Development of Crash Wall Design Loads from Theoretical Train Impact

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ABSTRACT

Crash walls may be required for the protection of overhead structures, and in some cases the Railway may consider a crash wall as an alternative to an earthen berm for the protection of structures or facilities adjacent to the track. There is, however, a lack of direction available for the design of such walls. AREMA provides minimum dimensions for crash wall protection of a pier supporting a bridge over a railway, but does not specify design loads; AASHTO design loads are only for tractor-trailer collision forces.

Considering basic principles of physics and behaviours exhibited by derailed trains, an energy balance approach was used to determine a reasonable impact load for the design of a crash wall. Two possible modes of impact were considered: a glancing blow from a derailed train running in the direction of the track; and a direct impact as a result of cars piling up in an accordion fashion. The results were compared with previously used guidelines. Comments were solicited from Canadian National, Canadian Pacific Railways, and GO Transit, the regional rail transit system operator in the Toronto Area. Their input was incorporated in the final design guideline which is now being provided to consultants submitting to those railways.

This paper reviews the development of the design equations for both modes of impact, and the resulting design guidelines. Minimum requirements for crash walls are included.

1. BACKGROUND

Railway tracks are frequently constructed under overhead structures or adjacent to developments or public areas, and protection against damage to those facilities in the event of a derailment may be required. The use of an earthen berm to provide this protection is not always practical, and so a reinforced concrete wall, a crash wall, may be used.

In August, 2012, a crash wall was required by GO Transit in Toronto, Canada, for the protection of a structure adjacent to a railway track. The designer applied the energy balance approach outlined in the 2011 Submission Guidelines for Crash Walls provided by AECOM (1), but found the results of the design to be unreasonable. The question was raised at AREMA Committee 8 meetings, and the consensus was that AREMA has not and is not likely to specify design loads for crash walls in the foreseeable future.

Given the requirement to provide functional guidelines for our clients, this prompted a thorough review of the design guidelines issued by AECOM. The intent was to evaluate whether the 2011 AECOM guidelines provide reasonable and justifiable design principles, and if there were refinements that could be made. On completion of the review, the design loads of the energy balance approach were compared to the findings of Hirsch (2) and design requirements by AREMA (3), and AASHTO 2010 (4) and 2012 (5). Finally, a set of suitable design guidelines were developed that may be applied to future projects. These new guidelines meet the requirements of AREMA, CN, and CP, and provide a reasonable and functional approach to this common design question.
2. THE ENERGY BALANCE APPROACH

2.1 Design Intent
The intent of the crash wall is to limit damage to adjacent structures by absorbing a portion of the collision force, and redirecting or deflecting derailed railroad equipment. In the case of overhead bridges, the supplementary goal is to avoid structural collapse on the tracks.

2.2 The energy balance
The 2011 AECOM design guidelines are based on the principle of an energy balance. This assumes the collision stops all movement in the direction perpendicular to the wall, such that

\[ Fd = \frac{1}{2} m v^2 \]  

[1]

In words: the force of the collision times the distance through which that force acts (in this case the deformation of the train, since the deflection of the wall is considered negligible) equals the kinetic energy the train had before the collision in the direction perpendicular to the wall, \( \frac{1}{2} mv^2 \), where \( m \) is the mass of the derailed consist and \( v \) is the velocity at impact.

2.3 Specified constants
Several factors given in the 2011 AECOM Guidelines were first reviewed to ensure their practicality.

The specified consists were:
1. a freight train - three 200 ton locomotives and six 143 ton (286 kip) cars,
2. a passenger train – two 148 ton locomotives and six 74 ton passenger cars.

Kehnemui (6) states that “damaging collisions due to derailments usually involve uncontrolled box cars and passenger cars much more so than the locomotives”. This sentiment is also expressed by Hirsch et al. (2). While these statements warrant further review for currency, the concept was adopted and the consists changed to nine freight cars or eight passenger cars, eliminating the inclusion of locomotives but retaining the number of cars as there was no evidence to dispute those numbers. Car weights are considered accurate, and the length of freight cars set to 56 feet (17 m) which is the average in use on the CN system, and to 85 feet (26 m) for passenger cars.

Also in the 2011 Guidelines, the height of application of the impact force is set at 3 feet (0.9 m) above the ground. Again this value may be confirmed by further study, but in light of the following references, the height of impact was revised to 6 feet (1.8 m) above the adjacent groundline.

- Hirsch et al. (2) found the center of gravity of common rail cars to be at 5’-6” (1.7 m) above the rail.
- AREMA (3) requires the minimum wall height to be a minimum of 6 feet (1.8 m) above the top of rail. Presumably this is to prevent the train from rolling over the top of the wall, assuming the center of gravity will be at or below that height.
- CN’s Protection and Minimum Clearances for Overhead Bridges (7) requires the minimum wall height to be 7 feet (2.135 m) above the top of rail.
- CP guidelines (8) require that the design load be applied 5’-8” (1.8 m) above the adjacent ground elevation.

Given the AREMA requirement for a crash wall to extend 4 feet (1.2 m) below grade (3), and assuming a footing of 32” (0.8 m) thickness, the distance from the point of impact to the top of the footing was taken as 7’-3” (2.2 m) for comparative calculations. Also for comparative purposes, loads are assumed to be distributed along the wall over a length equal to the length of impact plus the height of impact above the base of wall in either direction; in other words, at a 1:1 slope from the end of the impact length. Impact must be considered at any point along the wall. The validity of this distribution was confirmed with a finite element model of a wall 2’-6” (0.76 m) thick.
The energy balance equation depends on the distance through which the impact force travels. The 2011 AECOM Guidelines set the plastic deformation of an individual car due to direct impact as 1 foot (0.3 m) maximum. The total compression of linkages and equipment for either the freight train or passenger train consist is 10 feet (3.048 m) maximum. These values were retained for lack of any better information, and may be a subject for further study.

3. DESIGN LOAD CASES FOR THE ENERGY BALANCE APPROACH

3.1 Glancing Blow

A train, or portion of a train, may jump the track and continue to travel in the direction of the track, but drift sideways and into an adjacent structure - in this case a crash wall. Under this scenario, the movement of the train is primarily in the direction of initial travel, and parallel to the crash wall.

Forces due to the pull of the ground on the wheels, centrifugal forces, lateral impacts including those that may have caused the derailment in the first place, or loads shifting or not remaining centered over the wheels, may all cause lateral forces, and therefore lateral movement of the cars. These forces are all but impossible to quantify, and so the resulting velocity of the car in the direction perpendicular to the track is very difficult to determine.

By the 2011 AECOM guidelines, when calculating the design force from the energy balance equation in 2.2, the velocity of the train at impact was to be taken as track speed, and the angle of impact as $10^\circ$.

3.1.1 Velocity

Considering Figure 1, it can be seen that the component of velocity parallel to the wall is approximately equal to the velocity of the train for a small angle, $\theta$, which is assumed to be the case for the glancing blow. The velocity of the train, however, will begin to decrease immediately on derailing, under a deceleration not accounted for in the original AECOM guidelines.

![Figure 1](https://example.com/figure1.png)

Figure 1  Velocity parallel and perpendicular to the crash wall

No documented test program could be found that quantified the deceleration of a derailed train. In attempt to approximate the deceleration, the principles of highway runaway lane design were applied. A runaway lane aims to slow a vehicle by placing loose material across the lane to offer rolling resistance, and sometimes by having the lane go up a steep grade. The resistance of the material is expressed in terms of an equivalent percent grade, R, plus the grade of the land, G, and the length of the lane required to stop a vehicle on the runaway lane is
\[
L = \frac{v^2}{30(R + G)} \quad \text{[2]}
\]
\[
L = \frac{v^2}{254(R + G)} \quad \text{[2M]}
\]

Where \(L\) is the length to stop in feet (m) and \(v\) is the entry speed of the vehicle in mph (km/h) (9), (10).

Assuming a constant rate of deceleration, basic equations of physics were used to determine the rate of deceleration given an initial velocity and the calculated stopping distance.

From the geometry of the arrangement, the distance the train would have to travel prior to impact was determined and the final velocity at impact was then calculated. Hence, it required the introduction of the variable \(d_{CL}\), distance from centerline of track, which previously hadn’t been included in the guidelines.

The resulting equation for velocity of the train at impact, \(v_G\) is

\[
v_G = \sqrt{v_0^2 + 2a \left(\frac{d_{CL} - 5.331}{\sin \theta_G}\right)} \quad \text{[ft/s]} \quad \text{[3]}
\]
\[
v_G = \sqrt{v_0^2 + 2a \left(\frac{d_{CL} - 1.625}{\sin \theta_G}\right)} \quad \text{[m/s]} \quad \text{[3M]}
\]

Where
- \(d_{CL}\) is the distance from the crash wall to the centerline of track in feet (m).
- \(v_0\) is the track speed in ft/s (m/s)
- \(a\) is the acceleration in ft/s², calculated as \(-32(R + G)\)
  (in metric, acceleration is in m/s², calculated as \(-9.8(R + G)\))
- \(\theta_G\) is the angle of impact

The \(R\) value was selected as 0.25, which is the equivalent of truck tires through pea gravel. While it remains somewhat arbitrary and could warrant additional investigation, the use of \(R = 0.25\) can be validated by Hirsch et al. (2) which found a derailed train on level grade stopped in a distance of 290 feet (88 m) from the point of derailment when initially travelling 50 mph (80 km/h). Assuming a constant rate of deceleration, that deceleration would be approximately -8.2 ft/s² (-2.5 m/s²). A close approximation of this acceleration is obtained from the above equation with \(R = 0.25\). The grade, \(G\), can be generally taken as zero, as the short distances involved and the common presence of a ditch along the tracks will minimize the effect of this factor.

### 3.1.2 Angle of impact

Again referring to Figure 1, it can be seen that the component of force perpendicular to the wall is highly dependent on the angle of impact, \(\theta_G\). By the 2011 AECOM Guidelines, \(\theta_G\) is set to be 10 degrees; however, a 10 degree angle is actually quite a steep angle, and may not represent the train sliding along the wall, particularly where the wall is very close to the tracks.

Consider the geometry of a derailed car shown in Figure 2. If it is assumed that the rear wheels of the car do not move beyond the outside rail of the track, the maximum angle, \(\theta_G\), at which the train can impact the wall is a function of the length of car and the distance from the centerline of track to the face of the wall.
For a standard 56 foot (17 m) freight car, the distance from back axle to front of the car is 51'-6" (15.7 m). For this length of car, this maximum angle is shown in Figure 3.

For impact shortly after derailment, where the crash wall is very close to the tracks, the angle of impact may be dictated by the geometry in Figure 2. As the train travels farther after derailment, the glancing blow mode of impact becomes less about the leading car, and more about the forward momentum of the train as it travels largely parallel to the crash wall. From Hirsch et al (2), the first derailed car impacts the wall 10 feet (3.0 m) from the centerline at 177 feet (54 m) from the point of initial derailment, or at an overall angle of motion of 3.2 degrees. The limiting value of the angle of impact was therefore conservatively selected to be 3.5 degrees.

Fitting a simple linear approximation to the data points in Figure 3, and to a similar plot for passenger cars 85 feet (26 m) long, the angle of impact for the glancing blow can be estimated as:

\[
\theta_G = 1.2d_{CL} - 6.2 \leq 3.5^\circ \text{ for freight trains} \quad \text{[4]}
\]
\[
\theta_G = 3.8d_{CL} - 6.2 \leq 3.5^\circ \text{ for freight trains} \quad \text{[4M]}
\]
\[
\theta_G = 0.8d_{CL} - 4.3 \leq 3.5^\circ \text{ for passenger trains} \quad \text{[5]}
\]
\[
\theta_G = 2.6d_{CL} - 4.3 \leq 3.5^\circ \text{ for passenger trains} \quad \text{[5M]}
\]

where \(d_{CL}\) is the distance from the crash wall to the centerline of track in feet (m). Given that the angle will be limited to 3.5 degrees for all but the longest cars and the closest track spacing, the value of 3.5 degrees may be used conservatively in lieu of calculating the impact angle.
3.1.3 Length of action of impact force

The length of wall, $l_G$, along which the impact force should act was calculated from the length of deformation specified by the 2011 AECOM guidelines and the angle of impact as shown in Figure 4:

$$l_G = \frac{10}{\cos \theta_G}$$

[6]

$$l_G = \frac{3.048}{\cos \theta_G}$$

[6M]

where $l_G$ is in feet (m). For an angle of 3.5 degrees, the length along which the force acts is 10 feet (3.1 m). Due to the forward momentum of the train, it is likely that the length of impact along the wall is still being conservatively estimated.
3.2 Single Car Impact

The single car impact load case considers the observed derailment behaviour of an “accordion style” pileup. In these derailments, cars move with a combination of forward and angular momentum, and the angular momentum component may cause an impact force normal to the face of the crash wall.

In order to calculate the angular momentum of a single car at impact, several simplifying assumptions were made:

- The center of gravity of the car stays approximately over the centerline of track.
- All cars in the accordion rotate with approximately the same angular momentum.
- The accordion is caused by the front of the train decelerating faster than the rear. The difference in deceleration of the front of one car relative to the rear of the next was taken to be -8.2 ft/s² (-2.5 m/s²) as discussed in 3.1.1 using the values of R=0.25 and G=0.

3.2.1 Velocity at impact

With these assumptions, the velocity of the end of the derailed car, \( v \), and the angle of impact, \( \theta \), were calculated from the geometry of the arrangement shown in Figure 5.

![Figure 5 Geometry of the accordion style derailment](image)

The time required to move through \( dx \) is calculated from the difference in acceleration of the two points A and C, taken as 8.2 ft/s² (2.5 m/s²) and 0 ft/s² (0 m/s²) respectively. This time is the same required for the movement through the angle \( \theta \), giving an approximate angular velocity. The angular velocity is then translated into an impact velocity, \( v \), as the product of angular velocity and radius of rotation, which is half the length of the car. Because \( v \) is directly proportional to the radius of rotation, the specified car lengths of 56 feet (17 m) for freight and 85 feet (26 m) for passenger cars were used.

The resulting equations for velocity of the train at impact, \( v_A \), is

\[
v_A = \frac{7.6\theta}{\sqrt{1 - \cos \theta}} \quad \text{[ft/s] for freight cars} \quad [7]
\]

\[
v_A = \frac{2.3\theta}{\sqrt{1 - \cos \theta}} \quad \text{[m/s] for freight cars} \quad [7M]
\]

\[
v_A = \frac{9.4\theta}{\sqrt{1 - \cos \theta}} \quad \text{[ft/s] for passenger cars} \quad [8]
\]
\[ v_A = \frac{2.9\theta_f}{\sqrt{1 - \cos \theta_f}} \] [m/s] for passenger cars \[8M\]

Where \( \theta_f \) is the angle of impact, in radians, shown in Figure 5.

### 3.2.2 Angle of Impact

Theoretically, the angle of impact could be 0°, but only if the wall was immediately beside the railway car as it began its rotation. The further the wall is from the track, the larger the angle \( \theta_f \) before impact and the lower the perpendicular design force. Angle of impact must therefore be calculated from the geometry of the arrangement, and is again a function of the distance from the centerline of track to the face of the wall.

\[ \theta_f = \arcsin \left( \frac{d_{CL}}{28} \right) \] for freight cars \[9\]

\[ \theta_f = \arcsin \left( \frac{d_{CL}}{8.5} \right) \] for freight cars \[9M\]

\[ \theta_f = \arcsin \left( \frac{d_{CL}}{42.5} \right) \] for passenger cars \[10\]

\[ \theta_f = \arcsin \left( \frac{d_{CL}}{13.0} \right) \] for passenger cars \[10M\]

Where \( d_{CL} \) is the distance from the crash wall to the centerline of track in feet (m).

### 3.2.3 Length of action of impact force

The length of impact was again calculated from the length of deformation specified by the guidelines and the angle of impact as shown in Figure 5, only in this case the deformation of the car is 1 foot (0.3048 m) and the angle used is \( \theta_f \)

\[ l_A = \frac{1}{\sin \theta_f} \] [ft] \[11\]

\[ l_A = \frac{3048}{\sin \theta_f} \] [m] \[11M\]

This has the effect of increasing the length of impact from a point load to 3 feet (0.9 m) length for a freight car against a wall at 9'-10" (3.0 m) from the centreline of track; and to 4'-3" (1.3 m) for a passenger car.

### 3.2.4 Two cars in accordion

Sections 3.2.1 to 3.2.3 assume that the two sequential cars in the accordion impact the wall far enough apart that the area of influence along the base of the wall does not overlap.

Conservatively assuming the cars stay coupled, and therefore the space between the points of impact of two sequential cars is a minimum, the value \( s \) can be introduced to define the distance between the impact points as shown in Figure 6. Given an \( s' \) value of approximately 3'-4" (1.0 m) and a car width of 10'-8" (3.25 m), the value of \( s \) can be defined as

\[ s = 2 \left[ \frac{10.7}{2} \sin \theta_f + 1.6 \cos \theta_f \right] \] [ft] \[12\]

\[ s = 2 \left[ \frac{3.25}{2} \sin \theta_f + 0.5 \cos \theta_f \right] \] [m] \[12M\]

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Taking a load distribution from the elevation of the impact to the base of wall at ground level of 1:1, the wall must be designed for a moment at the base of the wall distributed over the length shown in Figure 7. Given a height of application of 7’-3” (2.2 m) above the top of footing, the case of two impact loads with an overlapping length of application will govern the design.

### 3.3 Minimum Wall Thickness

The CP Requirements for Protection of Structures Adjacent to Railroad Tracks includes a minimum thickness requirement: “crash walls 8 m or less from the centerline of the nearest existing or proposed track shall be at least 750 mm thick… [and] walls greater than 8 m… shall be at least 450 mm thick” (8). AREMA requires pier protection crash walls to be at least 2’-6” (0.76 m) thick regardless of the CL track to wall distance (3).

Forces along the length of the wall, $F_{II}$, are a result of friction between the derailed train and the wall

$$F_{II} = F_{I} \times \mu$$

Where $F_{I}$ is the force for each load case,

$\mu$ is the coefficient of friction, taken as 0.45 for concrete to steel (11), (12)

Providing a minimum wall thickness of 2’-6” (.76 m) for walls 25’ (7.6 m) or less from the centerline of track, or 18 inches (.45 m) for walls greater than 25’ (7.6 m) away, will ensure the quantity of reinforcing steel required to resist longitudinal forces will be less than that required for the resistance of impact forces; in essence, that the lateral impact resistance governs the crash wall design.
4. COMPARISON OF DESIGN GUIDELINES

4.1 Energy Balance Load Cases

A summary of the required design moment at the base of the wall for each of the four load cases in the energy balance approach is presented in Figure 8 (see 2.3 for geometry of comparative calculations). The case shown is for a freight train speed of 50 mph (80 km/hr) and passenger speed of 70 mph (112 km/hr). While changing the speed does not significantly change the shape of the curves, it may change the relative position so that any one of the load cases may govern for a range of track speeds and distances from the centerline of track.

It should also be noted that in reality, the glancing blow and single car impact cases will not be exclusive: a derailed train will have both forward momentum and rotational motion in some combination. The objective here is to determine a probable upper bound load for design that will allow the wall to meet the requirement of limiting damage to adjacent structures and redirecting the moving railcars.

4.2 Comparison of alternative design criteria

One of the most detailed reviews of design loads available is that published by Hirsch et al. (2). In that study, a computer model was used to solve for the impact load of derailed freight trains against a crash wall at varying distances from the centerline of track. The model railcar was a 260kip car. Figure 9 shows the resulting impact loads for an initial train speed of 50mph (64 km/h) with results scaled by a factor of 286/260. Also on the plot is the governing design curve from the energy balance equations, also based on a train speed of 50 mph (80 km/h) for freight, and with a passenger train speed of 70 mph (112 km/hr).
In the 2010 edition, AASHTO required protection walls to be designed for a load of 400 kip (1800 kN), acting in any direction at a height of 4 feet above the ground (4). This load was the same for highway vehicle or railway collision loads, and referenced Hirsch et al. (1989) as the source of that load. The Hirsch paper did in fact recommend a design load of 400 kip (1800 kN), but based on 260 kip loading and for the ideal distance from centerline of track of 10 feet (3.0 m). Peak design forces exceeded 400 kip (1800 kN) for distances of 12 feet to 26 feet (3.7 m to 7.9 m) from centerline of track, which are in fact the most likely locations for these walls to be located.

In 2012, the AASHTO requirement for protection wall design loads from highway vehicle collisions increased to 600 kip (2700 kN) at 5 feet (1.5 m) above ground (5). The specification for train collision loads found in previous editions has been removed and replaced with reference to AREMA requirements or railway company guidelines. Given the lack of direction from AREMA and some railway companies, some states have adopted the 600 kip (2700 kN) load (13). The implications of the 400 kip (1800 kN) and 600 kip (2700 kN) design loads, applied at the selected height of 6 feet (1.8 m) are also shown in Figure 9.

At track speeds higher than 50 mph (80 km/hr) for freight and 70 mph (112 km/hr) for passenger trains, the energy balance equations yield design moments higher than those required for the 600 kip (2700 kN) design load. Further consideration to impact loading from higher speed trains may be a subject of further study.

4.3 Requirements for provision of crash walls

Given the required clearance envelope for railway construction, no crash wall can be built closer than 9 feet (2.7 m) from the centerline of track.
In Canada, the minimum distance from the centerline of track to the face of an abutment or pier is 18 feet (5.486m) (14).

Per AREMA 2.1.5, crash walls or heavy piers capable of achieving the same objective are required where piers are closer than 25 feet (7.6 m) from the centerline of track (3). Further provisions state that consideration may be given to providing protection for piers farther than that distance if conditions warrant.

CP’s Protection of Structures Adjacent to Railway Tracks states that protection is required for overhead bridge piers when they are up to 26 feet (8.0 m) away from the centerline of track, and that protection is required for all other structures up to 49 feet (15.0 m) away from the centerline of track (8).

Considering Figure 9 for the range of distance from centerline of track of 9 feet (2.7 m) to 25 feet (7.6 m) from the centerline of track, it is seen that a design load of 600 kip (2700 kN) is expected to be sufficient for walls within this range.

Crash walls should be required until a distance where the threat of collision is acceptably small. Beyond 25 feet (7.6 m), the design load of 600 kip (2700 kN) may be overly conservative, as shown in Figure 9. At this distance, the energy balance equations may be used to determine a reasonable design load.

The glancing blow calculations can also predict whether the train is expected to stop its forward momentum prior to reaching the proposed crash wall location, taking into account and the slope of the ground away from the track. As an example, applying the energy balance equations to a track on level grade with passenger train speed of 70 mph (112 km/hr), it is found that design impact loads are greater than zero to a distance of 44 feet (13.4 m).

5. RECOMMENDED DESIGN GUIDELINES

For the design of a crash wall adjacent to a railway track, one of the following methods may be chosen, or an alternative design load may be selected if it can be justified by the engineer responsible for the design. The simplified approach of Method 1 may be used in many cases. Method 2 may be used to optimize the design, or where factors such as distance from the track to the wall, track speeds, side slopes along the track, and consequences of collision may justify a different load.

5.1 Method 1

The wall shall be designed for a minimum point load of 600 kip (2700 kN) applied horizontally and normal to the face at any point along the wall. The point load shall be applied at a height of 6 feet (1.8 m) above the top of rail for walls up to 25 feet (7.6 m) from the centerline of track, or a height of 6 feet (1.8 m) above the groundline for walls farther than 25 feet (7.6 m) from the centerline of track. This method may be applied where track speeds do not exceed 50 mph (80 km/hr) for freight or 70 mph (112 km/hr) for passenger trains; where speeds exceed these limits, Method 2 shall be used.

5.2 Method 2

An energy balance approach considering collision by glancing blow and single car rotation may be used to determine the design load for a wall at a distance $d_{cl}$ from the centerline of track in feet (m). The closest existing or future/proposed track is to be used. The following four cases shall be considered:

- **Freight Train Load Case 1 - Glancing Blow:** nine cars weighing 143 tons (129 700 kg) each, impacting the wall at an angle $\theta_r$. The angle of impact will be a function of track curvature, and for tangent track may be taken as 3.5 degrees.

- **Freight Train Load Case 2 - Single Car Impact:** single car weighing 143 tons (129 700 kg) impacting the wall as it undergoes rotation about its center. The angle of rotation at impact is defined in [9]:

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\[
\theta_f = \arcsin \left( \frac{d_{CL}}{28} \right)
\]

\[
\theta_f = \arcsin \left( \frac{d_{CL}}{8.5} \right) \quad \text{(metric)}
\]

Where \( d_{CL} \) is in feet (m). Where \( d_{CL} \) is greater than 28 feet (8.5 m), this load case need not be considered.

**Passenger Train Load Case 3 - Glancing Blow:** eight cars weighing 74 tons (67120 kg) each impacting the wall at an angle, \( \theta_G \). The angle of impact will be a function of track curvature, and for tangent track may be taken as 3.5 degrees.

**Passenger Train Load Case 4 - Single Car Impact:** single car weighing 74 tons (67120 kg) impacting the wall as it undergoes rotation about its center. The angle of rotation at impact is defined in [10]:

\[
\theta_f = \arcsin \left( \frac{d_{CL}}{42.5} \right)
\]

\[
\theta_f = \arcsin \left( \frac{d_{CL}}{13.0} \right) \quad \text{(metric)}
\]

Where \( d_{CL} \) is in feet (m). Where \( d_{CL} \) is greater than 42'-6" (13 m), this load case need not be considered.

Where a track is designed for dedicated service by a particular train consist, variations to the design trains may be permitted by the Railway.

### 5.2.1 Speed

The analysis should reflect the specified track speeds for passenger and/or freight trains applicable within the subject corridor. For the glancing blow load cases, the speed of derailed equipment impacting the wall is reduced from the track speed, \( v_G \), as per [3]

\[
v_G = \sqrt{v_o^2 + 2a \left( \frac{d_{CL} \sin \theta_G}{\sin \theta_G} \right)} \quad \text{[ft/s]}
\]

\[
v_G = \sqrt{v_o^2 + 2a \left( \frac{d_{CL} \sin \theta_G}{\sin \theta_G} \right)} \quad \text{[m/s]}
\]

Where \( d_{CL} \) is the distance from the crash wall to the centerline of track in feet (m).

\( v_o \) is the track speed in ft/s (m/s)

\( a \) is the acceleration in ft/s\(^2\), calculated as \(-32(0.25 + G)\)

(in metric, acceleration is in m/s\(^2\), calculated as \(-9.8(0.25 + G)\))

\( \theta_G \) is the angle of impact defined in [4] or [5]

\( G \) is the grade in decimal unit of the groundline in the direction of travel defined by the angle of impact relative to the centerline of track; calculated as \( \frac{\text{Groundline at wall - Base of Rail}}{d_{CL} \sin \theta_G} \).

For the single car load cases, the speed of derailed equipment impacting the wall is defined in [7] and [8]:

\[
v_A = \frac{7.6 \theta_f}{\sqrt{1 - \cos \theta_f}} \quad \text{[ft/s] for freight cars}
\]

\[
v_A = \frac{2.3 \theta_f}{\sqrt{1 - \cos \theta_f}} \quad \text{[m/s] for freight cars}
\]

\[
v_A = \frac{9.4 \theta_f}{\sqrt{1 - \cos \theta_f}} \quad \text{[ft/s] for passenger cars}
\]
\[ v_A = \frac{2.9\theta_f}{\sqrt{1 - \cos \theta_f}} \text{ [m/s] for passenger cars} \]

Where \( \theta_f \) is the angle of impact, in radians, defined in [9] and [10].

### 5.2.2 Design Forces

In lieu of more rigorous analysis, these energy balance equations may be used to determine the design load perpendicular to the wall. Deflection of wall is considered negligible in equations [14] and [15]. Where the designer wishes to include it, those equations may be modified.

For the glancing blow load cases

\[
F_G = \frac{\frac{1}{2}m(v_G \sin \theta_G)^2}{32.17d_G} \quad \text{[14]}
\]

\[
F_G = \frac{\frac{1}{2}m(v_G \sin \theta_G)^2}{d_G} \quad \text{(metric)} \quad \text{[14M]}
\]

And the load is considered to act along the length \( l_G \) in feet (m) per [6]:

\[
l_G = \frac{10}{\cos \theta_G} \quad \text{[6]}
\]

\[
l_G = \frac{3.048}{\cos \theta_G} \quad \text{(metric)} \quad \text{[6M]}
\]

Where \( m \) is the mass of the derailed cars in lbm (kg).

\( v_G \) is the impact speed in ft/s (m/s), defined in [3]

\( \theta_G \) is the angle of impact defined in [4] or [5]

\( d_G \) is the deformation of the consist in the direction of the applied force, and \( d_G = 10 \sin \theta_G \), in feet 

\( d_G = 3.048 \sin \theta_G \), in m)

For the single car impact

\[
F_A = \frac{\frac{1}{2}m(v_A \cos \theta_f)^2}{32.17d_A} \quad \text{[15]}
\]

\[
F_A = \frac{\frac{1}{2}m(v_A \cos \theta_f)^2}{d_A} \quad \text{(metric)} \quad \text{[15M]}
\]

And the load is considered to act along the length \( l_A \) in feet (m) per [11]:

\[
l_A = \frac{1}{\sin \theta_f} \quad \text{[11]}
\]

\[
l_A = \frac{3.048}{\sin \theta_f} \quad \text{(metric)} \quad \text{[11M]}
\]

Where \( m \) is the mass of the derailed cars in lbm (kg).

\( v_A \) is the impact speed in ft/s (m/s), defined in [7] or [8]

\( \theta_f \) is the angle of rotation at impact defined in [9] or [10]

\( d_A \) is the deformation of the consist in the direction of the applied force, and \( d_A = 1.0 \cos \theta_f \), in feet

\( d_A = .3048 \cos \theta_f \), in m

Where the influence areas of two sequential cars in this accordion style of derailment overlap, the wall must be designed for the simultaneous impact of both cars.
5.3 Minimum Requirements

Regardless of the method selected, the following guidelines must be followed:

- The minimum thickness for walls up to 25 feet (7.6 m) from the centerline of track shall be 2’-6” (0.760 m); minimum thickness for walls farther than 25 feet (7.6 m) from the centerline of track shall be 18 inches (0.45 m).
- Crash walls less than 12 feet (3.6 m) from the centerline of track shall be a minimum of 12 feet (3.6 m) above the top of rail. Crash walls between 12 feet (3.6 m) and 25 feet (7.6 m) from the centerline of track shall be a minimum of 7 feet (2.135 m) above the top of rail. Crash walls greater than 25 feet (7.6 m) from the centerline of track shall be a minimum of 7 feet (2.135 m) above the adjacent groundline.
- The face of the crash wall shall be smooth and continuous, and shall extend a minimum of 6 inches (0.15 m) beyond the face of the structure (such as a building column or bridge pier) parallel to the track.
- The design must incorporate horizontal and vertical continuity to distribute the loads from the derailed train.
- The wall must be of solid, heavy construction, and separate precast blocks or stones will not be permitted.

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References

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List of Figures

Figure 1  Velocity parallel and perpendicular to the crash wall
Figure 2  The maximum angle of impact is a function of car geometry and the distance from the centerline of track to the crash wall
Figure 3  Maximum angle of impact as a function of distance from centerline of track to wall for a 56 foot (17 m) freight car
Figure 4  Length along which impact force acts
Figure 5  Geometry of the accordion style derailment
Figure 6  Distance between points of impact of sequential cars
Figure 7  Distribution of impact force along the base of the wall
Figure 8  Comparison of energy balance design loads by load case
Figure 9  Comparison of design criteria from different sources
Development of Crash Wall Design Loads from Theoretical Train Impact

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Crash walls as pier protection

Protection for adjacent facilities

Lack of direction for design

AREMA 8-2.1.5.1: “…piers supporting bridges over railways… shall be protected by a reinforced concrete crash wall…with adequate reinforcing steel”

Previously used energy balance equations gave impractical design loads

Goals for this study

Review existing guidelines that use energy balance equations
Compare the energy balance approach to other methods
Develop a set of practical design guidelines that may be applied to future projects

The energy balance

\[ F_d = \frac{1}{2} m v^2 \]

Constants

- Train consist
  - 286 kip freight cars, 56’ long
  - 148 kip passenger cars, 85’ long
- Height of impact
  - 6’
Constants continued

- Deflection of the train
  - Compression of linkages and equipment for a consist: 10’
  - Plastic deformation of an individual car: 1’
- Deflection of the wall

The glancing blow load case

- Derailed train travels in the general direction of the rails, but may drift sideways or roll over into the crash wall

Deceleration

- Previously no allowance for deceleration
- Add provision using runaway lane concept
- Select resistance of 0.25, equivalent to truck tires in pea gravel

Angle of impact

- Angle of impact is constrained by length of car and distance from centerline of track
- Also a function of forward momentum of the train
- Set maximum angle of impact to 3.5°

Length of action of impact force

- Apply load over \( L \) and not just as point load

Single car impact

- Derailed train travels in the general direction of the rails, but may drift sideways or roll over into the crash wall
Velocity and angle

- Angle of impact and velocity are a function of car length and distance from centerline of track

Two cars in accordion

- Design for simultaneous impact of adjacent cars

Design moment by energy balance load case

- Load case 1: freight train glancing blow
- Load case 2: single freight car
- Load case 3: passenger train glancing blow
- Load case 4: single passenger car

Compare design methods

Hirsch et al., 1989

- Computer model to solve for impact force of derailed train against a crash wall

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At what distance are walls required?

- Clearance envelope, minimum distance: 9’
- In Canada, minimum distance to face of pier: 18’
- Per AREMA, crash walls required to: 25’
- CP Rail, crash walls required for all other structures to: 49’

Effect of speed on design loads

- Design for a minimum point load of 600 kip (2700 kN)
- The point load shall be applied at a height of 6 feet
- Applicable where track speeds do not exceed 50 mph for freight or 70 mph for passenger
Recommended Design Guidelines – Method 2
• An energy balance approach may be used, considering 4 load cases:
  – Freight train glancing blow
  – Freight train single car impact
  – Passenger train glancing blow
  – Passenger train single car impact

Example: freight train glancing blow
• Velocity at impact
  \[ v_G = \sqrt{v_0^2 + 2a\left(\frac{d_{cl} - 5.331}{\sin \theta_G}\right)} \]
• Length of impact
  \[ l_G = \frac{10}{\cos \theta_G} \]

The energy balance
\[ F \cdot d = \frac{1}{2} m v^2 \]

Minimum Requirements
• Minimum thickness of wall
  – 2’-6” for walls within 25’ of centerline
  – 18” for walls beyond 25’
• Minimum height of wall
  – 12’ for walls within 12’ of centerline
  – 7’ for walls beyond 12’
• Face of wall smooth and continuous
• Face of wall extends beyond face of structure
• Design must incorporate horizontal and vertical continuity

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