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BARRIERS IN CONSTRUCTION ZONES

VOLUME 1

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Federal Highway Administration

Report No.
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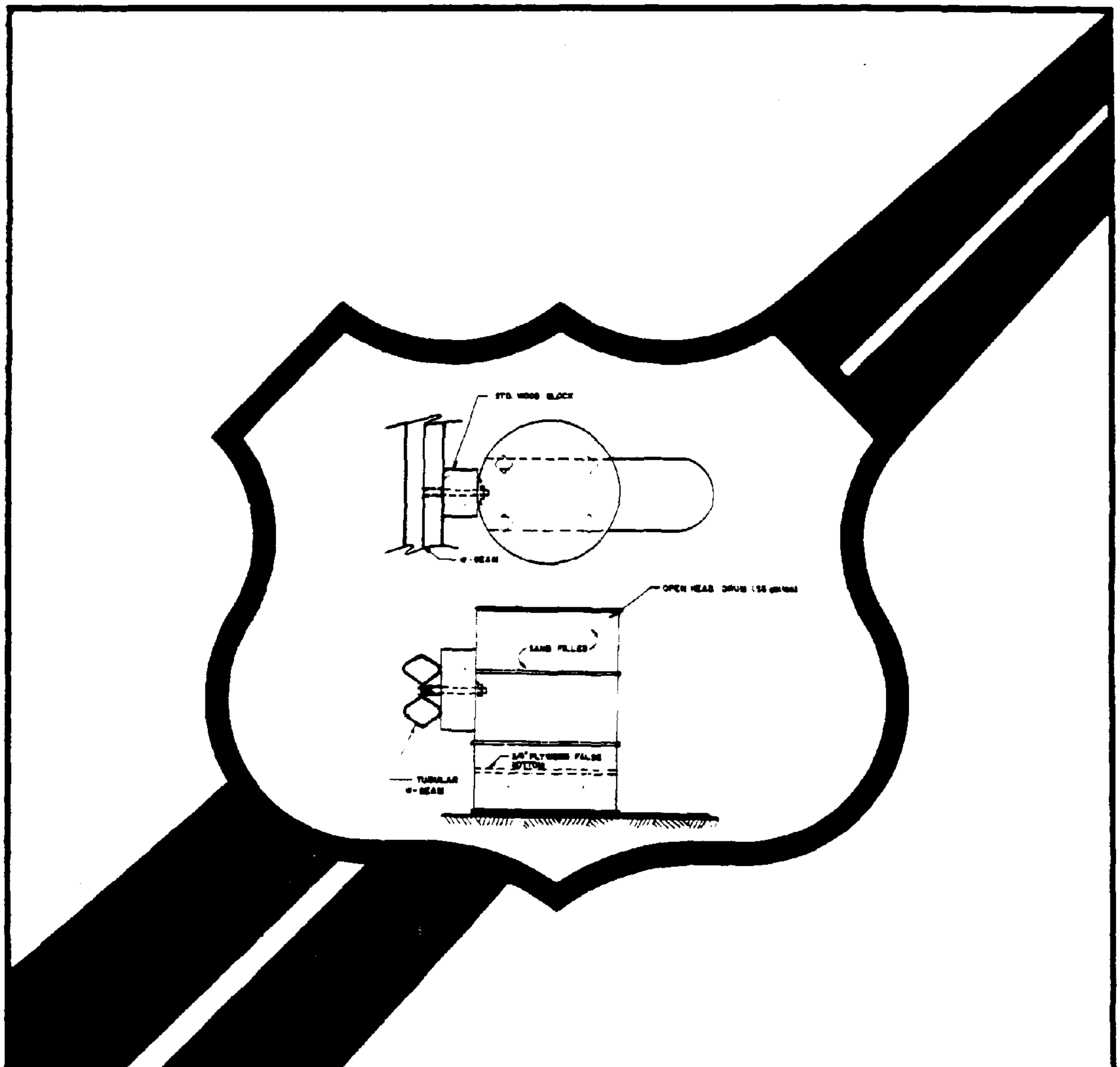
Final Report
April 1985



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16. Abstract

This study addressed the design and performance of longitudinal barriers for construction zones. The strengths of various connections for portable concrete median barriers (PCB) are analyzed and theoretical treatments of behavior of the PCB during a collision are presented. These analyses along with cost data and crash test information are used to develop a barrier performance rating and selection system. Crash tests on a non-deflecting PCB with various types and sizes of vehicles are reported.

This report consists of four volumes:

Volume 1: Summary Report
Volume 2: Appendix A - Documentation of Crash Tests
Volume 3: Appendices B, C, D, E and F - Theoretical and Economic Analyses
Volume 4: The Response Of Atypical Vehicles During Collisions With Median Barriers

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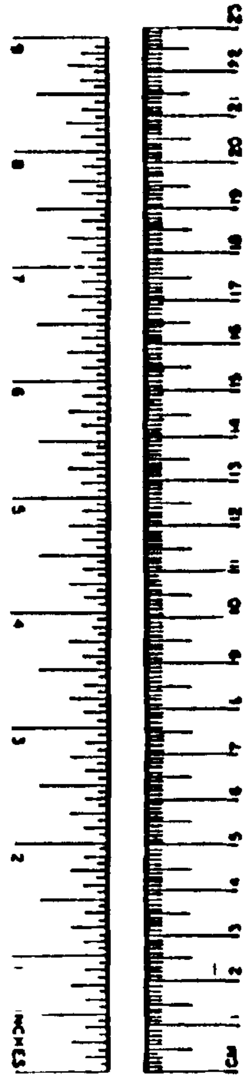
NOTICE

METRIC CONVERSION FACTORS

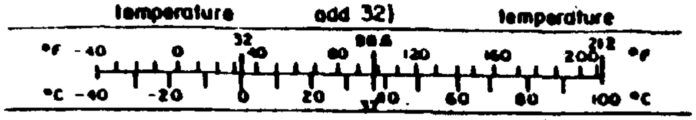
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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I. INTRODUCTION

Auxiliary structures, necessary elements of any highway system, have often been blamed for injuries and fatalities occurring in single vehicle accidents. Recognition of this fact and devotion to improved highway safety have brought about changes in designs of these structures. As a result they have now become an illustration of one way increased highway safety can be achieved.

Barriers such as bridge rails, guardrails and median barriers are a type of auxiliary structure intended to redirect errant automobiles. From the beginning, these barriers were viewed as functioning to increase safety. The movement of this type of structure from the permanent installation category to the temporary category for construction zone use has been difficult. Until the mid-1970's, redirectional barriers were not used in construction zones, but as maintenance and reconstruction activities on major freeways became common, the need for a variety of construction zone barriers was tragically illustrated by growing numbers of fatalities. In response to this need, highway engineers tried a number of temporary barriers with decidedly mixed results. Many of these stop-gap measures had not been adequately tested. Bronstad and Kimball (1), in 1977, reported tests of four temporary barriers. All of these systems exhibited either poor or marginal performance. New York (2) and California (3) conducted tests of portable concrete median barriers and of stacked timber barriers. Some designs of concrete median barriers proved adequate while tests of stacked timber barriers showed marginal performance.

It was apparent by 1978 that a systematic evaluation of portable construction zone barriers was needed to define the capabilities of some barrier systems and to upgrade those capabilities to provide better systems. The project initiated by the Federal Highway Administration at that time contained the tasks shown in Figure 1 (Original Contract). Tasks 1 and 2 were reported to the Federal Highway Administration in April of 1979 (4). At the conclusion of Task 3 and before beginning the detailed design element of Task 4 the project objectives were reoriented as shown in Figure 1 under "Revised Contract Objectives".

Task	Original Contract From July 1, 1978 to February, 1982	Revised Contract Objectives From September 1982 to April, 1984
1	Review existing barriers and failure modes and simulate barrier failures using computer programs.	Development of a construction barrier rating system.
2	Improve designs and upgrade barriers.	Estimate the strength characteristics of PCB connections.
3	Test and evaluation of barricade/barrier concepts.	Design a minimally deflecting 12 ft PCB barrier.
4	Develop new construction barrier concepts.	Develop a frequency distribution for available barrier deflection.
5	Full-scale test and evaluation of new concepts.	Completion of the original Tasks 1 through 4. Conduct 8 crash tests on a non-deflecting PCB and determine inertial properties of vehicles used in these tests.

Figure 1. Contract Tasks.

An assessment of the foregoing original and revised lists of tasks and activities demonstrates the many facets of this contract. In trying to organize all these activities in a three volume report, the reader may have difficulty in determining the overall accomplishments and advances in the state of the art without lengthy study. In an effort to help the reader selectively find those aspects of the work of primary personal interest, the following list of project accomplishments is given along with the sections of these reports under which they are documented.

<u>New Analytical Methods</u>

1. Two analytical methods of determining the influence of various parameters on PCB structural performance have been developed and used effectively. They are called the Advanced Dynamic Analysis (ADA) and the Simplified Energy Analysis (SEA). (See pages 7 through 25 and Appendices B and C).
2. An analytical method of modifying the performance of barriers that are not readily subject to closed form analysis was developed for use in upgrading the standard Barrel/W-Section barrier. An upgraded barrier demonstrating excellent performance was the result of applying this method. (See the "Method of Comparative Structural Analysis", pages 100 through 108.)

<u>Portable Concrete Barriers</u>

3. All PCB tests conducted to date have been included in a performance analysis. These include twenty crash tests. This analysis has allowed recommendations to be made for the strength of PCB connections to meet a variety of vehicle collision service levels. These strength characteristics are expressed in terms of shear, torsion and moment resisted by the connection. (See Chapter III, Construction Barrier Rating System. In particular, see page 67, Barrier Performance Levels.)
4. The load bearing capacities of most current PCB connection designs have been determined. (See Table 12, page 71 and Appendix D of Volume 3).
5. A detailed discussion of the ramping problem associated with PCB's is presented. Methods of reducing barrier deflection, a

way of reducing ramping, are described. (See pages 25 through 28.) Methods of limiting PCB deflection by use of shear connections at the ground plane have been developed. These tests illustrate the need for torsion capacity in the PCB connections so tipping of a section will not occur. (See pages 25 through 41)

6. A new PCB connection has been designed and is now under consideration by AASHTO-ARBA Task Force 13 as a hardware standard called the bottom T-Lock. It has improved strength and deflection characteristics.

Barrel W-Section Barriers

7. A new Barrel/W-Section barrier was developed that has demonstrated excellent performance during crash tests. The most severe test was a 4500 lb vehicle at 60 mph and 25 deg. Redirection was extremely smooth. The limit of performance of conventional Barrel/W-Section barrier was 4500 lb, 45 mph and 15 deg. (See pages 108 through 121)

Barrier Costs

8. A detailed method of assessing PCB barrier costs was developed. These costs included those for original construction, movement of barriers during the project and maintenance. All of these costs are related to the design of the PCB. By first assessing the need for a specific service level, and then using the cost estimating methods presented, the most efficient barrier design may be selected. The first objective assessment of PCB portability and the factors influencing portability has been made. Surprisingly, PCB segment length does not an appear to be important factor in portability. (See pages 73 through 91 and Appendix E of Volume 3.)

Space Requirements

9. A survey of construction zones was made to determine how often space limited the amount of deflection that could be accommodated by a barrier. It was concluded barriers with large deflections could be used in many construction zones, perhaps a majority of those zones. (See pages 29 through 31.)

II. PORTABLE CONCRETE BARRIER (PCB)

General

The portable concrete barrier (PCB), a precast portable version of the concrete median barrier, has become the most widely used type of construction zone barrier. It is in use throughout the nation and is actively marketed by several firms. It has performed extremely well in general, even though a number of questions were still to be answered and some limitations have been determined in the field performance.

Limitations

Consider first the limitations:

1. The PCB is a heavy barrier which is somewhat difficult to move. Barrier weight varies from 450 to 550 lb/ft (665 to 812 kg/m) depending on exact cross section geometry and degree of reinforcement. Segment lengths vary from 10 to 30 ft (3.0 to 9.1 m) and the weight of individual segments can vary from 4,500 to 16,500 lb (2,041 to 7,484 kg). These weights, even though varying by a factor of four, all require heavy equipment for movement.
2. Penetration of the barrier and/or overturning of individual segments under high intensity collisions sometimes occur. This is usually produced by vehicles larger than automobiles. This illustrates the need for longitudinal reinforcement and the increased stability that can be achieved by some degree of positive connection between segments. The capacity required of connections has been in doubt, with field practice showing differences all the way from 0 to 139 kip-ft (188 kN-m) of joint moment capacity, i.e., the moment a joint can develop about a vertical axis, the major bending axis for the predominately horizontal loads imposed by impacting vehicles.
3. The stability of vehicles, especially small vehicles, during collision is in doubt. The Road Research Laboratory reported unsatisfactory performance of mini-compacts during tests with permanent installations of concrete median barriers (CMB). More recently, Nordlin (5) reported data from California freeway accidents indicating fully 10% of the reported collisions with CMB resulted in vehicle rollover. Of this group, 73% of the passenger cars were

sub- or mini-compacts. Analysis of this data by Sides (6) and Viner (7) and comparisons with vehicle registration as a normalization scheme does appear to indicate overrepresentation of the small vehicle in rollovers. CALTRANS (3) had also reported tests of full-sized automobiles which showed the possibility of rolling if even rather small barrier rotational deflections were allowed. Rocking about the longitudinal axis may be prevented by connection torsional capacity.

In spite of these limitations, it must be emphasized that performance of the portable concrete barrier is generally good, redirecting thousands of vehicles every month: a record that fully justifies interim use, at least until better designs are established.

Need for Specific Answers (Portable Concrete Barriers)

Some specific questions that remained to be answered were:

1. How much joint structural capacity is required? It is obviously different for redirecting mini-compacts and redirecting school buses.
2. Would the barrier function adequately for a school bus? If not, what changes would be required to provide a safe school bus redirection?
3. What changes would be necessary to prevent vehicle ramping under high intensity collisions, i.e., speeds up to 60 mph (96.5 km/h) and impact angles up to 25 deg?
4. What changes can be made in either barrier segment size or connection details to increase portability?

Testing Program

To develop answers to these questions, a test program was set up which involved barriers composed of 12 ft (3.7 m) and 15 ft (4.6 m) PCB segments with different conditions of joint moment capacity. The objectives of these tests were: a) to determine empirically the capacity required to resist the impact of a 4,500 lb (2,041 kg) vehicle at 60 mph (96.5 km/h) and 25 deg (the most severe test advocated by TRC 191; b) to relate the theoretical analyses to actual tests; and c) to determine if a PCB could be designed with the structural adequacy to redirect a school bus.

Table 1 shows the tests that were performed in this series. Tests 1 through 4 were devoted to objectives a) and c). These tests were conducted in the order shown by test number, with the joint moment capacity reduced for each test until failure of the joint connection was achieved at a level of 26 kip-ft (35.3 kN-m). The cross section geometric properties and reinforcement are shown in Figures 2 and 3 for the 12-ft (3.7 m) and 15-ft (4.6 m) barrier segment lengths, respectively. Details of Tests 1 through 5 are given in Appendix A under the Proving Ground Designation Number.

The final test of this series, Test No. 5, was intended to demonstrate that a PCB could be expected to contain and redirect a school bus. In this test, the barrier shown in Figure 4 was used. The segment length was 15 ft (4.6 m) with a strong joint connection capable of 160 kip-ft (217 kN-m) of bending moment. This connection is shown in Figure 30 page 56. This barrier was originally designed by Ross (8). The bus was contained and redirected, but went through a slow roll, falling and remaining on the right side as shown in Figures 5 and 6. Details of this test are given in Appendix A under Test 3825-8. The accelerations were reasonable, but because the bus rolled, the result could only be considered marginally successful. It did demonstrate the capability of a PCB to accommodate the loads from a large vehicle impact.

Simplified Energy Analysis (SEA)

A comparison of these test results with the SEA predictions was presented in the Interim Report of Tasks 1 and 2 (4) is given by Figure 7. The details of this analysis are presented in Appendix B. SEA was developed specifically to predict barrier deflection during automobile impact and was, in part, based on methods presented by R. L. Stoughton of CALTRANS in an analysis he performed in 1977. This analysis has come to be called the Simplified Energy Analysis (SEA) to distinguish it from the more sophisticated analysis given in Appendix B.

The comparison indicates a degree of quantitative correlation in the tests of barriers with joint moment capacities between 50 and 100 kip-ft (68 and 136 kN-M), although the trend which seems self-evident, i.e. reduced barrier deflection with increased moment capacity, is not apparent from Tests 1, 2 and 3. Several factors must be considered here. Foremost

Table 1. Test Matrix.

Test No.	Proving Grounds Designation #	Vehicle Test Conditions (lb/mph/deg)	Joint Moment Capacity (kip-ft)	Static Deflection (ft)	Special Conditions and Results
1	3825-7 (12 ft segment)	4,500/59.2/25	103	1.8	Vehicle redirected with some ramping.
2	3825-6 (12 ft segment)	4,500/60.1/24	77	1.8	Vehicle redirected but ramped and rolled.
3	3825-5 (12 ft segment)	4,500/60.7/25	52	1.6	Vehicle redirected with some ramping.
4	3825-9* (12 ft segment)	4,510/63.4/25	26	6.5	Vehicle redirected without ramping. Three base plates failed.
5	3825-8 (15 ft segment)	20,000/57.7/15	160	1.8	Bus redirected but rolled on side.

*In this test a blocked-out steel W-section was placed on the side of the PCB to see if such an addition would reduce ramping. Thus the vehicle stability results are not representative of a normally shaped PCB.

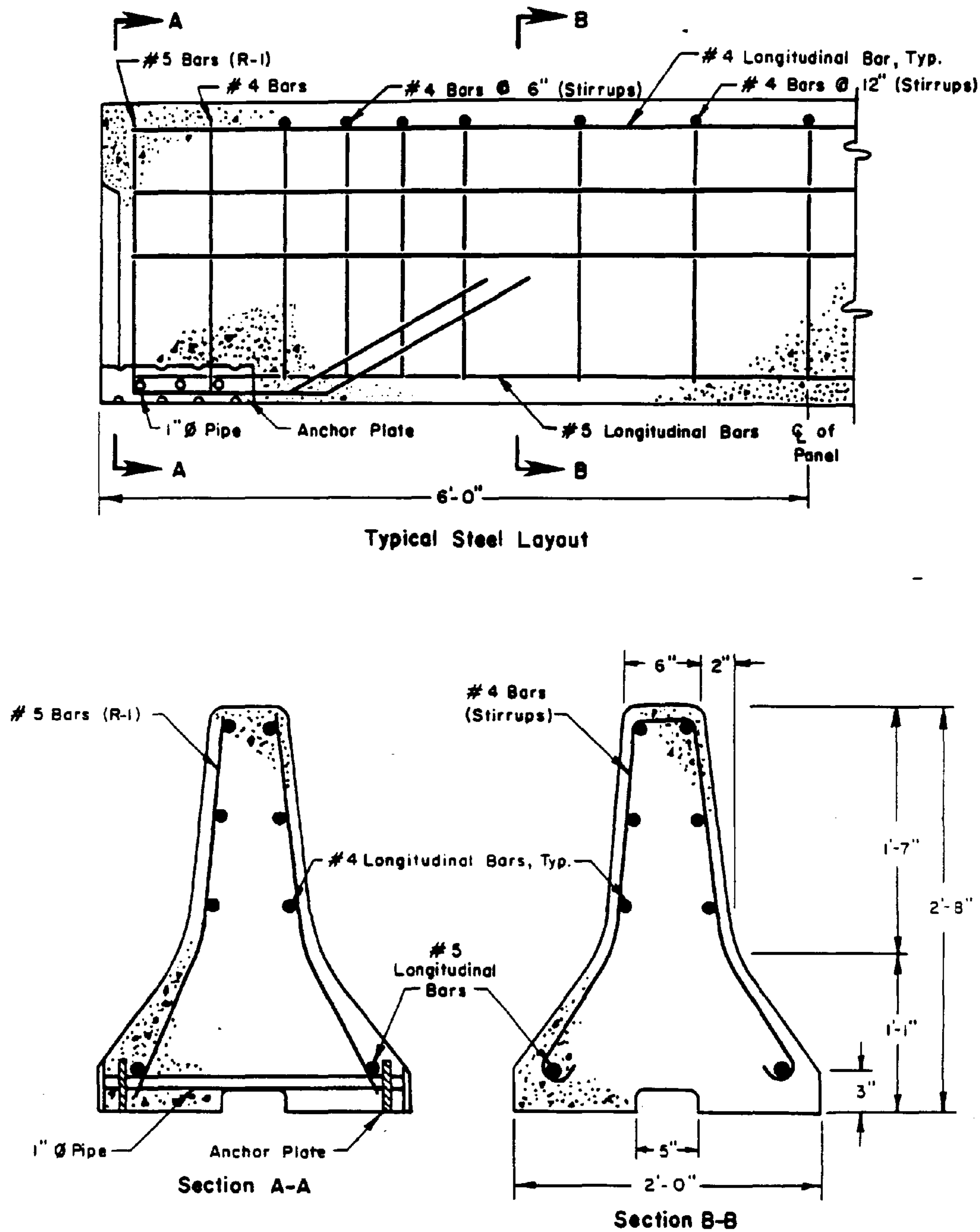


Figure 2. Details of Reinforcement for 12-ft Long CMB Segments.

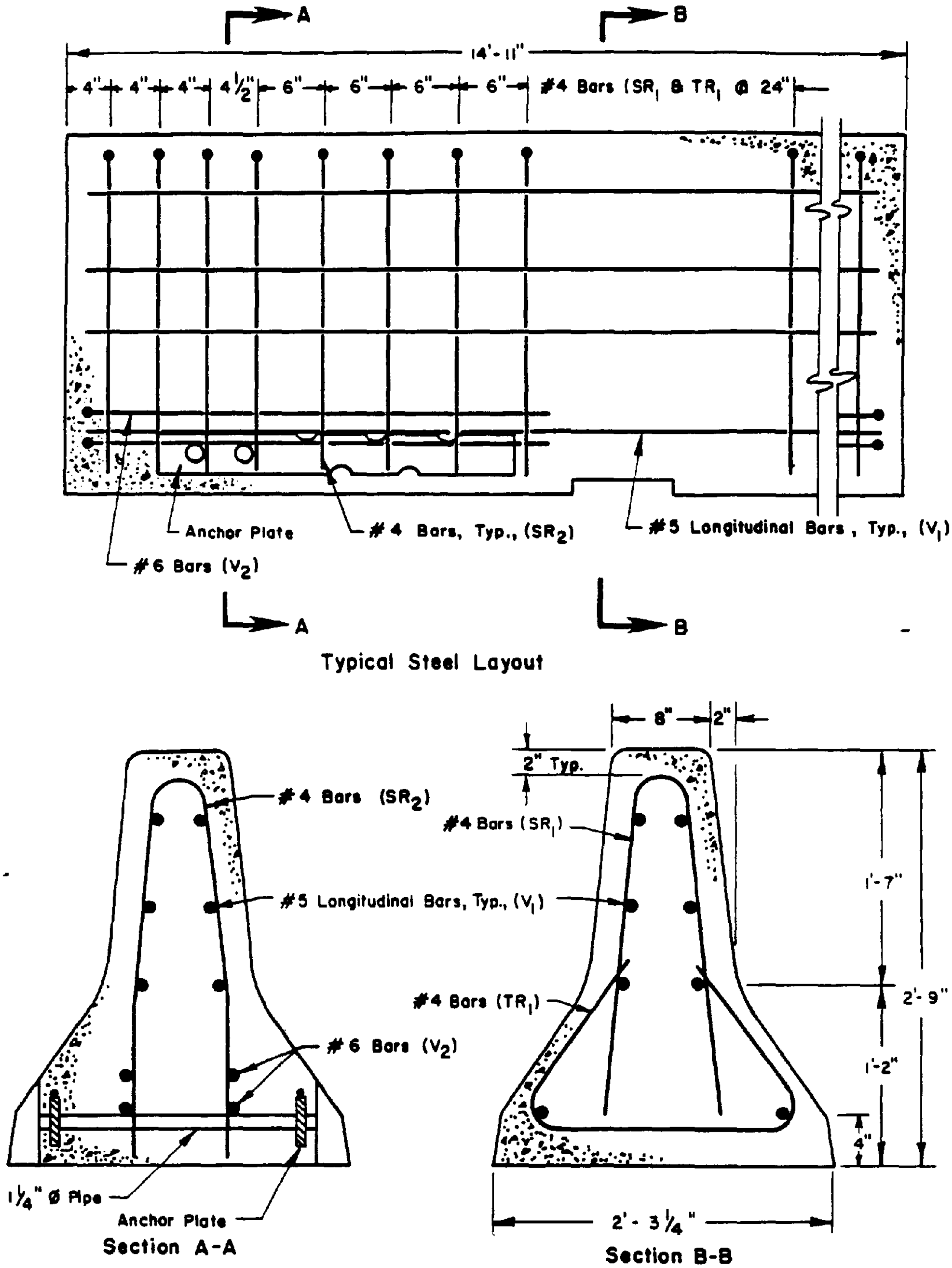


Figure 3. Details of Reinforcement for 15-ft Long CMB Supports.

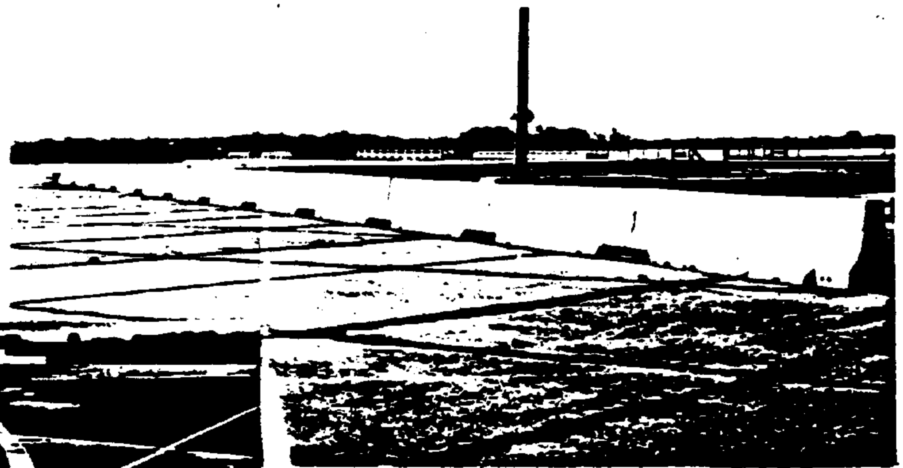
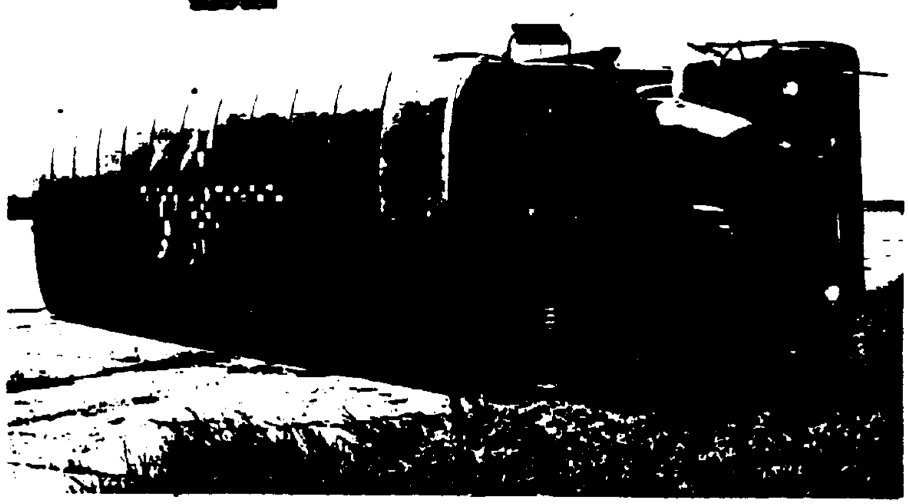


Figure 4. Barrier Before Test 5 (3825-8).

Figure 5. Barrier After Test 5 (3825-8).

Figure 6. Bus After Test 5 (3825-8).



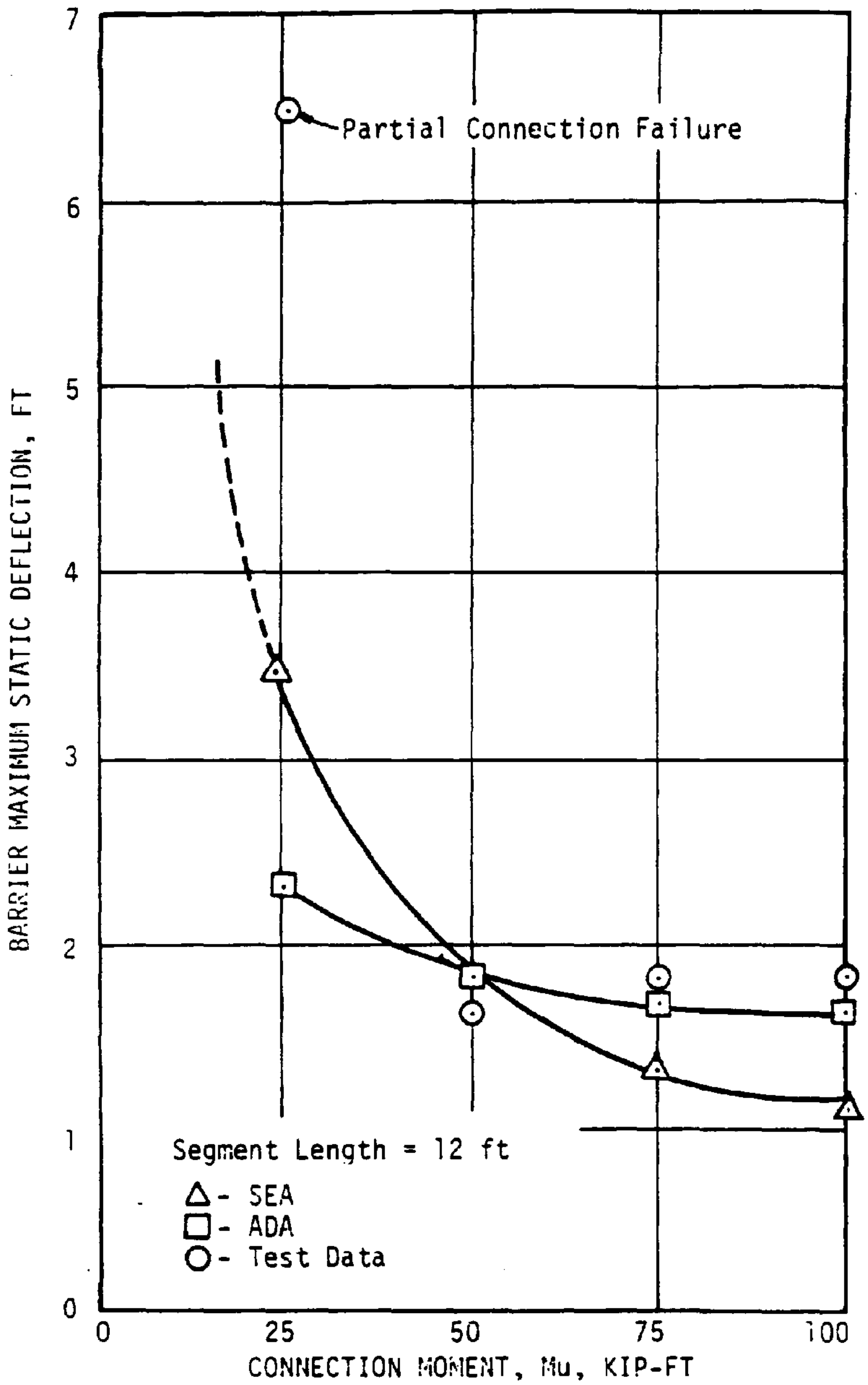


Figure 7. Comparison of Test Deflections with Simplified Energy Analysis (SEA) and with Advanced Dynamic Analysis (ADA).

may be the fact there was no yielding of the connection plates in these three tests. Since the yield condition was not achieved in any of these three tests, each connection functioned similarly with small differences in terms of connection slack in any specific installation accounting for some of the small differences in barrier deflection.

An assumption in SEA, that only two concrete segments move, plays a part here. The fact that many segments may actually move, seven in Test 2, allows energy absorption by joint slippage, thus reducing the requirement for connection yielding in the three connections closest to the impact zone.

One of the most critical parts of the interaction of barrier and vehicle during collision is the energy absorbed by vehicle structure, a factor subject to significant variation. The way in which the vehicle ramps during collision influences the amount of energy that is absorbed by vehicle crush. Ramping, apparent in Tests 1, 2 and 3, is a factor which reduces the energy both the vehicle and barrier are required to absorb. -

Considering Test 4, where failure of three connection plates occurred, the fact that the barrier deflection was nearly twice that predicted by the SEA is not surprising. The SEA does not provide for a discontinuity produced by a parting of several connection plates, even though it does provide for a reduction in connection moment to zero after full connection yielding has occurred. When the tension side plates parted, the only energy absorption mechanism remaining was the sliding friction of the barrier segments on the ground. Since the remaining deflection would be extremely sensitive to sliding friction, this estimated friction in the SEA analysis may be the factor that caused the main difference in predicted and observed barrier deflection.

Another factor is the equation which describes the energy absorbed by a vehicle during the collision. Since this equation was based on PCB tests where significant vehicle ramping occurred, and since the attached W-section in Test 4 prevented this ramping, a somewhat atypical interaction occurred.

The purpose of Tests 1 through 4 was to provide empirical data which could be compared to the analyses developed under this study and, if practical, to determine empirically the actual connection strength needed under critical test conditions. The accomplishment of the first comparison has been shown. The connection strength needed was also effectively determined by this sequence of tests.

In Test 3, with a connection strength for yielding of 52 kip-ft (71 kN-m), no yielding was apparent. When the yield strength of the connection was reduced to 26 kip-ft (35 kN-m) in Test 4, failure occurred. Since the ultimate strength of the 26 kip-ft (35 kN-m) connection in Test 4 was probably at least as high as 40 kip-ft (54 kN-m), and since this ultimate strength was exceeded, the lower limit of barrier connection capacity for PCB's designed for automobiles can be set at 50 kip-ft (68 kN-m), a level that is not difficult to achieve structurally and one which has successfully accommodated the most critical automobile test: 4,500 lb/60 mph/25 deg (2,041 kg/96.5 km/h/25 deg) (See Test 3). It should be noted that the addition of a W-section guardrail made the amount of energy to be absorbed by the connection somewhat greater. This is because ramping was prevented. Though this rendered the structure tested somewhat different from those tested in Tests 1, 2 and 3, the results are interpreted in a conservative way since the W-section would not add significant moment capacity.

Advanced Dynamic Analysis (ADA)

The analysis developed by Ross and Walker is presented in Appendix C. This analysis will be called the Advanced Dynamic Analysis or ADA. It has a number of advantages compared to SEA. A major advantage is that it allows movement of many joints. A limitation is that the force-time history of a specific impact is required as part of the program input data. Although this was something of a problem at the beginning of this project, there now have been sufficient tests conducted on PCB's and on flat wall surfaces with full instrumentation that a conservative prediction of the force-time history can be made with some confidence. Examples of how these force-time histories were estimated for use in the program can be found on pages 42 and 46 of Appendix C, Volume 3.

Figure 7 compares the ADA simulation with the four crash tests described in Table 1. It is seen, in the range of connection moments between 50 and 100 kip-ft (68 and 136 kN-m), the ADA analysis compares quite well with the test data. Again, as in the comparison with SEA, when

Table 2. Input Data and Correlation of Computer Programs.

TEST NO.	V (mph)	θ (deg)	W (lb)	L (ft)	Mu (k-ft)	ΔMAX OBSERVED (ft)	(ADA) DYNAMIC PROGRAM ΔMAX (ft)	(SEA) ENERGY BALANCE ΔMAX (ft)
CAL-291	65	7	4,860	12.5	9.00	0.52	0.7	0.2
CAL-294	39	25	4,700	20.0	12.00	0.46	0.6	0.5
CMB-24	56	24	4,500	20.0	8.00	3.42	3.5	3.8
NY-1	53	25	4,250	20.0	96.00	1.33	3.6	2.2
NY-2	55	25	4,230	20.0	96.00	0.92	2.1	0.6
CMB-2	60	24	4,540	30.0	50.00	1.10	2.1	0.8
TTI-1	60	17	4,500	15.0	150.00	1.00	1.1	0.8
TTI-2	60	26	4,500	15.0	160.00	1.33	1.6	1.3
1	59	25	4,500	12.0	103.50	1.80	1.6	0.9
2	60	24	4,500	12.0	77.38	1.80	1.8	1.1
3	61	25	4,500	12.0	52.25	1.60	2.2	2.0
4	63	25	4,500	12.0	26.13	6.50	2.3	3.4
5	60	15	20,000	15.0	160.00	1.83	5.7	2.2

connection failures occur, the predictions of deflection by ADA are inaccurate.

Comparison of SEA and ADA with Crash Tests

In an effort to secure the most comprehensive evaluation of the analytical methods developed, analyses were performed using documention available on thirteen barrier crash tests. These analyses were compared with observed values of barrier deflection. Table 2 documents these tests and the results of the two analytical methods. They include two California tests (3), one Southwest Research Institute test (9), two New York tests (2), and three Texas Transportation Institute tests (8), in addition to the five tests conducted during this study.

The major problem encountered was in estimating appropriate values for connection slack, the moments developed by joint rotation, and the barrier to ground friction values. Many of these factors could not be accurately determined, even for the current tests. Estimating them for tests reported in the literature was highly conjectural. Nevertheless, they are presented with the above severe limitations. The comparison of calculated and observed deflections is shown by Figure 8. This figure shows a fair grouping of the data about the "line of equality" with some notable exceptions.

The school bus test (3825-8 or Test No. 5) falls far above the line of equality for ADA. This would appear to be the result of barrier force reductions due to vehicle rolling, a factor neither analysis considers. It may be a fortunate compensating factor in SEA that the energy estimated for vehicle deformation is overstated. This results in predicting a reduced barrier deflection, to a level more consistent with the test results. Thus, the neglected energy going into rolling the vehicle may be accommodated by the apparent overstatement of vehicle crush energy.

The test where the connection failed (3825-9 or Test No. 4) also resulted in a major discrepancy between predicted and observed deflections. Neither analysis was successful in predicting this failure, which may be primarily due to the prevention of ramping, thus transmitting more force and energy to the barrier than in the usual concrete barrier tests.

Finally, the analysis of Test NY-1 compared unfavorably with the observed deflection, especially for the ADA analysis, which predicted a

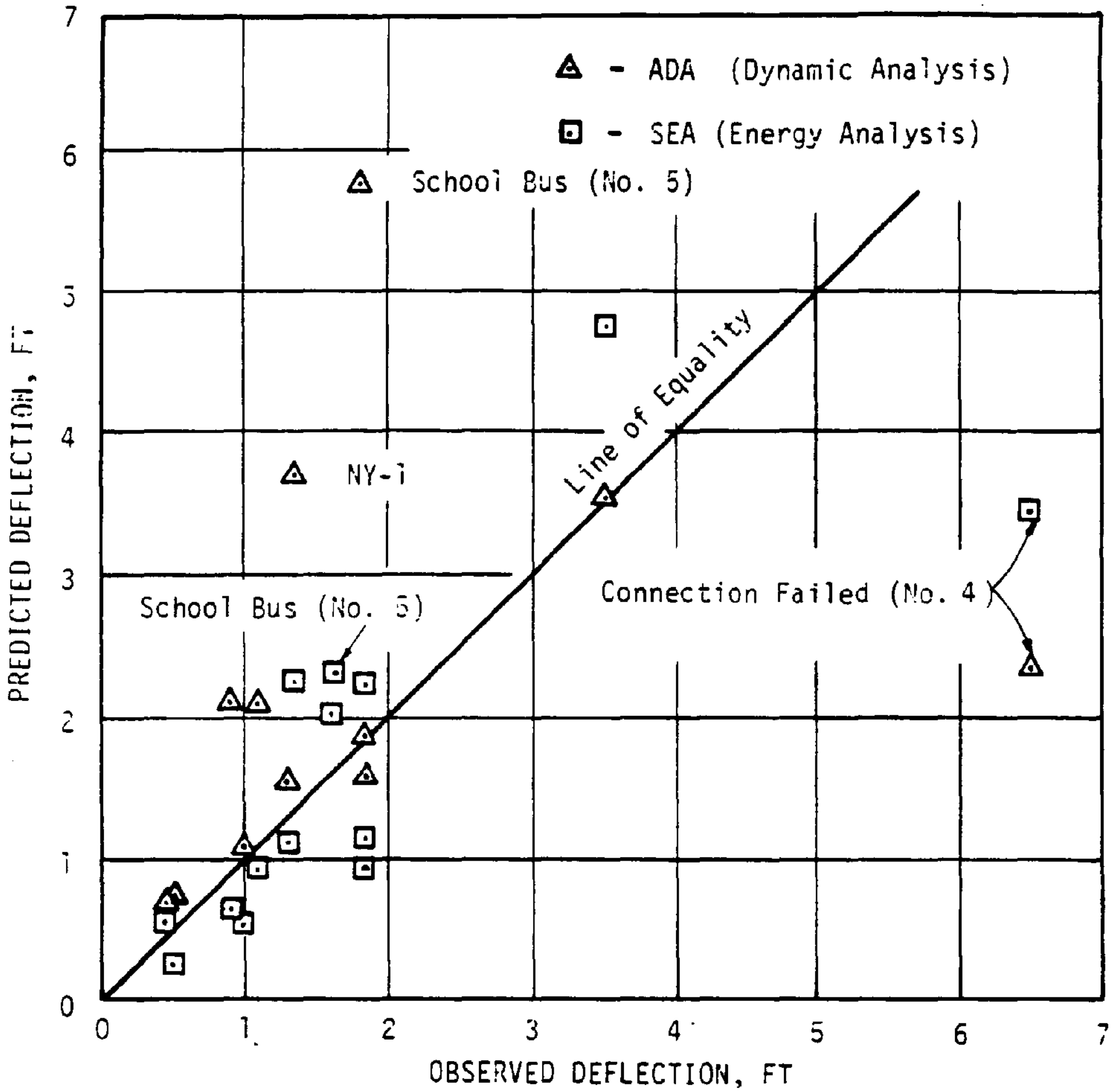


Figure 8. Correlation of Analytical Predictions with Observed Deflections.

deflection of 3.6 ft (1.1 m) compared with an observed value of 1.3 ft (0.4 m). SEA predicted a value of 2.2 ft (0.7 m), which is also high. A distinct possibility here is that an overestimate of the amount of slack in the barrier connection, a 10 degree value derived from the barrier plans, resulted in the high deflection predicted by analysis. Another possibility is that some barrier ground anchorage was achieved which was not considered in either analysis.

After considering the reasons for these apparent anomalies and the many estimates required in the analyses, it was concluded that a fair representation of barrier performance had been achieved. It would then be appropriate to use one analysis to perform parametric studies. These studies are given in Appendix C for ADA, and were given for SEA in a previous project report (4). The SEA study is repeated in the following section. It is used to illustrate the influence of important barrier characteristics.

Parametric Study Using Simplified Energy Analysis (SEA)

Parametric studies were conducted to determine, for the first time, barrier sensitivity to five different characteristics. These characteristics are connection moment capacity, barrier segment length, connection slack, barrier mass, and barrier to ground friction.

Concerning the connection moment capacity, Figure 9 shows a fairly low sensitivity of barrier deflection to connection moments between 50 kip/ft and 100 kip/ft (68 kN-m and 136 kN-m) for segment lengths of 12 to 30 ft (3.7 to 9.1 m). With connection moments below 50 kip/ft (68 kN-m), sensitivity increases (deflection increases rapidly) between 25 and 50 kip/ft (34 and 68 kN-m) for barrier lengths of 12 and 20 ft (3.7 and 6.1 m). Barrier segment lengths of 30 ft (9.1 m) are not very sensitive to connection moment at the 4,500 lb/60 mph/25 deg (2,041 kg/95.6 km/h/25 deg) test condition.

This does not imply, however, that no connection at all is appropriate for the 30 ft (9.1 m) segment length barrier. The program assumes continuity is maintained even though the moment developed at the connection is zero. This would be like the classic pinned connection during rotation before all the slack is taken up. That is, the connection was still assumed to develop shear and torsion forces sufficient to

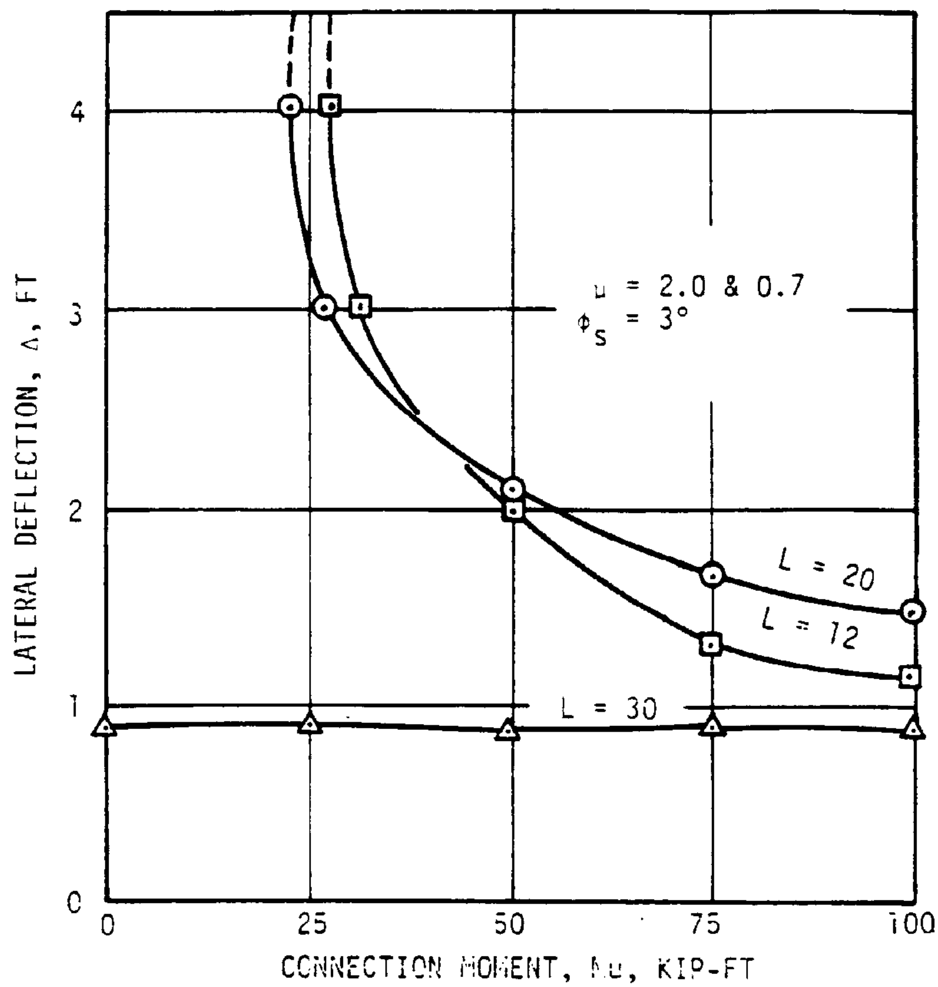


Figure 9. Influence of Connection Moment on Barrier Deflection. Test Conditions: 4,500 lb/60 mph/25 deg (2,041 kg/96.5 km/h/25 deg).

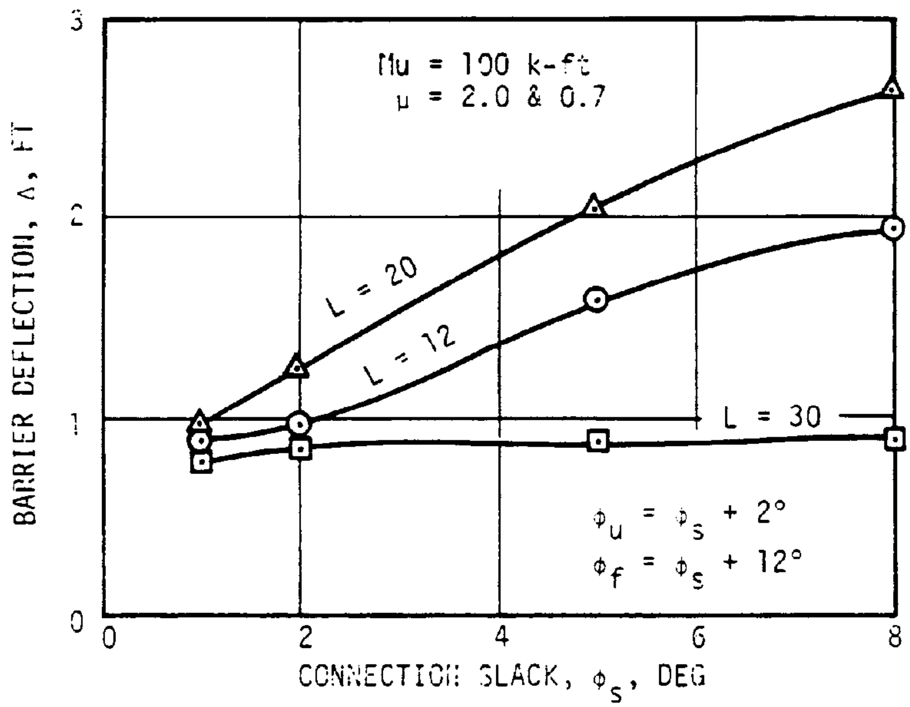


Figure 10. Influence of Connection Slack on Barrier Deflection.
Test Conditions: 4,500 lb/60 mph/25 deg
(2,041 kg/96.5 km/h/25 deg).

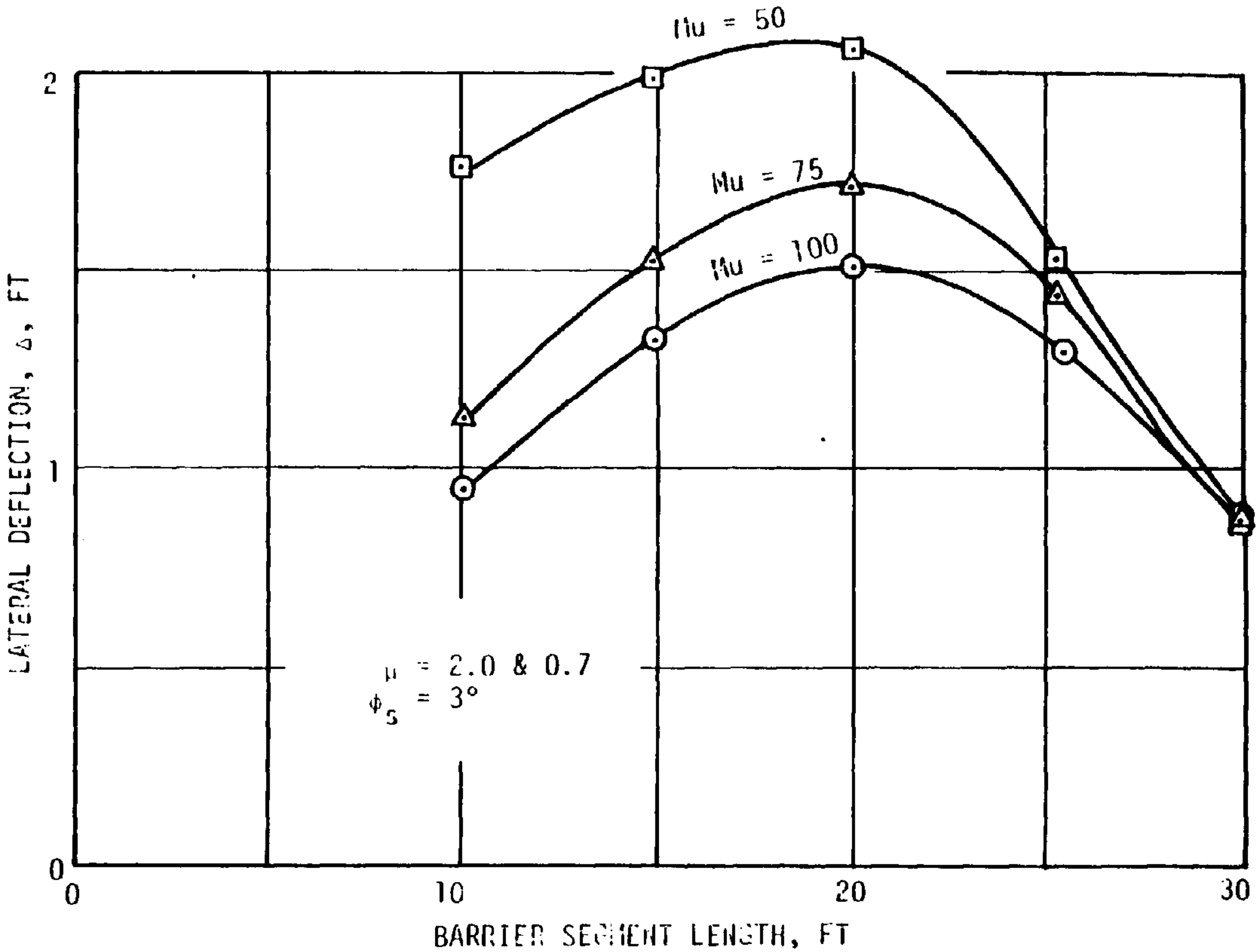


Figure 11. Influence of Barrier Segment Length.
Test Conditions: 4,500 lb/60 mph/25 deg
(2,041 kg/96.5 km/h/25 deg).

maintain alignment continuity between the adjoining ends of the segments.

The effect of connection slack, the amount one segment can rotate with respect to an adjacent segment before significant yaw moment is produced, is sensitive for segment lengths of 12 and 20 ft (3.7 and 6.1 m). Figure 10 shows an increase in deflection from 1 to 2 ft (0.3 to 0.6 m) as connection slack goes from 2 to 8 deg for a segment length of 12 ft (3.7 m); for a 30 ft (9.1 m) segment length, the sensitivity in this range is negligible.

An interesting, if perhaps predictable, phenomenon occurs when the influence of barrier segment length is determined for different values of connection moment capacity. Figure 11 shows barrier deflection first increases with length up to a maximum at about 20 ft (6.1 m) and then decreases for segment lengths above 20 ft (6.1 m). This phenomenon becomes less pronounced for smaller values of moment capacity and disappears entirely for a zero moment capacity. At zero moment capacity, deflection becomes continuously smaller as segment lengths increase. The reason for this is the interdependence of moment capacity and friction on segment length. With high joint moment capacities, the influence of relatively large joint rotations at fairly small deflections produces significantly higher energy absorption for short segment lengths. In contrast, as segment lengths become large, between 20 and 30 ft (6.1 and 9.1 m), the joint rotation at a given deflection becomes smaller and the overriding influence of barrier energy absorption due to friction becomes dominant.

The influence of static and sliding friction on barrier deflection is shown in Figures 12 and 13, respectively. The influence of positive connection techniques, such as the dowels provided by California on some temporary installations, can be accommodated by selecting an appropriately high value of static friction. It is also illustrated that an extremely large value of static friction, or a positive barrier to ground connection, would be required to make barrier deflections approach zero.

Concerning the lack of deflection sensitivity to sliding friction, this would be expected since the total deflection is only about one ft (0.3 m) for the conditions shown in Figure 13. For small deflections, the sliding friction would provide a very small part of the energy dissipated for the relatively small range sliding friction would normally span.

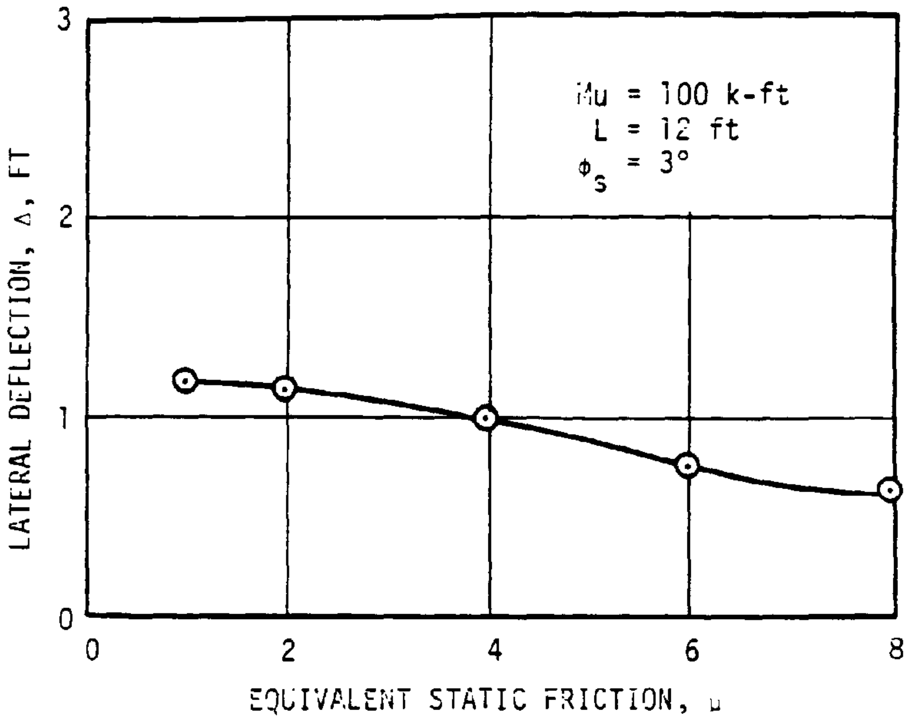


Figure 12. Sensitivity to Equivalent Static Friction.
Test Conditions: 4,500 lb/60 mph/25 deg
(2,041 kg/96.5 km/h/25 deg).

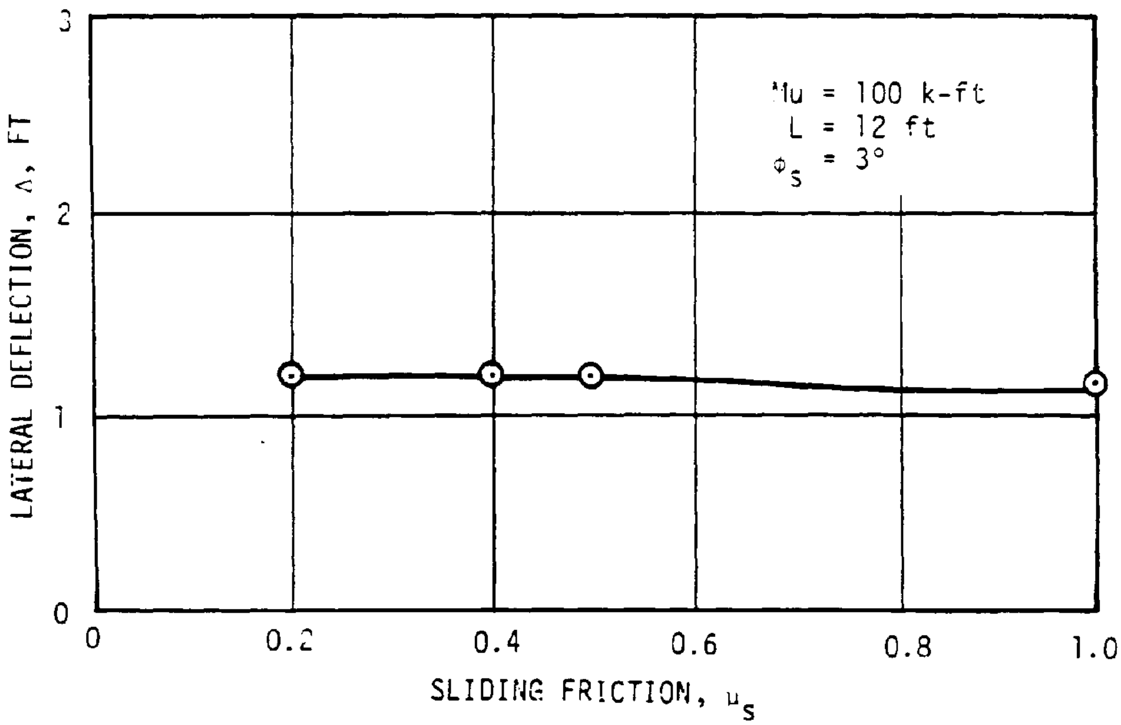


Figure 13. Sensitivity to Sliding Friction.
Test Conditions: 4,500 lb/60 mph/25 deg
(2,041 kg/96.5 km/h/25 deg).

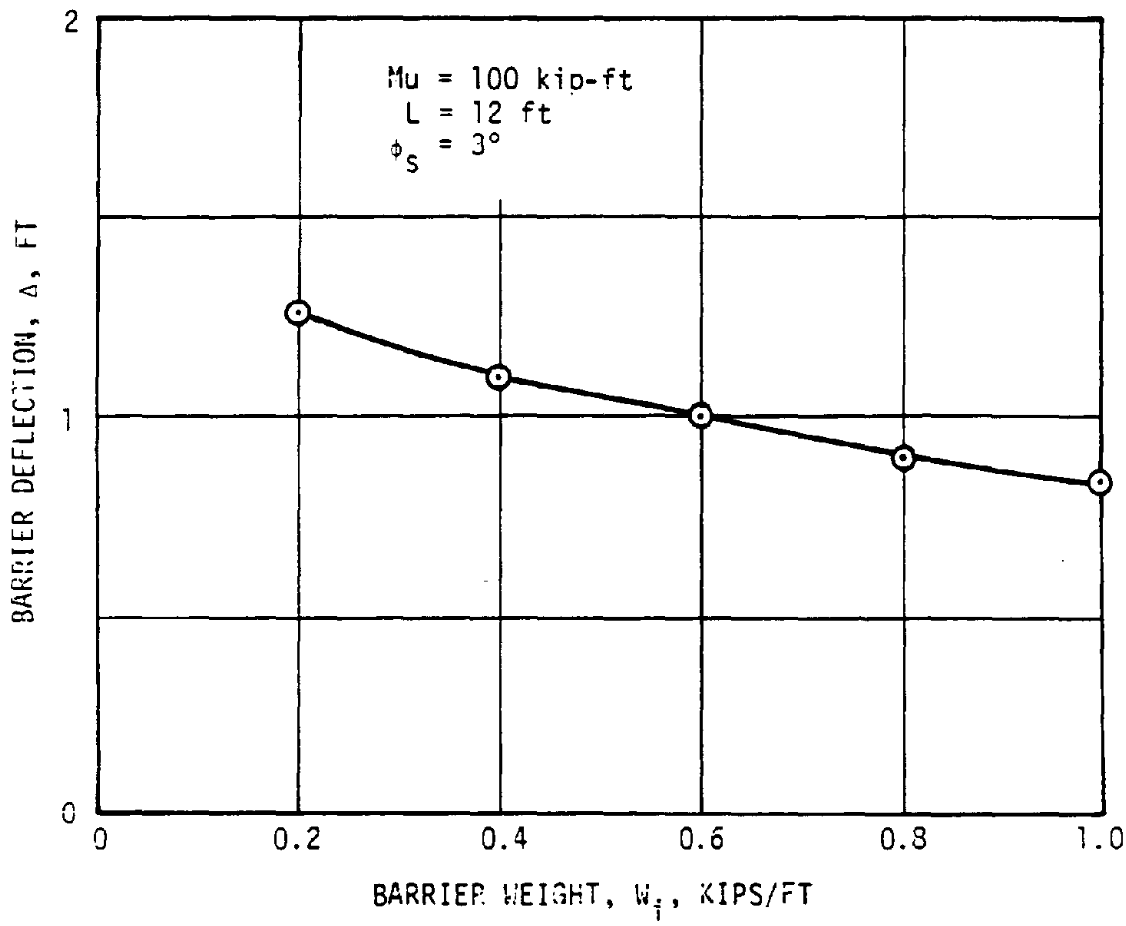


Figure 14. Influence of Barrier Weight.
Test Conditions: 4,500 lb/60 mph/25 deg
(2,041 kg/96.5 km/h/25 deg).

The influence of barrier mass, although perhaps academic at this stage, is shown in Figure 14. Increasing the mass of PCB's through the use of heavyweight aggregate is possible although it is unlikely to be economically justified. Decreasing the mass by the use of lightweight aggregate would not result in greatly magnified barrier deflections. It is doubtful, however, whether the increased cost of lightweight aggregate concrete would be justified in terms of increased barrier portability.

At this stage, the energy analysis appears to be a useful tool in predicting barrier deflection characteristics. It cannot, however, predict vehicle response. This must be inferred from actual test data. Nor does it predict whether connections have the necessary shear or torsion characteristics to avoid the development of structural discontinuities. The Advanced Dynamic Analysis (ADA) can be used to estimate shear and torsion capacity requirements as can the detailed analysis of the significant number of crash tests that have now been conducted. In Chapter III an analysis of these tests is presented to estimate shear, torsion and moment requirements.

Based primarily on the SEA analysis it appears that barrier deflection can be limited to 1.5 ft (0.5 m) or less under the extreme test condition of 4,500 lb (2,043 kg), 60 mph (97 km/h) and 25 deg, for practical values of barrier dimension, connection slack and connection strength. This limitation of deflection does not always assure that the impacting vehicle will not ultimately roll under these extreme conditions as will be demonstrated by Test No. 2 (3825-6).

Prevention of Ramping

As discussed earlier, a characteristic of major significance is the tendency toward ramping and possibly rolling in some vehicle tests. The obvious solution was to hold the vehicle front end down during the initial impact, for it is this rise of the vehicle front impacting corner that is the forerunner of a significant elevation of the whole vehicle. It was considered highly probable that any significant projection of the barrier out toward the vehicle at a position on the barrier face slightly above the initial impact zone would prevent the impacting vehicle corner from rising. This would eliminate tendencies to ramp.

This idea was tested by Test 4, where a blockout W-section guardrail was secured at a level of 26 in. (66.0 cm) above the road surface. This design modification is shown in Figures 15 and 16. It was anticipated the vehicle would start to ramp as the wheel on the impacting corner of the vehicle moved into contact with the sloped surface, but this rise would be opposed by the projecting W-section. The performance was as anticipated. All tendency toward ramping, so prevalent in Tests 1, 2 and 3, was eliminated. The impacting vehicle remained on a primarily two dimensional course through the test, finally spinning out downstream of the impact zone with heavy right front corner damage. The use of the blocked out W-section is not meant to imply PCB's or permanently installed CMB's should be retrofitted with W-section. The barrier was so fitted simply to demonstrate the effectiveness of some structural projection in preventing ramping. Further, the prevention of ramping has some inherent drawbacks in the case of a high mass barrier such as PCB.

While the danger of rolling is greatly reduced, a price is paid. A ramping vehicle gains potential energy, resulting in less kinetic energy being absorbed by the vehicle. By preventing ramping, more vehicle damage is produced in the initial impact, resulting in somewhat higher levels of longitudinal and transverse accelerations. For comparison purposes, consider the accelerations in Test 3 and 4 (shown in Table 3), where the vehicle test conditions are virtually the same.

The resultant of the ground plane accelerations shows 9.6 g for Test 3 (ramping occurred) compared to 13.2 g for Test 4 (no ramping). Although the barrier deflections are not directly comparable due to the connection plate failures in Test 4, this further indicates the probability of higher accelerations during the initial impact conditions if ramping is prevented. That is, if the deflection of the barrier in Test 4 had been only as large as the deflection in Test 3, the accelerations observed in Test 4 would probably have been significantly higher.

If it is ultimately determined that ramping must be reduced, due possibly to small vehicle roll sensitivity, a way to retrofit PCB's and CMB's has been demonstrated, although the performance of the resulting barrier profile has not been verified by testing. Figure 16 shows an existing CMB modified by fastening 2 in. x 6 in. x 10 ft precast concrete plates to the barrier face. The PCC plates are attached using a commercially available epoxy. Three epoxy formulations provided by the 3M Company were used in the demonstrations. All have performed adequately during exposure to the central Texas climate for two years.

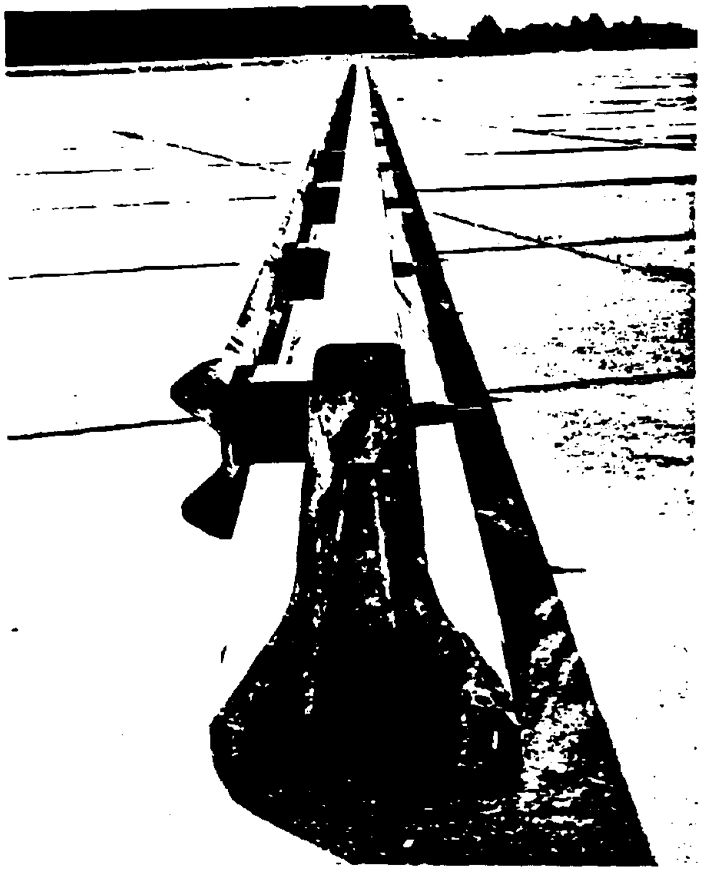


Figure 15. Portable Concrete Barrier With W-Section Attached, Test 4 (3825-9).

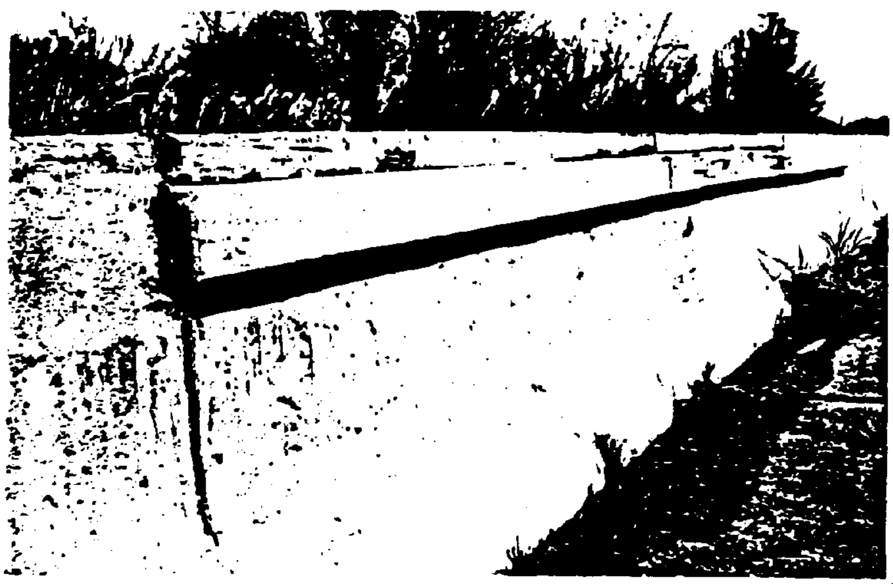


Figure 16. Concrete Median Barrier Modified by Concrete Plates to Reduce Ramping of Small Vehicles.

Table 3. Accelerations for Test 3 and 4.

	Test 3	Test 4
Weight	4,500 lb (2,041 kg)	4,500 lb (2,041 kg)
Speed	60.7 mph (97.7 km/h)	63.4 mph (102.0 km/h)
Angle	25 deg	25 deg
Permanent Deflection	1.6 ft (0.5 m)	6.5 ft (2.0 m)
Longitudinal Acceleration Max. 50 msec	6.2 g	8.8 g
Transverse Acceleration Max. 50 msec	7.5 g	9.9 g
Resultant of Transverse and Longitudinal Acceleration Max. 50 msec	9.6 g	13.2 g
Vertical Acceleration Max. 50 msec	8.2 g	2.2 g

The Space Available for Barrier Deflection

A question that has never been definitively answered is: What space is available behind construction barriers to allow deflection without having an influence on the safety of either the public or construction workers? Obviously the answer for a specific installation is sometimes zero and sometimes unlimited. As barrier designs with widely varying deflection characteristics are considered, however, some indication of the frequency with which certain conditions occur is needed.

For four years, the writers have made observations of construction site barrier installations, finally developing one hundred observations of barrier and site conditions. These observations were made in eight states, usually in daylight conditions. The states, the number of sites visited in each, and the information acquired is shown in Table 4. The number of observations is not the same as the number of sites. About 20% of the sites were subdivided into from two to four observation areas because of site variability.

Most of the observations were made in urban areas, consistent with the fact that most barrier use occurs there. The survey is neither random nor statistically designed. It is simply that which the project staff could manage to acquire during travel, for predominantly other purposes, during the contract. The types of barriers are predominantly portable concrete barriers, W-section on barrels and delineation devices (individual barrels, paddle boards or cones).

Figure 17 shows the frequency of available deflection distances weighted by barrier length. There was a total of 120,490 ft (36,725 m) of barrier surveyed. Less than 10% showed available deflection distances of one foot or less while over 70% exhibited distances of 10 ft or more. This heavily skewed distribution is also illustrated by Figure 18.

Information was also needed concerning the depth of pavement drops or elevation changes adjacent to traffic lanes through construction zones. Only 16 out of one hundred observations involved an elevation drop adjacent to a lane or immediately behind a barrier. For the sites that involved some type of barrier, a tabulation of cut depth, available deflection distance and estimated Service Level (See Chapter III, Table 8 for definition of construction barrier service levels) is given in Table 5. The non-relationship of cut depth (Depth of Drop) and available

Table 4. Details of Site Observations.

States:			
California	(5)*	Texas	(39)
Georgia	(4)	Virginia	(1)
Louisiana	(6)	Washington, D.C.	(4)
Maryland	(3)	Wisconsin	(1)
Data Acquired on Each Observation:			
Survey Date		Type of Barrier	
Surveyor		Length of Barrier	
Location of Site		Nature of Hazard/s	
Roadway Type		Type of Delineation	
Length of Zone		Available Deflection Distance	

Note: The available deflection distance was recorded at the time of observation only. Barriers having large available deflection distances during observation might have low values at other times due to construction machinery or personnel activities.

* Number of observations.

deflection distance is shown by Figure 19. The non-relationship between cut depth and estimated Service Level is shown by Figure 20. As these figures illustrate, the degree of hazard, as approximated by the combination of available deflection distance, and the Depth of Drop is not logically related to barrier Service Level, or at least such a relationship is not indicated by this data. If anything, this data points to the need for practical guidelines for construction barrier selection, the subject of Ross' (8) recent report.

Design of Portable Concrete Barriers for Minimal Deflection

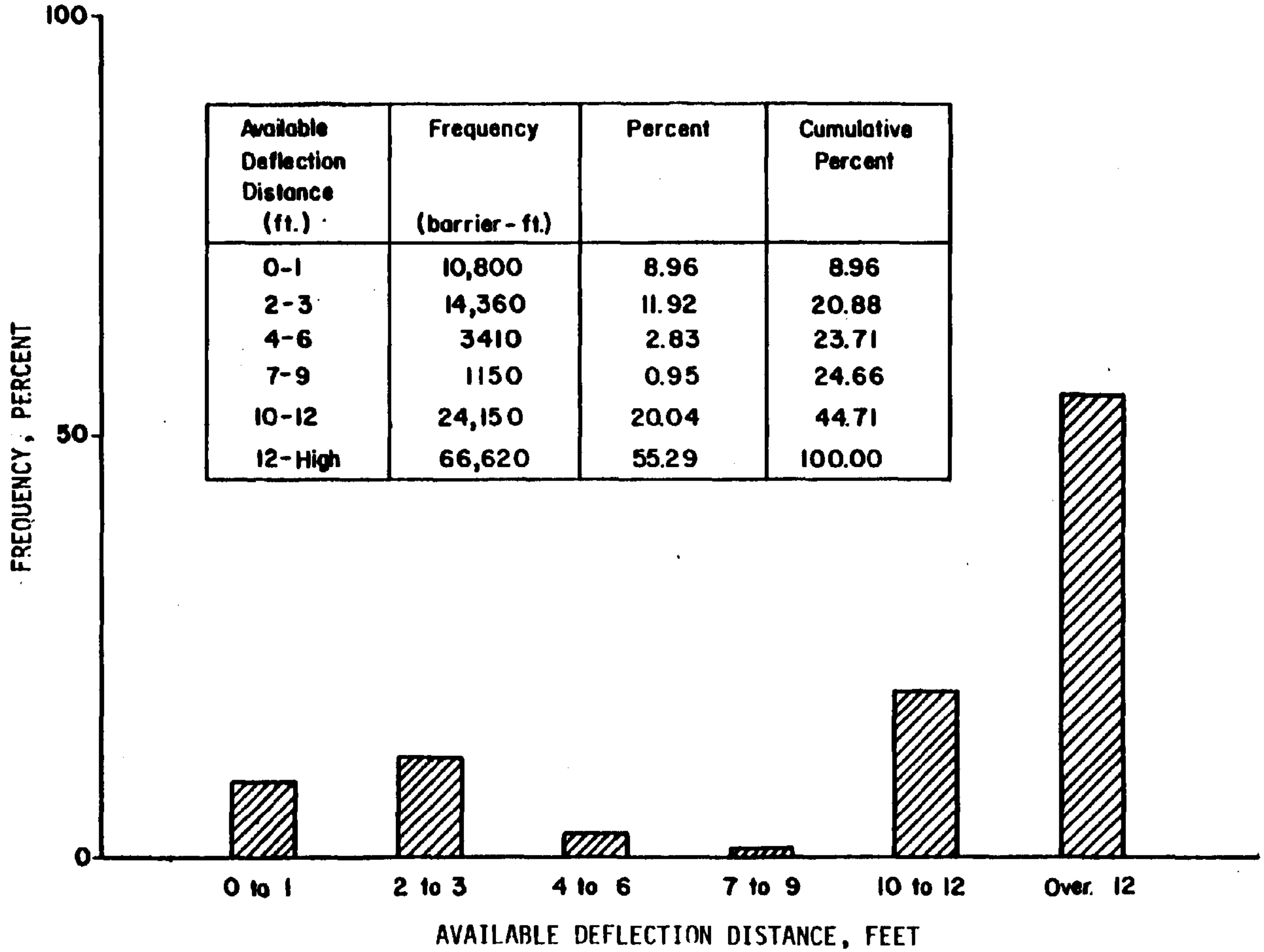
It has been shown by testing in this study that barrier deflections of free standing PCB's can be limited to about 1.5 ft (0.5 m) under the 4,500 lb/60 mph/25 deg (2,041 kg/96.5 km/h/25 deg) test condition, even for the condition of joint moment capacity as low as 50 kip-ft (68 kN-m). Increasing joint moment capacity to 150 kip-ft (203 kN-m) does not reduce this deflection significantly, since it is more a function of taking up the slack in several joints than of yielding the joint connection. Further, it does not seem practical to reduce connection slack to less than one degree since meeting construction tolerances would prove very costly and special connection construction would be necessary to provide a curved installation. A one degree slack corresponds to a movement of about 1/4 in. (0.6 cm) on each side of the PCB base before resistance of the connection is encountered. Actually, even this may be overly optimistic for construction tolerances. One state is using a connection design having a slack value of up to 18 degrees, but a more representative range of values, found on the vertical pin-type connection is 8 to 10 degrees. The result of these considerations is the conclusion that to reduce barrier deflection to a very small amount, only a few inches, it is necessary to provide a fairly rigid shear connection to the ground surface. It should be understood that reducing barrier transverse deflection by a shear attachment at the ground surface may put more pressure on a segment to rock and more pressure on a segment connection to resist torsional movements.

In some testing programs rotation of some barrier segments about their longitudinal axis has been judged to be a factor in producing a vehicle roll. This is probably true and was not objectively assessed in this work. Where dynamic deflections during tests are reported they are estimated from an overhead camera. This would be movement of the barrier top which included both lateral movement of the barrier base and rocking movement of the top with respect to the base. The difference in the dynamic deflection observed and the permanent deflection is the best indicator of how much a barrier segment rocked during a test. It should be noted however, that estimates of dynamic deflection made from these overhead views are subject to significant error. Therefore the dynamic deflection values given are somewhat questionable. If they are accepted as estimates only it would appear barrier segment rocking was nearly 10 degrees in Test 1 (3825-7), 4 degrees in Test 2 (3825-6) and zero in Test 3 (3825-5). This variation does not appear consistent with the test conditions: (1) That the kinetic energy and pre-impact trajectory of the vehicle was about the same in all tests, (2) Cracking of the connections indicated that all barriers were subject to some degree of differential rotation (or rocking), and (3) That the connection strength actually decreased with each Test 1, 2 and 3. The result of these considerations is that no conclusions should be drawn from these data regarding the influence of barrier rocking on vehicle reaction.

Figure 12, the influence of "Equivalent Static Friction" on the lateral deflection, indicates that increasing the value of equivalent static friction to eight would decrease the barrier deflection to about 7 in. (17.8 cm). This is equivalent to developing a lateral force of 36 kips (160 kN) during initial movement for a 12 ft (3.7 m) barrier segment length. This also does not seem to be a good approach since the engineering problem of developing that much force over the significant distance is considerable. It seems a more practical approach is to reduce barrier deflection to fundamentally zero by providing a ground connection that will resist the full impact load and thus not allow any movement. This requires the PCB to function like a permanent concrete barrier, and can be accomplished by providing a shear connection to the ground, consistent with the intensity of vehicle impact the designer chooses to accommodate.

Table 6 gives values of impact loads for different vehicle sizes and impact conditions taken from the work of Buth, et al (10) and Bronstad, et al (11). If the most intense automobile test is selected, a force of approximately 60 kips (267 kN) must be resisted. For a school bus, 20,000 lb (9,072 kg) vehicle, this should be increased to about 75 kips (334 kN) to accommodate the second impact, i.e., the "slap" of the rear of the bus against the wall. Force data during only the initial portions of the collisions are reported for tests CMB14 and CMB15 (11). Also, the vehicle rode up on the sloped face concrete median barrier. This vertical movement did not occur on the vertical faced concrete wall. Force values for the two CMB tests are the maximum values observed (i.e. not averaged over a time interval).

This is not to say that rigid ground connections should be provided on temporary barriers as a matter of course. The need for very small deflections (See Figure 18) should be a fairly rare occurrence, possibly only when PCB's are placed close to a large vertical drop or when it is projected that they may be subject to combined high speed-high angle impacts, and certainly only when the PCB is expected to remain for a fairly lengthly period of time, not likely to be less than a month. California has taken a less extreme approach to this need to limit deflection by requiring their Type K rail, 20 ft (6.1 m) pin connected PCB segments to be anchored by two one in. (2.5 cm) diameter steel rods driven three ft (0.9 m) into the soil or base at each segment end. Although experimental tests have not been conducted, field experience with this barrier is good, even though the variety of road and soil conditions under which it is used allows the actual rigidity of the shear support to vary widely. Although it might at first seem that moment capacity is not needed if the barrier segments are not allowed to rotate, structural capacity to resist torsion is still necessary to prevent one segment from being knocked over under the imposed overturning moment produced by the impact force and the base resistance. An adequate torque capacity is produced by most of the connections in use if they provide good moment capacity. Even those barriers which provide only shear capacity such as the vertical tongue and groove type joints provide some torsion capacity. Although the vertical tongue and groove seems to have a torsion capacity of only about 7 kip-ft (9.5 kN-m) statically, it may be adequate in that a 12 ft (3.7 m) barrier segment has a static resistance to overturning of about 5 kip-ft (6.8 kN-m), and the inertia of the barrier in concert with the relatively short duration of force works to reduce the needed torsion transfer capacity to prevent overturning.



Available Deflection Distance (ft.)	Frequency (barrier - ft.)	Percent	Cumulative Percent
0-1	10,800	8.96	8.96
2-3	14,360	11.92	20.88
4-6	3410	2.83	23.71
7-9	1150	0.95	24.66
10-12	24,150	20.04	44.71
12-High	66,620	55.29	100.00

Figure 17. Frequency of Available Deflection Distance. (Percent of 120,490 barrier-ft)

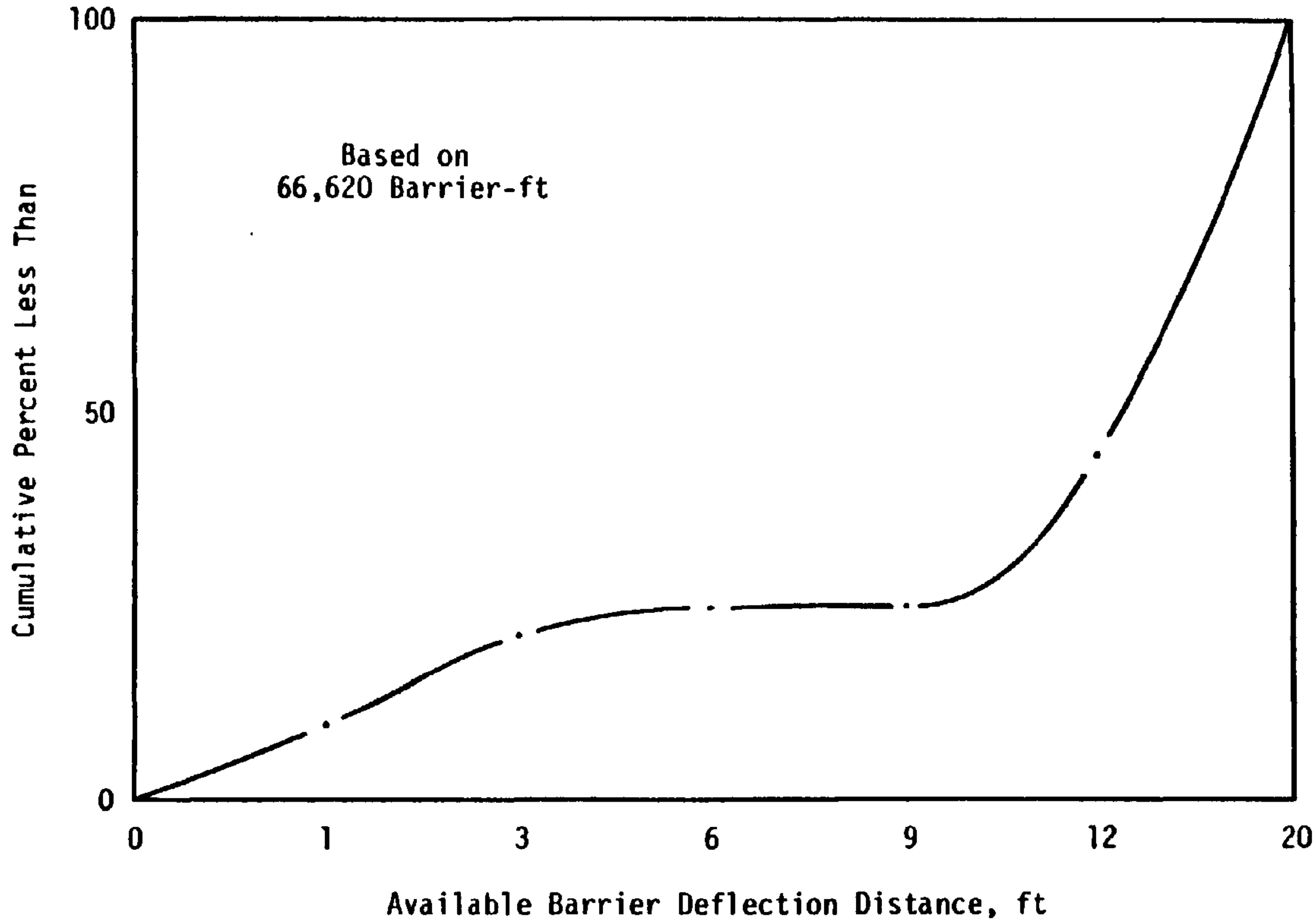


Figure 18 . Percentage of Barrier-ft Observed Having Less Than the Specified Available Deflection Distance.

Table 5. Service Levels of Barrier Adjacent to Roadway Drop Offs.

Site No.	Depth of Cut ft	Avail Defl. ft	Service Level				
			D*	A	1	2A	2B
3	10	2			x	x	
100	10	0		x			
33	1	8		x			
36	-		x				
29	-	12					
28	-	12	x				
34	1.5	3	x				
23 a	-	2	x				
b	-	12	x				
22	-	5		x			
18	-	6			x	x	
9	6	6		x			

*Performance as a "Delineator" only

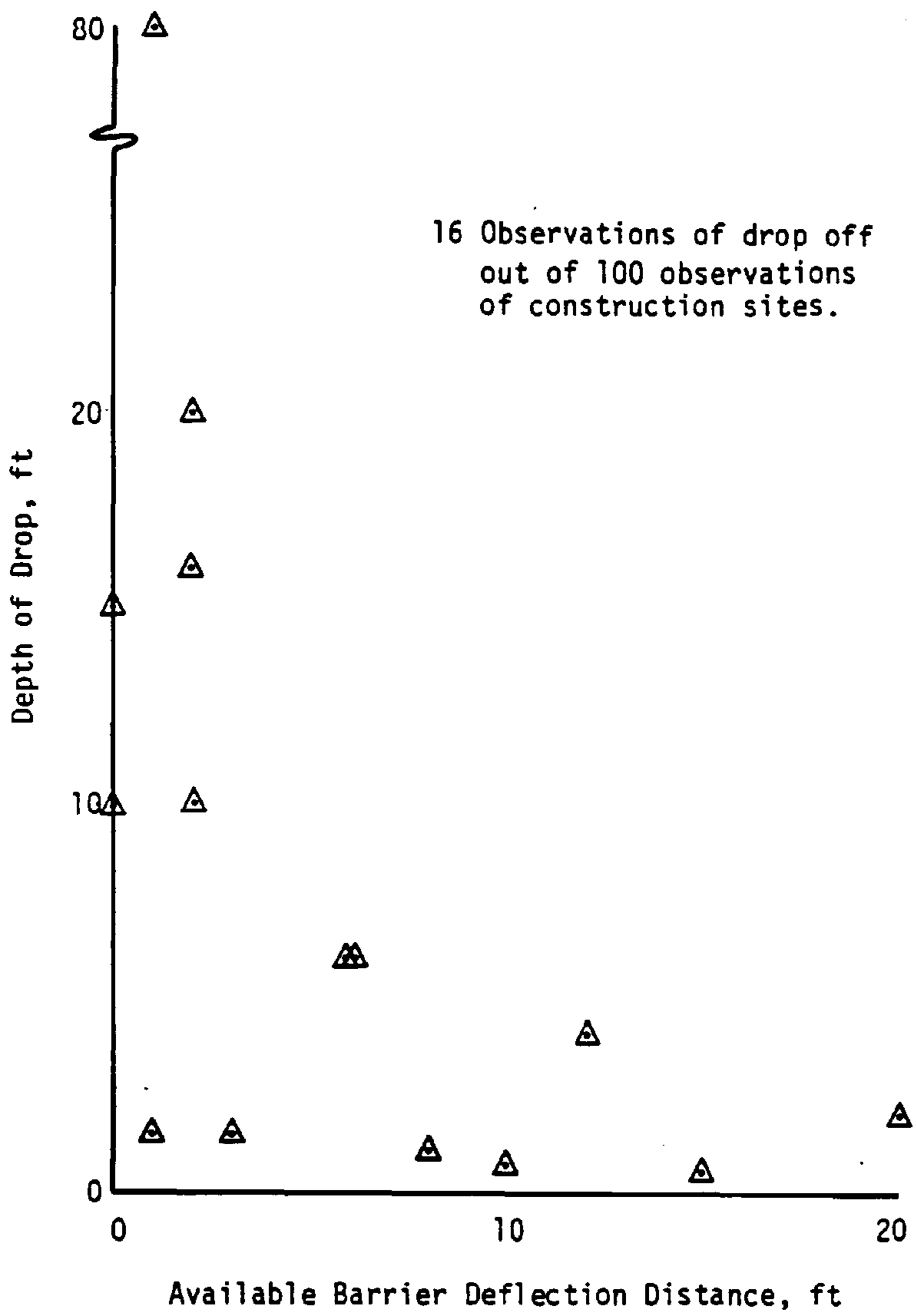


Figure 19 . Depth of Drop vs. Available Deflection Distance.

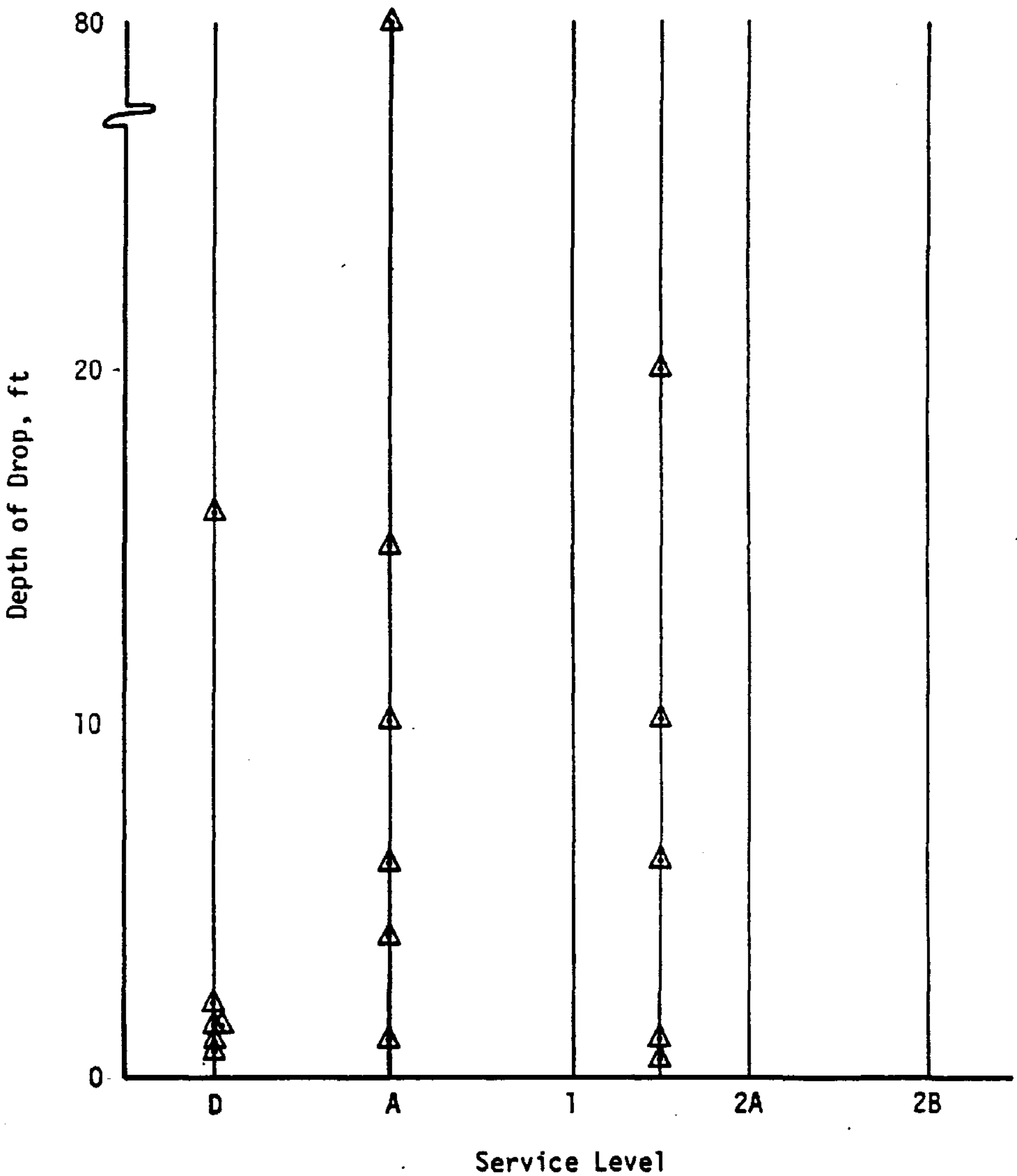


Figure 20 . Depth of Drop vs. Service Level.

PCB's can be designed to minimize deflection under severe collision loads, although it will be detrimental to portability and significantly increase costs. There may be some comparatively rare cases where it should be done.

Several techniques for minimizing barrier deflection may be employed and selected ones were investigated in this study. They included:

1. Angle segments (steel) connected to concrete pavement with driven studs (Figure 21)
2. Angle segments (steel) connected to concrete pavement with anchor bolts installed in drilled holes (Figure 21)
3. Triangular plates driven in soil (Figure 22)

Impact tests on segments of CMB were performed using a 5,000 lb (2,270 kg) bogie impacting at 90 deg to the barrier centerline. The test conditions were selected to produce lateral force values that would be similar to those produced by a 4500 lb (2,043 kg) vehicle at 60 mph (97 km/h) and 25 deg. Bogie impact speeds slightly greater than 20 mph (32 km/h) were achieved.

Bogie impact conditions were selected for the ground plane shear device tests in an effort to reduce costs, while producing lateral force conditions similar to those caused by an automobile impact. The bogie speed was selected to give about the same total kinetic energy as that portion of kinetic energy associated with the lateral component of velocity in a 4500 lb/60 mph/25 deg (2,043 kg/97 km/h/25 deg) automobile test. In comparing the actual force produced by these bogie conditions with forces produced by a vehicle on a rigid wall, it is seen that bogie forces of over 50 kips (222 kN) were produced in most tests, (See Table 7), while the maximum force acting on a vertical rigid wall is about 60 kips (267 kN) (See Table 6). It should also be recognized that the development of force in an automobile test occurs more slowly as crush of one frontal corner of the vehicle occurs. The force is also moving along the barrier longitudinally as it increases and approaches its maximum value. In the bogie situation the force develops more rapidly at a selected impact point. The impact point was selected to produce the most load on the shear connectors at one end. Although it is not alleged that the bogie test accurately simulates an automobile test, it was designed to achieve a test load of approximately the same magnitude.

Tongue and groove barrier segments, 12 ft (3.7 m) in length, were

Table 6. Nominal Forces Produced by Vehicles on a Vertical Rigid Wall.
[After Buth, et al. (10)]

Vehicle Type and Weight lb	Speed of Impact mph	Angle of Impact deg	Initial** Force kips	Final*** Force kips
Automobile 4,500	60	15	55	30
Automobile 4,500	60	25	60	30
School Bus 20,000	60	15	65	75
Intercity Bus 32,000	60	15	85	210
Automobile* 4,500	60 Configuration F - Test CMB 14	25	47.6	-
Automobile* 4,500	60 N.J. Safety Shape - Test CMB 15	25	39.9	-

*Test reported by Bronstad, et al. (11).
**Force when frontal corner of vehicle strikes barrier.
***Force when rear of vehicle slaps against barrier.

used in this experiment. The tongue and groove was selected so that development of connection movement would not influence the test. In this way only the shear connectors were tested rather than a combination of shear connectors and barrier mass. If moment and larger torsion capacities had been present less barrier movement would have been achieved.

The bogie, shown in Figure 23 was accelerated to impact speed with a pickup truck. An accelerometer, mounted on the rear of the bogie, was used to obtain data for the impact force-time history.

Results of these tests are presented in Table 7 and photographs of the installations are shown in Figures 24 through 29.

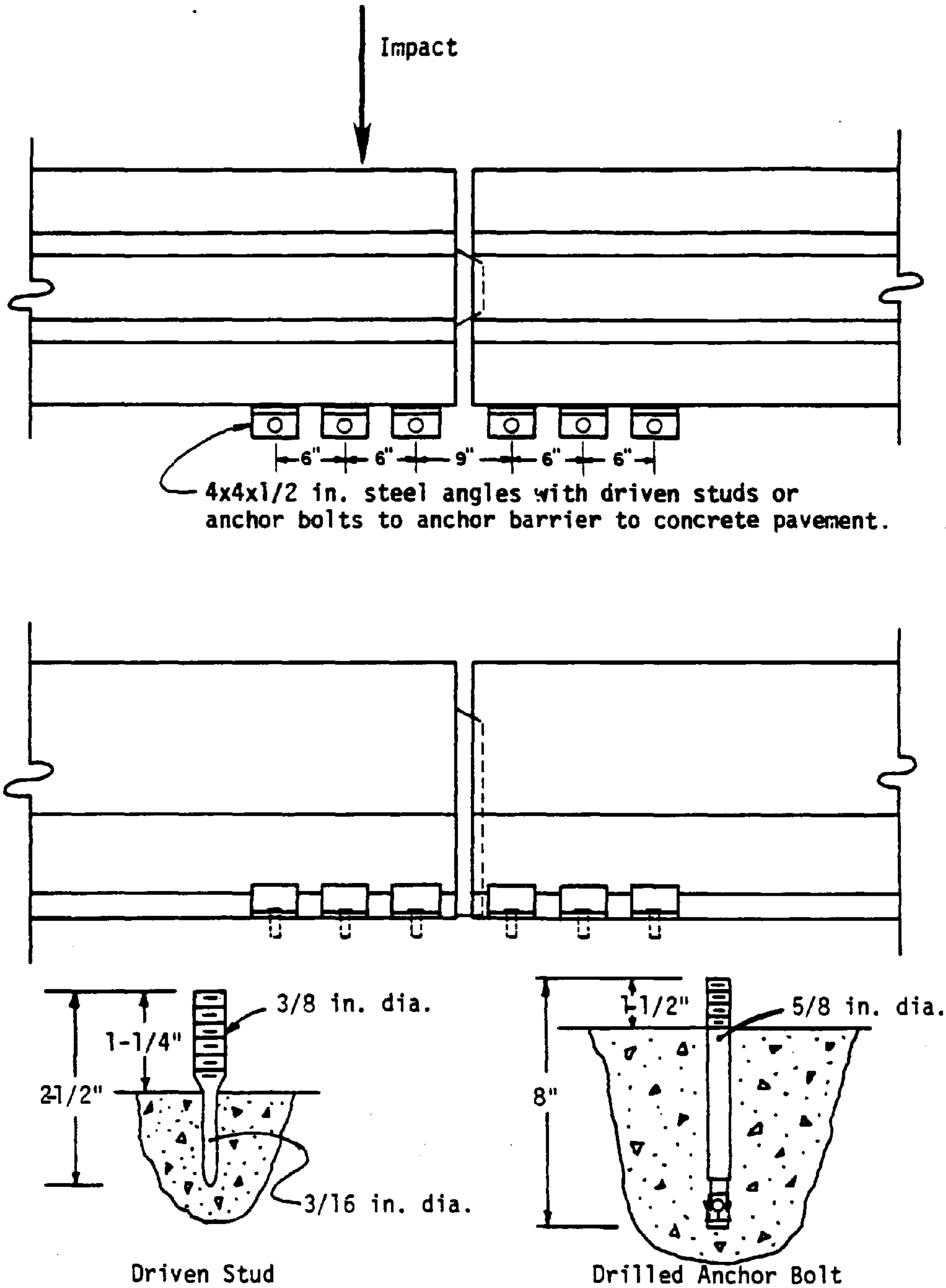


Figure 21. Barrier Anchorage Details for Concrete Pavement.

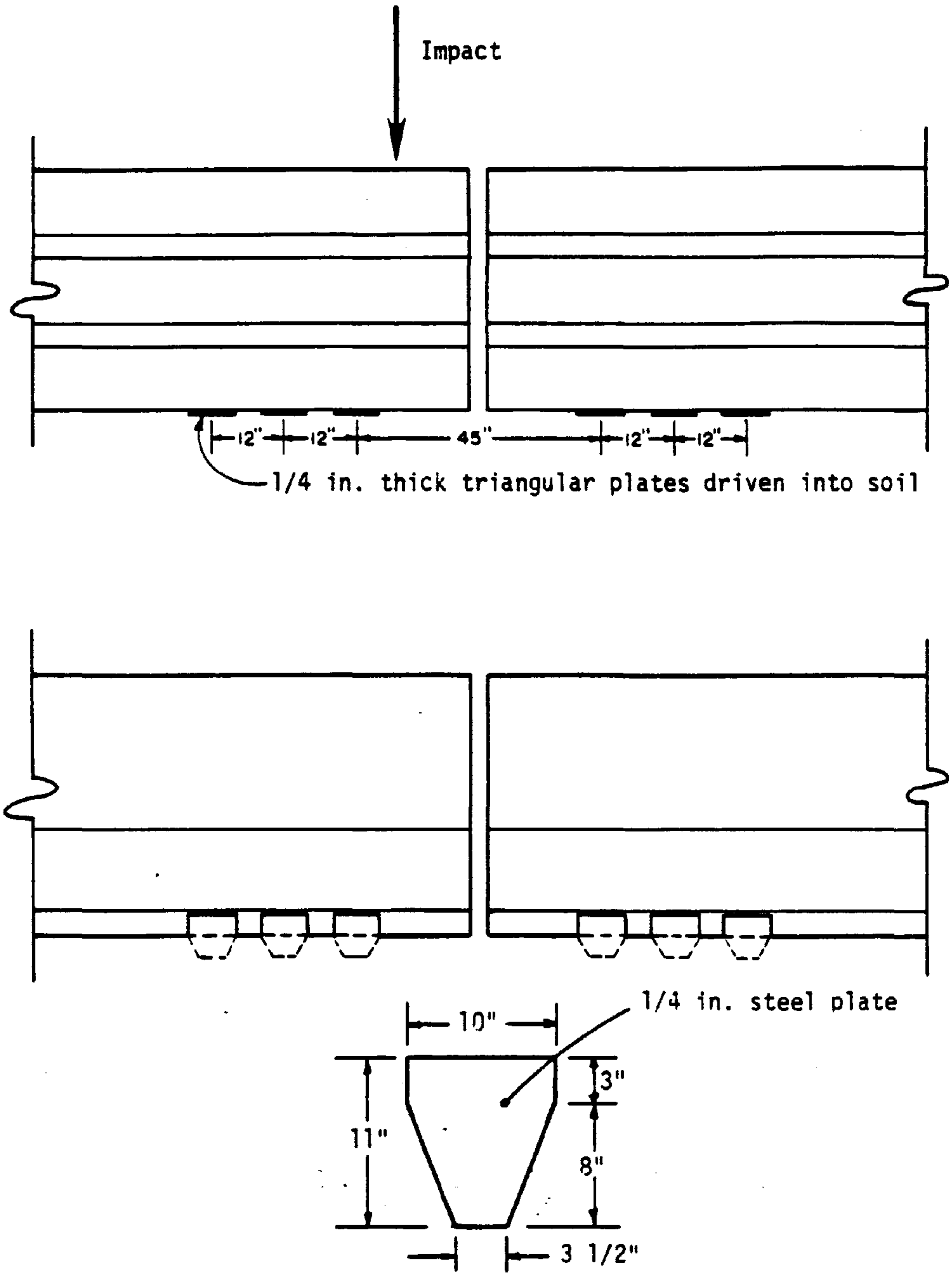


Figure 22. Barrier Anchorage Details for Soil Foundation.

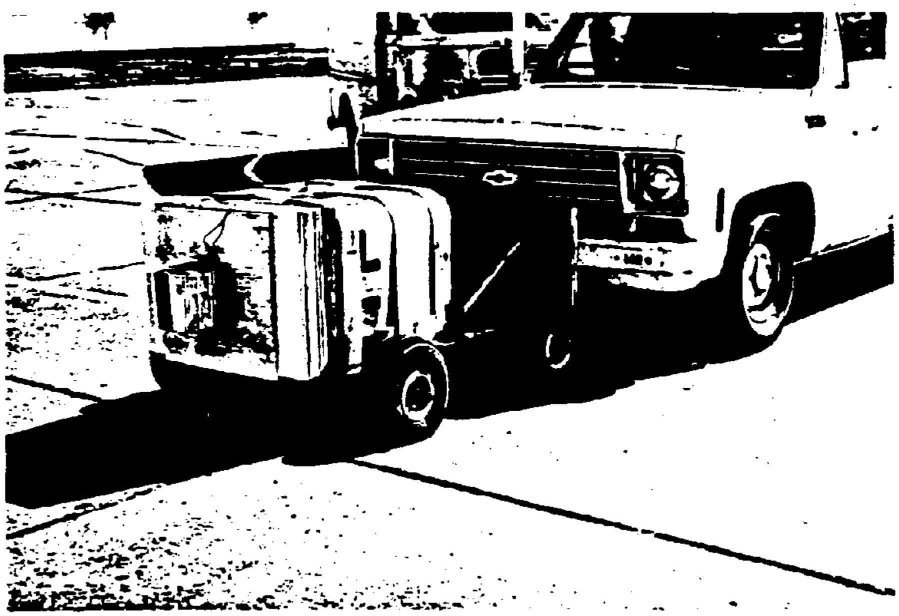


Figure 23. 5,000 lb Test Bogie.

Table 7. Results of Impact Tests on Barrier Anchorage.

Test Designation	Anchorage Design	Impact Location	Peak Impact Force (kips)	Impact Duration (sec)	Displacement of Barrier (in.)	Comments
3825-C3	Angles with Driven Studs	3 ft right of joint at 21 in. height	41.3	0.050	nil	Barrier segments rotated about lower edge during impact but returned to upright.
3825-C4	Angles with Driven Studs	3 ft right of joint at 21 in. height	48.8	0.050	18	Studs failed.
3825-C5	Angles with Drilled Anchor bolts	1 ft right of joint at 21 in. height	52.4	0.060	0	Barrier segments rotated about lower edge during impact but returned to upright.
3825-C6	Angles with Drilled Anchor bolt	1 ft right of joint at 21 in. height	53.6	0.047	36	Anchor bolt failed.
3825-C7	Angles with Drilled Anchor bolts	Centered on joint at 21 in. height	57.9	0.046	72	Anchor bolts failed.
3825-C8	Triangular Plates	2 ft right of joint at 21 in. height	69.5	0.060	0	Impacted barrier segment rotated about lower edge and came to rest on its side.

Figure 24. Barrier After Test 3825-C3.

Figure 25. Barrier After Test 3825-C4.

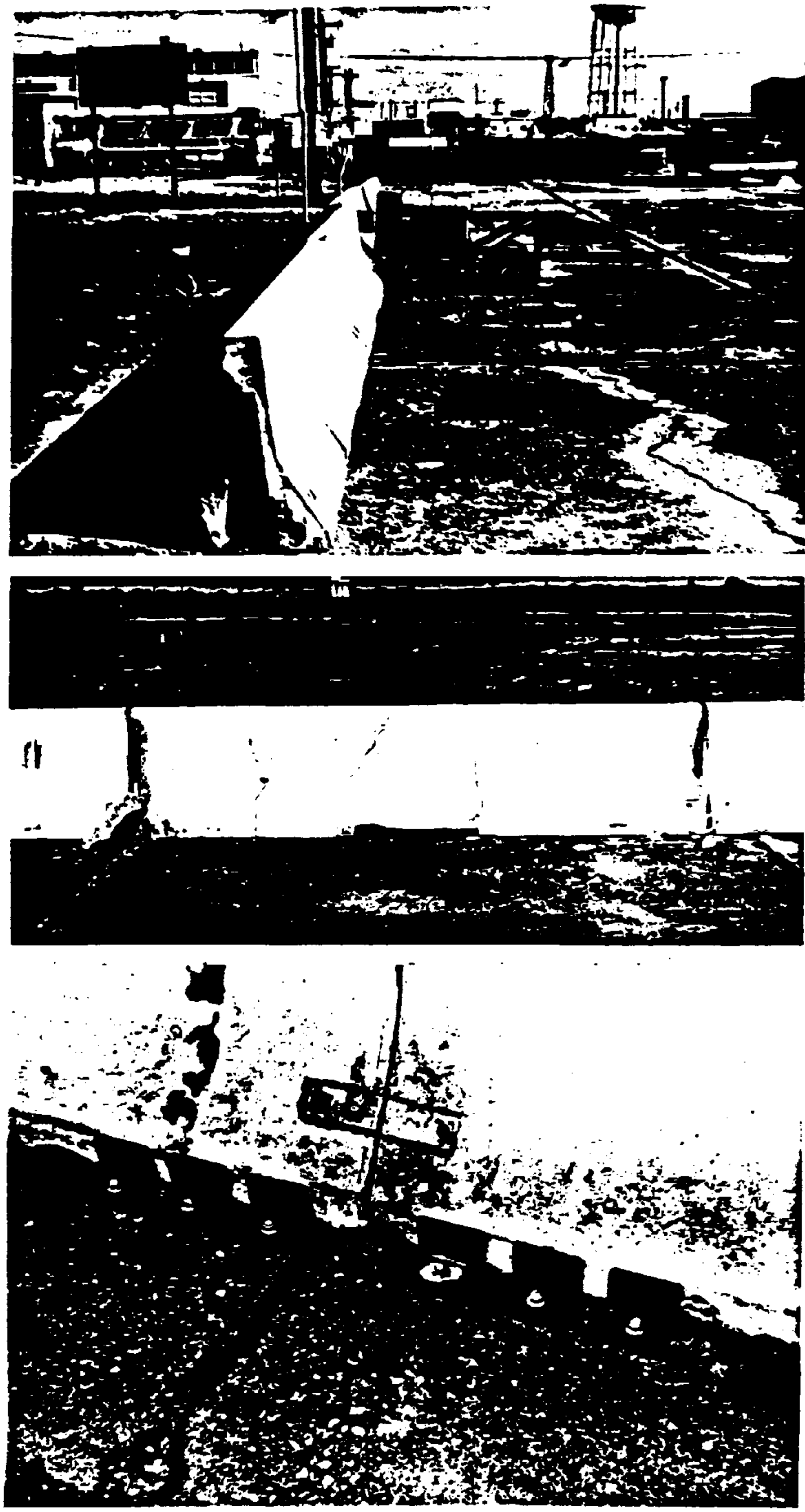


Figure 26. Barrier After Test 3825-C5.

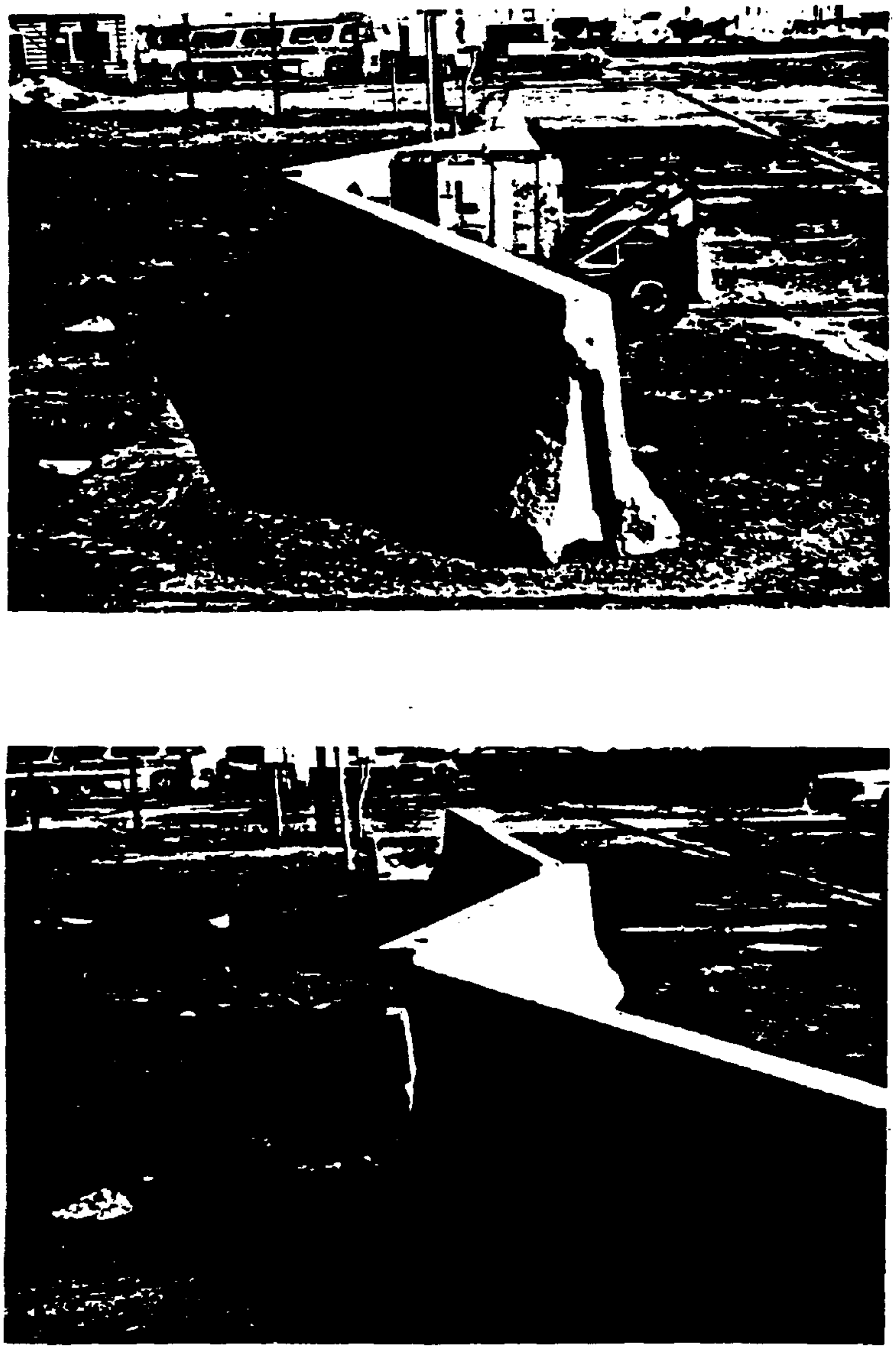


Figure 27. Barrier After Test 3825-C6.

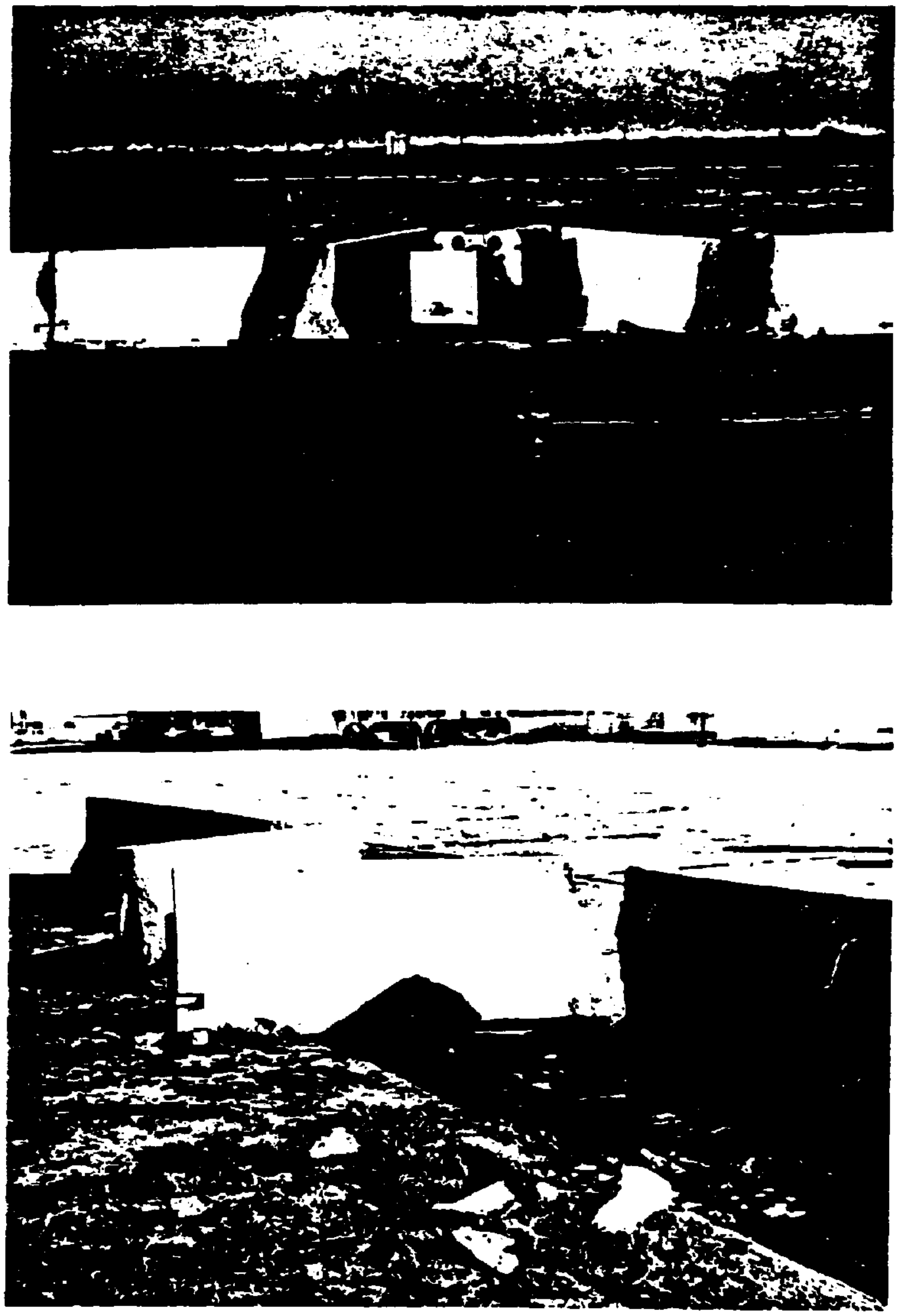


Figure 28. Barrier After Test 3825-C7.

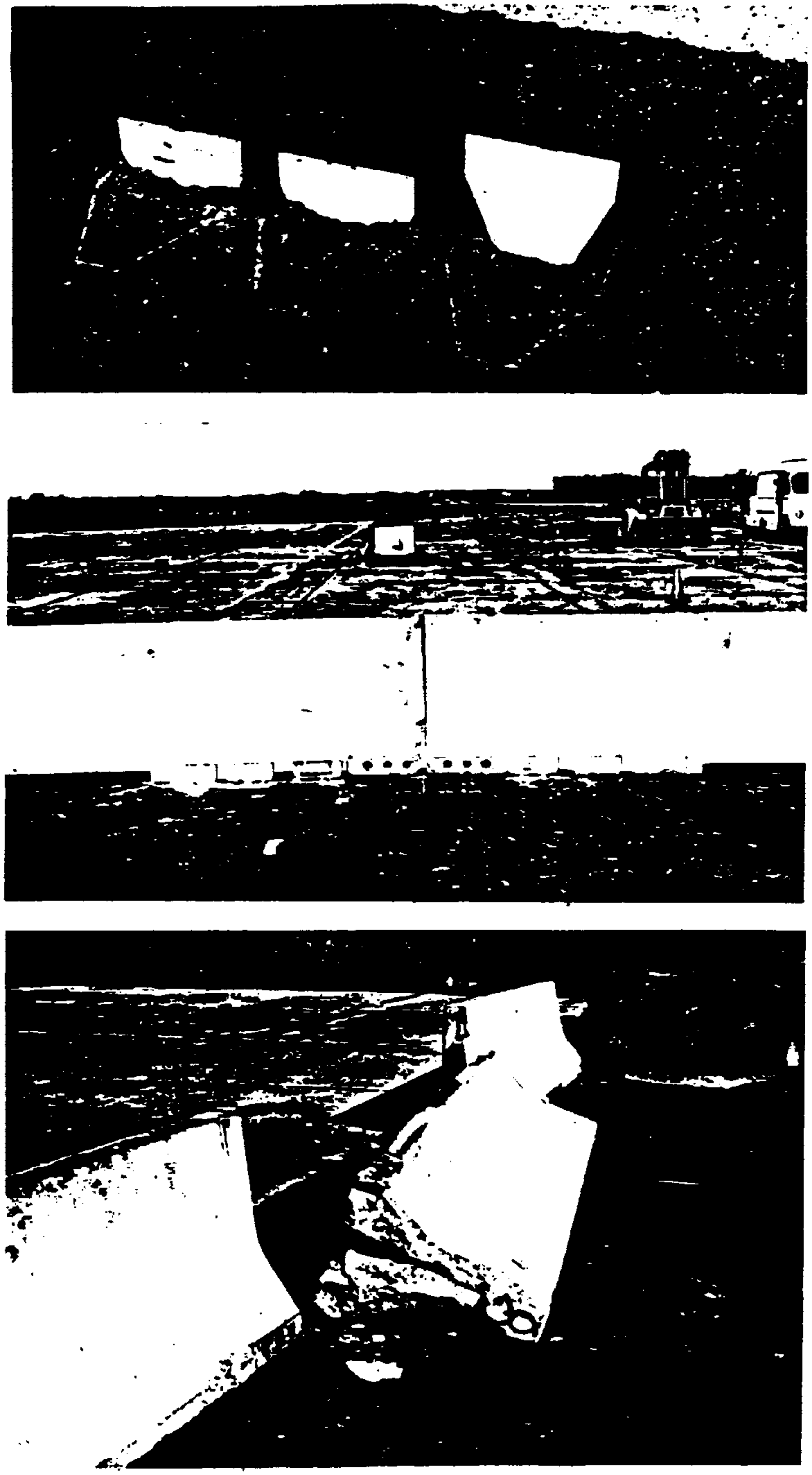


Figure 29. Barrier Before and After Test 3825-C8.

III. CONSTRUCTION BARRIER RATING SYSTEM

In early work on this project, a need was foreseen to have a method of comparing one construction barrier design to another. An extremely subjective system was developed which pointed out the need for certain evaluation factors. These factors included (1) functional adequacy, (2) cost of installation, (3) portability, (4) installed cost, and (5) repair costs.

Based on this first, extremely rough attempt, a more comprehensive method was developed for use here. The general factors listed above can be divided into two groups, functional adequacy (the "Service Level" of the barrier including deflection characteristics) and cost factors (initial cost, installation costs, maintenance costs, disassembly costs and transportation costs). These two groups of factors will be treated separately in the following sections.

Functional Adequacy

In determining the functional adequacy of a given barrier, reference is made to the work of Bronstad (17) and Michie (18) on bridge rail "Service Level". Rather than complicate the situation by choosing different service levels for construction zone application, the writers, to the degree possible, made use of the same service levels defined in NCHRP 230 (18). The levels defined are shown in Table 8.

Evaluation of Portable Concrete Barrier (PCB) crash tests has demonstrated the need for the structural properties of shear, moment and torsion resistance in the connections of these barrier segments. Using simplified structural analysis techniques Beason (Appendix C) has analyzed 15 barrier connections. Using the same analytical procedures, the capacities of the connections used in all available crash tests of PCB's were calculated and are tabulated in Table 9. Table 10 gives necessary details on test conditions and results. The tests so tabulated were conducted in California, Texas and New York over the past ten years. The values shown in Table 8 are not always the same as those calculated by Beason and given in Appendix C. Beason used nominal values of material strength levels. Connection capacities presented in Table 9 were obtained

Table 8. Portable Construction Barrier Service
Levels Compared to NCHRP 230 Service Levels.

Collision Characteristics kips-mph-degrees	Impact Severity $\frac{1}{2}\frac{W}{g}(V\sin\Theta)^2$ kip-ft	Construction Barrier Service Level		NCHRP 230 Service Level
4.5-45-15 (or 3.5-60-15)	20.4	A	←*→	-
4.5-60-15	36.5	1	←*→	1
4.5-60-25	97.3	2A	←*→	2
20-60-15	161.1	2B	←*→	-
40-60-15	322.2	3	←*→	3

*Corresponds to

Table 9. Summary of Portable Construction Connection Properties.

Test Numbers	Data Pt. No.	Connection Description	Connection Slack degrees	Connection Capacities Shear kips	Moment ft-kips	Torsion ft-kips
TX-1	1	Side plates (3'6" x 5" x ½", steel) See Figure 30.	5°	90	117	53
TX-2	2	Side channels (C5 x 9 x 3'6", steel) See Figure 30.	3°	90	117	53
3825-7	3	Vertical tongue and groove and side plates (3'0" x 4" x ½" steel) See Figure 31.	3°	76	103	67
3825-6	4	Vertical tongue and groove and side plates (3'0" x 4" x 3/8" steel) See Figure 31.	3°	57	77	52
3825-5	5	Vertical tongue and groove and side plates (3'0" x 4" x ¼" steel) See Figure 31.	3°	38	52	37
3825-9	6	Vertical tongue and groove and side plates (3'0" x 4" x 1/8" steel) See Figure 31.	3°	19	26	22
3825-8	7	Side channels (C5 x 9 x 3'6" steel) See Figure 32. plus 3 No. 8 x 18" steel rebar dowels.	3°	135	117	73
CMB-2	8	Three grouted dowels (No. 8 x 18") See Figure 33.	0°	60	50	37
291	9	Vertical steel pin (7/8" ø x 26") See Figure 34.	9°	46	31	35
292	10	Vertical steel pin (7/8" ø x 26") See Figure 34.	9°	46	31	35
293	11	Vertical steel pin (1" ø x 26") See Figure 34.	8°	55	40	42
294	12	Vertical steel pin (1" ø x 26") See Figure 34.	8°	55	40	42
CMB-18	13	Vertical tongue and groove and side plates (12" x 3" x ¼" steel) See Figure 35.	3°	27	9	16
CMB-24	14	Vertical tongue and groove and side plates (12" x 3" x ¼" steel) See Figure 35.	3°	27	9	16
NY-17	15	Vertical. I beam (3½ x 2") See Figure 36.	10°	208	61	87
NY-18	16	Vertical I beam (3½ x 2") See Figure 36.	0"	208	61	87
NY-17	17	Vertical I beam (3½ x 2") See Figure 36.	10°	208	61	87
NY-45	18	Vertical I beam (3½ x 2") See Figure 36.	10°	208	61	87
NY-46	19	Vertical I beam (3½ x 2") See Figure 36. (grouted joints)	0°	208	61	87
NY-47	20	Vertical I beam (3½ x 2") See Figure 36.	10°	208	61	87

Table 10. Summary of Portable Construction Barrier Tests.

Testing Agency	Test Number	Test Conditions Speed/Angle/Wgt./Energy* mph/deg/kips/kip-ft.	Segment Length (ft)	Static Deflection (ft)	Data Pt. No.	Test Results & Comments
TTI	TX-1	60.9/17.8/4.5/52.1	15	0.9	1	Smooth redirection, negligible barrier damage.
TTI	TX-2	55.9/26/4.51/90.5	15	1.3	2	Smooth redirection, negligible barrier damage.
TTI	3825-7	59.2/25/4.5/94.1	12	1.8	3	Smooth redirection, slight barrier damage.
TTI	3825-6	60.1/24/4.5/89.8	12	1.8	4	Vehicle redirected but rolled after recontact with pavement subsequent to primary collision, slight barrier damage.
TTI	3825-5	60.7/25/4.5/98.9	12	1.6	5	Smooth redirection, slight barrier damage.
TTI	3825-9	63.4/25/4.51/108.2	12	6.5	6	Smooth redirection, side plates failed, slight barrier damage.
TTI	3825-8	57.7/15/20.0/149.2	15	1.8	7	Bus redirected but rolled 90° onto side after collision, slight barrier damage.
TTI	CMB-2	60.0/24/4.54/90.3	30	1.1	8	Smooth redirection, negligible barrier damage.
CALTRANS	291	65/7/4.86/10.2	12.5	0.5	9	Smooth redirection, slight barrier damage.
CALTRANS	292	68/23/4.86/114.6	12.5	1.9	10	Vehicle redirected but penetrated over top of barrier and slid sideways along top. Segment fractured, major barrier damage.
CALTRANS	293	66/40/4.86/292.2	20	NA	11	Vehicle pentrated and rolled. Segment tipped over, major barrier damage.
CALTRANS	294	39/25/4.7/42.6	20	0.5	12	Smooth redirection. Steel vertical connection rods were severely bent. Significant barrier damage.
SWRI	CMB-18	62/25/4.5/103.2	20	NA	13	Vehicle redirected, flexural failure in the segments. Major barrier damage.
SWRI	CMB-24	56/24/4.5/77.8	20	3.4	14	Vehicle redirected, joint failures. Significant barrier damage.
New York	NY-17	53/25/4.25/71.2	20	1.3	15	Smooth redirection, slight barrier damage.
New York	NY-18	58/25/4.23/86.3	20	0.9	16	Vehicle redirected but rolled after recontact with pavement subsequent to primary collision, slight barrier damage.
New York	NY-44	65/25/4.3/108.4	8	1.4	17	Vehicle redirected but subsequently rolled, slight barrier damage.
New York	NY-45	66/15/2.18/21.2	8	0.3	18	Vehicle redirected but could have rolled, slight barrier damage.
New York	NY-46	61/25/4.35/96.6	8	0.6	19	Vehicle redirected, slight barrier damage.
New York	NY-47	61/15/2.18/18.1	20	0.3	20	Vehicle smoothly redirected, no significant barrier damage.

using the same computation methods but more specific material strength levels were used when available for a particular test.

These tables show the calculated structural capacities of the barrier connections in terms of shear, moment and torsion and the pertinent test conditions. The connections analyzed are detailed in Figures 30 through 36. It can be determined from these tests which barrier connections exhibited poor structural performance, and in which tests good performance was achieved.

Figures 37, 38 and 39 are plots of the connection capacities of shear, moment and torsion as a function of the impact severity. Impact severity is approximated by the kinetic energy associated with the lateral component of test vehicle velocity. This component of velocity is perpendicular to the longitudinal barrier axis. As seen, there is a boundary between good performance and poor performance for each strength property. Exactly where the boundary falls was not always accurately defined by these data points. This would be expected since the interactions of the three characteristics do not account for using this approach. It was necessary to conservatively select an appropriate boundary for use in setting connection capacities for different Service Levels.

In order to do this, three other areas of information were referred to. For the lowest service level, A, the characteristics of the Virginia tongue and groove connection was used as a control point. From Table 9, the values of shear (32 kip-ft) (43 kN-m), moment (0 kip-ft) (0 kN-m) and torsion (7 kip-ft) (9 kN-m) were assigned as the Service Level A control point on each graph. Although this may seem arbitrary, since this barrier has only been subjected to informal testing, the writers are satisfied this barrier will meet <u>at least</u> this Service Level based on the extremely good field performance level found by Lisle and Hargroves (<u>19</u>) and others. As a second control point, Buth (<u>10</u>) has observed the maximum force level to be expected for Level 2A and Level 2B tests. Plotting Buth's results on Figure 37 gives three points which are reasonably consistent with the boundary between good and poor performance as indicated by the 20 crash tests.

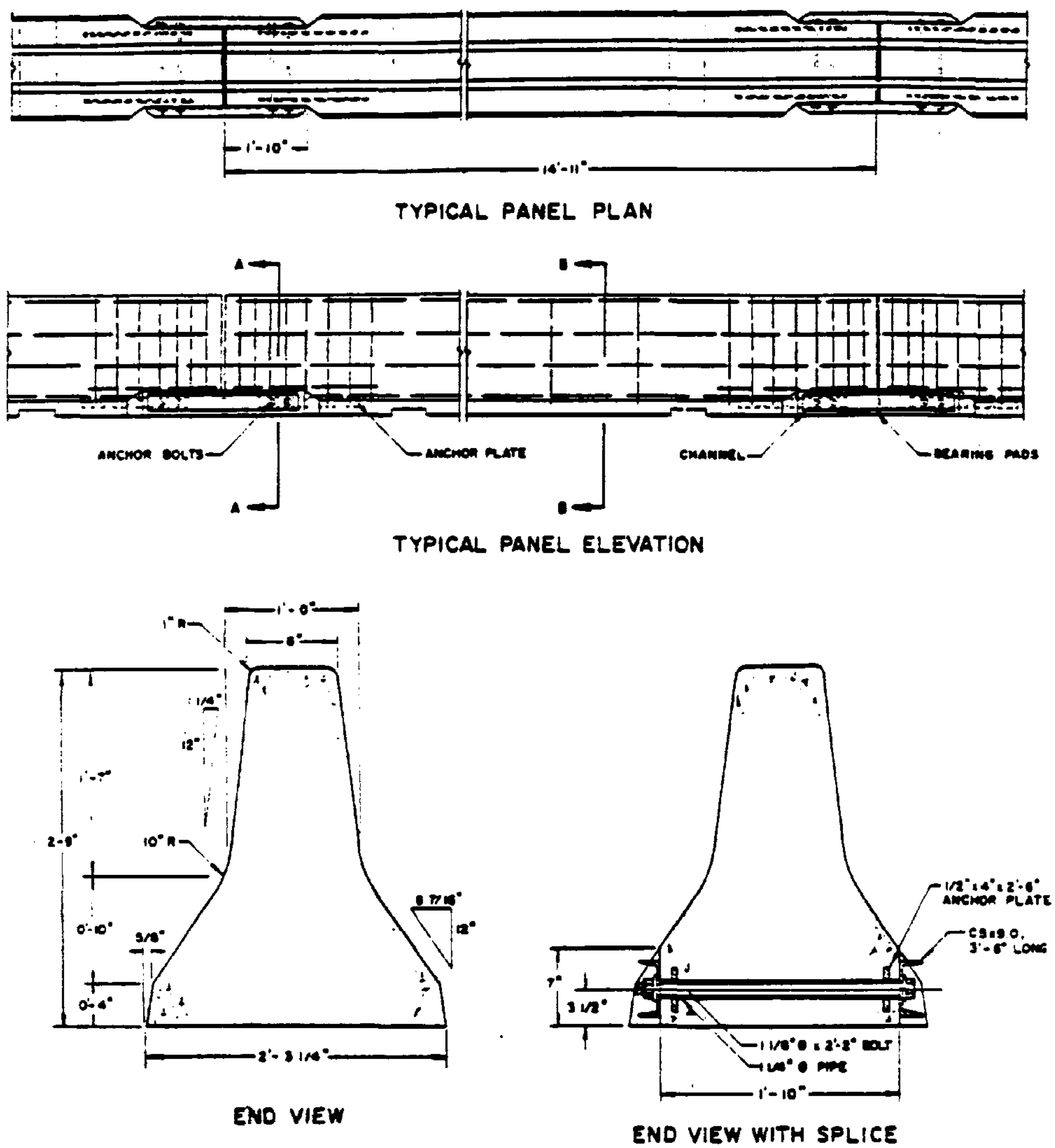


Figure 30. Side Plate/or Side Channels (Channel Splice).

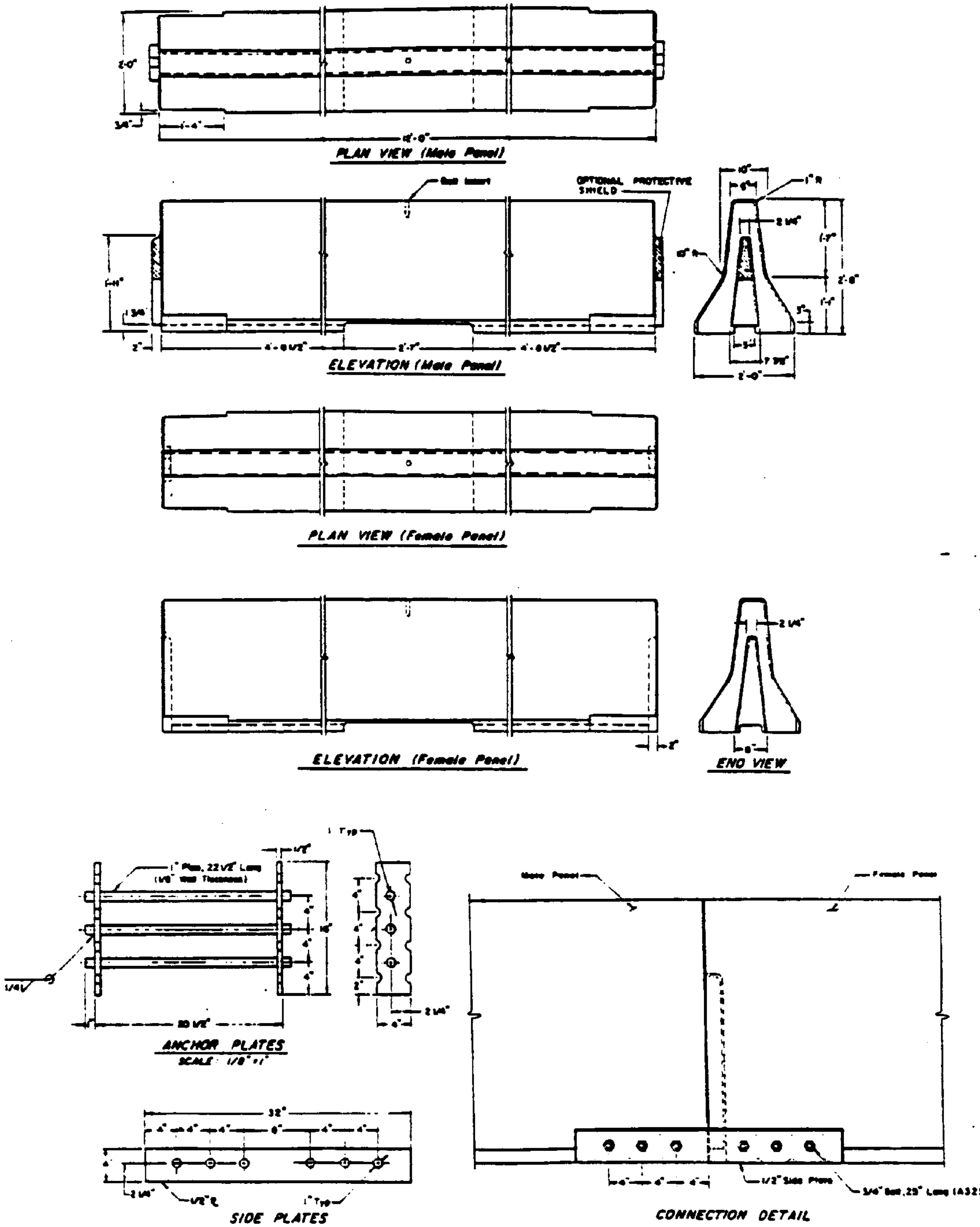


Figure 31. Vertical Tongue and Groove and Side Plates.

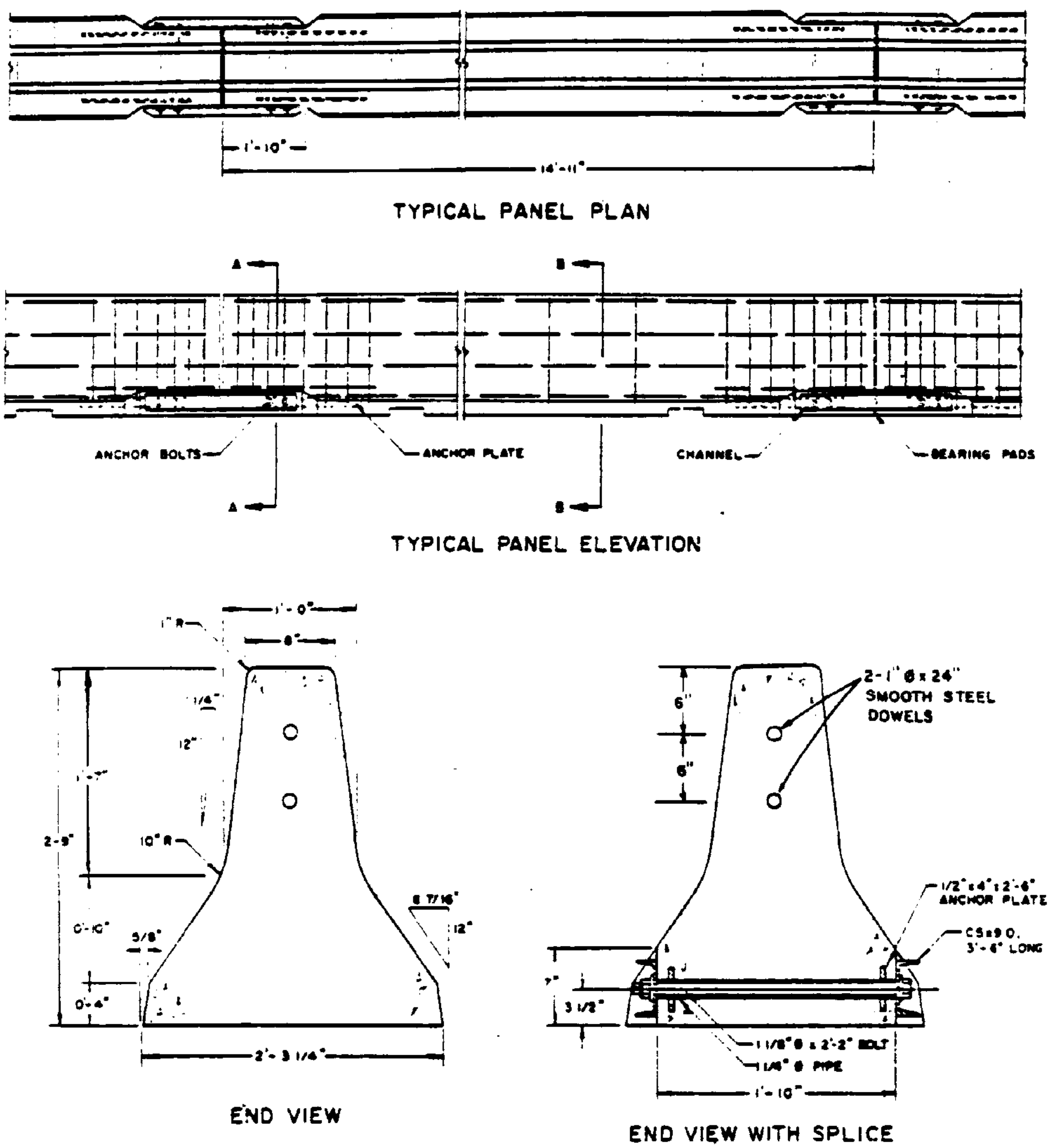


Figure 32. Side Plate/or Side Channels With Steel
Dowel Shear Connection (Channel Splice).

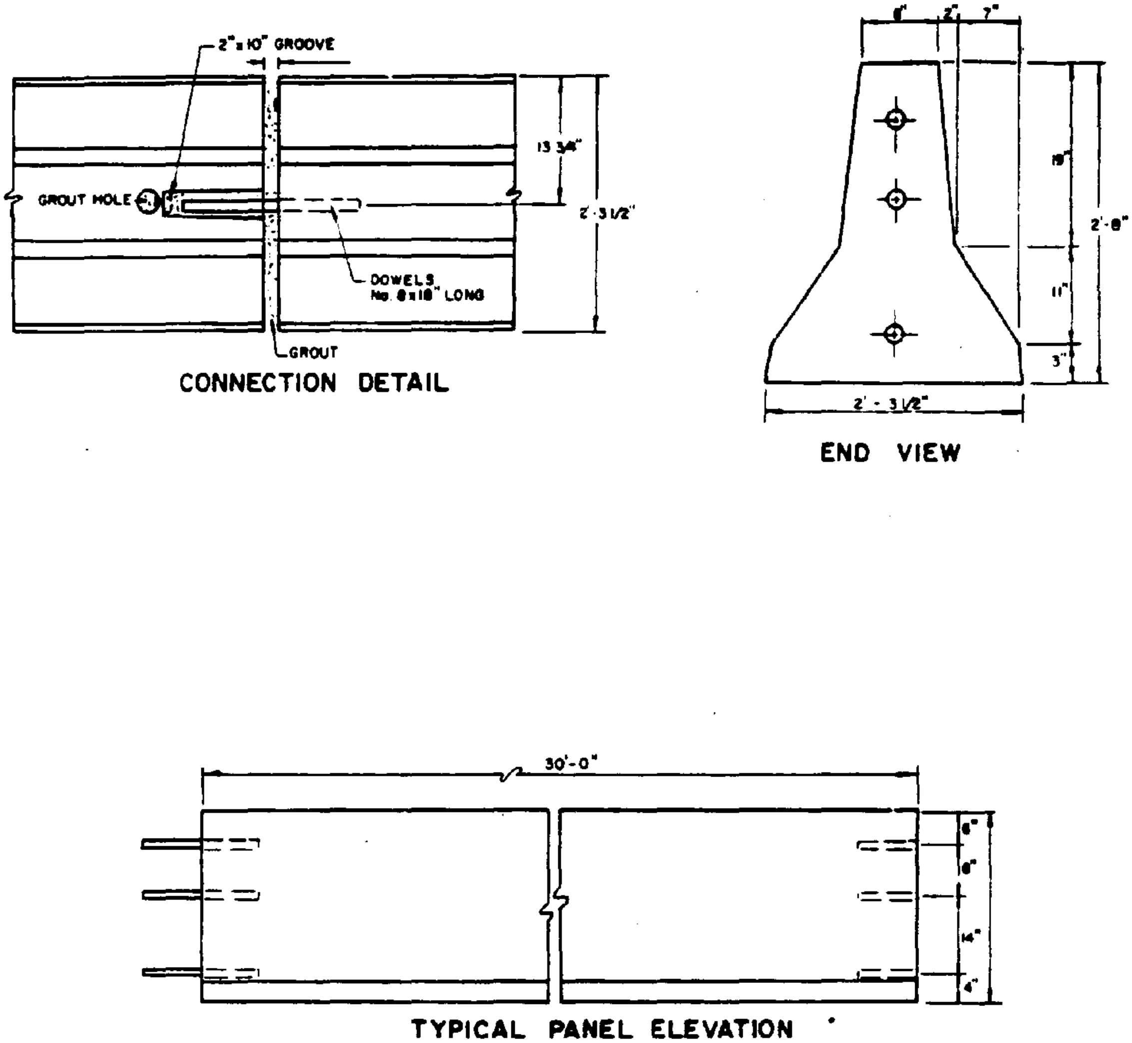


Figure 33. Grouted Dowels (Steel Dowel Joint).

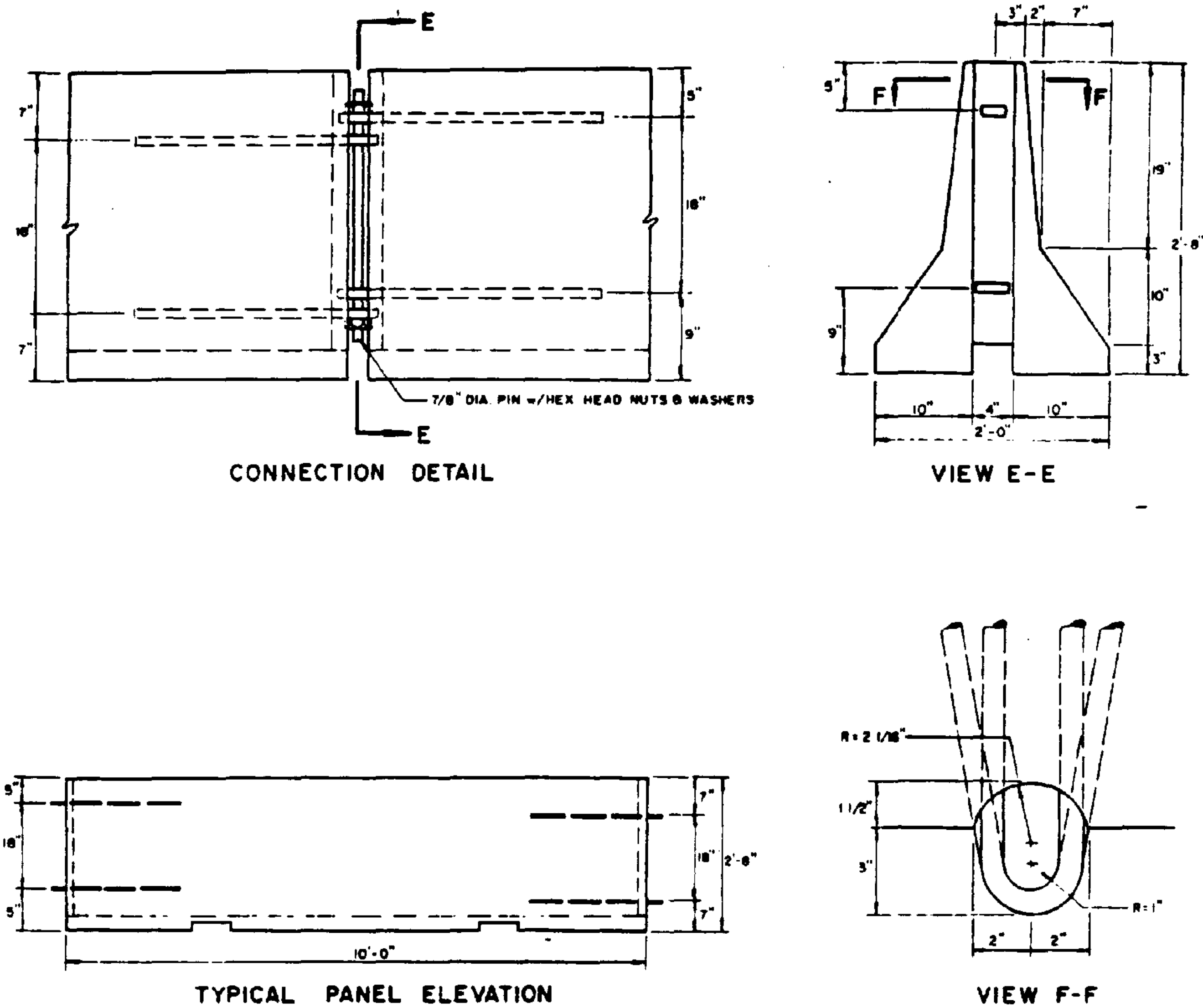


Figure 34. Vertical Steel Pin (Pin and Re-bar).

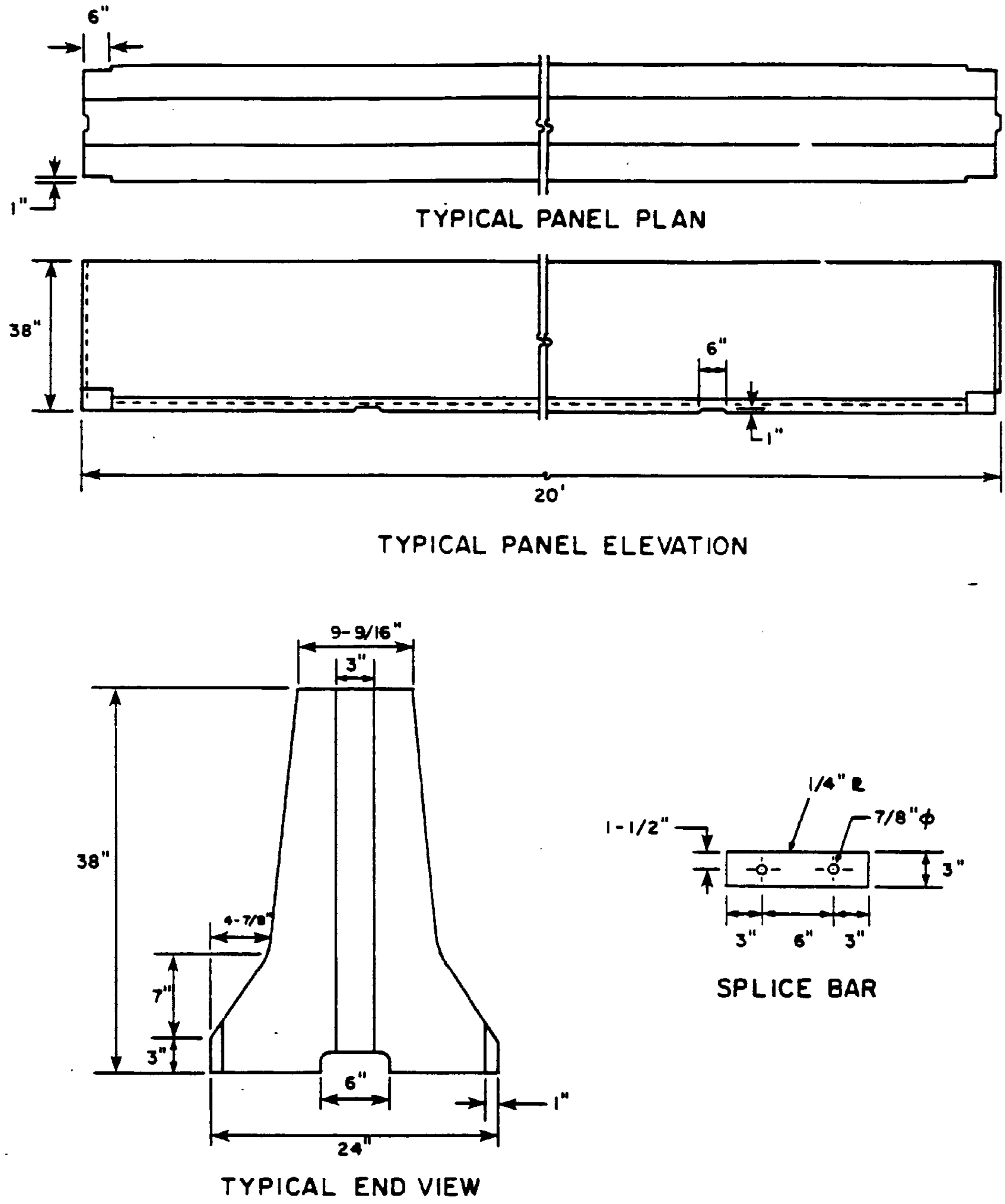


Figure 35. Vertical Tongue and Groove and Side Plates.

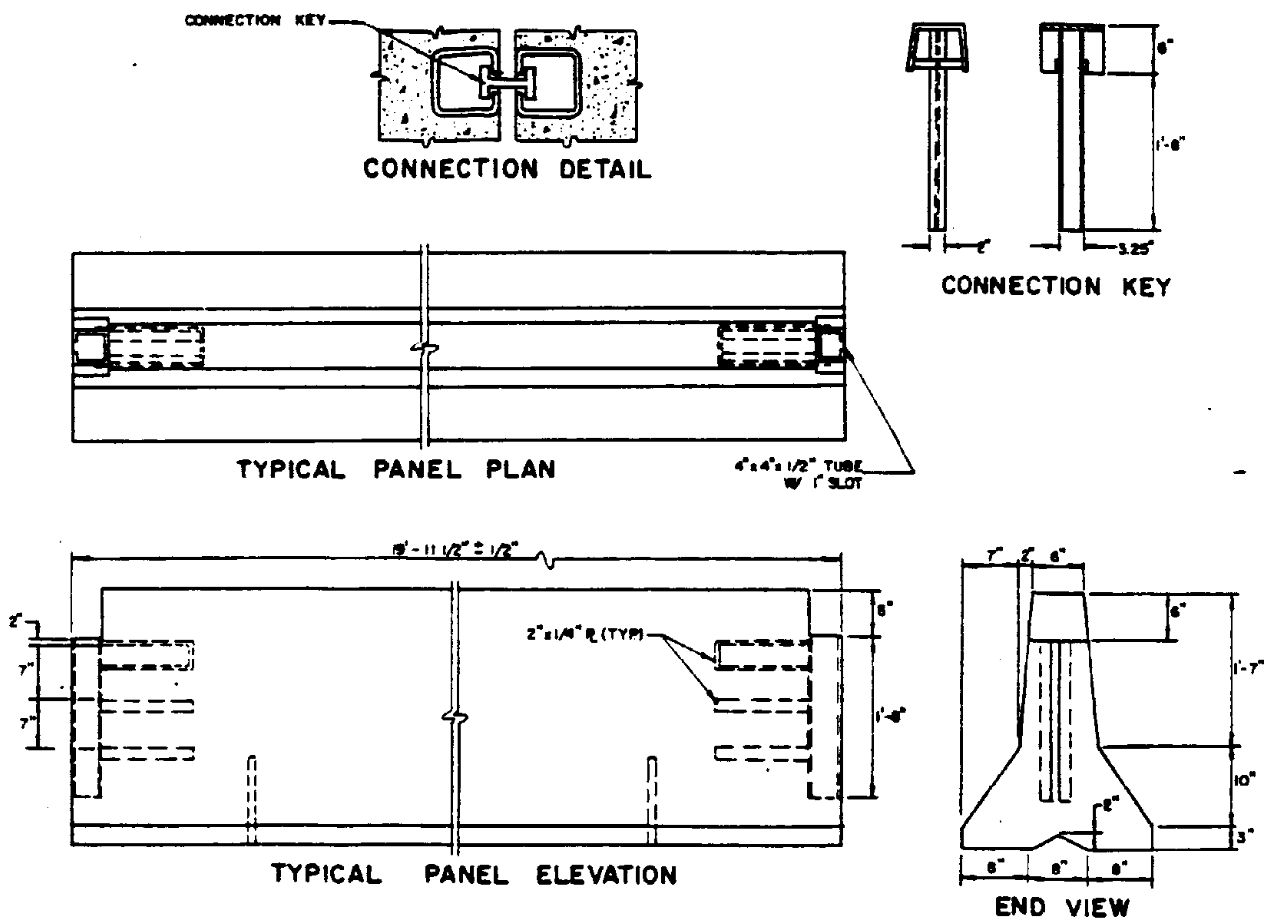


Figure 36. Vertical I-beam.

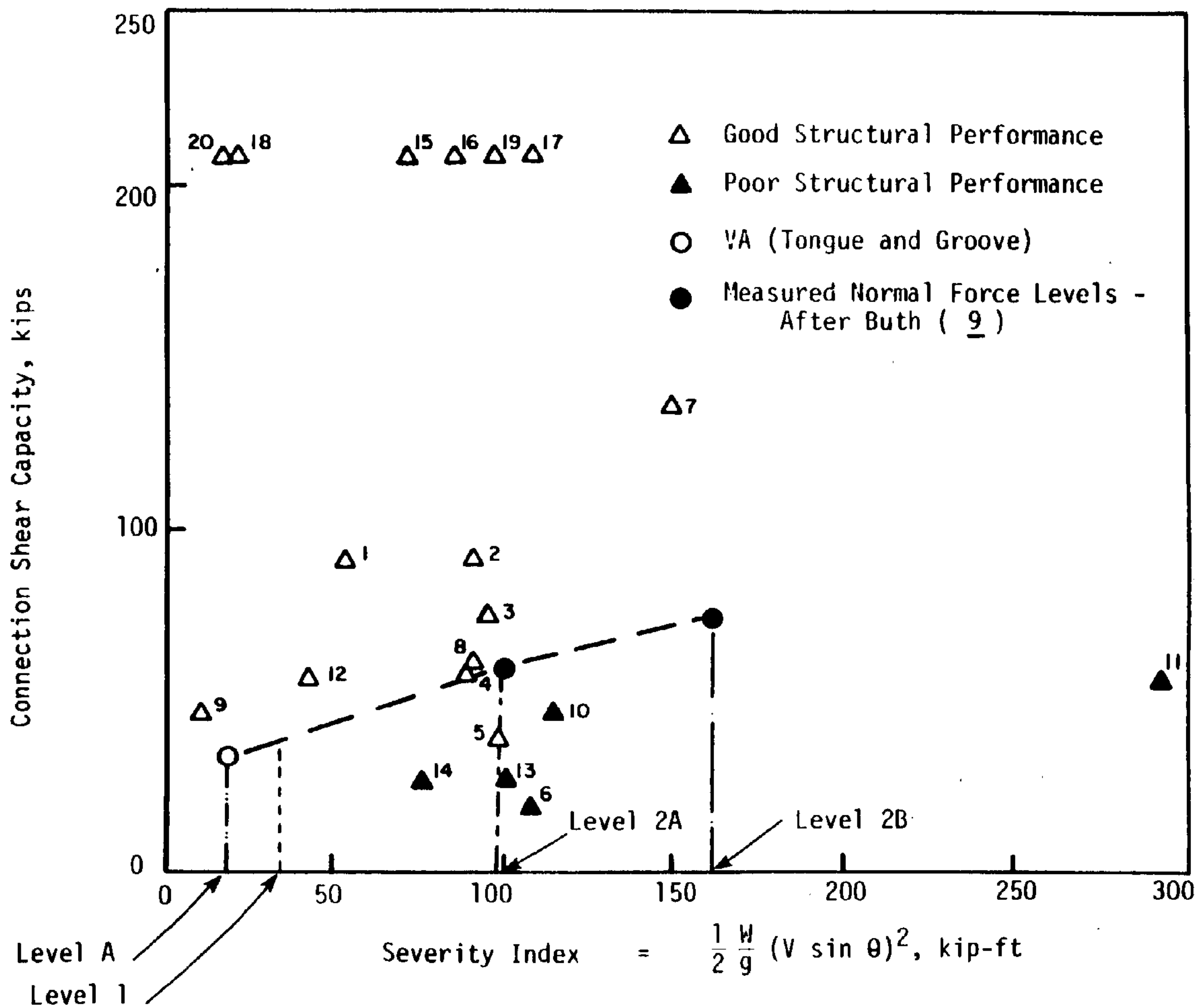


Figure 37. Shear Capacity of Connection vs. Severity Index.

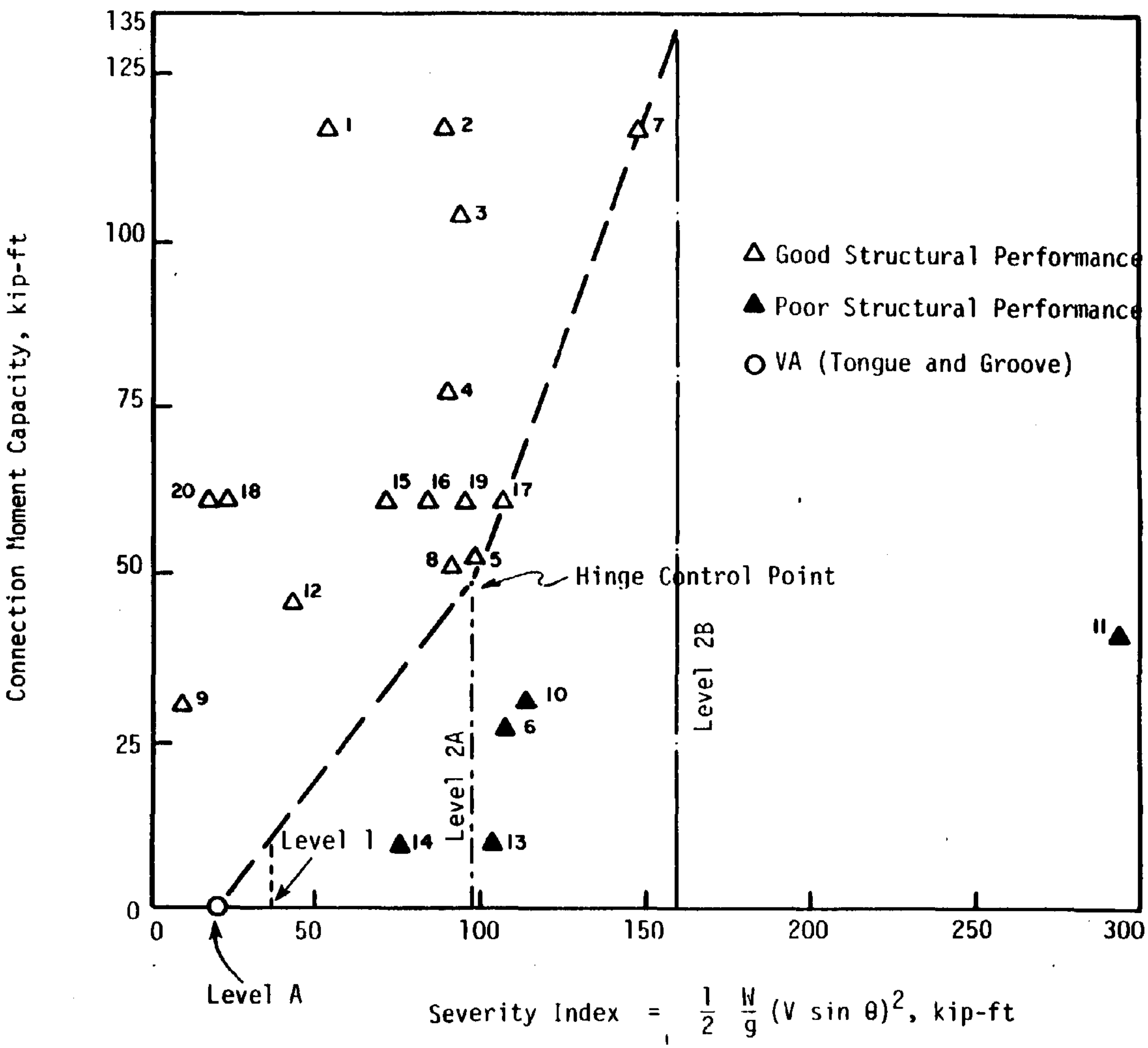


Figure 38. Moment Capacity of Connection vs. Severity Index.

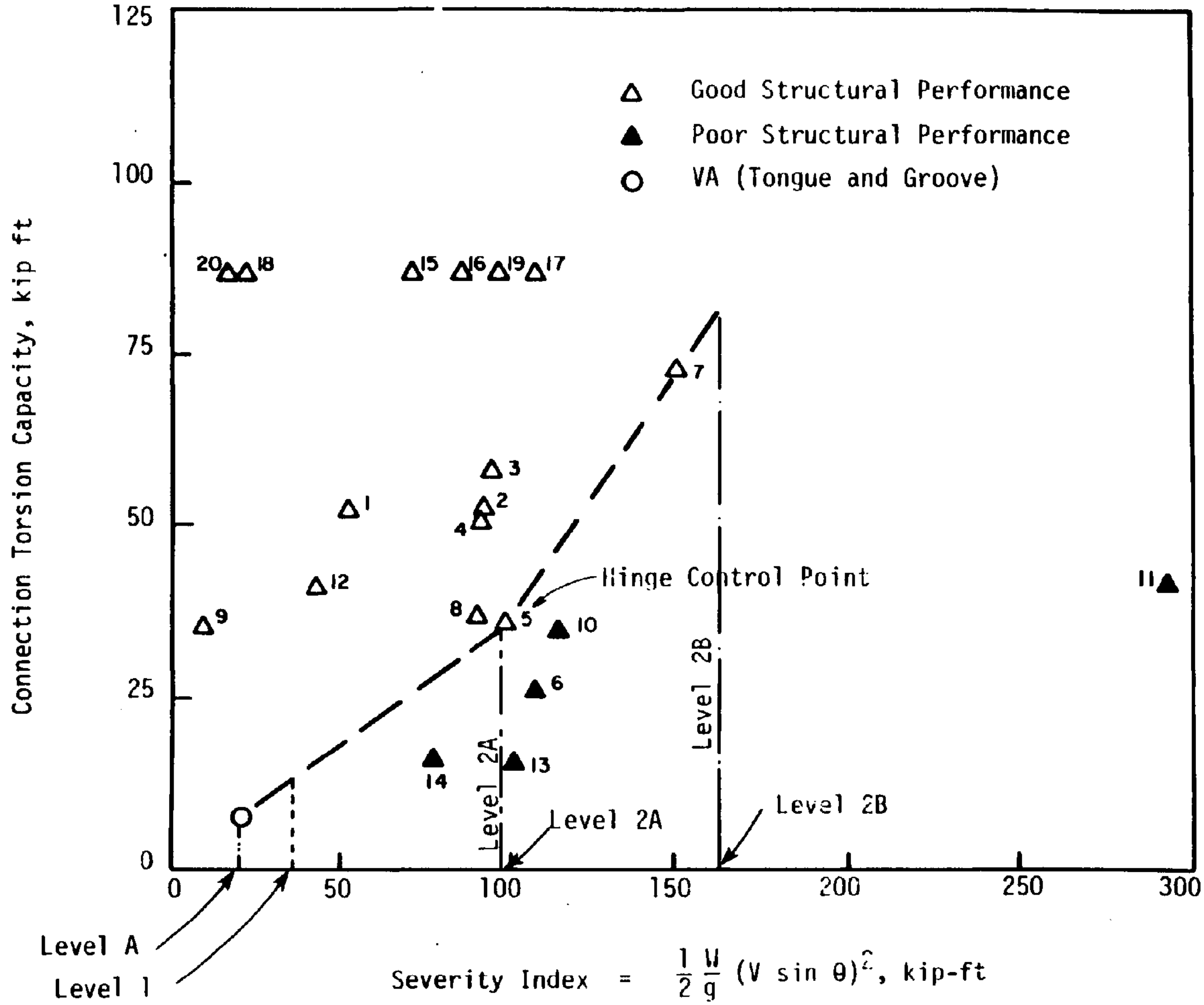


Figure 39. Torsion Capacity of Connection vs. Severity Index.

Considering moment capacity, Test series 3825-5, 6, 7 and 9 (data points 3, 4, 5 and 6) showed that a satisfactory performance could be achieved with a capacity of 50 kip-ft. This value for a Level 2A test was used as the "hinge" point in Figure 38. The final portion of the boundary in Figure 38 was set by data points 17 and 7. If anything, the boundary should be conservative, i.e. it should require moment capacities for the Service Levels above 2A that are greater than actually needed.

In Figure 39, on torsion capacity, Test 3825-5 (data point 5) was used again as the "hinge" point with the remainder of the boundary set by data point 7. Again, the line set in this way should be conservative. Using these boundaries in Figure 37, 38 and 39, it was then possible to set connection capacity requirements for Service Levels A, 1, 2A and 2B. These recommendations are described by the following summary.

BARRIER PERFORMANCE LEVELS -
RECOMMENDATIONS FOR CONNECTION STRENGTH

<u>Level A</u> (4500 lb/15 deg/45 mph) or (3500 lb/15 deg/60 mph)

Barrier connections should have strength characteristics that equal or exceed the following:

Moment - No requirement
Torsion - 10 kip-ft
Shear - 30 kips

The length of barrier segments should be twelve feet or greater unless the performance of shorter segments is verified by test:

Value of $\frac{1}{2}\frac{W}{g}(V \sin \theta)^2$ = 20.4 kip-ft

<u>Level 1</u> (4500 lb/15 degrees/60 mph)

Barrier connections should have strength characteristics that equal or exceed the following:

Moment - 10 kip-ft
Torsion - 15 kip-ft
Shear - 40 kips

Value of $\frac{1}{2}\frac{W}{y}(V \sin \theta)^2$ = 36.5 kip-ft

<u>Level 2A</u> (4500 lb/25 degrees/60 mph)

Barrier connections should have strength characteristics that equal or exceed the following:

Moment - 50 kip-ft
Torsion - 40 kip-ft
Shear - 60 kips

$\frac{1}{2}\frac{W}{g}(V \sin \theta)^2$= 97.3 kip-ft

Performance level 2A(1) would limit barrier deflections to approximately one foot or less. Performance level 2A(2) would limit barrier deflections to approximately two feet or less. Table 11, can be used to select combinations of barrier moment capacity and barrier segment length to meet either 2A(1) or 2A(2) requirements. Where different segment lengths are used, linear interpolation to determine probable barrier deflection is acceptable.

The expected deflections listed in Table 11 assume a "moment free" connection rotation of not more than three degrees. This corresponds to a slack space between the installed segments of approximately 5/8 inch.

Level 2B (20,000 lb/15 degree/60 mph)

Barrier connections should have strength characteristics that equal or exceed the following:

Moment	-	130 kip-ft
Torsion	-	80 kip-ft
Shear	-	73.8 kips

Value of $\frac{1}{2}\frac{W}{g}(V \sin \theta)^2 = 161.1$ kip-ft

Achievement of these strength levels along with barrier connection slack less than 3 degrees should result in deflections less than two feet.

Level 3 (40,000 lb/15 degrees/60 mph)

Moment	-	260 kip-ft*
Torsion	-	160 kip-ft*
Shear	-	150 kips*

Value of $\frac{1}{2}\frac{W}{g}(V \sin \theta)^2 = 322.2$ kip-ft

*The values are highly speculative. They were selected as double the 2B requirements since the severity index of a Level 3 test is double that of a Level 2B test. The performance of barriers so designed should definitely be verified by full scale crash tests. The reinforcement of most barrier segment designs would need to be increased so that the main segment structure could resist similar loads (i.e. the part of the barrier segment in between the connections).

Table 11. Combinations of Barrier Moment Capacity and Segment Length to Limit Barrier Deflections.

Connection Moment Capacity kip-ft	Barrier Segment Length feet	Expected* Barrier Deflection feet	Performance Level 2A (1 or 2)
50	10	1.8	2A(2)
50	20	2.1	2A(2)
50	30	0.9	2A(1)
75	10	1.2	2A(2)
75	20	1.7	2A(2)
75	30	0.9	2A(1)
100	10	1.0	2A(1)
100	20	1.5	2A(2)
100	30	0.9	2A(1)

*As predicted by the "Simplified Energy Analyses", Figure 9 through 14 and Appendix B. Connection slack must be equal or less than three degrees for these barrier deflections to be reasonably accurate.

Comparison of Connection Designs

Table 12 lists the strength characteristics for connection designations C1 through C10. The criteria used to estimate performance levels has been applied to these connections and is so tabulated. These connections represent the full spectrum of those used on portable concrete barriers. They do not represent any particular design in the computation of strength characteristics, but represent the values of those types of connections based on selected material strength levels which are described in Appendix D. The effort here is not necessarily to compare specific designs but to compare specific types of designs. Each design could be made stronger or weaker based on the selection of other material properties. It is seen connections C1, C2 and C3 only qualify in structural characteristics for Service Level A due to the fact that they do not have significant moment capacities. C4 and C5 qualify for Service Level 1. C6, C7, C8 and C9 qualify for Service Level 2A and one design, C10, the Welsbach, qualifies for Service Level 2B. Graphically comparing these connection designs with the performance level recommendation line as shown in Figure 40, shows how these barriers compare, based on moment capacity.

Further Performance Considerations

The first requirement of a barrier evaluation system is that the design be structurally adequate. The proposed connection design criteria given prior to this allows connections to be designed based on shear, moment and torsion strength characteristics for specific Service Levels. Note this does not assume that vehicle reaction to a specific design will comply with NCHRP Report 230. Testing is ultimately needed to verify that compliance. As an alternative to specifically designing a barrier to meet a Service Level, certain designs can now be selected which meet a specific Service Level. Once adequate performance has been determined for the Service Level selected, based on the probability of various kinds of collisions, the next item is to make sure the barrier deflection characteristics are acceptable for a particular site. In Chapter II, it has been shown that the amount of lateral distance available for deflection is highly variable. Also discussed in Chapter II, for Performance Level 2A, a portable concrete barrier may not perform

Table 12. Strength Characteristics of Connection Types.

Connection Designation	Connection Name	Strength Characteristics			Estimated Service Level
		Shear kips	Moment kip-ft	Torsion kip-ft	
C1	Tongue and Groove	32	0	7	A
C2	Steel Dowel	60	0	37	A
C3	Grid Slot	60	0	30	A
C4	Top T-Lock	190	11	56	1
C5	Lapped Joint	47	22	24	1
C6	Pin and Rebar	85	57	60	2A
C7	Vertical I-Beam	210	61	87	2A
C8	Bottom T-Lock	590	66	370	2A
C9	Channel Splice	67	80	36	2A
C10	Welsbach	160	139	94	2B

* These strength characteristics were calculated using average material strength levels given in Appendix D. In many cases these levels are not the same as specific designs used in some states.

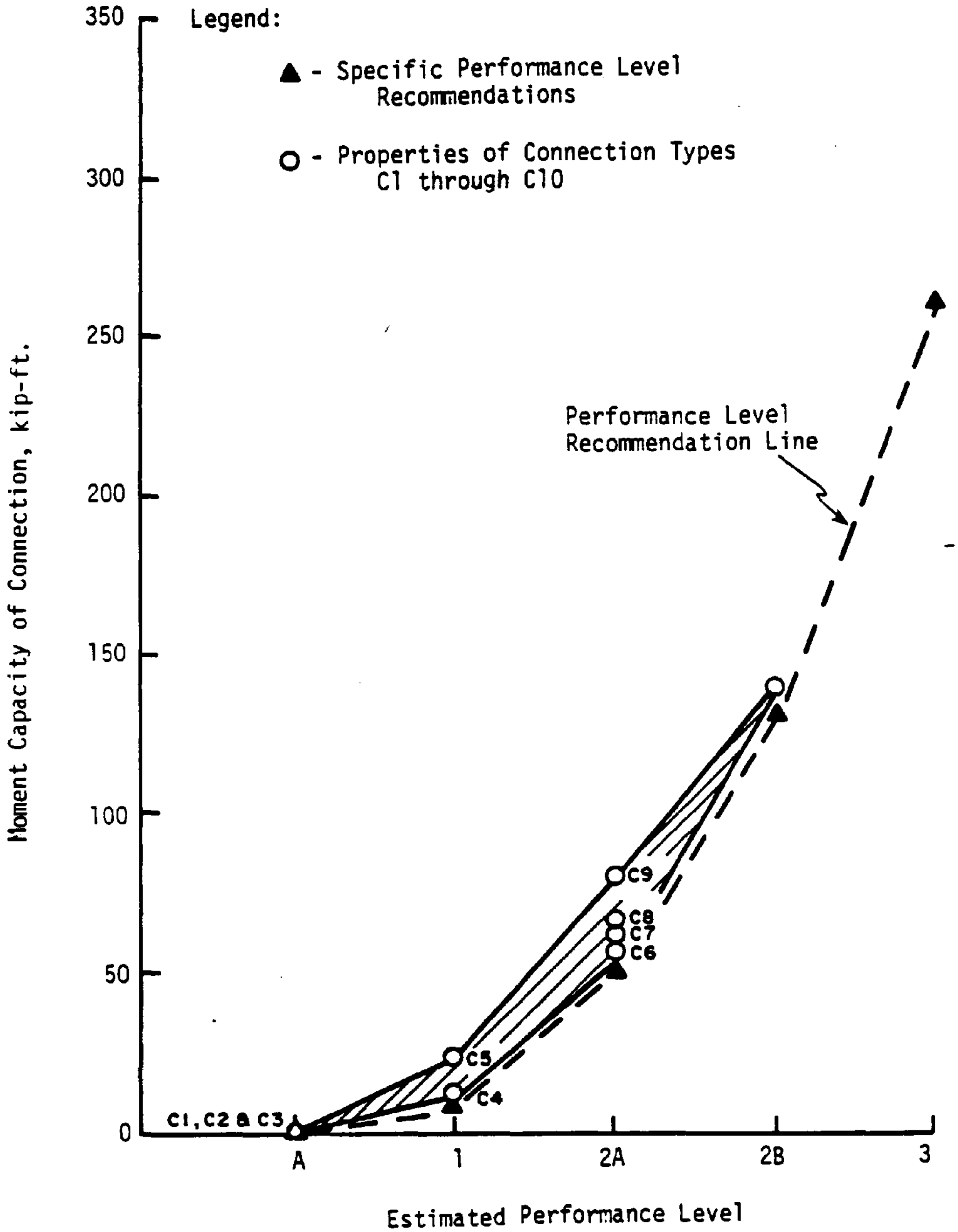


Figure 40. Estimated Service Levels For Various Connection Designs Based on Moment Capacity of Connection.

adequately, if deflections are greater than two feet. To be conservative, it seems appropriate to limit deflections to approximately one foot by selections shown in Table 11. For Service Level 2A and above, it is suggested that Table 11 be used to select appropriate combinations of moment and barrier segment length, with careful consideration being given to the effect of connection slack on deflection. The calculated values are based on an estimated connection slack of three degrees which is a fairly rigorous requirement for construction barriers. For example, the usual pin type connection, C6, requires some grouting of the joint to limit connection slack to less than eight degrees. As a further illustration of the problem of estimating barrier deflections, Figure 41 is presented. Although there appears to be a reasonable trend, based on the 20 available crash tests the scatter is large. Other characteristics being equal, the maximum static deflection should be somewhat proportional to the Severity Index. Such things as the friction of the barrier with respect to the supporting media, connection slack and segment length, all being somewhat variable in the crash tests available, contribute to this data scatter.

Costs and Convenience

Following the selection of a Performance Level, and appropriate consideration of deflection, the next consideration should be one of cost. In Appendix E Koppa has presented a rigorous analysis of barrier costs considering the ten connection types shown in Table 12. In Table 13, he has determined that connection costs can range from as little as three dollars per connection for the C1 (Tongue and Groove) to as much as $87 per connection for the relatively complex C10 (Welsbach). The influence of these connection costs on the average barrier cost per linear foot is illustrated by Figure 42. Plotting barrier segment length as a function of barrier connection costs yields a series of lines showing average costs per linear foot of barriers from $16 to $24 per foot. For the high cost connections the cost per linear foot will be closely related to the barrier segment length, which dictates the number of connections for a given length of barrier. The most economical barriers, if Service Level A is sufficient, is a 30 ft barrier length with a $3 connection cost. An opposite extreme, a 10 ft barrier segment length and $87 connection would

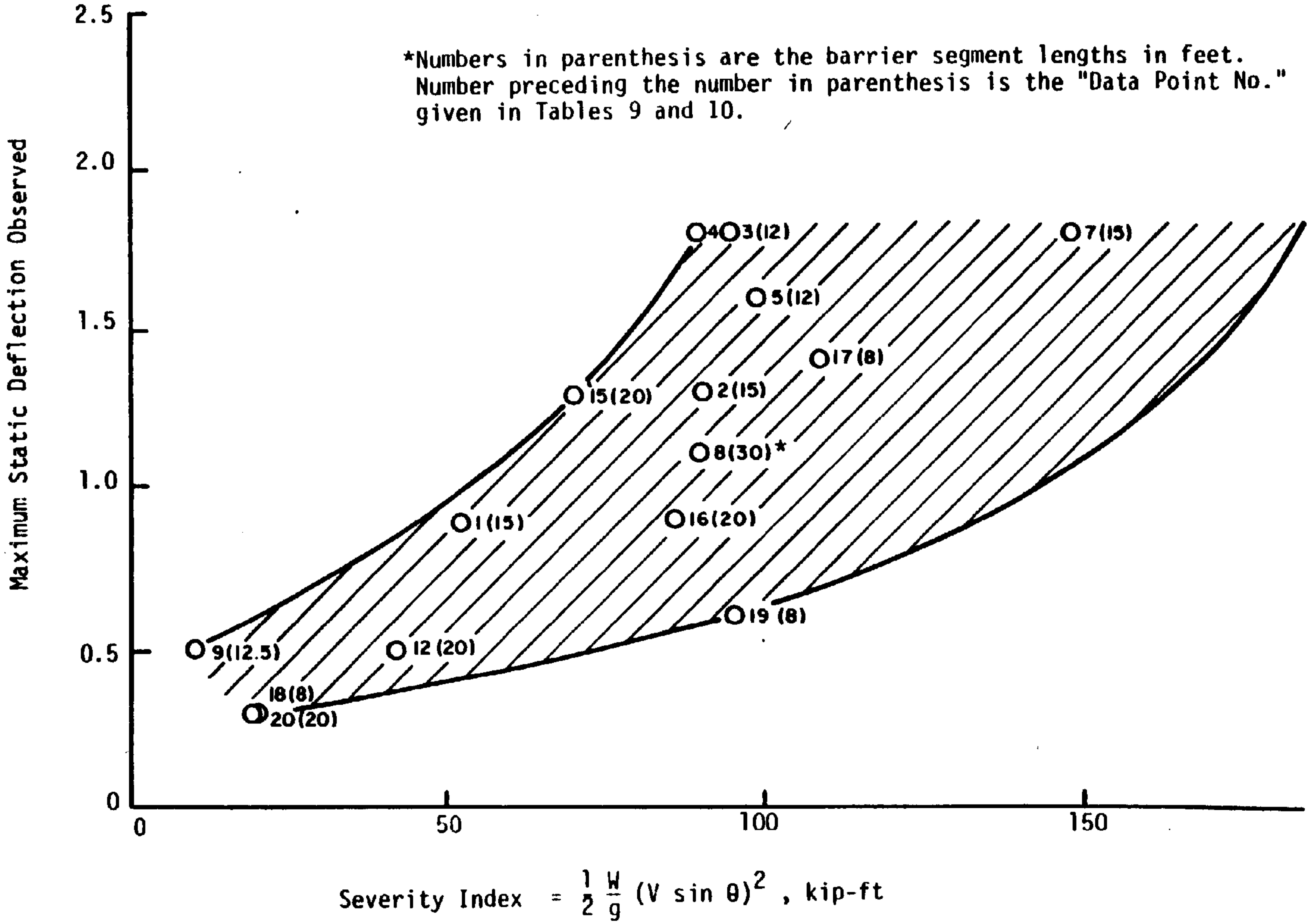


Figure 41. Static Deflections.

Table 13. Joint Fabrication Cost Analysis.

CONCEPT (1)	HARDWARE REQ'D (2)	MFG OPRNS (3)	NAT'L COST (4)	LABOR COSTS (5)	TOTAL DIRECT COST (6)	NEAREST $.50 (7)
C1-Tongue & Groove	Nose Cap over Tongue	Cut Stamp	$2.40	$ .69	$3.09	3.00
C2-Dowel	Steel Rods	Cut	$3.20	$ .33	$3.53	4.00
C3-Grid Slot	Grid of Steel Bar	Cut Weld	$5.33	$1.69	$7.02	7.00
C4-Top T-Lock	Channel Tubes Plates Pins	Cut Drill Weld	$9.00	$3.52	$12.52	13.00
C5-Lapped Joint	Bolt Re-Plates	Cut Notch Drill	$8.55	$1.72	$10.27	10.00
C6-Pin & Rebar	Rebars Bolt	Cut & Form Bars	$13.62	$7.08	$20.70	21.00
C7-Vertical I-Beam	I-Beam Tubes Re-Plates	Cut Slot Weld	$24.27	$14.82	$39.09	39.00
C8-Bottom T-Lock	Tube Base Pipe Tubes	Cut Split Weld	$34.00	$4.15	$38.15	38.00
C9-Channel Splice	Channel 4 Bolts Re-Plates	Cut Drill Clear	$50.00	$5.35	$55.35	55.00
C10-Welsbach	T-Rails L-Anchors Socket Assy. Anchors	Cut Form Bend Weld	$45.96	$41.16	$87.12	87.00

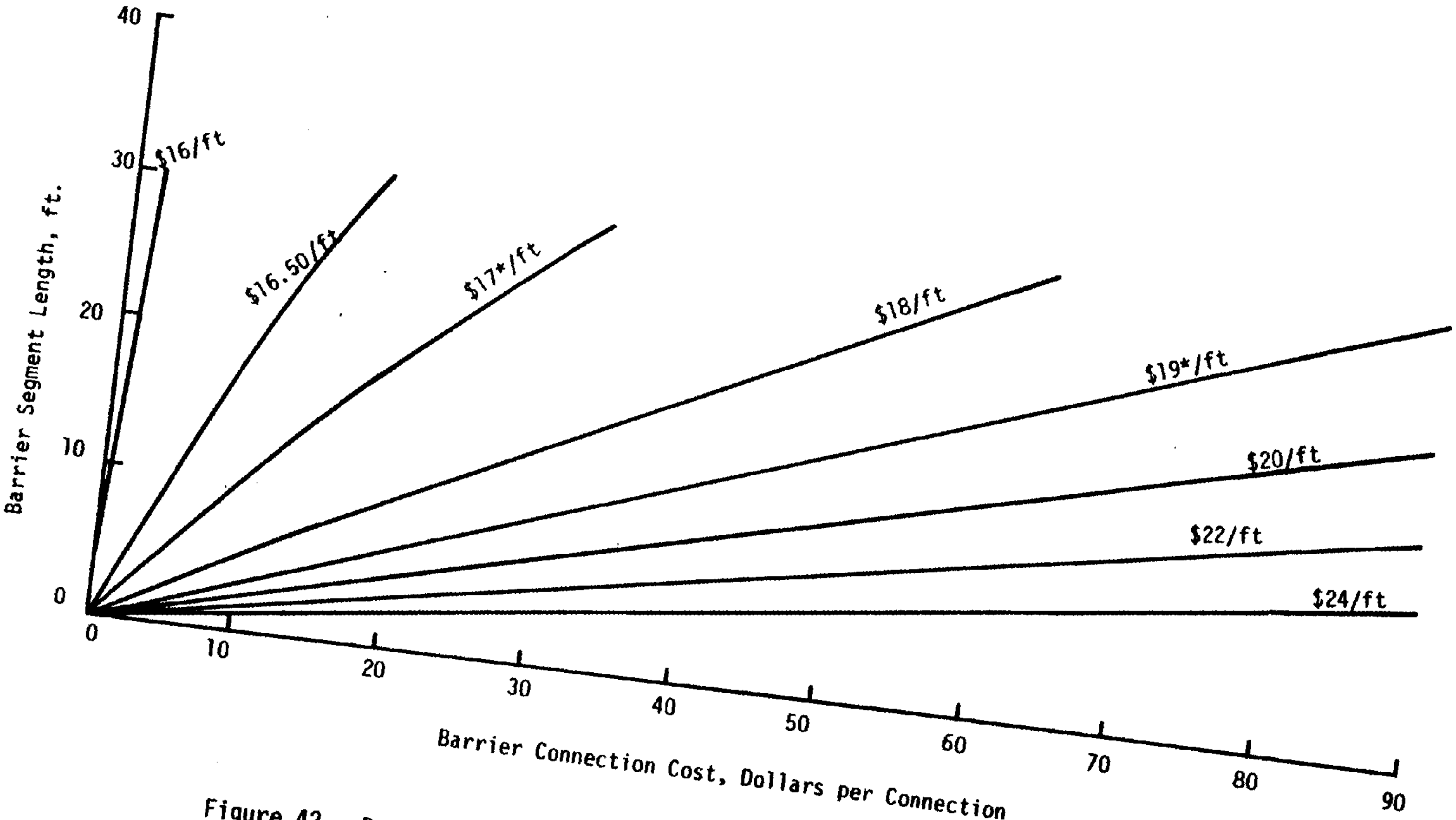


Figure 42. Barrier Fabrication Cost** as a Function of Barrier Segment Length and Individual Connection Cost.

*Fabrication cost per linear foot.
**F.O.B. point of fabrication - contractor profit not included - assumed longitudinal reinforcement approximately 10 lbs of steel per foot.

result in a construction barrier average cost of $24 per ft. The relationship between connection fabrication costs and estimated service level is shown in Figure 43. As would be expected, the performance level is highly related to the connection fabrication cost, with the highest Service Level achieved by the costly Welsbach design. There is a high degree of variability, however, in the connection costs for barriers meeting the 2A Service Level; this cost varying from $20 to $50 per connection. There certainly appears to be some advantage in selecting one of the lower cost connections. For example, C6, the venerable Pin and Rebar, appears to have a real cost advantage at the 2A performance level.

Portability

Barrier portability is one aspect of cost and convenience which has not been previously defined, but which was considered in both the field studies and estimates by Koppa. In Figure 44, as an indicator of portability, the parameter consisting of the sum of disassembly, pickup, placement and reassembly time in man-minutes has been compiled by Koppa. If this sum is plotted as a function of estimated Service Level, a trend may be seen toward decreased portability at the higher performance level. One would think that this trend is somewhat obvious given the extreme simplicity of assembling such designs as the Tongue and Groove, C1, at Service Level A.

There are, however, some significant outlyers. For example, the Welsbach design, is competitive and even better than many of the 2A Service Level designs in terms of portability. The Channel Splice design is a significant outlyer in that its portability is significantly poorer than all other designs considered. The problem of bolt-hole alignment and damaged bolt threads during assembly and disassembly seems to be an overriding factor in the Channel Splice design.

Koppa did not find a difference in portability of barrier segment length. This is discussed in some detail in Appendix E. With the equipment normally in use by contractors, it is possible to haul approximately the same length of barrier on a flat bed or low-boy independent of segment length. The major difference in portability comes about in the number of connections that require assembly. That number is a direct function of barrier segment length. The end result is that longer barrier segments can be moved and reassembled in shorter periods of time.

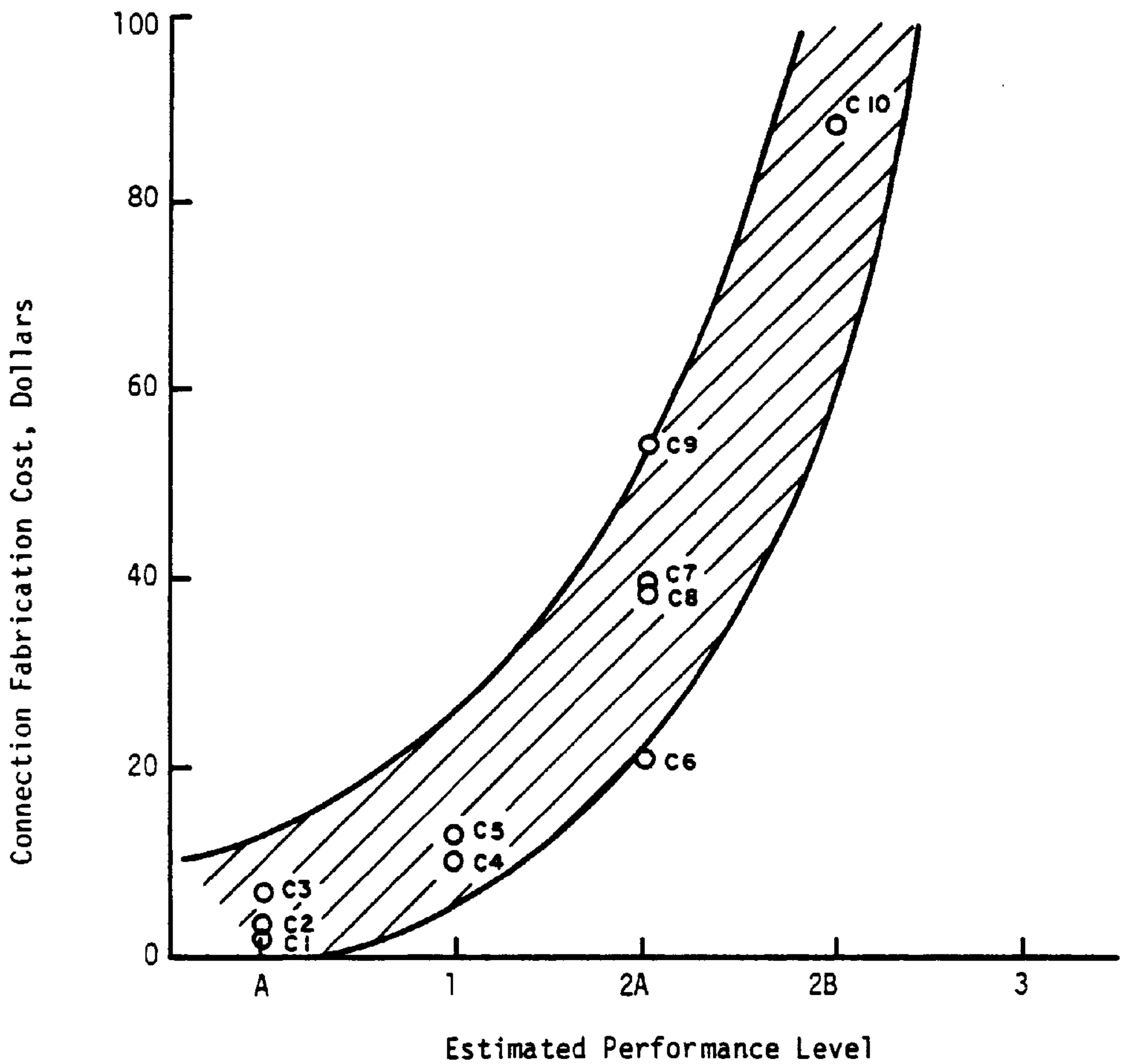


Figure 43. Relationship Between Connection Cost and Performance Level.

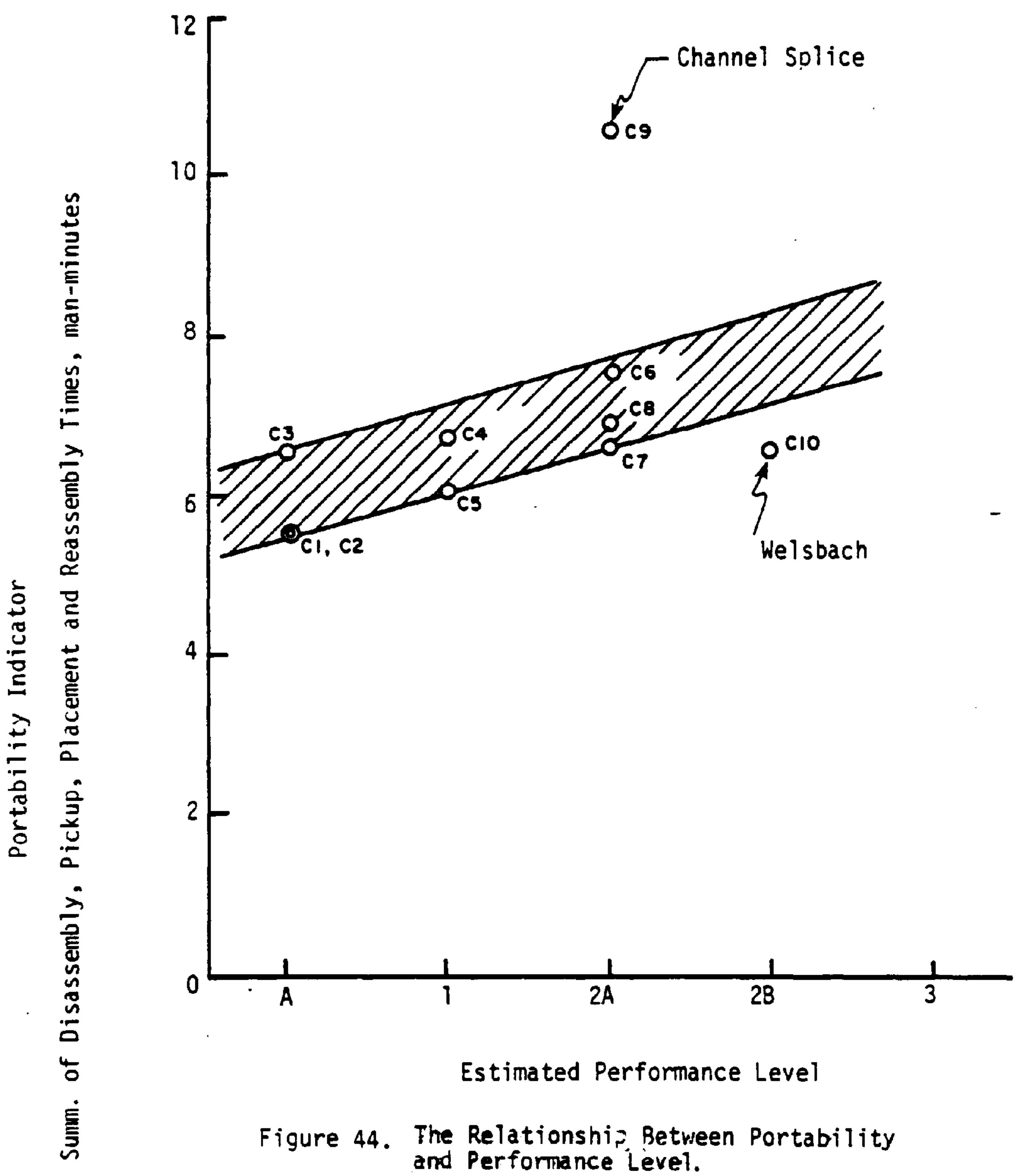


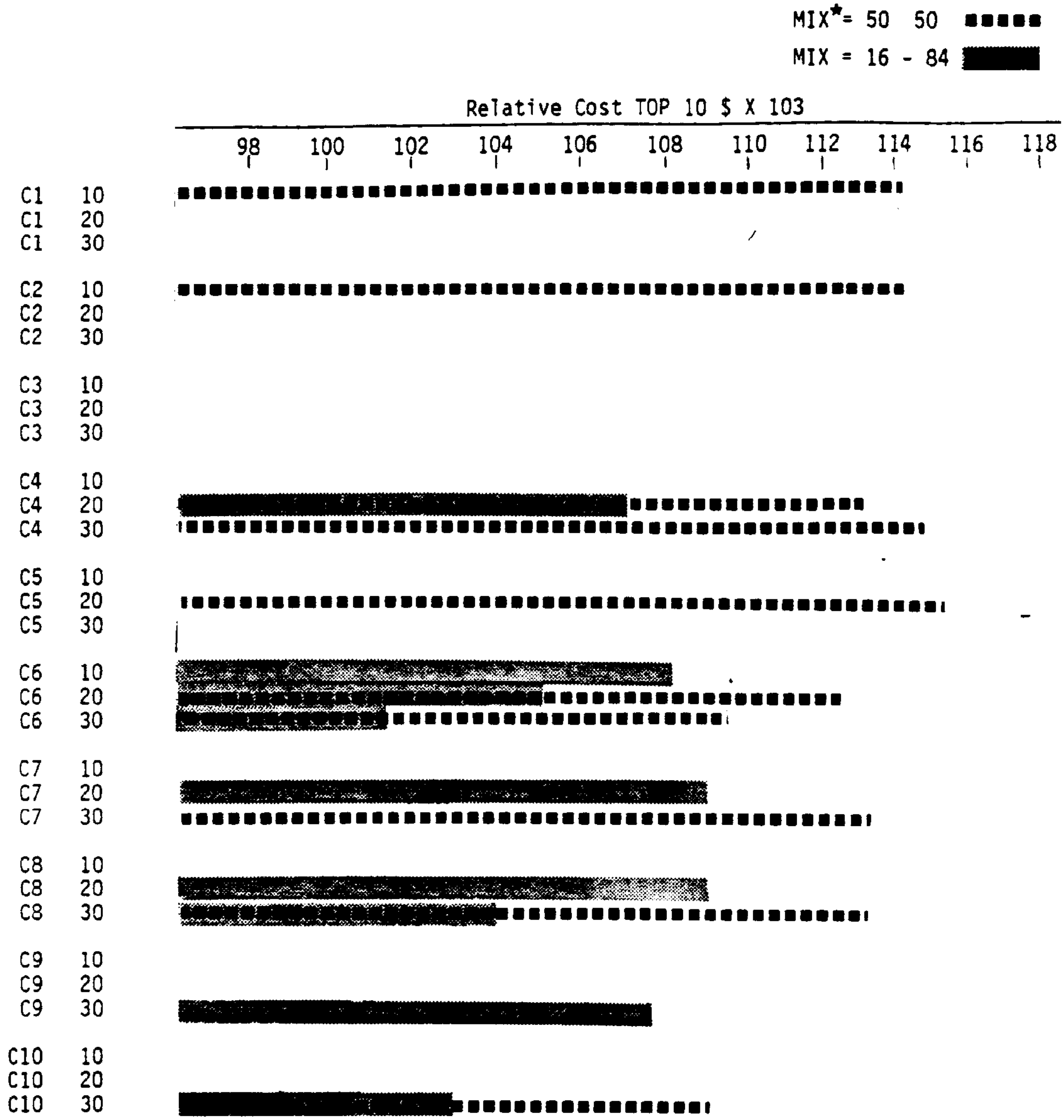
Figure 44. The Relationship Between Portability and Performance Level.

Koppa has presented a detailed process to determine total cost for particular barrier designs. Results of his complete analyses are illustrated by Figure 45. The total cost of one thousand feet of barrier for one year, which includes fabrication, placement, movement and maintenance costs is illustrated by the final column of Table 14. This gives a cost for all ten barrier connections with segment lengths of 10, 20 and 30 ft. In all cases, there is a lower cost with increased barrier segment length. From all these, the top ten combinations of connections and segment length were placed in bar graph form in Figure 45. This figure shows the lowest overall cost barrier connection-length combination is the C6, Pin and Rebar, with a 30 ft segment length. A close second, to the surprise of the writers, is the Welsbach at a 30-ft segment length. A key factor here is the estimate of damage caused by selected vehicle collisions, an estimate necessary to determine maintenance costs. While the Welsbach has a high joint connection cost, at a 30-ft length the overall cost per unit length of barrier is only about $19. The Pin and Rebar with a connection cost of only $21, at a 30-ft length has a barrier cost per linear foot of approximately $16.50. An initial cost differential of about $2.50 per ft exists, which is further reduced by consideration of maintenance cost.

The Welsbach design becomes uncompetitive at reduced segment lengths except for the situation when an extremely high percentage of the traffic is trucks. In that case, the Welsbach is close in overall costs to the Pin and Rebar.

The design developed in this contract, C8 Bottom T-Lock, has a connection cost of approximately $38. When used in a 30-ft joint length it is roughly two percent higher than the Pin and Rebar but eliminates the need for joint grouting to reduce slack. It is competitive with all designs except the Welsbach for the high truck mix traffic situation.

Considering further the very simple Tongue and Groove, while the barriers have the lowest initial cost and installation costs, the maintenance costs are significantly higher than the strong connection designs. This can result in higher overall costs than many of the more formidable connections. The way in which these data can be used systematically to make decisions on appropriate barrier designs is illustrated by the following:



*Percentage of trucks and passenger cars respectively.

Figure 45. Comparison of Ten Least Expensive PCB Concepts.

Table 14. Total 1 Year Costs With Maintenance for Trucks 16% - Passenger Cars 84%.

Concept	Length	Fabricate	Install	Level A	Level 1	Level 2A	Level 3	Main Cost	Total Cost
C1 Tongue	10	$ 81,500	$ 6,900	$ 765	$ 928	$ 1,254	$ 1,906	$ 21,559	$ 109,959
C1 Tongue	20	80,750	6,400	925	1,248	1,571	1,894	26,510	113,660
C1 Tongue	30	80,900	5,900	1,085	1,568	1,568	2,051	29,428	116,228
C2 Dowel	10	82,000	6,900	766	930	1,258	1,914	21,618	110,518
C2 Dowel	20	81,000	6,400	926	1,250	1,574	1,898	26,555	113,955
C2 Dowel	30	80,650	5,900	1,086	1,570	1,570	2,054	29,464	116,014
C3 Grid	10	83,500	7,050	769	936	1,270	1,938	21,793	112,343
C3 Grid	20	81,750	6,550	929	1,256	1,585	1,910	26,704	115,004
C3 Grid	30	81,150	6,050	1,089	1,576	1,576	2,063	29,572	116,772
C4 Top T	10	86,500	7,100	0	775	1,294	1,986	16,834	110,434
C4 Top T	20	83,250	6,600	0	935	1,268	1,934	17,460	107,310
C4 Top T	30	82,650	6,050	0	1,098	1,594	2,090	20,970	109,670
C5 Lapped	10	85,000	7,000	772	772	1,282	1,962	21,016	113,016
C5 Lapped	20	82,500	6,500	932	932	1,262	1,922	22,600	111,600
C5 Lapped	30	81,650	5,950	1,091	1,091	1,582	2,072	26,925	114,525
C6 Vert P	10	90,500	7,350	0	0	964	2,050	10,221	108,071
C6 Vert P	20	85,250	6,750	0	0	1,284	1,966	12,390	104,390
C6 Vert P	30	83,500	6,150	0	0	1,103	2,105	11,310	100,960
C7 Vert I	10	99,500	7,050	0	0	1,000	2,194	10,710	117,260
C7 Vert I	20	89,750	6,550	0	0	1,320	2,038	12,765	109,065
C7 Vert I	30	86,500	6,050	0	0	1,121	2,159	11,526	104,076
C8 Bottom	10	99,000	7,050	0	0	998	2,186	10,683	116,733
C8 Bottom	20	89,500	6,550	0	0	1,318	2,034	12,744	108,794
C8 Bottom	30	86,350	6,050	0	0	1,120	2,156	11,514	103,914
C9 Splice	10	107,500	8,050	0	0	1,032	2,322	11,146	126,696
C9 Splice	20	93,750	7,200	0	0	1,352	2,102	13,098	114,048
C9 Splice	30	89,150	6,350	0	0	1,169	2,302	12,100	107,600
C10 Welsb	10	123,500	7,050	0	0	1,096	2,084	11,226	141,776
C10 Welsb	20	101,750	6,550	0	0	1,416	1,823	13,112	121,412
C10 Welsb	30	94,500	6,050	0	0	0	1,736	2,778	103,328

Factor for Cars = 6.00
Factor for Trucks = 2.00

Economic Evaluation of Proposed Construction Barrier Designs

The best barrier for a specific project is a function of the degree of protection required, (i.e. the "Service Level"), and the various costs encountered in providing that level of protection. The service level for a specific design can be estimated by comparing the barrier connection characteristics with the guidelines presented in Chapter III. Table 12 may also be used for many designs. The cost for a given service level are a function of initial barrier costs, installation, movement and removal costs and maintenance costs. They are shown in Appendix E to be functions of total required barrier length, the project duration, the number of barrier movements and ADT for each barrier application. Once the Service Level has been selected or determined, preferably by using the criteria presented by Ross, et al (8), the next step is to select those designs meeting that Level and then to consider costs.

It is possible to use the same cost procedures outlined by Koppa (see Appendix E) to compare the total project cost of each barrier design. In an effort to simplify the cost estimating procedure, the examples developed by Koppa can be used as reference costs. Table 15 was developed from Appendix E data to produce unit costs of fabrication, installation, relocation, maintenance and removal. These costs can be used with the following equation to estimate total barrier costs, TBC, for the duration of tne project for any combination of joint type and segment length.

$$TBC = L \left[FAB + INS + REL\ (n_i) + REM + \frac{MAIN\ (C_t)\ t_i\ (ADT_i)}{2.4 \times 10^6} \right]$$

Where: L is the length of barrier to be installed in feet

FAB is the fabrication cost in dollars per foot (Column 3, Table 15).

INS is the installation cost in dollars per foot (Column 4, Table 15).

REL* is the relocation cost in dollars per foot (Column 5, Table 15).

REM* is the removal cost in dollars per foot (Column 6, Table 15).

* Due to certain simplifying assumptions made in the cost analysis Appendix E, the values of REL are equal to the values of REM.

Table 15. Simplified Cost Data.

DESIGNATION AND NAME	LENGTH FT	FABRICATION COST/FT	INSTALLATION COST/FT	RELOCATION COST/FT	REMOVAL COST/FT	MAINTENANCE COST/FT
C1-Tongue&Groove	10	16.30	1.38	1.11	1.38	4.31
	20	16.15	1.28	1.02	1.28	5.30
	30	16.10	1.18	0.92	1.18	5.88
C2-Dowell	10	16.40	1.38	1.11	1.38	4.32
	20	16.20	1.28	1.02	1.28	5.31
	30	16.13	1.18	0.92	1.84	5.89
C3-Grid Slot	10	16.70	1.41	1.15	1.41	4.36
	20	16.35	1.31	1.05	1.31	5.34
	30	16.23	1.21	0.95	1.21	5.91
C4-Top T Lock	10	17.30	1.42	1.17	1.42	3.37
	20	16.65	1.31	1.06	1.31	3.49
	30	16.43	1.21	0.95	1.21	4.19
C5-Lapped	10	17.00	1.40	1.15	1.40	4.20
	20	16.50	1.35	1.04	1.35	4.52
	30	16.33	1.19	0.93	1.19	5.39
C6-Pin and Rebar	10	18.10	1.47	1.25	1.47	2.04
	20	17.05	1.35	1.11	1.35	2.48
	30	16.70	1.23	0.98	1.23	2.26
C7-Vertical I-Beam	10	19.90	1.41	1.15	1.41	2.14
	20	17.95	1.31	1.05	1.31	2.55
	30	17.30	1.21	0.95	1.21	2.31
C8-Bottom T-Lock	10	19.80	1.41	1.14	1.41	2.14
	20	17.90	1.31	1.04	1.31	2.14
	30	17.27	1.21	0.94	1.21	2.55
C9-Side Channels	10	21.50	1.61	1.54	1.61	2.23
	20	18.75	1.44	1.30	1.44	2.62
	30	17.83	1.27	1.07	1.27	2.42
C-10 Welsbach	10	24.70	1.42	1.14	1.41	2.25
	20	20.35	1.31	1.04	1.31	2.62
	30	18.90	1.21	0.94	1.21	0.56

MAIN is the base maintenance cost in dollars per foot (Column 7, Table 15) This base cost is for a 16% truck mix and a 200,000 ADT for a period of 12 months.

n_i is the number of times the barrier will be moved during the course of the construction period.

C_t is the normalization coefficient for the percentage of truck traffic (See Figure 46).

t_i is the time the barrier will be used in months.

ADT_i is the average daily traffic expected through the construction zone.

It is recognized that this equation has certain limitations in accuracy brought about by the approximations discussed in Appendix E. Also, the probability of collisions is based on a very simple model using only project duration and ADT. However, it is likely this degree of sophistication, or lack of same, is as much as can be practically justified in discriminating between barrier designs. As a way of illustrating this method, the following example has been developed:

Example 1.

Given: A construction project requiring 1,000 ft of barrier to be moved three times during the course of an 8-month construction period. The percentage of truck traffic is 25%. The ADT of the highway is 50,000. Service Level A is considered sufficient.

Problem: Determine the total cost of three possible configurations to select the lowest cost alternative.

a. Twelve-ft sections of barrier with connection C1 (Tongue and Groove)
b. Twenty-ft sections of barrier with connection C6 (Pin and Rebar)
c. Thirty-ft sections of barrier with connection C10 (Welsbach)

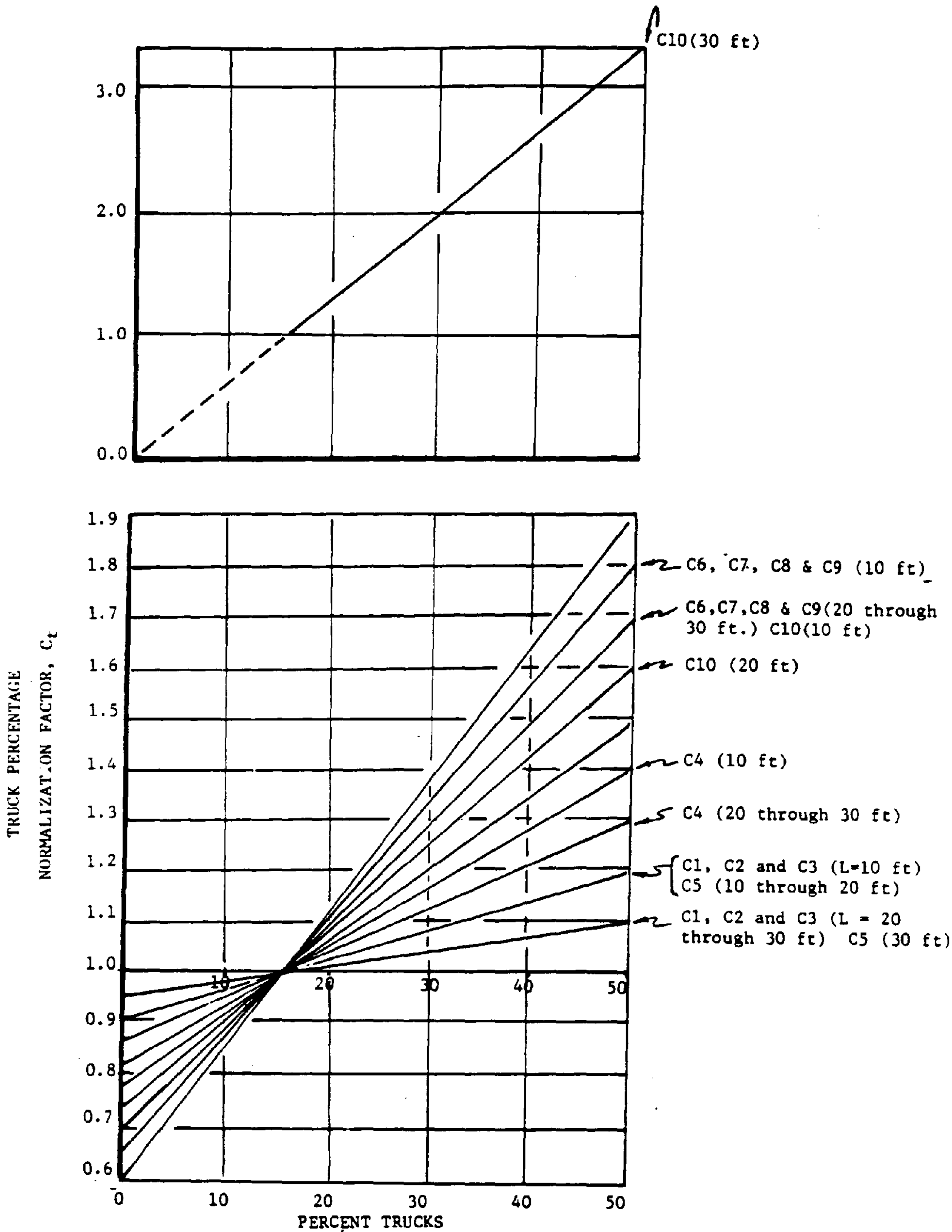


Figure 46. Truck Percentage Normalization Factor, C_t.

Choice a.

$$TBC_a = 1000\ [16.27 + 1.36 + 3(1.09) + 1.36 + \frac{4.31(1.06)8(50{,}000)}{2.4 \times 10^6}]$$

$$= 1000\ [16.27 + 1.36 + 3.27 + 1.36 + 0.76]$$

$$= 1000\ [\$23.02] = \$23{,}020$$

Choice b.

$$TBC_b = 1000\ [17.05 + 1.35 + 3(1.11) + 1.35 + \frac{2.48(1.18)8(50{,}000)}{2.4 \times 10^6}]$$

$$= 1000\ [17.05 + 1.35 + 3.33 + 1.35 + 0.49]$$

$$= 1000\ [\$23.57] = \$23{,}570$$

Choice c.

$$TBC_c = 1000\ [18.90 + 1.21 + 3(0.94) + 1.21 + \frac{0.56(1.4)8(50{,}000)}{2.4 \times 10^6}]$$

$$= 1000\ [18.90 + 1.21 + 2.82 + 1.21 + 0.13]$$

$$= 1000\ [\$24.27] = \$24{,}270$$

Based on these examples, several observation may be made. First, if Service Level A is selected as acceptable, it appears the C1 (Tongue and Groove) will usually give the lowest cost, unless a more expensive connection is used in concert with longer segment lengths. In this case, the difference in cost of 12-ft Tongue and Groove sections is not significantly different from 20-ft sections connected by Pin and Rebar.

Also note the Pin and Rebar would provide a barrier with a Service Level of 2A for fundamentally the same cost. Even the most expensive connection, the Welsbach (C10), is not unreasonable in cost when used with segments 30 ft long. Of course, the least expensive treatment to achieve Service Level A would be 30-ft sections with Tongue and Groove connections.

i.e.

$$TBC = 1000\ [16.10 + 1.18 + 3(0.92) + 1.18 + \frac{5.88(1.03)8(50{,}000)}{2.4 \times 10^6}]$$

$$= 1000\ [16.10 + 1.18 + 2.76 + 1.18 + 1.01]$$

TBC = 1000 [$22.23] = $22,230

Example 2.

A second illustration was developed by considering five barriers that are capable of meeting Service Level 2A. Consider the comparison of connections C6, C7, C8, C9 and C10, used on segments 20 ft in length. The cost computations are:

$$TBC(C6) = 1000\ [17.05 + 1.35 + 3(1.11) + 1.35 + \frac{2.48(1.18)8(50{,}000)}{2.4 \times 10^6}]$$

$$TBC(C6) = \$23{,}570$$

$$TBC(C7) = 1000\ [17.95 + 1.31 + 3(1.05) + 1.31 + \frac{2.55\ (1.18)8(50{,}000)}{2.4 \times 10^6}]$$

$$TBC(C7) = \$24{,}220$$

$$TBC(C8) = 1000\ [17.90 + 1.31 + 3(1.04) + 1.31 + \frac{2.55(1.18)8(50{,}000)}{2.4 \times 10^6}]$$

$$TBC(C8) = \$24{,}140$$

$$TBC(C9) = 1000\ [18.75 + 1.30 + 3(1.44) = 1.30 + \frac{2.62(1.18)8(50{,}000}{2.4 \times 10^6}]$$

$$TBC(C9) = \$26{,}190$$

$$TBC(C10) = 1000\ [20.35 + 1.31 + 3(1.04) + 1.31 + \frac{2.62(1.16)8(50{,}000)}{2.4 \times 10^6}]$$

$$TBC(C10) = \$26{,}600$$

Consideration of these equations would lead to the following observations:

1. Under most circumstances the C6 (Pin and Rebar) connection design will be the lowest in cost. It has the lowest initial cost and maintenance cost.
2. Under most circumstances, the C9 (Channel Splice) connection design will be more costly than C6, C7 and C8. All three factors, initial cost, portability cost and maintenance cost are the highest in this group.
3. Under most circumstances, the cost of C7 (Vertical I-beam) and

C8 (Bottom T-Lock) are fundamentally the same. (See the discussion of one further advantage of C8 in the section called "Portability", Chapter III.)

4. Under most circumstances, the cost of C10 (Welsbach) will be higher than C6, C7 and C8. When n_i, t_i and ADT_i are large and when 30 ft segment lengths are used the cost of C10 should be competitive and may even become the most economical due to the extremely low maintenance cost of the longer segments.

5. Under certain circumstances the cost of C7 and C8 could become lower than C6, i.e. when n_i is large and ADT_i is low. Consider, for example, the following input data: t_i = 24 mo., ADT_i = 5000, segment length = 20 ft. and percentage of trucks = 16.

Equating TBC (C6) and TBC (C8) gives:

$$L\ [17.05 + 1.35 + n_i(1.11) + 1.35 + \frac{2.48(1.0)24(5000)}{2.4 \times 10^6} =$$

$$L\ [17.90 + 1.31 + n_i\ (1.04) + 1.31 + \frac{2.55(1.0)24(5000)}{2.4 \times 10^6}\ ]$$

The value of n_i from this equation is:

$$19.87 + 1.11\ n_i = 20.65 + 1.04\ n_i$$

$$0.07\ n_i = 0.78$$

$$n_i = 11$$

For $n_i = 11$, the costs should be the same.

For $n_i > 11$, the cost of C8 should be lower.

Of the five barriers capable of performing at the 2A Service Level, some further discussion is required to compare certain facets of each, facets that were not accounted for in the cost analysis. These factors are give in Table 16.

While the fabrication, installation and portability costs of the Channel Splice are high, it does have the definite advantage of requiring

Table 16. Deflection Reducing Measures.

Connection Design	Connection Name	Special treatment needed to reduce slack in joint to prevent excessive deflection under impact conditions.
C6	Pin and Rebar	Joint must be grouted or shear connectors provided between barrier and surface.
C7	Vertical I-Beam	Joint must be grouted or shear connectors provided between barrier and surface.
C8	Bottom T-Lock*	Joint must be "frozen" by injecting sand.
C9	Channel Splice	No treatment needed.

* Experimental section, no field experience at this time.

no special treatment to reduce joint slack. California uses steel rods as a shear connector for their Type K Barrier (the best of the Pin and Rebar designs) and New York recommends grouting its Vertical I-Beam connection. Other methods of providing shear connectors between a barrier base and the surface on which it rests are shown in Figures 21 and 22.

The Bottom T-Lock (C8) was designed to overcome the cost problem of the Channel Splice (C9) and the inconvenience of shear connectors and/or grouting. Both grouting and cleaning are required when grouted sections are moved and reinstalled. By injecting the C8 connections with a special sand blasting "wand", the connection can be effectively "frozen" in place. When the sections are to be moved, the sand simply falls out when the joint is disassembled. Although this design (C8) has not been used in the field, it has been fabricated and used on barriers in Task 5 of this contract. That experience has been favorable. The connection is recommended for experimental use in the field.

The practical consideration of construction barrier reuse on many different projects over extended time periods will mean, in some cases, the initial cost will have been paid on an earlier project, rendering the available barriers the cost effective choice on subsequent projects. The most practical use of the relative evaluations made here may be in the selection of one of more "standard" barriers for a given governmental entity, based on the most dominant characteristics of their construction or rehabilitation projects. The selection of a barrier having low initial costs meeting Service Level A would be cost effective on low speed, low traffic and short duration projects while a Service Level 2A barrier could be selected for more severe traffic conditions on longer duration projects.

IV. BARREL/W-SECTION BARRIERS

Barrier Definition

Of the many barrier designs that have found use in construction and maintenance zones, the one that seems to have followed a reasonably well-defined evolutionary path is the Barrel/W-Section Barrier. Over the past ten years, steel barrels [55 gal (208 l) oil drums] have been put to a wide variety of uses by highway engineers. Relative to uses affecting traffic, the range is from simple delineation through barrel crash cushions. When barrels are effectively painted to achieve high visibility and arranged in lines to delineate the appropriate path of vehicles, depending on their spacing and ballast, they form a barricade of sorts to discourage vehicle entry into an inappropriate zone. The physical effectiveness of this barricade is almost negligible except where barrels are spaced closely and are filled with heavy ballast. In this case, an intruding vehicle will not be redirected by the lines of weighted barrels unless the impact angle is extremely low, but significant deceleration of the vehicle will result.

The next evolutionary step was the addition of a W-section (flex beam) guardrail. It is not known when the step was taken, but it was probably in the early 1970's. This step, which probably seemed so natural to an engineer at the time, since there were available quantities of used guardrail, suddenly converted the barrel delineation system from a barrier with inertia properties only to a barrier with some significant, positive redirection capability. It resulted in stabilization of barrier spacing in some division of 25 ft (7.6 m) which is the standard guardrail length.

The barrier of this type which has a significant automobile redirection potential is the Standard Barrel/W-Section barrier shown in Figure 47. It consists of steel barrels on a 6 ft, 3 in. (1.9 m) spacing with a section of standard steel flex beam (12 gauge) attached directly to the side of the barrels. The top edge of the flex beam is 27 in. (68.6 cm) above the ground. The ballast normally used in the barrels is sand, which produces a total barrel weight of approximately 800 lb (363 kg). Although barriers with larger spacing and lower amounts of ballast are commonly used, either of these changes results in decreased barrier performance.

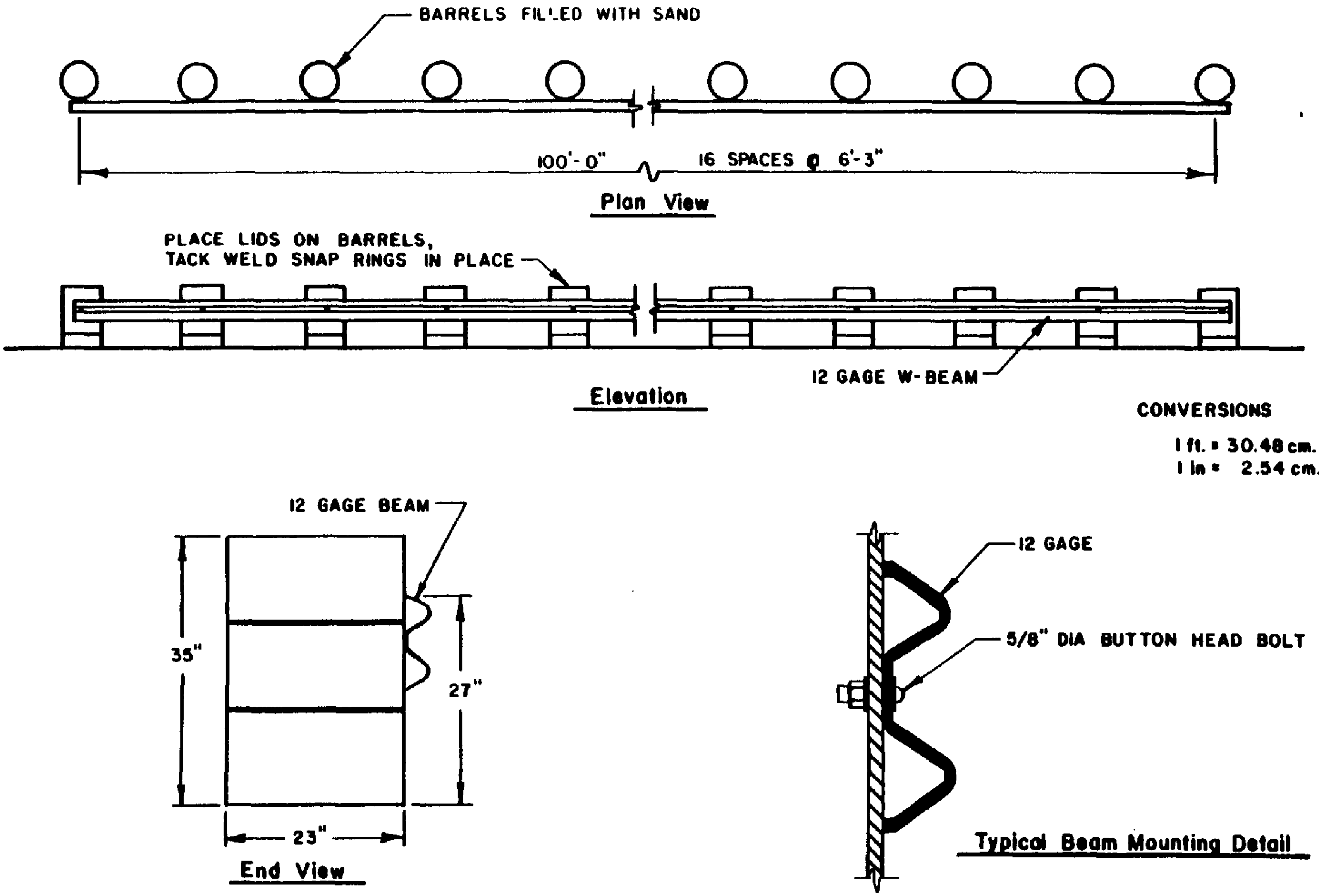


Figure 47. Standard Barrel/W-Section Barrier.

Test Results

Three tests of the Standard Barrel/W-Section Barrier were conducted and reported by Southwest Research Institute in 1977. These tests were described by Bronstad and Kimball (1) in their final report, "Temporary Barriers Used in Construction Zones", December 1977. Principal results and descriptions of these three tests are given in Table 17 which show Tests TB-3 and TB-4 were reasonably acceptable, but Test TB-5 was unacceptable.

In an extrapolation effort to produce maximum information from these three tests, Figure 48 was developed. In this figure, the impact angle is the ordinate and the automobile speed is the abscissa. The three tests are shown by two circles (TB-3 and TB-4) and one square (TB-5). From this the boundary between acceptable and unacceptable performance levels for the standard barrier was developed (4). It is based upon a 4,500 lb (2,041 kg) vehicle striking the barrier under various combinations of impact angle and speed. The performance boundary, designated as the area between curves A and B, must be viewed with some reservation since only the mid segment is reasonably justified by full-scale tests. The outer end of the boundary, in the 50 to 70 mph (80 to 113 km/h) zone, is probably accurate, due to the fact that the basic vehicle barrier interaction is reasonably well defined by the crash tests conducted at an angle of 15 deg. The inner end, between 25 and 35 mph (40 and 56 km/h) is somewhat more questionable, since the high impact angle between 20 and 30 deg could allow an interaction due to pocketing which has not been adequately defined by the previous tests. For this reason, the questionable zones of barrier performance are shown between 10 and 40 (16 and 64 km/h) and between 20 and 30 deg.

This barrier will perform adequately for Service Level A, but not for higher Service Levels. The reasons for the barrel/W-section barrier performance limitations can be summarized as:

- Structural Limitations
- Stability Limitations
- Connection Limitations
- Geometric Limitations

Table 17. Summary of SwRI Test Results. (After Bronstad and Kimball (1))

Test Number	TB-3	TB-4	TB-5
Vehicle	1969 Chevy Impala	1969 Chevy Impala	1975 Ply Gran Fury
Vehicle Weight	4303 lb	4303 lb	4424 lb
Test Speed	35.5 mph	45.4 mph	57.6 mph
Test Angle	14.3°	14.6°	15.8°
Exit Angle	-8.0°	-10.8°	-60°
Vehicle Accelerations (Max. 50 ms avg.)			
Lateral	-1.9 g	-2.7 g	-2.2 g
Longitudinal	-0.6 g	-1.2 g	-3.5 g
Vehicle Rebound Dist.	21 ft	23 ft	3 ft
Maximum Deflection			
Dynamic	1.9 ft	3.4 ft	5 ft*
Permanent	1.9 ft	3.4 ft	30 ft**

* Approximate dynamic deflection of barrier while in contact with vehicle.
** Position of one rail section which was dislodged from the barrier and knocked 30 ft inside the original barrier line.

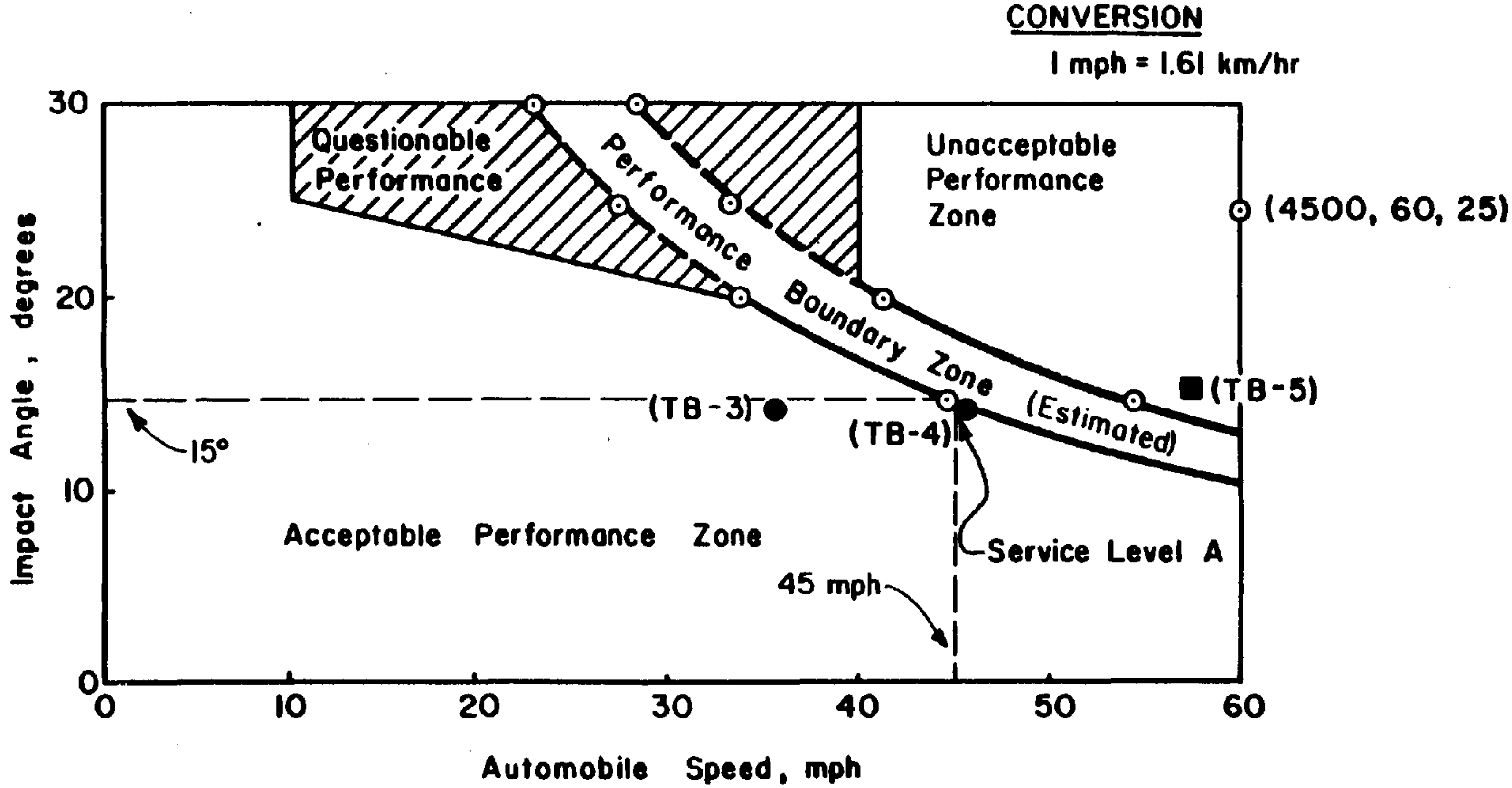


Figure 48. Performance Prediction for Standard Barrel/W-Section Barrier.

Considering those reasons in the order listed, Test TB-5 illustrates the limitation of the W-section bending stiffness. The vehicle severely deforms the W-section resulting in direct contact with the barrels.

This contact with the barrels is further agitated by overturning of the barrels in front of the vehicle allowing a ramping condition which brings elements of the vehicle undercarriage into contact with the lower end of the barrels. This is a problem of geometric stability, resulting in forces so large on individual barrels that they are torn free of the W-section and scattered about the construction zone. During this interaction, connections between W-section elements are also severed. Assuming the W-section to be strong enough to remain intact during a collision, the main problem is that of reducing the contact of the vehicle with the barrels. Obvious solutions seem to be (a) blocking out the W-section and (b) preventing barrel overturning.

Design of Upgraded Barrel/W-Section Barriers

The major elements to be considered in the design of a barrel/W-section barrier for increased performance are the same as those items listed as reasons for limited capacity of the standard barrier. Designs were developed which would accomplish the following:

- Increase the beam stiffness
- Increase overall barrier stability
- Strengthen all connections
- Correct geometric problems

Although numerous new designs were proposed during sessions of staff engineers, all but two of the designs were discarded for reasons ranging from low probability of performance to excess complexity. The two designs that were finally accepted for further analysis and possible testing are shown in Figures 49 and 50. They are designated Stabilized Barrel/W-Section No's 1 and 2. They will be called SBW1 and SBW2, respectively.

SBW1 is the barrier which will be demonstrated to have the highest performance potential. It is shown by Figure 49 to have four major changes from the standard system:

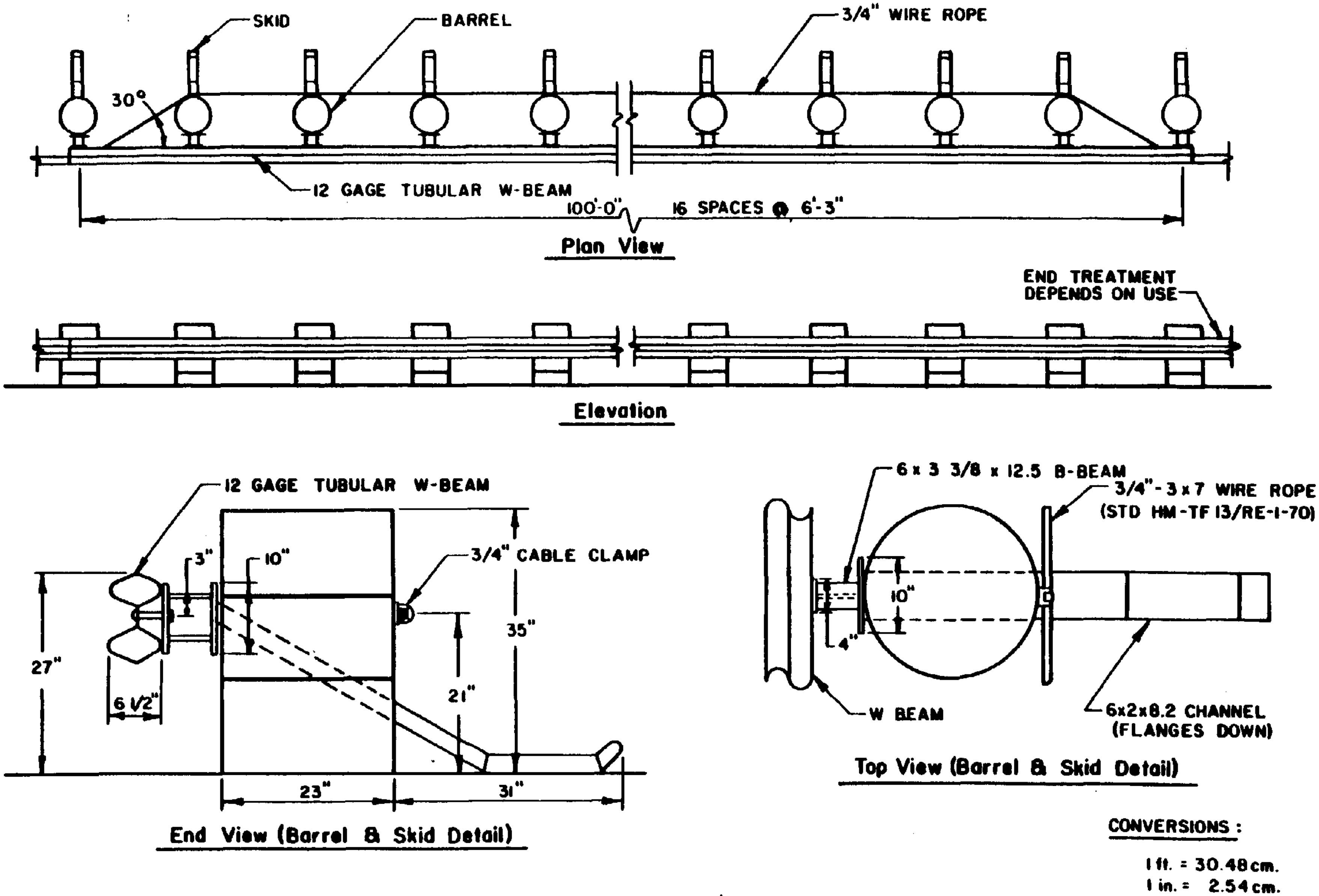


Figure 49. Stabilized Barrel/W-Section Barrier (SBW1).

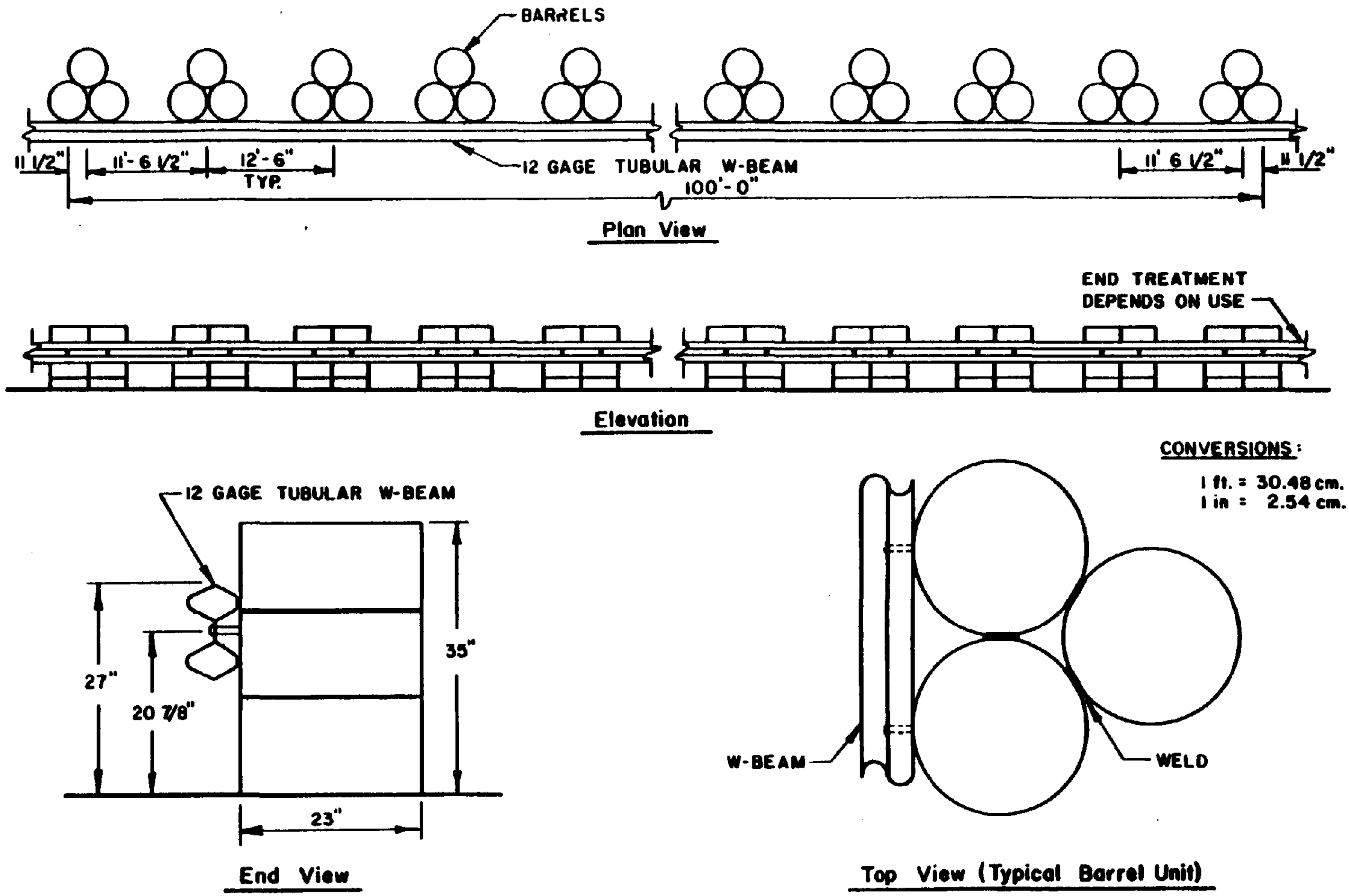


Figure 50. Stabilized Barrel/W-Section Barrier (SBW2).

- Use of the double, or closed W-section beam
- Addition of a 3/4 in. (1.9 cm) wire rope on the barrel side away from the impact plane
- A 6 in. (15.2 cm) block-out from the supporting barrels
- Use of a skid channel extending from the beam through the barrel to a point of support 40 in. (101.6 cm) behind the impact plane

SBW2 is shown in Figure 50. There are three major design changes from the standard barrier:

- Use of the double, or closed W-section beam [This also affects a 3 1/4 in. (8.3 cm) block-out compared to the standard barrier.]
- Grouping the barrels in sets of three
- Changing the distance between the centroid of barrel groups to 12.5 ft (3.8 m)

Each of the design changes for SBW1 and SBW2 is responsive to a specific limitation of the standard system with the exception of the final item under SBW2 which was required for practical reasons.

The two designs were submitted to FHWA Contract Managers and to certain other interested engineers, including Mr. Dexter Jones of the Texas State Department of Highways and Public Transportation. Mr. Jones reviewed these designs critically stating SBW1 was too complicated to construct and suggesting several changes. These suggestions were used to develop the design SBW3, shown in Figure 51.

The SBW3 design is very similar to the SBW1 design and incorporates three major changes from the standard system:

- Use of double, or closed W-section
- A 6-in. (15.3 cm) blockout from the support barrels
- Use of a skid plate welded to the base of the barrel

This design was developed to achieve the structural advantages of the SBW1 barrier, and reduce the complexity. The following analysis of the structural characteristics of all the designs will show its similarity.

Analysis of Proposed Systems

An approach, which may be termed "comparative structural analysis",

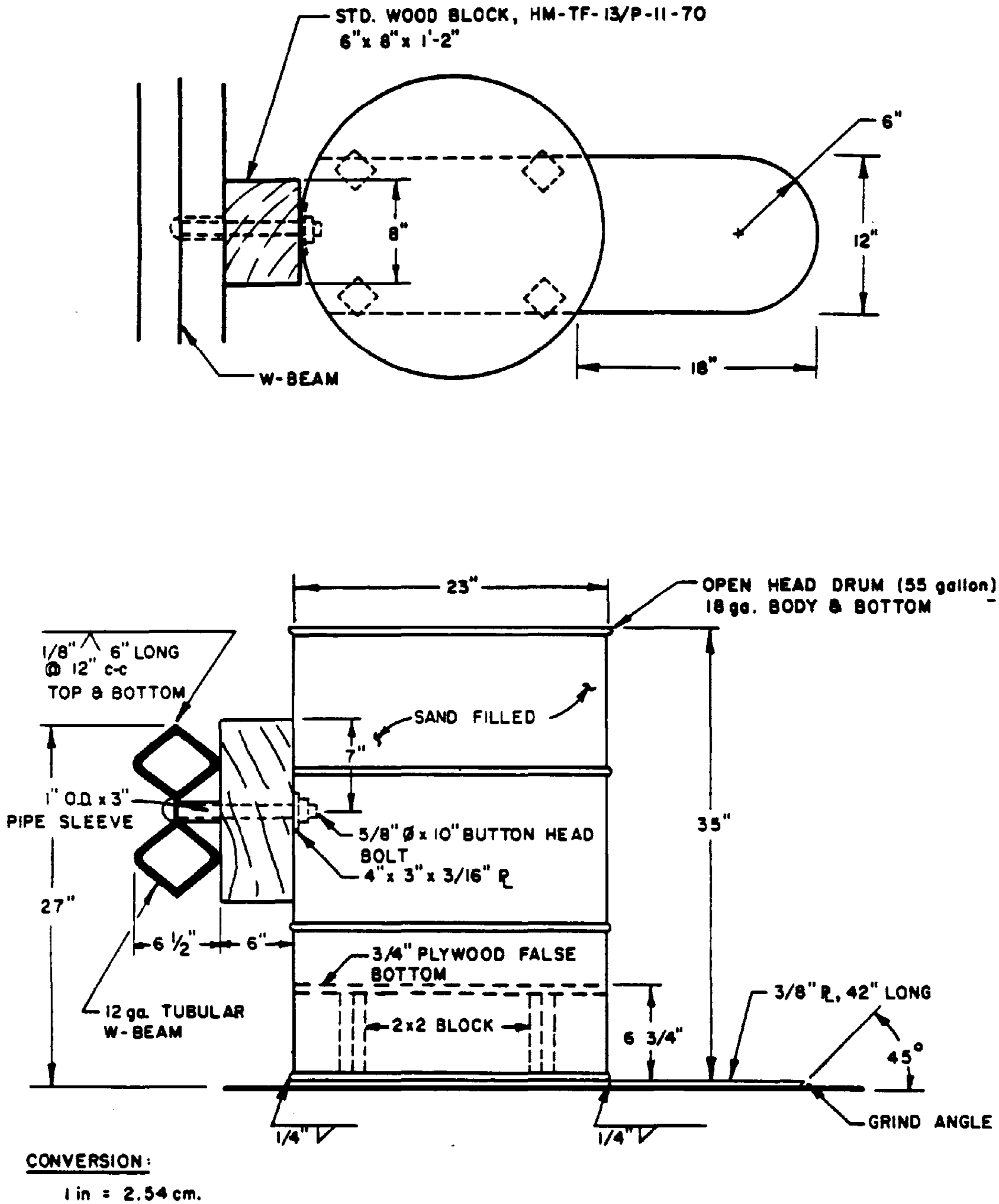


Figure 51. Stabilized Barrel/W-Section Barrier (SBW3).

was used to analyze the barrier systems. Comparative structural analysis requires the listing and/or development of a number of performance factors for new design which can be compared to the same factors in designs of known performance. For example, if it is known that the standard barrier performs reasonably well up to a certain level with a beam stiffness of BS_1, and if beam stiffness is one of the factors limiting the performance level of the standard barrier, it may be assumed that raising beam stiffness to the level BS_2 will have a positive effect on the performance of a new barrier. "Comparative structural analysis" is not new, except perhaps in name or in formal organization. It has often been informally practiced in the field of collision dynamics engineering and has resulted in some major design improvements.

The comparative factors developed here can be shown by theory and by analysis of test results to bear significantly on barrier performance (3). The factors developed are defined as follows:

Mass Mobilization Factor (MM) (lb) The average weight of the barrier in lb/10 ft (kg/3.0 m) of length.

Beam Stiffness Factor (BS) (in.) The moment of inertia of the beam cross section about the axis of major bending divided by the cube of the unsupported beam length between major attachment points.

$$BS = \frac{I_y}{L^3}$$

Torsional Stiffness Factor (TS) ($in.^3$) The equivalent polar moment of inertia (as defined for the determination of torsional rotation in response to an applied torque) divided by the unsupported beam length between attachment points.

$$TS = \frac{J\ eq.}{L}$$

Unit Stability Factor (US) (lb) The maximum force that can be applied at the automobile impact level to a 10 ft (3.0 m) length of barrier without creating a rotational barrier acceleration. (To be used only on systems not rigidly attached at the base.)

Unit Attachment Factor (UA) (lb) The maximum force that the attachment of the barrier to the pavement or ground surface generates in term of lb/10 ft (kg/3.0 m) of barrier. This includes friction forces and the lateral forces generated by adjacent pavement layers as well as the strength of positive attachments such as dowels, bolts, footings, and the like.

Each of these five factors has been calculated for the three new barrel/W-section barrier. Properties of the barrier systems and values of the factors for each are listed in Table 18.

Comparisons of these factors show, in general, relatively high values for the three new designs. The Mass Mobilization Factor (MM) increases to 1,530 lb/10 ft (694 kg/3.0 m) for SBW1 and SBW3, and 2,080 lb/10 ft (943 kg/3.0 m) for SBW2. These increases in barrier mass should result in lower barrier deflections.

The Beam Stiffness Factor (BS) increases radically for SBW1 and SBW3. The BS is calculated as I_y/L^3 where I_y was increased from 2.3 in.^{4} to 16.4 in.^{4} (96.5 cm^4 to 683.5 cm^4) due to the use of the double W-section beam. The length (L) remains constant at 6 ft, 3 in. (1.9 cm). BS increased only moderately for SBW2. Although the value of I_y is increased to 16.4 in.^{4} (683.5 cm^4) as in SBW3, the clear span of the beam in SBW2 is increased to 10.6 ft (3.2 m). The decrease in BS caused by increased length tends to offset the increase due to a larger I_y.

The Torsional Stiffness Factor (TS) is most important to barrier stability. It is the lack of torsional stiffness that allows the first few barrels to be overturned while other barrels remain upright and the connecting single W-sections are relatively unstressed. Although TS is calculated by dividing the equivalent section polar moment of inertia (J eq.) by the clear span between barrel supports, the major contribution is from the equivalent polar moment of inertia. The J eq. value for the closed double W-section is 4,790 times as large as J eq. for the open single section. Calculations indicate the torque necessary to produce a yielding shear stress on the closed section is 22.4 kip-ft (30.4 kN-m) at a rotation in 6 ft, 3 in. (1.9 m) of 2.8 deg, compared to a torque on the

Table 18. Structural Properties and Factors Indicating Barrier Performance.

Design	Standard Beam/W-Section Barrier	Stabilized Beam/W-Section SBW 1	Stabilized Beam/W-Section SBW 2	Stabilized Beam/W-Section SBW 3
PROPERTIES				
Beam				
Area (in.$^{-2}$)	1.99	3.98 (4.18)**	3.98	3.98
I_y (in. 4)	2.31	16.42 (245)**	16.42	16.42
I_x (in. 4)	30.0	60.0	60.0	60.0
$J_{eq.}$ (in. 4)	7.33 x 10^{-3}	34.38 (N.C.)**	34.38	34.38
L*	6.25	6.25	10.58	6.25
Barrel Spacing (ft)	1 @ 6.25	1 @ 6.25	3 @ 12.5	1 @ 6.25
Full Barrel Weight (lb)	800	800	800	800
FACTORS				
Mass Mobilization (MM), lb/10 ft	1360	1530	2080	1530
Beam Stiffness (BS), in.	5.47 x 10^{-6}	39.9 x 10^{-6} (580 x 10^{-6})**	8.04 x 10^{-6}	39.9 x 10^{-6} (580 x (10.6)**
Torsional Stiffness (TS), in.^{3}	0.098 x 10^{-3}	0.46 (N.C.)**	0.27	0.46 (N.C.)**
Unit Stablility (US), lb/10 ft	1070 (1150)***	13,570 (39,170)***	9590 (35,190)***	8850 (34,450)***
Unit Attachment (UA), lb/10 ft	680	770	1040	770

* Unsupported beam length

** The larger values are appropriate wherever the barrier is operating in the positive moment condition (i.e. the cable is in tension).

*** Including the torque generated by adjacent beam sections.

N.C. Not calculated but slightly larger.

open section of 0.13 kip-ft (0.18 kN-m) at a rotation in 6 ft, 3 in. (1.9 m) of 132 deg. This great stiffness increase in the torsion mode mobilizes much more of the barrier to resist overturning.

Probably the single most important factor indicating relative barrier performance is Unit Stability (US). This factor, based on the analysis of the structure shown in Figure 52, is a value of force, "F", that can be applied to a 10 ft (3.0 m) section of barrier without producing an angular acceleration (i.e. movement leading to overturning). It can be shown that this force is defined by the barrier weight and barrier dimensions as:

$$F = W \frac{(r - \mu y)}{(a - y)} \qquad (y < a)$$

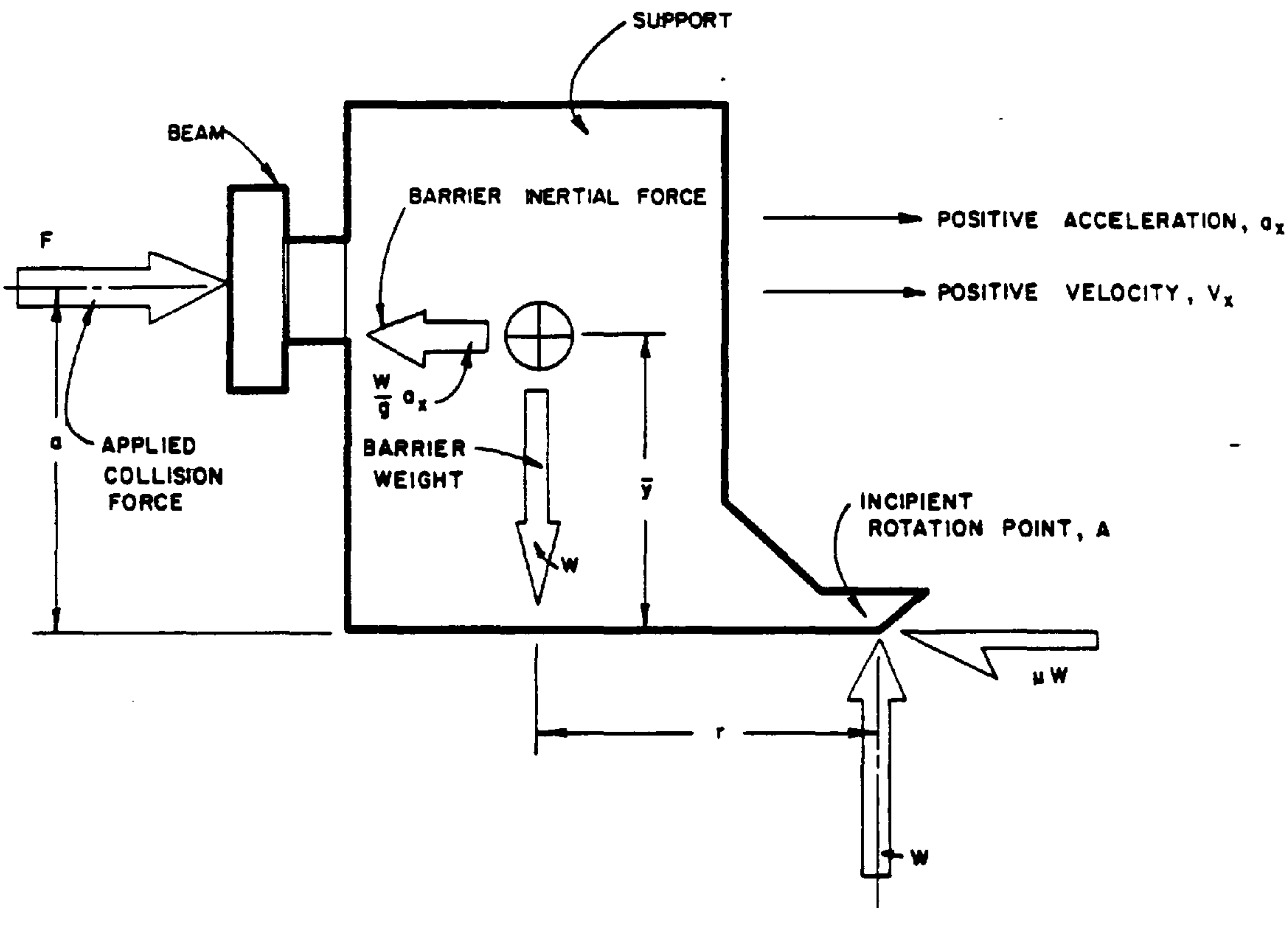
or

$$F = \frac{-W\ (r + \mu y)}{(a - y)} \qquad (y > a)$$

where:

- F is the lateral force applied by an impacting vehicle,
- r is the horizontal distance from the barrier c.g. to the required rotation point for overturning,
- a is the height of the force, F; and
- y is the vertical distance from the ground surface to the barrier c.g.

This equation is the result of eliminating a_x in the equations developed from a summation of moments about the point of incipient rotation, A, and a summation of forces in the x direction (Equations 1 and 2 of Figure 52). The force so derived is directly proportional to the weight of the barrier, a nonlinear function of the dimensions a, r, and y, and the coefficient of friction. The equation must not be taken literally for all imaginable values of a_x. For example, this equation implies that as "a" approaches y, F approaches ∞. Consideration of Equations 3 and 4 (Figure 52) indicates this is theoretically true as long as the coefficient of friction, μ, is less than the static overturning ratio of r/a. It is practically impossible, however, since consideration of Equation 1 indicates that a_x must approach ∞ in order for F to approach ∞. It is emphasized that the optimum position of the center of gravity of an inertially responding and sliding barrier of this type is on the same vertical level as the applied force position. Figure 53 illustrates this



$$\Sigma M_A = 0$$

$$(1) \quad Fa = \frac{W}{g} a_x \bar{y} + Wr$$

$$\Sigma F_H = 0$$

$$(2) \quad F = \frac{W}{g} a_x + \mu W$$

$$\Sigma M_A = 0 \quad (a_x = 0)$$

$$(3) \quad Fa = Wr$$

$$\Sigma F_H = 0 \quad (a_x = 0)$$

$$(4) \quad F = \mu W$$

Figure 52. Free Body Diagram and Equations Defining the Unit Stability Factor.

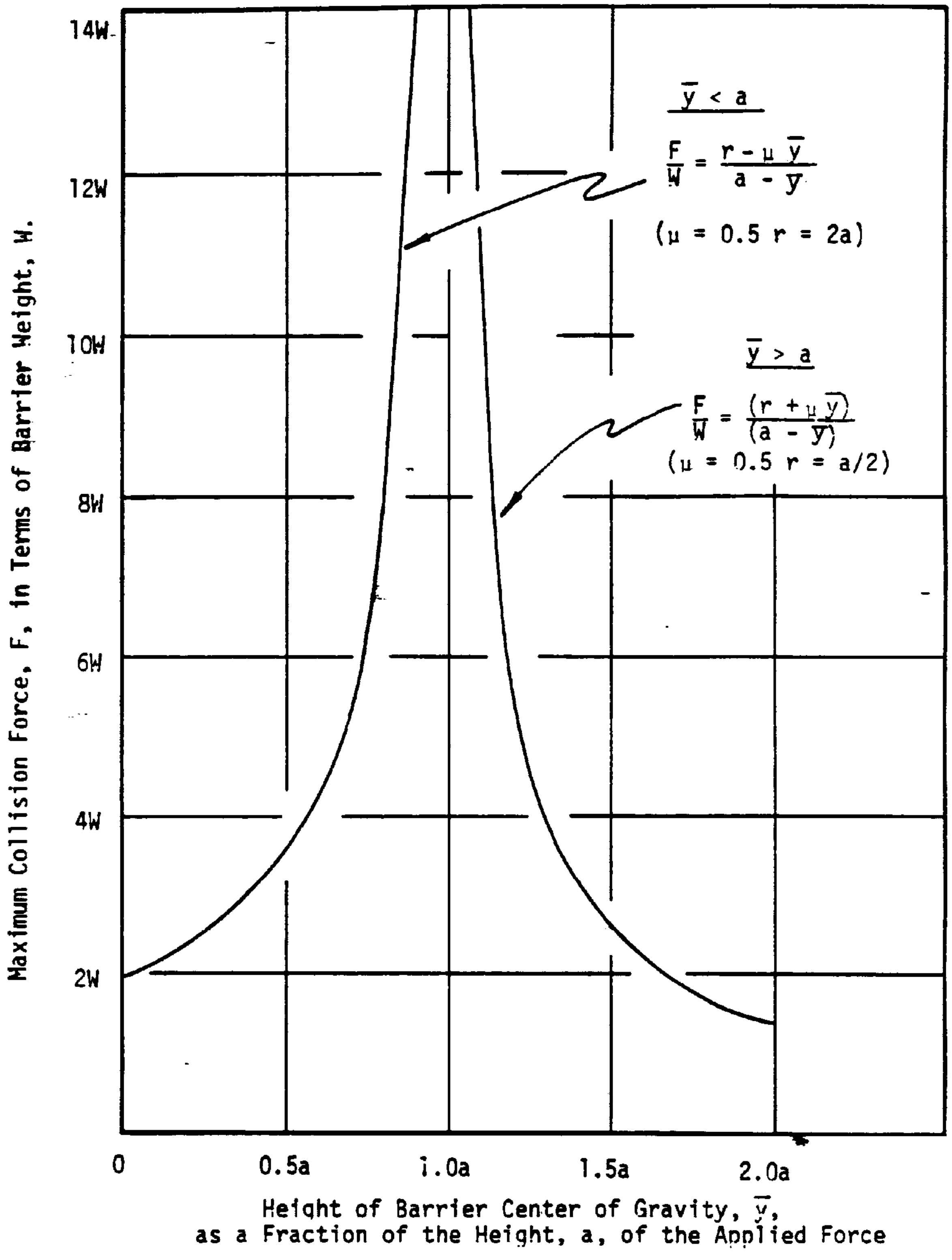


Figure 53. Relationship Between Maximum Collision Force and Height of Barrier Center of Gravity.

fact but does not show the applied force level above a practically achievable level.

The applicability of Equations 1 and 2 for any value of a_x does depend on μ being less than the r/a ratio. If μ is greater than r/a (see Equations 3 and 4) the barrier will tip over before it starts to move laterally (i.e. under zero lateral velocity conditions). This is the reason it is of fundamental importance to performance that the barrels skid on the surface rather than dig in.

The numbers in parenthesis in the Unit Stability row of Table 18 include the basic Unit Stability number to which is added a value of force, 2F'. This force is the value necessary to place adjacent segments of rail into a yield condition in torsion. As an example, adjacent beam segments of SBW1 and SBW3 are double closed W-beam sections 6 ft, 3 in. (1.9 m) in length. This beam can accept a torque of 22.4 kip-ft (30.4 kN-m) before the material yields in shear, when a total rotation of one end with respect to the other is 2.8 deg. By dividing this moment by the dimension a, the force necessary to produce this torque is calculated. The result is a hybrid Stability Factor which, to some degree, accounts for the tremendous increase in torsional stiffness of all the barriers.

The Unit Attachment Factor will be of significance for barriers which are mechanically attached to support media, but is only a reflection of the Mass Mobilization Factor in the case of a barrier subject only to friction acting at the base. In this case, the factors calculated are simply the MM value multiplied by the coefficient of friction, assumed to be 0.5.

<u>Test of Stabilized Barrel/W-Section No. 3 (SBW3)</u>

Based on the analyses of SBW1 and SBW2 and the comparable characteristics of SBW3, a decision was made to test the relatively simple SBW3. These tests were designated 3825-1 through 3825-4 and conducted on the installation shown in Figures 51 and 54. The test installation was placed on unpaved level soil similar to those found in construction zones. It was 250 ft (76.2 m) long, including a 25-ft (7.6 m) end treatment as shown in Figure 54. The details of each test and subsequent results are presented in Table 19.

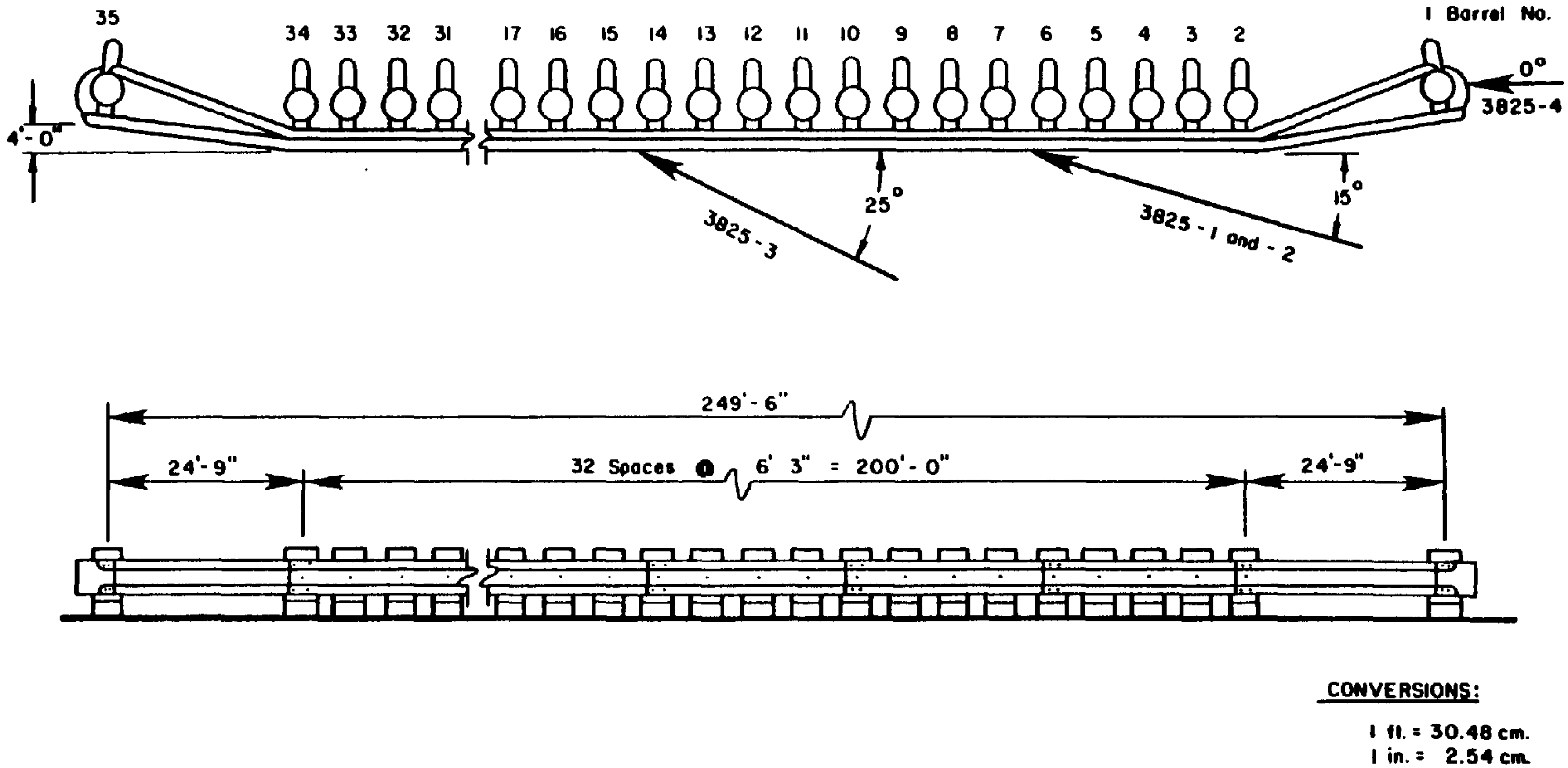


Figure 54. Barrel/W-Section (Design SBW3) Test Installation.

Table 19. Summary of SBW3 Test Results.

Test Number	3825-1	3825-2	3825-3	3825-4
Vehicle	1975 Plymouth Fury	1975 Plymouth Fury	1974 Fury II	1975 Plymouth
Vehicle Weight	4,500 lb (2,041 kg)	4,500 lb (2,041 kg)	4,500 lb (2,041 kg)	4,500 lb (2,041 kg)
Test Speed	44.3 mph (71.4 km/h)	61.7 mph (99.4 km/h)	62.4 mph (100.4 km/h)	61.4 mph (98.8 km/h)
Test Angle	15 deg	15.5 deg	22.5 deg	0 deg*
Exit Speed	33.3 mph (53.6 km/h)	51.9 mph (83.5 km/h)	45.4 mph (73.0 km/h)	NA
Exit Angle	3.5 deg	12.3 deg	18 deg	NA
Vehicle Accelerations (Max. 50 msec avg.)				
Transverse	4.0 g	4.6 g	5.4 g	-3.1 g
Longitudinal	-1.4 g	-2.0 g	-1.4 g	15.8 g
Maximum Deflection				
Dynamic	2.1 ft (0.6 m)	5.4 ft (1.6 m)	11.0 ft (3.4 m)	NA
Permanent	1.8 ft (0.5 m)	5.0 ft (1.5 m)	10.7 ft (3.3 m)	NA
Vehicle Damage				
TAD	1-RFQ-1	1-RFQ-2	1-RFQ-3	12-FD-3
SAE	01RFEW1	01RFEW2	01RFEW2	12FDEW2

* Impact parallel to the barrier at the end terminal.
NA Not applicable.

Test 1 - 4,500 lb/44.3 mph/15 deg (2,041 kg/71.3 km/h/15 deg)

A 1975 Plymouth Grand Fury weighing 4,500 lb (2,041 kg) including instrumentation was used in this test. Initial impact occurred 1.5 ft (0.5 m) downstream from barrel 6. The rear of the car contacted the rail near the point of initial impact. Contact with the barrier was maintained through barrel 14. The car was exceptionally stable during redirection and left the rail at a 3.5 deg exit angle. The maximum dynamic rail deflection was 2.1 ft (0.6 m). The rail rebounded 0.3 ft (0.1 m) leaving a 1.8 ft (0.6 m) deflection after collision.

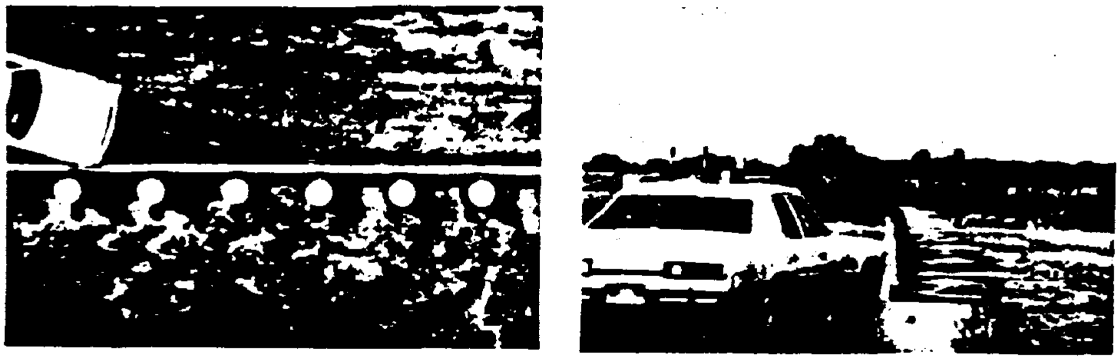
Sequential photographs are shown in Figure 55. The maximum 0.050 sec average transverse acceleration was 4 g, which is within the acceptable 5 g limit given in TRC 191 (11). The lateral acceleration when the vehicle motion became parallel to the barrier was only 1.3 g. The longitudinal 0.050 sec average was a modest -1.4 g. Damage to both vehicle and barrier was negligible. The same vehicle was used to conduct Test 2.

Restoration After Test 1

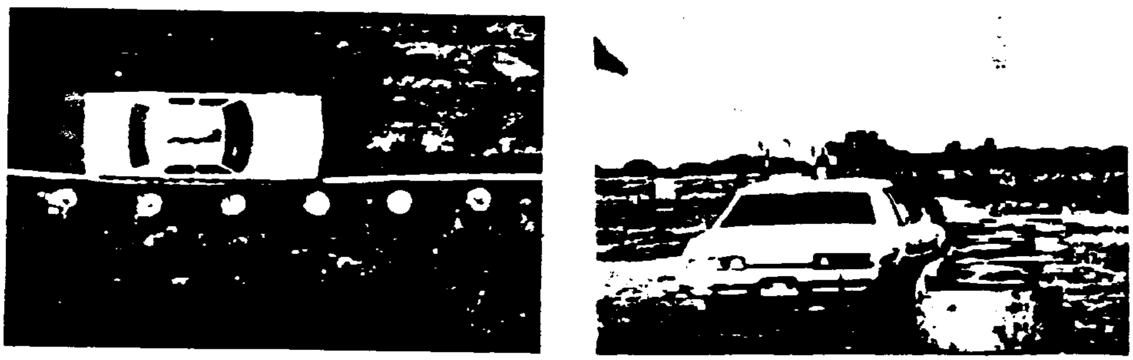
The barrier was pushed to its original position in 30 minutes by two men with a forklift. The extent of the permanent deformation was isolated to one 25 ft (7.6 m) rail segment. The rail segment was between barrels 6 and 10 with a 1/2 in. (1.3 cm) permanent set. The damage to the rail segment was so slight that replacement was not considered necessary. Four barrels in the immediate area of impact were deformed slightly adjacent to the wooden block. The barrels were not replaced because the deformations would not affect performance.

Test 2 - 4,500 lb/61.7 mph/15.5 deg (2,041 kg/99.3 km/h/15.5 deg)

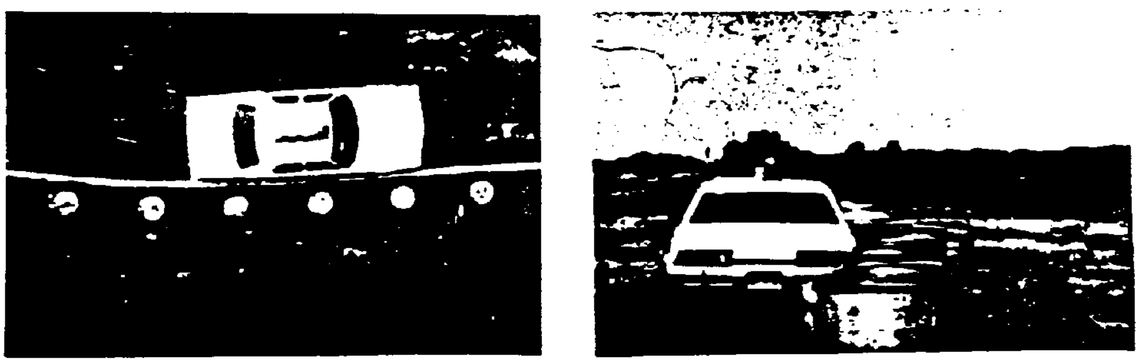
In Test 2, a 1975 Plymouth Grand Fury weighing 4,500 lb (2,041 kg) including telemetry equipment, impacted the barrier at 15.5 deg and 61.7 mph (99.3 km/h). Sequential photographs are presented in Figure 56. The vehicle remained quite stable during redirection, exhibiting no tendency to mount the rail. The vehicle exited the barrier at an angle of 12.3 deg and a speed of 51.9 mph (83.5 km/h). The maximum 0.050 sec average transverse acceleration was 4.6 g. This compares favorably with the 5 g acceptable limit from TRC 191 (11). The longitudinal acceleration was -2.0 g, well within the 5 g preferred limit. The maximum rail deflection



0.000 sec



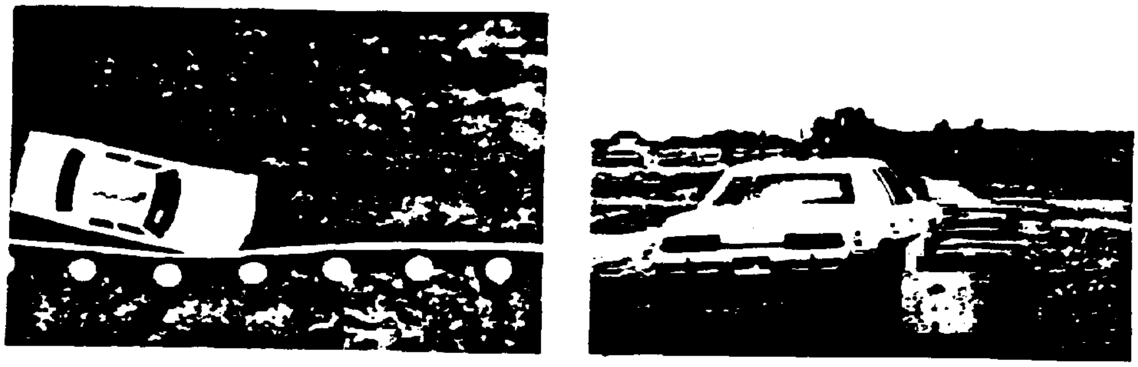
0.390 sec



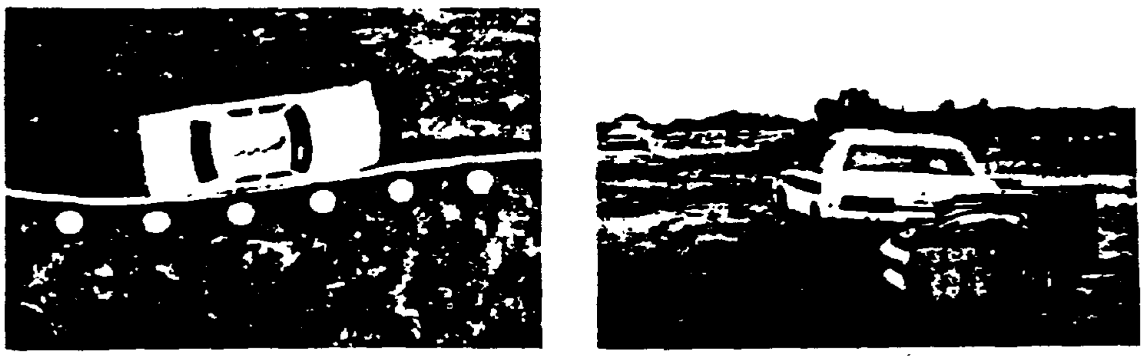
0.624 sec

0.780 sec

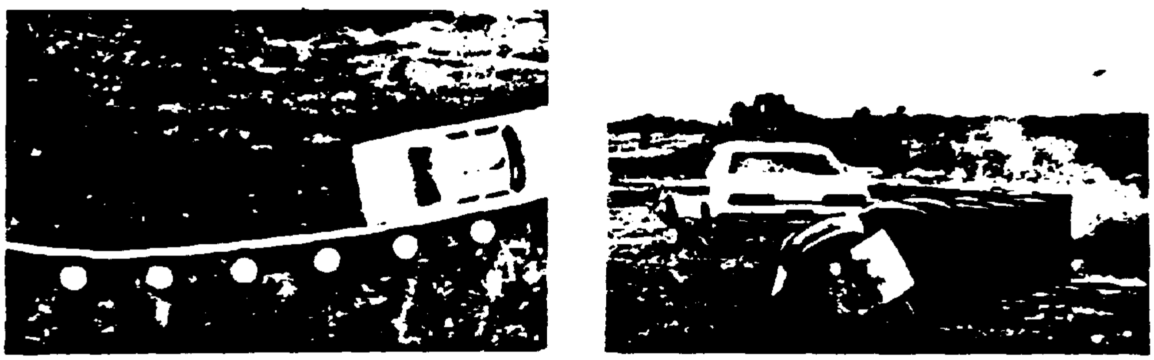
Figure 55. Sequential Photographs for Test 3825-1.



0.104 sec



0.312 sec



0.519 sec

0.696 sec

Figure 56. Sequential Photographs for Test 3825-2.

was 5.4 ft (1.6 m) but the vehicle only penetrated into the protected zone 4.7 ft (1.4 m). Photographs of the vehicle before and after Test 2 are shown in Figure 57.

Restoration After Test 2

Two men with a forklift were required to push the barrier back to its original position. Restoration was completed within 60 minutes. Significant permanent deformation was confined to the 25 ft (7.6 m) rail section between barrels 6 and 10. The maximum permanent set was 3.9 in. (9.9 cm) located 2.0 ft (0.6 m) downstream from barrel 7. This rail section and barrels 6 through 8 were replaced.

Test 3 - 4,500 lb/62.4 mph/22.5 deg (2,041 kg/100.4 km/h/22.5 deg)

The test vehicle, a 1974 Plymouth Fury, impacted the barrier at 22.5 deg with a velocity of 62.4 mph (100.4 km/h). The vehicle weighed a total of 4,500 lb (2,041 kg) including the telemetry equipment. Photographs of the vehicle and the barrier before and after the test are shown in Figures 58 and 59, respectively.

Point of impact occurred 3 ft (0.9 m) downstream of barrel 14. At approximately 0.210 sec, the vehicle swung into the rail 2.5 ft (0.8 m) downstream of barrel 15. By 0.236 sec, the upstream barrels were beginning to rotate. By 0.641 sec, the first of the upstream barrels fell over and succeeding downstream barrels began to fall. But in the vicinity of the vehicle, the barrels remained upright and resisting throughout the test. The vehicle exited the rail at an angle of 18 deg and a velocity of 45.5 mph (73.2 km/h). The maximum dynamic deflection of the barrier was 11 ft (3.4 m); this returned to 10.7 ft (3.3 m) after the test.

Restoration After Test 3

The barrier was returned to its original position by three men with two forklifts in 90 minutes. The extent of the permanent deformation after repositioning was between barrels 13 to 20. The maximum deformation occurred at barrel 16 of magnitude 5.7 in. (14.5 cm). The restored barrier is shown in Figure 60. The 25 ft (7.6 m) rail section between barrels 13 and 17 was replaced along with barrels 14 through 18 before testing continued.

Figure 57. Vehicle Before and After Test 3825-2.

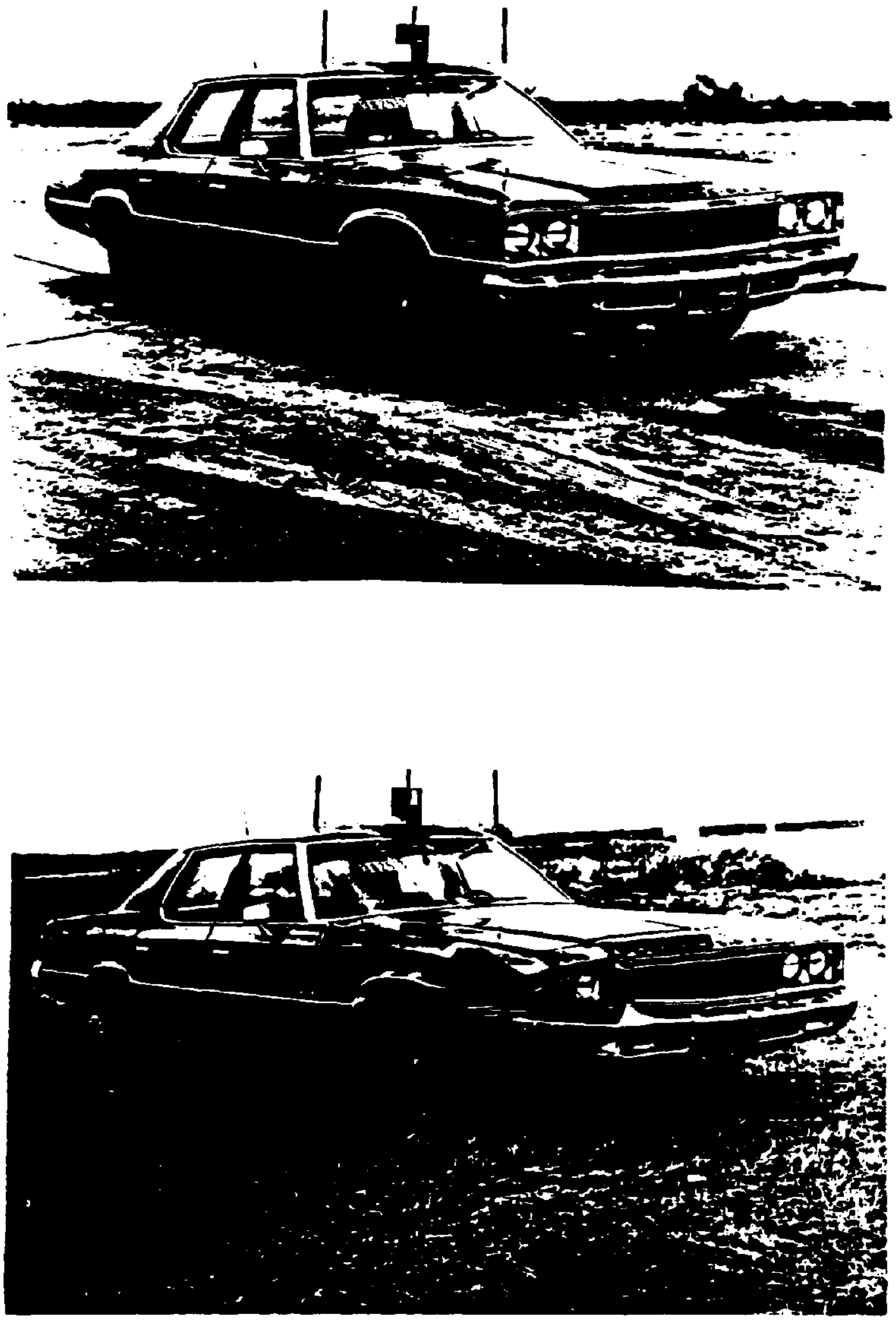


Figure 58. Vehicle Before and After Test 3825-3.

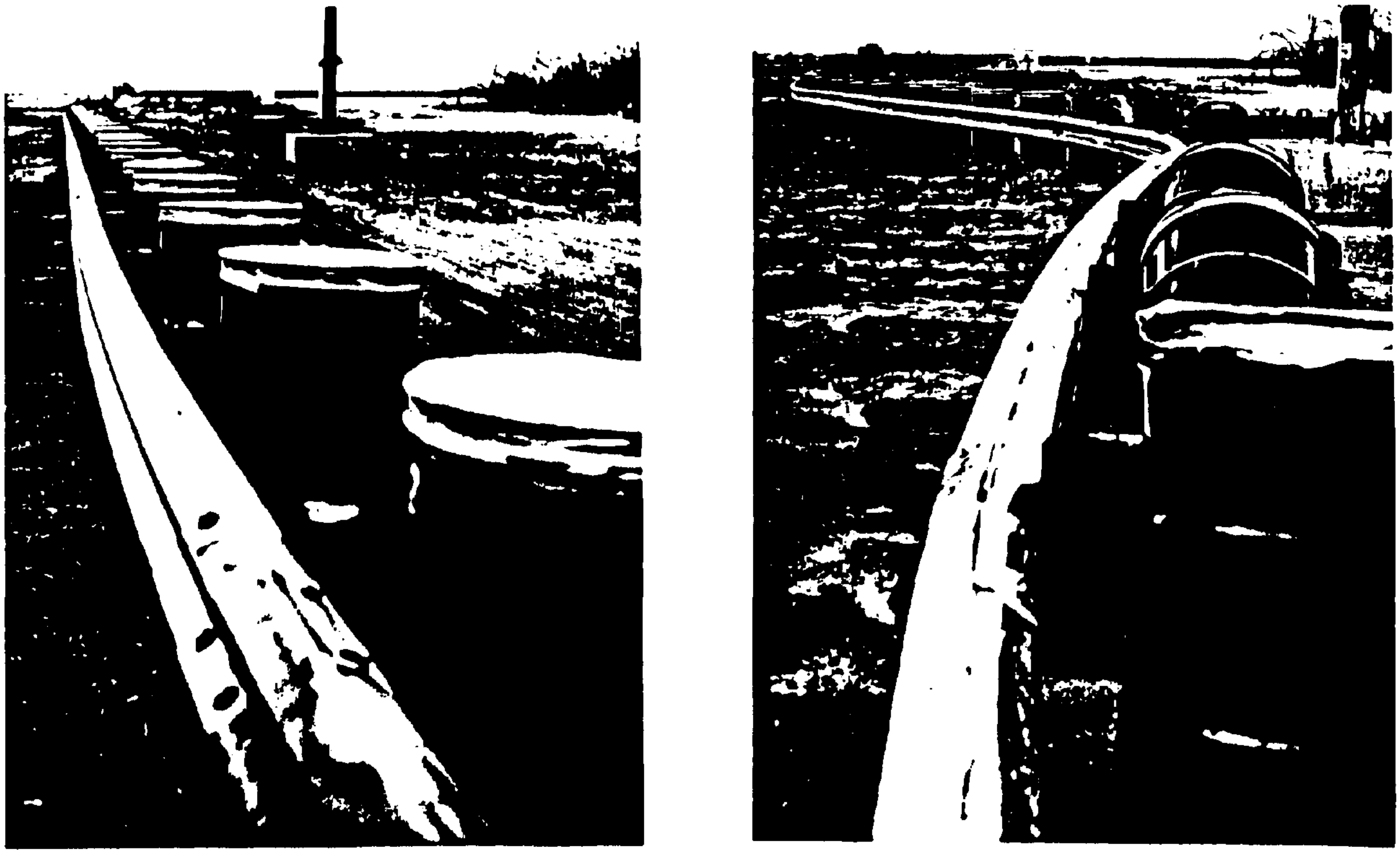


Figure 59. SBW3 Installation Before and After Test 3825-3.

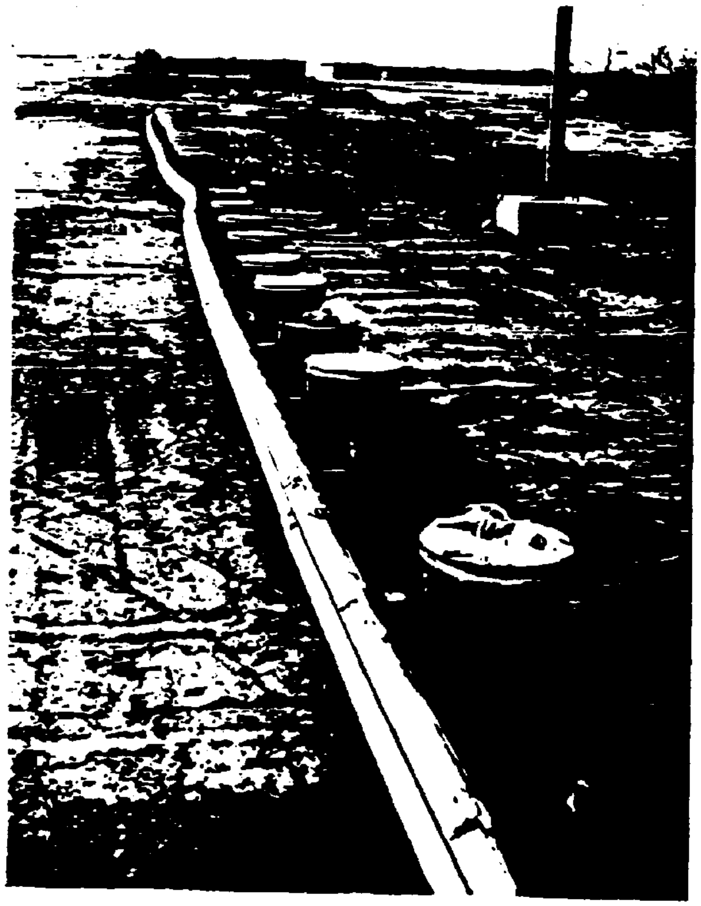


Figure 60. SBW3 Installation Restored After Test 3825-3.

Test 4 - End Treatment - 4,500 lb/61.4 mph/0 deg
(2,041 kg/98.8 km/h/0deg)

In Test 4, a 1974 Plymouth Grand Fury impacted the terminal of the barrier at 0 deg and 61.4 mph (98.8 km/h). The vehicle weighed 4,500 lb (2,041 kg) including telemetry equipment. Sequential photographs are presented in Figure 61.

As shown in the sequential photographs impact occurred at the end of the terminal. The W-beam began to buckle upstream of barrel 2, and folded inward toward the back side of the barrier causing the vehicle to ride up and over it. Outward buckling occurred at barrels 2 and 3. The vehicle yawed to the left and came to rest behind the barrier. Damage to the front of the vehicle was extensive.

The longitudinal acceleration, 15.8 g (0.050 sec average) was high and would have been much higher for a small vehicle. The writers, therefore, propose to reduce the sand ballast in the end barrel to roughly 200 lb (90.7 kg) and to elevate the c.g. of this sand to prevent the vehicle from ramping on the end barrel.

Restoration After Test 4

Although the barrier was not repaired following Test 4, the barrier was severely damaged upstream of barrel 3. Repairs that would have been required to restore the barrier included the replacement of the first two sections of W-beam and the first eight barrels.

Conclusion

The "comparative structural analysis" technique indicated the high probability that barriers SBW1, SBW2, and SBW3 would perform Service Levels higher than Service Level A as achieved by the standard barrel/W-section barrier.

This statement has been verified by the first three tests of SBW3. The performance of this design is excellent, achieving Service Level 2A with one major drawback: the relatively large barrier deflection. The barrier is not highly portable and should be considered for use only when needed at one point for a considerable period of time. Unless surplus barrels and the W-section are available, the cost is comparable to conventional portable concrete median barriers.

0.000 sec

0.203 sec

0.520 sec

0.749 sec

Figure 61. Sequential Photographs for Test 3825-4.

The writers recommend the use of this barrier design where cost factors warrant and where deflections during anticipated vehicle collisions can be accommodated.

V. END TREATMENT FOR PORTABLE CONSTRUCTION BARRIER

Portable construction barriers constitute a major safety improvement when used properly in the field. There appears to be a better understanding of the need for these barriers to prevent lateral intrusion into specified zones, however, than the understanding of the hazard a specific barrier may constitute if the barrier end is not carefully accommodated. Emphasis needs to be placed on good methods of accommodating barrier ends. A variety of treatments have been used, some treatments have recently been evaluated by crash testing and others are still under development.

These treatments range from simply flaring the end away from traffic to sophisticated and costly, crash cushions. A variety of treatments, along with a discussion of the strengths, weaknesses, and applicability of each will be presented.

The simplest treatment, and one which is often used in conjunction with other types of end treatments is a simple flaring of the barrier away from traffic on the upstream end. A flare that alone is reasonably safe requires considerable space, space not often available at construction sites. Lateral space, up to as much as 30 ft (9.1 m) to remove the unprotected end of the barrier to a safe position, is required along with longitudinal space, so the angle the flaring barrier makes with probable automobile trajectory is not too acute. Figure 62 shows a flared end treatment where the end is not sufficiently removed from the traveled way. A significant hazard may be constituted by the barrier end.

Where space is not available, certain combinations of modest flares and end treatments can be used. One of the least costly and reasonably effective treatments is shown in Figure 63. Here a slight flare allows the barrier end to be tucked behind a small earth berm. Although this prevents the extremely hazardous barrier end collision, the treatment has some disadvantages unless traffic speed is very low. An impacting vehicle at other than low speeds might ramp and go over the berm and barrier end, allowing penetration into the construction zone and producing a potential for vehicle roll.

Another treatment that has much the same limitations as the earth berm is the wedge-shaped end segment. One of the better designs of the

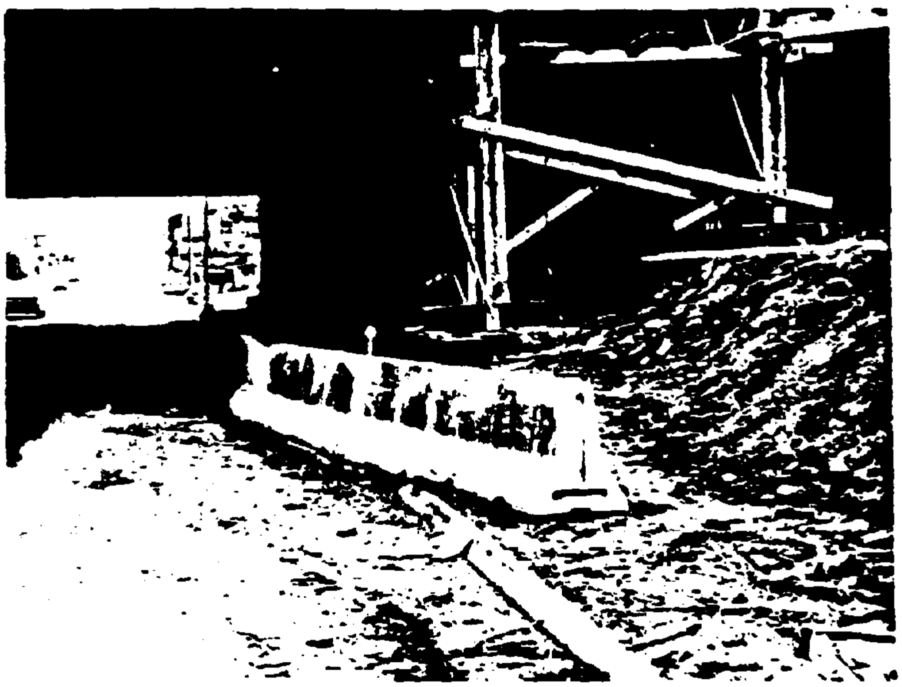


Figure 62. Inappropriate Flared End Treatment.

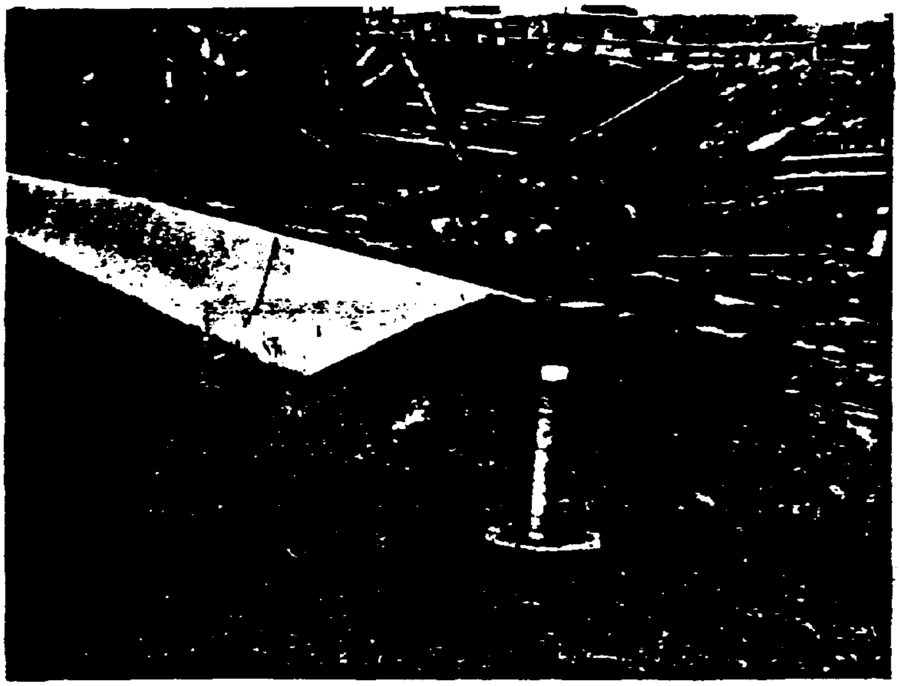


Figure 63. Earth Berm End Treatment.

Figure 64. Wedge End Treatment.

wedge end is shown in Figure 64. This does prevent the precipitous stop that would occur if a vehicle contacted a full height end segment, but would allow ramping and probable rolling at certain combinations of vehicle trajectory and speed, just as was produced under certain impact conditions by the turned down guardrail before it was modified to allow collapse. There are several other wedge designs in use which are shorter than the one shown in Figure 64. That is, the slope of the wedge is much more acute. These designs probably have less favorable performance characteristics as the angle the segment top makes with the horizontal becomes larger.

A good approach to eliminating the hazard of an end is to make the end tie into or be contiguous with an existing longitudinal barrier. If a positive connection is provided that will prevent a vehicle from snagging at the connection point, an excellent end treatment should result. Compared to many end treatments, Figure 65 shows an end protection that is probably reasonable considering the lateral distance from traffic. It would not function well, however, if a vehicle impacted the guardrail at a fairly high angle just in advance of the PCB end. Since a positive connection between the PCB and the guardrail has not been provided, the guardrail would deflect away from the PCB end and allow a fairly direct contact of the vehicle with that end.

An extremely positive treatment of a construction barrier end is provided by the use of a crash cushion. There are several types available including the GREAT, the Barrel Crash Cushion, and the Fitch Inertia Barrier. Examples of these treatments are shown by Figures 66, 67, and 68. Of these, the GREAT system has the best overall performance since it is specifically designed for this application and provides redirection capability at the point it joins the PCB end, a vulnerable zone when using some barrel cushion designs and the Fitch barrier. It is also well adapted to installations in very narrow areas.

There have been numerous applications of the Barrel Crash Cushion at the ends of PCB's. Most have been well designed. There have been some, however, where a positive structural connection has not been made between the rear of the barrel cluster and the end of the PCB. In this situation it would be possible for the crash cushion to rotate under an eccentric collision and allow the vehicle to strike the PCB end, or continue into

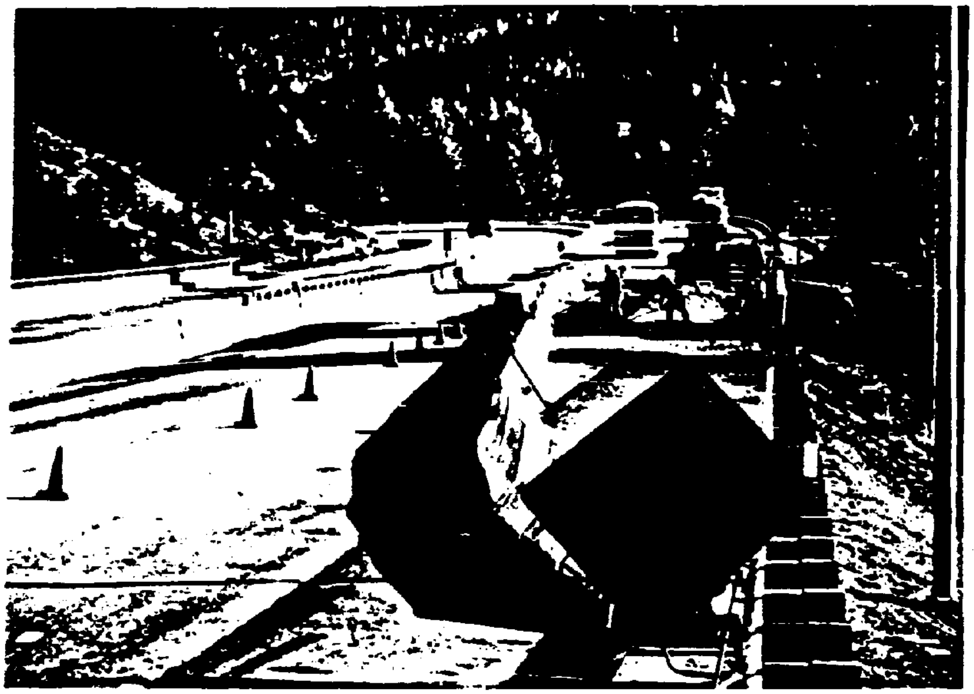


Figure 65. Structurally Tying Construction Barrier into Permanent Guardrail Is Needed Here to Produce a Safer Installation

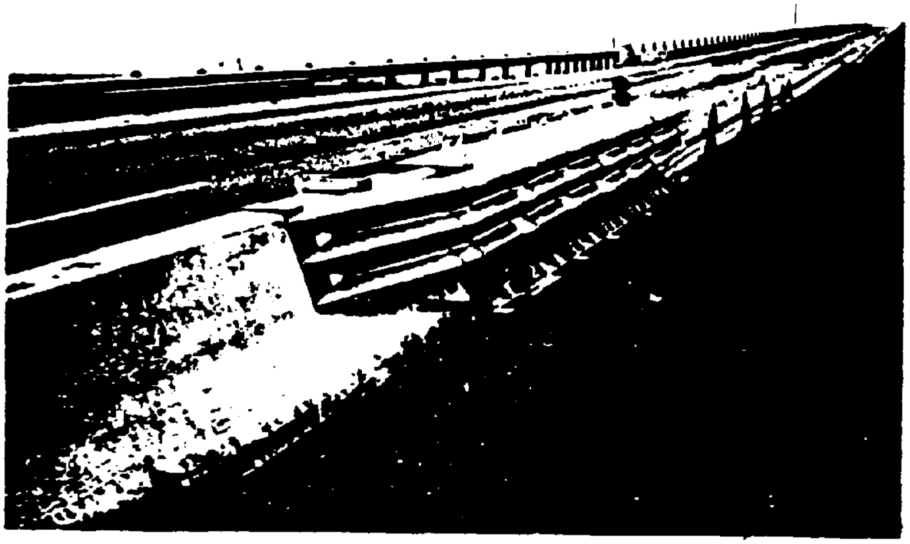


Figure 66. GREAT Installation.

Figure 67. Barrel Installation.

(Note: The writers believe the angle of the barrier installation is improper. Better protection would be provided if the barrier line was parallel to traffic or even angled slightly toward the traffic.)

Figure 68. Fitch Inertia Barrier Installation.

the construction zone. Care is required in providing an adequate connection to the PCB end or to the ground at the rear of the crash cushion.

The Fitch Inertia Barrier is easily installed, requires no connection to the PCB end and functions well in a relatively low speed head-on impact. Although Fitch Barriers can certainly be designed for higher speed collisions they have not normally been so designed for construction zone applications, usually consisting of a single row of sand-filled barrels as was shown by Figure 68. It appears that this installation would not be effective if struck close to the PCB end. Further, it is not understood why the line of barrels is directed at a significant angle away from the probable trajectory of vehicles leaving the roadway. It would seem a better alignment would be parallel to the traffic or even at a slight angle toward the traffic lanes.

An experimental design has recently been tested and shows rather good performance characteristics. It is a combination barrel-guardrail crash cushion which will fit into a very narrow area to provide a safe end for a PCB. Shown in Figure 69, this barrier was developed by Ross (8) under a contract with the State of Texas. In concept, it was derived in part from the tubular W-section/barrel design developed under this study. The barrier performs well for head-on collisions and for collisions close to the PCB end. The drawback is that the structure expands laterally under a head-on impact with collapsed guardrail sections extending laterally about 6 ft (1.8 m) in each direction. This is shown by Figure 70. Thus, if installed in a very narrow median, it would extend into adjacent traffic lanes. A related limitation is that the barrier can be knocked laterally by an angle collision close to the front of the barrier. Good vehicle redirection is achieved, but the barrier continues to move laterally after vehicle redirection. This results in the deformed barrier blocking the adjoining traffic lane. Overall the positive performance of this barrier would seem to outweigh the disadvantages. The cost also is very attractive compared to other devices although maintenance may be a problem.

All the treatments discussed to this point have had primary application to PCB's, the dominant barrier now in use. Some attention should perhaps be given to the barrier composed of a guardrail section (or

Figure 69. Experimental Barrel/W-Section Crash Cushion.

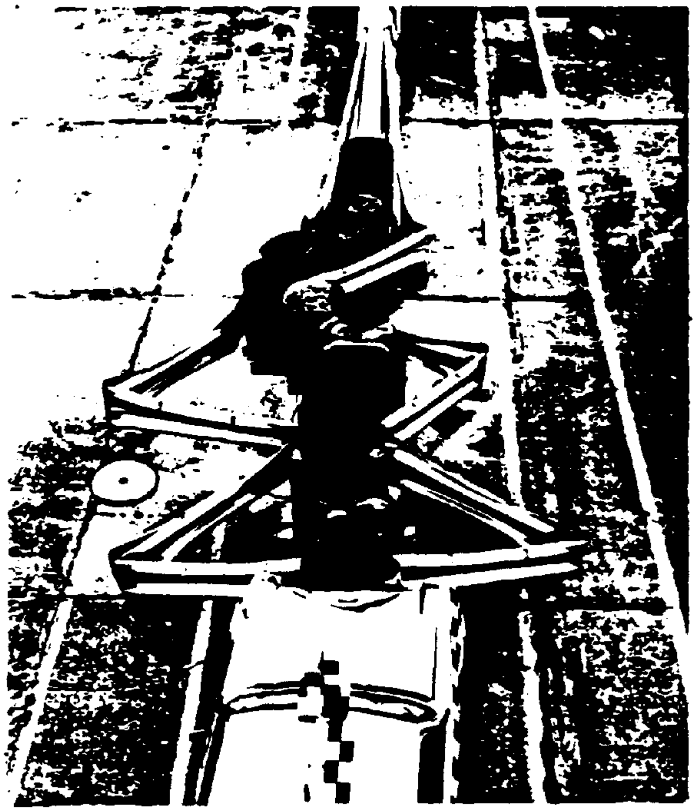


Figure 70. Lateral Extension of Experimental End Treatment After Head-on Impact.

W-section) attached to sand filled steel barrels. This barrier is used extensively in several states but is rarely installed to achieve a good end treatment condition. A typical installation is shown in Figure 71. Although some flaring of the end is normally encountered, it is usually not enough to preclude the possibility of snagging. A much larger flare with a standard end show would be a better end treatment.

A treatment often used is the "turned-down" shown in Figure 71. This is probably quite effective although it has not been tested. Although at first thought one might believe this treatment has the same disadvantages as the turned-down end on the conventional guardrail, the fact that the rigidity of this element is much lower for the construction barrier application would probably not make it a significant contributor to a hazardous ramping situation. It would also be possible to design a weak attachment to the first few barrels which would allow the end portion to "fall down" under impact.

Another end treatment which is applicable to the tubular W-section; sand filled barrel barrier which was developed and tested under this contract is shown by Figure 72. This end treatment was reported here as Test 3825-4. This test could be considered indicative of the need to lower the weight in the end barrel and needs further testing before it can be fully recommended. The principal of operation, collapse of the end, or split section of the tubular rail, appears sound and the basic design could conceivably be applied as a PCB end treatment as well, although its performance should be verified through full-scale testing.

A recently developed end treatment for W-section and thrie-beam guardrail, called the SENTRE, is commercially available. This end treatment functions by deforming longitudinally and, at the same time, by moving laterally to redirect the front of an impacting vehicle away from the guardrail. Performance of the design has been evaluated through full-scale tests but no in-service evaluation is available at this time (20).

There are a variety of end treatments available for use. Some can be adapted to almost any construction barrier, but others are somewhat specialized. Possible and obvious limitations of end treatments which have not been fully tested should be carefully noted and these end treatments used with caution. Even with the limitations, however, some constitute a considerable improvement over certain current practices.

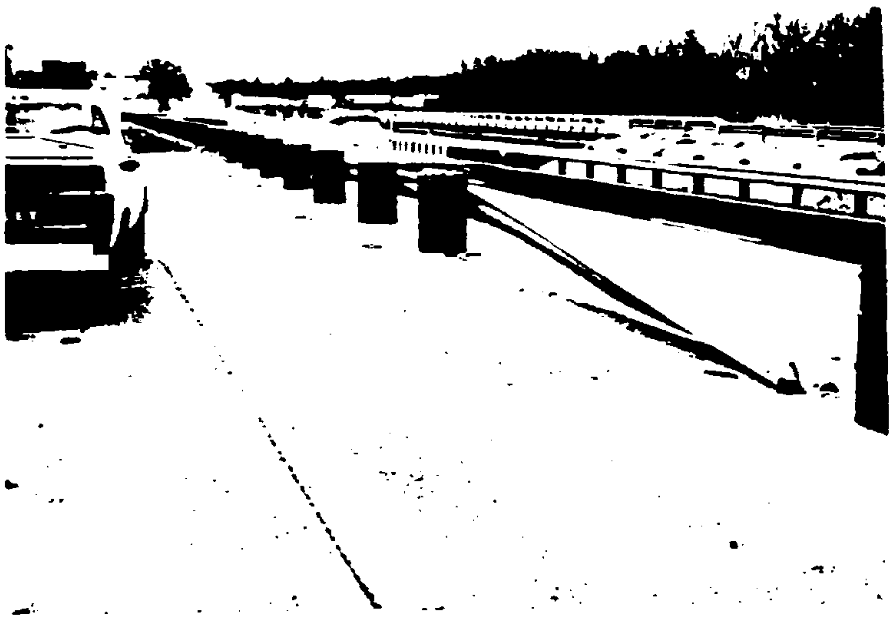


Figure 71. Turned-Down End Treatment on a Barrel/W-Section Barrier.

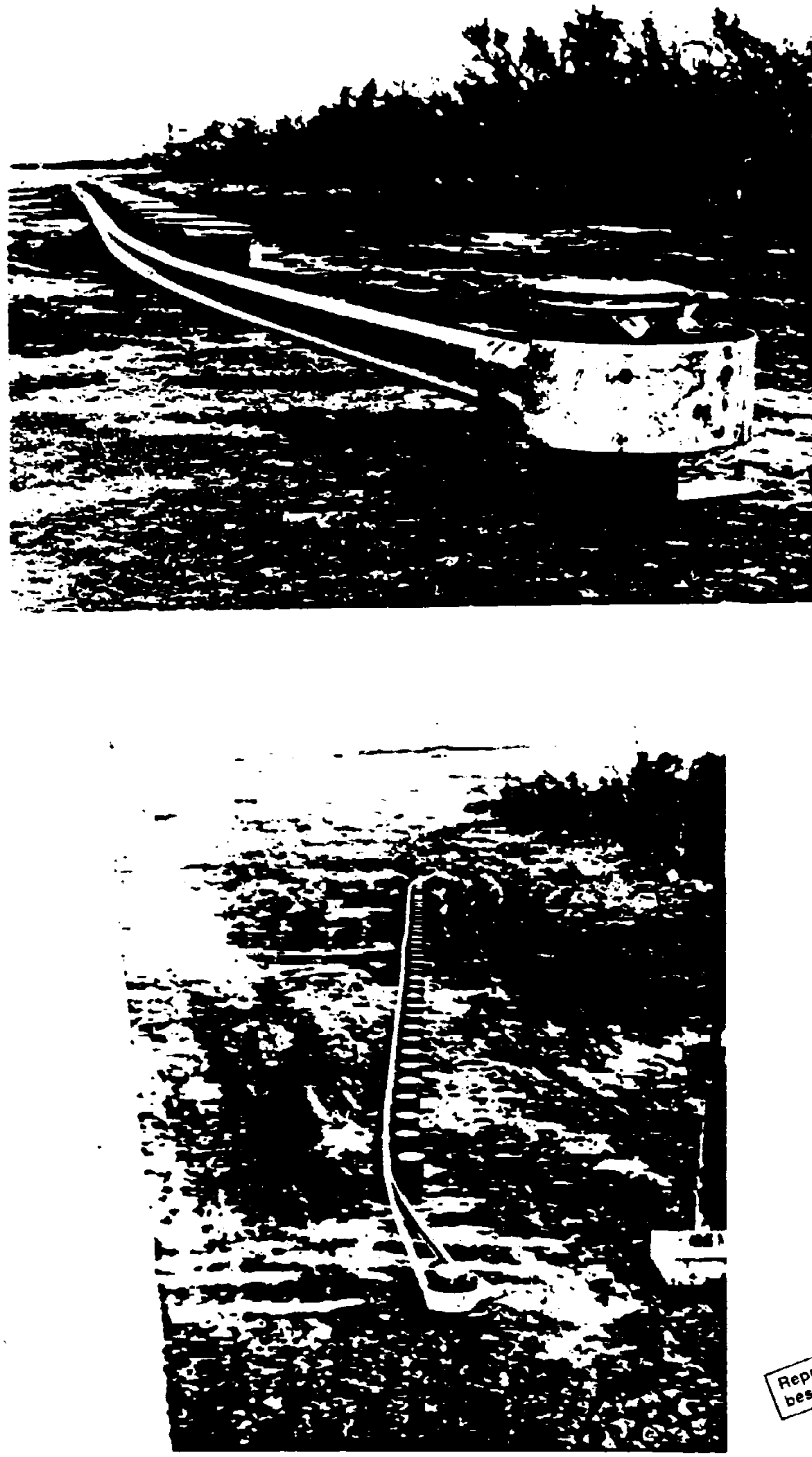


Figure 72. Experimental End Treatment for Barrel/W-Section Construction Barrier.

VI. CONCLUSION

Two types of longitudinal barriers for construction zones were studied. They were the Portable Concrete Barrier (PCB) and the Barrel/W-section barrier.

1. Several current designs of the Portable Concrete Barrier are useful to improve the safety of construction zones.
2. PCB's can be designed to meet various performance or service levels using the methods developed and demonstrated in this report.
3. Barrel/W-Section barriers as now used in construction zones provide a limited vehicle redirection capacity.
4. An improved Barrel/W-Section barrier was developed that provides a greatly improved vehicle redirection capacity.

A Simplified Energy Analysis (SEA) of a collision by an automobile was developed to predict barrier deflection. As the name implies, the procedure contains some simplifying assumptions and approximations, but does a reasonable job of showing the relationship of various parameters and of predicting results from a limited number of tests.

A more sophisticated procedure for analyzing collisions with a PCB, called the Advanced Dynamic Analysis (ADA), was also developed and was found to be a better tool for the structural design of a PCB.

Parametric studies were conducted to determine PCB sensitivity to five different characteristics. These characteristics are connection moment capacity, barrier segment length, connection slack, barrier mass, and barrier to ground friction.

5. Increasing the connection moment capacity of PCB's in general decreases the deflection during vehicle impacts. (Figure 9)
6. Increasing connection slack of PCB's in general increases the deflection during vehicle impacts. (Figure 10)
7. Increasing barrier segment length increases deflection as the length varies from ten to twenty feet, then decreases deflection as the length varies from twenty to thirty feet. (Figure 11)
8. Increasing shear resistance at the base of the barrier in general decreases the deflection during vehicle impacts. (Figure 12 and 13)

9. Increasing barrier weight modestly decreases barrier deflection during vehicle impacts. (Figure 14)
10. Barrier deflections for 30 foot long segments are rather insensitive to connection moment capacity and connection slack.

The space available for barrier deflection in a construction zone was studied in a field survey involving one hundred observations. The sampling procedure was neither random nor statistically designed but did provide useful information. A total of 120,490 ft of barrier was surveyed. Less than ten percent showed available deflection distances of one ft or less while over 70 percent exhibited distances of 10 ft or more.

11. In most applications observed, barrier deflections are small compared to the space in a construction zone available for deflection.
12. Special care should be paid to the consideration of available deflection space when Barrel/W-Section barriers are selected for use. Under some impact conditions, these barriers may deflect as much as fifteen feet laterally. (See Test 3825-3, Table 19)

A construction barrier rating system that will allow comparison of one PCB design to another was developed. The rating system considers two groups of factors to define functional adequacy and costs. The rating system will allow rational selection of an appropriate barrier design for a given situation.

13. Service levels are suggested for construction zone barriers that are somewhat analogous to service levels suggested in NCHRP Report 230. (See Table 8)
14. Existing barrier designs will meet service levels A, 1, 2A, and 2B. Strength criteria for meeting these service levels and also level 3 are suggested.
15. The overall cost of different PCB designs did not vary widely. The main differences were in the cost of connections and the maintenance costs. In general, the more costly connections are the strongest and least susceptible to collision damage. Thus initial costs and maintenance costs tend to offset each other during a period of barrier use.

16. The influence of PCB segment lengths on portability, thought to be of importance initially, does not appear to be significant. Contractor practices of using over-capacity equipment for any barrier movement results in making this real difference in portability of negligible influence.

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