DESIGN AND RATING FOR LONGITUDINAL FORCE

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ABSTRACT

This paper discusses the application of the new longitudinal force provisions of Chapters 8 and 15 of the AREMA Manual of Recommended Practice (/). It also discusses their eventual application to timber structures.

Emphasis is placed on following the load path to where the force leaves the structure and carefully selecting a load path that corresponds to the stiffest path, or at least selecting multiple paths in accordance with their structural stiffness.

Discussion covers both the design and evaluation aspects of longitudinal force and ends with a discussion of which structures are most likely to see the maximum longitudinal force.
INTRODUCTION

There are at least 4 recent innovations that give rise to the fact of much higher longitudinal forces from diesel-electric locomotives than those occurring previously.

• New High Adhesion Locomotives that start at the highest Adhesion.
• New Dynamic Braking Systems.
• Load/Empty Brake Systems.
• ECP Braking Systems.

The reduction made by AREA Committees 8 and 15 around 1968 to longitudinal force was a mistake based on data from a number of tests done with short, light trains. None of the tests was conducted under conditions that would approach the maximum possible longitudinal force available at the time, and consequently very low longitudinal forces were measured (4). A simple calculation of a typical locomotive of the day on a span equivalent to the out to out of the wheels would have indicated a much higher force at full adhesion demand on normal rail (2).

New High Adhesion Locomotives

Older D.C. Locomotives, such as SD-40’s, apply traction forces of about 12 kips or less per unit in normal operation and are usually dispatched to deal with the ruling grade at about 18% adhesion (roughly 72 kips) in poor weather conditions. They have a maximum capability of 80 kips in ideal conditions. At speeds above 10 mph, these forces are much reduced.
Furthermore, the method of applying traction requires that the load be applied in 8 steps (from low to high adhesion) and often wheel slip occurs long before notch 8 is ever reached.

A.C. Locomotives and many of the newer D.C. Locomotives are designed to be dispatched at 32% adhesion in poor weather conditions, but are capable of 50%+ adhesion in good weather conditions (up to 220 kips measured on a 420 kip unit (4)).

These units apply the adhesion starting in throttle position 8 so that maximum force is applied immediately on starting. Wheel slip is computer controlled, as is sanding. As technology develops these high adhesion rates could be applied at ever increasing speeds.

The result is that any location where a locomotive stops or slows considerably can get these maximum forces over a short to long time duration depending on train length (load) and grade.

At least one railroad has issued instructions limiting application of traction to 100 kips to protect their existing plant until appropriate measures are taken.

Note: Catenary fed electric units have always been capable of higher tractive efforts and are outside the scope of the current Manual recommendations.
**New Dynamic Braking Systems**

The new A.C. technology and better cooling has doubled the potential Dynamic Braking force due to dynamic locomotive braking and future developments being tested now will increase this further.

Again, one railroad (a different one) has restricted dynamic braking to half the unit’s capability to protect their infrastructure.

**Load/Empty Brake Systems**

Older systems were designed to limit the braking pressure to what an empty car could absorb. Recall $f = \mu N$.

These new systems for unit trains apply one brake application when the train is empty and a much higher pressure when the train is full using the spring deflection to determine how much to apply.

**ECP Braking Systems**

This brake control system converts the application of train braking from one where braking is applied successively over time from car to car to one where the brakes from each car are applied at the same instant thus stopping the train in a shorter time frame and distance.
Backed by Test Results (AAR- TTCI)

The first test (3) was conducted and the test results indicated that measured longitudinal forces were in the order of 25 times those predicted by what was in the Chapter 8 and 15 sections of the Manual (1).

Subsequent tests (4, 5 ) confirmed the first test and led to a number of interesting conclusions:

- The behavior of short spans does not justify any reduction in longitudinal force for such spans.
- The behavior of ballast deck spans does not justify a reduction in longitudinal force for such spans.
- Longitudinal forces were measured over a number of situations and found to be common where new technologies would be expected to produce high longitudinal forces.
- Certain locations where high longitudinal force would be rare events were delineated.

Concrete Bridge Before Change

Consider as an example a 965-foot Prestressed Concrete Box Girder and ballast deck bridge with 23 spans of 39’ and two spans of 34’. A design made before the change to higher longitudinal forces resulted in being able to take out all the longitudinal force at
the abutments where there were longitudinally battered piles for that purpose.

Although all of the intermediate bents had transversally battered piles none were longitudinally battered. Four 24-inch square precast-prestressed concrete piles were sufficient. This in spite of poor soil conditions.

CONCRETE BRIDGE AFTER CHANGE

The task of re-designing the structure to handle a longitudinal force that is in the order of 25 times higher is quite a challenge.

The first reaction from a designer is to question to see if there is any way to get around the new requirement.

Since the precast box girders sit on elastomeric pads, and since there is no connection of the rails to the substructure is it permissible to transfer the load to the top of the cap without the moment?

NO! Test results indicate that this is not what happens and the load must be taken as acting at 8 ft. above top of rail for braking or 3 ft. above top of rail for traction (Article 8.2.2.3j (1)) (1)

Why? Because the vertical loads are high enough to ensure sufficient friction. Nothing slips. Earlier tests done with much lighter and shorter trains gave contrary results (4) as there was insufficient load to ensure that nothing slipped.
The current design has 20 intermediate bents with transversely battered piles, and four intermediate bents with longitudinally battered piles. The abutments have battered piles in both directions.

The design longitudinal forces for E 80 are (J):

• **Braking force** = $45 + 1.2L$ applied 8 feet above top of rail
  
  $$45 + 1.2 \times 965' = 1203 \text{ kips}$$

• **Traction Force** = $25 (L)^{0.5}$ applied 3 feet above top of rail
  
  $$25 (965)^{0.5} = 777 \text{ kips}$$

Clearly in a structure this long the braking force controls.

Nevertheless, it is important to look at each element of the structure to see if it is isolated in which case a different L would apply. The most obvious case would be a swing span in an otherwise long bridge. Similarly, a portion of a steel deck with multiple floor beams might attract a high local traction force. This case is covered later in the paper.

Examining the structure at hand there is no need to consider any isolated parts other than the anchor bolts that need to be strong enough to handle the transfer of local longitudinal forces.
Soil Model

The AREMA Recommended Practice, Article 8.2.2.3j, (1) mandates that longitudinal force be distributed to the various elements according to their relative stiffness.

From Soil Borings, the soil model properties are:

–To El. –27: Clay, c = 0.37 ksf
–To El. – 50: Clay, c = 1.40 ksf
–To El. – 53: Clay, c = 0.61 ksf
–To El. – 90: Clay, c = 1.16 ksf

In soft clay, the initial assumption is to set the point of fixity at 15 feet below the ground line. Each bent is then assumed to be a cantilever from the point of fixity to the centerline of the cap.

Longitudinal Force Distribution

Each bent is assigned a longitudinal stiffness consisting of three components (the hollow in each pile is neglected for simplicity):

Horizontal Stiffness – Kh is the sum of the individual moments of inertia plus the sum of Ad².

Force = Deflection * 3EI/L³

Battered Stiffness Kb is based on the batter angle of the piles and is composed of a horizontal and a battered component where
Force = Kh*(cos(angle))^2 (horizontal) + AE/L * (sin(angle))^2 (battered)

Soil Resistance for the resisting abutment.

Assume 200 k/(in-ft) per foot width of an 8-foot wall height. (FHWA-SA-97-006, pages 4-15 and 4-16 after Caltrans) (6,7), thus

Force = 1031 k per inch deflection (only at the resisting abutment).

The entire structure will deflect uniformly with the stiffer elements attracting more of the longitudinal force.

Both directions of longitudinal force were examined because effects on the abutments differed with a change in the direction of longitudinal force.

Resolving Force to Centerline of Cap.

Each span receives half of the Longitudinal force taken by its supporting bents, except the end span which takes the entire longitudinal force from the abutment and half of the force from the first intermediate bent. The couple is resolved to the substructure as follows:

Span longitudinal force multiplied by the distance from the centerline of the cap to 8 feet above top of rail divided by the average span length. One bent would experience an increase in pile load and the other bent would be uplifted by this force. The longitudinal
force is now applied at the centerline of the cap, where it is resisted by pile batter and by moments on the piles.

_Resisting Longitudinal Force with Piles._

The vertical piles attract little of this force, so that can be handled by the pile moments. One row of piles in each of the battered bents supplies resistance by the horizontal component of pile capacity. The actual pile load was used as opposed to its capacity. This is good for subsequent rating purposes as the capacity of driven piles may be much higher, and occasionally lower than assumed and in such soil conditions is always in some doubt. The longitudinal force component in excess of the battered resistance is split to all six piles as moment.

**At abutments:** One row of piles resists by using the horizontal component of the battered pile loads. The loads on the piles are determined after a stability analysis is performed on the abutments, accounting for earth loads. As before, the longitudinal force in excess of the battered resistance is split evenly to all six piles as moment.

**Moments on Piles:** The pile moments are calculated by applying the excess of the longitudinal force to the centerline of cap, multiplying by the length to fixity, and dividing by the number of piles. This produced large moments.

Since this approach does not account for soil interaction with the pile, the Florida Pier
Model (8) was used to compare a typical pile to that of the simple approach. The result was a 50% reduction in moment. This reduction was used for all piles.

**Analysis of Moment Effects on Piles:**  Louisiana DOTD Standard 24” Square PPC

Piles with the following properties are used:

–Area = 463 square inches

–Section Modulus = 2219 inches³

–Concrete f’c = 5000 psi

–Effective prestress = 814 psi after losses

–Allowable Fa = 0.4f’c = 2 ksi (AREMA (1))

–Allowable Fb = 0.4f’c = 2 ksi (AREMA (1))

–Allowable tension = -6*(f’c)0.5 = -424 psi (PCI transient load allowable (9)).

Two step Check:

• (fa/Fa) + (fb/Fb) < 1.25 (overstress allowed- AREMA Group III (1))

• (fa) – (fb) > -424 psi (Transient Tension (9))

In order to demonstrate, a possible design process going from the original design for a much lower longitudinal force to an appropriate design for this structure, several trials will be illustrated.
Trial 1

The original design for much lighter longitudinal force, had the piles battered only at the abutments. The resulting overall structure longitudinal deflection for this case is 3/16”, and the maximum moment due to longitudinal force in the abutment piles is 672 foot-kips which is too high.

Trial 2

Assume the piles battered longitudinally at the abutments and at two bents, 9 & 17. That is a maximum spacing of 347 feet. Note: This might work if the soil wasn’t of such marginal quality.

The result is a structure deflection of 1/8” longitudinally, and a maximum pile moment of 412 foot-kips due to longitudinal force, which is still too high.

Trial 3

Assume longitudinally battered piles at the abutments and also at five bents 5,9,13,17 and 22. The resulting structure longitudinal deflection is 1/16”, and the maximum pile moment is 275 foot-kips due to longitudinal force which is OK, but at the abutment the combined abutment \((fa/FA) + (fb/FB) = 1.28 > 1.25\) which is slightly over, but more important, the allowable tension in an abutment pile is not acceptable viz: abutment \((fa) – (fb) = -630 \text{ psi vs. allowable} – 424 \text{ psi}.\)
**Trial 4**

In order to eliminate piles from the main channel, the designer tried longitudinally battered piles at the abutments and at six bents 4,7,10,13,16 and 25. The resulting structure longitudinal deflection was 1/16” and the maximum pile moment was 275 foot-kips due to longitudinal force, which is OK. This represents a battered group every 150 feet. Although an acceptable solution, one more iteration was done to improve the design.

**Final Solution**

The designer increased the batter to 3 on 12, and used battered piles at the abutments and at only four bents 5, 9, 15 and 24. The longest distance between bents is now 346 feet. This is compatible with typical structures in the area designed before the 1968 reduction of longitudinal force that had spacing between longitudinal force resisting bents roughly inversely proportional to their design loading. That is E 80 / E 60 times 346 feet.

–Note: Bent 25 cannot be battered with bent 26 (abutment) because the 3 on 12 batter would create conflicts with pile lengths around 80 feet and spans of 39 feet.

**TIMBER STRUCTURES**

AREMA Chapter 7 (1), which covers timber structures, did not reduce the design longitudinal force in 1968. So their chapter has not as yet adopted the new formulae.
Nevertheless, it is expected that will occur shortly as the new formulae approximate what is in Chapter 7, Article 7.2.5.5.4, and is easier to use.

One point about timber railroad bridges, that is different from say steel structures, is that the bracing cannot carry longitudinal forces, due to the capacity of the connections and L/d ratios. Article 7.2.5.5.4 b states this very clearly. Such bracing is to ensure the L/d stability to the posts. In fact on some railroads, particularly on the IC/CN, which crosses so many tributaries of the Mississippi, many structures over 20 feet high have no longitudinal bracing. This is because debris would wipe them out all too frequently. There is no longitudinal timber bracing.

In a structure such as the Mile Long Bridge (See photo below) the longitudinal force goes...
from stringers to cap to stringers to abutments or similar strong points. The longitudinal forces in this bridge are taken by the deck to the concrete piers that act as a couple to take the loads to ground. Near the ends, the longitudinal force is taken out via the dump wall (abutment wall).

STEEL BRIDGE RATING FOR LONGITUDINAL FORCE

The proportioning and distribution of forces is similar to the concrete example given previously: that is distributed in proportion to stiffness.

The first step is to determine from the standard rating (I) the equivalent Cooper’s E Loading (E-60, E 80, ?) to be used for Longitudinal force evaluation as indicated in Article 15.7.3.3.8 a (I). Then derive the corresponding longitudinal force

Is high Longitudinal Force likely?

A couple of questions need to be answered:

• Is the structure located where the speed is likely to be below 25 mph?
• Is the structure located on or near ruling grade or a momentum grade?
• If the answer to the two questions above is no, then apply Article 15.7.3.3.8 c (I) and
• Check for Article 15.7.3.3.8 f. (I) as shown below.

If the rating is restricted to apply for only a few years, the 25 mph above could be
lowered to whatever value the railroad mechanical or transportation Engineer deems appropriate for all company owned and foreign locomotives operating on or likely to operate on the line. This might be as low as 12 mph. When the technology of the locomotives used catches up, the rating would have to change.

**High Longitudinal Force not likely**

- Use Article 15.7.3.3.8 c (I).
- A. Determine typical traction force likely at that location (get help from railroad mechanical or transportation engineer).
- B. Ascertain Maximum Traction force of highest adhesion locomotive run on system (Must include foreign units).
- Apply ratio (A/B) to the appropriate rating longitudinal force

*Example*

- Suppose same location as concrete example
- Controlling longitudinal force was 1203 kips for E- 80 loading.
- Suppose Normal Rating for a steel structure at that location was E 60.
- Then by Article 15.7.3.3.8a (I), longitudinal force is reduced to 1203 *60/80 = 902 kips for normal rating.
Suppose at this location, baring derailment or rare emergency, freight trains are expected to move at 60 MPH, and the largest locomotive on the system is a foreign locomotive that is an AC 90 Max. Given that speed and that location, the Mechanical Officer says maximum traction typically will be less than 10% of the maximum the unit is capable of generating.

Then, the longitudinal Force for this location for rating becomes 10% of 902 kips or 90 kips. But! Article 15.7.3.3.8 f (I) requires a check at full design (read normal rating in this case) longitudinal force at 1.5 x allowable stresses. Thus 902 kips at 1.5 x allowable stresses (in this case at 0.55 \( F_y \times 1.5 = 0.825F_y \) where \( F_y \) is the yield point).

**Load Combinations (I)**

- Longitudinal Force + Wind + Lateral force from equipment and force between bracing members are checked at the appropriate allowable stress.
- When longitudinal force is combined with Dead, Live, Impact and centrifugal force, if applicable, use 1.25 x appropriate allowable stress.

**Places where full Longitudinal Rating Force must be used**

If Article 15.7.3.3.8 b (I) applies use the full force, which in the previous example would be 902 kips, were:

- Where speed is likely to be less than 25 mph and maximum tractive effort is likely. For diesel-electric units in use today the 25 mph is a bit excessive and in the short term a lower number, say 12 mph, could be used –Note: Max effort is applied at start up on these units.
Where maximum braking effort is likely to be used to hold train speed

Where trains are likely to stop

Where is this likely?

- Meet and pass locations
- Ruling grades
- Momentum Grades
- On long grades
- Certain Siding and spur locations
- Where trains wait for clearance

What to do if the structure doesn’t rate for longitudinal force.

- Modern locomotives have dial gages with a read out of longitudinal force. One railroad limited such force on critical bridges to 100 kips total.
- Another railroad limited the application of dynamic brake to half the unit’s capability.
- Another railroad prohibited the use of the load/empty brake on certain runs.

FAILURE EXAMPLE

The photograph below shows an example of a failure due to longitudinal force, where the local effect of traction needed to be considered. The structure spans the double track main line and maintenance roadways of another railroad with a center to center of bearings of 72.25 feet. In order to get better clearance, the design consists of a steel plate
deck with direct rail fixation to the plate which is attached to 40+ transverse floor beams at 1.77’ spacing. The span is skewed at both ends. There is a gap between the floor plate and the outside through plate girders. There is a channel down the under-centerline of the steel floor plate, which does not extend to the abutments, to distribute forces.

The structure was built for E 70 loading in 1963 using a design longitudinal force of slightly more than half that specified in the AREA Manual of the day. The load used was 74.5 kips which is not too far off of an SD-40 power unit at maximum traction (80 kips).

Longitudinal force was not explicitly considered in the design of the deck as it was assumed that the traction force would be evenly distributed over the entire deck plate, and then, since the load to each floor beams would be so small, it could be transferred by torsional moments or jump from the deck plate to the floor beams to the bearings. No calculation was made.

Under this assumption the load per floor beam per side would have been Force ÷ number of floor beams ÷ two sides = 74.5kips/40/2 = 0.93 kips applied 3 feet above the deck which the 8 rivets should have been able to handle. The skewed floor beams have been ignored in this calculation.

When cracking started, a set of traction braces were installed at the plane of the bottom of the floor beams to take the loads to the bearings. Unfortunately, this did nothing for the
torsion in the floor beams.

Sadly, the bridge is on the ruling grade between two cities roughly 500 miles apart and moderately high traction occurred frequently.

The assumption that the longitudinal traction force would be uniformly distributed over the entire deck proved not correct. The traction force on typical locomotives operating over this bridge was applied by each truck over a distance of roughly 13 feet, and the longitudinal force applied by each wheel when applied to a floor beam would be even higher locally.

The load should have been calculated per floor beam, (Article 15.1.3.4.1b (I) applies as
this is not a ballasted deck, so there is no longitudinal distribution of wheel load), in
which case even using half the AREA design per connection of longitudinal force over
each floor beam would have been 25% of 70 kips/2/2 = 4.375 kips using the designer’s
method which is far too much torsion for this connection. A more precise modeling of
the wheel load distribution might produce a lower number, but it would still be too high a
load for the connection to withstand, and it would not be in accordance with the AREMA
Manual Recommendations (1).

After 43 floor beams had cracked due to torsional fatigue, having sustained an average of
about 45 mgt per annum, the decision was made to replace the structure after 37 years of
service as it was cheaper than repairs.

CONCLUSIONS

The key to dealing with longitudinal force for design is to determine the resulting load
path due to stiffness and check each component on the way. In the end the load must be
taken through and out of the structure.

For rating, the key is to determine the likelihood of maximum longitudinal force – most
locations are likely to have lower than the maximum.

Timber bridges designed for the full longitudinal force criteria of Chapter 7 (1) should
not present a problem up to their original design load.
Most concrete and steel bridges designed for the full longitudinal force criteria of Chapters 8 and/or Chapter 15 before 1968, but after roughly 1932 (1,4), should not be a problem up to their original design load.

Most steel structures designed for the full longitudinal force criteria of Chapter 15 between 1905 (1,4) and 1932 will be within 80% of the current criteria up to their original design load.

Short spans and ballast deck spans might be an exception to the above.
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