BRIDGE RAILS FOR
MESSINA STRAIT BRIDGE

Final Report
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SUMMARY

This report presents an External and an Internal Bridge Rail Design for use on the Messina Strait Bridge.

The External Bridge Rail is 89.5 in. (2273 mm) high and is designed to restrain and redirect a 30.7 metric ton (67,500 lb) tank truck at 80 km/hr (50 mph) and 20° angle. This External Rail is constructed of three 11 in. (280 mm) diameter aluminum tubes supported by 12 in. (305 mm) deep steel wide flange posts spaced 8 ft-8 in. (2642 mm) center to center. This External Rail weighs 109 lb/ft (162 kg/m) which is considerably less than early preliminary designs (185 lb/ft or 276 kg/m).

The Internal Bridge Rail is 62 in. (1575 mm) high and is designed to restrain and redirect a van-type truck at the same weight, speed and impact angle as above. This Internal Rail is constructed of two 12 in. (305 mm) diameter aluminum tubes supported by 12 in. (305 mm) deep steel posts as above. The Internal Bridge Rail weighs 82 lb/ft (122 kg/m) which is considerably less than early preliminary designs (165 lb/ft or 246 kg/m).

The two bridge rails were analyzed and designed by an ultimate strength and plastic failure mechanism procedure. For this procedure to be valid, ductile material and good connections of structural elements are required. These special requirements are presented in the report.
METRIC - ENGLISH CONVERSION FACTORS

MASS

1 lbm = 0.4535924 kg

FORCE

1 lb = 4.4482219 N (Newtons)
1 kip = 1000 lb = 4.4482219 kN

LENGTH

1 in. = 2.54 cm = 25.4 mm
1 ft = 0.3048 m
1 mile = 1609.344 m = 1.609344 km

STRESS - FORCE/AREA

1 psi = 6.895 K N/m² = 6895 Pascals
1 ksi = 6.895 N/mm²

SPEED

1 mph = 0.44704 m/sec = 1.609344 km/hr
1 fps = 0.3048 m/sec

DERIVED UNITS

1 lb/ft = 1.488164 kg/m
INTRODUCTION

In June 1986 the Texas Transportation Institute began making analytical evaluations of several proposed bridge rail designs for use on the Messina Strait Bridge. The bridge rail requirements were set forth in the January 30, 1986 letter from G. D. Gilardini of Stretto Di Messina, Sp. A., to T. J. Hirsch. The bridge rail should be minimum weight and maximum ultimate strength. The bridge rail should be of high strength aluminum or steel. The design vehicle should be a 50 metric ton truck traveling 80 km/hr and impacting at 20° angle (110,000 lb truck at 49.7 mph).

Some typical design trucks are shown by Figure 1. Stretto Di Messina proposed an external bridge rail as shown by Figure 2 and internal bridge rail as shown by Figure 3. Both rails used a New Jersey type steel safety shape in the lower 550 mm (22 in.).

A preliminary progress report was submitted to Stretto Di Messina on July 17, 1986. This report pointed out that the proposed design vehicle for the Messina Strait Bridge will produce an impact force of about 1557 kN (350 kips) which is considerably larger than the 890 kN (200 kips) impact force now used in the USA (80,000 lb truck impacting at 50 mph and 15° angle). Stretto Di Messina therefore reduced the design impact force to 1000 kN (225 kips) which is equivalent to a 30.7 metric ton truck impacting at 80 km/hr and 20° (67,500 lb truck, 50 mph and 20°).

The analytical evaluation of the proposed external bridge rail (Figure 2) indicated its strength to be 560 kN (126 kips) if made of aluminum with a yield strength of 240 N/mm² (35 ksi). The total weight of this proposed rail was 185 lb/ft (275 kg/m).

Since the lower 550 mm New Jersey type steel safety shape contributes
nothing toward the strength of this rail, it was recommended that it be
deleted. This steel safety shape weighs about 186 kg/m (125 lb/ft) so a
considerable saving in weight and costs results from deleting this with no
sacrifice in strength of rail. In addition, recent research such as
Reference 26 "Rollover Caused by Concrete Safety Shaped Barrier, Task A
Report" would indicate the expensive, heavy New Jersey Safety Shape is not
desirable for the Messina Strait Bridge.
Figure 1. Typical Design Trucks.
RAILS & N.J. STEEL GUIDE RAIL
& WIND REDUCTION ELEMENTS
PROPOSED FOR
STRAIT OF MESSINA BRIDGE
ULTIMATE CAPACITY 40 mt Tonn.

Figure 2. Proposed External Bridge Rail.
TOTAL WEIGHT: 165 lb/ft

DOUBLE B.B. & N.J. STEEL BARRIER
(PROPOSED FOR INTERNAL EDGES)
STRAIT OF MESSINA BRIDGE

Figure 3. Proposed Internal Bridge Rail.
IMPACT CONDITIONS, IMPACT FORCES
AND GEOMETRICS

In our preliminary progress report, the 48 metric ton (105.6 kip) tank truck shown on Figure 1 was scaled up to 50 metric tons (110 kips) for our design vehicle. The center of gravity of this vehicle was 2000 mm (79 in.). Figure 4 shows how this 110 kip truck compares with the 80 kip design truck typical of the USA.

Using these typical trucks as design vehicles, the theoretical procedures presented in references 1 and 2 were used to compute (see Appendix B) the design impact forces shown on Figure 5. Figure 5 shows the 50 ms average impact force for various weight vehicles at various speeds and angles of impact.

The heavy truck bridge rails constructed and tested to date in the USA were designed for an 80 kip truck (Figure 4) impacting at 50 mph (80 km/hr) and 15° angle or a 200 kip (890 kN) average 50 ms impact force. The Federal Highway Administration (FHWA) has proposed that we increase the impact speed to 55 mph (88 km/hr). This would produce an impact force of 245 kips (1090 kN) as shown on Figure 5.

The proposed Italian design truck, 110 kips, impacting at 50 mph (80 km/hr) and 20° would produce an impact force of 350 kips (1557 kN).

These impact forces were discussed with engineers of Stretto Di Messina and a compromise design impact force of 1000 kN (225 kips) was agreed upon. This impact force would be equivalent to the following:

(1) 30.7 metric ton (67.5 kip) truck impacting at 80 km/hr (50 mph) and 20° angle or,
(2) 32.7 metric ton (72 kip) truck impacting at 88 km/hr (55 mph) and 15° angle, or
Figure 4. Bridge Rail Design Truck.
Figure 5. Comparison of Vehicle Impact Forces to Total Vehicle Weight-Theory and Test Results-Stiff Rails.
(3) 40 metric ton (88 kip) truck impacting at 80 km/hr (50 mph) and 15° angle.

The initial design truck had a center of gravity of 2000 mm (79 in.), and it was agreed that this could be reduced to 1800 mm (71 in.). Using this center of gravity height, 1800 mm (71 in.), and the formulas presented in references 1 and 2, we get a maximum $G_{lat} = 3.31$ g's and an effective barrier height of (1420 mm) 56 in. as shown in Figure 6 (see Appendix B). These design values were arrived at considering that Stretto Di Messina can control the weight and speed of trucks using the bridge.
Figure 6. Comparison of Required Barrier Height to Vehicle Center of Gravity-Theory and Test Results.
Ultimate strength and plastic failure mechanism analysis procedures were used in this study (see reference 5). In this procedure, plastic hinges are assumed to develop at points of maximum bending moment. At failure, a sufficient number of hinges are developed to form a mechanism. No factor of safety is used in this analysis. The critical failure mechanism will barely resist the design impact force of 1000 kN (225 kips) in this case. When impacted by the design force the rail is expected to yield and deform permanently. Our margin of safety is in the inertia of the barrier and in the ductility and toughness of the aluminum and steel materials used. In order for a structure to behave as assumed, materials with adequate ductility must be used. Materials considered in this study have adequate ductility to meet this requirement. Also, joints must be designed to resist the full bending moment strength of the member being joined. This means that joints in the rail elements and the connection between the post and deck must resist bending. Joints or splices in the rail elements should be at quarter points in the span when they occur. It is not necessary for the connection between the rail elements and post to resist bending moment, however the rail elements should be continuous across this connection.

Failure mechanisms for one, two and three spans are illustrated in Figure 7. Mechanisms involving more spans are also possible.

For the failure mechanisms considered, the rail elements function as beams as shown in Figure 8. The failure load for each rail element is expressed by:

\[ W = w \ell = 8 \, M_p/(L - \ell/2) \]
Figure 7. Typical Failure Mechanisms for Bridge Rails.
(A) Single Span Failure Mode

(B) Two Span Failure Mode

(C) Three Span Failure Mode

\[ M_p = \text{plastic moment capacity of rail} \]
\[ P_p = \text{ultimate load capacity of a single post} \]
\[ w\ell = \text{total ultimate vehicle impact load} \]
\[ w\ell = \frac{8M_p}{L-(\ell/2)} = W \]

Figure 8. Possible Failure Modes for Metal Rails.
where:

- \( W \) is total applied load, kips
- \( M_p \) is plastic bending moment capacity of rail, ft-kip
- \( L \) is total span length of failure mechanism, ft
- \( \lambda \) is length over which applied load is distributed, ft

For one span the shear force, \( V \), in each rail element at the ends of the mechanism is \( W/2 \). For a mechanism involving only one span, the total load carried by the railing system would be the number of rail elements multiplied by the total load for each rail element. It would be possible for a one-span mechanism to occur if each post were capable of supporting a load equal or greater than one-half the total load for all rail elements for a one-span mechanism. If the post is weaker than this, the mechanism that will develop will involve more than one span. If such a mechanism occurred, the total load carried by the rail system would be the load carried by each rail element as a beam mechanism plus the load carried by each post that is located within the mechanism (not including posts at each end of the mechanism).
EXTERNAL BRIDGE RAIL

Figure 9 shows our proposed alternate External Rail design number 3 (number 1 is shown by Figure 2 and number 2 was presented in the preliminary report of July 1986). This design consists of three aluminum tubes 11 in. O.D. x 5/16 in. wall thickness structural aluminum with yield strength of 35 ksi (245 N/mm²). These three tubes along with the steel wide flange posts W 12 x 50 spaced at 8 ft-8 in. center to center (2642 mm) provide the design strength of 225 kips (1000 kN). The steel posts are of yield strength 50 ksi (345 N/mm²). The lower 6 in. O.D. x 5/16 in. wall thickness aluminum tube is a rub-rail to prevent automobile tires from snagging the posts (see Appendix B for supporting computations).

Figure 10 summarizes the load capacity analysis of this External Bridge Rail. The three span failure mechanism is the critical mechanism which produces a load capacity of 237 kips (1054 kN). With this mechanism two posts within the 26 ft (7.9 m) span will develop plastic hinges. The 8 ft-8 in. (2642 mm) post spacing was chosen since the floor beams of the bridge appear to be at this spacing. The steel post can be attached to these floor beams to develop these plastic moment strength (see Appendix B).

This External Bridge Rail weighs about 109 lb/ft (172 kg/m) whereas the one shown by Figure 2 weighs about 185 lb/ft (276 kg/m).

Table 1 compares three External Bridge Rails designed with three different span lengths or post spacings. With the 5 ft (1500 mm) post spacing the rail sizes and weight decreases but the number and weight of post per length of bridge increases. The net results in an increase in the total weight of bridge rail, 123 lb/ft (183 kg/m) in this case. For
WEIGHT OF RAIL PER FOOT OF BRIDGE

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight per Foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum Tubes</td>
<td>44.8 lb/ft</td>
</tr>
<tr>
<td>Post</td>
<td>43.2 lb/ft</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>21.0 lb/ft</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>109.0 lb/ft</strong></td>
</tr>
</tbody>
</table>

**ALUMINUM TUBES** 11 in. O.D. x 5/16 in. wall
- $F_y = 35 \text{ ksi}$
- $y = 245 \text{ N/mm}^2$

**W12x60 STEEL POST**
- $F_y = 50 \text{ ksi}$
- $y = 345 \text{ N/mm}^2$
  - at 8 ft-8 in. spacing (2642 mm spacing)

**6 in. O.D. x 5/16 in.**

**1/2 in. STIFFNERS**

**6 - 1-1/4 in. diameter A325 BOLTS**
- $F_u = 120 \text{ ksi} = 827 \text{ N/mm}^2$

*Figure 9. Alternate External Rail Design No. 3.*
Figure 10. Load Capacity Analysis of Alternate External Rail Design No. 3.

STRENGTH OF FAILURE MECHANISM
3 Alum. Tubes 11 in. O.D. x 5/16 in. wall

STRENGTH OF POSTS
W12 x 50
64.6 kips each

CRITICAL MECHANISM
237 kips
3 Span 26 Feet

LOAD CAPACITY OF RAIL - KIPS

LENGTH OF FAILURE MECHANISM - FEET

No. Spans
1
2
3
4
0
8.7
17.3
26.0
34.7

0
100
200
300
400
500
Table 1. Comparison of Bridge Rail Designs with Various Span Lengths.

<table>
<thead>
<tr>
<th>Type</th>
<th>Rail Span Post Spacing</th>
<th>Alum. Tubes No. x Diam. x thickness</th>
<th>Steel Post</th>
<th>Load Capacity kips</th>
<th>Weight lb./ft</th>
<th>Failure Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>External</td>
<td>5.0 ft</td>
<td>3-8 in. x 5/16 in.</td>
<td>W12x45</td>
<td>229</td>
<td>123</td>
<td>3 span - 15 ft</td>
</tr>
<tr>
<td>External</td>
<td>8.67 ft</td>
<td>3-11 in. x 5/16 in.</td>
<td>W12x50</td>
<td>237</td>
<td>109</td>
<td>3 span - 26 ft</td>
</tr>
<tr>
<td>External</td>
<td>10.0 ft</td>
<td>3-11 in. x 1/2 in.</td>
<td>W12x58</td>
<td>227</td>
<td>112</td>
<td>2 span - 20 ft</td>
</tr>
<tr>
<td>Internal</td>
<td>5.0 ft</td>
<td>2-10 in. x 5/16 in.</td>
<td>W12x35</td>
<td>238</td>
<td>84</td>
<td>3 span - 15 ft</td>
</tr>
<tr>
<td>Internal</td>
<td>8.67 ft</td>
<td>2-12 in. x 3/8 in.</td>
<td>W12x40</td>
<td>240</td>
<td>82</td>
<td>3 span - 26 ft</td>
</tr>
<tr>
<td>Internal</td>
<td>10.0 ft</td>
<td>2-11 in. x 1/2 in.</td>
<td>W12x50</td>
<td>236</td>
<td>93</td>
<td>2 span - 20 ft</td>
</tr>
</tbody>
</table>
the 10 ft (3 m) post spacing, the size and weight of the rail increases but the net weight of post per length of bridge decreases. Note also the length of failure mechanism reduced to 2 spans or 20 ft (6 m). Remember the External Bridge Rail posts are about 7.5 ft (2.3 m) high. The optimum rail span or post spacing might be about 7.5 ft (2.3 m) in this case.

The 84 in. (2130 mm) height of the top rail was chosen so that it would engage the outer edge of large diameter tank trucks to prevent them from rolling over during redirection or impact.

The center rail at 56 in. (1422 mm) height was selected so that it will engage the floor system of most van-type trucks. The lower structural rail at 28 in. (711 mm) height will engage the 42 in. (1067 mm) diameter bus and truck tires. The tires and axles are hard points and transmit a significant amount of impact force into the bridge rail system. The lower rub-rail is to prevent small passenger car tires from snagging on the posts.
INTERNAL BRIDGE RAIL

Figure 11 shows our alternate Internal Bridge Rail design number 3 (number 1 is shown by Figure 3 and number 2 would have been a modification from preliminary report of July 1986). This design consists of two 12 in. O.D. x 3/8 in. wall thickness aluminum tubes with yield strength of 35 ksi (245 N/mm²). These two tubes have the same strength as the three tubes used on the External Rail. These two tubes along with the steel wide flange posts W12x40 spaced at 8 ft-8 in. center to center (2642 mm) provides the design strength of 225 kips (1000 kN). The steel posts are of yield strength 50 ksi (345 N/mm²). The lower 6 in. O.D. x 5/16 in. wall thickness aluminum tube is a rub-rail to prevent automobile tires from snagging the posts.

Figure 12 summarizes the Load Capacity Analysis of this Internal Bridge Rail. The three-span failure mechanism is the critical mechanism which produces a load capacity of 240 kips (1068 kN). With this mechanism two posts within the 26 ft (7.9 m) span will develop plastic hinges.

This Internal Bridge Rail weighs about 82 lb/ft (122 kg/m) whereas the one shown on Figure 3 weighs about 165 lb/ft (246 kg/m).

Table 1 compares three Internal Bridge Rail designs with three different spans. The 8 ft-8 in. (2642 mm) span seems to be a good one.

The 56 in. (1422 mm) height of the top rail was chosen because it would engage the floor systems of most van-type trucks. The lower structural rail at 28 in. (711 mm) height will engage the 42 in. (1067 mm) diameter bus and truck tires. The tires and axles are hard points and transmit a significant amount of impact force into the bridge rail system.

One final point is that the Internal Bridge Rail appearance and geometry is compatible with the lower two thirds of the External Bridge...
Rail. Messina Strait engineers may want to select the same diameter tubes for both the Internal and External Rails. For example, use three 12 in. O.D. x 1/4 in. wall aluminum tubes for the External Rail instead of the three 11 in. O.D. x 5/16 in. wall tubes. If this is done, all rails will be 12 in. in diameter and the splices and post-rail connections could be the same. This should reduce fabrication costs.
WEIGHT OF RAIL PER FOOT OF BRIDGE

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (lb/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum Tubes</td>
<td>38.8</td>
</tr>
<tr>
<td>Post</td>
<td>25.0</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>18.2</td>
</tr>
<tr>
<td>TOTAL</td>
<td>82.0</td>
</tr>
</tbody>
</table>

**Figure 11. Alternate Internal Rail Design No. 3.**
Figure 12. Load Capacity Analysis of Alternate Internal Rail Design No. 3.
CONNECTIONS

Rail Splices

Structural aluminum tubing used for the rail elements should be as long as possible to minimize the number of splices that must be provided. Such splices can also accommodate thermal expansion and contraction. Splices, made in rail elements, should be adequate to develop the full plastic bending moment capacity of the rail element. For the designs considered here it is not necessary to develop significant tensile axial strength.

Splices may be made in the proposed rail elements by inserting a smaller diameter round tube sleeve inside the main rail elements. The plastic section modulus of the splice tube should be equal to or greater than that of the rail elements. Splices should be located at quarter points between post spans.

Figure 13 shows a typical tube rail splice. The outside diameter (O.D.) of the sleeve splice tube should be about 1/4 in. (6 mm) smaller than the inside diameter (I.D.) of the main rail tube. The sleeve splice tube should extend at least two diameters into the main tube.

Rail to Post Connections

Rail elements must be continuous across posts so that the full plastic bending moment can be developed in the rail elements at the posts. Connections between rail elements and posts need only hold the rail in place and transfer lateral forces in the posts. The primary lateral force is directed outward and places the connections in compression. In addition, an inward (tension) lateral force of about the same magnitude will occur just beyond each end of a loaded area as a result of the deflected shape of the continuous rail elements. This force will place
STEEL POST W12x50

1-1/4 in. Joint

2 ft

ALUMINUM TUBE 11 in. O.D. x 5/16 in. wall

1/2 in. Ø PIN (Driving Fit)

ALUMINUM SPLICE TUBE
10 in. O.D. x 7/16 in. wall 4 ft-1 1/4 in. long

EXTERNAL RAIL

Figure 13. Typical Tube Rail Splice Detail.
the connection in tension. For the External and Internal Rails shown by Figures 9 and 11, the post compression force is about 65 kips (289 kN) and tension force is about 56 kips (249 kN). The post forces are divided in three rails for the External Rail and divided into two rails for the Internal Rail. A cast or welded connector similar to the concept shown in Figure 14 is recommended. The connector would be bolted to the post flange with standard structural bolts. The rail element would be bolted to the connector with two toggle bolts similar to those shown in Figure 15.

Post Base Plate

Figure 16 shows a typical post base plate and its full bending moment connection to a floor beam. The 12 in. wide flange steel post should be welded to the base plate with fillet welds to develop the full plastic moment capacity of the post. The weld metal should be equal or better than the post yield strength of 50 ksi (345 N/mm²). The 3/8 in. and 5/8 in. fillet welds shown would develop the moment capacity of the W12x50 steel post.

The base plate is bolted to the floor beam flange with six high strength ASTM A325 bolts which have an ultimate tensile strength of 120 ksi (827 N/mm²). The bolt pattern shown will develop the full plastic moment of the post in both directions (outward and inward). These bolts should be tightened by the "turn of the nut" method or by torque wrenches or other means to achieve 70% of their ultimate strength in pretension.

The floor beams of the bridge should be reinforced with web stiffeners as shown to prevent local buckling of the web.

The size of the base plate and anchor bolts for the two posts shown on Figures 9 and 11 are as follows:
Ultimate Rail Force = 60 kips (267 kN) compression from Barrier VII

= 47 kips (209 kN) tension

Figure 14. Typical Rail to Post Connector.
SPECIFICATIONS

Toggles shall conform to A.S.T.M., A570, Grade D or better; toggle bolt to A.S.T.M A354, Grade BD; hex nut to A.S.T.M. A563, Grade D; rivet to A.S.T.M. A276, Type 304 Condition B.

Plain washers shall be made of steel and shall meet the dimensional requirements of A.N.S.I. B27.2 Type A Plain Washers. Spring lock washers shall meet the requirements of A.N.S.I. B27.1.

Steel components shall be cadmium plated to A.S.T.M. A165, Type OS, or better.

Required minimum tensile load shall equal 9000 pounds when in open position and tested through a 1" φ hole.

Dimensional tolerances not shown or implied are intended to be those consistent with the proper functioning of the part, including its appearance, and accepted manufacturing practices.

INTENDED USE

This toggle bolt is used to connect 5" OD tubular rails in bridge designs BR1 (Aluminum) Type A, BR2A (Aluminum) Type A, BR2 (Aluminum) Type A, and BR3A (Aluminum) Type A.

1/2" TOGGLE BOLT

HM-TF-13

F-2573

Figure 15. Example Toggle Bolt from AASHTO Hardware Standards.
Figure 16. Typical Post Base Plate Details.
W12 x 40 Post

Base Plate 1-1/2" x 10" x 20"

Bolts 6 - 1-1/8" φ A325

$F_y = 50 \text{ ksi (345 N/mm}^2\text{)}$

$F_u = 120 \text{ ksi (827 N/mm}^2\text{)}$

W12 x 50 Post

Base Plate 1-3/4" x 10" x 20"

Bolts 6 - 1-1/4" φ A325

$F_y = 50 \text{ ksi (345 N/mm}^2\text{)}$

$F_u = 120 \text{ ksi (827 N/mm}^2\text{)}$
A significant consideration in the design of bridge rail for the proposed structure is aerodynamics. Several cross sections for posts were considered. The post is subjected to bending loads, and a wide flange shape is very efficient for resisting bending. However, a wide flange shape has a relatively high wind drag coefficient. Other shapes such as square and round tubes have lower wind drag coefficients but are less efficient (strength-to-weight ratio) in resisting bending.

For comparisons of various post shapes, it is necessary to consider the drag coefficient and width of post only because the various post shapes would be the same length. The shapes considered are shown in Table 1. A wide flange post would be 8 in. (0.67 ft) wide and have a drag coefficient of about 2.3. Multiplication of these factors gives a load factor of 0.67 x 2.3 = 1.54.

For a square structural tube the width would be 10 in. (0.83 ft) and have a drag coefficient of 1.2. Multiplication of these factors gives a load factor of 0.83 x 1.2 = 1.00. Similar calculations for a 12 in. diameter round tube post would give a load factor of 0.80.

If a wide flange were filled with foam or some other means of making it present a rectangular shape, its load factor would become 0.80, which is as low as the round tube of the same bending strength. This latter design is recommended since it also has the least weight.
LOAD FACTOR

0.67 ft x 2.3 = 1.54

W12x45 lb/ft
f = 1.11
S = 58.1 in.³
Z = 64.7 in.³
I = 350 in.⁴
C_d = 1.7 to 2.8
C_d = 2.3 avg.
Wt = 45 lb/ft
Effectiveness = 1.54 x 45 = 69.3

LOAD FACTOR

0.83 ft x 1.2 = 1.00

ST 10 x 10 x 1/2
f = 1.125
S = 54.2 in.³
Z = 61 in.³
I = 271 in.⁴
C_d = 1.2
Wt = 62.5 lb/ft
Effectiveness = 62.5

LOAD FACTOR

1 ft x 0.8 = 0.80

Tube 12 in. O.D. x 1/2 In. wall
f = 1.27
S = 50.95 in.³
Z = 64.7 in.³
I = 300 in.⁴
A = 18.06 in.
C_d = 1.1 to 0.5
Wt = 61.4 lb/ft
C_d = 0.8 avg.
Effectiveness = 49.2

LOAD FACTOR

0.67 x 1.2 = 0.80

Wt = 45 lb/ft

Figure 17. Post Section - Post Aerodynamics
MATERIALS

Aluminum Rail Elements

The specified yield strength for the tubular aluminum rail elements is 35 ksi. The material should also have sufficient ductility to withstand the deformations that will occur in regions of plastic hinges when a mechanism is developed. Material meeting the Aluminum Association (Washington, D.C.) designation 6061-T6 should be adequate. This material has a specified minimum yield strength of 35 ksi and an ultimate strength of about 38 to 42 ksi. The elongation in two inches is 17 percent. This is an aluminum alloy having the following nominal chemical composition:
(Reference: Aluminum Association Specifications)

<table>
<thead>
<tr>
<th>Composition</th>
<th>Percentage by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Copper</td>
<td>0.25</td>
</tr>
<tr>
<td>Silicon</td>
<td>0.6</td>
</tr>
<tr>
<td>Magnesium</td>
<td>1.0</td>
</tr>
<tr>
<td>Chromium</td>
<td>0.25</td>
</tr>
<tr>
<td>Aluminum</td>
<td>97.9</td>
</tr>
</tbody>
</table>

Steel Posts

The steel wideflange posts have a specified yield strength of 50 ksi. Material meeting American Society for Testing and Materials specification A441 should be used. This material has a specified minimum yield strength of 50 ksi and a minimum tensile ultimate strength of 70 ksi. The elongation for this material is 18 percent. The posts should also possess adequate ductility to allow rotation in the plastic hinges that must form in the posts immediately above the base plates when a mechanism forms in the bridge rail.
Base plates should also be fabricated from material meeting specification A441 and should be welded to the posts using properly designed welds and welding procedures.

**Rail-to-Post Brackets**

It is anticipated that these brackets will be made of aluminum alloy and will be either fabricated from sheet and plate or will be cast. The Texas State Department of Highways and Public Transportation has had very good experience with a cast aluminum bridge rail post that is somewhat similar to the bracket proposed for the Messina Strait Bridge. The post is made of alloy A444-T4 in accordance with American Society for Testing and Materials specification B108. A material such as this is recommended for the brackets connecting the rail members to the posts in the Messina Strait Bridge rails.

**Post Mounting Bolts**

Bolts used to connect the posts to the floor beam should be high strength structural steel bolts meeting American Society for Testing and Materials Specification A325. These bolts should be installed according to specifications using the turn-of-the-nut method to assure that adequate preload is achieved.
REFERENCES


APPENDIX A
COMPUTER ANALYSIS WITH BARRIER VII PROGRAM

Messrs. Dean Sicking and Roger Bligh of TTI performed a BARRIER VII computer program analysis (reference 25) of a truck impact with the External Bridge Rail (Figure 9 and simulated as shown in Figure A1), and summaries of the results are presented in Figures A2 through A11 and Tables A1 through A6. TTI has used the BARRIER VII computer program to simulate numerous automobile and bus impacts with longitudinal barriers and found it to give good comparisons. The BARRIER VII program only has provisions for two rails so the middle and lower aluminum tubes are combined into one as shown on Figure A1.

Impact was with the tractor of the Messina truck shown by Figure 4 with 8.6 kips on the front axle and 28.2 kips on the rear axle. The total weight of the tractor was 36.8 kips. Previous crash tests with tractor-trailer trucks such as this have shown the rear axle of the tractor delivers the major impact force to the bridge rail, and this BARRIER VII analysis confirms this. The rear axle of the trailer frequently does not even touch or impact the traffic rail.

The truck tractor impacts the bridge rail at 50 mph (80 km/hr) and a 20° angle with the centroid of the tractor directed at Post No. 4 as shown by Figure A3. The left front of the tractor contacts the rail midway between Post No. 2 and 3.

Before proceeding, the following is a summary of steel post and aluminum rail strength properties:

**POST W12 x 50**

Plastic Bending Moment Strength = $F_y Z = 3620$ kip-in. (strong axis)
Plastic Deformation will occur about strong axis when the top post deflection $\geq 0.6$ in.

Shear Yield Strength about x-x axis = $0.55 F_{yd} = 124.0$ kips

Plastic Bending Moment Strength = 1070 kip-in. (weak axis)

**ALUMINUM RAILS 11 in. O.D. x 5/16 in. Wall**

Plastic Bending Moment Strength = $F_yZ = 1211$ kip-in.

Moment reduced 21 percent to allow for some local buckling and possible loss of shape because of aluminum

Thus $M_p = 956$ kip-in. TOP RAIL

and $M_p = 1912$ kip-in. BOTTOM RAIL

In Tables A3 and A4 you will note that the posts and rails yield when moments are within plus or minus 10 percent of these values. BARRIER VII program does this to assure numerical stability of the computer program.

Figure A4 shows the tractor and bridge rail 0.08 sec after impact. Since the transverse deflections of posts 2, 3, and 4 exceed 0.6 in., they have already yielded.

Figures A5, A6 and A7 show the truck tractor after it loses contact with the rail and rotates from a heading of 12.2° to -4.2°.

Figure A8 at 0.29 sec shows the rear tandem axles of the tractor impacting the rail and generating the maximum acceleration of 7.37 g's. At this time the aluminum rails have achieved a plastic moment at posts 3 and 4. From Table A5 one can see that the rear of the tractor actually impacts the rail at 0.25 sec.

At 0.33 sec Figure A9 shows that the aluminum rail has achieved a plastic moment at Posts 2, 3, 4 and 5 and the design failure mechanism has formed (3 span, 2 posts). At 0.39 sec Figure A10 shows the design failure mechanism more clearly. Posts 3 and 4 are clearly yielding and the
aluminum rail has plastic moments at posts 2, 3, 4 and 5 as shown by Figures 7 and 8 of the main report. The 3-span failure mechanism which was found critical in the static design analysis is verified by the BARRIER VII dynamic analysis.

Figure A11 shows the tractor leaving the bridge rail at about 0.44 sec and the post deflections shown are the final permanent deformations.

Table A1 is a summary of the post deformation at selected times.

Table A2 is a summary of the post shear force at various times. Note there are both transverse and longitudinal shear forces on the posts. This is because a coefficient of friction of 0.3 was used between the tractor and the aluminum rails. The maximum post shear is 118.7 kips which is less than the post shear yield strength of 124 kips.

Table A3 presents a summary of the post bending moments at selected times. The posts never yield about the weak axis. Note post 3 has yield moments about the strong axis of 3680 kip-in. and 3550 kip-in. which are within plus or minus 10 percent of the yield moment of 3620 kip-in.

Table A4 presents a summary of the aluminum rail bending moments at selected times. Note the bottom rail is twice as strong as the top rail (Figure A1). The yield moments shown are also within plus or minus 10 percent of the rail plastic moment strength of 956 kip-in. (top rail) and 1912 kip-in. (bottom rail).

Table A5 presents a summary of the truck tractor accelerations, speed, etc., at 10 ms intervals. The transverse acceleration vector is perpendicular to the rail. Note the maximum 50 ms average acceleration is 6.25 g's. This would yield a maximum transverse force of

\[ 6.25 \times 36.8 \text{ kips} = 230 \text{ kips} \] (50 ms average)

which was used to design this bridge rail (Figures 5 and 10).
Table A6 presents a summary of the kinetic energy lost during the tractor-bridge rail impact. After the impact was over 0.44 sec, the tractor lost 39.6 percent of its initial kinetic energy. Of the total kinetic energy lost, 17.7 percent was absorbed by bending of the posts and rails, 5.3 percent by crushing of the truck tractor, and 77.0 percent due to friction losses between the tractor and rails and tire-pavement friction.

Conclusions The results of the BARRIER VII computer analysis of the EXTERNAL RAIL confirm the validity of the relatively simplified analysis and design procedures used to design the External and Internal Bridge Rails (Figures 9 and 11). This analysis indicates the maximum deflection of the top rail under the design impact will be about 17.9 in. This would indicate the maximum roll angle of the impacting truck would be about 12° (Figure A2) which is acceptable.

The BARRIER VII analysis indicates the three span/two post failure mechanism found critical in our simplified static analysis and design procedure was correct.
ALUMINUM TUBE  11 in. O.D. x 5/16 in. wall

\[ F_y = 35 \text{ ksi} \]
\[ y = 245 \text{ N/mm}^2 \]

W12x50 STEEL POST

\[ F_y = 50 \text{ ksi} \]
\[ y = 345 \text{ N/mm}^2 \]

at 8 ft-8 in. spacing
(2642 mm spacing)

2 ALUM. TUBES COMBINED INTO ONE

FIGURE A1. EXTERNAL BRIDGE RAIL AS SIMULATED BY BARRIER VII.
ALUMINUM TUBES 11 in. O.D. x 5/16 in. wall

\[ \begin{align*}
F_y &= 35 \text{ ksi} \\
y &= 245 \text{ N/mm}^2
\end{align*} \]

\[ 17.9'' \]

W 2x50 STEEL POST

\[ \begin{align*}
F_y &= 50 \text{ ksi} \\
y &= 345 \text{ N/mm}^2
\end{align*} \]

at 8 ft-8 in. spacing
(2642 mm spacing)

DEFLECTED POST

FLOOR BEAMS
Spaced 8 ft-8 in. (2642 mm)

1/2 in. STIFFNERS

6 - 1-1/8 in. diameter A325 BOLTS

\[ F_u = 120 \text{ ksi} = 827 \text{ N/mm}^2 \]

FIGURE A2. DEFLECTED EXTERNAL BRIDGE RAIL POST NO. 3.
(See Figure A)
FIGURE A3. PLAN VIEW OF TRACTOR AND RAIL.  
(Time = 0.00 sec)
FIGURE A4. PLAN VIEW OF TRACTOR AND RAIL. 
(Time = 0.08 sec)
FIGURE A5. PLAN VIEW OF TRACTOR AND RAIL.
(Time = 0.12 sec)
FIGURE A6. PLAN VIEW OF TRACTOR AND RAIL
(Time = 0.18 sec)
FIGURE A7. PLAN VIEW OF TRACTOR AND RAIL.  
(Time = 0.24 sec)
TIME = 0.29 sec.

FIGURE A8. PLAN VIEW OF TRACTOR AND RAIL.
(Time = 0.29 sec)
FIGURE A9. PLAN VIEW OF TRACTOR AND RAIL.
(Time = 0.33 sec)
FIGURE A10. PLAN VIEW OF TRACTOR AND RAIL.  
(Time = 0.39 sec)
FIGURE A11. PLAN VIEW OF TRACTOR AND RAIL.
(Time = 0.44 sec)
TABLE A1. SUMMARY OF POST DEFLECTION VS. TIME FROM BARRIER VII ANALYSIS OF EXTERNAL BRIDGE RAIL

<table>
<thead>
<tr>
<th>TIME (sec)</th>
<th>POST 1</th>
<th>POST 2</th>
<th>POST 3</th>
<th>POST 4</th>
<th>POST 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
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<td>5.97</td>
<td>1.13</td>
<td>-0.03</td>
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<tr>
<td>0.16</td>
<td>0.06</td>
<td>2.63</td>
<td>5.97</td>
<td>1.13</td>
<td>-0.03</td>
</tr>
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<td>0.20</td>
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<td>5.97</td>
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<td>-0.03</td>
</tr>
<tr>
<td>0.24</td>
<td>0.06</td>
<td>2.63</td>
<td>5.97</td>
<td>1.13</td>
<td>-0.03</td>
</tr>
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<td>-0.13</td>
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<td>16.83</td>
<td>5.66</td>
<td>0.08</td>
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<td>5.22</td>
<td>17.6</td>
<td>8.85</td>
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</table>

No contact t = 0.12 to 0.24 sec.

**Max. Top of Post Deflection**
TABLE A2. SUMMARY OF POST SHEAR FORCE VS. TIME FROM BARRIER VII ANALYSIS OF EXTERNAL BRIDGE RAIL

<table>
<thead>
<tr>
<th>TIME (sec)</th>
<th>POST SHEAR FORCE - KIPS</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>POST 1</td>
</tr>
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</tr>
<tr>
<td>0.4</td>
<td>2.11</td>
</tr>
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<td>0.8</td>
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</tr>
<tr>
<td>0.12</td>
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<tr>
<td>0.16</td>
<td>-0.6</td>
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<td>0.18</td>
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<td>-0.3</td>
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<tr>
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</tr>
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<td>5.2</td>
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<td>9.2</td>
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<td>6.8</td>
</tr>
<tr>
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<td>0.8</td>
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</table>

No contact t = 0.12 - 0.24 sec

*Max. Post Shear Transverse
TABLE A3. SUMMARY OF POST BENDING MOMENT VS. TIME FROM BARRIER VII ANALYSIS OF EXTERNAL BRIDGE RAIL

<table>
<thead>
<tr>
<th>TIME (sec)</th>
<th>POST 1 Weak Axis</th>
<th>POST 1 Strong Axis</th>
<th>POST 2 Weak Axis</th>
<th>POST 2 Strong Axis</th>
<th>POST 3 Weak Axis</th>
<th>POST 3 Strong Axis</th>
<th>POST 4 Weak Axis</th>
<th>POST 4 Strong Axis</th>
<th>POST 5 Weak Axis</th>
<th>POST 5 Strong Axis</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.0</td>
<td>0.0</td>
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<td>-92.0</td>
<td>2908.0</td>
</tr>
</tbody>
</table>

No contact t = 0.12 - 0.24 sec

*Yield Moment (For numerical stability, yield moment is ±10%).
TABLE A4. SUMMARY OF ALUMINUM RAIL BENDING MOMENT VS. TIME FROM BARRIER VII ANALYSIS OF EXTERNAL BRIDGE RAIL

<table>
<thead>
<tr>
<th>TIME (sec)</th>
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<th>SPAN 1</th>
<th>POST 2</th>
<th>SPAN 2</th>
<th>POST 3</th>
<th>SPAN 3</th>
<th>POST 4</th>
<th>SPAN 4</th>
<th>POST 5</th>
<th>SPAN 5</th>
<th>POST 6</th>
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<tbody>
<tr>
<td></td>
<td>Top Rail</td>
<td>Bottom Rail</td>
<td>Top Rail</td>
<td>Bottom Rail</td>
<td>Top Rail</td>
<td>Bottom Rail</td>
<td>Top Rail</td>
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No contact t = 0.12 - 0.24 sec

*Yield Moment
### TABLE A5. SUMMARY OF ACCELERATION AND VELOCITY DATA OF TRACTOR IN BARRIER VII ANALYSIS

<table>
<thead>
<tr>
<th>TIME AFTER IMPACT (sec)</th>
<th>ACCEL. VECTOR (g's)</th>
<th>VECTOR ANGLE (deg)</th>
<th>TRACTOR VELOCITY (mph)</th>
<th>HEADING ANGLE (deg)</th>
<th>TRANSVERSE ACCEL. (g's)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.25*</td>
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<td>.43</td>
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*Rear Impact of Tractor Begins*
TABLE A6. SUMMARY OF KINETIC ENERGY LOST
DURING TRACTOR IMPACT - BARRIER VII ANALYSIS

<table>
<thead>
<tr>
<th>ENERGY BALANCE, TIME = .4400 SECS</th>
<th>PERCENT OF</th>
<th>PERCENT OF</th>
<th>PERCENT OF</th>
</tr>
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<tbody>
<tr>
<td>TYPE OF ENERGY</td>
<td>ENERGY</td>
<td>ORIGINAL AUTO KE</td>
<td>ORIGINAL KE</td>
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<tr>
<td>Translational KE of Auto</td>
<td>60.4</td>
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<td></td>
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<tr>
<td>Rotational KE of Auto</td>
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</tr>
<tr>
<td>Barrier KE</td>
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<td></td>
</tr>
<tr>
<td>Elastic Energy in Members</td>
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<tr>
<td>Beams</td>
<td>.4</td>
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<td></td>
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<tr>
<td>Posts</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Inelastic Work on Members</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Beams</td>
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<tr>
<td>Posts</td>
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<tr>
<td>Elastic Energy in Auto</td>
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<tr>
<td>Inelastic Work on Auto</td>
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<td>Damping Losses</td>
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<tr>
<td>Auto-Barrier Friction Loss</td>
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<tr>
<td>Auto-Pavement Friction Loss</td>
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<tr>
<td>Sum of All Contributions</td>
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<tr>
<td></td>
<td></td>
<td>39.6 Total Loss</td>
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</tr>
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</table>

Original Kinetic Energy of Tractor = 3,036,806 lb-ft
- 44 Sec. Kinetic Energy of Tractor = 1,834,308 lb-ft
Total Energy Lost in Collision = 1,202,498 lb-ft
Bridge Rail Absorbed 17.7% = 212,842 lb-ft
Tractor Absorbed 5.3% = 63,732 lb-ft
Barrier and Pavement Friction 77.0% = 925,924 lb-ft
APPENDIX "B" ANALYSIS AND DESIGN COMPUTATIONS

THEORY IMPACT FORCE (see Fig. 4)

50 mph = 80 km/hr = 72.9 ft/sec

20° Angle

\[ G_{lat.} = \frac{(72.9 \text{ ft/sec})^2 \sin^2 20°}{2 \times 32.2 \text{ ft/sec}^2 \left[ 14.1 \text{ ft} \sin 20° - 4 \text{ ft} (1 - \cos 20°) \right]} \]

= 2.11 g's avg. Theory

\[ G_{max} = 2.11 \times \frac{\pi}{2} = 3.31 \text{ g's max. Theory} \]

Corrected \( G_{max} = 3.31 \times 1.78 = 5.89 \text{ g's max. 50ms} \)

Force \( F_{max} = 5.89 \text{ g's} \times 60 \text{ kips} = 353.4 \text{ kips} \) for 110,000 lb Truck

(see Fig. 5)

\[ \text{Theory } F_{max} = 3.31 \text{ g's} \times 60 \text{ kips} \]

50 mph 20°

= 198.6 kips for 110,000 lb Truck

= \( \overline{3.31 \times 45 \text{ kips}} \)

= 149 kips for 80,000 lb Truck

(see Fig. 5)

= \( \overline{3.31 \times 37} \)

= 123 kips for 67,500 lb Truck

Note: The 1.78 factor was used to adjust Theory to the 0.050 sec avg. experimental.
THEORY — EFFECTIVE HEIGHT OF RAIL

\[ H = \frac{\text{max. } G_{\text{lat}} \times C - B/2}{\text{max. } G_{\text{lat}}} \]

\[ = \frac{3.31 \text{ g/s} \times 71 \text{ in.} - 48 \text{ in.}}{3.31 \text{ g/s}} \]

\[ H = 56 \text{ in.} \quad (1422 \text{ mm}) \]

50mph - 20° Angle

(see Fig. 6)
SAMPLE BRIDGE RAIL DESIGN - EXTERNAL FIG. 9

Post Spacing = 8 ft - 8 in = 8.67 ft = 2642 mm

3 Main Rails Located at 84", 56" and 28"

ALUM. TUBE 11" O.D., by 0.313" thickness (3 will be used)

\[ S = 27.26 \text{ in}^3 \]
\[ Z = 1.27 S = 34.62 \text{ in}^3 \]

\[ M_p = F_y Z = 35 \text{ ksi} \times 34.62 \text{ in}^3 = 1211 \text{ in-kips} = M_p \]

Assume \( L = 8' \) or 96"

**Mechanism Ultimate Load each Rail**

<table>
<thead>
<tr>
<th>1 Span ( L = 8.67' = 104&quot; )</th>
<th>( w = \frac{8 M_p}{L - \frac{8}{2}} = \frac{8 \times 1211}{104 - 48} = 173 \text{ kips} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Span ( L = 208&quot; )</td>
<td>( w = \frac{8 \times 1211}{208 - 48} = 60 \text{ kips} )</td>
</tr>
<tr>
<td>3 Span ( L = 312&quot; )</td>
<td>( w = \frac{8 \times 1211}{312 - 48} = 36 \text{ kips} )</td>
</tr>
<tr>
<td>4 Span ( L = 416&quot; )</td>
<td>( w = \frac{8 \times 1211}{416 - 48} = 26 \text{ kips} )</td>
</tr>
</tbody>
</table>

**TRY POST**

\[ M_p = F_y Z = 50 \text{ kips/in}^2 \]

\[ 72.4 \text{ in}^3 = 362.0 \text{ in-kips} \]

\[ P_p = \frac{362.0 \text{ in-kips}}{5.6 \text{ in.}} = (64.6 \text{ kips} = P_p) \]
## Total Capacity of Rail Design - External

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>Load Capacity of Rails + Post</th>
<th>Mech. Total</th>
<th>Load Capacity Post Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Span</td>
<td>Rails $173^k \times 3$</td>
<td>$519^k$</td>
<td>$2 \times 64.6^k$</td>
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<tr>
<td>L = 8.67'</td>
<td>Post</td>
<td>$0$</td>
<td>$129.2$ kips</td>
</tr>
<tr>
<td>2 Span</td>
<td>Rails $60^k \times 3$</td>
<td>$180^k$</td>
<td>$3 \times 64.6$</td>
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<tr>
<td>L = 17.34'</td>
<td>Post $64.6 \times 1$</td>
<td>$64.6^k$</td>
<td>$193.8$ kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$244.6$ kips</td>
<td>$193.8$ kips</td>
</tr>
<tr>
<td>3 Span</td>
<td>Rails $36^k \times 3$</td>
<td>$108^k$</td>
<td>$4 \times 64.6$</td>
</tr>
<tr>
<td>L = 26'</td>
<td>Posts $64.6 \times 2$</td>
<td>$129.2^k$</td>
<td>$258.4$ kips</td>
</tr>
<tr>
<td>4 Span</td>
<td>Rails $26^k \times 3$</td>
<td>$78.0$</td>
<td>$5 \times 64.6$</td>
</tr>
<tr>
<td>L = 34.7'</td>
<td>Posts $64.6 \times 3$</td>
<td>$193.8$</td>
<td>$323.0$ kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$271.8$ kips</td>
<td>$323.0$ kips</td>
</tr>
</tbody>
</table>

See Fig. 10
DESIGN OF ALUM. TUBE RUB RAIL - FIG. 9

Design for One Span
4500 lb Car 60 mph 25° CG = 20 in
From FIG. 5 F_max = 50 kips
Since CG is at 20 in. 1/2 F_max will
be resisted by RUB Rail = 25 kips
other 1/2 by 11 in. Ø Tube l/2 = 12 in.

\[ F = \frac{8 M_p}{L - \frac{l}{2}} \]

\[ M_p = \frac{F (L - \frac{l}{2})}{8} = \frac{25 \text{ kips} (104'' - 12'')}{8} \]

\[ M_p = 287.5 \text{ in.-kips} \]

\[ Z_{\text{reqd}} = \frac{M_p}{F_y} = \frac{287.5}{35} = 8.21 \text{ in.}^3 \]

\[ S_{\text{reqd}} = \frac{Z}{1.27} = 6.47 \text{ in.}^3 \]

USE ALUM. TUBE 6 in. O.D. x 5/16 in. wall

\[ S = 7.55 \text{ in.}^3 \quad \text{OK} \]

see FIG. 9 and 11.
WEIGHT OF RAILING SYSTEM
EXTERNAL RAIL

ALUM TUBES

1\(\frac{1}{4}\) in. O.D. \times 0.313 \text{ in.}

\[ 12.7 \text{ lb/ft} \times 3 = 38.1 \]

6 in. O.D. \times 0.313 \text{ in.}

\[ 6.6 \div 44.8 \text{ lb/ft} \]

POSTS

\[ \frac{89.5}{12} \text{ lb/ft} \times 50 \frac{1}{4} \text{ ft} \times \frac{1}{8.67 \text{ ft}} = 43.2 \text{ lb/ft} \]

BASE PLATE, ANCHOR BOLTS, BRACKETS & STIFFENERS

Base PL 6 lb/ft

Bolts 2 \(\frac{1}{4}\) lb/ft

Brackets 9 lb/ft

Stiffeners 4 lb/ft = 21.0 lb/ft

\[ 109 \text{ lb/ft} \]
INTERNAL BRIDGE RAIL - FIG. 11
Post SPACING = 8.67 ft
2 MAIN RAILS located at 56" and 28"
ALUM TUBE 12" O.D. by 3/8" thickness
\[ S = 38.8 \text{ in.}^3 \quad Z = 1.27 S = 49.3 \text{ in.}^3 \]
\[ M_p = F_y Z = 35 \times 49.3 = 1725 \text{ in.-kips} \]
\[ \ell = 8' = 96" \]
Mechanism Ultimate Load each Rail

1 SPAN \[ W = \frac{8 M_p}{L - \frac{4}{2}} = \frac{8 \times 1725}{104 - 48} = 246 \text{ kips} \]
L = 104"

2 SPAN \[ W = \frac{8 \times 1725}{208 - 48} = 86 \text{ kips} \]
L = 208"

3 SPAN \[ W = \frac{8 \times 1725}{312 - 48} = 52 \text{ kips} \]
L = 312"

4 SPAN \[ W = \frac{8 \times 1725}{416 - 48} = 37 \text{ kips} \]
L = 416"

TRY W12 x 40 Steel Post \[ Z = 57.5 \text{ in.}^3 \]
\[ N_p = F_y Z = 50 \text{ ksi} \times 57.5 \text{ in.}^3 = 2875 \text{ in.-kips} \]
\[ P_p = \frac{2875 \text{ in.-kips}}{42 \text{ in.}} = 68.4 \text{ kips} = P_p \]
# Total Capacity of Internal Rail

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>Load Capacity</th>
<th>Mech. Total</th>
<th>Load Capacity Posts Only</th>
</tr>
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<tbody>
<tr>
<td>1 Span</td>
<td>Rails 2 x 246</td>
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<td>2 x 68</td>
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<td>492 kips</td>
<td>136 kips</td>
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<tr>
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<td>Rails 2 x 86</td>
<td>172 kips</td>
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<td>Post 1 x 68</td>
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<tr>
<td></td>
<td></td>
<td>240 kips</td>
<td>204 kips</td>
</tr>
<tr>
<td>3 Span</td>
<td>Rails 2 x 52</td>
<td>104 kips</td>
<td>4 x 68</td>
</tr>
<tr>
<td>L = 26'</td>
<td>Posts 2 x 68</td>
<td>136 kips</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>240 kips</td>
<td>272 kips</td>
</tr>
<tr>
<td>4 Span</td>
<td>Rails 2 x 37</td>
<td>74</td>
<td>5 x 68</td>
</tr>
<tr>
<td>L = 34.7'</td>
<td>Posts 3 x 68</td>
<td>204</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>278 kips</td>
<td>340 kips</td>
</tr>
</tbody>
</table>

See Fig. 12
WEIGHT OF INTERNAL RAIL

ALUM TUBES

12 in. O.D. x \( \frac{3}{8} \) in.

\[ 2 \times 16.1 \text{ lb/ft} = 32.2 \]

6 in. O.D. x \( \frac{5}{16} \) in.

\[ 6.6 \text{ lb/ft} \]

38.8 lb/ft

POSTS

\[ 40 \text{ ft} \times 5.17 = 25.0 \text{ lb/ft} \]

BASE PLATE, ANCHOR BOLTS

\[ \frac{5}{2} \]

BRACKETS, STIFFNERS

\[ \frac{7}{2} \]

\[ \frac{4}{8.67} \]

18.2 lb/ft

\[ 82.0 \text{ lb/ft} \]
BASE PLATE DESIGN

W12 x 50 and W12 x 40

see FIG. 16

BOLT TENSION

\[ 2T \times 14" + 2T \times 6" = M_p \]

\[ T = \frac{M_p}{40 \text{ in.}} \]

W12 x 50 POST

\[ M_p = 3620 \text{ kip-in.} \]

\( T = 90.5 \text{ kips} \)

Use 1/4" \( \phi \) A324 BOLTS

W12 x 40 POST

\[ M_p = 2875 \text{ kip-in.} \]

\( T = 71.9 \text{ kips} \)

Use 1/8" \( \phi \) A324 BOLTS
BASE PLATE THICKNESS

\[ M_p = 2 \times 90.5 \text{ kips} \times 2 \text{ in.} = 362 \text{ k-in.} \]

\[ Z = \frac{M_p}{F_y} = \frac{362 \text{ k-in.}}{50 \text{ ksi}} = 7.24 \text{ in.}^3 \]

\[ \frac{b t^2}{4} = 7.24 \]

\[ t^2 = \frac{7.24 \times 4}{10} = 2.90 \]

\[ t = 1.70 \text{ in.} \]

use 1\(\frac{3}{4}\)" for W12x50

\[ M_p = 2 \times 71.9 \text{ kips} \times 2 \text{ in.} = 287.6 \text{ k-in.} \]

\[ Z = \frac{M_p}{F_y} = \frac{287.6}{50} = 5.75 \]

\[ \frac{b t^2}{4} = 5.75 \]

\[ t^2 = \frac{5.75 \times 4}{10} = 2.30 \]

\[ t = 1.5" \]

use \(1\frac{1}{2}\)" for W12x40

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