario is given in the combination where:

- Lane load of 2.5 kN/m², All Deck
- Lane load of 9 kN/m², Lane 1
- Truck 300 kN, Lane 1
- Truck 200 kN, Lane 3
- Truck 100 kN, Lane 2

This combination of loads gives the biggest moment in the first beam, \( M_{ULE,\text{max}} = 12305.97 \text{ kN m} \).

For the minimum, we only consider the dead load and the lower value of the pavement so \( M_{ULE,\text{min}} = 4425.74 \text{ kN m} \). This moment is given in the first and the last beam at its mid-span.

**Service Limit State**

- Frequent Combination

The frequent combination only considers the lane loads with a reduction factor. Its maximum is giving in the loading of the first lane at the mid-span of the first beam \( M_{SLS,freq,\text{max}} = 5443.91 \text{ kN m} \).

For the minimum only the dead load and the pavement are considered so \( M_{SLS,freq,\text{min}} = 4425.74 \text{ kN m} \).

- Quasi-permanent Combination

For the quasi-permanent combination only the dead load is considered and it is the same for the minimum and maximum \( M_{SLS,q} = 4425.74 \text{ kN m} \).

**Comparison**

In the following tables we see a summary of the moments computed for both codes in international units and American units.

<table>
<thead>
<tr>
<th></th>
<th>Moment (kN m)</th>
<th>Moment (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strength I Max</strong></td>
<td>10012.88</td>
<td>7385.2</td>
</tr>
<tr>
<td><strong>Strength I Min</strong></td>
<td>5632.08</td>
<td>4154.01</td>
</tr>
<tr>
<td><strong>Service I</strong></td>
<td>6920.11</td>
<td>5104.01</td>
</tr>
<tr>
<td><strong>Service III</strong></td>
<td>6416.78</td>
<td>4732.78</td>
</tr>
</tbody>
</table>

*Table 19. Moments for AASHTO*
<table>
<thead>
<tr>
<th></th>
<th>Moment (kN m)</th>
<th>Moment (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ULE</strong></td>
<td>13381.23</td>
<td>9869.49</td>
</tr>
<tr>
<td><strong>Dead Load</strong></td>
<td>3447.30</td>
<td>2564.72</td>
</tr>
<tr>
<td><strong>SLS Frequent</strong></td>
<td>5443.91</td>
<td>4015.22</td>
</tr>
<tr>
<td><strong>SLS Quasi</strong></td>
<td>4425.74</td>
<td>3264.26</td>
</tr>
</tbody>
</table>

Table 20. Moments for IAP
Design of the prestressed beam

1st Step: Predesign

For both codes we will use the same beam and the same materials. The materials concerning only the beam are the following:

**Concrete:**

- $f_{ck} = 59$ MPa (8.5 ksi)
- $E_c = 34523$ MPa

**Steel:**

- $f_y = 420$ MPa
- $f_{max} = 1860$ MPa
- $f_{yk} = 1675$ MPa

We define the yield strength of the prestressed steel as the stress at 1% of strain, this is as define in AASHTO designation: M 203M/M 203-07, which is in the same range of values established in EHE 34.3.

In order to start with the design we will check the capacity of the final section, deck and beam, with both codes. We will do this by iterating with different amounts of prestressed strands. In both codes we will use strands of a diameter of 0.5 in (12.7 mm) composed by 7 wires of 1/6 in.

**AASHTO**

After a few iterations we have seen that with 14 strands distributed in three layers and 4 strands in the fourth layer the section holds up. We will try the distribution and amount of steel shown in the side figure.

We will use a prestressed force of 134.94 kN represents 73.4% of $f_{max}$. Meeting the criteria in AASHTO 5.9.3.1.

**EHE**

From the predesign of the beam with the Spanish code we have obtained that we will need 5 levels of strands as shown in the side figure.

We will use a prestressed force of 134.94 kN represents 73.4% of $f_{max}$. Meeting the criteria in EHE 20.2.1.
**2nd Step: Beam Only**

Here we will compute de stresses and strains of the beam when the deck has just been poured. Meaning that the beam has to endure its self-weight and the weight of the deck but the deck has no compression capacity. This means the following cross-section:

![Figure 26. Cross-Section of the Standard beam T x 54 (in).](image)

**AASHTO**

The geometric data for the homogenized section is:

<table>
<thead>
<tr>
<th>Area (m²)</th>
<th>0.5498</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center of Gravity (m)</td>
<td>0.5776</td>
</tr>
<tr>
<td>Inertia (m⁴)</td>
<td>0.1299</td>
</tr>
</tbody>
</table>

The American code lets us consider that the concrete has a resistance to tensile stresses. The tensile stress that we can use is \( f_{ct} = 0.19 \sqrt{f_c} = 3.80 \text{ MPa} \), AASHTO 5.9.4.2.2-1. We use this value for the Bending Moment - Curvature diagram.

The compressive strength limits established in AASHTO 5.9.4.1.1. at stressing is \( f_c = 0.6 f'_c = 35.40 \text{ MPa} \).

As shown previously the maximum bending moment of self-weight the beam has to endure is 3477 kN - m. With this data we can proceed to draw the bending moment - curvature diagram that will allow us to determine stresses and strains:
From the diagram we obtain the curvature that allows us to obtain the following data:

![Bending Moment vs Curvature Diagram](image)

**Figure 27. Moment - Curvature Diagram**

We can see that the beam will have no problem supporting the weight of the deck while the deck doesn’t develop compressive strength.

**EHE**

The geometric data for the Spanish code will be different because we have more strands:

<table>
<thead>
<tr>
<th></th>
<th>0.5573</th>
<th>0.5731</th>
<th>0.1307</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (m²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center of Gravity (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inertia (m⁴)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The tensile strength that the EHE let’s use is:

\[
f_{ct,m} = 0.58 f_{ck}^{1/2} = 4.45 \text{ MPa} \quad \text{if } f_{ck} > 50 \frac{N}{mm^2}
\]

Notice that the tensile limit is a little higher than AASHTO’s.

With the maximum moment of 3477 kNm we can proceed to draw the bending moment – curvature diagram to obtain stresses and strains:
From this diagram we obtain the curvature of the section. This allows us to obtain the stresses and strains of the concrete and steel:

![Bending Moment vs Curvature Diagram](image)

**Table 22.**

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Stress</th>
<th>Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top (MPa)</td>
<td>Bottom (MPa)</td>
</tr>
<tr>
<td></td>
<td>15.39</td>
<td>12.72</td>
</tr>
</tbody>
</table>

Here again the beam will have no problem supporting the deck.

**Comparison**

The differences between codes are due to the number of strand needed (AASHTO 46 and EHE 62). This makes that for the same bending moment EHE has less deformation than AASHTO. This is reflected in the following figure.

![Bending Moment vs Curvature](image)

**Figure 29. Comparison between codes**
We can also compare the strain and stresses for both codes in the following figures:
**3rd Step: Beam and Deck**

At this point not only the beam is able to resist the loads but also the deck. To proceed with this step of the design we will impose the deformations on the beam that we have computed previously. This will give us a cross-section where the beam has deformation and the deck has a different concrete with no initial deformations.

![Diagram of beam and deck with dimensions and labels](image)

**Figure 30. Tx54 beam and the slab. (in)**

The data material for the deck is the following for both codes:

\[
f_c = 35.0 \text{ MPa} \quad \text{E}_c = 29,778 \text{ MPa}
\]

In both codes we will consider a typical quantity of losses of 12% in the prestressed strands. The losses are due to shrinkage, relaxation and creep.

**ASSHTO**

Once we include the slab in the cross-section the geometric homogenized data is the following:

| Area (m²) | 1.1683 |
| Center of Gravity (m) | 1.0470 |
| Inertia (m⁴) | 0.3610 |

With the deformation that we had earlier for the beam and with no deformation on the deck we have the following initial situation for stresses and strains:
We have not yet taken into consideration the losses of the prestressed strands. We will now consider a 12% in the prestressed strands. As the American code says we will check tensile stress at service III and compressive stress at service I. At the service limit state the American code makes us use a prestressed force of \(0.80 f_{py}\) after losses.

The compression limits for service I (due to permanent loads) established at 5.9.4.2.1 is \(0.45 f'_c = 26.55\, MPa\). In prestressed beams with a deck on top it is never the limiting factor due to the addition of fresh concrete on top of the beam.

Once we compute the curvature of the cross-section, the stresses and the strains we have to check that at no point in the section the concrete reaches the tensile or compressive limits:

For tensile stresses, that at no point the stress is under \(-3.80\, MPa\) at service III.

For compressive stresses, that at no point the stress is above \(26.55\, MPa\) at service I.

For this composite cross-section we obtain the following bending moment – curvature diagram:

![Figure 31. Bending Moment – Curvature for the composite section](image-url)
For our moment of service III, \( M_{\text{Service,III}} = 6420 \text{ kN} \cdot \text{m} \), as done before we can from the curvature of the section obtain the following:

<table>
<thead>
<tr>
<th>Beam</th>
<th>Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stress (Mpa)</strong></td>
<td><strong>Strain</strong></td>
</tr>
<tr>
<td>Bottom</td>
<td>Top</td>
</tr>
<tr>
<td>18.60</td>
<td>2.45</td>
</tr>
<tr>
<td>-2.93</td>
<td>-8.49E-05</td>
</tr>
<tr>
<td>-4.14E-04</td>
<td>1.32E-04</td>
</tr>
</tbody>
</table>

**Table 23. Stresses and Strains of the beam at Service III**

We can see that the tensile limit is not reached.

When checking for compressive stresses in service I only permanent loads have to be applied. For prestressed beams this condition is always met.

**EHE**

We proceed as with the American code but with the initial deformations corresponding to the EHE:

As done before, we will consider a 12% in the prestressed strands and analyze the composite cross-section for the frequent combination of loads. For this composite cross-section we obtain the following bending moment – curvature diagram:
If we search for the value of our frequent moment, $M_{\text{frequent}} = 5444 \text{ kN m}$, we will find a curvature and from there we can compute the following values:

| Concrete |
|------------------|------------------|
| **Stress**       | **Strain**       |
| Top (Mpa)        | Bottom (Mpa)     | Top      | Bottom        |
| 2.78             | 5.48             | 9.57E-05 | 1.12E-04      |

**Table 24.**

We can see that the beam has no tensile stresses, so the section will not crack and it is not necessary to check the section with the quasipermanent combination of loads.

**Comparison**

Comparing the bending-curvature of these cross sections gives no significant data. This is because the quantity of prestressed strands is bigger in EHE than in AASHTO requirements. Obviously this will give more capacity to the Spanish cross section.
**4th Step: Capacity of the Section**

We will now check the capacity of our section for both codes. We will compare the capacity of the section to the Strength moment for AASHTO and the Ultimate moment for EHE.

**AASHTO**

We will proceed to check the bending moment – axial force interaction diagram, where when the axial force is 0 kN we will obtain the capacity of the section.

![Bending Moment (kN - m)](image)

**Figure 34.**

For the diagram above, we obtain the capacity of the section and we can compute the stresses and strains for the cross-section at the capacity moment:

<table>
<thead>
<tr>
<th>Moment</th>
<th>Beam Stress (Mpa)</th>
<th>Beam Strain</th>
<th>Slab Stress (Mpa)</th>
<th>Slab Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>kN m</td>
<td>Bottom</td>
<td>Top</td>
<td>Bottom</td>
<td>Top</td>
</tr>
<tr>
<td>11256.11</td>
<td>0.00</td>
<td>12.07</td>
<td>1.11</td>
<td>20.10</td>
</tr>
<tr>
<td>Strain</td>
<td>-4.00E-03</td>
<td>6.36E-04</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 25.**

We should point out is the resistance factor for tension-controlled prestressed concrete (AASHTO 5.5.4.2.1) is $\phi = 1.00$

$M_r = \phi M_n \quad M_r = 11256 \text{ kN m}$

We can see the capacity moment ($M_r$) is higher that the Strength moment, 10012 kN m.
**EHE**

We will repeat the same process as done for the American code. We will check the capacity to make sure that the moment is higher that the one given by the IAP.

![Bending Moment (kN - m)](image)

**Figure 35.**

From the diagram above we can obtain the following data for the capacity moment:

<table>
<thead>
<tr>
<th>Moment (kN m)</th>
<th>Beam</th>
<th>Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress (Mpa)</td>
<td>0.00</td>
<td>2.88</td>
</tr>
<tr>
<td>Strain</td>
<td>-1.00E-02</td>
<td>1.23E-03</td>
</tr>
</tbody>
</table>

We can see that the capacity moment, 13 840 kN m, is higher that the $M_{ELU} = 13\,381$ kN m, so the beam will support all the loads.

**Comparison**

The capacity of the EHE beam is higher than the AASHTO beam for the reasons mentioned before. The EHE loads are bigger that the AASHTO loads this makes that the EHE beam has more strands to meet the requirements and gives it more capacity.
Conclusions

Thru the work done we’ve seen small and big differences between the two codes. The differences are most notable in the load combination and factors. Both codes use the same philosophy of ultimate and service states but they use different coefficients for the loads. These states will then be use in very similar ways by both codes. They may use different factor but the idea behind it is very similar: we first magnify the loads and the compute the bending moment, shear and axial forces acting.

Once the moments of all the ultimate and service limit states we can see that the Spanish code gives us bigger ultimate moments, this does not mean that we will need more steel. We saw in the comparison of the ultimate limit states that the American code has bigger capacities for the sections, due to the factors on the resistances of materials that the Spanish codes uses.

On top of that, the main difference when computing the ultimate state is the deformation domains. EHE a set of different domains with different breaking planes. On AASHTO the domains consist in tension breaking or compression breaking. With this domains we get different factor and not breaking planes.

Furthermore, ultimate state may not be the state that tells us the amount of steel needed. We saw that the service states are bigger in the American code. Also, the approach to live loads is very different and the combination factors are considered in very different ways, this leads to completely different moments.

Apart from the differences mentioned, the design calculations all come from the same theory and approach the design of beams the same way. This was something to expect because the theory behind the codes is based on the same.
Bibliography

Codes


Books

*Bridge Design and Evaluation: LRFD and LRFR.* Fu. G. 2013

Special Software

SAP2000.