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Safety of High Speed Guided Ground Transportation Systems

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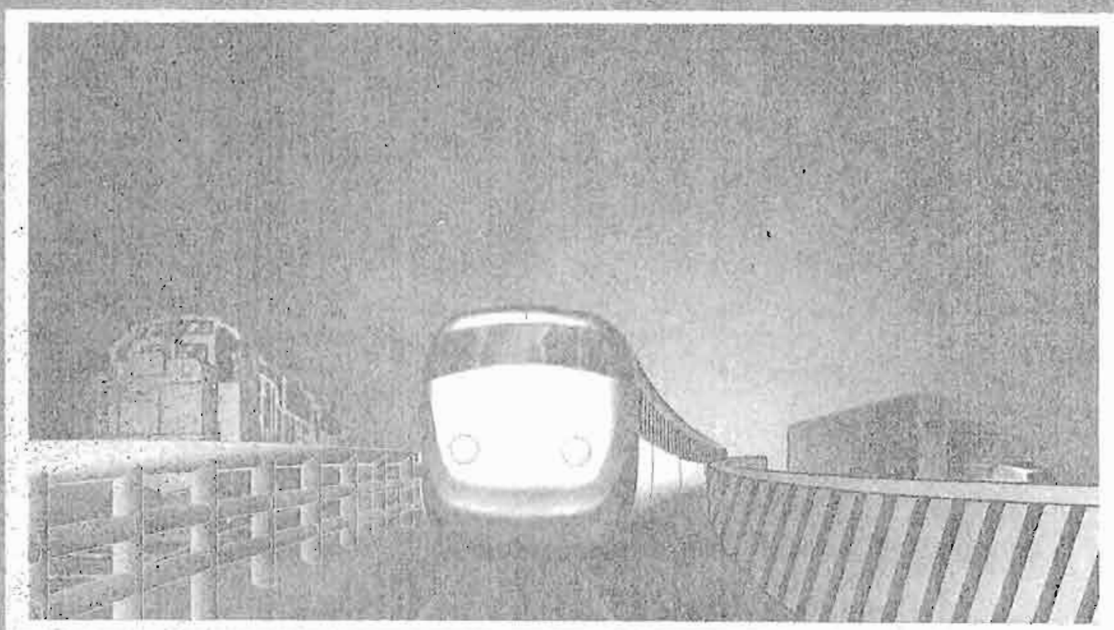
Intrusion Barrier Design Study

Parsons Brinckerhoff
Quade & Douglas, Inc.
Boston, Massachusetts



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13. ABSTRACT (Maximum 200 words)
Intrusion hazard in shared rights-of-way is a key safety issue of High Speed Guided Ground Transportation (HSGGT) systems. Minimizing this hazard will support the feasibility of locating HSGGT systems adjacent to existing transportation facilities. The objective of this study is to evaluate the feasibility of intrusion barriers that will: 1) prevent errant railroad or highway vehicles from intruding into the operational space of an HSGGT guideway from an adjacent or overhead facility; 2) prevent a derailed HSGGT vehicle from intruding into the operational space of an adjacent railroad or highway; and 3) prevent a derailed HSGGT vehicle from falling from an elevated track or guideway.

This study addresses Maglev, High Speed Rail, Conventional Railroad and Highway vehicles. Alternatives for intrusion barriers along HSGGT guideways are explored, and the feasibility and effectiveness of the various types of intrusion barriers are evaluated. An analysis method is presented and prototype designs shown which can provide a basis for future HSGGT intrusion barrier design nationwide. Eight alternative steel and concrete structural barrier designs are detailed. Barrier construction costs are estimated along with an assessment of the collision damage and repair costs likely to be incurred by vehicles and barriers.

14. SUBJECT TERMS
High speed guided ground transportation, maglev, high speed rail, conventional railroad, highway vehicles, shared rights-of-way, safety issues, structural barriers, earthwork barriers, feasibility study, dynamic analysis, computer modeling, barrier design, barrier construction costs, barrier repair costs, hazards evaluation.

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PREFACE

High Speed Guided Ground Transportation (HSGGT) systems are in the early stages of development in the United States. A major safety issue affecting the feasibility of HSGGT systems is the protection of HSGGT facilities from intrusion hazards associated with shared rights-of-way.

Shared rights-of-way offer potential for the siting of HSGGT facilities. Adjacent transportation modes within a shared right-of-way, however, pose potential intrusion hazards unique to these sites. HSGGT vehicles are vulnerable to collision in the event of an intrusion of a vehicle into the HSGGT guideway. There is also a collision hazard in the event of an intrusion of an HSGGT vehicle into an adjacent transportation corridor. Elevated HSGGT structures are vulnerable to damage from vehicle impact at their base (e.g., elevated guideway piers located adjacent to roadways or railroads). HSGGT vehicles are exposed to the hazard of a vehicle on an overhead structure falling onto the HSGGT guideway. The consequences of any of these scenarios are unacceptable, and the Federal Railroad Administration (FRA), with the support of the Volpe National Transportation Systems Center (Volpe Center), has undertaken a study on the feasibility of using intrusion barriers to minimize the consequences of these events.

This study was managed by the Volpe Center in support of the Federal Railroad Administration. Parsons Brinckerhoff Quade & Douglas, Inc. (PB) was retained by the Volpe Center to perform the requisite engineering services for a comprehensive program for the study of intrusion barriers. The objective of the study is to develop designs for barriers that can effectively mitigate intrusion hazards associated with shared rights-of-way, and assess their effectiveness and feasibility. Assisting PB in this effort was the Texas Transportation Institute at Texas A&M University, which has performed much of the recent research on the subject of intrusion barriers.

The opinions stated in this report are those of the authors, not necessarily those of the United States Department of Transportation, the Federal Railway Association, or the Volpe Center.

The authors acknowledge the assistance and support of Mr. Arne J. Bang, former Program Manager for the United States Department of Transportation Federal Railroad Administration, Office of Research and Development; and Mr. Robert Dorer and Dr. Norman Knable of the Volpe Center.

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO METRIC UNITS

Symb.	When you know	Multiply by	To find	Symb.
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters square	mm^2
ft^2	square feet	0.0929	meters square	m^2
mi^2	square miles	2.59	kilometers square	km^2
ac	acres	0.405	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb_m	$pounds_m$	0.454	kilograms	kg
kip_m	$kilopounds_m$	454	kilograms	kg
		FORCE AND ENERGY		
lb_f	$pounds_f$	4.48	Newtons	N
kip_f	$kilopounds_f$	4.48	kiloNewtons	kN
		VOLUME		
fl oz	fluid ounces	28.40	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0283	meters cube	m^3
yd^3	cubic yards	0.765	meters cube	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* SI (System International) is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO ENGLISH UNITS

Symb.	When you know	Multiply by	To find	Symb.
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters square	0.0016	square inches	in^2
m^2	meters square	10.764	square feet	ft^2
km^2	kilometers square	0.39	square miles	mi^2
ha	hectares	2.471	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	$pounds_m$	lb_m
kg	kilograms	0.00221	$kilopounds_m$	kip_m
		FORCE AND ENERGY		
N	Newtons	0.2248	$pounds_f$	lb_f
kN	kiloNewtons	0.2248	$kilopounds_f$	kip_f
		VOLUME		
mL	milliliters	0.035	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cube	35.320	cubic feet	ft^3
m^3	meters cube	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

In this report, values are shown in Metric units, followed by English units in parenthesis. In some instances, such as tabulated values and graphs where given in English units by source documents, English units are shown along with conversions to SI units.

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GLOSSARY

AASHTO	American Association of State Highway and Transportation Officials.
ACI	American Concrete Institute
AREA	American Railway Engineering Association
Barrier	A device which provides a physical limitation through which a vehicle would not normally pass. It is intended to contain or redirect an errant vehicle.
Barrier Height	Height of barrier above the top of rail or guideway.
Bridge Railing	A longitudinal barrier whose primary function is to prevent an errant vehicle from going over the side of the bridge structure.
Barrier Offset Distance	Lateral distance from centerline of vehicle guideway to face of barrier, or other trackside or roadside object or feature.
Coefficient of Friction	Ratio of friction force to normal force.
Coupler	Mechanism that provides connection between railroad cars.
Crashworthy	A feature that has been proven acceptable for use under specified conditions either through crash testing or in-service performance.
Crush Stiffness	The force required to crush a corner of a railroad car, or high speed vehicle one foot.
g	Acceleration due to gravity - 9.8 m/sec^2 (32.2 ft/sec^2)
HSGGT	High Speed Guided Ground Transportation.

GLOSSARY (cont.)

Impact Angle	For a longitudinal barrier, it is the angle between a tangent to the face of the barrier and a tangent to the vehicle's (or rail car's) path at impact.
Intrusion Barrier	A barrier intended to prevent an errant vehicle from entering into or exiting out of an HSGGT guideway by redirecting the errant vehicle back into its right-of-way.
Kip	A unit of force equal to 1000 pounds.
NCHRP	National Cooperative Highway Research Program.
Slope	The relative steepness of the terrain expressed as a ratio or percentage. Slopes may be categorized as positive (backslopes) or negative (foreslopes), and as parallel or cross slopes in relation to the direction of traffic.
Spring Stiffness	The ratio of force to deflection, based on the idealized model of a spring, where force exerted by the spring is equivalent to the product of its stiffness multiplied by its deflection from the at rest position (Force = stiffness x deflection).
TBIP	Train Barrier Interaction Program: A dynamic computer program that models conventional railroad and high speed guided ground transportation systems and their interaction with an adjacent barrier.
TTI	Texas Transportation Institute
TGV	Train a Grande Vitesse. French high speed train.
Warrants	The criteria by which the need for a safety treatment or improvement can be determined.
WMATA	Washington Metropolitan Area Transit Authority.

EXECUTIVE SUMMARY

The intrusion hazard within shared rights-of-way is a potential safety issue for High Speed Guided Ground Transportation (HSGGT) systems. The ability to cost-effectively mitigate this hazard will affect the feasibility of locating HSGGT systems on and adjacent to existing transportation facilities. The objective of this study is to evaluate the feasibility of intrusion barriers that will serve to reduce intrusion hazards, and develop designs based on rational analysis that will perform the following functions:

- Prevent a derailed railroad car or errant highway vehicle, or dislodged load from intruding into the operational space of the HSGGT guideway from an adjacent or overhead transportation corridor.
- Prevent a derailed HSGGT vehicle from intruding into the operational space of an adjacent railroad or highway, when such intrusion represents a significant increase in hazard to the safety of operations of all affected modes.
- Prevent a derailed HSGGT vehicle from leaving an elevated track or guideway, or from colliding with some other trackside hazard.

This report summarizes an approach to intrusion barrier design, describes the findings and offers conclusions and recommendations. The report consists of the following elements:

- Study of alternative types of intrusion barriers along HSGGT guideways
- Determination of the feasibility and effectiveness of the various types of intrusion barriers
- Development of a design method for those barrier systems found to be feasible and effective
- Development of designs to provide a basis for HSGGT intrusion barrier design nationwide
- Estimation of barrier construction costs
- Assessment of damage and repair costs likely to be incurred by the barriers due to vehicle impact
- Evaluation of potential hazards related to the use of intrusion barriers, including vehicle damage and passenger safety.

The scope of the study includes maglev, high speed rail, conventional railroad and highway vehicles. The full range of operating speeds for these vehicles is considered, up to 483 km/h (300 mph) for maglev, 322 km/h (200 mph) for high speed rail, 127 km/h (80 mph) for conventional railroad, and 105 km/h (65 mph) for highway. Three classes of barrier system types are evaluated: earthwork systems consisting of earth berms and ditches; structural systems consisting of steel and concrete barriers; and systems utilizing components of both. A total of 22 scenarios with various combinations of vehicle and barrier types are considered for study and analysis. These scenarios are described in Chapter 2 and are listed in Table 2-1.

Earthwork Barriers

Earthen berms and ditches are considered for use as intrusion barriers in Section 3.3. Energy methods are employed as the basis for the analysis of earthwork barriers. This analysis considers changes in potential energy of the derailed vehicle from travel across slopes, and the frictional energy losses which dissipate the initial kinetic energy of the vehicle. It is concluded that the earthwork berm and ditch barrier systems are not well suited as barriers for high speed systems, for a number of reasons. The kinetic energy of a high speed vehicle traveling at 320 km/h (200 mph) is so great that, neglecting friction, a berm over 400 meters high would be required to convert the kinetic energy to potential energy and stop the vehicle. Friction would dissipate some of the energy, but either high berms, long unobstructed stopping distances, or a combination of the two would be necessary to effectively stop high speed vehicles. Data from highway studies indicates that even slight changes in grade can cause a vehicle to become airborne resulting in loss of control. High speed vehicles would be even more sensitive to changes in grade. Errant vehicles could dig into the side of berm or ditch slopes, stopping the vehicle suddenly, causing tumbling or airborne motion and subjecting passengers to violent forces. Earthwork barriers are generally not effective in safely containing high speed consists.

Structural Barriers

Structural barriers consisting of rigid concrete or steel walls are feasible for many crash scenarios. These barriers perform their function by preventing penetration into the protected guideway, and redirecting the errant vehicle back into its own guideway. It is not the intent that structural barriers slow the vehicle down. High speed rights-of-way have controlled terrain with flat slopes and no obstructions. If contained within this environment, the vehicle would not be exposed to significant vertical movement. Structural barriers can be designed to keep an errant vehicle within its guideway until friction between the wheels and the ground gradually brings the vehicle to a stop. There is better control after derailment, and less damage and injury.

Structural barriers can be used in single or dual applications. Single barriers would be located on one side of a guideway where the hazard occurs on only one side, as in the case of high speed rail guideway adjacent to a freight railroad. In this application, the barrier would protect the high speed railroad from a derailed freight train, and would keep a derailed high speed train within its guideway. Dual barriers would be located on both sides of a set of tracks where the hazard occurs on both sides. A third barrier could be used between pairs of tracks for protection from opposing traffic of the same high speed facility, but they were considered to be impractical. Protection can be provided more efficiently through proper scheduling and communication between opposing vehicles.

Structural barriers are modeled using the Train-Barrier Interaction Program (TBIP), a computer program previously used by the Texas Transportation Institute (TTI) in a study for the Washington Metropolitan Area Transit Authority. This program, now modified for HSGGT vehicles, simulates the physical properties and kinematics of a moving rail vehicle which derails and then impacts a barrier. The program performs a two dimensional (horizontal) dynamic simulation to determine the path of the train, and the magnitude of the forces experienced by the cars and barriers at collision. Many of the parameters used in the analyses were based on previous studies of barriers designed to contain highway vehicles. Manual calculations supplement the program to evaluate three dimensional out of plane effects, such as rotation about the longitudinal car axis and vertical buckling. The analysis method and findings are described in Section 3.1.

The analysis yields interesting results. Barrier impact loads vary from 890 to 4900 kN (200,000 to 1,100,000 pounds). Impact loads from high speed vehicles are within the range of conventional vehicles. Loads from conventional freight trains, in fact, yield the highest loads - higher than those from high speed vehicles. Contrary to common expectations, the highest impact loads are observed at lower derailment speeds, in the range of 120 to 160 km/h (75 to 100 mph). At high speeds the vehicle experiences a "glancing" blow with the barrier. The train cars rebound from the barrier and travel down-track without additional impacts, and come to rest in a shallow "zig-zag" pattern. By contrast, at low speeds, the vehicles undergo a "snagging" collision. The cars remain in contact with the barrier longer during the collision and ensuing travel, and come to rest in a sharper "zig-zag" pattern. Dual barriers straddling a guideway experience the highest loads, both for high speed and conventional vehicles. This is due to the tendency of cars getting wedged between the two barriers, and getting pushed into the barriers by the cars behind.

A design methodology for structural barriers is presented which uses loads from the TBIP model as a basis. Performance specifications are included in Appendix B describing this methodology in detail. These specifications can be used as a basis for future designs. Twelve types of structural barrier designs have been developed. Grouping the 22 vehicle/barrier scenarios by barrier load and developing designs for each of the twelve barrier types produces a total of 35 different designs. The barrier alternates include precast concrete, cast in place concrete, structural steel, and retaining wall systems. These designs are shown in Figures 4-6 through 4-31.

Barrier Costs

In Chapter 5, construction costs are estimated for each of the different structural barrier systems described above, based on estimated costs of materials, labor, equipment and miscellaneous items for each system. The costs are significant. The range of costs for the barriers are given below for each type of vehicle:

At-Grade Barriers

Maglev	\$1.115M/km to \$2.64M/km	(\$1.795M/mile to \$4.25M/mile)
High Speed Rail	\$1.115M/km to \$3.38M/km	(\$1.795M/mile to \$5.44M/mile)
Conventional Rail	\$1.250M/km to \$3.38M/km	(\$2.01M/mile to \$5.44M/mile)
Highway	\$1.170M/km to \$1.320M/km	(\$1.874M/mile to \$2.11M/mile)

Elevated Barriers

Maglev	\$0.445M/km to \$1.260M/km	(\$0.713M/mile to \$2.03M/mile)
High Speed Rail	\$0.530M/km to \$2.28M/km	(\$0.845M/mile to \$3.67M/mile)
Conventional Rail	\$1.160M/km to \$2.71M/km	(\$1.874M/mile to \$4.36M/mile)
Highway	\$0.645M/km to \$0.690M/km	(\$1.056M/mile to \$1.109M/mile)

The ranges above illustrate the cost variation with the type of system. The precast concrete wall barrier designs are the least expensive alternates and are recommended for use as structural intrusion barriers. Cast-in-place retaining walls are the most expensive alternative. They are only recommended where the adjacent guideways are located at different elevations and walls would be necessary to accomplish the grade differential anyway. The above costs are average total costs for single barriers assuming new guideway construction in mid-1993 dollars. Elevated barriers generally require dual barriers, and the costs should be doubled for these situations. It is important to note that the costs above are for typical situations. Local prices, material availability and unique site features could make other barrier types preferable in some areas.

An estimate of barrier system costs can be made for a selected train route. The costs will depend on such factors as the mix of adjoining transportation systems, what fraction of the system is elevated, the number of overpasses, and what fraction of the system requires barriers. Passages where the adjoining areas are not vulnerable to derailment nor do the areas pose a threat to the high speed line, do not require barriers.

Using data contained in an as yet unpublished Commercial Feasibility Study of High-Speed Ground Options, sponsored by the FRA, a cost estimate has been made of an American high-speed rail system rangin from \$4.3M/km to \$29.8M/km (\$7M/mi to \$48M/mi) with an average of \$15.5M/km (\$25M/mi). Estimates of barrier cost (p. xviii) range from \$0.5M/km for an elevated barrier to \$3.3M/km for an at-grade barrier (\$.8M/mi to \$5.4M/mi). From these data one may expect the barrier costs to range from less than ten percent of the system cost to as much as twenty percent. Further study of siting criteria (p. xx) will permit a better assessment of these costs.

Hazards Evaluation

An assessment of the consequence of a derailment and impact with a structural barrier is made based on the impacts observed in the TBIP runs, and using estimated repair and replacement costs. Results indicate that barrier repair costs may range from \$50,000 to \$1.2M per incident. These costs do not account for costs for repair of vehicle damage.

Vehicle damage is assessed based on impact forces estimated by the TBIP analyses. Results indicate that most vehicle accident damage is expected to be minor, with less than 0.6 meters (2 feet) of crushing at the impacting corner of the car. Intuitively, much more damage would be expected. The analyses, however, illustrate that predicted movement although rapid in the longitudinal direction, would be somewhat limited laterally, and side impacts would be lower than expected. Observations of actual high speed rail derailments support this finding. A recent derailment in France resulted in very little lateral movement, and the train remained in a straight line with little "zig-zagging." For dual barrier installations, where barriers are located on both sides of a pair of tracks, higher forces and more significant vehicle damage is expected.

Passenger safety during derailment is measured by determining the acceleration of the mass center of the cars and comparing it to threshold limits accepted by the automobile industry. On this basis, it is concluded that accelerations during derailment and barrier impact are at acceptable levels for all but the dual barriers for high speed trains, where current automobile standards are exceeded.

Recommended Further Study

There are many areas where further study would be beneficial to addressing intrusion hazards along shared rights-of-way. Of critical importance is an examination of where barriers are warranted, a topic that was not covered in the current study. Decisions must be made to determine where intrusion hazards warrant the cost of barriers. It may not be necessary to locate barriers at all locations on shared rights-of-way, as was assumed in the case study. More prudent siting criteria could reduce barrier installation costs significantly. High speed consists are designed and maintained to minimize derailments. Actual performance indicates a good track record. It may be more reasonable to locate protection type intrusion barriers to exclude errant conventional vehicles from high speed guideways at locations where there is a record of derailments of adjacent conventional trains, or errant highway vehicles. Containment of HSGGT vehicles provided by intrusion barriers may be necessary only at HSGGT terminals and in urban areas, and may be unnecessary in remote areas.

Further study is also needed to verify parameters used in the analysis and design of the barriers. In the current study, many of the parameters have necessarily been based on assumptions. Although reasonable values have been selected based on previous research in the automobile industry and elsewhere, the assumptions should be verified. An example is the assumed value used in the TBIP program of the crush stiffness of the high speed vehicle structure in a collision. This value has been extrapolated from results of tests performed on automobiles, trucks and buses. Analysis indicates that the predicted impact force is dependent on assumed values of crush stiffness. This and other parameters could best be verified with crash testing or detailed analytical techniques that are outside of the scope of this study.

This study has developed methods for the design of intrusion barriers, and barrier designs have been prepared. Barrier costs have been estimated both in terms of construction cost and damage repair cost. The hazard to impacting vehicles and their passengers have been evaluated. The conclusion of the study is that intrusion barriers can be designed and constructed that can effectively reduce hazards and risks associated with vehicular intrusion on adjacent transportation corridors.

1. BACKGROUND

In 1992, the Battelle Memorial Institute prepared a report, "Safety of HSGGT Systems: Shared Right-of-Way Safety Issues," [28] which identified the protection of HSGGT facilities from intrusion hazards associated with shared rights-of-way as a safety issue affecting the feasibility of HSGGT systems. HSGGT vehicles are vulnerable to collision in the event of intrusion of a vehicle into the HSGGT guideway. There is also a collision hazard in the event of intrusion of an HSGGT vehicle into an adjacent transportation corridor. Elevated HSGGT structures are vulnerable to damage from vehicle impact at their base (e.g., elevated guideway piers located adjacent to roadways or railroads). HSGGT vehicles are exposed to the hazard of a vehicle on an overhead structure falling onto the HSGGT guideway. Intrusion barriers may represent the most effective means for mitigation of these intrusion hazards.

The current state of transportation technology does not include a methodology or criteria for the design of intrusion barriers for HSGGT vehicles. Shared right-of-way hazards are similar to hazards inherent in more conventional transportation modes such as highways and railroads. There is some research and development that has been carried out in these areas that forms the basis of much of the work in the current HSGGT study.

Extensive research has been performed in the area of highway vehicle barriers. The American Association of State Highway and Transportation Officials (AASHTO) has developed design and analysis techniques for concrete barriers, guard rails, bridge rails and crash attenuation barriers for highway facilities. To a large extent, their work is based on full scale crash tests.

Limited research has been performed in the area of railroad barriers. Criteria are provided by the American Railway Engineering Association (AREA) for the design of crash walls for pier protection along railroads. The expense associated with full scale crash tests of trains has discouraged the kind of study that has been accomplished in the automobile industry. Until the recent development of computer models, the complexity of the dynamics of a train derailment and subsequent crash has put the analysis of crash scenarios beyond the reach of conventional analytical methods.

developed for transit vehicles on rights-of-way shared with railroads. A two-dimensional computer model, the Train/Barrier Interaction Program (TBIP) was developed based on previous work by T. H. Yang to dynamically model the train/barrier impacts and determine the forces generated by the impact. This model has been modified for HSGGT vehicles and used for the analysis and design of structural barriers in the current study.

The current intrusion barrier design study is intended to further current technology toward the development of a means by which barriers can be designed that can effectively mitigate intrusion hazards associated with shared rights-of-way on high speed guided ground transportation corridors. The study develops designs for intrusion barriers, and assesses their effectiveness and feasibility. In Chapter 2, the study defines the conditions for which designs will be developed. Methods for modeling and analyzing errant vehicles and their interaction with various types of barriers are described in Chapter 3, and the effectiveness of various barrier types is described. Structural barriers, consisting of concrete or steel walls are found to be feasible, while earthwork berms and ditches are not. The development of structural barrier designs is described in Chapter 4, and detailed drawings of various types of intrusion barriers, capable of deflecting both high speed vehicles and conventional railroad and highway vehicles are presented in Figures 4-6 through 4-31. In Chapter 5, costs are estimated for construction of the barriers and for repair of barriers damaged by collision. An estimate of barrier system cost is made in Section 5.1.4. Chapter 6 evaluates the hazards associated with the introduction of barriers into a right-of-way, both in terms of vehicle damage, and passenger safety. Conclusions and recommendations are given in Chapter 7.

2. INTRUSION SCENARIOS

It is intended that this study cover the possible HSGGT systems likely to be used in the U.S. There are many possible combinations of vehicles, speeds and types of intrusion barriers to be evaluated. Combinations of potential vehicle accidents have been assembled into 22 scenarios. Each scenario is defined by values selected for different variables. This section of the report describes the rationale behind the selection of the scenarios. Variables considered include vehicle type, barrier function, barrier type, number of barriers, barrier offset distance and vehicle speed. These variables are described below, along with a discussion on how they will be used in the analysis methods presented in later sections of the report.

2.1 VEHICLE TYPE

This study is intended to evaluate vehicles representative of the consists likely to be used in the United States. Vehicle types to be studied have been narrowed down to a manageable number that is representative and gives meaningful results. The study does not cover atypical vehicles, such as double-stacked railroad cars. A methodology is given, however, that can be used for the design of barriers for any vehicle. The following vehicle types have been evaluated in this study:

Maglev — *German Transrapid 07.* This vehicle has an undercarriage structure that wraps around the guideway, dramatically reducing derailment hazards. Nonetheless, barriers have been designed that could contain this vehicle in the event of a derailment. This barrier design can then represent potential barrier requirements for other maglev vehicles that do not have the wrap-around design.

High Speed Rail - Type 1 — *Articulated - TGV Atlantique.* This vehicle has articulated couplers that limit the angular rotation between the cars. It was hypothesized that this type of car would behave differently than the more conventional non-articulated car in a derailment event.

High Speed Rail - Type 2 — *Non-Articulated - ICE InterCity Express.* The X2000 was also considered, but it was determined that the ICE vehicle is a heavier, more conservative choice. Barrier requirements would be expected to be less severe for the X2000.

Railroad - Type 1 — *Uniform freight car consists.* This consist would be made up of freight cars having the same weight and dimensions.

Railroad - Type 2 — *Mixed freight car consists.* Derailed trains have been found to behave quite differently with mixed and uniform car consists.

Highway - Type 1 — *36,300 kg (80,000 lb) tractor-trailer van truck.*

Highway - Type 2 — *36,300 kg (80,000 lb) tractor trailer tank truck.* The tractor trailer tank truck has a higher center of gravity than the tractor-trailer van truck.

2.2 BARRIER FUNCTION

Intrusion barrier systems have been divided into two classes, depending on their intended function, protection or containment.

HSGGT Protection — For protecting HSGGT operations from intrusion by external railroad or highway vehicles as shown in Figure 2-1. Protection barriers protect against vehicular intrusion into the HSGGT's path.

HSGGT Containment — For containing an HSGGT vehicle within its guideway in the event of a derailment, thereby reducing risks to and from adjacent hazards, as shown in Figure 2-2.

Further, these types of barriers can perform their functions either at-grade, on elevated structures, or at pier bases as pier protection barriers.

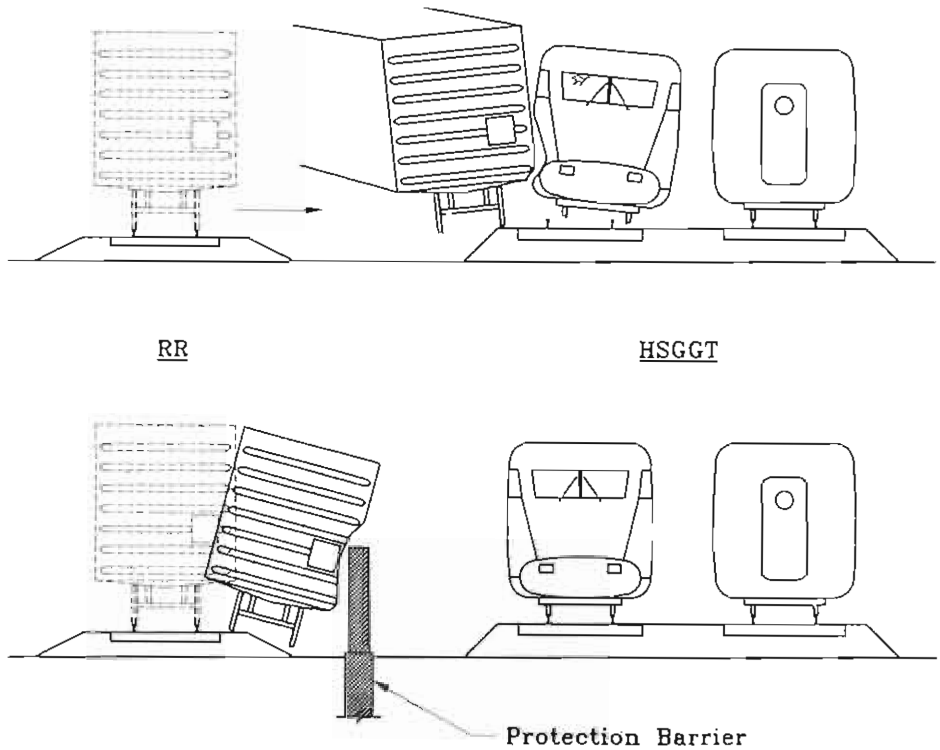

FIGURE 2-1. PROTECTION BARRIER

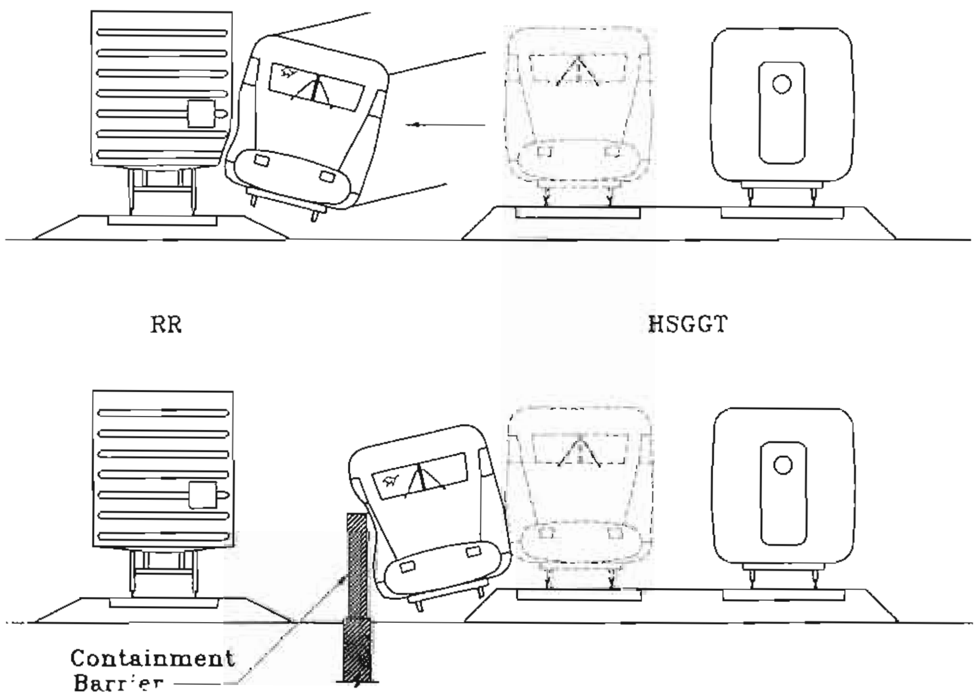

FIGURE 2-2. CONTAINMENT BARRIER

At-Grade Barriers The usual application for both protection and containment barriers is at-grade, where the HSGGT and adjacent facilities are approximately at the same grade, as shown in Figures 2-1 and 2-2.

Elevated Barriers Where the two modes are at different elevations, an elevated barrier would be used. Figure 2-3 shows an elevated barrier used for containment of a high speed facility located on a bridge or viaduct. This barrier would serve to contain the vehicle on the guideway and prevent it from falling off, protecting the elevated vehicle from the falling hazard, and the vehicles below.

Pier Protection Barriers Where a highway or railroad guideway is adjacent to an HSGGT viaduct or bridge, intrusion barriers can be used for protecting the elevated HSGGT structure from damage from an errant vehicle impacting its base, as shown in Figure 2-4.

There are situations where one barrier will perform both protection and containment functions, such as a barrier between an HSGGT facility and a freight railroad facility. In this case, the barrier would serve to contain the derailed HSGGT vehicle and also protect the HSGGT facility from derailed railroad vehicles.

2.3 BARRIER TYPE

Barrier types have been considered to include structural barriers, composed of concrete and steel components; earthwork barriers, composed of earth berms and ditches; and combination barriers composed of elements of structural and earthwork barriers.

Structural Barriers Structural barriers consist of concrete and/or steel barriers designed to contain or deflect a vehicle in an impact situation. Examples are shown in Figures 2-1 through 2-4.

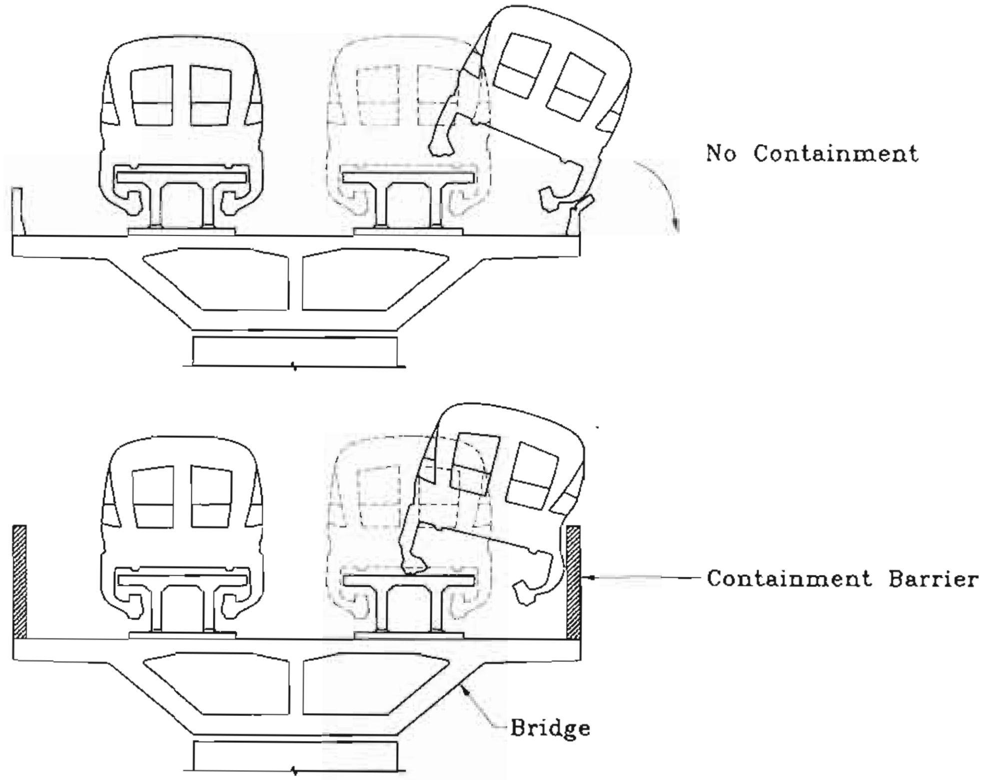

FIGURE 2-3. ELEVATED BARRIER

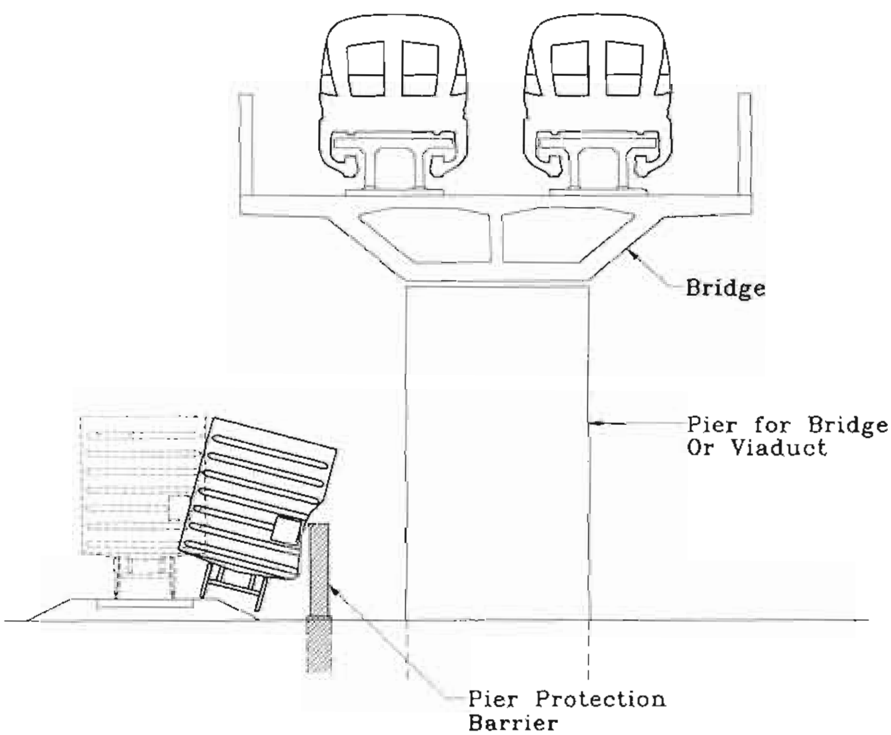

FIGURE 2-4. PIER PROTECTION BARRIER

These barriers perform their function by preventing penetration into the protected guideway, and redirecting the errant vehicle back into its own guideway. It is not the intent that structural barriers slow the vehicle down. Structural barriers are designed to keep the vehicle within its guideway until friction between the wheels and the ground gradually bring the vehicle to a stop. Conventional and high speed rights-of-way have controlled terrain with flat slopes and few obstructions. Hazards can be minimized, therefore, if the vehicle can be contained within this area.

Structural HSGGT *Protection* Barriers are placed adjacent to the source of errant vehicles (i.e. near trackside of a nearby conventional railroad, which could be located at various distances from the HSGGT guideway) or near the HSGGT guideway, whichever is more advantageous for the protection of HSGGT operations. Protective barrier systems are also placed on adjacent elevated structures to prevent vehicles from falling onto the HSGGT guideway below.

Structural HSGGT *Containment* Barriers are usually placed near the HSGGT guideway, because the intended function is to contain the HSGGT vehicles within its guideway and keep it away from nearby hazards in the event of derailing. This is based on the hypothesis that the hazards are nearby and therefore require protection close to the guideway. Studies indicate that usually the impact is less violent when the barrier is nearer the derailing vehicle. Section 3.1.3.1 describes this variation in force with barrier distance from tracks.

When the HSGGT guideway is located close to another facility, a single barrier can perform both the protection and containment functions. In these situations it may be more advantageous to place the barrier close to the adjacent guideway if this produces a lower maximum derailment force. Recommendations for offset distance are covered in detail in Section 4.1.2.10.

Earthwork Barriers

Earthwork barriers consist of earth berms and ditches, or gravel beds for energy dissipation similar to the runaway truck escape ramps that are used on highways. Earthwork barriers serve as either protection barriers, containment barriers, or both. They offer protection through redirecting the vehicle, or by slowing it down.

Berm barriers (Figure 2-5) have been considered as either protection or containment barriers where vehicles or their loads can be kept from invading the transportation envelope of another mode within a shared right-of-way. Berms may best be used in dissipating kinetic energy through the use of embankment slopes.

Ditch barriers (Figure 2-6) have been considered as either protection or containment barriers where vehicles or their loads can be kept from encroaching into the transportation envelope of another mode within a shared right-of-way. Ditch barriers are intended to contain both vehicle and vehicle loads. The ditch's side slope could further dissipate the energy of an errant vehicle.

Analysis indicates that earthwork barriers are not effective or feasible as intrusion barriers. The analysis used to reach this conclusion is included in Section 3.3. Earthwork barrier scenarios have not been included, however, in the scenario list.

Combination Barriers

These barriers combine structural and earthwork components to perform either the protection or containment functions, or both. Possible combination barriers include a retaining wall type barrier (Figure 2-7) and a concrete barrier wall enclosed in an earth berm (Figure 2-8). It was theorized that combination barriers could perform the required functions more efficiently than either structural or earthwork barriers acting alone. As discussed in Section 3.3, combination barriers have been found to be ineffective.

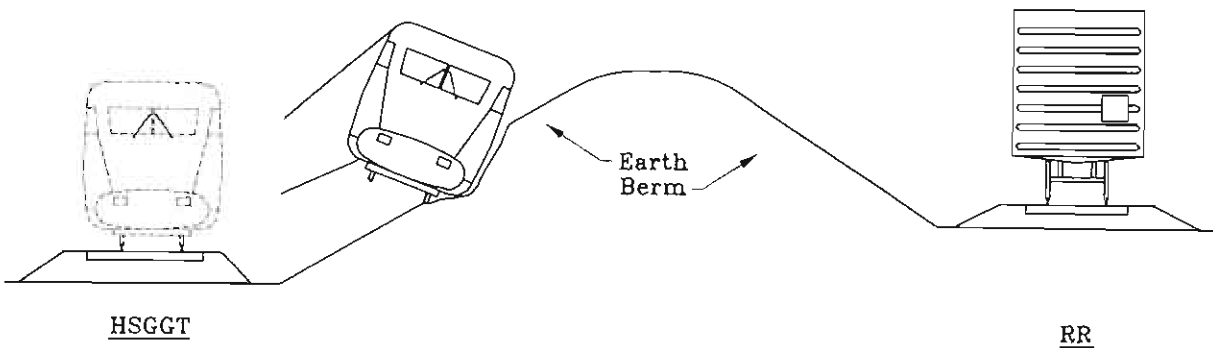

FIGURE 2-5. EARTHWORK BERM BARRIER

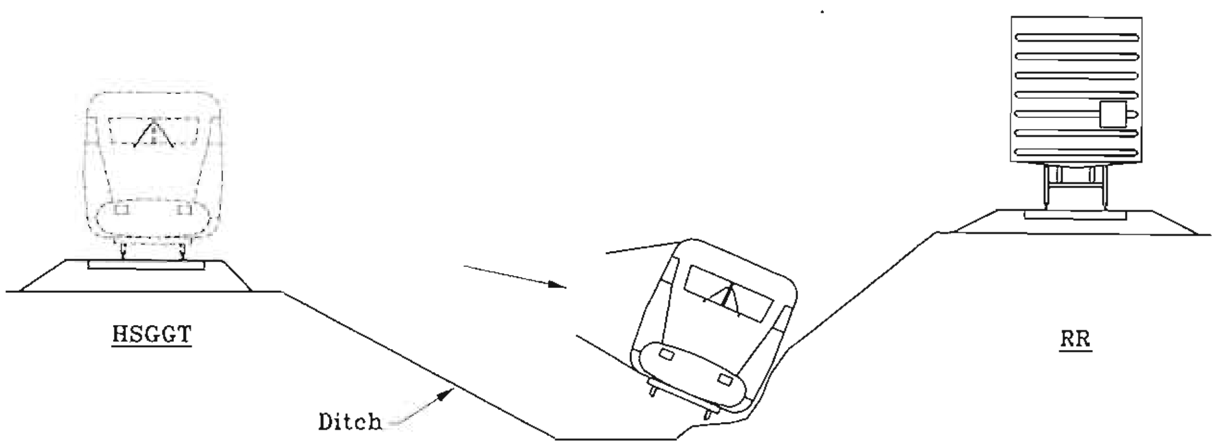

FIGURE 2-6. EARTHWORK DITCH BARRIER

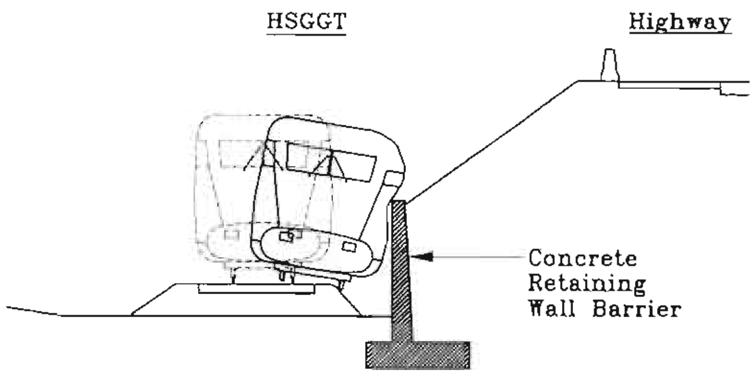

FIGURE 2-7. COMBINATION BARRIER - RETAINING WALL

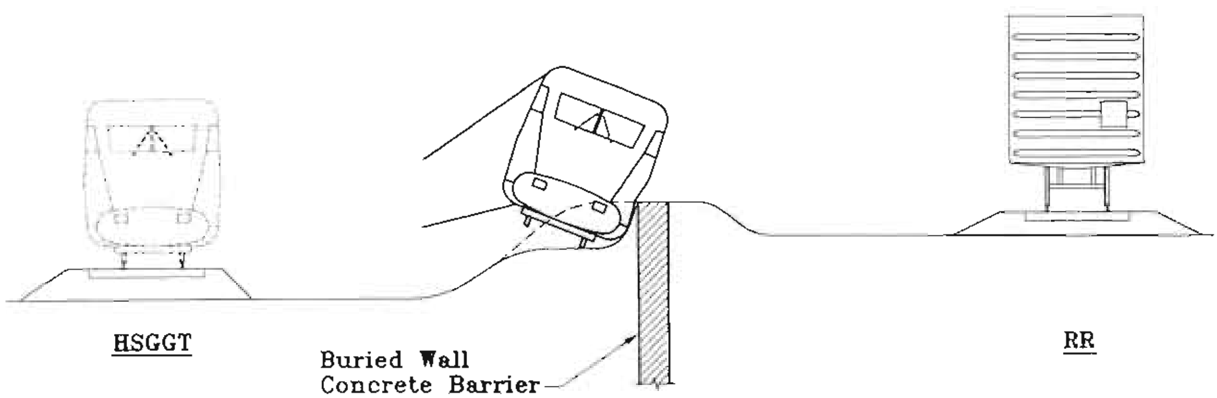

FIGURE 2-8. COMBINATION BARRIER - BURIED CONCRETE WALL

2.4 NUMBER OF BARRIERS

In the course of the study it has been observed that derailed vehicles behave quite differently, depending on the number and placement of adjacent barriers. Two situations have been considered, as follows:

Single Barrier — A single barrier is located on one side of the guideway if the hazard exists on only one side, as in the case of high speed rail guideway adjacent to a freight railroad, as shown in Figures 2-1 and 2-2. In this application, the barrier would protect the high speed railroad from a derailed freight train, and would keep a derailed high speed train safely within its guideway.

Dual Barriers — Dual barriers are located on both sides of the guideway; for example, straddling both tracks of a high speed rail facility if hazards exist on both sides, as shown in Figure 2-9. Dual barriers are also used on overhead structures to protect an HSGGT facility that passes underneath, or to contain an elevated HSGGT facility as shown in Figure 2-3.

A third barrier could be used between pairs of tracks for protection from opposing traffic of the same high speed facility, but they are considered impractical. Protection can be provided more efficiently through proper scheduling and communication between opposing vehicles.

2.5 BARRIER OFFSET DISTANCE

Impact forces have been found to be dependent on the distance and perpendicular to the track of the barrier, from the centerline of the vehicle guideway to the face of the barrier (see Section 3.1.3.1). This distance is known as the *barrier offset distance* (see Figure 2-10). A range of barrier offset distances have been studied and forces generated for each. The minimum barrier offset distance considered is the minimum allowed by code for clearance of the various vehicles. The forces resulting from barrier placement at different offset distances is smallest when the barriers are located at large and small distances from the track. Maximum values are reached when barriers are placed at intermediate distances. Barriers experience no force at large offset distances, where the barrier is located beyond the lateral travel of the vehicle.

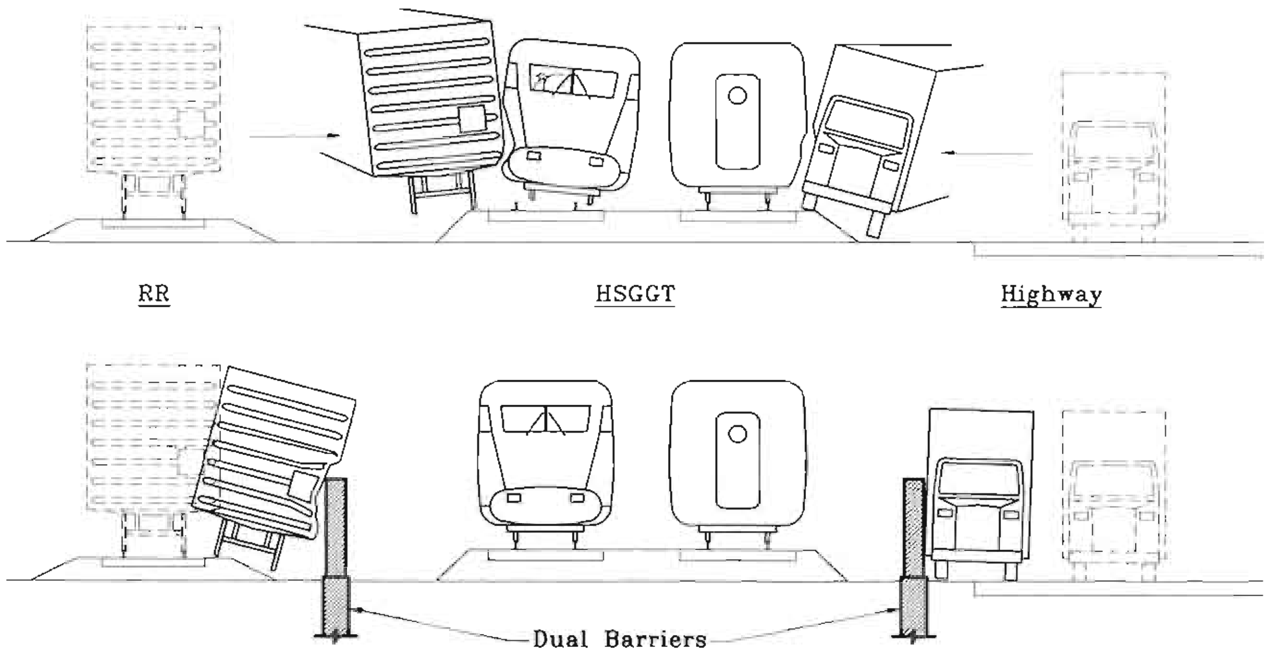

FIGURE 2-9. DUAL BARRIER

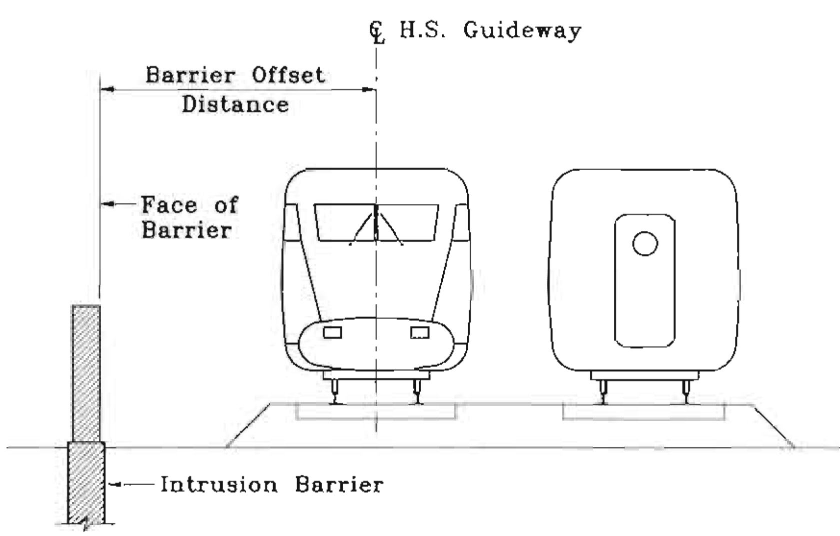

FIGURE 2-10. BARRIER OFFSET DISTANCE

2.6 VEHICLE SPEED

It is intended that this study cover the full range of speeds likely to be encountered for the selected vehicle types. The following speeds indicate the upper and lower bounds that have been considered:

	Minimum Speed	*Maximum Speed*
Maglev:	80 km/h (50 mph)	483 km/h (300 mph)
High Speed Rail:	80 km/h (50 mph)	322 km/h (200 mph)
Railroad:	80 km/h (50 mph)	127 km/h (80 mph)
Highway:	89 km/h (55 mph)	105 km/h (65 mph)

2.7 ANALYSIS METHODOLOGY

Different analysis methodologies for the establishment of design parameters have been determined to be appropriate for the different types of barriers. These methodologies are summarized below:

Structural Train Barriers: *Train-Barrier Interaction Program:* A dynamic computer modeling program is used to model the behavior of a moving train type vehicle (high speed rail, conventional, or maglev vehicle) as it derails and impacts the barrier. The program determines the force applied to the barrier which is then used for design. This methodology is described in detail in Section 3.1.

Structural Highway Barriers: *AASHTO Methods:* The American Association of State Highway and Transportation Officials (AASHTO) is currently developing new methods for designing highway barriers for incorporation into its Standard Specifications for Highway Bridges [1]. These methods, described in detail in Section 3.2, are used for intrusion barriers for highway vehicles.

Earthwork Barriers: *Energy Methods:* Energy equations of motion are used to describe the interaction of moving vehicles with earthworks comprised of berms and ditches. This methodology is described in detail in Section 3.3.

Combination Barriers: *Modifications of Above:* Modifications to the above methods are used as appropriate for the analysis of combination barriers and are dependent on the characteristic behavior of the barrier system: whether primarily structural or earthwork. This methodology is described in Section 3.4.

2.8 SUMMARY

Examination of the above variables suggests a large number of permutations of variables to be considered. Representative scenarios have been selected to cover the cases of greatest concern for safety. These scenarios are listed in Table 2-1.

All extraneous and unlikely scenarios have been eliminated from the list to ensure that the critical scenarios get adequate attention. For example, dual at-grade barriers are listed for high speed passenger vehicles, but not for freight vehicles. Dual barrier freight scenarios are not included because the protection of a high speed guideway requires the placement of a single barrier between the high speed and freight guideway. Dual freight barriers would only be necessary if there were high speed guideways on both sides of a freight railroad -- an unlikely situation. High speed guideways, on the other hand, often will have freight or other guideways on both sides, requiring dual barriers. Elevated freight barriers are included because a common situation is a freight railroad bridge spanning a high-speed facility. Protection is necessary on both sides of such a bridge so that freight trains will not fall off of either side of the bridge onto the high speed guideway.

TABLE 2-1. LIST OF SCENARIOS

Scen. No.	*Vehicle Type*	*Intrusion Barrier*		*Offset Dist. - m(ft)*		*Speed - km/h(mph)*		*Analysis Methodology*[2]
		Barrier Type	*Barrier Function*	*Max*[1]	*Min*[1]	*Max*	*Min*	
At Grade Barriers								
1	Maglev	Single-Structural	Containment-At Grade	12.2(40)	3.4(11)	483(300)	80(50)	TBIP
2	HSR-Articulated	Single-Structural	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
3	HSR-Nonarticulated	Single-Structural	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
4	Freight-Uniform Car	Single-Structural	Protection-At Grade	12.2(40)	2.7(9)	127(80)	56(35)	TBIP
5	Freight-Mixed Car	Single-Structural	Protection-At Grade	12.2(40)	2.7(9)	127(80)	56(35)	TBIP
6	Maglev	Single-Combination	Containment-At Grade	12.2(40)	3.4(11)	483(300)	80(50)	TBIP
7	HSR-Articulated	Single-Combination	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
8	HSR-Nonarticulated	Single-Combination	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
9	Freight-Uniform Car	Single-Combination	Protection-At Grade	12.2(40)	2.7(9)	127(80)	56(35)	TBIP
10	Freight-Mixed Car	Single-Combination	Protection-At Grade	12.2(40)	2.7(9)	127(80)	56(35)	TBIP
11	Maglev	Dual-Structural	Containment-At Grade	12.2(40)	3.4(11)	483(300)	80(50)	TBIP
12	HSR-Articulated	Dual-Structural	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
13	HSR-Nonarticulated	Dual-Structural	Containment-At Grade	12.2(40)	2.7(9)	322(200)	80(50)	TBIP
Elevated Barriers								
14	Maglev	Dual-Structural	Containment-Elevated	4.9(16)	3.4(11)	483(300)	80(50)	TBIP
15	HSR-Articulated	Dual-Structural	Containment-Elevated	4.9(16)	2.7(9)	322(200)	80(50)	TBIP
16	HSR-Nonarticulated	Dual-Structural	Containment-Elevated	4.9(16)	2.7(9)	322(200)	80(50)	TBIP
17	Freight-Uniform Car	Dual-Structural	Protection-Elevated	4.9(16)	2.7(9)	127(80)	56(35)	TBIP
18	Freight-Mixed Car	Dual-Structural	Protection-Elevated	4.9(16)	2.7(9)	127(80)	56(35)	TBIP
Highway Barriers								
19	Highway-Van	Single-Structural	Protection-At Grade	NA	NA	105(65)	89(55)	Analytic/Test
20	Highway-Tank	Single-Structural	Protection-At Grade	NA	NA	105(65)	89(55)	Analytic/Test
21	Highway-Van	Single-Structural	Protection-Elevated	NA	NA	105(65)	89(55)	Analytic/Test
22	Highway-Tank	Single-Structural	Protection-Elevated	NA	NA	105(65)	89(55)	Analytic/Test

1. "Offset Dist." = Barrier offset distance measured from centerline of track to face of barrier.
2. Analysis Methodology: TBIP Train-Barrier Interaction Program developed by TTI.
Analytic/Test: For highway vehicles, analytical methods currently employed by AASHTO.

3. MODELING AND ANALYSIS OF CRASH SCENARIOS

The general design approach for intrusion barriers begins with determining the barrier requirements for each type of vehicle. Then, since barriers will usually have different vehicles on both sides, a barrier is designed that meets the requirements of both vehicles for a given location and can withstand potential intrusion from either. Chapter 3 describes methods for determining the requirements, or parameters for the design of intrusion barriers consisting primarily of the geometry of the barrier, and the force that it must resist in a crash event.

The first problem to be addressed is how to model the crash scenario numerically to arrive at these design parameters. This chapter describes approaches and analysis results for four different barrier types: Structural Train (Railroad and HSGGT) Barriers in Section 3.1, Structural Highway Barriers in Section 3.2, Earthwork Barriers in Section 3.3, and Combination Structural/Earthwork Barriers in Section 3.4.

The barrier geometries and impact forces developed here are used in Chapter 4 where a methodology for the design of barriers is presented, along with barrier designs for the scenarios listed in Table 2-1.

In addition to discussions of modeling and analysis, preliminary findings on the feasibility of the four categories are presented here, particularly the Earthwork and Combination Barriers which were found to be impractical for use as intrusion barriers.

3.1 STRUCTURAL TRAIN BARRIERS

Structural barriers provide protection by means of a rigid barrier or wall. The analysis of structural barriers designed to contain *trains* is treated separately from barriers designed to contain *highway vehicles* because the analysis and modeling techniques used for each are quite different. A dynamic computer model is used for modeling and analyzing the train or HSGGT vehicle as it impacts the structural barrier. This methodology is applied to right-of-way intrusion from derailed high-speed rail (HSR) trains, magnetically levitated (Maglev) trains, and conventional freight or passenger trains. As described in Section 3.2, highway barriers are generally designed and validated with the use of crash testing. Although reasonable for highway vehicles, this technique is prohibitively expensive for the routine design of train barriers, thus, the justification for analytical means.

In the structural train barrier model, a vehicle is assumed to derail and depart its guideway, traveling along the tracks while slowing due to ground friction. The vehicle crashes into a barrier at an angle some distance from the point of derailment, then deflects back toward the tracks and comes to a stop with the cars following a zig-zag "accordion" pattern. The vehicle impacts the barrier at various locations (see Figure 3-1). The determination of the magnitude of the forces imposed by the impacting vehicles on the structural barrier is the primary objective of the analysis.

The movement of the train is affected by many different variables, including initial velocity, characteristics of the couplers (whether articulated or non-articulated), ground friction, vehicle mass and mass distribution, the structural strength characteristics of the vehicle, and the lateral distance of the barrier from the vehicle. A computer program was developed to model these variables and predict the movement of a derailing train and estimate the force of impact with a trackside barrier.

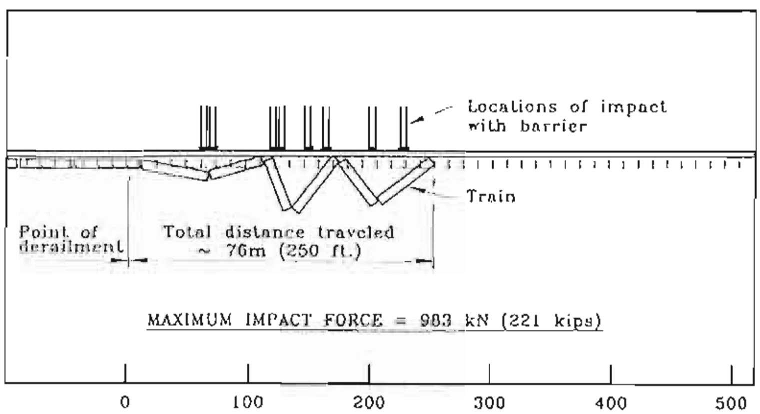

FIGURE 3-1. DERAILMENT/IMPACT MODEL

Note: Figure shows a freight train after derailment. High-speed vehicles are characterized by less severe zig-zag movement, but greater longitudinal travel down the tracks (See Section 3.1.3.1)

3.1.1 Methodology

3.1.1.1 Train-Barrier Interaction Program (TBIP)

Modeling the interaction of the structural barrier and the derailing railroad or HSGGT vehicles is accomplished using a two-dimensional computer model simulating the derailment of train consists. In a derailment, each car can roll, pitch, yaw, and translate in three dimensions. A review of past accidents reveals that the pattern is, in fact, extremely complicated. This study is limited to the most significant motions (those in a horizontal plane) in order to simplify the simulation. The computer program models the physical properties and kinematics of the moving vehicle, barrier, ground, and rail during derailment. It performs a dynamic simulation to determine the path of the train or high speed vehicle, and the magnitude of the forces experienced by the cars and barriers in their collisions. The analysis is based on the following assumptions:

- The cars are coupled together with a certain resisting moment between cars
- Simple ground friction is applied at the trucks of the derailed cars
- Emergency braking is applied to non-derailed cars
- Cars remain coupled
- The rail is interrupted by the first derailed car (i.e., once one car derails, all subsequent cars derail at the same point in the track)
- A rigid barrier is located a specified offset distance from the centerline of tracks
- The vehicle's resistance to impact with the barrier is modeled with a spring stiffness analogy
- Simple barrier friction is applied at car-barrier impact points

The computer model employed, referred to here as the Train-Barrier Interaction Program, or TBIP, originated as a two-dimensional model of derailing freight trains developed by Yang et al. (1972) in an attempt to study the influence of several variables and parameters on vehicle derailment behavior [2]. Variables studied include the number of cars in a train, car length, car weight, and initial velocity of train. Model parameters studied include braking effectiveness, coupler moment-rotation characteristics, and ground friction. The Yang model did not incorporate the affects of intrusion barriers.

In 1989, the original computer model was modified by TTI in a study sponsored by the Washington Metropolitan Area Transit Authority (WMATA) [3,4]. WMATA operates trains in shared right-of-way with freight train tracks, and the threat of derailments on the adjacent freight lines and possible intrusion into the commuter train operational space led WMATA managers to study the feasibility of construction of structural barriers to positively separate the WMATA trackway from the adjacent freight tracks. The model was modified to incorporate a simple model of a rigid barrier interaction with derailing cars through a linear spring model [3,4]. A nominal width of 3.05m (10 feet) was incorporated for the cars. Also a graphical presentation program was developed to display the simulated derailment as a plan-view, slow-motion presentation on a microcomputer monitor. Other enhancements, including a model of coupler separation and car-to-car impact were also studied [5].

The model employed to perform the analysis in the present study is essentially the same model employed in the WMATA study. Minor modifications have been made, however, to accommodate the high velocities at which the HSGGT vehicles travel, and to incorporate the different characteristics of the high speed vehicles.

TBIP simulates the train movement during a derailment and subsequent impact with a barrier by performing calculations of mathematical formulae at specified time intervals. The unknowns in the formulae consist of: (1) the movement of each car defined by translation along the track, translation parallel to the track, and rotation about a vertical axis through the center of mass of the car, and (2) the forces acting on each car, including forces transmitted through the couplers from one car to the next, ground friction forces applied at the trucks, and barrier impact and friction forces applied at the car corner as it impacts the barrier. The following calculations are performed to solve for the unknowns at each instant during the derailment:

Equations Of Motion: Equate the summation of forces acting on each car to the product of the car's mass multiplied by the acceleration in each of the three directions.

Equations of Constraint: Define the location and acceleration of the back end of one car as being the same as that of the front end of the next car.

With these equations, the program can solve for the unknown movement and forces acting on each car at each instant of time. The results form the basis for design of the barriers, as described in Chapter 4.

The parameters used in the TBIP are either built into the FORTRAN code or supplied in the input dataset. Those parameters provided in the dataset include the following, listed in the order in which they appear in the dataset, by the name used in the code. A sample input dataset is shown in Figure 3-2.

Time Increment The fixed time increment (in seconds) used in the dynamic simulation. To prevent numerical instability, a short time increment is used. The critical value depends on mass, stiffness, and velocity parameters. A parameter study of T1 should be accomplished initially to ensure that numerical instability is not a problem.

ICE11C.OUT

Time Incr.	Init. Angle	Barrier Frict.	No. Barriers		
0.0001	0.05	0.40	1		
Dist. to near barrier	Distance to far barrier				
12	27				
ICE 1P+12CC+1P[1]	189200 POWER[2]	110000 COACH[3]			
No Cars	Ground Friction	Velocity (fps)			
14	1.00	295.16			
Coupler Parameter	m0	m1	m2		
	-0.70238	1.67024	-0.72043		
Brake parameters	A0	A1	A2	A3	A4
	21774.8	-267.2	3.351	-0.1572	0
Car	INER. MOM.	M(Slug)	L(ft)	W.B.(ft)	SKCB(lb/ft)
1	2.165E06	5876.	66.5	38.0	260000.
2	1.822E06	3416.	80.0	56.0	170000.
3	1.822E06	3416.	80.0	56.0	170000.
4	1.822E06	3416.	80.0	56.0	170000.
5	1.822E06	3416.	80.0	56.0	170000.
6	1.822E06	3416.	80.0	56.0	170000.
7	1.822E06	3416.	80.0	56.0	170000.
8	1.822E06	3416.	80.0	56.0	170000.
9	1.822E06	3416.	80.0	56.0	170000.
10	1.822E06	3416.	80.0	56.0	170000.
11	1.822E06	3416.	80.0	56.0	170000.
12	1.822E06	3416.	80.0	56.0	170000.
13	1.822E06	3416.	80.0	56.0	170000.
14	2.165E06	5876.	66.5	38.0	260000.

FIGURE 3-2. SAMPLE TBIP INPUT DATASET *(Note: program input is in English units)*

---

[1] "ICE 1P+12CC+1P" denotes the vehicle type and the arrangement of cars in the consist (in this example, an ICE vehicle with one power car, twelve coach cars and one power car).

[2] "189200 POWER" denotes the weight of the power car (in this example, 189,200 pounds)

[3] "110000 COACH" denotes the weight of the coach cars (in this example, 110,000 pounds)

Initial Derailment Angle	The initial value of the derailing angle (in radians). When the first car derails, it veers off the track and rotates about the vertical axis of the rear truck, forming an angle between the longitudinal axis of the car and the track (See Figure 3-3).
Initial Derailment Angle	TBIP initiates the derailment sequence by imposing an to the first derailing car. The rear truck of the car remains on the rails, and the front truck is displaced in the direction of rotation. The initial velocity vector of the mass center of the first car remains parallel to the track centerline. The forces of the ground on the front truck tend to cause the front of the derailing car to displace further and strike any barrier provided near the rails (See Figure 3-3). Maximum barrier forces appear to depend strongly on this parameter because different initial derailment angles result in different path lengths before impacting the barrier, with different impact angles and velocities.
Barrier Friction Coefficient	A value of 0.25 is used for steel barriers and a value of 0.40 is used for concrete barriers. This imposes a friction force on the traveling vehicle in a direction parallel to the barrier, numerically equal to the product of the impact force and the barrier coefficient of friction.
Number of Barriers	A value of either 1 or 2 is input to denote the number of barriers present in the simulation. Where equal to 2, a barrier is present on either side of the pair of tracks. In this case it is assumed that the tracks are 4.6 m (15 feet) apart from centerline to centerline.
Distance to Near Barrier[1]	Distance (ft) from the track centerline to the face of the barrier on the left side of the tracks.
Distance to Far Barrier	Distance (ft) from the track centerline to the face of the barrier on the right side of the tracks.
Number of Cars	The total number of cars in the consist behind the point of initial derailment. TBIP assumes that the train separates at the derailment, with the cars in front of the point of derailment (if any) not influencing the derailment. This assumption is based on the hypothesis that the couplers between derailed cars and preceding cars break upon derailment so that the preceding cars cannot influence the movement of the derailed cars. With freight trains, typical derailments occur in mid-consist, however with high

[1] Note: the TBIP program uses English units. The primary units shown here, therefore, are English units, not metric.

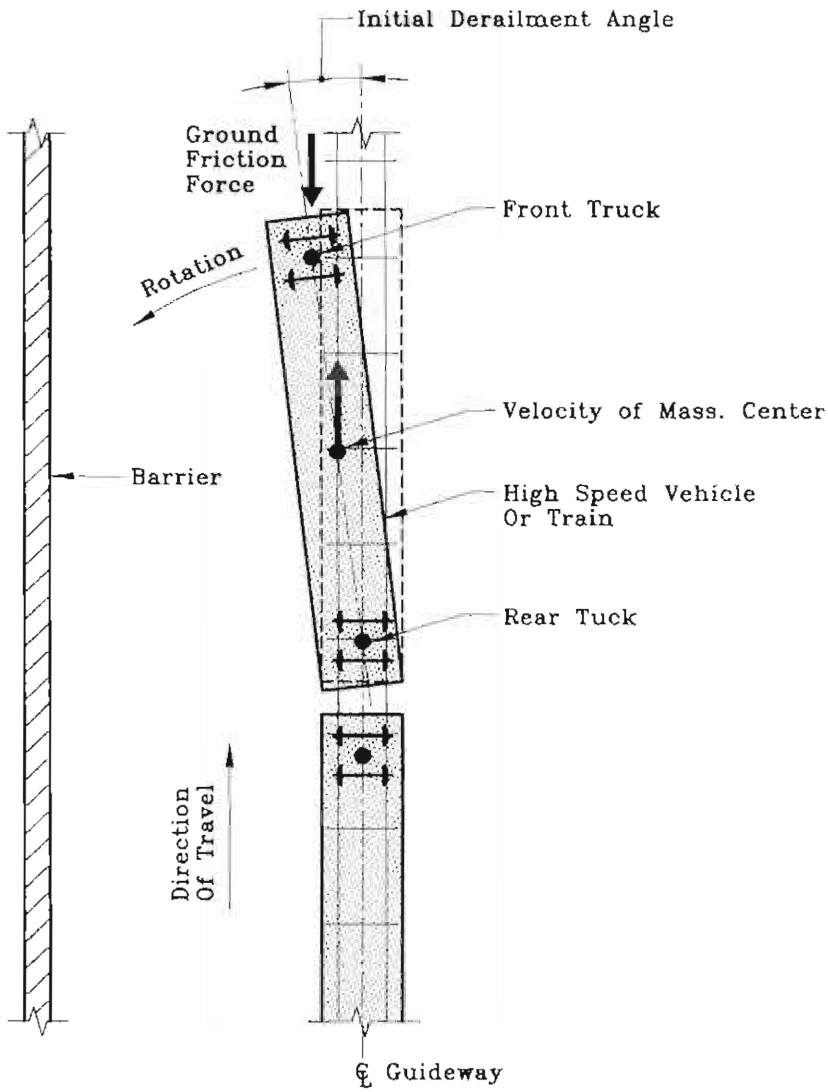

FIGURE 3-3. PLAN VIEW ILLUSTRATING INITIAL DERAILING ANGLE

speed vehicles, derailments of leading cars (power cars) are assumed. Therefore, the number input is the total number of cars of all types in the consist.

Ground Friction Coefficient

A simple friction model is used to compute the ground friction forces on the derailed trucks. Although a constant value of ground friction is not a completely accurate representation of the discontinuous impacts of vehicle wheels, brake discs and various other components with rails, guard rails, ties, etc., it has been found to yield results that agree well with observed accident data. Values of ground friction have been obtained by calibration of the model against the June 21, 1970 Crescent City, Illinois derailment of a 90-car, mixed freight consist, and the June 19, 1987 Washington, DC derailment of a 135 uniform freight car consist. In the reported simulations, a value of ground friction of 1.0 to 1.5 was employed with results in close agreement with the number of derailed cars and the maximum longitudinal distance of travel of the derailed cars actually observed in the actual accidents. With HSR derailing velocities, it is speculated that a lower value of ground friction might be appropriate; i.e., if "hydroplaning" of wheels over soil occurs. Without other evidence or data, a value of 1.0 is retained for the present.

Velocity

Velocity of the train at the instant of derailing (ft/sec).

Coupler Parameters

Coupler moment-rotation characteristics are developed for three types of coupler connections used in freight cars, corresponding to variations of typical E-type and F-type couplers: EE, EF and FF [2]. The moment-rotation curves were obtained from static test data provided by an earlier study [5]. Input is provided in the input dataset corresponding to the appropriate type of coupler (EE, EF, FF for freight). For high speed vehicles, coupler parameters are modified to better approximate the stiffness and strength characteristics of these vehicles, based on available data. These modifications consist of revised coupler stiffness and reduction in the amount of unrestricted coupler displacement and swing angle. Two types of couplers are evaluated: for articulated consists, like the TGV vehicle, and for non-articulated consists, like the ICE vehicle.

Brake Parameters

Yang et al. developed a model of braking representative of the automatic emergency application of air brakes to freight trains which have suffered a mid-consist separation and subsequent derailment of the rearward portion of the consist. The model includes a time delay for application of brakes and a speed-dependent braking force which is applied to each car, independent of its weight. This resulting braking force is applied to those cars still on the tracks (i.e. behind the point of derailment).

The following data are read once for each car in the consist.

Car Mass Moment of Inertia The mass moment of inertia (I) of the car about its vertical axis through the car's mass center ($slug\text{-}ft^2$). The usual assumption is that the mass center is at the geometric center of the car, and $I=ml^2/12$ (where m = mass and l = length of car); i.e., the car is modeled as a uniform bar.

Mass (M) Car mass (slug).

Length (L) Length of car (ft).

Wheelbase (W.B.) Wheelbase, or distance between truck kingpins (ft). It is assumed that trucks are the point of action for ground friction forces and that kingpins are equidistant from the car center.

Crush Stiffness (SKCB) A simple linear spring model is used to represent the interaction of a barrier and impacting car. The crush stiffness is the force required to crush a corner of a railroad or high speed vehicle car one foot in a collision. Appropriate spring properties are used as suggested by a method proposed by Emori [7], supported with data developed in crash testing various vehicles into an instrumented wall [8].

Emori's theory has been applied to develop estimates of appropriate spring constant, based on empty vehicle mass. Values of 1168 to 1752 kN (80 kip/ft to 120 kip/ft) are used for typical freight cars.[1] Values for HSGGT vehicles are indicated in Table 3-1. This is based on 1.5 and 1.7 times empty weight of power car and coach respectively. Further discussion on corner crush stiffness is provided in Section 3.1.2.

The program simulates the kinematics of the dynamic system and outputs certain information about the derailment. Quantitative output includes the critical value of maximum predicted barrier impact forces (oriented perpendicular to the barrier). A sample output file is presented in Figure 3-4. Analysis of the simulation results involves viewing a graphically displayed slow-motion plan-view depiction of the derailment and barrier impact, as shown in Figure 3-5.

The resulting derailment pattern indicated in the output is examined for the possible presence of numerical instability or other behavior contradictory to any of the various simplifying assumptions upon which the model is based. In particular, it is possible for the model to indicate that two cars can jackknife such that the two cars will pass through each other. Any such observed response triggers the need for a more thorough analysis of the simulation to supplement the TBIP analysis. Finally, the simulation yields information about how many cars, and what length of barrier are involved in the incident, allowing estimates of the extent of damage to both the train and the barrier.

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[1] The unit "kip" is equivalent to 1,000 pounds

TABLE 3-1. PHYSICAL PROPERTIES OF VARIOUS HIGH SPEED CONSISTS

	ICE (Express)	*TGV (Atlantique)*	*MAGLEV (Transrapid 07)*
Classification	HSR [non-articulated]	HSR [articulated]	Maglev
Service Speed ***Max. Test Speed*** ***Design Speed**, km/h (mph)*	250 (155) [1,2] 406 (252) [2] 322 (200) [1]*	300 (186) 515 (320) 322 (200)*	401 (249) 483 (300)*
Consist	one 11 to 14-car trainset [1,4]	one/two 10 to 12-car trainsets	one 6-car trainset
Trainset *PC=power car* *CC=coach car*	PC+9CC+PC PC+10CC+PC PC+12CC+PC	PC+8CC+PC PC+10CC+PC	6PC/CC
***Wt.-Power Car**, kg (lb)*	85,900 (189,200) [2]	74,600 (164,200)	same as coach
***Max. Power Car Axle Load**, kg (lb)*	19,500 (43,000) [2]	17,000 (37,400)	N/A
***Power Car Crush Stiffness**, kN/m (lb/ft)*	4,086 (280,000) [est.]	3,576 (245,000) [est.]	2,481 (170,000)
***Wt.-Coach Cars** (empty), kg (lb)*	50,000 (110,000) [2]	30,000 (66,000) [est.]	51,730 (113,806)
***Wt.-Coach Cars** (loaded), kg (lb)*	55,000 (121,000) [est.]	33,680 (74,100)	59,000 (129,806)
***Coach Car Crush Stiffness**, kN/m (lb/ft)*	2,481 (170,000) [est.]	1,635 (112,000) [est.]	2,481 (170,000) [est.]
Passengers/Car	60 (avg.) [1]	48.5 [avg.], 485 total for 1+10+1 cars	100 48/66
***Power Car Length**, m (ft)*	20.57 (67.5) [1] 20.16 (68) [2], 20.27 [66.5]*	22.16 (72.7)	25.5 (84)
***Power Car Wheelbase**, m (ft)*	11.46 (37.6) [1,2] 11.58 (38)*	14.0 (45.93)	12.8 (42)
***Coach Car Length**, m (ft)*	26.4 (86.6) [1,2] 26.21 (86)*	18.7 (61.35) Typ. 21.84 (71.65) @ PC	25.5 (84)
***Coach Car Wheelbase**, m (ft)*	19.0 (62.3) [1] 18.9 (62)*	18.7 (61.35)	12.8 (42)
***Izz-Power**, $kg\text{-}m^2$ ($slug\text{-}ft^2$)*	3.00x10[illegible] (2.165x10^{6})[est.]	3.11x10[illegible] (2.246x10^{6}) [est.]	3.28x10[illegible] (2.370x10^{6}) [est.]
***Izz-Car**, $kg\text{-}m^2$ ($slug\text{-}ft^2$)*	3.20x10[illegible] (2.316x10^{6})[est.]	1.00x10[illegible] (0.722x10^{6})[est.] 1.36x10[illegible] (0.984x10^{6})[est] PC	3.28x10[illegible] (2.370x10^{6})[est.]
***CG Height**, m (ft)*	1.65 (5.42) [power], 1.67 (5.48) [coach]	1.47 (4.81) [power], 1.37 (4.50) [coach]	0.85 (2.79)
***Floor Height**, m (ft)*	1.22 (4.00)	1.22(4.00) [est.]	0.82 (2.69)
***Car Width**, m (ft)*	3.02 (9.92) [typ.]	2.82 (9.25)	3.70 (12.14)
***Car Height**, m (ft)*	3.83 (12.58) [typ.]	4.10 (13.45) [power], 3.56 (11.67) [coach]	4.06 (13.32)
Coupler Coeff.	-0.702, 1.670, -0.720	-0.702, 1.670, -0.720	-0.702, 1.670, -0.720
***Braking Coeff.**, %*	5.50	5.50	5.50
Information Sources	[1] Texas Fastrack [2] Bing 1990 * Selected for Design	Bing 1990, SNCF, Texas TGV * Selected for Design	Bing 1990 Hadden et al. 1992

					Comments
ICE 1P+12CC+1P 189200 POWER; 110000 COA					*Information on consist (See Figure 3-2 for explanation of terms).*
14 12.000 -500.000 295.160					*14 cars, 12.000' near barrier offset, -500.000' far barrier offset (in this case far away to simulate single barrier), 295.160 fps velocity*
66.50					*Wheelbase of first car*
80.00					*Wheelbase of second car*
80.00					*Wheelbase of third car*
80.00					*Wheelbase of fourth car*
80.00					*Wheelbase of fifth car*
80.00					*Wheelbase of sixth car*
80.00					*Wheelbase of seventh car*
80.00					*Wheelbase of eighth car*
80.00					*Wheelbase of ninth car*
80.00					*Wheelbase of tenth car*
80.00					*Wheelbase of eleventh car*
80.00					*Wheelbase of twelfth car*
80.00					*Wheelbase of thirteenth car*
66.50					*Wheelbase of fourteenth car*
.00 1					*Time, No. of derailed cars (Time = 0, No. of derailed cars = 1)*
.05000	1026.458	1.662	.000	.000	*Output: Column 1 - angle of car axis to track (radians)*
.00000	953.250	.000	.000	.000	*Column 2 - car location along track (feet)*
.00000	873.250	.000	.000	.000	*Column 3 - car location perpendicular to track (feet)*
.00000	793.250	.000	.000	.000	*Column 4 - impact force on left barrier (pounds)*
.10 1					*Column 5 - impact force on right barrier (pounds)*
.05003	1055.821	1.662	.000	.000	
.00000	982.744	.000	.000	.000	
.00000	902.757	.000	.000	.000	
.00000	822.757	.000	.000	.000	
.20 1					
.05037	1084.956	1.663	.000	.000	
.00000	1012.086	.000	.000	.000	
.00000	932.221	.000	.000	.000	
.00000	852.244	.000	.000	.000	*(... Output Truncated ...)*
3.20 12					*Maximum barrier force occurs at time = 2.80 seconds:*
.17966	1903.285	-6.291	.000	.000	*12 cars derailed*
-.21230	1831.346	-3.589	.000	.000	*maximum barrier force = 216,179.059 pounds*
.22150	1752.898	-4.139	.000	.000	*car number 5*
-.25800	1674.956	-2.742	187316.632	.000	
.15909	1597.047	1.170	216179.059	.000	
-.09696	1517.997	-1.383	.000	.000	
.04634	1438.239	.731	.000	.000	
-.02088	1358.422	-.334	.000	.000	
.00878	1278.507	.191	.000	.000	
-.00310	1198.987	-.066	.000	.000	
.00097	1119.384	.021	.000	.000	
-.00001	1039.758	-.000	.000	.000	
.00000	960.147	.000	.000	.000	
.00000	887.238	.000	.000	.000	*(... Output Truncated ...)*
10.95 14					*Consist comes to rest at time = 10.95 seconds*
.37299	2826.185	-23.801	.000	.000	*Results indicate final configuration*
-.20138	2755.940	-27.905	.000	.000	
.52611	2682.158	-40.008	.000	.000	
-.57058	2613.939	-38.492	.000	.000	
.39673	2543.317	-32.377	.000	.000	
-.31094	2468.512	-35.633	.000	.000	
-.00055	2390.513	-23.419	.000	.000	
-.16895	2311.003	-16.620	.000	.000	
-.01285	2231.748	-9.375	.000	.000	
-.17704	2152.415	-1.815	.000	.000	
.09187	2073.408	1.581	.000	.000	
-.02683	1993.644	-1.001	.000	.000	
-.00175	1913.686	.142	.000	.000	
.00486	1840.248	.051	.000	.000	
99999 14	216179.059	3.103			

FIGURE 3-4. SAMPLE TBIP OUTPUT *(Note: program output is in English units)*

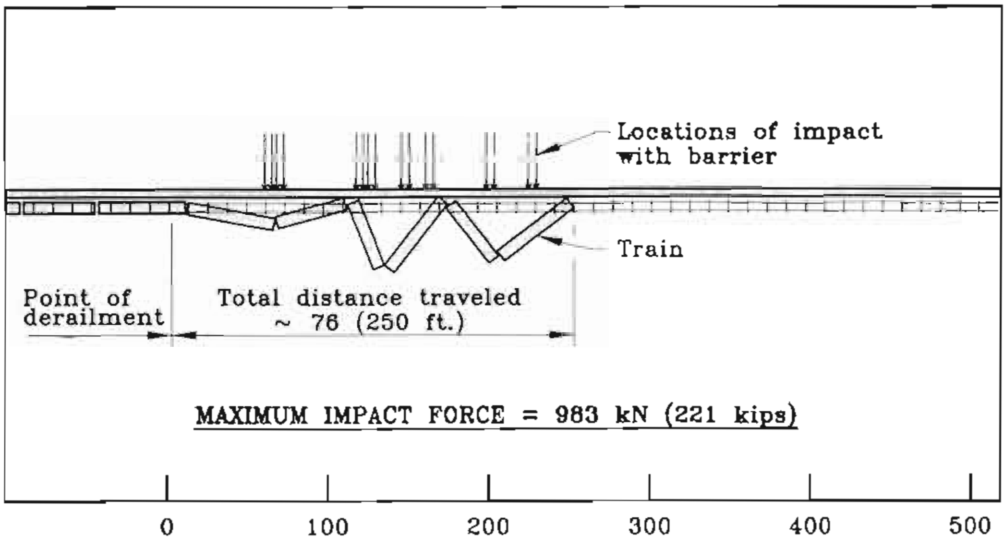


FIGURE 3-5. SAMPLE TBIP OUTPUT DISPLAY

3.1.2 Assumptions

The TBIP program is based on a number of assumptions in addition to those discussed above. It is important to understand these assumptions in order to properly assess the limitations of the program. As described in Section 7.5, these assumptions should be verified in the future with testing and/or analytical means. The critical assumptions are described below.

2-D Motion The most important premise of this model is that the predominant motion occurs in the horizontal plane. In actuality, a derailed train behaves in three dimensions. Each car can move horizontally (both longitudinal and transverse), vertically, and it can rotate in three directions. The model does not account for vertical movement, nor rotation about any horizontal axis. Supplemental calculations indicate that out-of-plane movements, such as rotation about the horizontal axes of the car contribute a negligible amount of energy to the system. Nevertheless, allowance was made in the design of the barriers for three-dimensional effects (See Section 3.1.3.2).

Energy Losses: All energy losses occur in the form of ground friction and barrier friction. It is assumed that there is no energy loss due to crushing of the vehicle. Research has shown that this energy loss from vehicle crushing is less than 5% for automobiles and freight trains and it should also be low for high speed vehicles.

Barrier Stiffness: No distinction is made in the model between stiff (concrete) barriers or flexible (steel) barriers. Other uncertainties in the model make this refinement meaningless. Vehicle stiffness, ground friction and other parameters are more variable than the range of barrier stiffnesses that would result from this refinement.

Effect of Rail: Once the vehicle is derailed, the effect of the rail is ignored. Any energy losses resulting from impact with the rail and ties, or jumping the rail have been included in the single term for ground friction.

Vehicle Deformation: Changes in vehicle and barrier geometry due to crushing or deformation are not taken into account in later collisions. Throughout the derailment and collision process, the length, width, mass and moment of inertia are assumed to be constant.

Track Curvature: A tangent track is assumed. The radius of curvature is so great for high speed vehicles compared to derailing distances, that the difference between tangent and curved track is negligible. For example, the minimum radius for 320 km/h (200 mph) high speed rail vehicle is approximately 6000 m (19,900 ft.). The distance from derailment to barrier impact is on the order of 100 to 150 m (325 to 500 feet) according to the TBIP runs. This results in an angular difference of less than 1.5 degrees at the point of impact. The total centrifugal force of 26.7 kN (6 kips) resulting from this radius of curvature is also insignificant compared to the barrier impact force of 980 kN (220 kips).

Airborne Motion: It is assumed that once derailing has been initiated, the vehicle remains on the ground, and frictional forces are applied through the trucks throughout its travel. While it might be theorized that an incident could occur resulting in a derailing car striking the barrier while airborne, such incidents would be unlikely and have not been studied. The use of large initial derailing angles in effect simulates partial airborne movement of the front trucks.

Stiffness Model: One of the most critical assumptions is that barrier/car stiffness can be modeled based on a linear force-displacement relationship. Stiffness is the value of force divided by displacement. This is a linear relationship when stiffness is constant over all ranges of displacements. This concept, originally employed by Emori (1968) [7] to model head-on collisions of automobiles with rigid objects, does not model the energy lost due to plastic crushing, and therefore predicted times and total distance traveled are expected to be higher than actual, other factors being equal. Nonetheless, it is believed that this simplifying assumption can be effectively used to predict peak barrier forces, if an appropriate value is selected for the slope of the force-displacement relationship. The difficulty is that little data exists that allows direct calculation of an equivalent force-displacement relationship.

Emori, however, hypothesized that stiffness was a function of the weight of the vehicle. He developed an empirical relation between the weight of an impacting automobile and the appropriate stiffness, given by:

$$k = (12.5\ ft^{-1})W(lb)$$

where k is the stiffness in kips/ft, and W is the mass of the automobile in kips. In various subsequent studies, as summarized by Hirsch et al. [3] and by DeLeuw, Cather [4], it was further hypothesized that this relationship can be extended to heavier highway vehicles, up to and including 18,200 kg (40,000 lb) buses and 36,300 kg (80,000) tractor-trailer combinations, by introducing a variable coefficient A which depends on the *empty* vehicle weight W_e in kips:

$$k = A(W_e)$$

A series of tests was conducted by TTI (Beason and Hirsch 1989) [8] which allowed the determination of $A(W_e)$ for numerous vehicles. The results of the tests are summarized in Figure 3-6. From this figure it is concluded that the parameter A decreases with increasing vehicle empty weight.

In the absence of a more defensible method, values of the stiffness k for high speed vehicles and freight trains were extrapolated from Figure 3-6, based solely on their weight. It should be noted that these stiffness values are higher than those used for the freight cars because the empty weight of the ICE cars is greater than the empty weight of freight cars. The validity of the assumptions leading to these values has not been demonstrated, and in fact cannot be demonstrated convincingly short of a full-scale test.

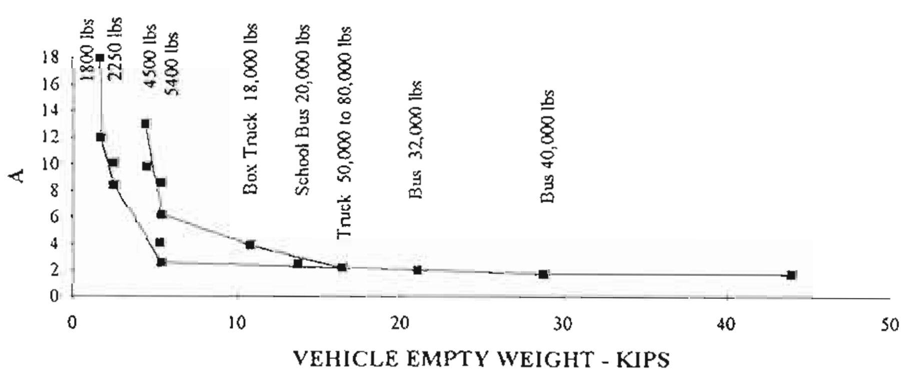


FIGURE 3-6. VEHICLE STIFFNESS VS. EMPTY WEIGHT *($K = A\ W_{empty}$)*

Note: Metric Equivalent 1 kip= 454 kg

The linear elastic model of car corner-barrier interaction is a gross simplification of a process which is inelastic and nonlinear. The basis for this simplification is the traditional use of such a simplified model (in highway barrier analysis) and the lack of required data for more sophisticated models. Emori's method for simplification of a motor vehicle model has been used in a modified fashion by TTI engineers for comparative analysis of crash test data and for analysis and design of highway barriers, where peak crash loads are of primary concern. The experience gained in the use of this model is a significant factor in the application of it in TBIP.

During the present study of high-speed rail vehicles, three numerical models of representative HSR vehicle structures have been discovered. The first model, the more pertinent of the three, is a model used to simulate an actual collision that occurred between a TGV train and a large machine tool which the train struck at a grade crossing. Not all details of the accident or the resulting modeling effort have been released to the researchers in the present study because of their proprietary nature. From the available details, which are not reported here, again because of their proprietary nature, the modeling effort appears to be a simulation of a direct, head-on impact. The impact force estimated by the model is plotted against displacement, or crushing distance that the vehicle experiences. The load-displacement curves developed to represent the elements of the TGV train presumably are based on longitudinal crushing of the main longitudinal structural members of the locomotive and cars. Consequently, the components have stiffnesses (stiffness = slope of load displacement curve) much higher than those predicted using the modified Emori model.

The second pertinent model was developed by IABG[1] to simulate a Maglev Transrapid-07 during direct head-on impact with a rigid wall (or

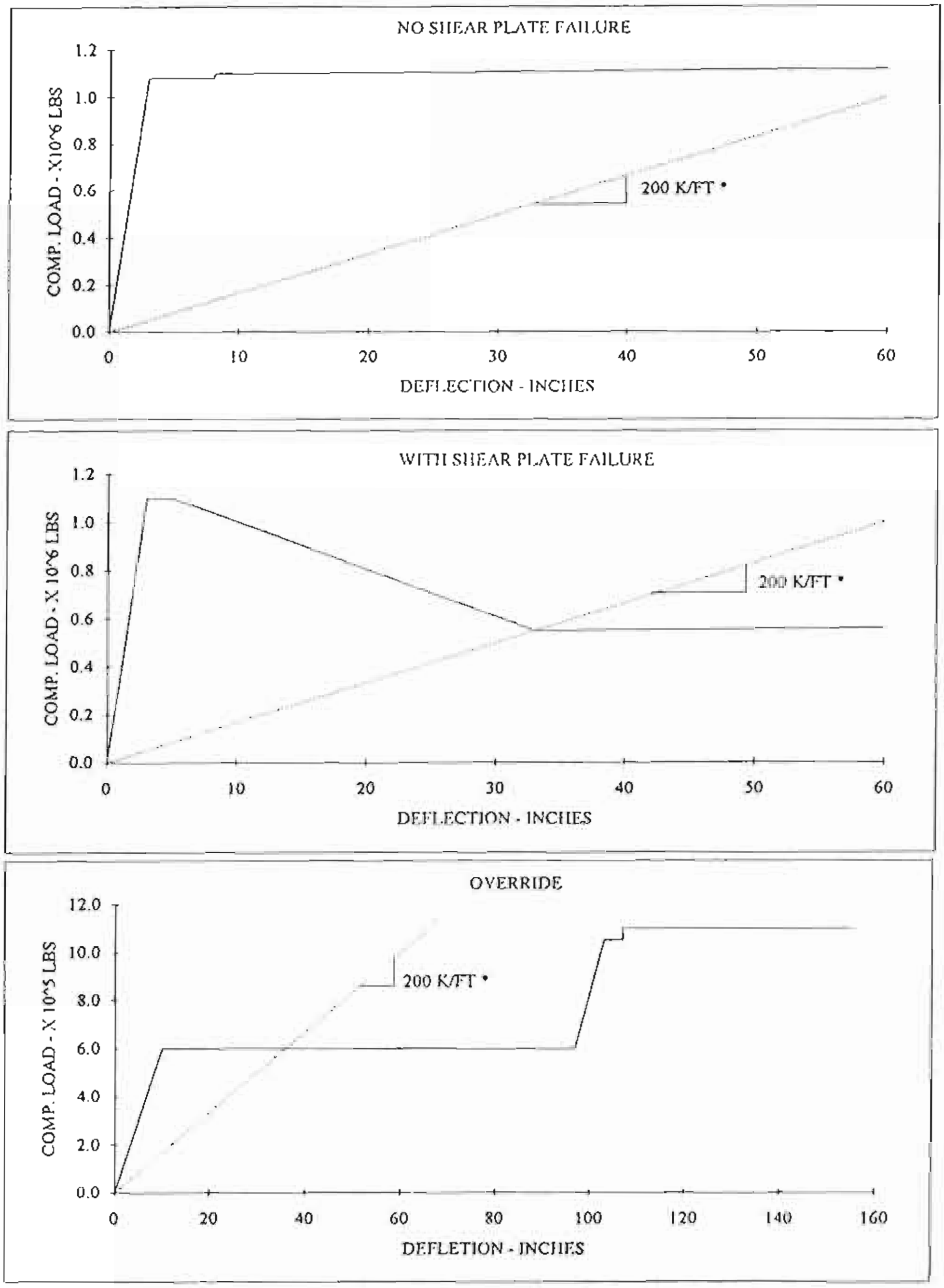
[1] IABG: Industrieanlagen-Betriebsgesellschaft mbH, a German firm involved in the development of the Transrapid-07 Maglev vehicle.

an identical train). The model is a finite element model taking into consideration buckling of the longitudinal structural members under dynamic loading. The car is assumed constrained by the guideway, which it envelopes, so gross buckling of the car is prevented. The assumed static load displacement curve differs significantly from that used in the TGV model in that a large stiffness is immediately experienced, followed by a rapid loss of load-carrying ability due to the buckling of the frame members. This data is not presented because of proprietary interests.

A third model which is reported in the literature [9] involves three force-deflection curves for a double deck "Highliner" railroad car. Each of three structural response conditions are represented, all involving longitudinal crushing with or without failure of an associated shear plate and with or without override. Detailed structural analyses were performed to determine force-displacement relationships resulting from a head-on collision. While the car modeled is probably not representative of the types of vehicles being studied in the present study, several aspects of the model are pertinent. Figure 3-7 shows the reported overall static axial force-deflection characteristics for the Highliner under the three failure modes considered.

For comparison, it is possible to arrive at an "equivalent" simplified linear elastic model from the parameters in either of the three models discussed above. This is done by determining the spring constant parameter in the simplified model such that the energy of structural deformation is the same under the two curves, up to some specified displacement. That is, the area under the load-displacement curve is equated to that of a curve with constant slope. This slope is the average stiffness for that range of displacements. Table 3-2 indicates the equivalent spring constant for the three models obtained in this way for two specified displacements.

All the equivalent spring constant values calculated in this manner are significantly higher than the typical values of 1,635 kN/m to 4,090 kN/m



* *Value shown (*) (200 k/ft) is approximate value used in TBIP analysis, based on Emori approach*

FIGURE 3-7. STATIC AXIAL FORCE-DEFLECTION CHARACTERISTICS OF A "HIGHLINER" PASSENGER CAR [10]

Note: Metric Equivalent 1 inch= 25.4mm
10^6 lb= 4.45 MN

TABLE 3-2. APPROXIMATE EQUIVALENT SPRING CONSTANTS FOR SIMPLIFIED MODEL OF "HIGHLINER"

Comparison Model		Specified Crush Deformation 0.15 m (0.5 ft)	Specified Crush Deformation 0.30 m (1.0 ft)
1.	TGV Locomotive	46,700 kN/m (3,200 kip/ft)	32,120 kN/m (2,200 kip/ft)
2.	Transrapid-07 (lower portion of car structure)	15,180 kN/m (1,040 kip/ft)	5,260 kN/m (360 kip/ft)
3.a.	Highliner	64,240 kN/m (4,400 kip/ft)	10,220 kN/m (700 kip/ft)
3.b.	Highliner w/ shear plate failure	64,240 kN/m (4,400 kip/ft)	24,820 kN/m (1,700 kip/ft)

(112 to 280 kip/ft) suggested for the TBIP application for the vehicles studied. A significant difference in modeling objective is noted: the TBIP attempts to model an impact of the car corner, while the models compared above are based on axial crushing. The load-displacement characteristics of a crushing car corner will intuitively reflect a softer structure. First, the hard points representing the ends of the main longitudinal members will not be engaged immediately, as an axial impact, but only after some significant deformation of the car corner, depending on impact angle. Secondly, the oblique loading will probably result in lower buckling resistance than that exhibited by the main longitudinal structural members located toward the middle of the car. The quantitative effect of these factors is of course unknown.

Only two methods can practically be used to quantify the oblique crushing characteristics, and both are outside the scope of the present limited study. A finite or discrete element analysis of the car corner structure was contemplated, but uncertainties in car materials and structural details (for future designs) and effects of dynamic loading rates led to the abandonment of this proposal. Full-scale crash testing could allow accurate determination of the crush characteristics, and in fact such testing would probably be required to calibrate any finite element model developed. Until such test data can be obtained, it is recommended that the values for the model parameter be determined using the modified Emori approach. Since the predicted barrier forces depend strongly on the value used for the model parameter, such testing should be required to validate any barrier design contemplated (See Section 7.5.1).

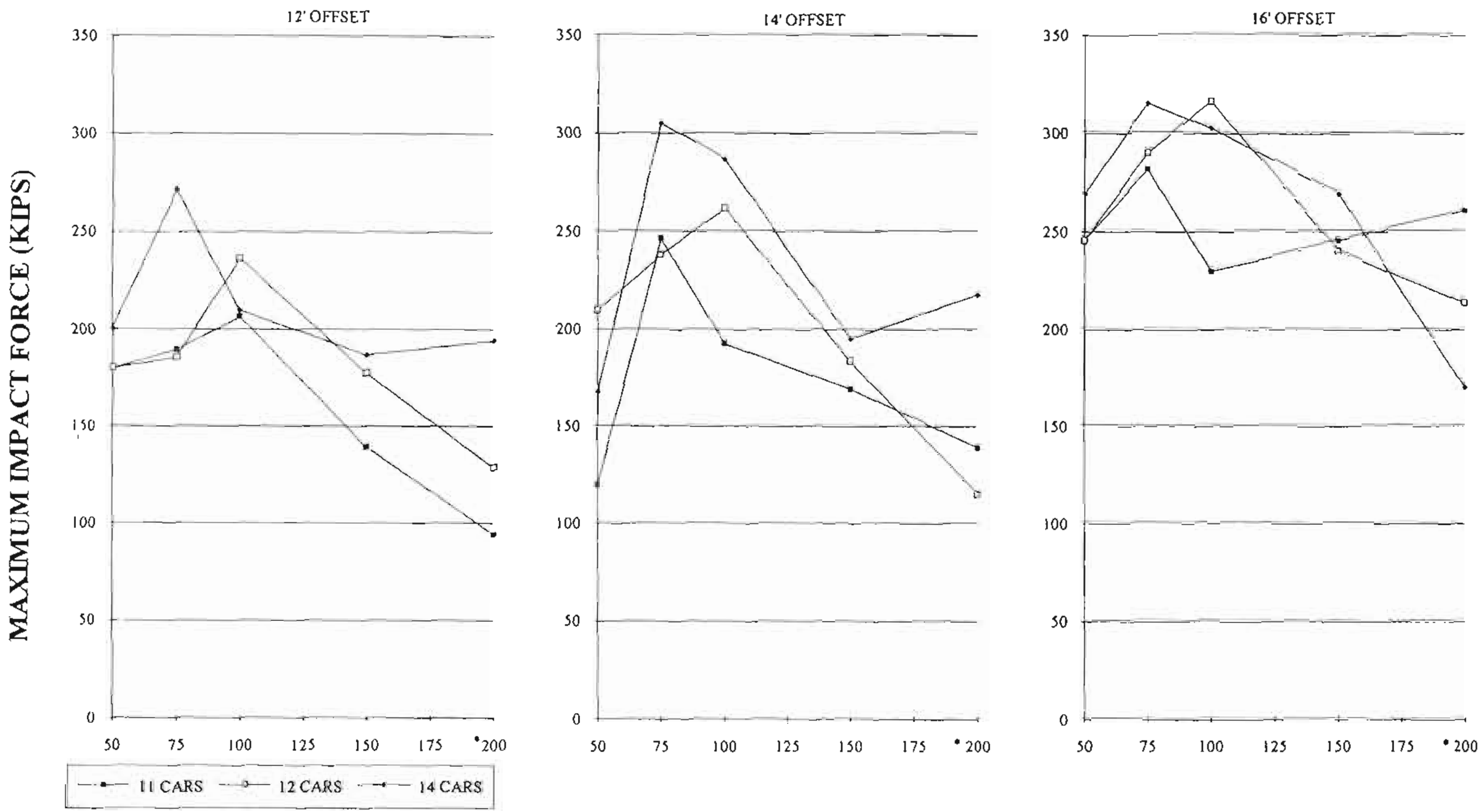
3.1.3 Findings

3.1.3.1 Parametric Study for HSGGT Vehicles

The impact load generated by the TBIP model is affected by many different parameters, such as vehicle speed at derailment, car and barrier stiffness, ground and barrier friction, barrier location, braking coefficient, coupler properties, number of cars in a consist, initial derailing angle, and number of barriers. Some of the parameters are well documented and values easily assigned. Others were not so clear at the beginning of the study, and it was unknown what effect the values assumed for these parameters might have on the results. Because of the large number of scenarios to be evaluated, and the intention that study results be representative of the full range of possible installations, it is important to understand what values yield conservative and reasonable results. A parametric study has been undertaken, using the ICE vehicle, to help understand the uncertainties. The results allow for the judicious selection of parameters for modeling of other vehicles and eliminate the needless analysis of inconsequential cases.

The TBIP program has been run for various permutations of parameters. The program calculated the maximum barrier impact force, which has been plotted for the variations in parameter values (See Figures 3-8 through 3-15). The following effects have been noted for variations of each of the different parameters:

Vehicle Speed: Speeds varying from 80 km/h (50 mph) up to the ICE vehicle's top speed of 320 km/h (200 mph) have been studied for various barrier offset distances (horizontal distance from centerline of track to face of barrier) and consist lengths. A peculiar phenomenon is noted. Intuition would say that the highest speed would yield the highest impact load on the barrier. The opposite is observed. Figure 3-8 indicates that the highest loads occur at lower derailment velocities, below 160 km/h (100 mph).



*Base Run

SPEED (MPH)

FIGURE 3-8. IMPACT FORCE VS. DERAILING VELOCITY

(See Section 3.1.3.1 for explanation of results)

Note: Metric Equivalent 1 mph= 1.609 km/h
1 kip = 4.45 kN
1 ft = 0.305 m

It was originally theorized that the higher forces observed at lower speeds were a result of a greater angle of impact of the slower moving vehicle. Comparison of two cases where identical trains were modeled at different speeds illustrates why this is not the correct explanation. Examination of Figure 3-8 indicates an impact force of 1,206 kN (271 kips) at 120 km/h (75 mph), and 863 kN (194 kip) at 320 km/h (200 mph). The slower moving train does trace a trajectory on the ground plane with a greater angle of impact: however, the explanation for the higher forces at the lower speeds is not this simple. If this were the governing factor, then the peak forces for the 80 km/h (50 mph) impact would be expected to be higher than the forces in the 120 km/h (75 mph) impact, which is not the case. The difference in impact angle of the trajectory would influence the initial impact conditions, but as noted below, the initial impacts do not yield the highest impact forces in a derailment event. The higher velocity vehicle, however, does yield a higher initial impact force than that of the lower velocity vehicle.

A comparison of the two cases reveals that the peak impact forces occur at very different times and locations. In the case of the 120 km/h (75 mph) derailment, the peak impact force of 1206 kN (271 kips) is created by the impact of the fifth car about 7.2 sec after initial derailing. In the case of the 320 km/h (200 mph) derailment, the peak force of 863 kN (194 kips) occurs when the third car strikes the barrier about 2.8 sec after derailing. If the initial impacts of the first car in each consist are compared, the 200 mph initial impact force of 514 kN (115 kips) is seen to be significantly greater than the 120 km/h (75 mph) impact force of 227 kN (51 kips). Therefore, the explanation for higher peak impact loads in the developing 120 km/h (75 mph) collision is not straightforward.

The collision events must be studied as a whole, rather than simply comparing two single impacts from different times within each event. A study of the two entire events reveals differences in the observed movement and subsequent crashes. The higher speed collision differs from the lower speed collision in one significant way - the collision with the barrier ends comparatively more quickly, with the train cars rebounding from the barrier and traveling down-track without additional impacts with the barrier and in a shallow zig-zag pattern. By comparison, the slower speed collision does not exhibit such rebounding - the collision event is characterized by the cars remaining in contact with the barrier, with the zig-zag pattern being more exaggerated, and with the peak load occurring much later in the event. The difference in collision pattern, especially the steeper zig-zag pattern, is apparently significant. The two impact events could be characterized as a "glancing" blow for the higher speed event, as opposed to a "snagging" collision for the slower speed event.

Conclusion: The simulation should be run for all consists over a range of speeds to determine the maximum barrier load. It is this load which should be used in the barrier design.

Vehicle Crush Stiffness

Values of crush stiffness from 4,086/2,480 kN/m (280/170 kips/ft) to 146,000/87,600 kN/m (10,000/6,000 kips/ft) (power car/coach car) have been tested for various barrier offset distances (See Section 2.5) and consist lengths. The values of 4,086 and 2,480 kN/m (280 and 170 kips/ft) correspond to estimates made in accordance with Emori's methods. Values as high as 146,000/87,600 kN/m (10,000/6,000 kips/ft) have been tested because values in this range have been observed in a recent crash in Voiron, France (see Section 3.1.2).

Impact loads have been found to increase significantly with increased vehicle crush stiffness in all cases (See Figure 3-9). Increasing the stiffness 37 times, from 4,086 kN/m (280 kips/ft) to 46,000 kN/m (10,000 kips/ft), yields an increase in impact load of 4.9 times, from 1,161 kN (261 kips) to 5,725 kN (1287 kips). The force increases with stiffness at a rate of less than 0.5 to 1.

Conclusion: In the absence of better empirical data, Emori's method should be used. It is recognized that this important parameter can best be determined with full scale crash tests of actual high speed rail vehicles.

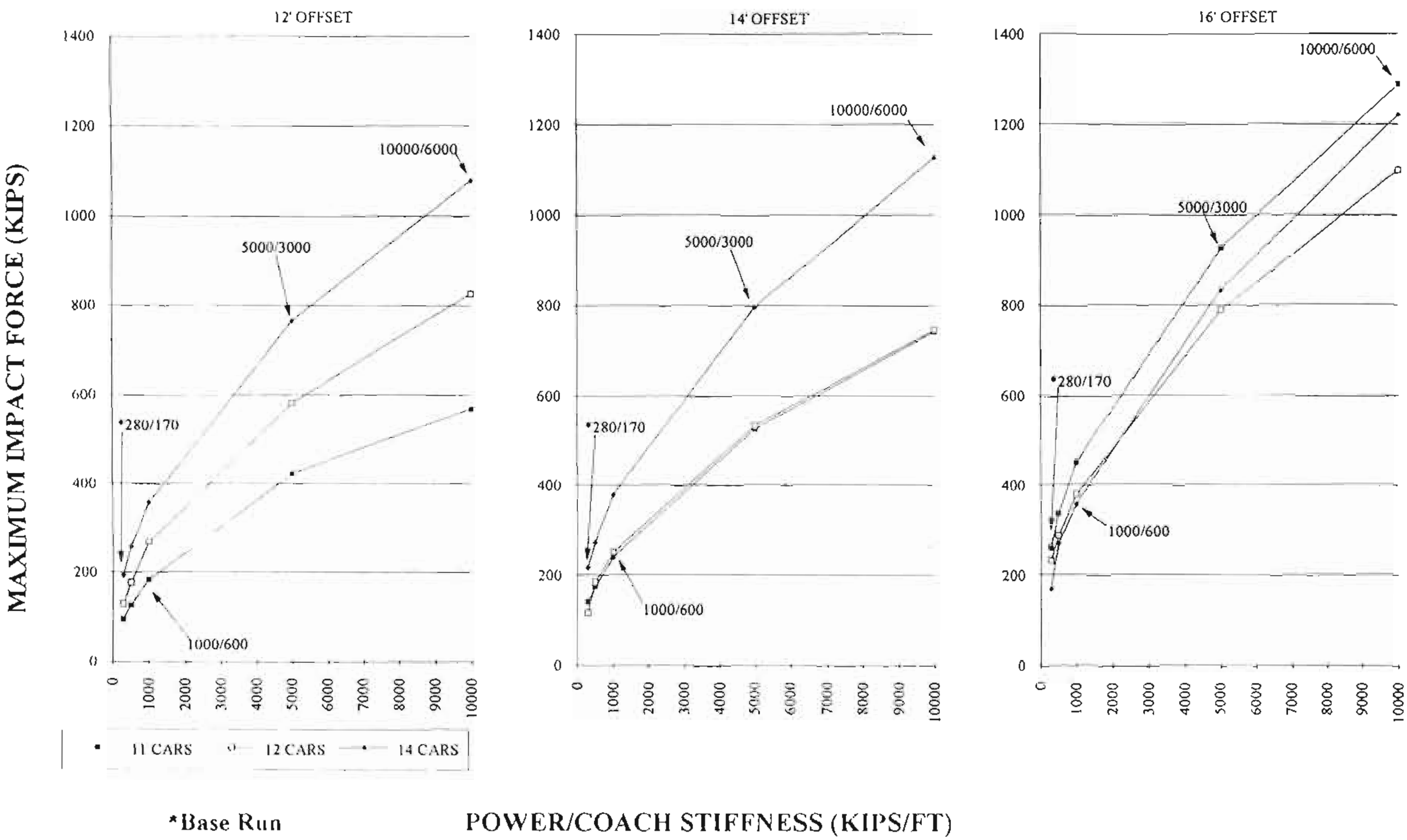


FIGURE 3-9. IMPACT FORCE VS. POWER COACH STIFFNESS

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip/ft= 14.59 kN/m
1 kip = 4.45 kN
1 ft = 0.305 m

Section 7.5.1 makes more specific recommendations for further study of vehicle crush stiffness.

Ground Friction

Values from 0.25 to 2.0 have been tested. Ground friction has a significant effect. Again, intuition proves incorrect. Greater values of ground friction increase the impact load, contrary to what one might expect, as indicated in Figure 3-10. Higher friction causes the cars to buckle into the accordion configuration more quickly, increasing lateral motion and forces.

Values traditionally used for highway vehicles vary from 0.75 to 1.1. Steel wheels would be expected to have a lower coefficient than rubber tires, but this could be offset by the wheels digging into the soil under high vertical load. As previously discussed in Section 3.1.1.1, a value of 1.0 was used for freight trains in a previous WMATA intrusion barrier study [3]. This yielded results corresponding well to actual accident data. A value of 1.0 is a reasonable approximation of the coefficient that might be expected.

Conclusion: A ground friction coefficient of 1.0 should be used.

Barrier Offset Distance:

Values of barrier offset distances (See Section 2.5) of 2.74 to 12.19 meters (9 to 40 feet) have been studied. Nine feet is considered a minimum value, since this would result in approximately 3 feet of clearance between the side of the car and the face of barrier. It is unlikely that barriers would be installed at distances greater than 40 feet.

The force exerted on the barrier is very sensitive to this parameter. Up to a point, greater impact forces are observed when barriers are situated at greater barrier offset distances (See Figure 3-11). This probably results from the cars' ability to achieve a more oblique impact angle, and therefore a larger force, when barriers are located further from the tracks. Impact force reaches a maximum at a certain value of barrier offset

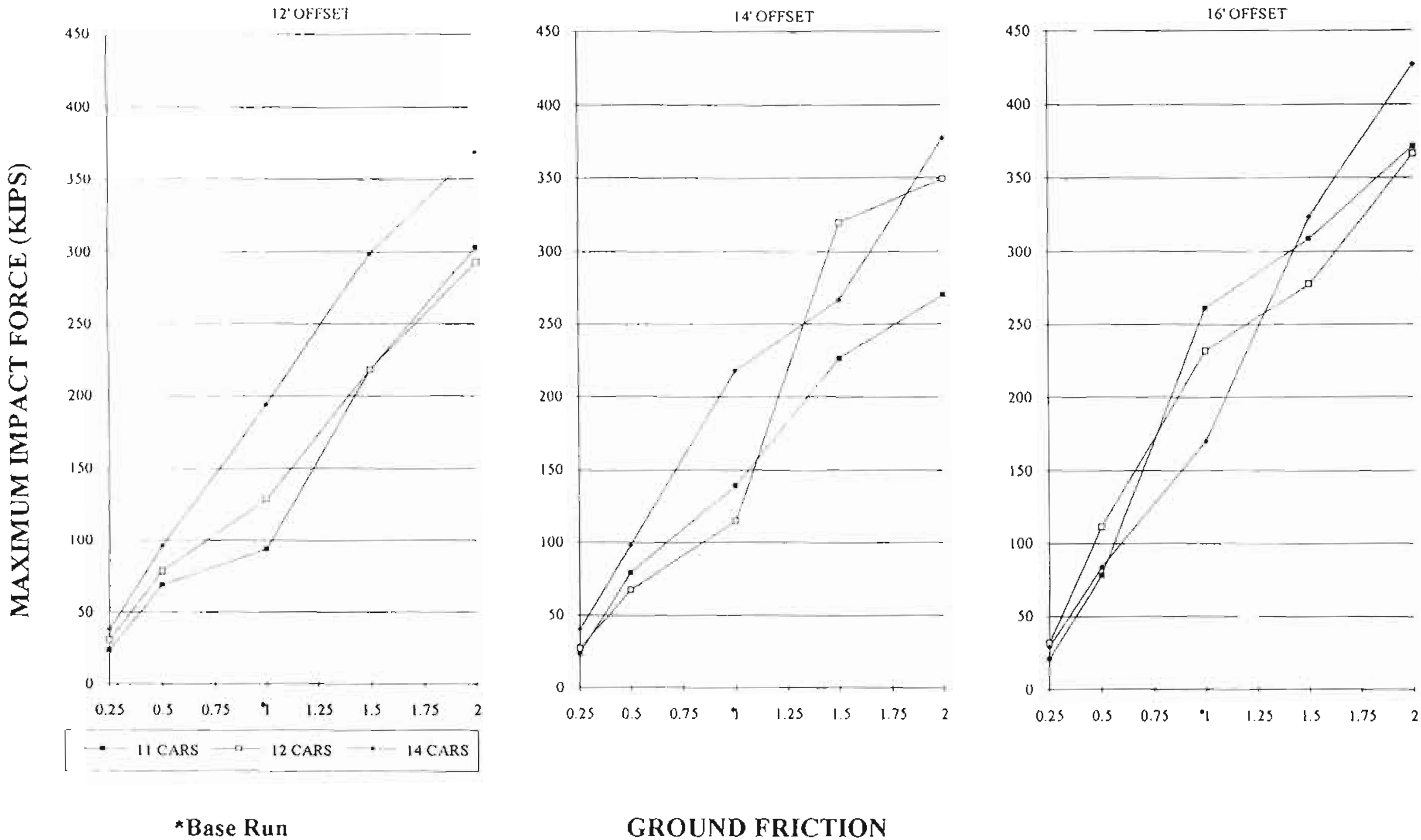


FIGURE 3-10. IMPACT FORCE VS. GROUND FRICTION

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip = 4.45 kN
1 ft = 0.305 m

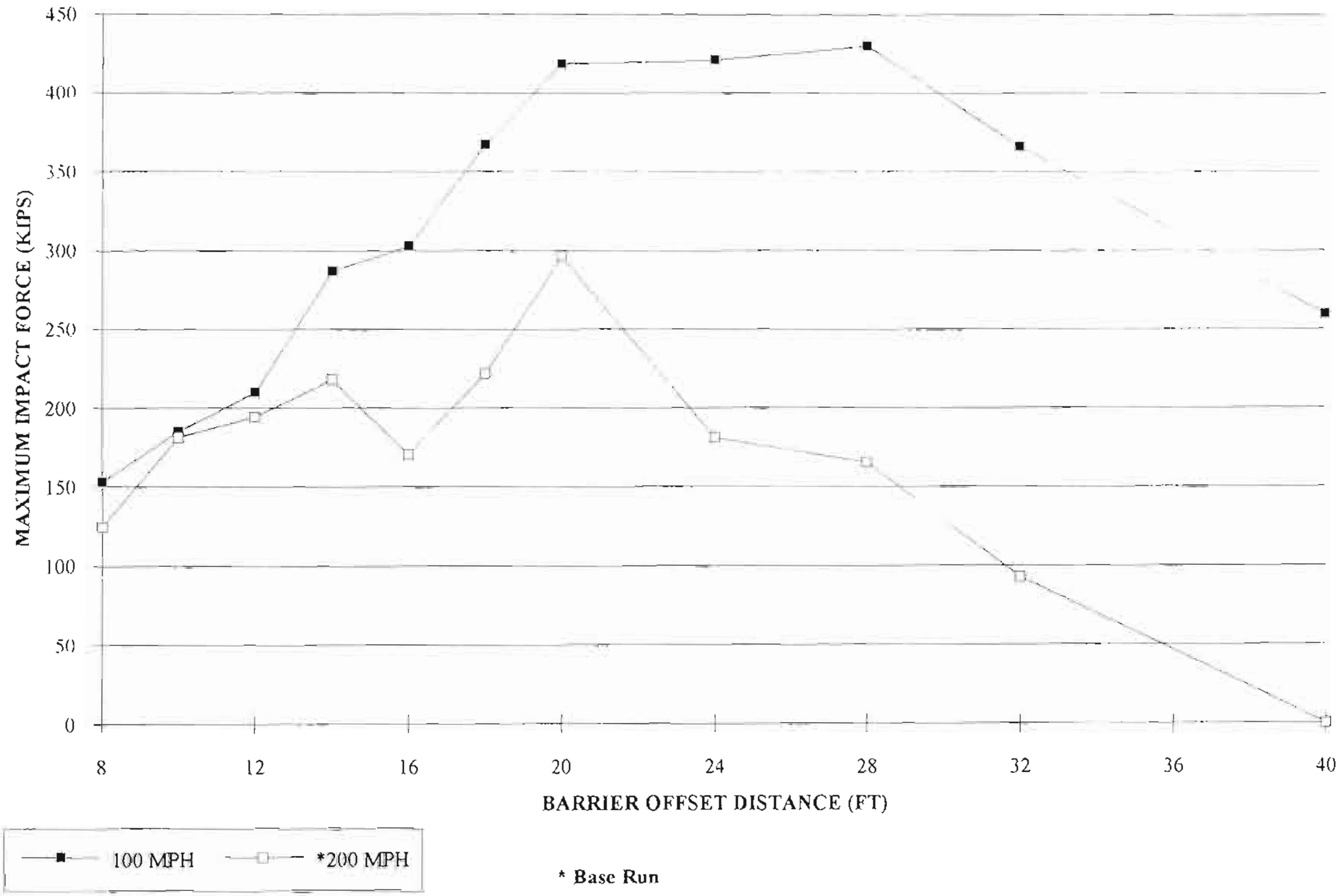


FIGURE 3-11. IMPACT FORCE VS. BARRIER OFFSET DISTANCE (GROUND FRICTION = 1.0, 14 CARS)

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip = 4.45 kN
1 ft = 0.305 m

distance, and is less at greater barrier offset distances where frictional forces become more predominant. Past work on the WMATA project [2,3] confirms this trend.

Similar results are observed using a ground friction coefficient of 0.50, but with lower impact forces (See Figure 3-12).

Conclusion: This parameter can be prescribed by design. For rail trackways, a barrier offset distance of 2.74 m (9 feet) should be used, in conformance with American Railway Engineering Association (AREA) standards. For Maglev guideways a barrier offset distance of 3.35 meters (11 feet) should be used. Impact loads corresponding to these barrier offset distances should be used for design.

Barrier Friction Coefficient: Values of 0.25 have been measured for steel barriers and 0.40 for concrete barriers. This parameter has very little effect, although higher barrier friction increases impact forces slightly.

Conclusion: Values of 0.25 should be used for steel and 0.40 for concrete surfaces.

Braking Coefficient: Values of braking friction coefficient of 5.5% to 7.0% have been measured. Variation of this parameter has very little if any effect.

Conclusion: A single value should be used, 5.5% is suggested.

Coupler Types: For freight trains, characteristics of three types of couplers have been tested: EE, EF, and FF. For high speed consists, coupler models have been modified to approximate the stiffness and strength characteristics for articulated and non-articulated vehicles. Impact forces are affected very little by the type of coupler.

Conclusion: EE, EF, and FF couplers will be used for freight trains. Coupler models will be modified appropriately to approximate the stiffness and strength characteristics of HSGGT vehicles.

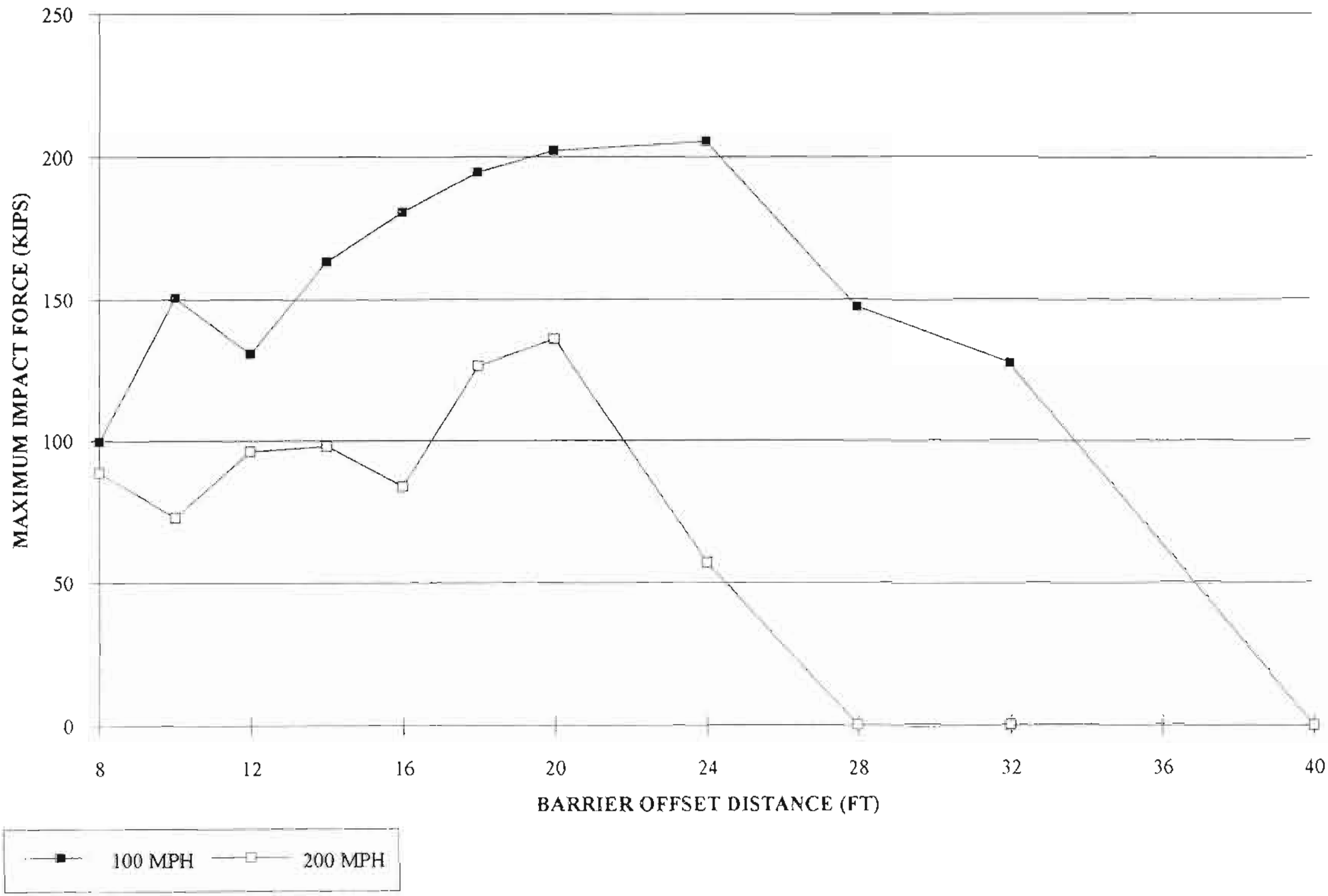


FIGURE 3-12. IMPACT FORCE VS. BARRIER OFFSET DISTANCE (GROUND FRICTION = 0.5, 14 CARS)

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip= 4.45 kN
1 ft = 0.305 m

Number of Cars:

Consists of 11 (i.e., 1 power car, 9 coaches, 1 power car), 12 and 14 cars have been studied.

Barrier force increases generally with the number of cars, with the 14-car trainset causing greater impact forces than the 11- or 12-car trainsets modeled, as indicated in Figure 3-8. This would be expected due to the increased total kinetic energy of the consist. Due to the somewhat random nature of the collisions, there are exceptions, and there is some scatter. It can be generalized, however, that a longer consist produces higher impact forces.

Conclusion: The longest anticipated consist with the most cars should be used for design purposes.

Initial Derailing Angle:

Initial derailing angles (see Section 3.1.1.1 and Figure 3-3) from 0.02 to 0.10 radians have been studied. For an ICE power car, the value 0.01 corresponds roughly to a lateral displacement of the front trucks equal to the combined width of the head of rail and flange of wheel (approximately 8"). This is thought to be a lower bound on realistic initial angles of derailing. An angle of 0.10 radians corresponds to approximately 1140 mm (45 inches) of lateral displacement of the front trucks. This is thought to be an upper bound on this parameter. For small track/barrier offset distances, the angle is constrained by the presence of the barrier (e.g., for a 2.74 meter (9 foot) barrier offset, the maximum angle is 0.02 radians - any higher and the vehicle overlaps the barrier).

The maximum angle before barrier overlap has been used, up to a maximum of 0.10 radians: 0.02 radians for a 2.74 meter (9 foot) offset or less, 0.05 for 3.0 meters (10 feet) to 3.66 meters (12 feet) and 0.10 for barrier offsets greater than 4.25 meters (14 feet). The larger initial derailing angles result in larger barrier impact forces, as would be expected (See Figure 3-13).

BARRIER OFFSET DISTANCE = 16 FT

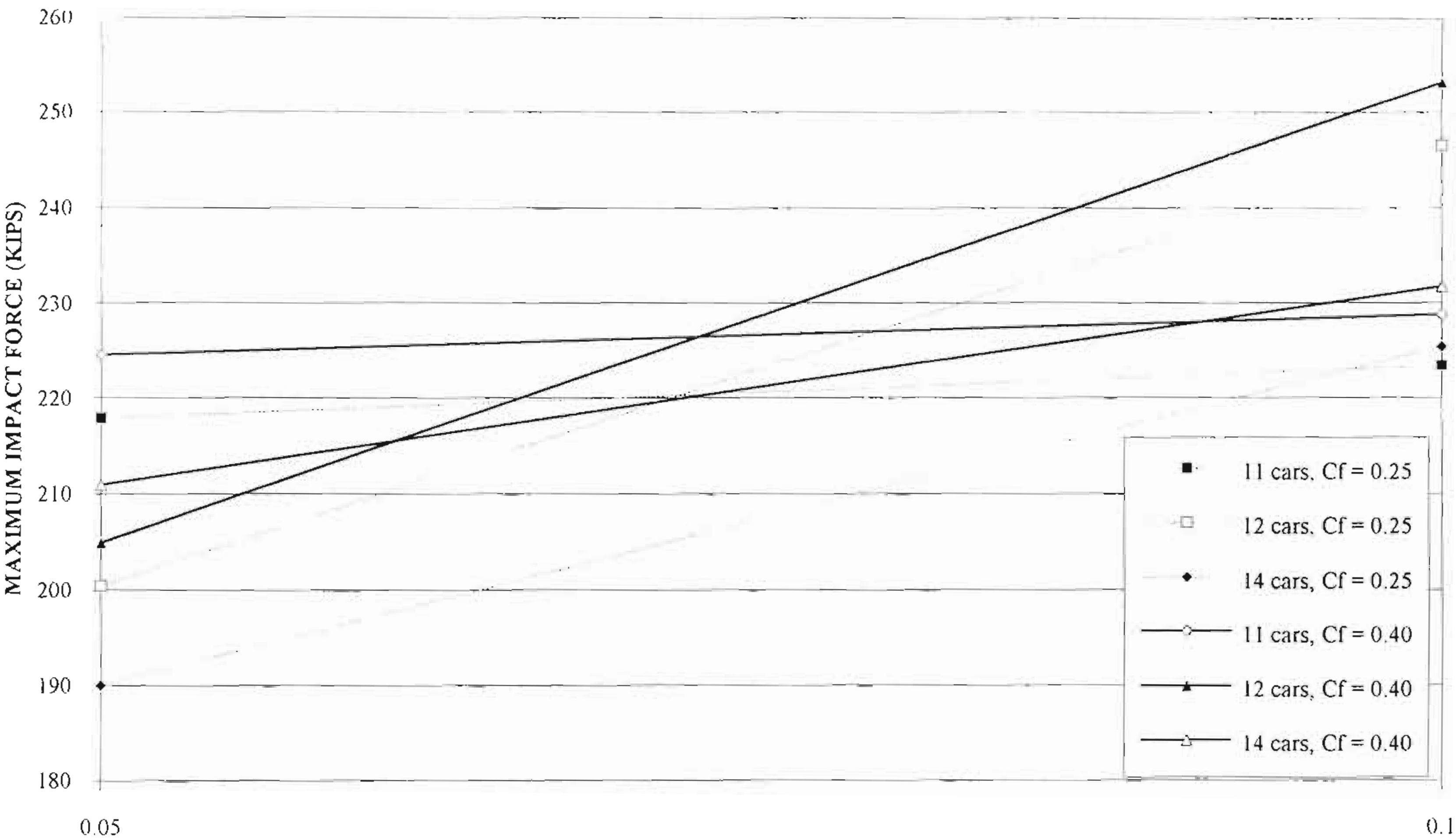


FIGURE 3-13. IMPACT FORCE VS. INITIAL DERAILING ANGLE

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip = 4.45 kN
1 ft = 0.305 m

While it might be theorized that an incident could occur resulting in a derailing car striking the barrier while airborne, before the front truck ever touches the ballast, such incidents have not been studied. The use of large initial derailing angles simulates partial airborne movement.

Conclusion: An initial derailing angle of 0.02 should be used in the case of barrier offset distances equal to or less than 2.74 meters (nine feet), 0.05 for barrier offset distances between 3.0 and 3.66 meters (10 and 12 feet), and 0.10 for barrier offset distances greater than 12 feet 3.66 meters (12 feet).

Dual Barriers:

Dual barriers, located on both sides of a pair of tracks, have been tested for various offset distances and speeds (See Figure 3-14). Dual barriers are placed on either side of a set of tracks. There is, therefore, a near barrier and a far barrier for each track (the far barrier being on the other side of the adjacent track).

Comparison of Figures 3-11 and 3-14 illustrate that forces are much higher for dual barriers than for single barriers. The maximum dual barrier force for a near barrier distance of 5.49 meters (18 feet) is over 11,700 kN (2,631 kips), where the maximum single barrier force is 1,912 kN (430 kips). This is due to the cars getting wedged between the two barriers, and getting pushed into the barriers by the cars behind.

Conclusion: Use dual barriers where necessary due to hazards on both sides, such as on overhead bridges. To minimize forces, minimum barrier offset distances should be used.

Triple Barriers:

Triple barriers have also been evaluated. These barriers would be laid out similar to dual barriers, but would also have a barrier between the two tracks. Barrier distance would therefore be equal for any one track.

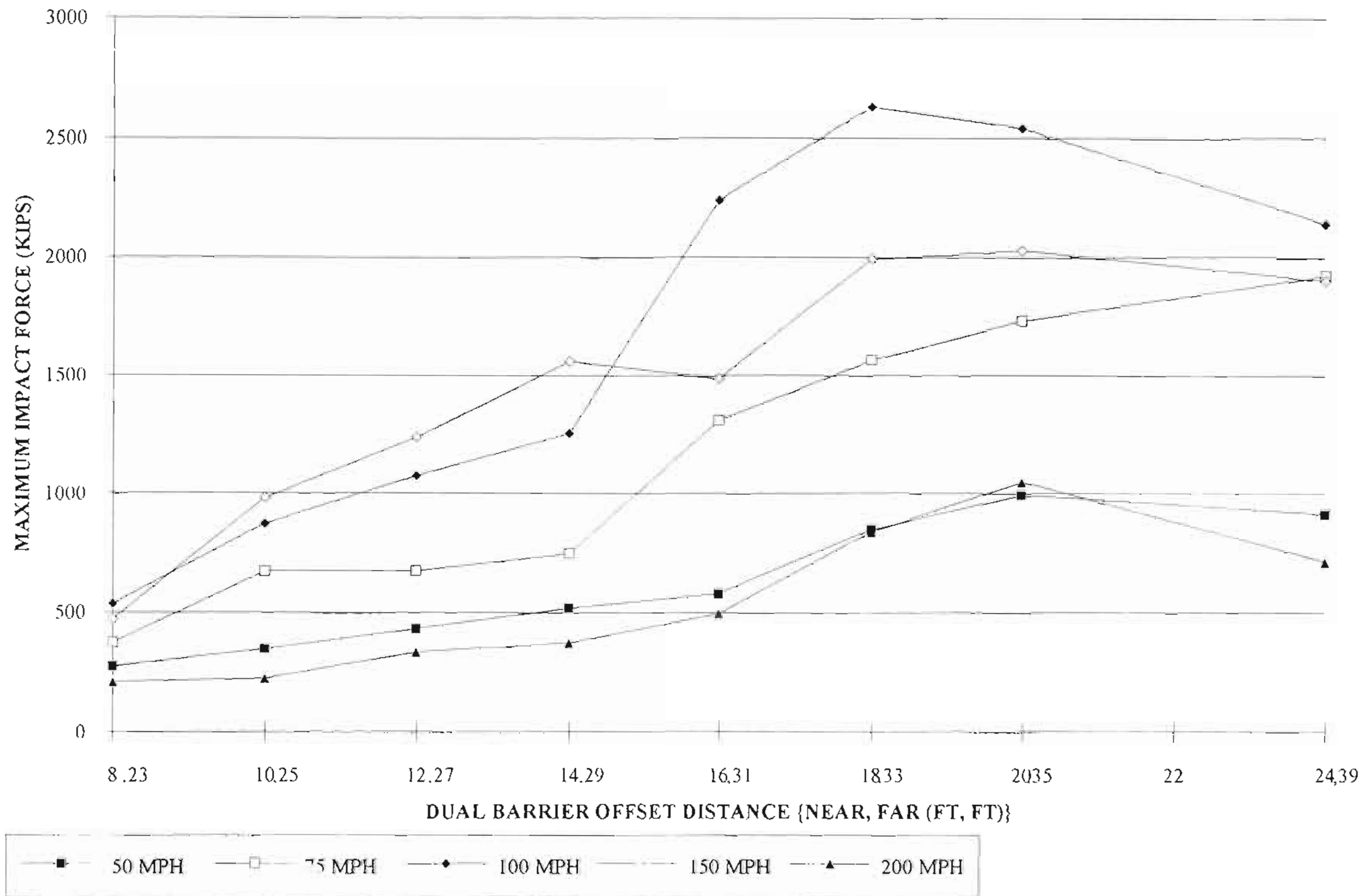


FIGURE 3-14. IMPACT FORCE VS. DUAL BARRIER OFFSET DISTANCE

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip = 4.45 kN
1 ft = 0.305 m

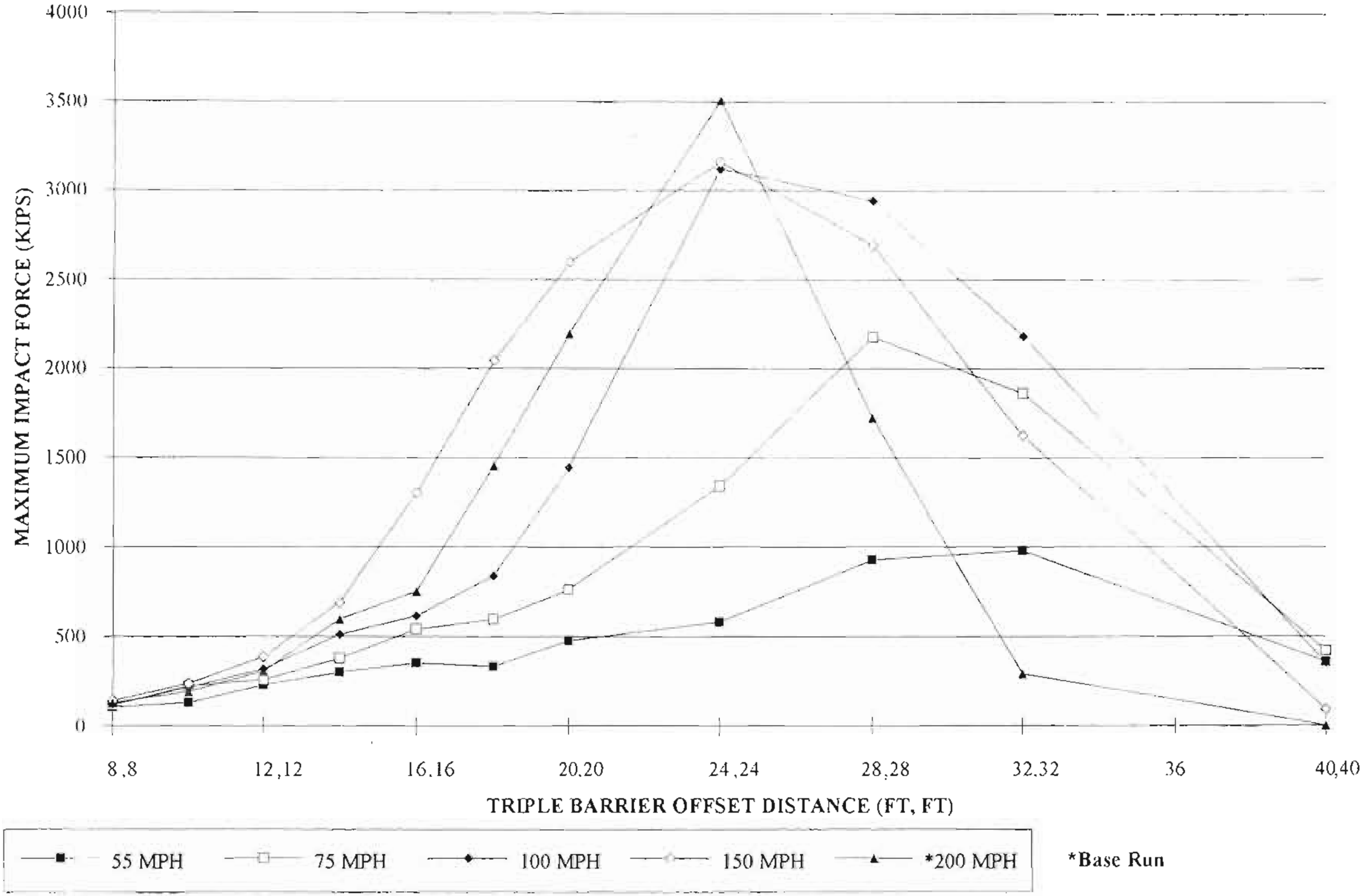
Like dual barriers, triple barrier loads are higher than single barriers, but they are lower than dual barriers at the small offset distances that would be used in practice (See Figure 3-15). Triple barriers could be effective where loads must be kept to a minimum, for example where attachment to bridge decks would otherwise overload the bridge deck structure. They could also be used between on-coming HSGGT tracks to protect HSGGT vehicles from opposing HSGGT vehicles.

It was decided, however, not to pursue triple barriers further for a number of reasons: (1) they would intrude into established vehicle clearance envelopes requiring more right-of-way; (2) they would cost substantially more than dual barriers; i.e., they would not decrease the loads so much as to reduce their size enough to offset the cost of the third barrier; (3) the probability of derailing at the instant an on-coming vehicle approaches is more remote than other scenarios; (4) HSGGT systems are considered to be safer and better maintained and are less likely to derail; and (5) opposing HSGGT vehicles, being on the same system, would have the benefit of direct communication thereby giving more advance warning in the event of derailment of one of the vehicles.

Conclusion: Triple barriers should not be used and have not been considered further.

In summary, the following conclusions have been drawn from the parametric study and have been followed for all other vehicle types:

Vehicle Speed:	Speeds from 80 km/h (50 mph) to maximum speed should be studied
Vehicle Crush Stiffness	The Emori model should be used
Ground Friction	1.0 should be used
Barrier Friction Coefficient:	0.25 should be used for steel, 0.40 for concrete
Barrier Offset Distance:	2.74 m (9 ft) for railroad vehicles, and 3.35 m (11 ft) for Maglev vehicles
Braking Coefficient:	5.5% should be used
Coupler Types:	EE, EF, FF types should be used for freight, modified models for HSGGT
Number of Cars:	Maximum number of cars should be used
Initial Derailing Angle:	0.10 radians should be used, or maximum before overlapping barrier
Dual Barriers:	Railroad vehicles: 2.74 m (9 ft) near barrier, 7.32 m (24 feet) far barrier



3-15. IMPACT FORCE VS. TRIPLE BARRIER OFFSET DISTANCE

(See Section 3.1.3.1 for explanation of results)
Note: Metric Equivalent 1 kip = 4.45 kN
1 ft = 0.305 m

	Maglev vehicles: 3.35 m (11 ft) near barrier. 8.46 m (27.75 feet) far barrier
Triple Barriers:	Should not be used

3.1.3.2 Out-of-Plane Effects

The TBIP Program models two-dimensional effects in the horizontal plane. These effects represent the majority of the energy and forces involved in the derailment incident. Supplementary calculations have been performed to determine the effects of three-dimensional movements, including rotation about the longitudinal car axis and vertical buckling or override. It has been concluded that the effects of out-of-plane motion on impact forces are minor.

Vertical Buckling — The tendency of the train to buckle vertically, or override, under axial compression loads has been checked. The compression loads vare found to be insufficient to lift the cars. The car weights are great enough to resist any vertical instability. The tendency of buckling horizontally is determined by TBIP.

Rollover — Calculations have been performed to determine the barrier height necessary to prevent the vehicle from rolling over the top of the barrier. A stable condition is achieved when the *restoring moment* exceeds the *overturning moment*. The overturning moment is equal to the horizontal impact force multiplied by the vertical distance of the vehicle's mass center above the top of the barrier. The restoring moment is equal to the weight of the vehicle multiplied by the horizontal distance to the barrier (See Section 4.1.1.5 and Figure 4-5). Results indicate that heights varying from 1.52 m to 1.83 m (5 to 6 feet) above the top of the guideway will be sufficient to prevent overtopping of the barrier. This calculated height is the basis for establishing barrier height for the designs shown on the design drawings in Section 4.1.2.

Rotation — Rotation about the car's longitudinal axis will result as the car travels laterally down the ballast slope toward the barrier. This rotation results when the car's wheels on the barrier side travel down the slope causing the car to tilt. Calculations have been performed to determine the angular velocity of this tilting, and estimate the resulting contribution to barrier impact load. Results indicate that this rotation will not cause a significant increase in impact force with an estimated increase of less than 2%.

Kinetic Energy Increase After derailment, as the vehicle loses potential energy during its travel down the ballast slope toward the barrier, kinetic energy is gained by the system and the vehicle's speed would be expected to increase. The increase in kinetic energy that results is estimated to be less than 1%.

Since models for three-dimensional behavior are less rigorous than the two-dimensional models, a factor of 20% has been added to the impact force generated by TBIP to allow for any out-of-plane effects. This factor is certainly conservative, yet still yields reasonable barrier sizes, not unlike barriers developed for highway and railroad use.

3.1.3.3 Barrier Design Forces

With the insight gained in the parametric study, additional TBIP runs have been made for the remaining scenarios and other vehicles. Numerous runs were made to determine maximum forces for each scenario. The results of these runs are included in Appendix C. The maximum forces generated are summarized for each scenario in Table 3-3. These forces include the 20% allowance for three-dimensional effects.

3.2 STRUCTURAL HIGHWAY BARRIERS

This section describes methods used to design highway barriers - barriers designed to deflect errant highway vehicles and protect adjacent high speed corridors from these vehicles. These barriers are intended to mitigate hazards to high speed vehicles from collisions with errant highway vehicles. The barriers are also intended to mitigate hazards from damage sustained in highway vehicle collisions with high speed vehicle support structures such as bridge piers. As described in Section 3.1, this is distinct from the design of structural train barriers which are designed to deflect derailed trains and high speed vehicles.

Much research has already been done on the analysis and design of structural highway barriers. The current AASHTO Standard Specifications for Highway Bridges [1] and Guide Specifications for

TABLE 3-3. BARRIER FORCE SUMMARY - STRUCTURAL RAILROAD AND HSGGT BARRIERS

Vehicle Type	No. of Barriers	Range of Derailment Speeds Studied				Derailment Speed at MIF[1]		Maximum Impact Force (MIF)		3-D Effects: 20% of MIF		Total Barrier Force		Barrier Design Load	
		Min.		Max.											
		km/h	(mph)	km/h	(mph)	km/h	(mph)	kN	kips	kN	kips	kN	(kips)	kN	(kips)
ICE	Single	80	(50)	322	(200)	121	(75)	1063	(239)	213	(48)	1276	(287)	1334	(300)
ICE	Dual	80	(50)	322	(200)	161	(100)	3892	(875)	778	(175)	4671	(1050)	4893	(1100)
TGV	Single	80	(50)	322	(200)	161	(100)	543	(122)	109	(24)	651	(146)	890	(200)
TGV	Dual	80	(50)	322	(200)	161	(100)	2189	(492)	438	(98)	2626	(590)	2669	(600)
Maglev	Single	80	(50)	483	(300)	121	(75)	730	(164)	146	(33)	875	(197)	890	(200)
Maglev	Dual	80	(50)	483	(300)	121	(75)	956	(215)	191	(43)	1148	(258)	1334	(300)
Freight - Uniform	Single	56	(35)	129	(80)	89	(55)	3688	(829)	738	(166)	4425	(995)	4893	(1100)
Freight - Uniform	Dual	56	(35)	129	(80)	105	(65)	9417	(2117)	1883	(423)	11300	(2540)	11298	(2540)
Freight - Mixed	Single	56	(35)	129	(80)	89	(55)	1072	(241)	214	(48)	1286	(289)	1334	(300)
Freight - Mixed	Dual	56	(35)	129	(80)	129	(80)	8581	(1929)	1716	(386)	10297	(2315)	11298	(2540)

[1] MIF = Maximum Impact Force

Bridge Railings [11] include a design methodology for bridge railing. The AASHTO Roadside Design Guide [12] includes recommendations for guard rails adjacent to at-grade roadways. Bridge and guard railing systems are usually proven through crash testing. Many tested designs currently exist such as concrete New Jersey safety shapes, steel bridge rails, concrete parapets, and combination steel and concrete systems. Most of these designs, however, have been developed and tested for light trucks and automobiles. HSGGT intrusion barriers must also be capable of resisting larger vehicles, weighing up to 36,300 kg (80,000 lbs) (the maximum legal highway limit).

AASHTO is currently developing new specifications for bridge railings, to be incorporated into their LRFD Bridge Design Specifications and Commentary. These specifications give a methodology for designing barriers capable of resisting 80,000 pound trucks. It is recommended that the methodology described in this new code be adopted for Intrusion Barriers for Highway Vehicles. The provisions can be applied directly where the barrier is located on a bridge. The provisions can be modified to incorporate new provisions for foundations where the barrier is located at-grade.

Much of the following section is taken from the AASHTO Standard Specifications for Highway Bridges, the Roadside Design Guide and the Draft LRFD Bridge Design Specifications and Commentary, March 1993 [13].

3.2.1 Methodology

The primary purpose of all roadside highway barriers is to prevent a vehicle from leaving the roadway and striking a fixed object or terrain feature that is considered more hazardous than the barrier itself. HSGGT structural highway intrusion barriers are also intended to protect the high speed vehicle from intrusions from errant highway vehicles. This is accomplished by containing and redirecting the impacting vehicle. Since the dynamics of a crash are complex, the most effective means of assessing barrier performance for highway vehicles is through full scale crash tests. The new methodology for modeling and analysis used by AASHTO is, in fact, based on crash testing.

3.2.1.1 Crash Test Criteria

A study was made by the National Cooperative Highway Research Program (NCHRP) in Report No. 230, "Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances"

[14]. This report currently recommends two tests on standard sections of longitudinal barriers, one with an 820 kg (1800 lb) vehicle impacting at 96 km/h (60 mph) and 20 degrees to evaluate occupant risk, and one with a 2050 kg (4500 lb) vehicle impacting at 96 km/h (60 mph) and 25 degrees to evaluate the structural integrity of the barrier. After collision vehicle trajectory is also evaluated in these tests.

NCHRP Report No. 230 also gives recommendations for a series of optional tests using cars, buses, and trucks with weights up to 36,300 kg (80,000 lbs) to evaluate the effectiveness of safety features. The heavy truck impact test uses a vehicle speed of 80 km/h (50 mph) and a 15 degrees impact angle. It should be noted that NCHRP Report 350 "Recommended Procedures for the Safety Performance Evaluation of Highway Features" [15], has now replaced NCHRP Report 230.

Crash tests have been performed on various prototype barrier designs. Instrumentation systems measured the forces experienced by the barriers, while the performance of the barriers was visually observed. Thus, barrier designs have been developed that have performed satisfactorily under the impact of the various vehicle types.

3.2.1.2 Warrants

Barrier *warrants* are the criteria by which the need for a safety treatment or improvement can be determined. They are based on the premise that a traffic barrier should be installed only if it reduces the severity of potential accidents. It is important to note that the probability or frequency of run-off-the-road accidents is not directly related to the severity of potential accidents. Typically, guardrail warrants have been based on a subjective analysis of certain roadside elements or conditions. If the consequences of a vehicle striking a fixed object hazard or running off the road are believed to be more serious than hitting a traffic barrier, then the barrier is considered warranted. While this approach can be used often, there are instances where it is not immediately obvious whether the barrier or the unshielded condition presents the greater hazard. Furthermore, the subjective method does not directly consider the probability of an accident occurring nor the costs associated with the shielded and unshielded conditions.

Thus, warrants may also be established by using a benefit-to-cost analysis whereby factors such as design speed and traffic volume can be evaluated in relation to barrier need. Costs associated with the barrier (installation cost, maintenance costs, and accident costs) are compared to similar costs associated with the unshielded hazard.

Highway hazards that warrant shielding by a roadside barrier can be placed in one of three basic categories: embankments, roadside obstacles, or bystanders.

Embankments

Traditionally, barriers have been used for protection of highway vehicles from hazards related to embankments. Embankment height and side slope are the basic factors considered in determining barrier need. These criteria are based on studies on the relative severity of encroachments on embankments versus impacts with roadside barriers.

Roadside Obstacles

Another traditional use of barriers is for protection from roadside obstacles. Roadside obstacles may be nontraversable hazards or fixed objects and may be either man made (such as culvert inlets) or natural (such as trees). Barrier warrants for roadside obstacles are a function of the obstacle itself and the likelihood that it will be hit. However, a barrier should be installed only if it is clear that the result of a vehicle striking the barrier will be less severe than the accident resulting from hitting the unshielded object. HSGGT guideways are a new type of obstacle hazard, since they present more of a hazard to the highway vehicle than the presence of the barrier itself.

Bystanders

A bystander is any adjacent presence that should be protected from the errant highway vehicle. Examples include pedestrians and buildings. HSGGT guideways adjacent to highway facilities also fall into this category.

3.2.1.3 Performance Level Selection Procedures

Traditionally, most roadside barriers were developed, tested and installed with the intention of containing and redirecting passenger motor vehicles weighing up to 2050 kg (4500 pounds). Properly designed and installed barrier systems have proven to be very effective in reducing the amount of damage and lessening the severity of personal injuries when struck by automobiles and similar-sized vehicles at relatively shallow angles (less than 25 degrees) and at reasonable impact speeds, less than 112 km/h (70 mph). However, it has long been understood that barriers designed for automobiles should not be expected to perform equally well for larger vehicles, such as buses and trucks. Recognizing this fact, several highway agencies have developed and used barrier systems capable of redirecting vehicles as heavy as 36,300 kg (80,000 pound) tractor trailer combination trucks. Although objective warrants for the use of higher performance traffic barriers do not presently exist, subjective factors most often considered for new construction or safety upgrading include:

- high percentage of heavy vehicles in traffic stream
- adverse geometrics such as sharp curvature oftentimes combined with poor sight distance
- severe consequences associated with penetration of a barrier by a large vehicle.

Five performance levels have been defined to account for different types of highways and the anticipated type of vehicle including its weight and geometry (height). The crash testing requirements vary by performance level. The performance levels are given in Table 3-4. Crash testing requirements are given in Table 3-5.

The hazards inherent in adjacent HSGGT facilities requires a performance level of either PL-4, or PL-5, depending on the nature of the highway traffic. It is generally recommended that the PL-5 performance level be used, unless the volume of tank trucks is extremely low, such as may result from traffic restrictions.

3.2.2 Findings

Figures 3-16, 3-17, 3-18 and 3-20 show the dimensions, weights, and center-of-gravity (C.G.) heights of typical automobiles, buses and trucks. Also shown are several longitudinal barriers which have successfully redirected them in crash tests [16]. It can be seen that to redirect a 36,300 kg (80,000 lb) van-type tractor-trailer takes a barrier approximately 1.27 m to 1.37 m (50 to 54 in) high. The barrier should push on the *hard point* or floor of the van to redirect it. It can be seen that to redirect a 36,300 kg (80,000 lb) fluid tank truck will take a barrier 2.13 to 2.29 m (84 to 90 in) high. The barrier should push on the fluid tank which is frequently a cylinder. These heights are required to prevent the truck from rolling over the barrier. Figures 3-19 and 3-21 show barrier heights in graphical form.

TABLE 3-4. PERFORMANCE LEVEL SELECTION CRITERIA

PL-1	Performance Level One - Used for short, low level structures on rural highway systems, secondary expressways, and areas where a small number of heavy vehicles are expected and speeds are either posted or reduced.
PL-2	Performance Level Two - Used for high-speed main line structures on freeways, expressways, highways, and areas with a mixture of heavy vehicles and maximum tolerable speeds.
PL-3	Performance Level Three - Used for freeways with variable cross slopes, reduced radius of curvature, higher volume of mixed heavy vehicles and maximum tolerable speeds. Site specific justification shall be made for use of this performance level.
PL-4	Performance Level Four - Used where there are a high percentage of heavy *van* type vehicles in the traffic stream and where there are severe consequences associated with penetration of a barrier by a large vehicle.
PL-5	Performance Level Five - Used where there are a high percentage of heavy *tank* type vehicles in the traffic stream and where there are severe consequences associated with penetration of a barrier by a large vehicle.

TABLE 3-5. BRIDGE RAILING PERFORMANCE LEVELS AND CRASH TEST CRITERIA

	Test Vehicle Descriptions and Impact Angles					
	Small Automobile	Pickup Truck	Medium Single-Unit Truck	Van-Type Tractor-Trailers	Large Van-Type Tractor Trailers	Large Tank Trucks
Weight	820 kg (1.8 kips)	2,430 kg 5.4 kips	8,160 kg (18 kips)	22,680 kg (50 kips)	36,300 kg (80 kips)	36,300 kg (80 kips)
Track	1.67 m (5.5 ft)	1.98 m (6.5 ft)	2.29 m (7.5 ft)	2.44 m (8.0 ft)	2.44 m (8.0 ft)	2.44 m (8.0 ft)
C. G. Height	508 mm (20.0 in)	686 mm (27.0 in)	1245 mm (49.0 in)	1626 mm (64.0 in)	1626 mm (64.0 in)	1981 mm (78.0 in)
Impact Angle	q = 20°	q = 20°	q = 15°	q = 15°	q = 15°	q = 15°
Perf. Level	*Test Speeds - km/h (mph)*					
PL-1	72 (45)	72 (45)	NA	NA	NA	NA
PL-2	97 (60)	97 (60)	80 (50)	NA	NA	NA
PL-3	97 (60)	97 (60)	80 (50)	80 (50)	NA	NA
PL-4	NA	NA	NA	NA	80 (50)	80 (50)
PL-5	NA	NA	NA	NA	80 (50)	80 (50)

Figure 3-19 shows the approximate vehicle impact force imposed on a rigid barrier by these types of vehicles. The magnitude of the impact force and its distribution on the barrier is very complex because of the numerous points of collision with the vehicle body, as well as its variation over time. The "Draft LRFD Bridge Design Specifications and Commentary," March 1993 [13] recommends the design forces shown in columns PL-1, PL-2 and PL-3 of Table 3-6. Columns for PL-4 and PL-5 for the 36,300 kg (80,000 lb) van and fluid tanker respectively have been added based on subsequent studies. Reference [13] shows how these design forces are to be used to design a longitudinal barrier.

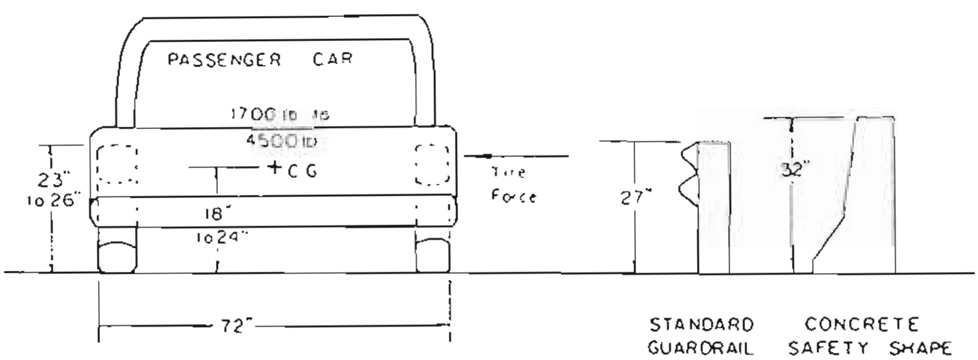


FIGURE 3-16. BASIC PROPERTIES OF PASSENGER AUTOMOBILE AND EFFECTIVE LONGITUDINAL BARRIERS [16]

Note: Metric Equivalent 1 lb = 4.45 N
1 in = 25.4 mm

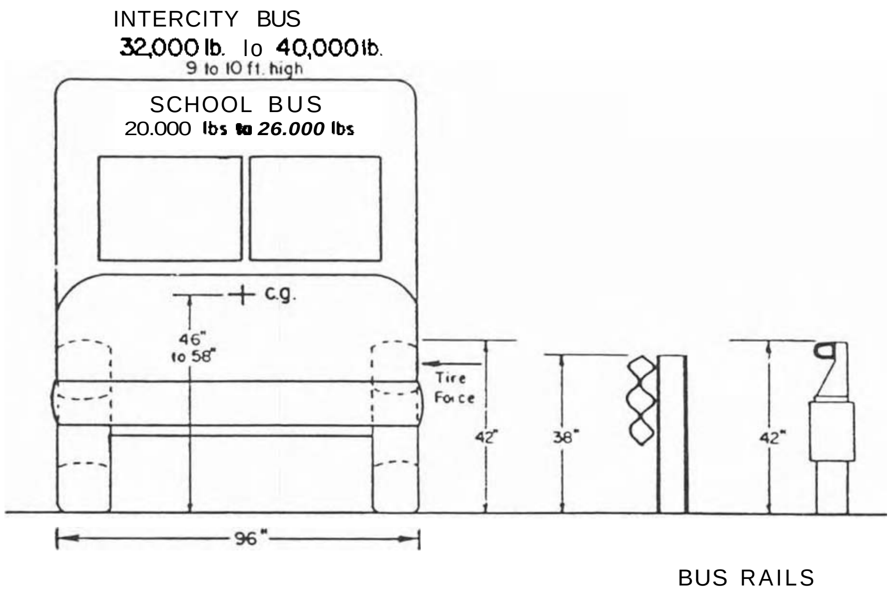


FIGURE 3-17. BASIC PROPERTIES OF BUSES AND TWO EFFECTIVE LONGITUDINAL RARRIERS [16]

Note Metric Equivalent l lb= 4.45 N
l in= 25.4 mm

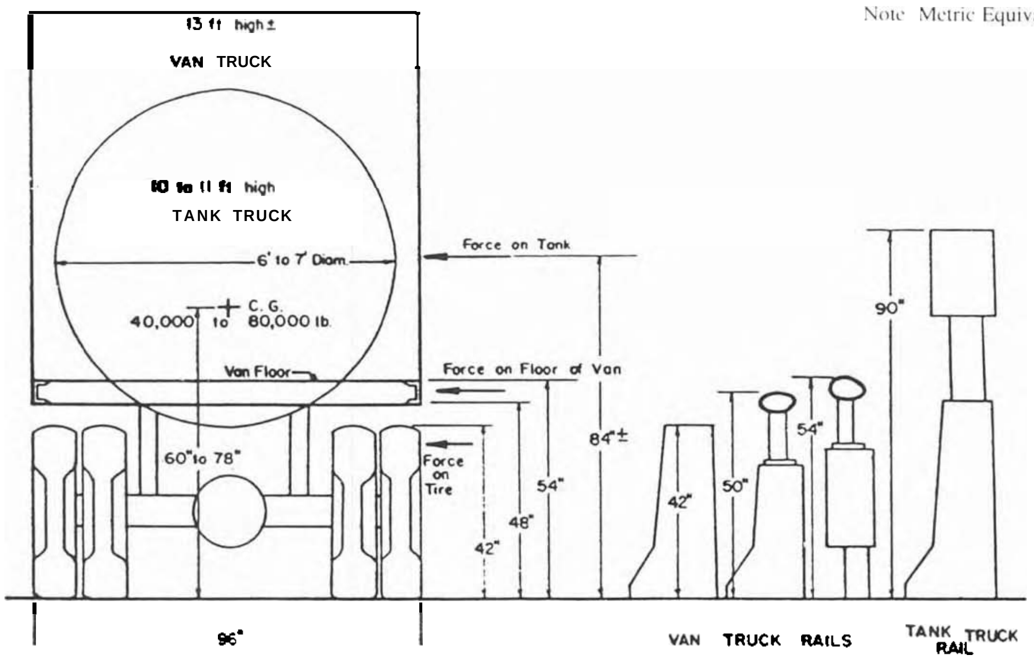


FIGURE 3-18. BASIC PROPERTIES OF TRACTOR-TRAILER TRUCKS (VAN AND TANK TYPES) AND SOME EFFECTIVE LONGITUDINAL RARRIERS [16]

Note: Metric Equivalent l lb= 4.45 N
l in= 25.4 mm

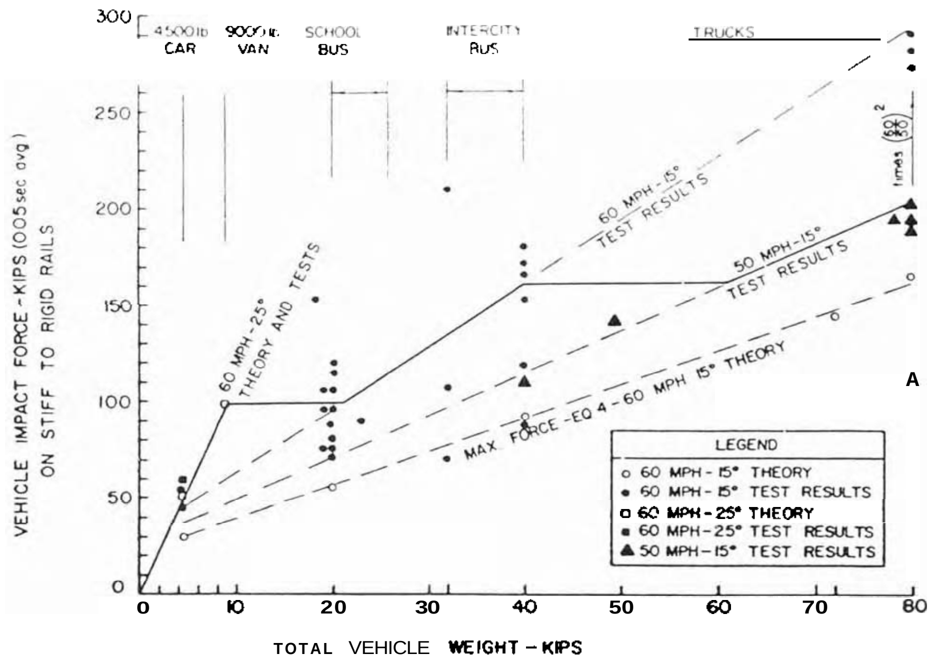
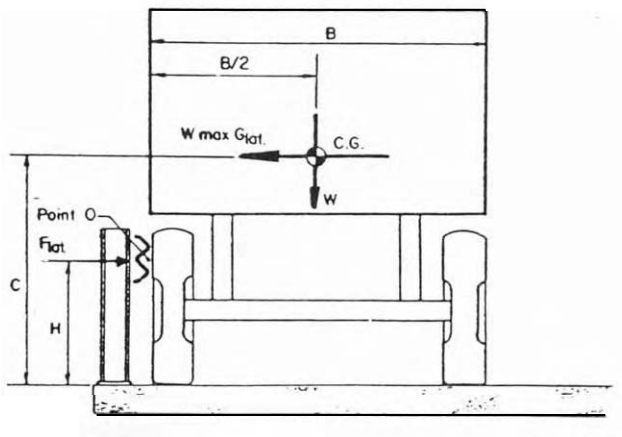


FIGURE 3-19. COMPARISON OF VEHICLE IMPACT FORCES AND TOTAL VEHICLE WEIGHT. THEORY AND TEST RESULTS FOR STIFF RAILS [16]

Note: Metric Equivalent 1 lb = 4.45 N
1 in = 25.4 mm
1 mph = 1.609 km/h
1 kip = 454 kg



W = weight of vehicle
maxGlat = maximum lateral deceleration of vehicle
C = height to vehicle center of gravity
H = effective height of barrier rail. in.
O = center of overturning rotation located at centroid of rail or top of concrete parapet
B = width of vehicle. in.
Flat = resisting railing force located at effective rail height
= x maxGlat
Mo = Overturning moment about point "O"
= Flat(C-H)-B/2 = 0

$$H = \frac{FlatC - (B/2)}{Flat}$$

FIGURE 3-20. APPROXIMATE ANALYSIS OF BRIDGE RAIL EFFECTIVE HEIGHT REQUIRED TO PREVENT VEHICLE FROM ROLLING OVER RAIL [16]

Note: Metric Equivalent 1 lb = 4.45 N
1 in = 25.4 mm

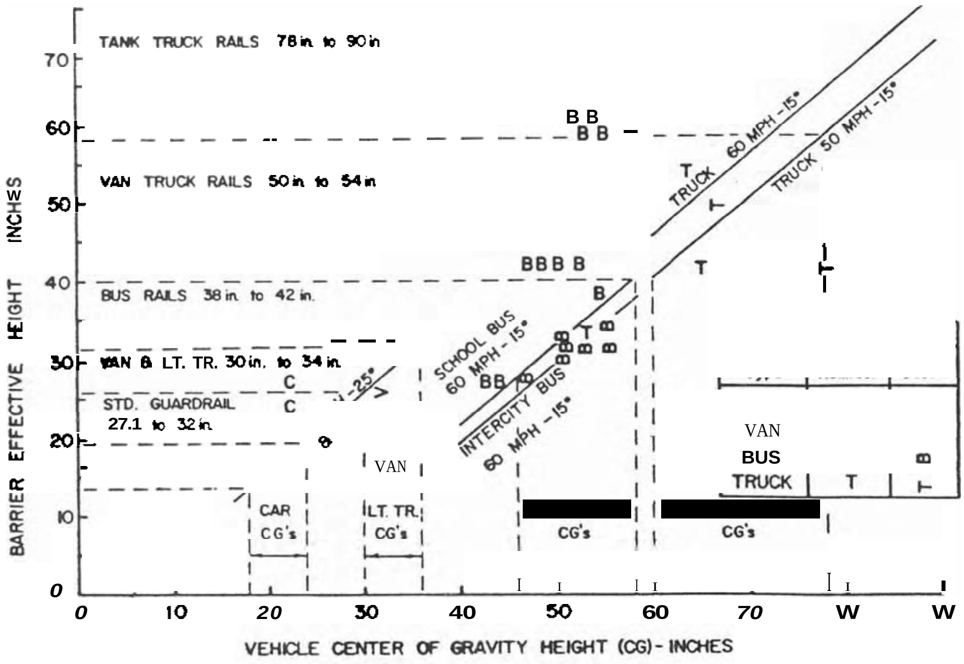


FIGURE 3-21. COMPARISON OF REQUIRED BARRIER HEIGHT AND VEHICLE CG, THEORY AND TEST RESULTS [16]

Note: Metric Equivalent 1 in = 25.4 mm
1 mph = 1.609 km/h

TABLE 3-6. DESIGN FORCES FOR HIGHWAY BARRIERS

Design Forces and Designations	*Railing Performance Levels*					
	Units	*2,430 kg (5,400 lb) Truck PL-1*	*8,160 kg (18,000 lb) Truck PL-2*	*22,680 kg (50,000 lb) Van PL-3*	*36,300 kg (80,000 lb) Van PL-4*	*36,300 kg (80,000 lb) Tank PL-5*
F_t	kN (kips)	120 (27.0)	240 (54.0)	516 (116.0)	552 (124)	778 (175)
F_l	kN (kips)	40 (9.0)	80 (18.0)	173 (39.0)	182 (41)	258(58)
F_v	kN (kips)	24 (5.4)	80 (18.0)	222 (50.0)	356 (80.0)	356 (80.0)
L_t and L_l	mm (in.)	102 (4.0)	89 (3.5)	203 (8.0)	203 (8.0)	203 (8.0)
L_v	mm (in.)	457 (18.0)	457 (18.0)	1016 (40.0)	1016 (40.0)	1016 (40.0)
H_{eff}	mm (in.)	508 (20.0)	813 (32.0)	1016 (40.0)	1067 (42.0)	1422 (56.0)
H_{min}	mm (in.)	508 (20.0)	813 (32.0)	1016 (40.0)	1372 (54.0)	2286 (90.0)

F_t = Transverse force on barrier
F_l = Longitudinal force on barrier
F_v = Vertical force on barrier
L_t, L_l, L_v = Distribution length of transvcrsc, longitudinal and vertical forces
H_{eff} = Effcctivc height of vehicle rollover force
H_{min} = Rail height

3.3 EARTHWORK BARRIERS

3.3.1 Methodology

This analysis evaluates the effectiveness and feasibility of using engineered ditches, berms, and various combinations to create functional intrusion barriers for HSGGT systems placed in shared corridors. These barriers are considered for use as protection barriers, containment barriers, or both. Past research performed with passenger vehicles and modeling performed by the Texas Transportation Institute (TTI) is the basis for this evaluation.

In the past, various earthwork configurations have typically been associated with highway engineering. Upslopes and downslopes have been used to adapt roadways to existing terrain with cuts and fills. Ditches have been used to channelize drainage, and berms have been used as protection from hazards such as roadside signs. Usually, however, ditches and berms are considered to be roadside hazards. The severity of the hazard depends upon the degree of slope over which a vehicle would be forced to traverse. Generally, errant highway vehicles are prevented from traveling on these slopes through the use of guardrails, concrete barriers or the like. The use of ditches and berms as intrusion barriers is a new concept.

Other earthen systems have been used to dissipate energy. An example is the truck runaway escape ramp. These ramps are typically sand or gravel filled to a depth of 305 mm (12 inches). It was initially proposed that the earthwork barrier utilize two primary concepts to prevent intrusion. First, the earthwork barrier should provide a means of redirecting the vehicle. That is, it should provide a barrier to contain the high speed vehicle within its right-of-way or deflect an intruding vehicle to protect a high speed right-of-way. Second, it should, in some fashion, dissipate the kinetic energy of the derailed train set.

The dissipation of energy must occur without substantially damaging the trainset. While this concept may be difficult to achieve in combination with the redirection aspects of the barrier, it remains a goal of this study.

A review of NTSB accident reports for conventional railroad derailments shows that substantial forces are applied during derailment. In some cases, unbalanced forces have been sufficient to force the train to flip end over end, or roll on its longitudinal axis. Our examination of energy dissipation recognizes this condition and attempts to identify where this hazard is a concern.

Energy dissipation has been used primarily by the Federal Highway Administration for run-away trucks. Similar technology could be used as a means of dissipating energy during derailment. Further, by creating a vertical slope adjacent to the track or guideway, a derailed vehicle would convert at least some kinetic energy to potential energy as it travels uphill. Both energy dissipation and redirection are considered as the primary components for the earthwork barrier.

The work-energy principle is the basis for analysis and modeling of earthwork barriers. This principle states that the change in kinetic energy (DKE) equals the work performed on the system (U), or:

$$\Delta KE = U$$

which reduces to:

$$\tfrac{1}{2} \times MV_i^2 + WH_i - W(H_f) - (F_f d) = 0$$

where:

V_i =	initial velocity at derailment
M =	mass of vehicle
W =	weight of vehicle
H_i =	initial elevation of vehicle
H_f =	final elevation of vehicle
$(F_f d)$ =	summation of all friction forces multiplied by their distance of application

In the case of high speed vehicles, the potential energy contribution is minimal as compared to kinetic energy, and the equation reduces to:

$$\tfrac{1}{2} \times MV_i^2 = (F_f d)$$

This simple formula is used to predict the total distance traveled by the vehicle before it comes to rest.

To complete the analysis, research on highway barriers has been reviewed to evaluate the redirection characteristics of berms and ditches.

3.3.2 Findings

Earthwork berm and ditch barrier systems are not well suited as intrusion barriers for high speed systems for the following reasons:

High Vertical Accelerations: At velocities of 320 km/h (200 mph), even slight changes in the vertical gradients of the earthwork would result in substantial vertical accelerations. Previous testing of highway vehicles, and modeling of the high speed vehicles suggest that shoulder gradients greater than 6:1 would create a condition where the high speed vehicle would become airborne. Once airborne the vehicle would lose control, creating unpredictable and violent movement. In addition, vertical accelerations and decelerations would create unacceptable forces for passenger safety.

High Vehicle Deceleration: Changes in grade could cause the vehicle to dig into the side of slopes, stopping the vehicle suddenly, creating unacceptably high deceleration and causing tumbling or airborne motion. This would subject passengers to violent forces and would increase rather than decrease hazards.

Rollover Hazard: For highways, the maximum recommended slope for an earthwork berm without guard rails is 3:1. Steeper slopes produce vehicle rollover. At speeds of 320 km/h (200 mph), the maximum slope would have to be much flatter to prevent rollover, perhaps flatter than 8:1. These flat slopes would not be effective for redirecting high speed vehicles.

Poor Energy Dissipation: Given that earthwork barriers would be incapable of redirecting high speed vehicles, their effectiveness at dissipating energy through translation to potential energy and frictional heat was studied. Calculations using the energy formulae given above indicate that predicted performance of earthworks barriers for dissipation of energy would also be poor.

The kinetic energy of a high speed vehicle traveling at 320 km/h (200 mph) is so great that both frictional losses and potential energy components require great dimensions to be effective. Neglecting the effects of potential energy over 400 m (1300 ft) would be required to stop the train through ground friction alone. Without effective redirection of the vehicle, this distance would translate into large horizontal movements requiring wide rights-of-way.

Assuming all kinetic energy is translated to potential energy (neglecting ground friction), a berm over 400 meters (1300 ft) high would be required to convert the kinetic energy to potential energy and stop the vehicle.

Even considering the combination of frictional losses and potential components, earthwork systems would not be effective as energy dissipators.

Right-of-Way Requirements: Earthwork barriers with gradients acceptable for vertical accelerations (more shallow than 6:1, say 8:1), would require substantial right-of-way. For example, a 3 m (10 ft) vertical displacement would require a horizontal distance of 24 m (80 ft). The lateral distance required for deceleration of the vehicle would also be large. Acquisition costs would make this type of barrier impractical.

Earthwork barriers would be impractical, costly and would create unacceptable safety hazards. They have not been considered further in this study. Structural barriers, by contrast, do not impose the vertical movement and sudden deceleration that earthwork barriers would. They remain the more practical choice for intrusion barriers.

3.4 COMBINATION STRUCTURAL/EARTHWORK BARRIERS

Earth berm and ditch-type combination barriers are not recommended because of the safety concerns cited above. A more feasible design alternative is the use of engineered earth retaining walls, as shown in Figure 2-7. This is a combination barrier design that takes advantage of the retained earth behind the wall to increase the structural resistance of the wall, and forms an effective intrusion barrier. The barriers would behave essentially as rigid barriers. The vertical face of the wall would reduce hazards related to any loss of vehicle control due to overturning and airborne movement. Right-of-way costs would also be reduced for this barrier system.

The methodology to be followed for the modeling and analysis of combination barriers will therefore use theories developed in the TBIP model and will also apply to combination barriers. The TBIP model has been used for the determination of forces that are used for the design of the retaining walls.

4. INTRUSION BARRIER DESIGN

The objective of the design effort is to define engineering solutions and to identify provisions which must be made in the design and construction of intrusion barriers. A general discussion is given here. Complete requirements for the design and construction of intrusion barriers are given in the Performance Specifications (Appendix B).

The derailment barrier impact forces generated by the TBIP computer analyses were used to develop intrusion barrier designs for the various scenarios. System components have been laid out and sized to resist the loads and requirements developed in the analysis. Detailed drawings have been prepared for each barrier design indicating barrier layout, geometry, and component size to a level of detail adequate for the preparation of cost estimates. Barrier loads and requirements are grouped, and alternative designs developed that are representative of scenarios with similar requirements.

As discussed in the previous section, earthwork barriers are deemed to be impractical and ineffective, as are combination barriers using earth berm or ditch concepts. The designs presented in this section, therefore, are limited to structural barriers and retaining wall type combination barriers. Earthwork barriers are not considered.

4.1 TRAIN BARRIER DESIGN

4.1.1 <u>Methodology</u>

4.1.1.1 General

This section summarizes and presents the major structural design aspects and methods used for the determination of the physical requirements of an effective barrier structure. The barrier structure is designed to perform the function of preventing a derailed vehicle from intruding into an adjacent right-of-way without collapse of the barrier and without the vehicle rolling over it. In addition to resisting the lateral impact forces imposed by a derailed vehicle, the wall must be strong enough to redirect the vehicle and resist further multiple impacts by the following derailed vehicles.

Three major items of barrier behavior are of practical interest for design:

1. The ultimate strength of the barrier system, i.e., that magnitude of the maximum impact load from a derailed vehicle that a structure can sustain without failure,

2. The deformations, such as deflections and extent of cracking, which the structure will undergo when impacted by a vehicle, and

3. The geometry of the barrier as it relates to that of the vehicle such that the vehicle is prevented from rolling over the top of the barrier.

Since collapse of the barrier structure is not allowed under dynamic impact loads, while damage and repairs are anticipated, the total ultimate capacity of the structure is of concern, and the design of the barrier will consist of determining the ultimate strength capacity of the various structural members necessary to resist the total ultimate vehicle impact load. Therefore, failure mode analysis is used and is the recommended method of design since stability may be maintained well beyond the elastic deformation of concrete or steel (during inelastic behavior). This failure theory also known as the *ultimate strength design method* or the *yield-line theory* is used for concrete wall barriers, and the *plastic theory* for structural steel wall barriers.

The ultimate strength method and the plastic theory evaluate the structure's ability to withstand loads based on the capacity of structural elements at their point of failure. For example, the ultimate moment capacity of a concrete beam is the bending moment that initiates yielding (stretching beyond safe limits) of the reinforcing steel and/or crushing of the concrete. For a steel beam the ultimate moment capacity (also known as the *plastic* moment) is the bending moment that initiates yielding of the steel beam. Further bending beyond these limits causes continued movement without significant increase in load. This ultimate strength approach is in contrast to *allowable stress methods,* used for other types of structures, that evaluate the structure's ability to withstand loads based on the capacity of structural elements at safe or allowable stress levels (e.g., ultimate stresses divided by some factor of safety).

Deformations, or deflections, are checked to ensure that they are not so great that adjacent transportation corridors would be intruded upon by the deflected barrier. Otherwise, deflections are not critical to the design. Because the barrier is designed for ultimate strength, much larger deflections can be tolerated than with conventional building or bridge design.

The barrier's geometry is based on the vehicle geometry, including its center of gravity and the *hard point* of the vehicle structure, or the location of the stiffest and strongest framing (usually the floor). The height of the barrier is sized to prevent overtopping by the vehicle, and to resist the impact forces at the vehicle's hard point.

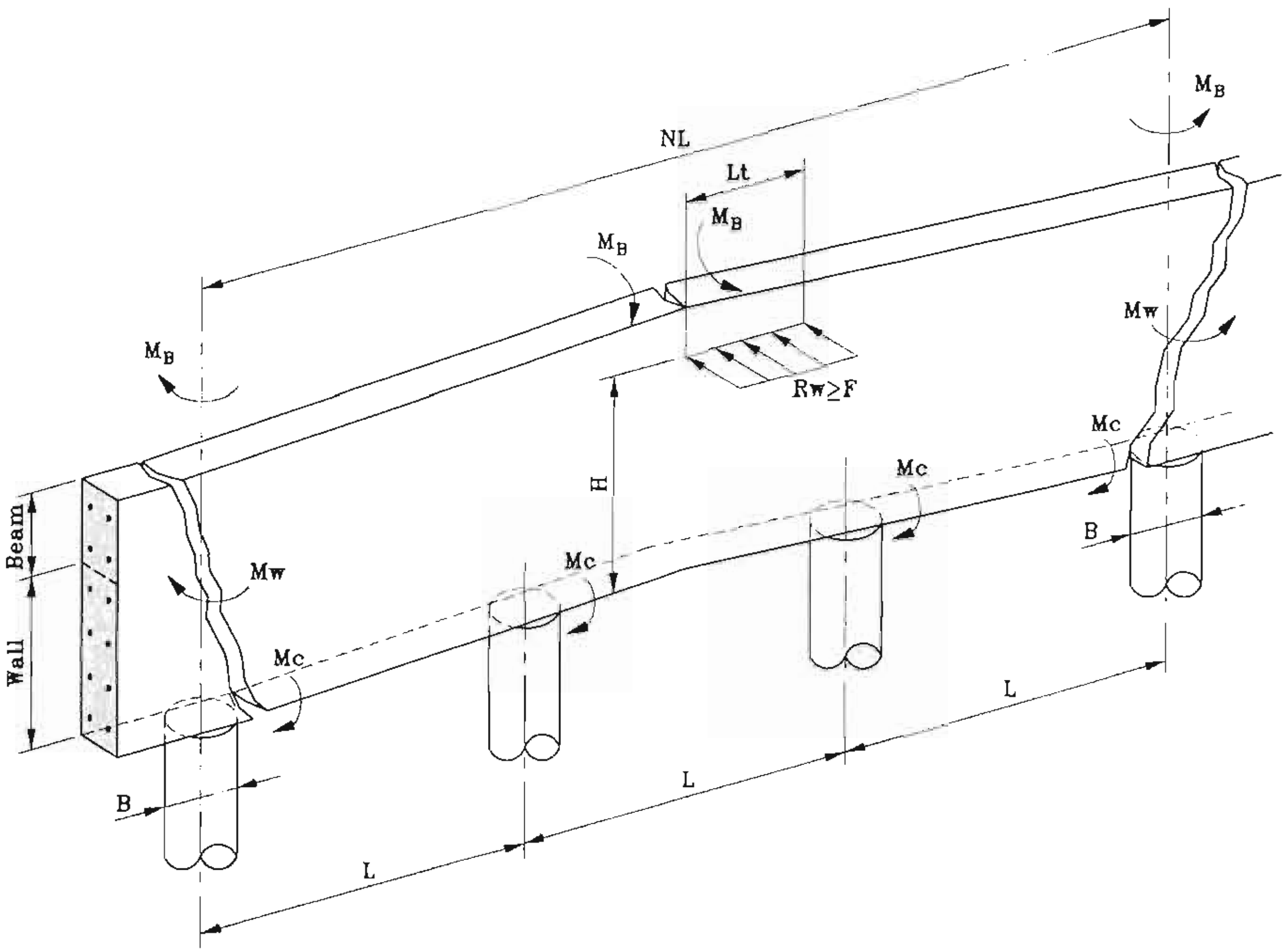
The barrier structure, whether concrete or steel, is designed to resist the effects of impact load, including flexure, shear, and torsion behavior. Eight alternative barrier designs have been developed (five at-grade and three elevated barrier designs) utilizing cast-in-place concrete, precast concrete and structural steel. These designs demonstrate that a structural barrier system is feasible and capable of deflecting a derailed high-speed vehicle.

4.1.1.2 Concrete Wall Barriers

Figure 4-1 shows a concrete wall supported on concrete caissons or piles, and subjected to a horizontal impact load near the top of the wall. Figure 4-2 shows a similar wall on an elevated structure. This load will tend to bend the wall into a dished shape surface in two directions: (1) horizontally between supports; and (2) vertically as a cantilever at support points (wall/column). The bending and deformation of the concrete wall indicates the capacity of the wall to be a function of its moment capacity.

The total ultimate moment capacity of the concrete barrier wall (see Chapter 4.1.1.1) is a function of the moment capacity of the localized beam at the top of the wall, the moment capacity of the wall below the beam, the cantilever moment capacity of the wall/column at the support, and the moment capacity of the supporting foundation or deck slab. The failure mechanism for this wall with a partially uniform distributed load (wl) will develop plastic hinges at the center and at supports. The plastic moments or moment capacities are determined by the ultimate strength method in accordance with ACI 318. The capacity-moment equations shown are arrived at by equating the external work with the internal energy absorbed. These equations are based on a study entitled "Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks" [17], modified for at-grade barriers to account for the lack of fixity between foundations otherwise provided by a bridge deck.

In order to achieve a failure mechanism or formation of plastic hinges, the concrete sections must be able to rotate and deform considerably. Therefore, the sections should be lightly reinforced in order to achieve yielding of the reinforcement and avoid crushing of the concrete.



Even Spans: Rw = 16Mb/(2NL-Lt)+16 Mw/(2NL-Lt)+(N-2)NMcL/(H(2NL-Lt)+4McB/H(2NL Lt)+Mc/H

Odd Spans: Rw = 16Mb/(2NL-Lt)+16 Mw/(2NL-Lt)+(N-1)(N+1)McL/(H(2NL-Lt)+4McB/H(2NL-Lt)

where:

F = maximum impact force, kN (kips)

H = distance from top of foundation to impact force, meters (feet)

L = foundation centerline spacing, meters (feet)

NL = critical length of wall failure, meters (feet)

N = number of spans in failure mechanism

Rw = total ultimate load capacity of barrier wall, kN (kips)

Mb = ultimate moment capacity of beam at top of wall, kN-m (ft-kips)

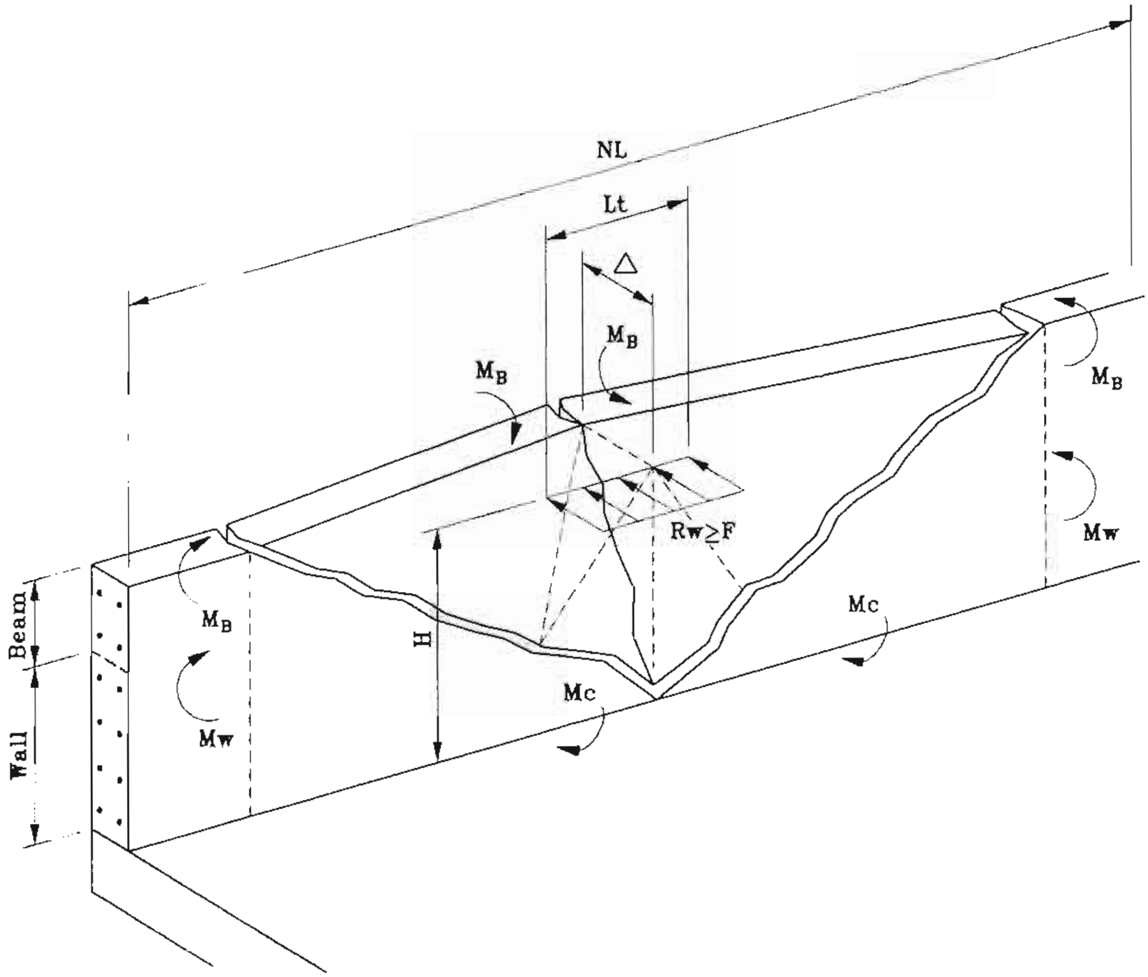
Mw = ultimate longitudinal moment capacity of wall, kN-m (ft-kips)

Mc = ultimate vertical moment capacity of wall/column at foundation, kN-m (ft-kips)

Lt = transverse length of distributed vehicle impact load, meters (feet)

B = width of foundation, meters (feet)

FIGURE 4-1. YIELD LINE ANALYSIS OF AT-GRADE CONCRETE BARRIER WALL



L = $Lt/2 + ((Lt/2)^2 + 8H(Mb+Mw)/Mc)^{1/2}$

Rw = $16Mb/(2L-Lt) + 16\ Mw/(2L-Lt) + 2McL^2/(H(2L-Lt)$

LR = Rw/Pc

where:

F = maximum impact force, kN (kips)

H = distance from top of slab to impact force, meters (feet)

L = critical length of wall failure, meters (feet)

Rw = total ultimate load capacity of barrier wall, kN (kips)

Mb = ultimate moment capacity of beam at top of wall, kN-m (ft-kips)

Mw = ultimate longitudinal moment capacity of wall, kN-m (ft-kips)

Mc = ultimate vertical moment capacity of wall cantilever up from bridge deck per unit length of wall, kN-m/m (ft-kips/ft)

Lt = transverse length of distributed vehicle impact load, meters (feet)

LR = total length of wall resisting impact load, meters (feet)

FIGURE 4-2. YIELD LINE ANALYSIS OF ELEVATED CONCRETE BARRIER WALL

4.1.1.3 Steel Wall Barriers

Figure 4-3 shows some possible failure modes for a steel beam and post barrier. As with the concrete barrier system, the total ultimate moment capacity of the steel barrier wall is a function of the moment capacity of all the structural elements that must work together to produce the ultimate strength of the barrier: namely, the top beam, posts, base plate and foundation or deck slab. In order to determine the total ultimate vehicle impact load, all possible failure modes shall be considered, including weak beam-strong post and strong beam-strong post systems.

The plastic moment or moment capacity of the beam and post members is calculated by the following equation:

$$M_p = F_Y.Z$$

Where:
- M_p = Plastic Moment in inch-pounds
- F_Y = Specified Minimum Yield Stress of Steel in pounds per square inch
- Z = Plastic Section Modulus in in^3

4.1.1.4 Foundations

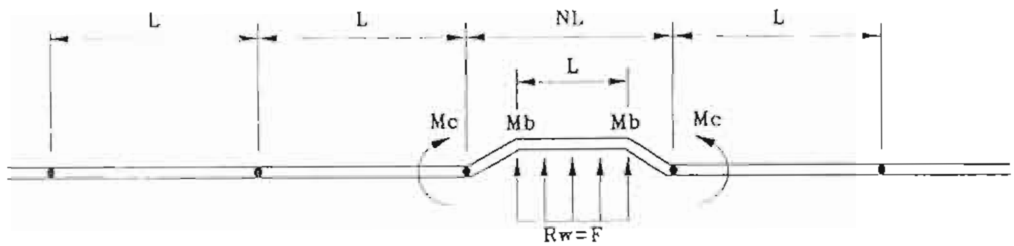
Figure 4-4 shows a typical deep foundation used to support the barrier wall system with the typical soil parameters that were used to design the piles. It should be noted that foundation conditions could differ dramatically based on actual site soil or rock occurring at a given site. Actual foundation designs should be developed based on actual site conditions determined with a subsurface exploration program. Like the steel and concrete components described above, the depth of embedment of the concrete caisson, precast concrete or steel pile foundation is determined by failure mode analysis. The ultimate lateral resistance in cohesionless (sand and gravel) and cohesive (clay) soils is based on Brom's pressure distributions [18]. The embedment depth required to safely resist the applied loads is determined based on the static load, and then accounting for the increased dynamic strength of the soil. The following equation relates the dynamic load to the static load [19]:

$$P_{Dynamic} = P_{Static} (1 + JV)$$

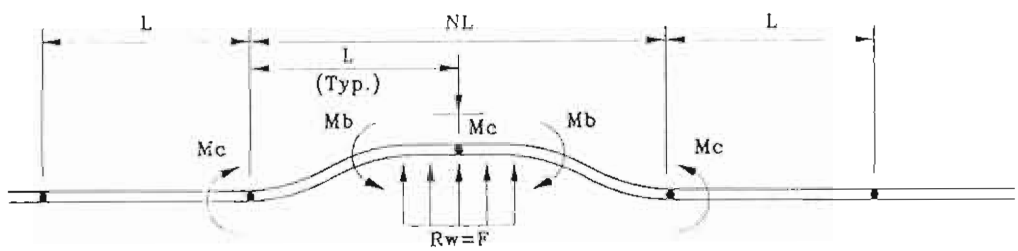
Where: V = Impact velocity in m/s (ft/s)

J = Damping constant = 0.46 s/m (0.14 s/ft)

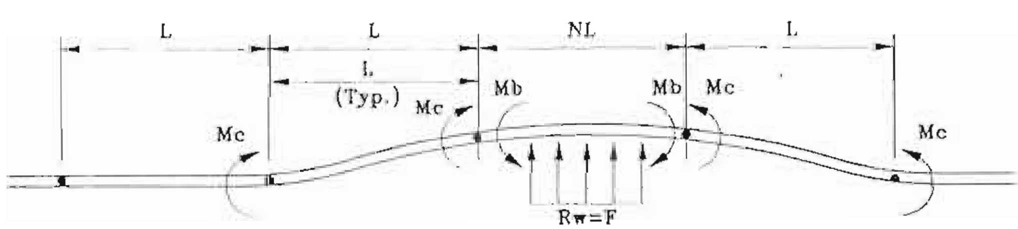
(a measure of the energy dissipating characteristics of the soil)



(A) SINGLE SPAN FAILURE MODE, N = 1



(B) TWO SPAN FAILURE MODE, N = 2



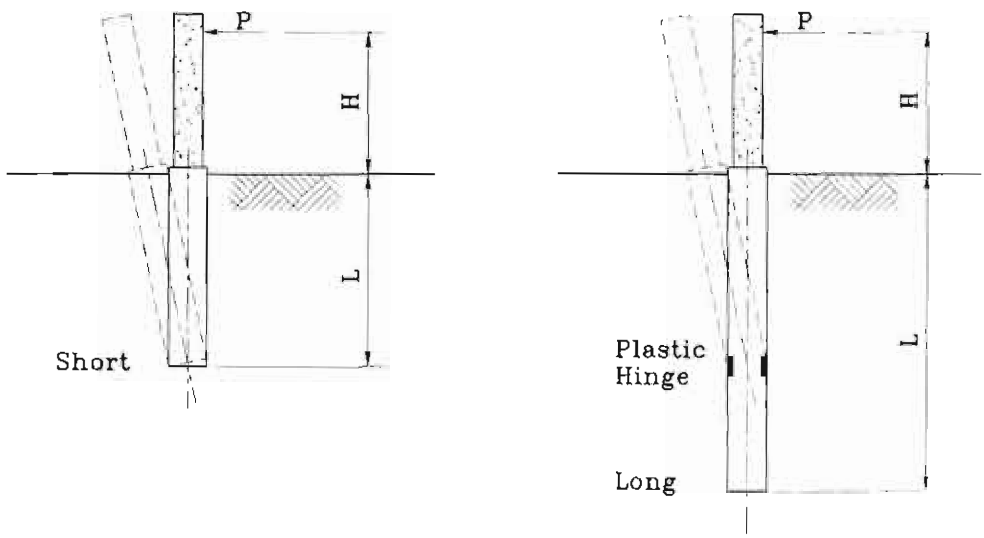
(C) THREE SPAN FAILURE MODE, N = 3

$$Rw = 16Mb/(2NL - Lt) + Mc(N-1)/H$$

where:

F =	maximum impact force, kN (kips)
H =	distance from top of slab/foundation to impact force, meters (feet)
L =	post/column centerline spacing, meters (feet)
NL =	critical length of wall failure, meters (feet)
N =	number of spans in failure mechanism
Rw =	total ultimate load capacity of barrier wall, kN (kips)
Mb =	plastic moment capacity of beam, kN-m (ft-kips)
Mc =	plastic moment capacity of post/column, kN-m (ft-kips)
PC =	ultimate load capacity of single post/column, kN (kips)
Lt =	transverse length of distributed vehicle impact load, meters (feet)

FIGURE 4-3. POSSIBLE FAILURE MODES FOR STEEL BEAM AND POST BARRIER



FAILURE MODE FOR LATERALLY LOADED PILES/CAISSONS

Brom's procedures to design piles for lateral loads shall be used based on the following (assumed) soil parameters:

Cohesion,	c = 71 kN/m^2 (1.50 ksf)
Average effective soil unit weight,	g = 1766 kg/m^3 (110 pcf)
Angle of internal friction,	f = 30 degrees

FIGURE 4-4. ULTIMATE LATERAL RESISTANCE OF FOUNDATION FOR SOILS RELATED TO EMBEDMENT DEPTH

4.1.1.5 Overturning Analysis

Figure 4-5 shows a typical vehicle-barrier height relationship and analysis. It is not sufficient that a wall be strong enough to resist the impact forces generated by a derailed vehicle. It must also be high enough to prevent the vehicle from overturning and rolling over the wall.

The analysis is consistent with that performed for the WMATA study [17], and is considered conservative. However, regardless of the barrier height determined by analysis, a minimum barrier height of 300 mm (1 foot) above vehicle floor level is recommended since this is usually the location of the *hard point* created by the floor framing system. This criterion should be modified appropriately if the vehicle framing system is not consistent with this assumption. The barrier height ideally should not impede an on objectionable line of vision from the train windows. Therefore, consideration shall also be given to maximum as well as minimum barrier height.

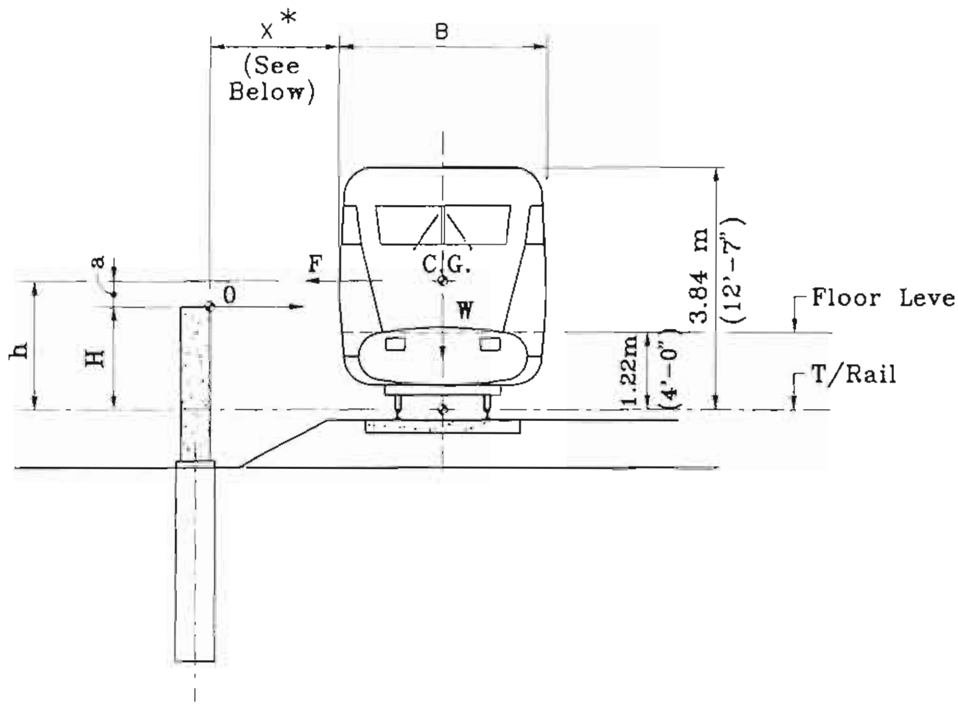
4.1.1.6 Deflections

General

The horizontal deflection of the barrier resulting from the impact is important in determining whether adjacent corridors are affected. Deflections have been calculated to determine the magnitude of deformation that the barriers undergo during an impact event. This analysis applies the impact loads determined from the TBIP runs onto the barrier, incorporating the physical properties of the structure. Resulting deflections are calculated using standard elastic theory. Since the barriers are designed to yield, however, the elastic deflections have to be modified to account for the plastic deformation that occurs as the portions of the structure deform beyond the elastic region. Maximum deformations have been estimated to a point just prior to collapse of the barrier.

The general procedure consists of the following steps:

1. Structural analysis to determine member elastic and plastic stresses and strains.
2. Determination of the regions of the member that undergo plastic deformations.
3. Calculation of the combination of elastic and plastic deflection using the moment area method.



*Note: X = 0 upon impact

Summing moments about point "0":

$$\Sigma Mo = F(h\text{-}H) - W(B/2) = 0$$

$$Fh - FH = W(B/2)$$

$$H = \frac{Fh - W(B/2)}{F}$$

FIGURE 4-5. MINIMUM BARRIER HEIGHT TO PREVENT OVERTURNING

4. Check of displacement ductility (the ratio of total deflection to the deflection at first yield) to ensure that collapse of the barrier does not occur prior to engagement of the required number of posts based on the failure mode analysis.

Concrete Barriers

Maximum deformations of the reinforced concrete barrier are directly dependent on the ductility of the members. The curvature ductility is a measure of the internal stresses and strains and can be expressed as the ratio of the ultimate curvature to the curvature at first yield. This ductility is strongly influenced by the amount of compression reinforcement and by the use of reinforcing steel hoops within the plastic hinge. Both of these factors are used to increase the maximum concrete strain in the compression zone to allow for larger deflections prior to collapse. The ACI 318 Building Code Requirements concerning *Special Provisions For Seismic Design* can be used to ensure adequate ductility.

The deformation ductility (μd) is a measure of a member's deflection just prior to collapse to its deflection at first yield (Δy) and is dependent on the estimated length of the plastic hinge (lp) that can form. The deformation ductility ratio can be used to ensure that collapse of the barrier does not occur prior to engagement of the required number of posts based on the failure mode analysis.

The total deflection of the post is determined by modeling it as a cantilever fixed at its base with a height, H. The plastic hinge will form at its base for a height approximately equal to one half of the thickness of the post. Therefore the total deflection is:

$$\Delta total = \mu d_y, \text{ where } \mu d = 1+3(\mu-1)(lp/H)(1-0.5(lp/H)),$$

where μ_c is the curvature ductility ratio and is defined in Appendix D.

The total deflection is the summation of the maximum deformations due to the post and the beam/wall members.

Steel Barriers

Maximum deformations of the steel barrier are highly dependent on the ductility of the members. The curvature ductility based on the internal stresses and strains are most affected by the strain-hardening properties of the steel and on the inelastic rotations that can occur.

The deformation ductility (μ_d) is a measure of a member's deflection just prior to collapse to its deflection at first yield (Δ_y) and is dependent on the length of the plastic hinge (l_ρ) that can occur. This is affected by strain hardening as well as by local buckling considerations of the member. The deformation ductility ratio can be used to ensure that collapse of the barrier does not occur prior to engagement of the required number of posts based on the failure mode analysis.

The total deflection of the post is determined by modeling it as a cantilever fixed at its base with a height, H. The plastic hinge will form at its base for a height approximately equal to l_ρ = tH as defined in Appendix D. Therefore the total deflection is:

$\Delta total = \mu_{dy}$, where $\mu_d = 1+3(\mu_c-1)(l_\rho /H)(1-0.5)$, and

where μ_c is the curvature ductility ratio and is defined in Appendix D.

The total deflection is the summation of the maximum deformations that occur in the post and the beam members.

4.1.2 Findings

Alternative barrier designs capable of reducing intrusion hazard are described here in detail to reflect the differences between alternates and to demonstrate their feasibility from an engineering and constructability standpoint. The designs have been developed to a high level of detail, not only determining required concrete sizes, for example, but also determining reinforcing steel requirements and critical connection details. This detail is sufficient to enable estimating of construction costs and to evaluate constructibility. This detail should not create a false sense of trust in the designs, however. As stated in Chapter 3.1.2, the analysis methodology used to estimate impact forces is based, of necessity, on a number of assumptions. Many of these assumptions have never been tested; for example, the crush stiffness of the HSGGT or railroad vehicles. One of the recommendations of this study, made in Chapter 7.5.1, is that these assumptions be verified through a testing program before the designs presented in this report are used in practice.

Barrier design loads were determined using the TBIP computer program (See Chapter 3.1) for all of the railroad and HSGGT scenarios shown in Table 2-1. The loads, summarized in Table 3-3, represent the maximum loads resulting from literally hundreds of TBIP runs made for different values of the variables previously discussed. Allowance has been made for rotational-induced loads resulting from three-dimensional effects as described in Chapter 3.1.3.2 to arrive at the loads shown in the table.

Eight alternative railroad and HSGGT types of barrier designs, each capable of resisting the loads in Table 2-1 applied at the top of the barrier, are presented below. Five alternates are for at-grade applications, and three for elevated structures such as bridge decks. These designs represent common construction techniques that have been widely used for other types of structures throughout the United States. All of the designs can effectively resist intrusion from errant vehicles. The choice of alternate will be made primarily based on local economies of the different construction materials and methods.

The 18 train vehicle/barrier scenarios have been grouped by impact force magnitude resulting from the TBIP analyses, and designs have been developed for each force level for the eight barrier types. A total of 31 different designs have thus been developed.

Figure 4-6 shows the intrusion scenarios associated with each barrier type.The designs are shown in Figures 4-7 through 4-31. Preliminary plans, sections and details are shown for a longitudinal free-standing wall or railing system supported by an at-grade deep foundation system, or by an elevated bridge deck. Retaining wall barriers are also shown. The eight barrier design alternatives developed in this study consist of:

At-Grade Barriers

AG-1: Precast Concrete Wall and Foundation (See Figures 4-7 through 4-9)
AG-2: Precast Concrete Wall and Steel Foundation (See Figures 4-10 through 4-12)
AG-3: Cast-In-Place Concrete Wall and Foundation (See Figures 4-13 through 4-15)
AG-4: Structural Steel Railing and Foundation (See Figures 4-16 through 4-18)
AG-5: Cast-In-Place Concrete Retaining Wall (See Figure 4-19)

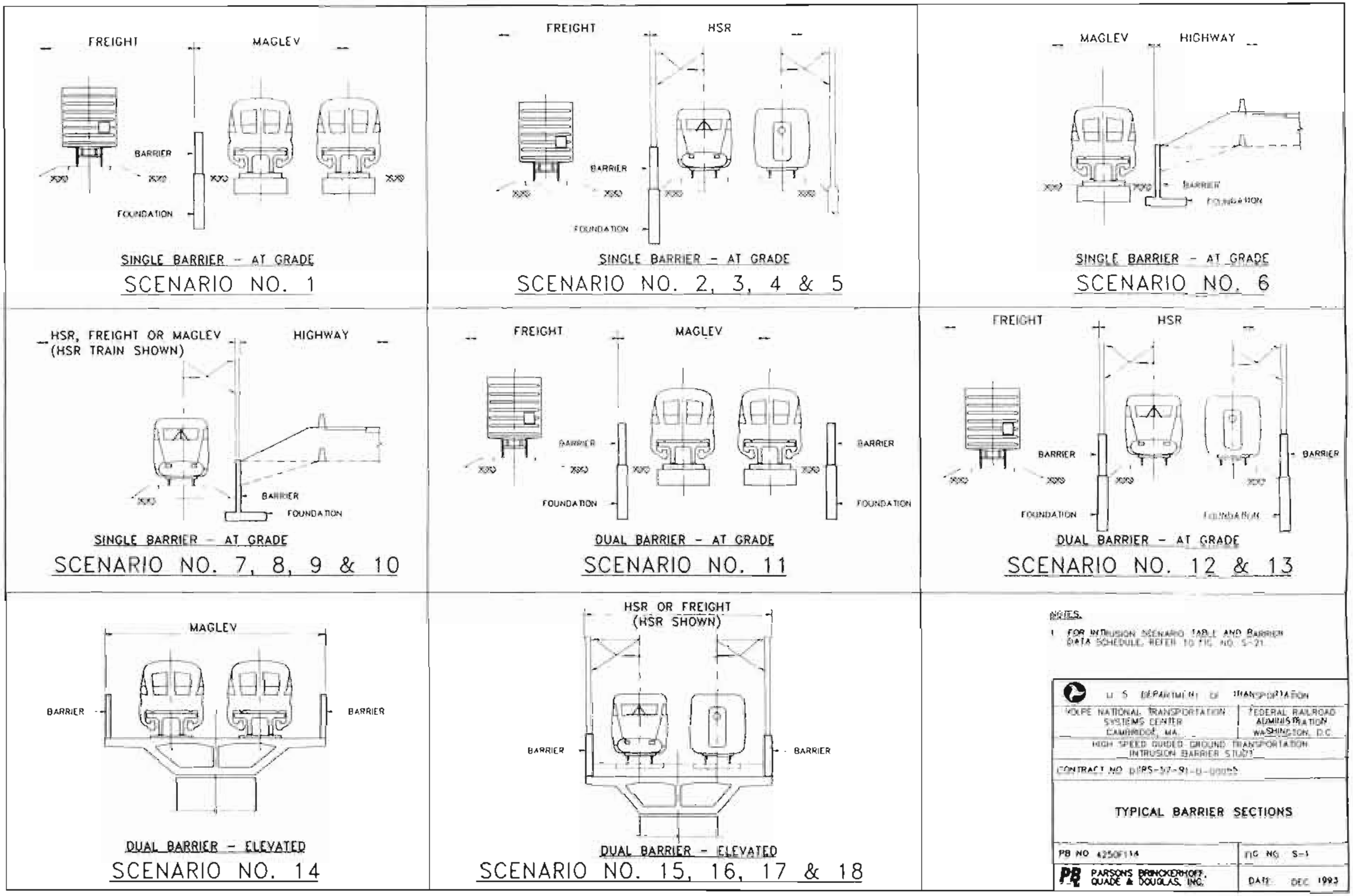


FIGURE 4-6. TYPICAL BARRIER SECTIONS

Elevated Barriers

EL-1: Precast Concrete Wall (See Figures 4-20 and 4-21)

EL-2: Cast-In-Place Concrete Wall (See Figures 4-22 and 4-23)

EL-3: Structural Steel Railing (See Figures 4-24 and 4-25)

Figure 4-26 summarizes all the intrusion scenarios, barrier types, design alternates, design loads, and structural dimensions.

All designs utilize an essentially linear wall structure minimizing the need for right-of-way acquisition, in contrast to frame type structures. Another feature common to all alternates is the detailing of the reinforcement and the connections. Since more than one column or post is relied on to effectively distribute the impact load, all members of the barrier structure (wall - column - foundation) are continuously tied together and the reinforcement is continuous throughout each member and at the supports. This serves to provide the continuity needed to bridge the damaged or yielded support. By making the reinforcement continuous and the connections capable of resisting shear and moment reversals, the integrity of the overall structure is greatly improved and thereby better able to maintain its effectiveness as an intrusion barrier, even after impact.

4.1.2.1 At Grade Alternate 1 (AG1): Precast Concrete Wall and Precast Concrete Foundation

In this alternate, which is shown in Figures 4-7 through 4-9, the entire barrier structure is constructed of precast concrete, with the following components:

- Prestressed square piles below grade with either a solid or hollow core
- Square columns above grade with conventional (not prestressed) reinforcing steel
- Concrete wall panels with conventional reinforcing steel

The piles are driven into the ground at a spacing ranging from 2.74 meters (9 feet) to 4.57 meters (15 feet) and project 150 millimeters (6 inches) above the subgrade. The piles vary in size from 559 mm x 559 mm (22" x 22") solid sections to 914 mm x 914 mm (36" x 36") hollow core sections. They are driven to embedment depths ranging from 4.27 meters (14 feet) to 7.31 meters (24 feet). The connection

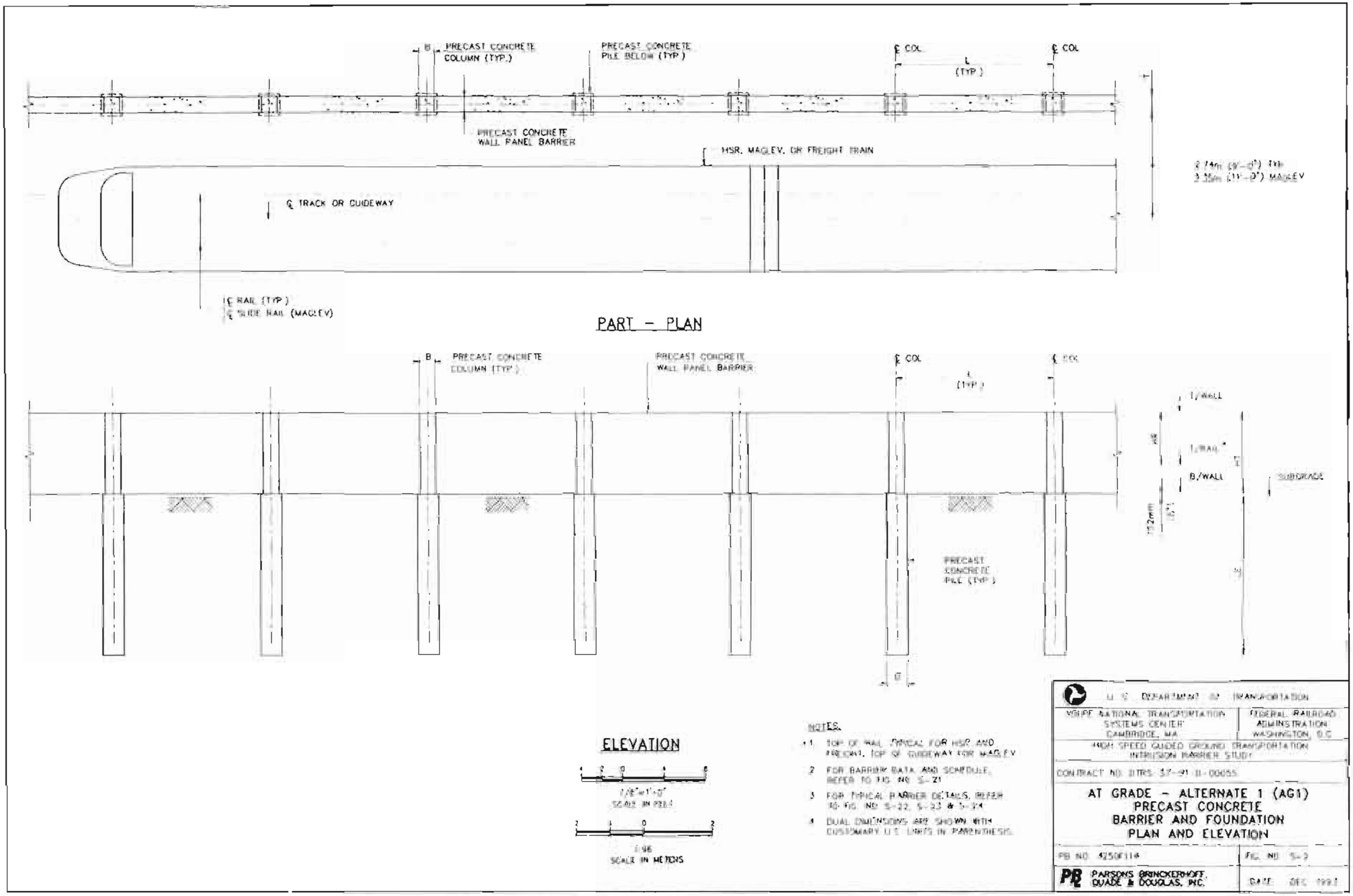


FIGURE 4-7. AT GRADE ALTERNATE 1: PRECAST CONCRETE (P/C) WALL AND P/C FOUNDATION - PLAN AND ELEVATION

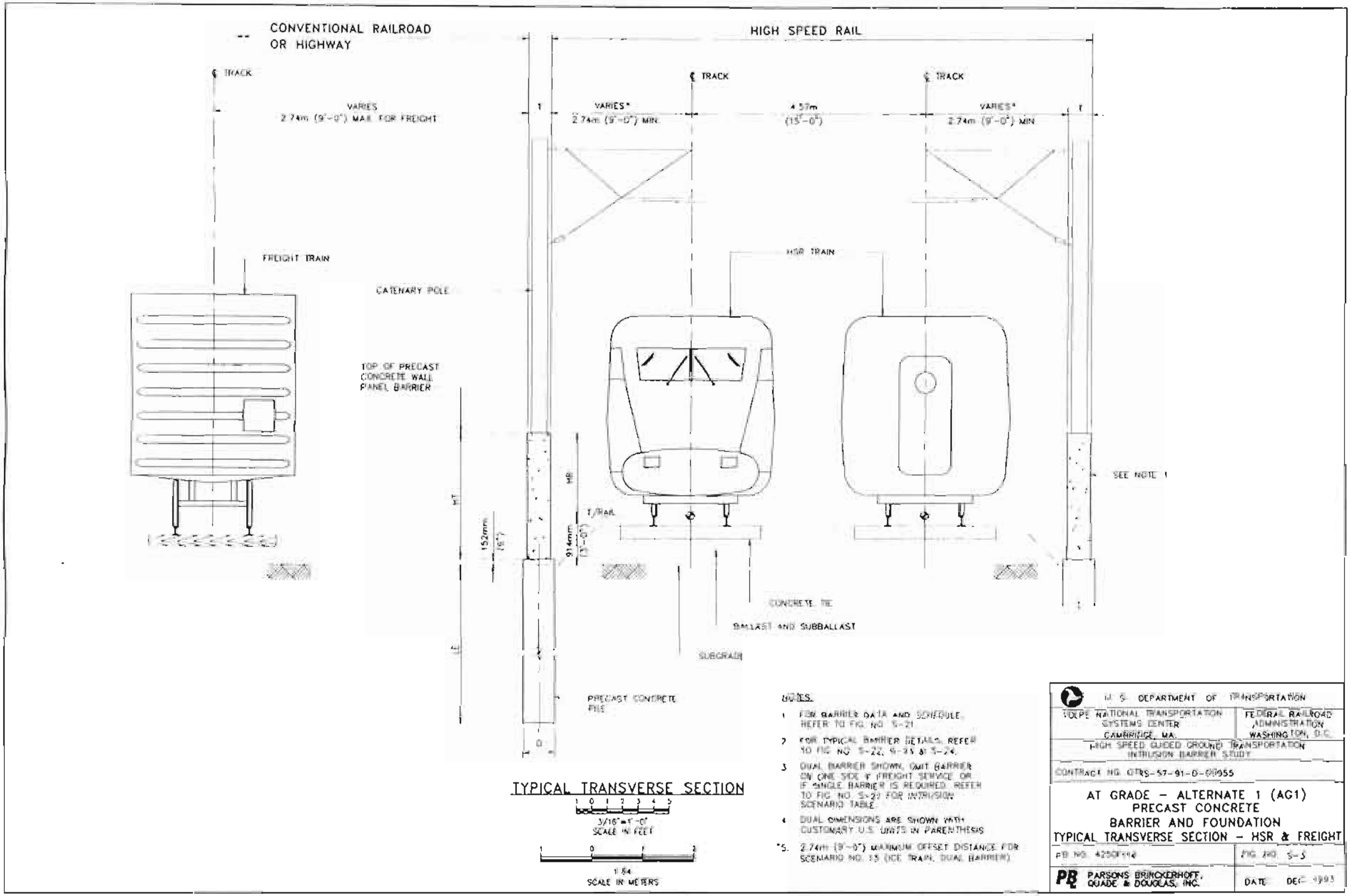


FIGURE 4-8. AT GRADE ALTERNATE 1: P/C WALL AND P/C FOUNDATION - TRANSVERSE SECTION/RR

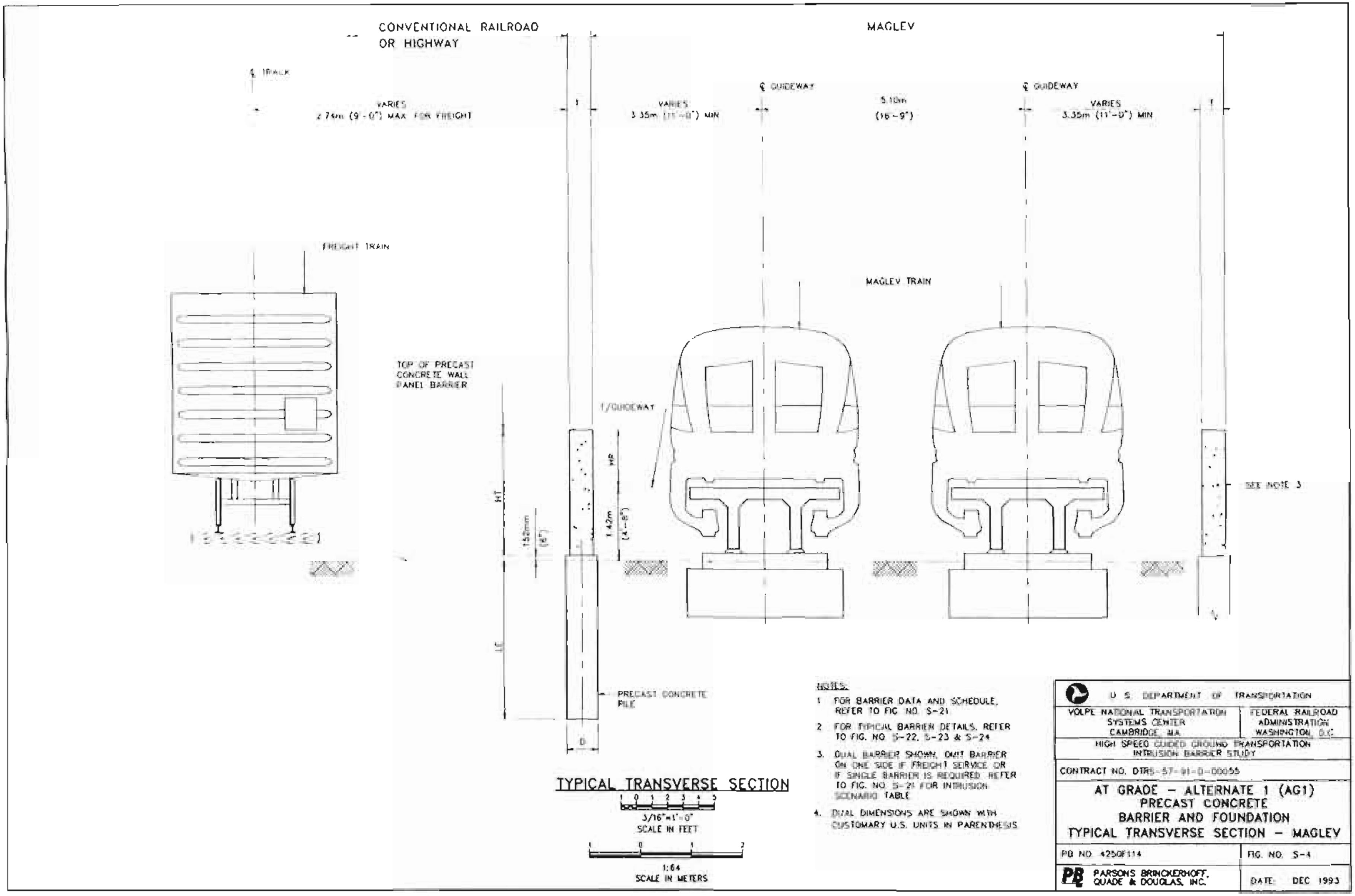


FIGURE 4-9. AT GRADE ALTERNATE 1: P/C WALL AND P/C FOUNDATION - TRANSVERSE SECTION/MAGLEV

between the pile head and the base of the column is achieved through the use of mechanical connections consisting of splice sleeves filled with high-strength epoxy grout. These connections must be designed to be capable of achieving over 125% of the yield strength of the reinforcement both in tension and compression. This connection is shown in Figure 4-23. The top of the pile has reinforcement embedded in oversized sleeves cast-in to allow for construction tolerances.

The columns assume the same spacing as the piles and vary in size from 457mm x 457mm (18" x 18") to 762 mm x 762 mm (30" x 30"). The height of the columns above grade is the same for all at-grade alternates and varies according to scenario from 2.44 meters (8 feet) above subgrade, typically, to 2.74 meters (9 feet) for the larger impact forces (1.52 meters (5 feet) and 1.83 meters (6 feet) above the top of the rail).

The wall panels vary in thickness from 457 mm (18") to 762 mm (30") and are installed between the columns with a 25 mm (1 inch) joint spacing at each end. The wall-to-column connection, shown in Figure 4-29, is accomplished with four rows of plates along the height of the column. These plates are welded to plates embedded in the column and the panels. To achieve continuity of reinforcement and fixity at the joints, the embedded plates are provided on both faces of the joint and the horizontal wall reinforcement is welded to the embedded plates. After the plates are welded, the joint between the column and the wall is filled solid with non-shrink grout and sealed all around to prevent water intrusion.

4.1.2.2 At Grade Alternate 2 (AG2): Precast Concrete Wall and Steel Foundation

This alternate, shown in Figures 4-10 through 4-12, is similar to Alternate 1 except that the columns and piles are structural steel wide flange sections. The barrier structure consists of the following components:

- Wide flange structural steel piles
- Wide flange structural steel columns encased in concrete
- Precast concrete wall panels with conventional reinforcing steel

The piles are driven into the ground at spacing ranging from 4.27 meters (14 feet) to 4.57 meters (15 feet) and project 152 mm (6 inches) above the subgrade. The piles sections vary from W254 x 89 kg/m (W10 x 60 lbs/ft) to W356 x 635 kg/m (W14 x 426 lbs/ft), and have a welded cap plate at the top to

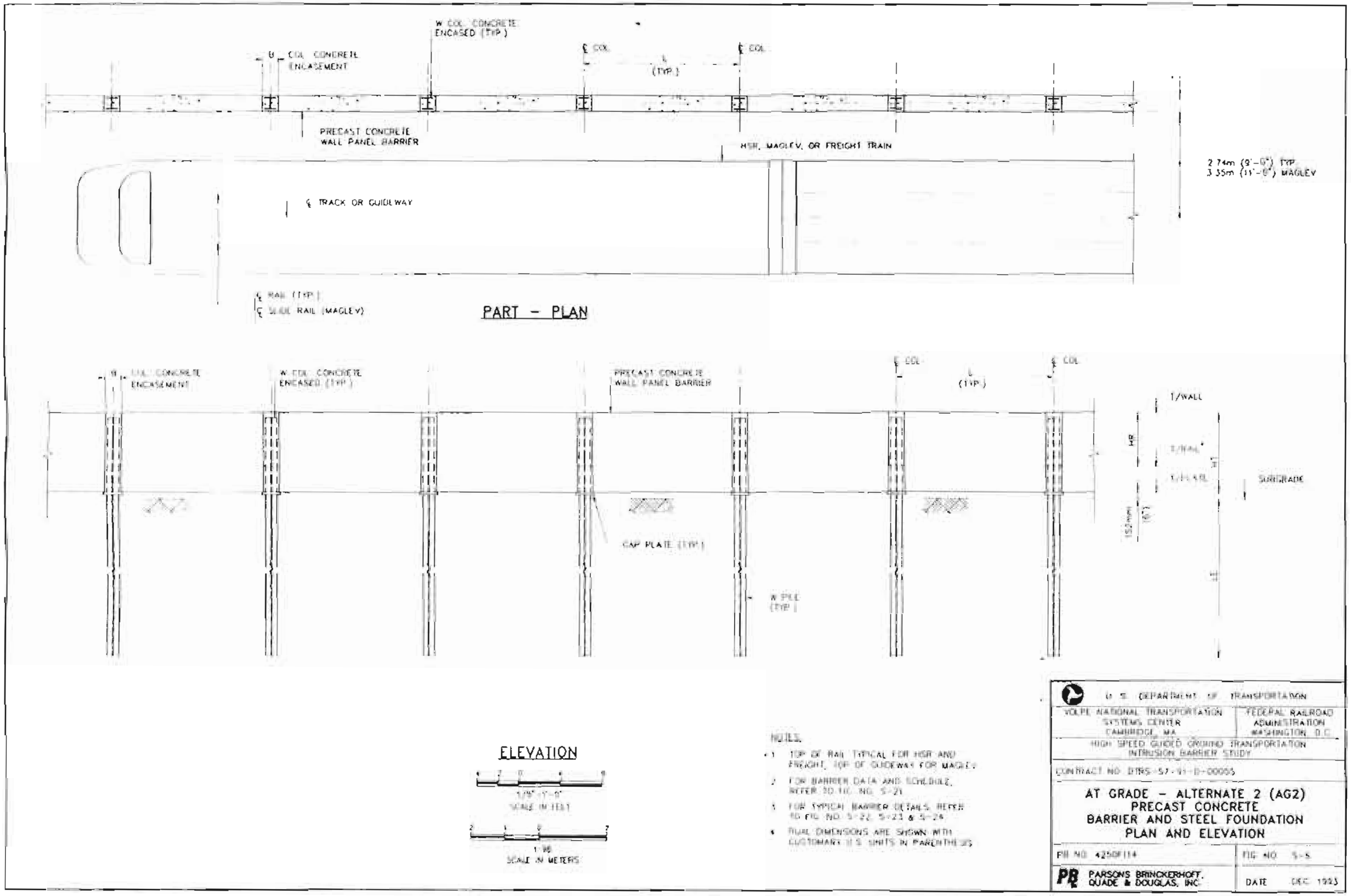


FIGURE 4-10. AT GRADE ALTERNATE 2: P/C WALL AND STEEL FOUNDATION - PLAN AND ELEVATION

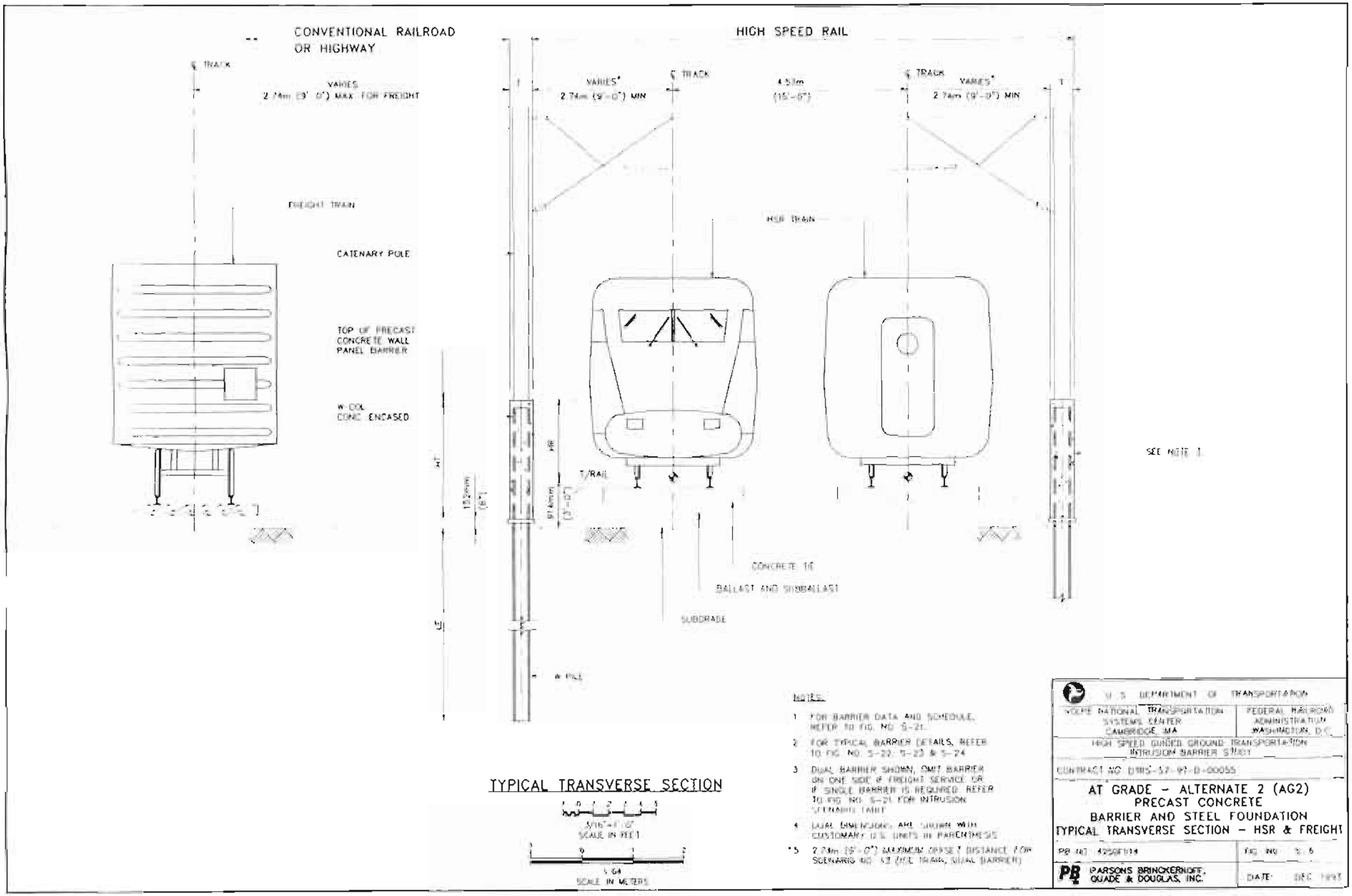


FIGURE 4-11. AT GRADE ALTERNATE 2: P/C WALL AND STEEL FOUNDATION - TRANSVERSE SECTION/RR

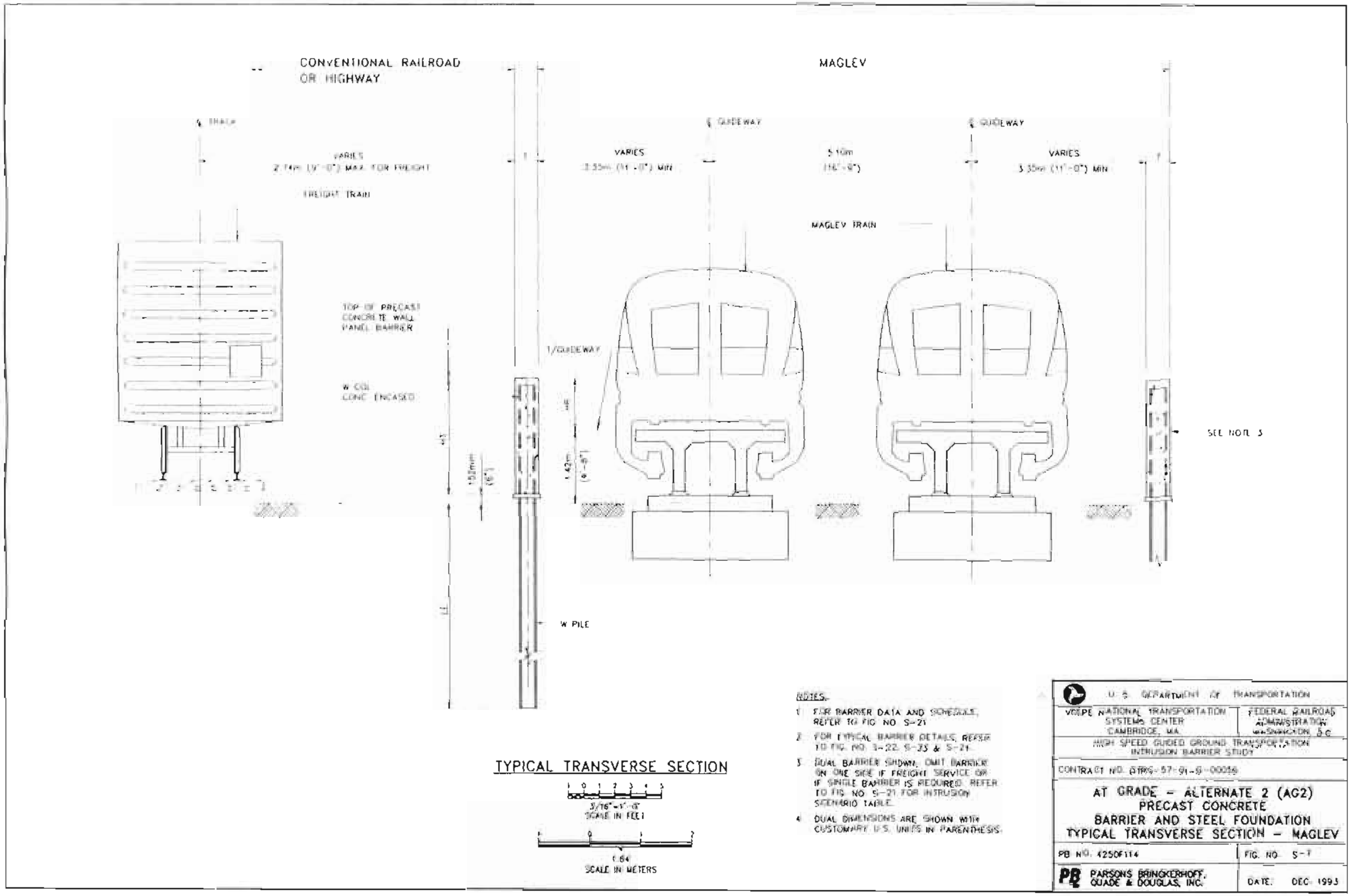


FIGURE 4-12. AT GRADE ALTERNATE 2: P/C WALL AND STEEL FOUNDATION - TRANSVERSE SECTION/MAGLEV

support the steel columns. The connection between the pile top and column base is achieved by field welding the base of the column to the pile cap plate to facilitate the welding. The column base extends beyond the concrete encasement by 51 millimeters (2 inches) and this joint is drypacked with non-shrink grout after welding is completed.

The column size and spacing are the same as for the piles. The top of the steel columns is set 76 millimeters (3 inches) below the top of concrete encasement for corrosion protection.

Wall panel construction and characteristics are the same as for Alternate 1 except that thickness varies from 457 mm (18") to 711 mm (28"). This slightly reduced thickness is attributed to the greater strength provided by the steel column sections.

4.1.2.3 At Grade Alternate 3 (AG3): Cast-in-Place (C.I.P.) Concrete Wall and C.I.P. Concrete Foundation

This alternate design is shown in Figures 4-13 through 4-15. It consists of an all cast-in-place concrete barrier structure with the following components:

- Reinforced concrete pier foundations (caissons)
- Reinforced cast-in-place concrete wall

The caissons are installed in the ground at spacings ranging from 3.66 meters (12 feet) to 4.88 meters (16 feet). As with the first two alternates they project 152 millimeters (6 inches) above the subgrade. Caisson diameter varies from 762 mm (30") to 1219 mm (48"), with embedment depths ranging from 3.96 meters (13 feet) to 6.71 meters (22 feet).

The wall is cast on top of the caissons with reinforced column sections at the caisson locations as shown in Figure 4-27. The wall/column reinforcement extends into the caisson for fixity. The wall thickness varies from 508 mm (20") to 1016 mm (40") with the column section sizes ranging from 610 mm x 508 mm (24" x 20") to 914 mm x 1016 mm (36" x 40"). The height of the wall barrier is the same as the other alternates. As before the horizontal wall reinforcement is continuous through the wall/column sections to provide moment transfer and fixity.

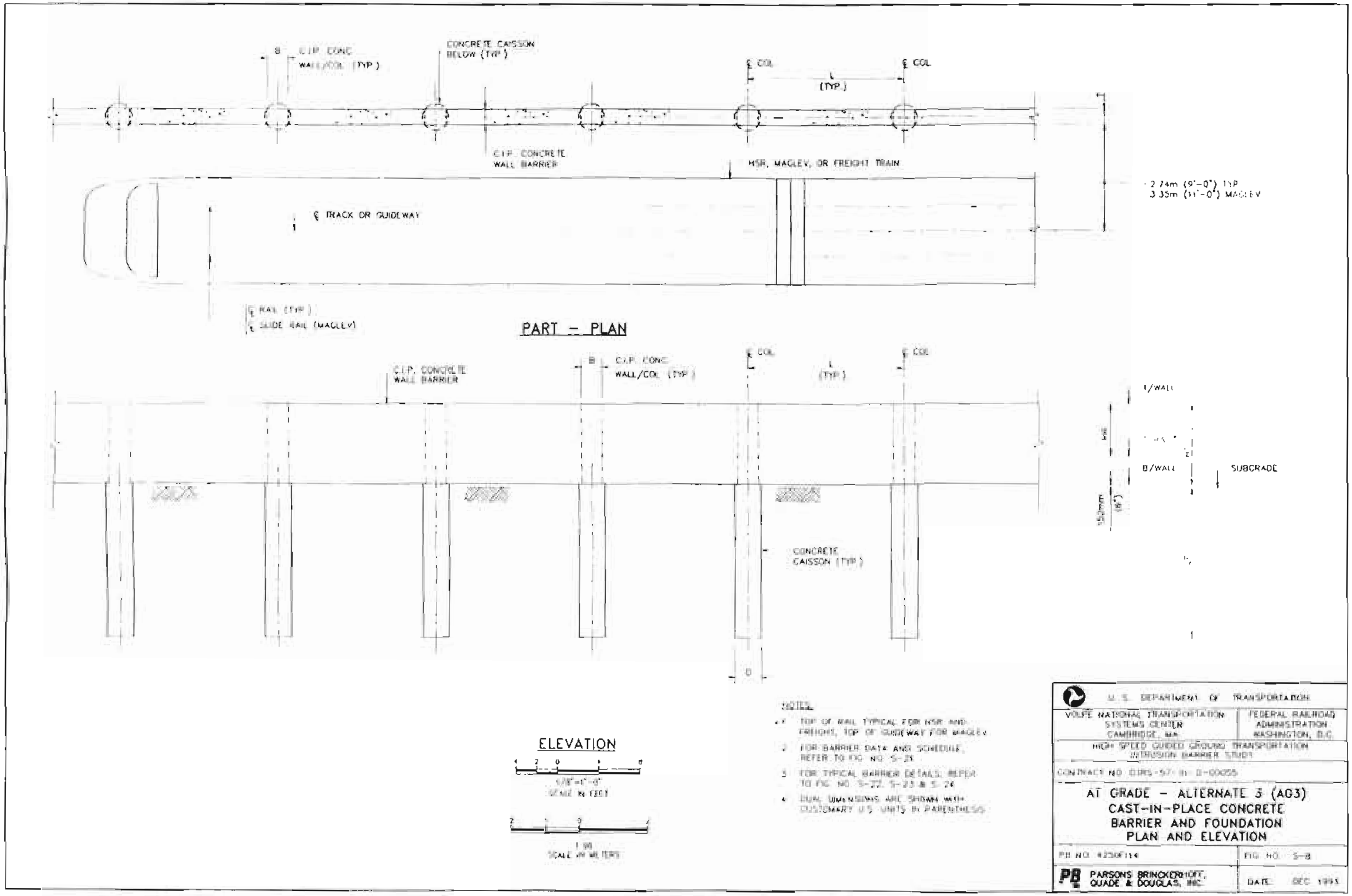


FIGURE 4-13. AT GRADE ALTERNATE 3: CAST-IN-PLACE (C.I.P.) CONC. WALL AND C.I.P. FOUNDATION - PLAN AND ELEVATION

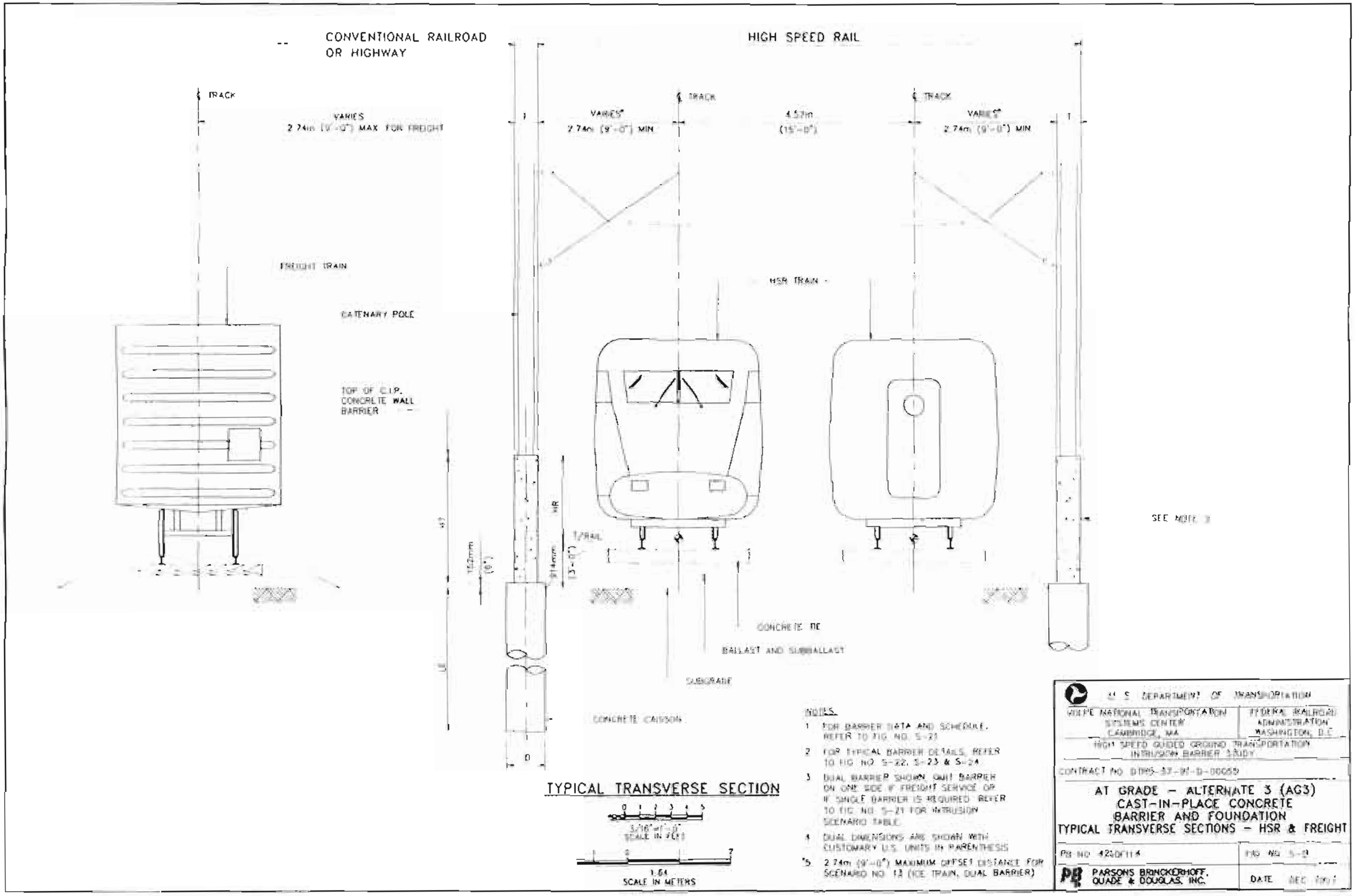


FIGURE 4-14. AT GRADE ALTERNATE 3: C.I.P. CONC. WALL AND C.I.P. CONC. FOUNDATION - TRANSVERSE SECTION/RR

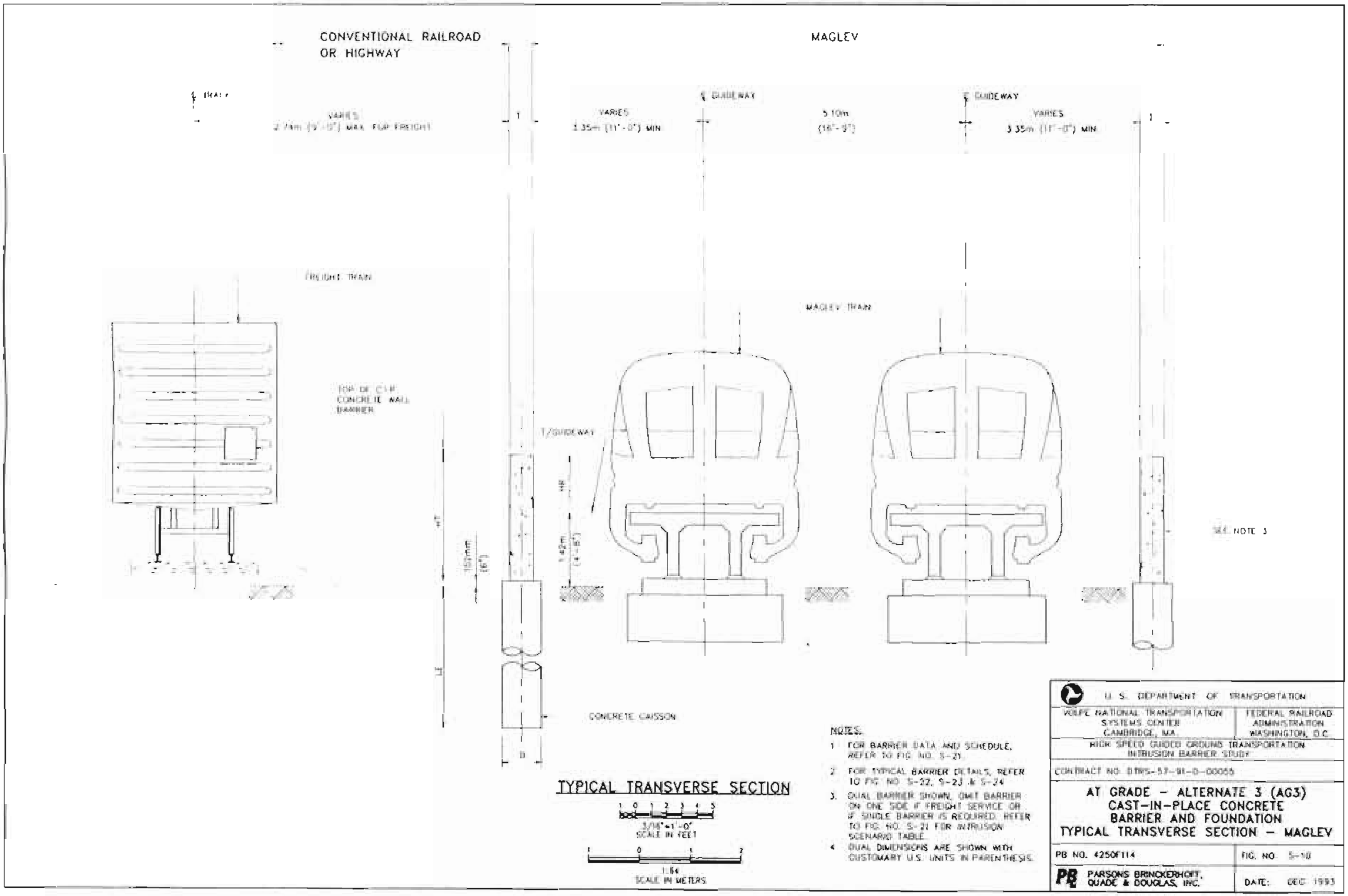


FIGURE 4-15. AT GRADE ALTERNATE 3: C.I.P. CONC. WALL AND C.I.P. CONC. FOUNDATION - TRANSVERSE SECTION/MAGLEV

4.1.2.4 At Grade Alternate 4 (AG4): Structural Steel Post, Railing and Foundation

This all structural steel alternate is shown in Figures 4-16 through 4-18 and consists of the following components:

- Steel pipe piles
- Steel pipe columns
- Steel pipe beams and rails
- Steel stiffener wall plate

The piles are driven into the ground at spacings ranging from 3.05 meters (10 feet) to 5.79 meters (19 feet) and, similar to all alternates, they project 152 millimeters (6 inches) above the subgrade. Embedment depths range from 5.18 meters (17 feet) to 8.84 meters (29 feet). The pile, column and top beam sizes are the same for economy, and to minimize snagging hazards. This also simplifies field connections which are all welded to achieve fixity and continuity and to ensure the proper load distribution. These member sizes vary from 406 mm diameter pipe by 16.7 mm wall thickness (16" diameter by 0.656 inch wall) to 610 mm diameter pipe by 31 mm wall thickness (24" diameter by 1.218 inch wall). Pipe sections were selected because they have the same strength in all directions, they are efficient sections with the ability of achieving great structural capacity with relatively small sizes, and their smooth profile minimizes snagging potential.

The pipe rails and the stiffener wall plate are provided to brace the top beam in the vertical and longitudinal directions as well as to prevent intrusion and snagging on the columns or beams. Where this system is used between HSGGT and RR guideways to prevent intrusion from both sides, the stiffener plate would be provided on both sides.

The connection between the pile top and the column base is accomplished by field welding the base of the column to the pile cap plate. The beam-to-column connection is a field welded full moment connection. These details are shown in Figures 4-28 and 4-29.

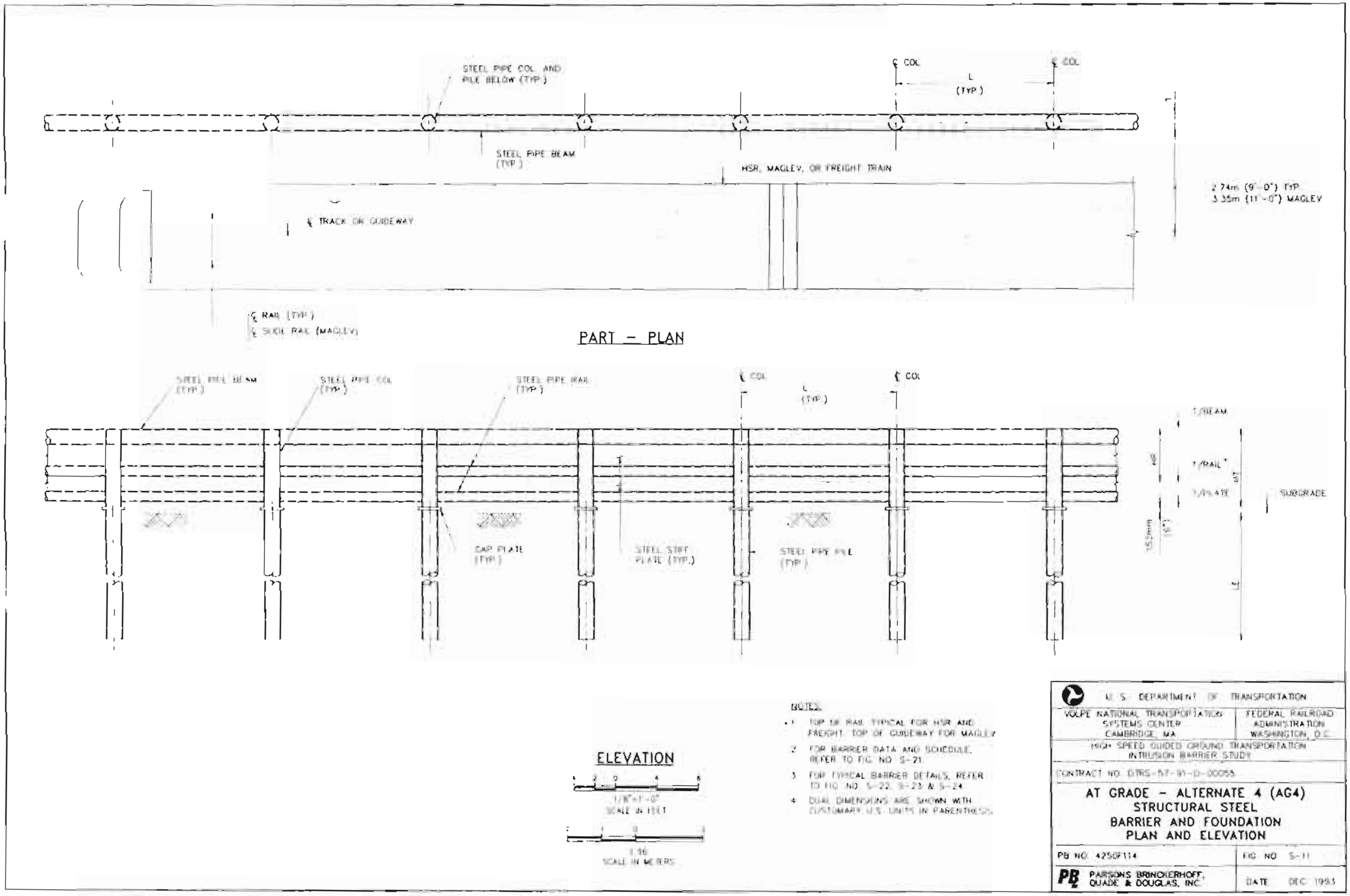


FIGURE 4-16. AT GRADE ALTERNATE 4: STRUCTURAL STEEL POST, RAILING AND FOUNDATION - PLAN AND ELEVATION

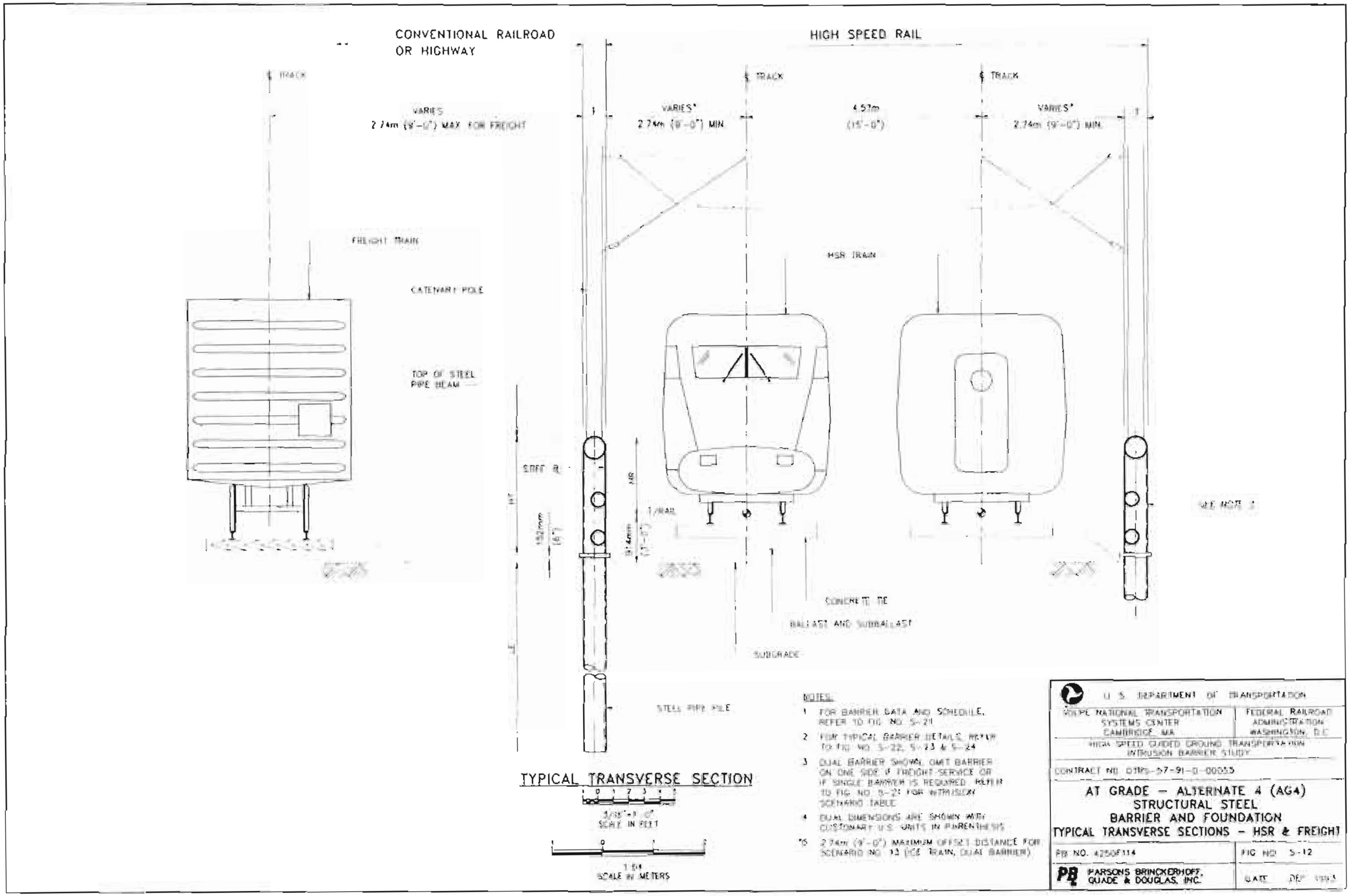


FIGURE 4-17. AT GRADE ALTERNATE 4: STRUCTURAL STEEL POST, RAILING AND FOUNDATION - TRANSVERSE SECTION/RR

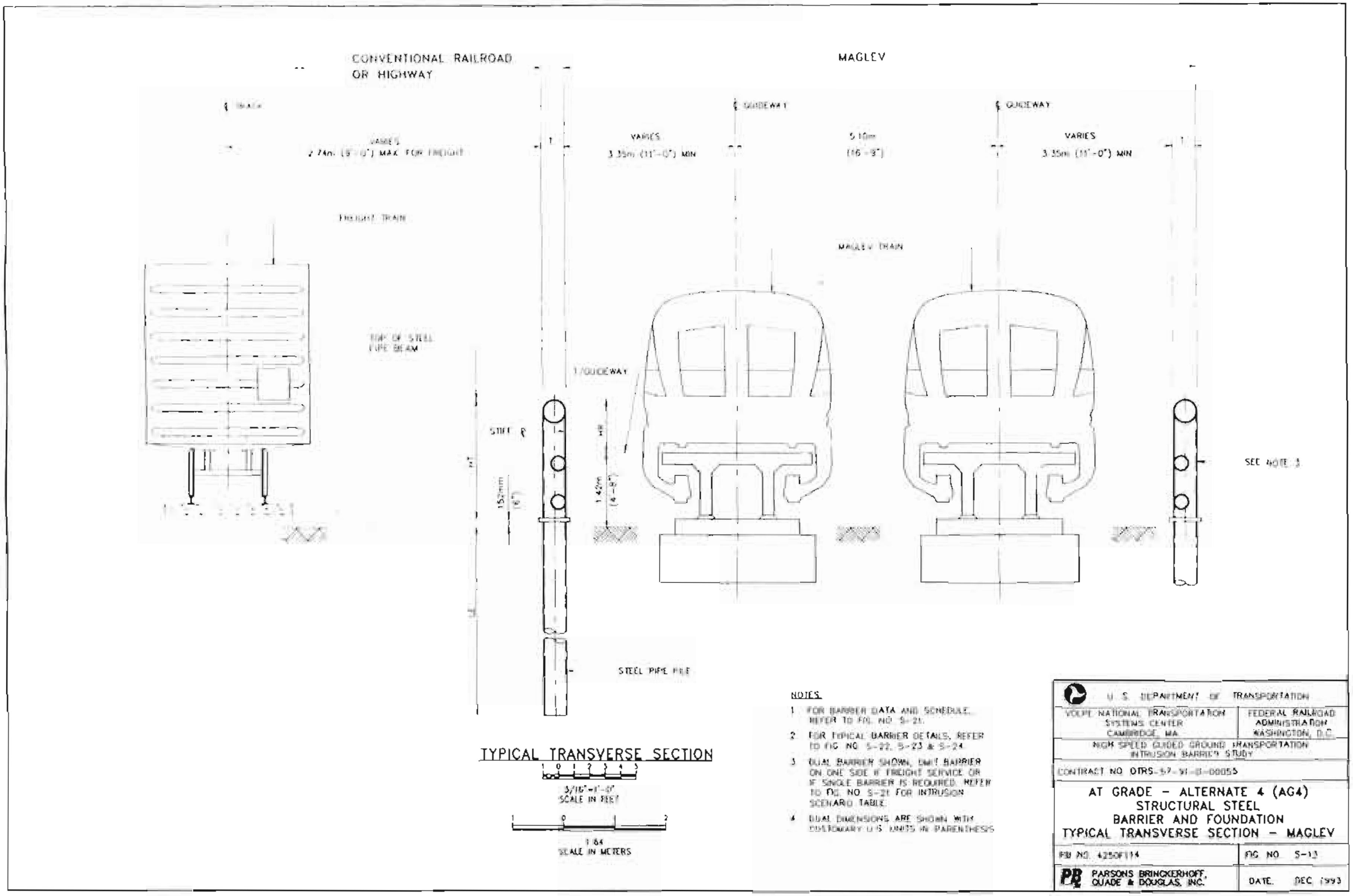


FIGURE 4-18. AT GRADE ALTERNATE 4: STRUCTURAL STEEL POST, RAILING AND FOUNDATION - TRANSVERSE SECTION/MAGLEV

4.1.2.5 At Grade Alternate 5 (AG5): C.I.P. Concrete Retaining Wall Barrier

This alternate, shown in Figure 4-19, consists of a conventional cast-in-place concrete retaining wall designed to resist both the lateral earth pressures and the impact forces generated by a derailed train. Unlike the structural barriers, this is a combination structural/earthwork system. The lateral impact loads are resisted by a combination of earth pressure (developed through a passive lateral earth pressure force) and the ultimate moment capacity of the reinforced concrete wall itself.

The typical wall reinforcing details are shown in Figure 4-27. The wall thickness varies from 305 mm (12") typically to 457 mm (18") for the larger impact forces. The bottom of the wall footing is set at 1219 mm (48") below the top of subgrade for frost protection in cold weather areas (note: this depth will vary by local climate, as will the other dimensions along with it). The footing width varies from 2.44 meters (8 feet) typically to 2.74 meters (9 feet) and the thickness from 457 mm (18") to 610 mm (24").

The retaining wall height above subgrade varies from 2.44 meters (8 feet), typically, to 2.74 meters (9 feet), and the width of solid backfill required to resist the design impact loads is 9.14 meters (30 feet) measured horizontally from the back face of the retaining wall.

Overall, this prototype design illustrates a typical two-lane highway situation within a shared right-of-way with a high speed rail line. Here the HSGGT guideway layout is shown vertically depressed in relationship to the elevation of the highway. Further, two possible cases are presented with respect to the vertical elevation of the adjacent roadway. Case I illustrates the condition where the vertical alignment of the adjacent roadway is higher than the HSGGT guideway. Case II illustrates the condition where the existing roadway is lower - closer to the elevation of the HSGGT.

These barriers are designed utilizing loads from the TBIP. Parameters for active earth pressures behind the wall are used for resistance of these loads, assuming granular soil. The walls are designed as normal retaining walls, resisting lateral earth loads acting in one direction, and are also designed to distribute the impact loads in the longitudinal direction.

A minimum setback distance of 9.1 meters (30 feet) to the adjacent guideway is specified. It is anticipated that soil in this area would be disturbed by an impact from an errant HSGGT vehicle. In order to minimize disruption to the adjacent facility, it should be located outside of this zone. Where this is

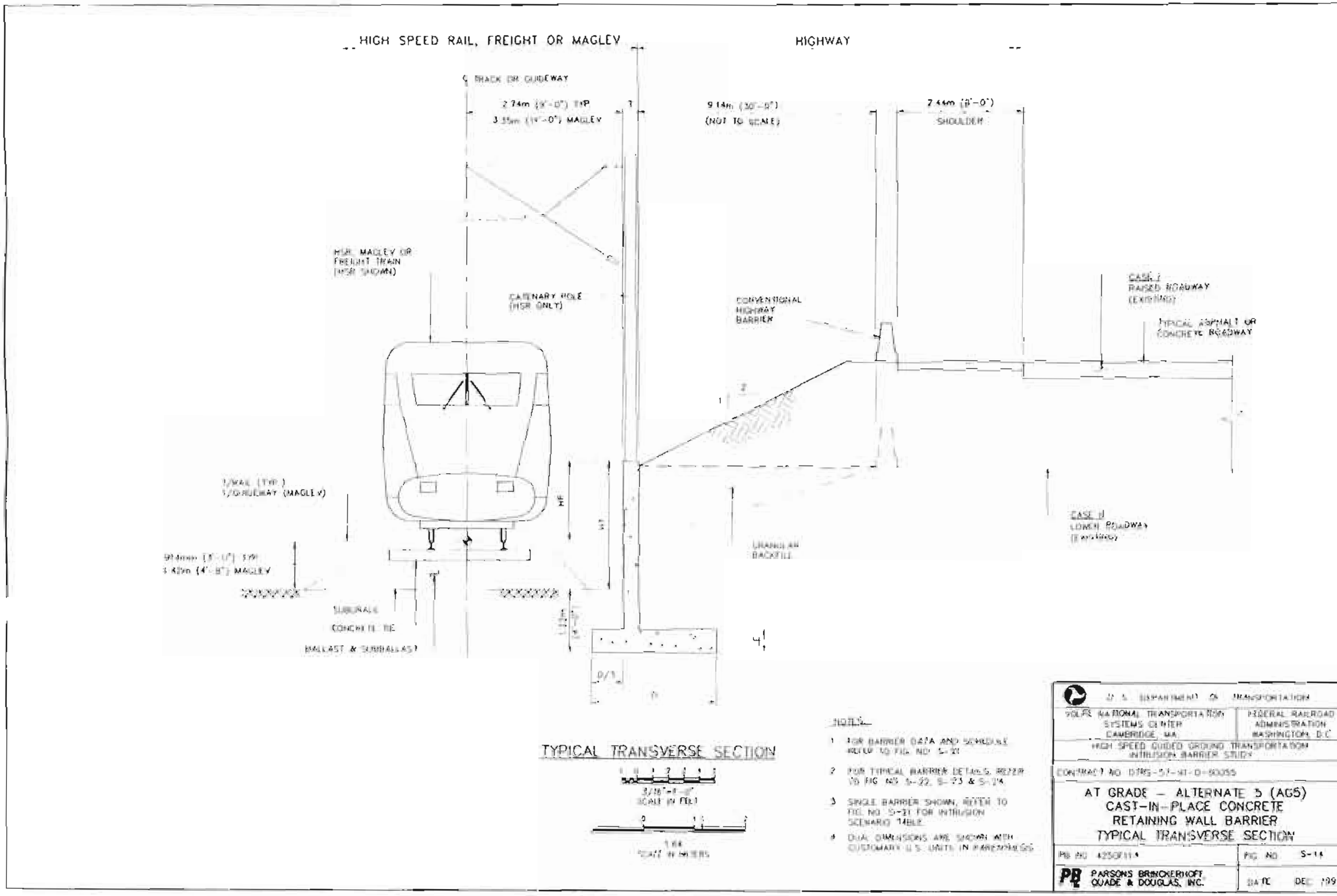


FIGURE 4-19. AT GRADE ALTERNATE 5: C.I.P. CONCRETE RETAINING WALL BARRIER - TRANSVERSE SECTION/RR & MAGLEV

geometrically impossible or difficult to accomplish, the setback could be reduced, with the understanding that a vehicle impact could cause damage to the adjacent facility.

Although cast-in-place reinforced concrete retaining walls are shown, precast concrete Reinforced Earth or Doublewal designs (both proprietary) are other options that may prove to be cost effective in certain areas due to local practices and availability. Also the precast elements could prove beneficial for modular replacement of damaged elements.

Reinforced Earth walls are composed of precast wall panels with metal reinforcing strips extending backward into the soil. These barriers have the disadvantage that more right-of-way is required for their construction. Doublewal systems are composed of large precast concrete blocks similar to masonry blocks. Both of these types could offer the additional benefit of quick repair via replacement of the modular precast components.

The designs for the Reinforced Earth and Doublewal designs have not been shown. Both of these proprietary types of walls are commonly designed by the manufacturer for the loading conditions specified in the contract documents. Costs are typically approximately equivalent to the cast-in-place design shown, again with local variation. For the purpose of constructability and cost estimating, therefore, the presentation of only the cast-in-place retaining wall is adequate.

4.1.2.6 Elevated Alternate 1 (EL1): Precast Concrete Wall

The elevated alternates would be installed on overhead bridge structures. When barrier structures are installed on existing construction, the existing slab or supporting members must be strong enough to resist the forces imposed by the barrier on the slab or supporting member. Depending on the scenario involved, the existing construction may have to be modified and strengthened. Due to the magnitude of impact forces involved, the modifications would likely be significant.

The EL1 alternate which is shown in Figures 4-20 and 4-21 consists of precast concrete wall panels continuously attached to the reinforced concrete bridge deck slab or beam. The wall thickness varies from 305 mm (12") to 1016 mm (40") and is fixed to the slab with mechanical connections consisting of splice sleeves filled with high-strength epoxy grout, as shown in Figure 4-30. This connector is similar to that used with the precast concrete at grade alternative AG-1. In the case of an existing concrete deck slab, dowels would have to be installed by drilling and grouting in the slab.

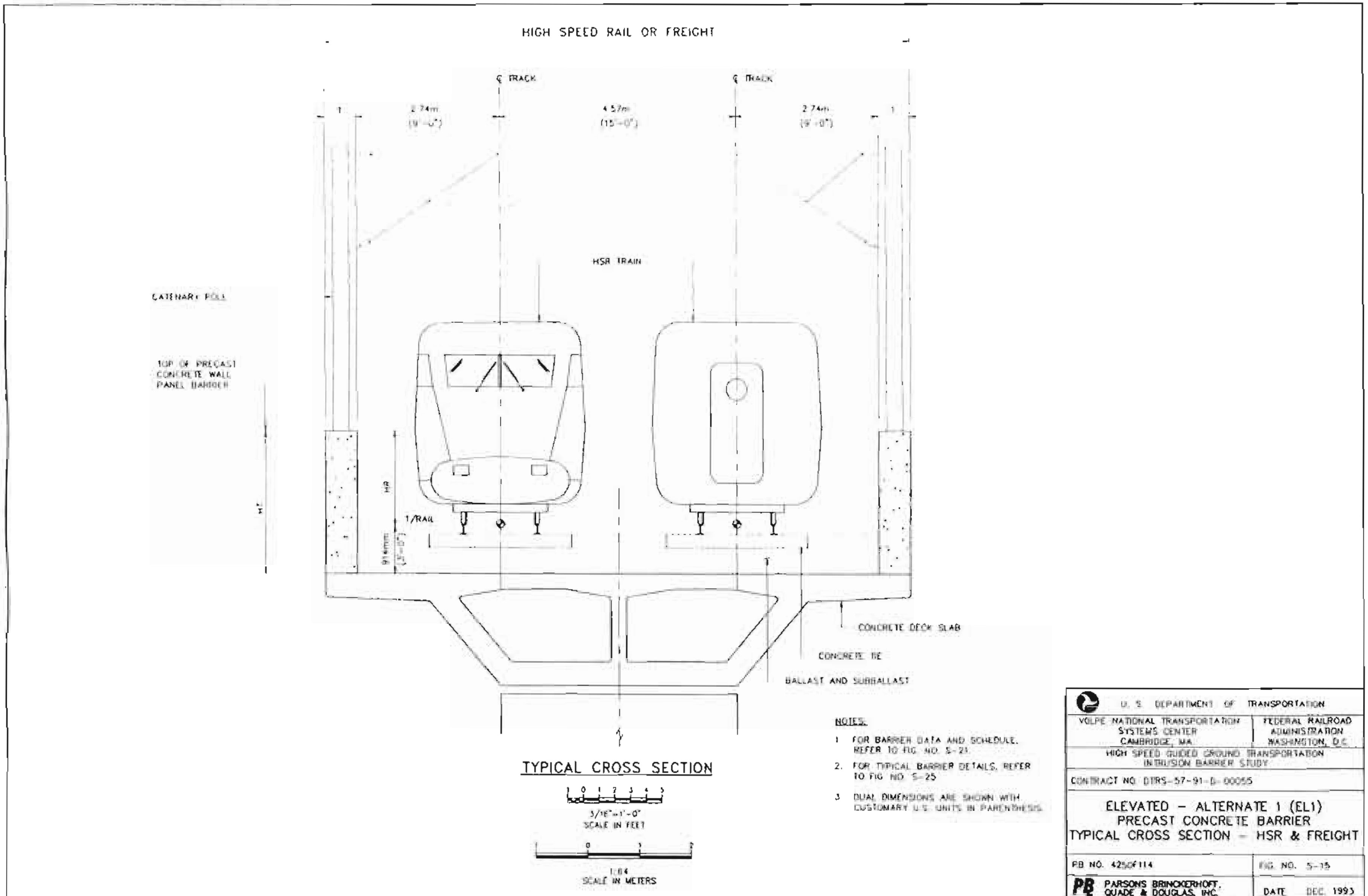


FIGURE 4-20. ELEVATED ALTERNATE 1: PRECAST CONCRETE WALL - TRANSVERSE SECTION/RR

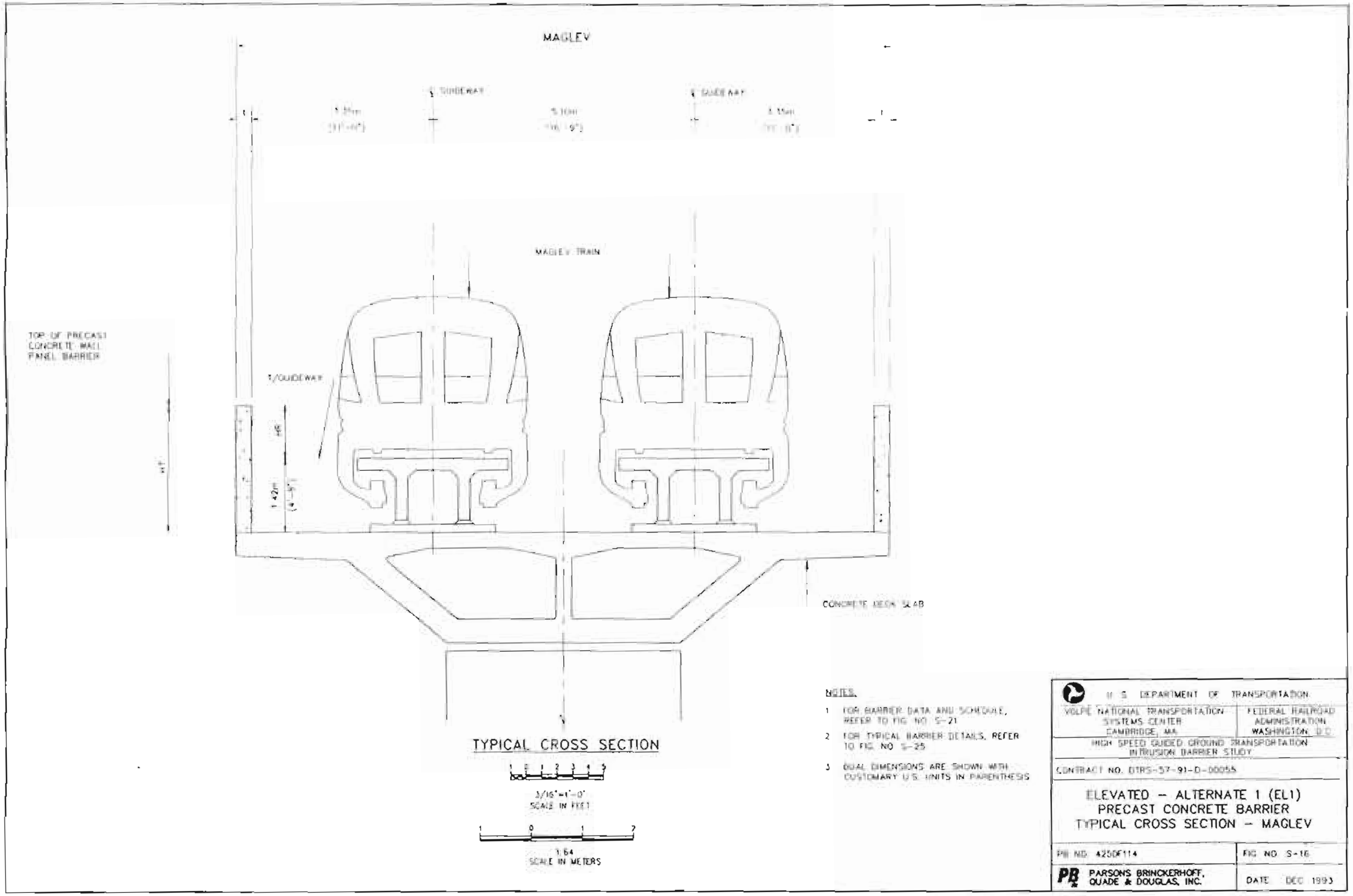


FIGURE 4-21. ELEVATED ALTERNATE 1: PRECAST CONCRETE WALL - TRANSVERSE SECTION/MAGLEV

The total height of the wall from the top of the slab is the same for all elevated alternates and varies depending on the vehicle type from 2.44 meters (8 feet) to 2.90 meters (9' - 6") to 1.52 meters (5 feet) and to 1.98 meters (6'-6") above the top of the rail). The length of each wall panel is only limited by weight and transportation requirements. The wall-to-wall connection as shown in Figure 4-30 is also achieved using mechanical connections similar to the wall base connector. In both connections, the joint is filled solid with non-shrink grout and sealed all around to prevent water intrusion.

4.1.2.7 Elevated Alternate 2 (EL2): C.I.P. Concrete Wall

The cast-in-place concrete alternate shown in Figures 4-22 and 4-23, and detailed in Figure 4-30 consists of a reinforced concrete wall cast on top of a concrete deck slab. The wall thickness varies from 356 mm (14") to 106 mm (40") and is anchored to the slab with projecting dowels. As with the previous alternate EL1, in the case of an existing deck slab, dowels would have to be installed by drilling and grouting into the slab.

4.1.2.8 Elevated Alternate 3 (EL3): Structural Steel Post and Railing

The structural steel alternate is shown in Figures 4-24 and 4-25, and detailed in Figure 4-30. It is similar to the at-grade steel alternate AG4 with the exception that the columns are fixed to a deck slab instead of a deep pile foundation. The connection to the deck slab is achieved through the use of base plates and anchor bolts as shown on Figure 4-30. Modification and strengthening of the deck structure may be more significant for this elevated alternate because the load applied by the posts is a concentrated load, in contrast to the uniformly distributed load applied by a wall type barrier.

4.1.2.9 Deflections

Deflections were calculated as described in Section 4.1.1.6. Calculations were not performed for all barriers. Instead, representative barriers were analyzed to determine the order of magnitude deflections that could be expected. The AG3 and AG4 barriers, under a loading of 1335 kN (300 kips) were considered to be representative of the usual loads that the steel and concrete barriers would sustain. A summary of the deflection results is given in Table 4-1. The calculations, and a derivation of the analytical approach is included in Appendix D.

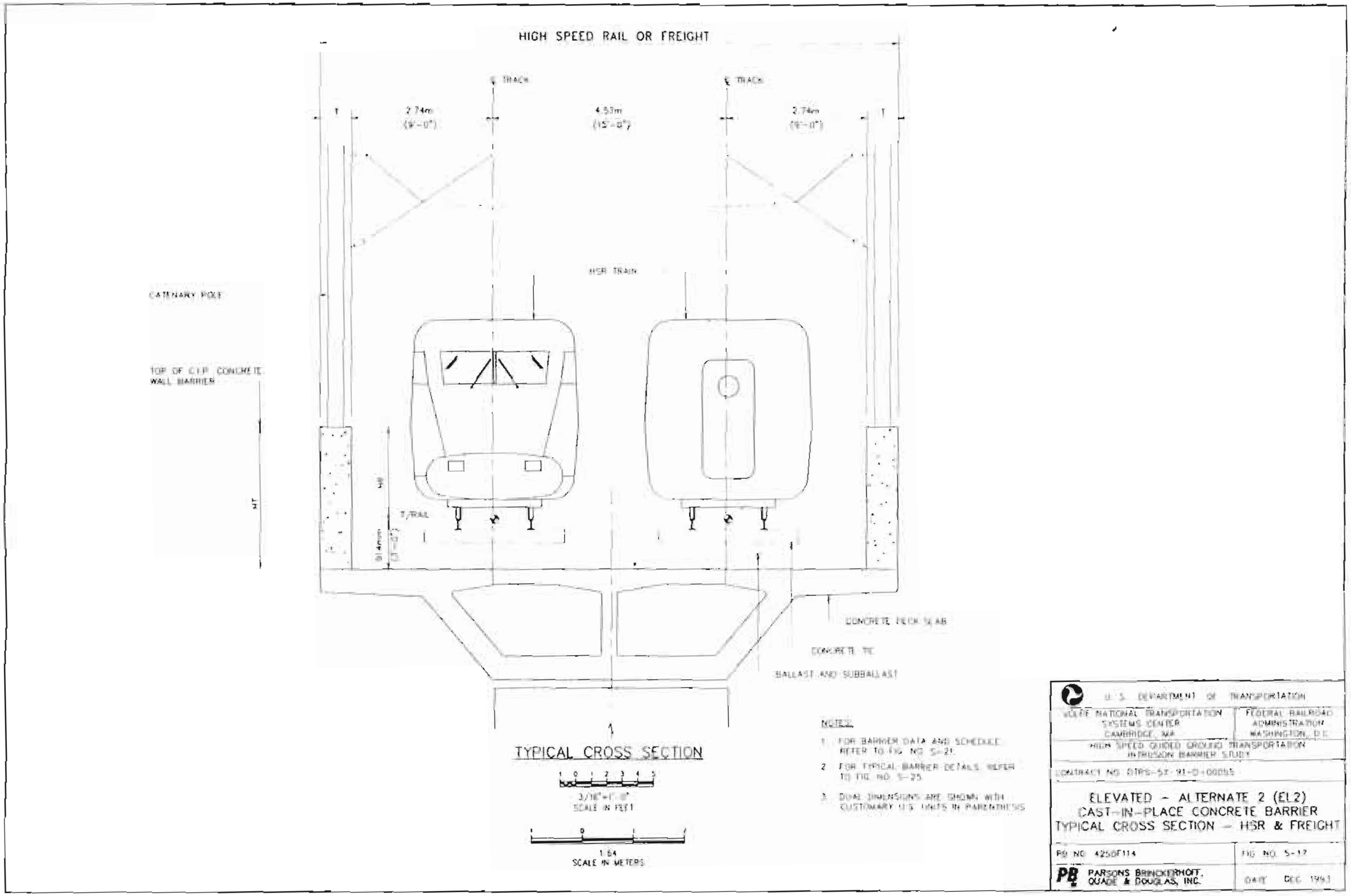


FIGURE 4-22. ELEVATED ALTERNATE 2: C.I.P. CONCRETE WALL - TRANSVERSE SECTION/RR

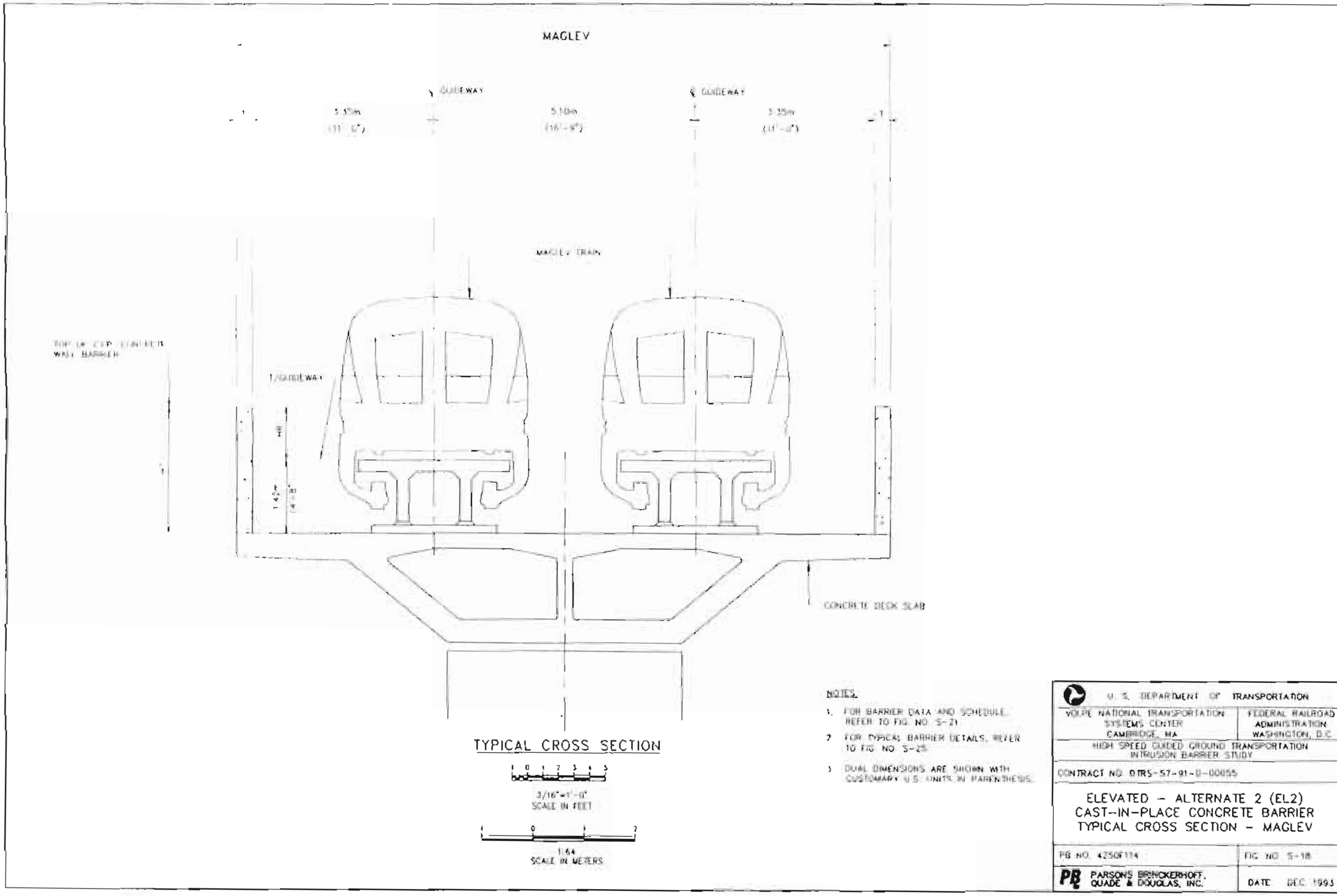


FIGURE 4-23. ELEVATED ALTERNATE 2: C.I.P. CONCRETE WALL - TRANSVERSE SECTION/MAGLEV

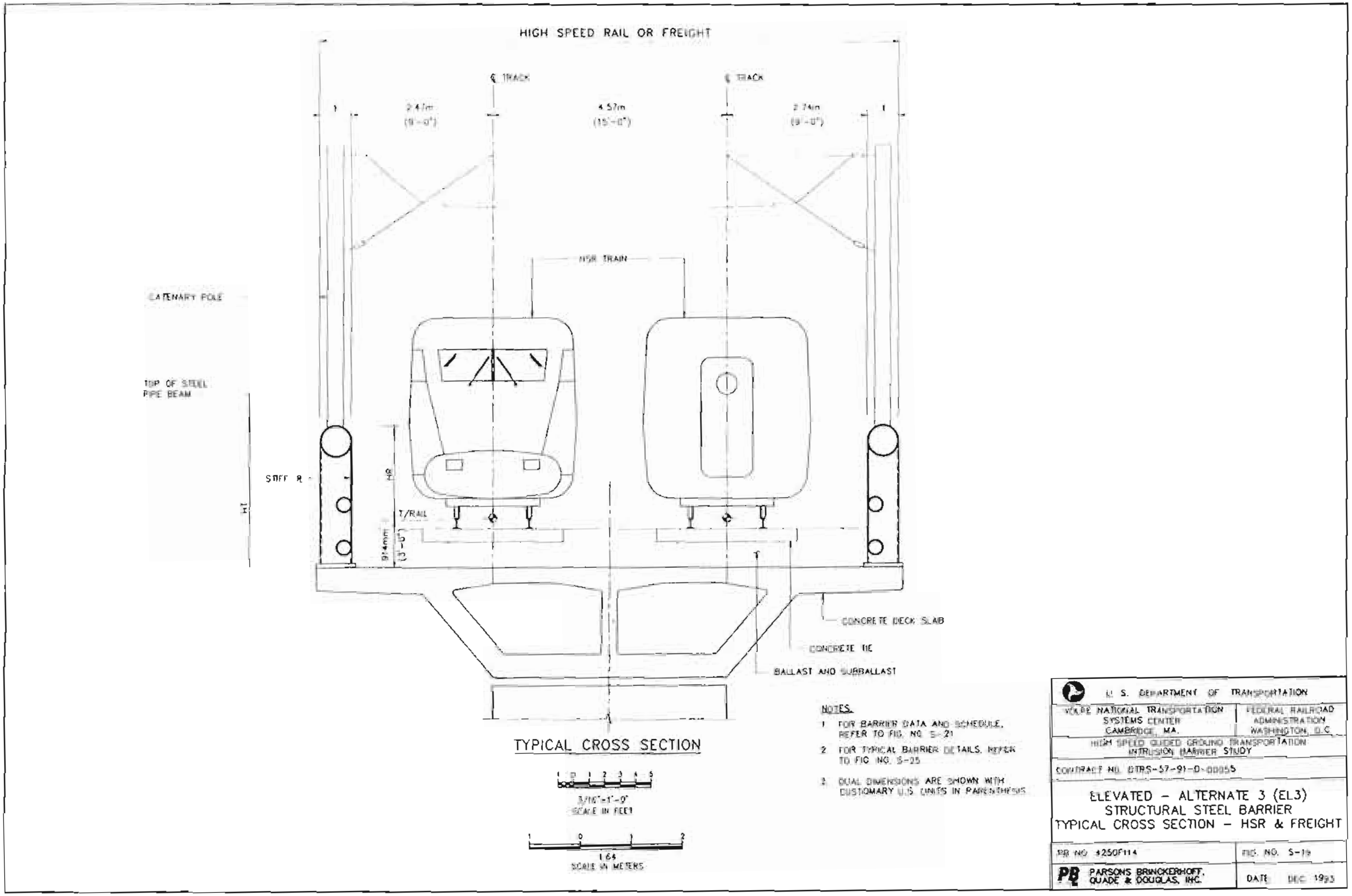


FIGURE 4-24. ELEVATED ALTERNATE 3: STRUCTURAL STEEL POST AND RAILING - TRANSVERSE SECTION/RR

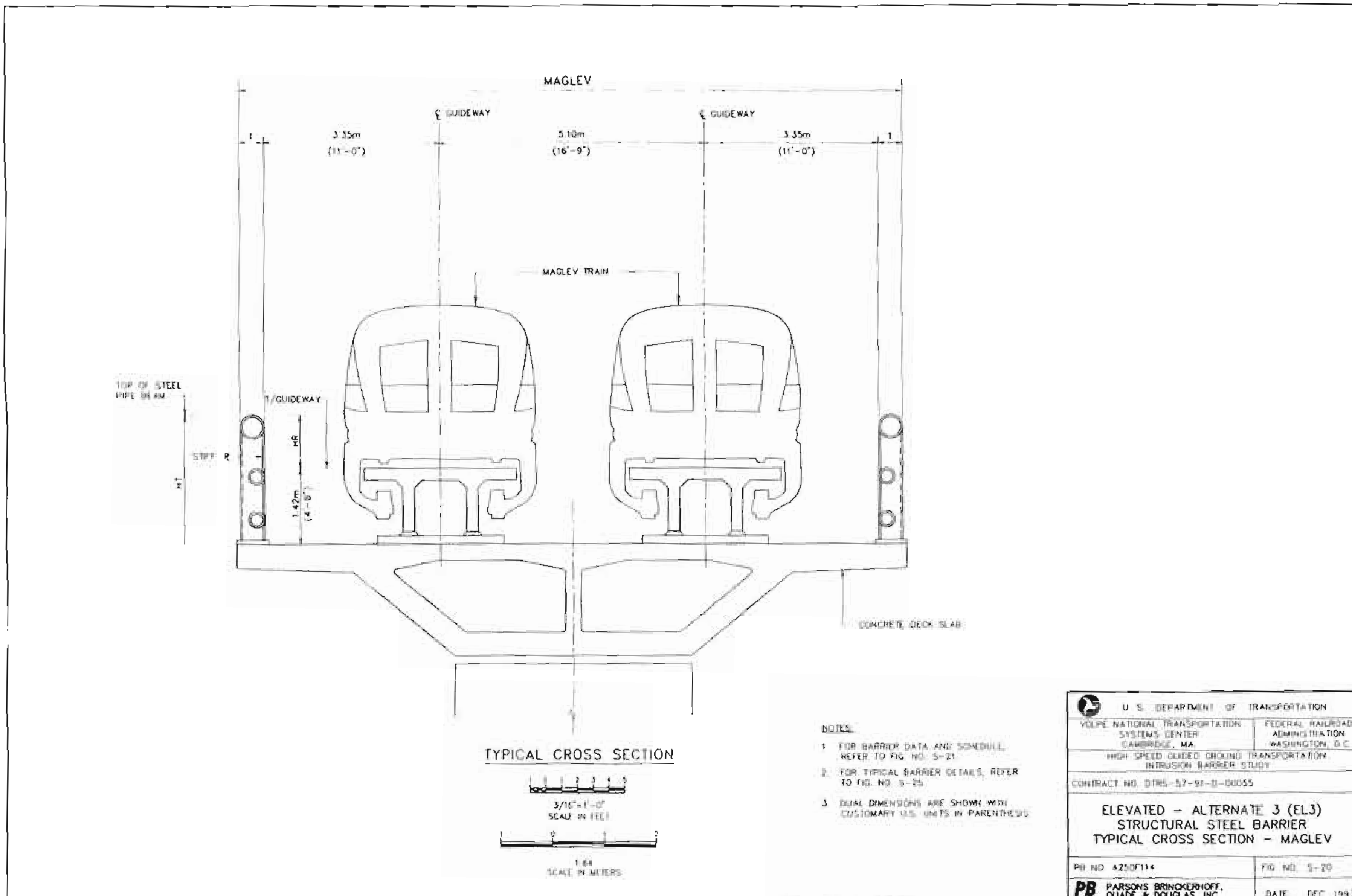


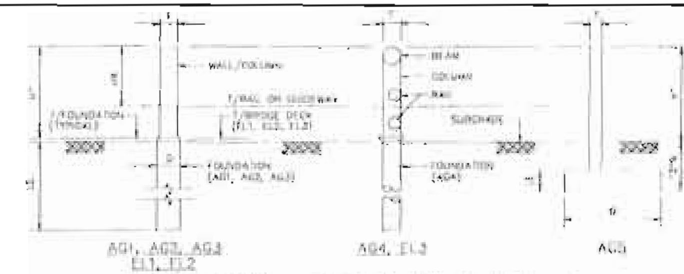
FIGURE 4-25. ELEVATED ALTERNATE 3: STRUCTURAL STEEL POST AND RAILING - TRANSVERSE SECTION/MAGLEV

TABLE 1 - INTRUSION SCENARIO AND BARRIER TYPE

SCENARIO NO.	VEHICLE TYPE	LOCATION	BARRIER TYPE	DESIGN ALTERNATES	BARRIER HEIGHT HR (m)	HR (FT)	HT (m)	HT (FT)	DESIGN LOAD (KN)	DESIGN LOAD (KIPS)
1	MAGLEV	AT GRADE	SINGLE-STRUCT.	AG1, AG2, AG3, AG4	0.86	2.83	2.29	7.5	890	200
2	HSR-ARTICULATED (TGV)	AT GRADE	SINGLE-STRUCT.	AG1, AG2, AG3, AG4	1.37	4.5	2.29	7.5	890	200
3	HSR-NONARTICULATED (ICE)	AT GRADE	SINGLE-STRUCT.	AG1, AG2, AG3, AG4	1.37	4.5	2.29	7.5	1334	300
4	FREIGHT-UNIFORM CAR	AT GRADE	SINGLE-STRUCT.	AG1, AG2, AG3, AG4	1.68	5.5	2.59	8.5	4893	1100
5	FREIGHT-MIXED CAR	AT GRADE	SINGLE-STRUCT.	AG1, AG2, AG3, AG4	1.37	4.5	2.29	7.5	1334	300
6	MAGLEV	AT GRADE	SINGLE-COMB.	AG5	1.02	3.33	2.44	8.0	890	200
7	HSR-ARTICULATED (TGV)	AT GRADE	SINGLE-COMB.	AG5	1.52	5.0	2.44	8.0	890	200
8	HSR-NONARTICULATED (ICE)	AT GRADE	SINGLE-COMB.	AG5	1.52	5.0	2.44	8.0	1334	300
9	FREIGHT-UNIFORM CAR	AT GRADE	SINGLE-COMB.	AG5	1.83	6.0	2.74	9.0	4893	1100
10	FREIGHT-MIXED CAR	AT GRADE	SINGLE-COMB.	AG5	1.52	5.0	2.44	8.0	1334	300
11	MAGLEV	AT GRADE	DUAL-STRUCT.	AG1, AG2, AG3, AG4	0.86	2.83	2.29	7.5	1334	300
12	HSR-ARTICULATED (TGV)	AT GRADE	DUAL-STRUCT.	AG1, AG2, AG3, AG4	1.37	4.5	2.29	7.5	2669	600
13	HSR-NONARTICULATED (ICE)	AT GRADE	DUAL-STRUCT.	AG1, AG2, AG3, AG4	1.68	5.5	2.59	8.5	4893	1100
14	MAGLEV	ELEVATED	DUAL-STRUCT.	EL1, EL2, EL3	1.02	3.33	2.44	8.0	1334	300
15	HSR-ARTICULATED (TGV)	ELEVATED	DUAL-STRUCT.	EL1, EL2, EL3	1.52	5.0	2.44	8.0	2669	600
16	HSR-NONARTICULATED (ICE)	ELEVATED	DUAL-STRUCT.	EL1, EL2, EL3	1.83	6.0	2.74	9.0	4893	1100
17	FREIGHT-UNIFORM CAR	ELEVATED	DUAL-STRUCT.	EL1, EL2, EL3	1.98	6.5	2.90	9.5	11298	2540
18	FREIGHT-MIXED CAR	ELEVATED	DUAL-STRUCT.	EL1, EL2, EL3	1.98	6.5	2.90	9.5	11298	2540
19**	HIGHWAY - VAN TRUCK	AT GRADE	STRUCTURAL	HAG1, HAG5	—	—	1.27	4.17	552	124
20**	HIGHWAY - TANK TRUCK	AT GRADE	STRUCTURAL	HAG2, HAG5	—	—	2.29	7.5	778	175
21**	HIGHWAY - VAN TRUCK	ELEVATED	STRUCTURAL	HEL1, HAG5	—	—	1.27	4.17	552	124
22**	HIGHWAY - TANK TRUCK	ELEVATED	STRUCTURAL	HEL2, HAG5	—	—	2.29	7.5	778	175

TABLE 2 - BARRIER ALTERNATES DESCRIPTION

AG1	AT GRADE - ALTERNATE 1	PRECAST CONCRETE BARRIER AND FOUNDATION
AG2	AT GRADE - ALTERNATE 2	PRECAST CONCRETE BARRIER AND STEEL FOUNDATION
AG3	AT GRADE - ALTERNATE 3	CAST-IN-PLACE CONCRETE BARRIER AND FOUNDATION
AG4	AT GRADE - ALTERNATE 4	STRUCTURAL STEEL BARRIER AND FOUNDATION
AG5	AT GRADE - ALTERNATE 5	CAST-IN-PLACE CONCRETE RETAINING WALL BARRIER
EL1	ELEVATED - ALTERNATE 1	PRECAST CONCRETE BARRIER
EL2	ELEVATED - ALTERNATE 2	CAST-IN-PLACE CONCRETE BARRIER
EL3	ELEVATED - ALTERNATE 3	STRUCTURAL STEEL BARRIER



KEY SECTIONS - SCENARIOS NO. 1 TO NO. 18

TABLE 3 - BARRIER DATA SCHEDULE

AT GRADE ALTERNATES - AG1, AG2, AG3, AG4, AG5

SCENARIO NO.	UNITS	AG1 WALL SIZE T	AG1 COLUMN SIZE B x T	AG1 FOUNDATION SIZE D	AG1 FOUNDATION SPA L	AG1 FOUNDATION EMBED. LE	AG2 WALL SIZE T	AG2 COLUMN SIZE (ENCAS.) B x T	AG2 COLUMN SIZE (STEEL)	AG2 FOUNDATION SIZE (STEEL)	AG2 FOUNDATION SPA L	AG2 FOUNDATION EMBED LE
1, 2, 6, 7	METRIC	457	457x457	559x559	4.57	4.57	457	457x457	W250x89	W250x89	4.57	6.10
	(U.S.)	(1'-6")	(18"x18")	(22"x22")	(15'-0")	(15'-0")	(1'-6")	(18"x18")	(W10x60)	(W10x60)	(15'-0")	(20'-0")
3, 5, 8, 10, 11*	METRIC	508	508x508	610x610	4.27	4.88	508	508x508	W250x149	W250x149	4.27	7.01
	(U.S.)	(1'-8")	(20"x20")	(24"x24")	(14'-0")	(16'-0")	(1'-8")	(20"x20")	(W10x100)	(W10x100)	(14'-0")	(23'-0")
12	METRIC	711	711x711	914x914 HC	4.57	4.27	711	711x711	W360x196	W360x196	4.27	5.18
	(U.S.)	(2'-4")	(28"x28")	(36"x36") HC	(15'-0")	(14'-0")	(2'-4")	(28"x28")	(W14x132)	(W14x132)	(14'-0")	(17'-0")
4, 9, 13*	METRIC	762	762x762	914x914 HC	2.74	7.32	711	711x711	W360x634	W360x634	4.27	13.41
	(U.S.)	(2'-6")	(30"x30")	(36"x36") HC	(9'-0")	(24'-0")	(2'-4")	(28"x28")	(W14x426)	(W14x426)	(14'-0")	(44'-0")

SCENARIO NO.	UNITS	AG3 WALL SIZE T	AG3 COLUMN SIZE B x T	AG3 FOUNDATION SIZE D	AG3 FOUNDATION SPA L	AG3 FOUNDATION EMBED. LE
1, 2, 6, 7	METRIC	508	610x508	762	4.88	4.27
	(U.S.)	(1'-8")	(24"x20")	(30")	(16'-0")	(14'-0")
3, 5, 8, 10, 11*	METRIC	559	610x508	762	3.66	4.27
	(U.S.)	(1'-10"	(24"x22")	(30")	(12'-0")	(14'-0")
12	METRIC	711	762x711	914	4.57	3.96
	(U.S.)	(2'-4")	(30"x28")	(36")	(15'-0")	(13'-0")
4, 9, 13*	METRIC	1016	914x1016	1219	3.66	6.71
	(U.S.)	(3'-4")	(36"x40")	(48")	(12'-0")	(22'-0")

SCENARIO NO.	UNITS	AG4 WALL SIZE T	AG4 BEAM SIZE (STEEL)	AG4 RAIL SIZE (STEEL)	AG4 COLUMN SIZE (STEEL)	AG4 FOUNDATION SIZE (STEEL)	AG4 FOUNDATION SPA L	AG4 FOUNDATION EMBED. LE
1, 2, 6, 7	METRIC	406	406x17	324x10	406x17	406x17	4.57	6.10
	(U.S.)	(1'-4")	(16x0.656)	(12x0.406)	(16x0.656)	(16x0.656)	(15'-0")	(20'-0")
3, 5, 8, 10, 11*	METRIC	457	457x24	356x11	457x24	457x24	5.79	6.40
	(U.S.)	(1'-6")	(18x0.937)	(14x0.437)	(18x0.937)	(18x0.937)	(19'-0")	(21'-0")
12	METRIC	559	559x29	406x13	559x29	559x29	4.57	5.18
	(U.S.)	(1'-10")	(22x1.125)	(16x0.500)	(22x1.125)	(22x1.125)	(15'-0")	(17'-0")
4, 9, 13*	METRIC	610	610x31	457x14	610x31	610x31	3.05	8.84
	(U.S.)	(2'-0")	(24x1.218)	(18x0.562)	(24x1.218)	(24x1.218)	(10'-0")	(29'-0")

SCENARIO NO.	UNITS	AG5 WALL SIZE T	AG5 FOUNDATION WIDTH D	AG5 FOUNDATION DEPTH LE	AG5 FOUNDATION HEIGHT HT
1, 2, 6, 7	METRIC	305	2.44	457	2.44
	(U.S.)	(1'-0")	(8'-0")	(1'-6")	(8'-0")
3, 5, 8, 10, 11*	METRIC	305	2.44	457	2.44
	(U.S.)	(1'-0")	(8'-0")	(1'-6")	(8'-0")
12	METRIC				
	(U.S.)				
4, 9, 13*	METRIC	457	2.74	610	2.74
	(U.S.)	(1'-6")	(9'-0")	(2'-0")	(9'-0")

ELEVATED ALTERNATES - EL1, EL2, EL3

SCENARIO NO.	UNITS	EL1 WALL SIZE T	EL2 WALL SIZE T	EL3 WALL SIZE T	EL3 BEAM SIZE (STEEL)	EL3 RAIL SIZE (STEEL)	EL3 COLUMN SIZE (STEEL)	EL3 COLUMN SPA L
14	METRIC	305	356	457	457x24	356x11	457x24	4.57
	(U.S.)	(1'-0")	(1'-2")	(1'-6")	(18x0.937)	(14x0.437)	(18x0.937)	(15'-0")
15	METRIC	406	457	559	559x29	406x13	559x29	4.57
	(U.S.)	(1'-4")	(1'-6")	(1'-10")	(22x1.125)	(16x0.500)	(22x1.125)	(15'-0")
16	METRIC	610	610	610	610x31	457x14	610x31	2.44
	(U.S.)	(2'-0")	(2'-0")	(2'-0")	(24x1.218)	(18x0.562)	(24x1.218)	(8'-0")
17,18	METRIC	1016	1016	914	914x29	508x15	914x29	2.74
	(U.S.)	(3'-4")	(3'-4")	(3'-0")	(36x1.125)	(20x0.593)	(36x1.125)	(9'-0")

NOTES:

1. DUAL DIMENSIONS ARE SHOWN WITH CUSTOMARY U. S. UNITS IN PARENTHESIS. METRIC DIMENSIONS (SI UNITS) WITH THREE DIGITS OR LESS ARE SHOWN IN MILLIMETERS (E.G. 300), LARGER DIMENSIONS ARE SHOWN IN METERS (E.G. 3.15). TYPICAL UNLESS OTHERWISE NOTED
2. UNITS OF STEEL SIZES, E.G. W250x89 (AG2) ARE IN mmxKg/m (W10x60 IN InchxPounds/Foot), 406x17 (AG4,EL3) ARE PIPE DIAMETER x WALL THICKNESS IN mmxmm (16X0.656 IN Inchxinch).
* 3. ALTERNATE AG5 DOES NOT APPLY FOR SCENARIOS NO. 11 AND NO. 13.
4. THE DESIGN LOADS SHOWN IN TABLE 1 REPRESENT THE MAGNITUDE OF THE HORIZONTAL DERAILMENT LOADS FOR WHICH THE BARRIER STRUCTURES HAVE BEEN DESIGNED TO RESIST
5. FOR MATERIAL PROPERTIES, REFER TO THE NOTES ON FIG. NO. S-24.
** 6. FOR HIGHWAY BARRIER DIMENSIONS AND DETAILS, REFER TO FIG. NO. S-26.

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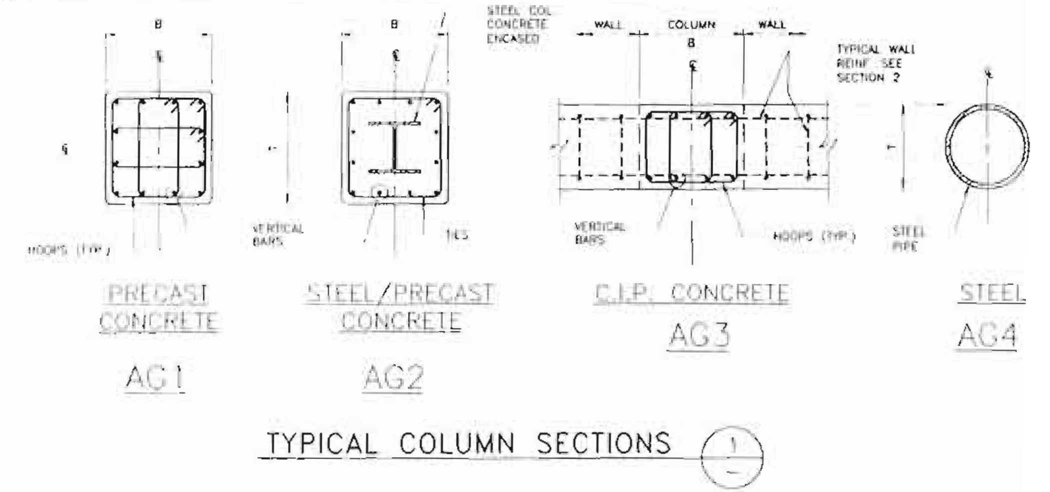
HIGH SPEED GUIDED GROUND TRANSPORTATION INTRUSION BARRIER STUDY
CONTRACT NO. DTRS-57-91-D-00056

INTRUSION SCENARIO TABLE AND BARRIER DATA SCHEDULE

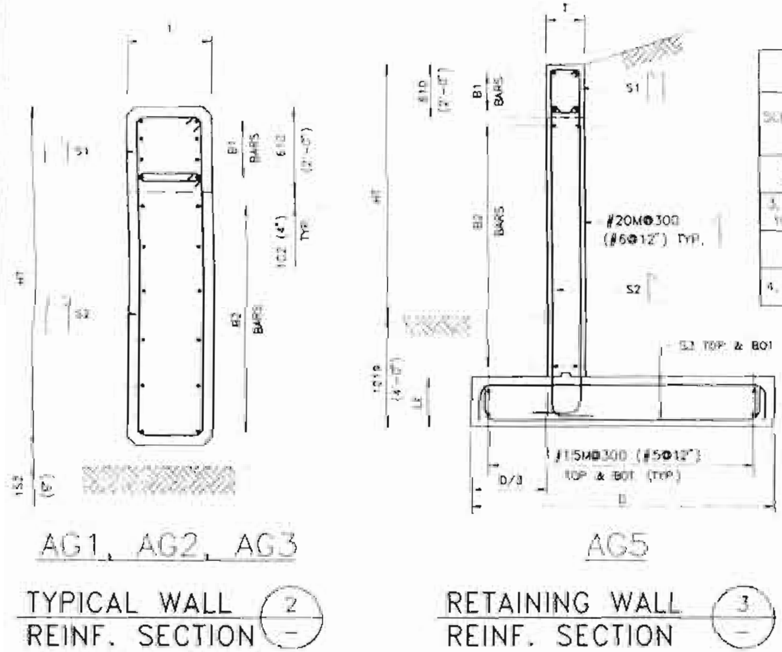
PB NO. 4250F114 | FIG. NO. S-25
PARSONS BRINCKERHOFF, QUADE & DOUGLAS, INC. | DATE DEC 1993

FIGURE 4-26. BARRIER DATA AND REINFORCING DATA



COLUMN REINFORCING SCHEDULE						
SCENARIO NO.	AG1		AG2		AG3	
	VERTICAL BARS	HOOPS	VERTICAL BARS	TIES	VERTICAL BARS	HOOPS
1, 2, 6, 7	10-#30M (8-#10)	#10M@80 (#4@4")	8-#20M (8-#6)	#10M@120 (#4@6")	10-#30M (10-#9)	#10M@80 (#4@4")
3, 5, 8, 10, 11	10-#35M (12-#10)	#10M@80 (#4@4")	10-#20M (8-#7)	#10M@120 (#4@6")	10-#30M (8-#10)	#10M@80 (#4@4")
12	20-#35M (20-#11)	#10M@80 (#4@4")	8-#30M (8-#9)	#10M@120 (#4@6")	14-#35M (14-#11)	#10M@120 (#4@6")
4, 9, 13	20-#35M (20-#11)	#10M@100 (#4@5")	8-#30M (8-#9)	#10M@120 (#4@6")	16-#35M (16-#11)	#10M@120 (#4@6")

RETAINING WALL REINF. SCHEDULE					
SCENARIO NO.	AG5				
	B1 BARS	B2 BARS	HOOPS S1	S2	S3
6, 7, 8, 10	8-#10M (6-#4)	22-#10M (18-#4)	#10M@120 (#4@6")	#15M@300 (#5@12")	#15M@300 (#5@12")
9	8-#20M (5-#7)	24-#20M (18-#7)	#10M@120 (#4@6")	#20M@300 (#6@12")	#20M@300 (#6@12")



TYPICAL WALL REINFORCING SCHEDULE												
SCENARIO NO.	AG1				AG2				AG3			
	B1 BARS	B2 BARS	HOOPS S1	HOOPS S2	B1 BARS	B2 BARS	HOOPS S1	HOOPS S2	B1 BARS	B2 BARS	HOOPS S1	HOOPS S2
1, 2, 6, 7	10-#30M (10-#9)	18-#30M (18-#9)	#10M@80 (#4@4")	#10M@80 (#4@4")	10-#30M (10-#9)	18-#30M (18-#9)	#10M@80 (#4@4")	#10M@80 (#4@4")	6-#30M (6-#9)	18-#30M (18-#9)	#10M@80 (#4@4")	#10M@80 (#4@4")
3, 5, 8, 10, 11	8-#30M (6-#10)	20-#30M (20-#9)	#10M@80 (#4@4")	#10M@80 (#4@4")	8-#30M (6-#10)	20-#30M (20-#9)	#10M@80 (#4@4")	#10M@80 (#4@4")	8-#30M (8-#9)	16-#35M (20-#10)	#10M@80 (#4@4")	#10M@80 (#4@4")
12	8-#35M (10-#10)	20-#35M (24-#10)	#10M@120 (#4@6")	#10M@120 (#4@6")	8-#30M (10-#10)	20-#35M (24-#10)	#10M@120 (#4@6")	#10M@120 (#4@6")	8-#35M (10-#10)	20-#35M (20-#11)	#10M@100 (#4@5")	#10M@100 (#4@5")
4, 9, 13	10-#35M (10-#11)	24-#35M (24-#11)	#10M@120 (#4@6")	#10M@120 (#4@6")	8-#35M (10-#10)	16-#35M (20-#10)	#10M@120 (#4@6")	#10M@120 (#4@6")	10-#35M (10-#11)	20-#35M (20-#11)	#10M@120 (#4@6")	#10M@120 (#4@6")

NOTES:

1. DUAL DIMENSIONS ARE SHOWN WITH CUSTOMARY U.S. UNITS IN PARENTHESIS. METRIC DIMENSIONS (SI UNITS) ARE SHOWN IN MILLIMETERS.
2. FOR CONCRETE DIMENSIONS AND STEEL SIZES, REFER TO FIG. NO. 5-21
3. FOR MATERIAL PROPERTIES, REFER TO THE NOTES ON FIG. NO. 5-24

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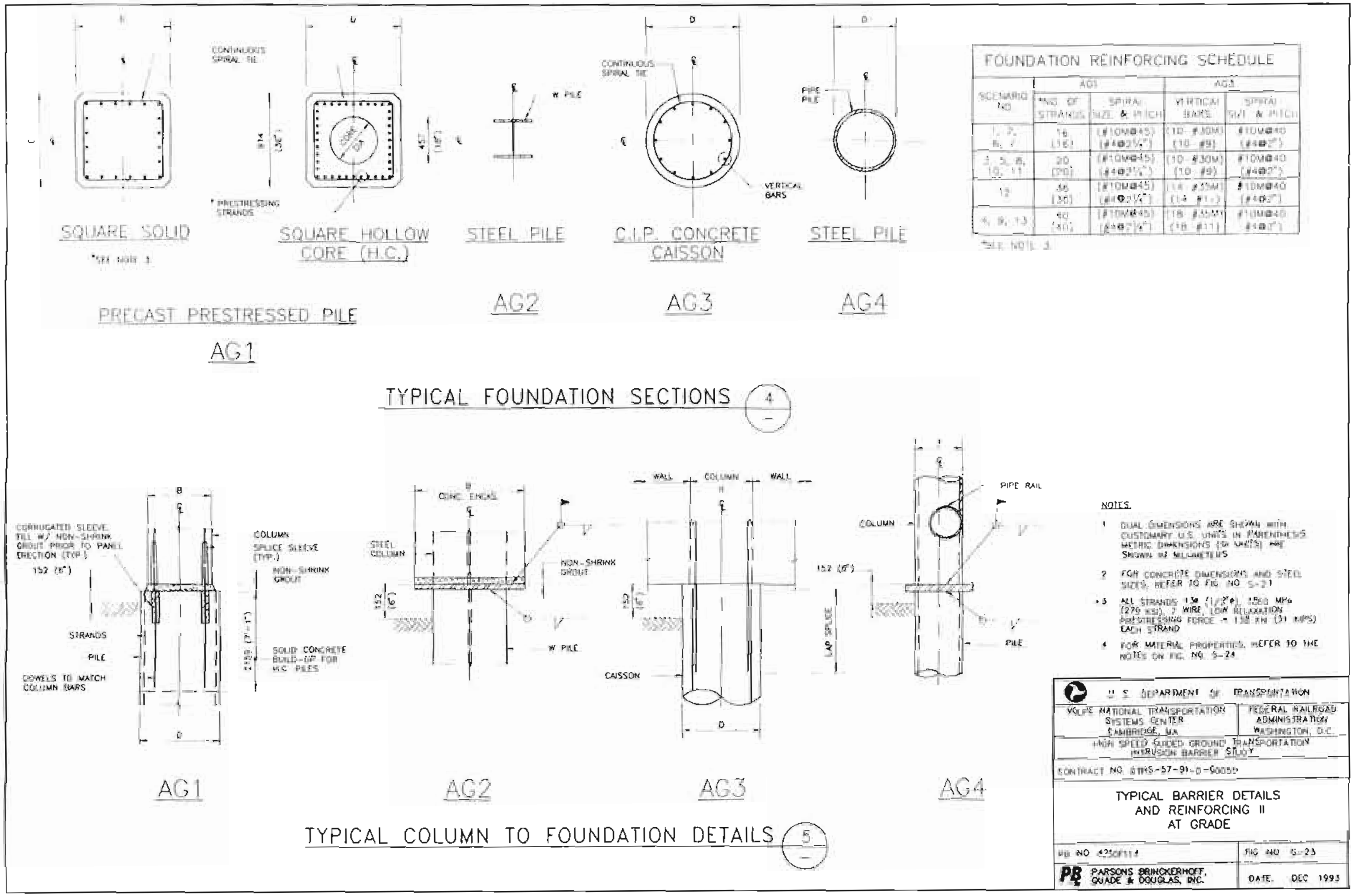
TYPICAL BARRIER DETAILS AND REINFORCING I AT GRADE

PB NO. 1250F114 | FIG. NO. 5-22

PARSONS BRINCKERHOFF, QUADE & DOUGLAS, INC. | DATE DEC 1993

FIGURE 4-27. TYPICAL BARRIER DETAILS AND REINFORCING AT GRADE (I)



FOUNDATION REINFORCING SCHEDULE

SCENARIO NO.	AG1		AG3	
	*NO. OF STRANDS	SPIRAL SIZE & PITCH	VERTICAL BARS	SPIRAL SIZE & PITCH
1, 2, 6, 7	16 (16)	(#10M@45) (#4@2¼")	(10-#30M) (10-#9)	#10M@40 (#4@2")
3, 5, 8, 10, 11	20 (20)	(#10M@45) (#4@2¼")	(10-#30M) (10-#9)	#10M@40 (#4@2")
12	36 (36)	(#10M@45) (#4@2¼")	(14-#35M) (14-#11)	#10M@40 (#4@2")
4, 9, 13	40 (40)	(#10M@45) (#4@2¼")	(18-#35M) (18-#11)	#10M@40 (#4@2")

*SEE NOTE 3

FIGURE 4-28. TYPICAL BARRIER DETAILS AND REINFORCING AT GRADE (II)

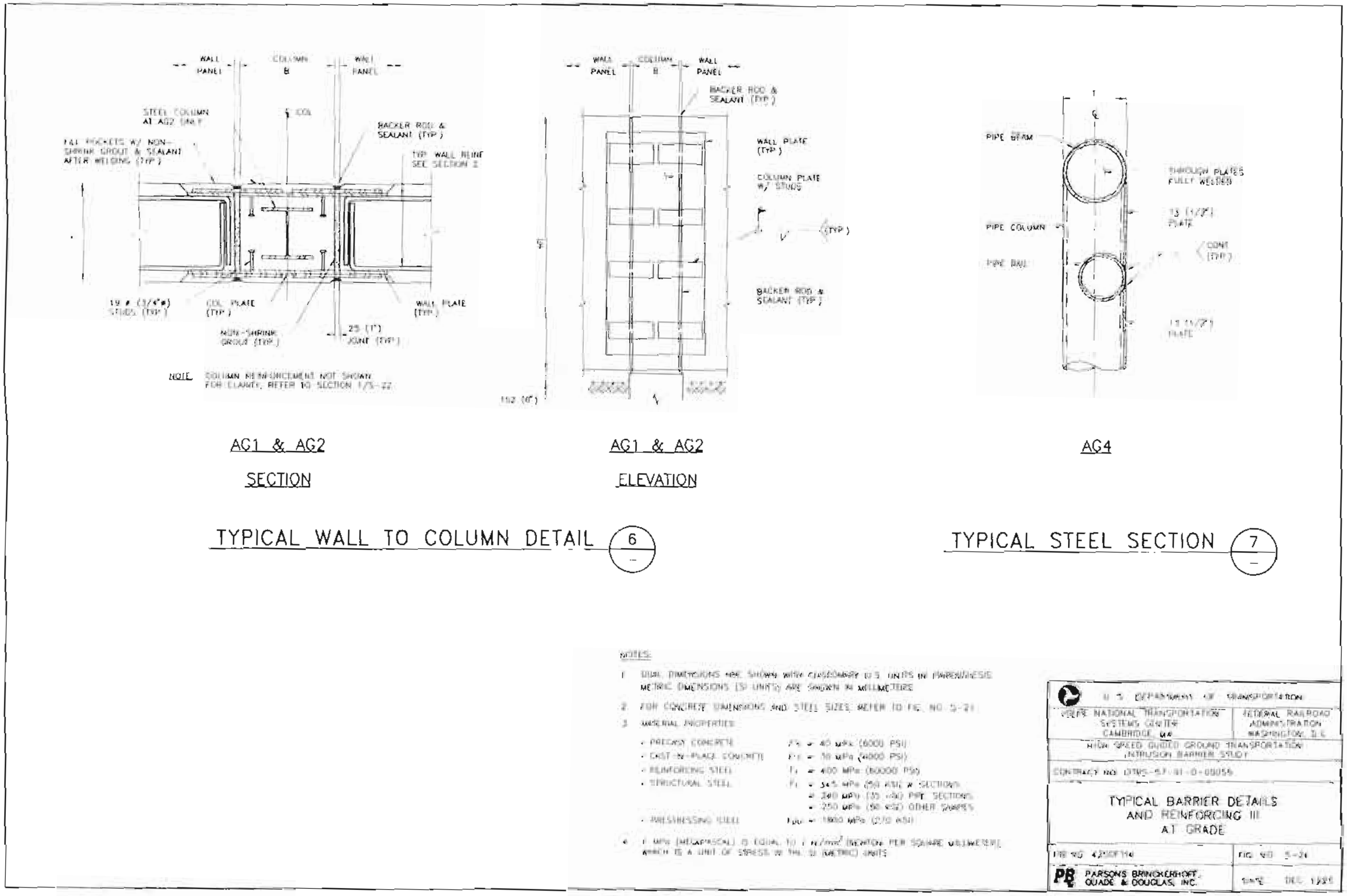
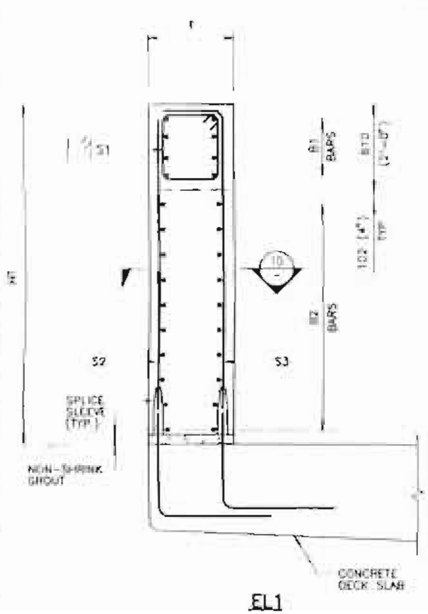
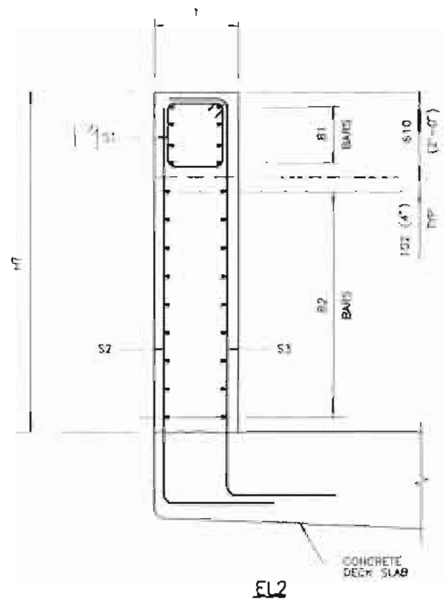
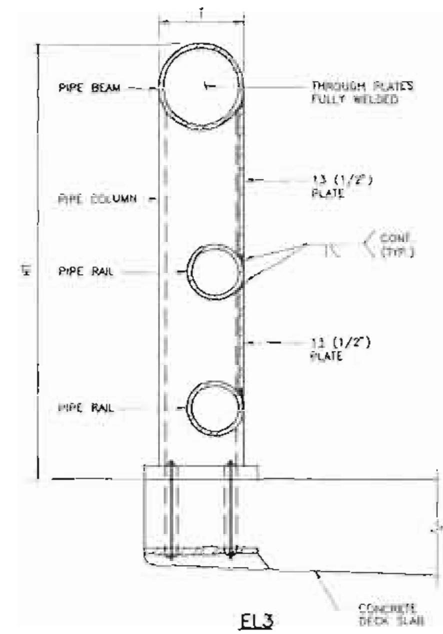


FIGURE 4-29. TYPICAL BARRIER DETAILS AND REINFORCING AT GRADE (III)





TYPICAL WALL REINF. SECTION (8)



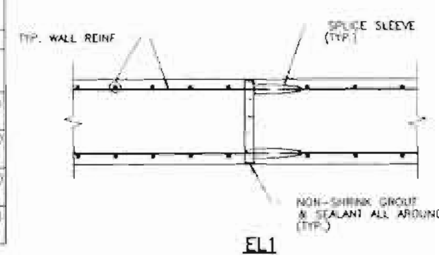
TYPICAL STEEL SECTION (9)

NOTES:

1. DUAL DIMENSIONS ARE SHOWN WITH CUSTOMARY U.S. UNITS IN PARENTHESIS. METRIC DIMENSIONS (SI UNITS) ARE SHOWN IN MILLIMETERS
2. FOR CONCRETE DIMENSIONS AND STEEL SIZES, REFER TO FIG. NO. S-21
3. FOR MATERIAL PROPERTIES, REFER TO THE NOTES ON FIG. NO. S-24

WALL REINFORCING SCHEDULE

SECTION NO	EL1 B1 BARS	EL1 B2 BARS	EL1 HOOPS S1	EL1 S2	EL1 S3	EL2 B1 BARS	EL2 B2 BARS	EL2 S1	EL2 S2	EL2 S3
14	10-#20M (8-#7)	20-#25M (20-#8)	#10M@80 (#4@4")	#20M@230 (#7@12")	#20M@115 (#7@6")	10-#20M (8-#7)	18-#25M (18-#8)	#10M@80 (#4@4")	#20M@230 (#7@12")	#20M@115 (#7@6")
15	8-#25M (8-#8)	20-#30M (20-#9)	#10M@120 (#4@6")	#30M@300 (#9@12")	#30M@150 (#9@6")	8-#25M (8-#8)	26-#25M (26-#8)	#10M@120 (#4@6")	#25M@300 (#8@12")	#25M@150 (#8@6")
16	10-#30M (8-#10)	16-#35M (20-#10)	#10M@120 (#4@6")	#30M@300 (#9@12")	#30M@175 (#10@8")	10-#30M (10-#9)	24-#30M (24-#9)	#10M@120 (#4@6")	#30M@300 (#9@12")	#30M@150 (#9@6")
17, 18	8-#25M (8-#8)	20-#35M (24-#10)	#10M@120 (#4@6")	#35M@300 (#10@10")	#35M@150 (#10@5")	8-#25M (8-#8)	24-#35M (28-#10)	#10M@120 (#4@5")	#35M@300 (#10@10")	#35M@150 (#10@5")



TYPICAL PRECAST WALL TO WALL DETAIL (10)

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CONTRACT NO. DTRS-57-91-D-00055

TYPICAL BARRIER DETAILS AND REINFORCING I ELEVATED

PB NO. 4250F114 | FIG. NO. S-25

PARSONS BRINCKERHOFF, QUADE & DOUGLAS, INC. | DATE DEC. 1993

FIGURE 4-30. TYPICAL ELEVATED BARRIER DETAILS

TABLE 4-1. SUMMARY OF DEFLECTION CALCULATION RESULTS

			Deflections		
Type	Alternate	Impact Load	Post	Beam	Total
Concrete	AG3	1335 kN (300 kips)	116 mm (4.6 in.)	358 mm (14.1 in.)	475 mm (18.7 in.)
Steel	AG4	1335 kN (300 kips)	112 mm (4.4 in.)	168 mm (6.6 in.)	279 mm (11.0 in.)

The deflections, although extreme for structures designed by elastic theory, are not considered excessive for these structures designed using limit state theories. The deformations are not large enough to compromise the barrier's ability to prevent intrusion.

4.1.2.10 Barrier Offset Distances

As discussed in Section 3.1.3.1, the distance from the centerline of the guideway to the face of the barrier has a great affect on the impact forces exerted on a barrier. Forces are low when the barrier is close to the guideway, they increase as the barrier location is moved away from the guideway, reaching a maximum at some distance, and then decreasing eventually to zero at large barrier offset distances in the vicinity of approximately 15 meters (50 feet).

The issue gets more complicated when the barrier doubles as a protection and containment barrier, such as when located between railroad (RR) and high speed rail (HSR) guideways. The forces imposed by the RR are higher than those imposed by the HSR. It is advantageous, therefore, to locate the barrier close to the RR, allowing the HSR forces to increase with higher offsets from the HSR, until they surpass the RR forces. Beyond this offset distance, the barrier design in question would not work, because its design load would be exceeded.

For those HSGGT applications adjacent to conventional railroads analysis, results indicate that it is advantageous to locate the barrier as close to the railroad as possible. The designs shown are valid for the case where the barrier is located 2.74 m (9 feet) from the railroad track centerline, and any distance from the high speed guideway. There is one exception, however, for ICE consists where dual containment barriers are required. Analysis indicates that impact loads from the ICE trainset for this scenario are very high. In these situations, the design is valid only for an offset from the railroad of 2.74 m (9 feet) and offsets from the ICE centerline of 2.74 m (9 feet) for the near barrier, and 7.32 m (24 feet) for the far barrier. For other offsets, the impact force from the ICE trainset exceeds that from the railroad, and a site-specific barrier design would have to be developed.

4.2 HIGHWAY BARRIER DESIGN

4.2.1 Methodology

As previously discussed, it is proposed that intrusion barriers be designed in accordance with provisions currently under development [13] that will be incorporated into the AASHTO Standard Specifications for Highway Bridges [1]. The methodology, as described in a draft of these provisions, is summarized below. It must be stressed that this procedure has not been officially adopted by AASHTO. The actual design procedure should follow recommendations given in the final document when it is issued.

Establish Warrants: Determine the need for intrusion barriers considering the conditions at the site including adjacent hazards; volume and nature of vehicular, HSGGT, and pedestrian traffic; geometry of the site and location of relevant features. Additional guidance is provided in AASHTO's Roadside Design Guide [12] and Guide Specification for Bridge Railings [11].

Select Performance Level: In consideration of the established warrant, select the performance level, PL-1 through PL-5, as described in Section 3.2.1.3 and Table 3-5. It is recommended that PL-4 or PL-5 be used as a minimum for highways adjacent to HSGGT guideways. PL-5 should be used where Tank truck traffic is common. Where this traffic is infrequent, such as where there are traffic restrictions, PL-4 can be used.

Select Crashworthy Designs: Use designs already proven through crash testing to be capable of deflecting the vehicles identified by the selected performance levels. The highway barrier designs described herein have been crash tested for the elevated (bridge deck) application, but not for at-grade. If these designs are selected for at-grade use, they will have to be tested.

or:

Develop New Design: New designs can be developed for the selected performance level using the loads given in Table 3-6 and the methodology given in the AASHTO specifications (or the current draft [13]). This is similar to that described for structural barriers in section 4.1 of this report.

and:

Crash Test: In order to comply with AASHTO specifications, new designs must be crash tested using the testing criteria set forth in Table 3-5 to confirm that they meet the structural and geometric requirements of the specified performance level.

Detail End Treatments: An untreated end of a roadside barrier is extremely hazardous to the highway vehicle if hit, since the beam element can penetrate the passenger compartment and will generally stop the vehicle abruptly. A crashworthy end treatment is therefore considered essential if the barrier terminates within the clear zone and/or is in an area where it is likely to be hit head-on by an errant motorist. To be crashworthy, the end treatment should not spear, vault, or roll a vehicle for a head-on or angled impacts.

4.2.2 Findings

The designs presented in this report were developed for elevated bridge deck application in previous studies [20,21]. These studies included crash tests of the designs. As previously stated, therefore, the designs are considered crashworthy as elevated barriers. The designs have been modified in this study with the incorporation of foundation elements for use at-grade. In order to comply with AASHTO requirements, these modified designs must be tested for crashworthiness as at-grade barriers.

Figure 4-31 shows a 1.27 m (50 in) high concrete safety shape with a metal rail on top which successfully redirected a 36,300 kg (80,000 lb) van truck traveling 80 km/h (50 mph) and impacting at a 15 degree angle [20]. Figure 4-31 also shows a 2.29 m (90 in) high concrete barrier which successfully redirected an 36,300 kg (80,000 lb) fluid tank truck at 80 km/h (50 mph) and a 15 degree angle [21]. This barrier design has been constructed on I-10 in San Antonio, Texas. A barrier very similar to it has been installed on I-68 near Cumberland, Maryland. This barrier has been impacted several times by trucks [22], and has effectively redirected them away from the adjacent hazard.

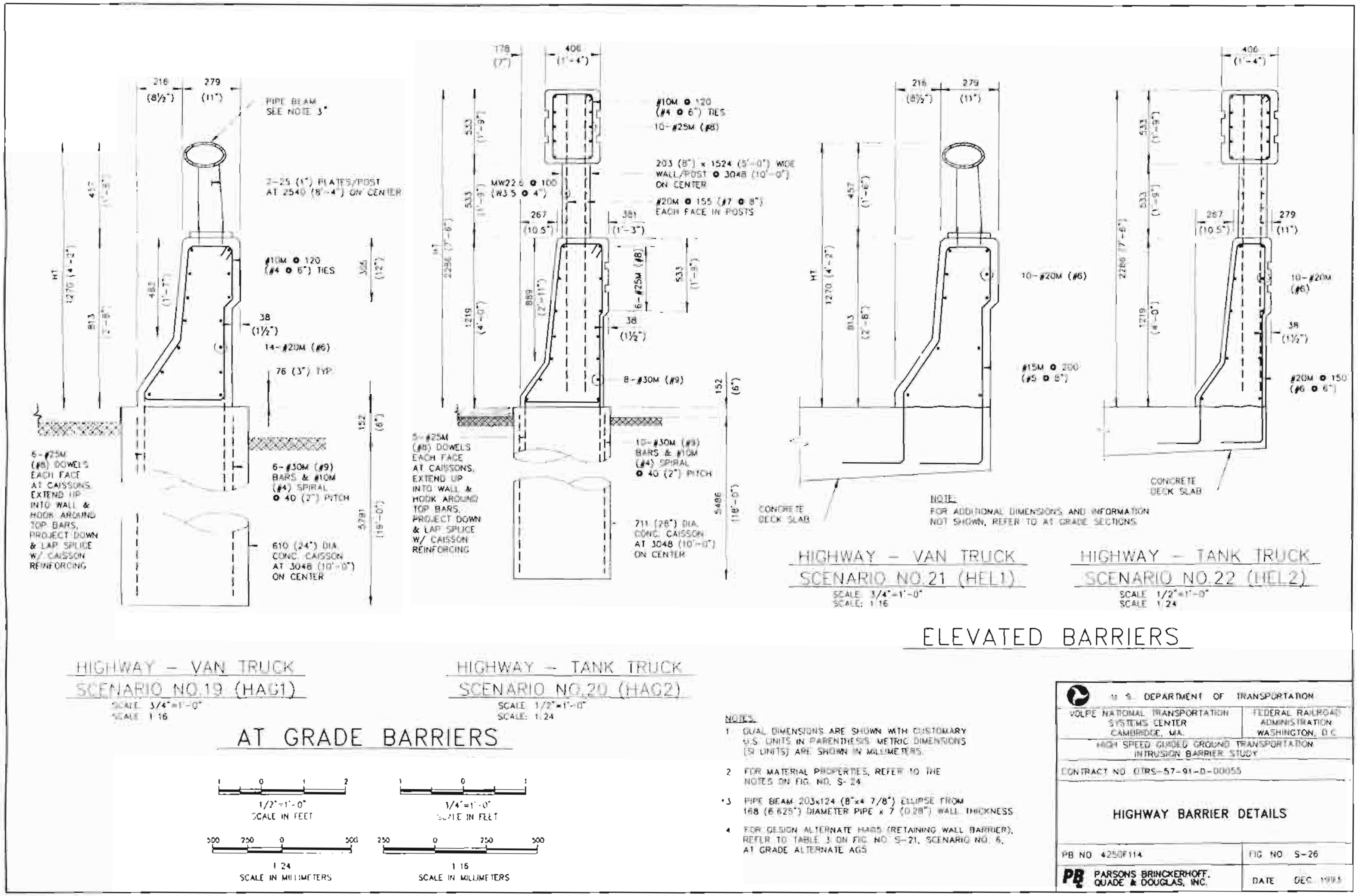

FIGURE 4-31. TYPE 1 AND 2 HIGHWAY BARRIER DETAILS

5. INTRUSION BARRIER COSTS

5.1 CONSTRUCTION COSTS

5.1.1 Methodology

Order-of-magnitude cost estimates for the HSGGT intrusion barriers and scenarios developed under this study have been prepared using the designs contained in Figures 4-6 through 4-31, which serve as the basic source documents. Unit prices have been developed from standard references, including Means Building Construction Cost Data [23], Engineering News Record [24], and other similar indices; discussions with material suppliers and vendors; recent bids for similar elements of work; and from current engineering cost estimates for projects with similar items of work. All estimated costs are stated in mid-1993 dollars.

Linear unit costs of intrusion barrier designs and alternatives have been estimated for each of the crash scenarios developed for this study. Estimates for each barrier design have been broken down and summarized into four separate elements of cost: material, labor, equipment, and miscellaneous. These elements of cost are provided in Appendix E of this report. Material costs are based on quantities computed from the applicable barrier design figure(s). Labor and corresponding equipment costs are based on production rates developed from the aforementioned standard references and indices. The miscellaneous cost elements include allowances for expendable construction materials, agency or abutting transportation system flagging protection costs as appropriate, railroad protective liability insurance cost as applicable, and contractor's mobilization and demobilization costs. All elements of cost are intended to include contractor's overhead and profit.

A contingency factor has been included in all estimates. The contingency selected for this study is 20 percent and is considered standard in the industry for this level of analysis. No allowance has been provided for the costs of final engineering design of specific application's engineering design support during construction management services, nor agency/owner administration costs.

5.1.2 Assumptions

The following assumptions have been used in the preparation of construction cost estimates:

- Cost estimates have been prepared for 1993 costs. Inflation factors should be incorporated for the anticipated construction dates of particular installations.

- National average costs have been used. Geographical cost variations should be considered in evaluating the use of barriers in particular areas of application.

- Inasmuch as the extent and location of intrusion barriers are unknown, geotechnical data has been assumed for average conditions of cohesionless soils (sands and gravels) for foundation requirements. Estimates have been prepared using these soil conditions. Preliminary assessments indicate that linear unit costs of intrusion barriers supported on foundations constructed in cohesive soils (clays) will be approximately five percent less than those presented here. No assessments have been made for foundations in rock or in poor quality soils.

- Estimates are based on construction of 1.6 km (1.0 mile) of continuous barriers.

- For estimating purposes, HSGGT systems are considered to be new construction, and activities to construct intrusion barriers are assumed to have minimal impact or interference on operations of an adjacent transportation system. Based on these assumptions, average access to the construction site(s) and normal 8-hour daytime work shifts have been used for estimating production rates.

- Where intrusion barriers are to be installed between two adjacent operating transportation systems, i.e., freight and commuter rail systems, costs will be increased resulting from:

    - limited access
    - construction adjacent to existing operating systems
    - limitation of construction windows, i.e., night time and weekend work
    - premium wages
    - working in territories with catenary and other overhead structures

The premium costs for construction of intrusion barriers between two existing adjacent operating transportation systems has been estimated at 25 percent. When HSGGT barrier systems designed for operating corridors and shared with other transportation systems are completed or further defined, comparable order-of-magnitude cost estimates for installation of intrusion barrier can be made based on the aforementioned assumptions. At such time, cost estimates can be developed to approximate the differential costs of intrusion barriers for these variable conditions.

5.1.3 Estimated Intrusion Barrier Construction Costs

Estimated construction costs of intrusion barrier designs and alternatives for the crash scenarios developed under this study can be classified into three general categories: (1) at-grade barriers, (2) elevated barriers for elevated structures, and (3) highway barriers. In the first two categories, design of intrusion barriers and alternates are based on maximum impact loads (as determined from the TBIP) resulting from derailments of designated HSGGT equipment consist scenarios enumerated in Table 5-1. Highway barrier designs are based on crash tested designs accepted by AASHTO, although two have been modified for at-grade applications.

In the cost summary table that follows, at-grade barriers are designated with the prefix AG. With the exception of the retaining wall barrier (AG5), four separate structural design alternatives were studied for at-grade intrusion barriers. The unit costs for such at-grade barrier alternatives are included in the cost summary table.

Similarly, barriers on elevated structures are designated with the prefix EL in the cost summary table. For barriers on elevated structures, three design alternatives were studied for each of the HSGGT equipment crash scenarios on structures. The unit costs for such alternate barriers on elevated structures are included in the cost summary table. It should be noted that the unit costs are for barrier elements only on new elevated structure construction. This study does not address the additional foundation and superstructure cost required to support the increased loads and forces of an intrusion barrier system on an existing structure.

Highway barriers are split into two categories: at-grade, with an HAG prefix; and elevated, with an HEL prefix. The two types have been designed for the two types of highway vehicles considered in this

TABLE 5-1. SUMMARY OF ESTIMATED INTRUSION BARRIER CONSTRUCTION COSTS

At-Grade Train Barriers

Alternate		Scenarios[1,2]			
		1,2,6,7,19,20	3,5,8,10,11*	12*	4,9,13*
		Unit Costs in $Million/kilometer ($Million/Mile)			
AG1	Precast Pile Foundations w/ Precast Concrete Wall Panels	$1.115 ($1.795)	$1.250 ($2.01)	$1.490 ($2.40)	$2.76 ($4.44)
AG2	Steel Bearing Pile Foundations w/ Precast Concrete Wall Panels	$1.200 ($1.927)	$1.410 ($2.27)	$1.605 ($2.59)	$3.27 ($5.25)
AG3	Caisson Foundations w/ Cast In-Place Concrete Walls Panels	$1.275 ($2.06)	$1.490 ($2.40)	$1.605 ($2.59)	$2.71 ($4.36)
AG4	Steel Pipe Pile Foundations w/ Structural Steel Wall	$1.365 ($2.19)	$1.430 ($2.30)	$1.900 ($3.06)	$3.28 ($5.28)
AG5	Cast-in-Place Concrete Retaining Wall Barrier *(scenario 6-10 only)*	$2.64 ($4.25)	$2.64 ($4.25)	------ ------	$3.38 ($5.44)

Elevated Train Barriers

Alternate		Scenarios			
		14*	15*	16*	17*,18*
		Unit Costs in $Million/kilometer ($Million/Mile)			
EL1	Precast Concrete Wall Panels	$0.445 ($0.713)	$0.530 ($0.845)	$0.745 ($1.188)	$1.160 ($1.874)
EL2	Cast-in-Place Concrete Wall Panels	$0.755 ($1.214)	$0.950 ($1.531)	$1.370 ($2.19)	$2.38 ($3.83)
EL3	Structural Steel Wall Barrier	$1.260 ($2.03)	$1.475 ($2.38)	$2.28 ($3.67)	$2.71 ($4.36)

Highway Barriers

Alternate		Scenarios			
		19	20	21	22
		Unit Costs in $Million/kilometer ($Million/Mile)			
HAG1	Cast-In-Place Concrete Wall Panel w/ Steel Railing, for Van Truck	1.170 ($1.874)	------ ------	------ ------	------ ------
HAG2	Cast-In-Place Concrete Wall Panel w/ Concr. Railing, for Tank Truck	------ ------	$1.320 ($2.11)	------ ------	------ ------
HEL1	Cast-In-Place Concrete Wall Panel w/ Steel Railing, for Van Truck	------ ------	------ ------	$0.645 ($1.056)	------ ------
HEL2	Cast-In-Place Concrete Wall Panel w/ Concr. Railing, for Tank Truck	------ ------	------ ------	------ ------	$0.690 ($1.109)

---

[1] Refer to Table 2-1 for scenario list.

[2] Scenario numbers designated by asterisks are dual barrier systems. Unit costs should be doubled to obtain total estimated construction costs of dual barrier systems.

study, a 36,300 kg (80,000 pound) tractor trailer van truck, and a 36,300 kg (80,000 pound) tractor trailer tank truck.

Table 5-1 summarizes the linear costs of intrusion barrier designs and alternatives for crash scenarios considered in this study. Separate sub-tables are given for at-grade train barriers, elevated train barriers, and highway barriers. A list describing the scenarios identified by the scenario numbers is given in Table 2-1.

Costs vary according to the scenario for which the design is intended. For each sub-table, scenarios have been separated into four groups, or columns. These groups are based on the barrier forces expected for the various scenarios. Different designs have been developed for each of these groups in Chapter 4, and costs have been developed for each of these designs. The first column represents scenarios with the lowest loading; the last column, the highest loading. The tables indicate that single barrier maglev and articulated high speed rail scenarios require the least costly barriers, whereas dual barriers, freight and non-articulated high speed rail scenarios require the most expensive barriers.

Costs also vary according to the barrier alternate indicated in the different rows. Clearly, the precast concrete panel alternatives are the least expensive. They cost less than other alternatives because they are less labor intensive. Precast panels can be shop-fabricated using efficient mechanized processes, and labor requirements in the field are reduced. In terms of difficulty of construction, the precast wall panels provide additional advantages when compared to the construction operations of forming and casting-in-place walls, and to a lesser extent, by the structural steel alternatives due to the continuously welded construction. Other advantages which appear to make the precast wall panel barrier construction the system of choice are summarized in Section 7.1.

The cast-in-place concrete retaining wall barrier (alternate AG5) is the most expensive alternate because of the large quantities of concrete, reinforcing steel, excavation, and backfill. This alternate should only be used where naturally occurring grade differentials occur between the adjacent corridors. In these situations, there is little or no differential in cost between conventional retaining walls and those designed as intrusion barriers.

5.1.4 Estimated Intrusion Barrier System Costs

An estimate of barrier system costs can be made for a selected train route. The costs will depend on such factors as the mix of adjoining transportation systems, what fraction of the system is elevated, the number of overpasses, and what fraction of the system requires barriers. Passages where the adjoining areas are not vulnerable to derailment nor do the areas pose a threat to the high speed line, do not require barriers.

Using data contained in an as yet unpublished Commercial Feasibility Study of High-Speed Ground Options, sponsored by the FRA, a cost estimate has been made of an American high-speed rail system ranging from $4.3M/km to $29.8M/km ($7M/mi to $48M/mi) with an average of $15.5M/km ($25M/mi). Estimates of barrier cost (p. xviii) range from $0.5M/km for an elevated barrier to $3.3M/km for an at-grade barrier ($.8M/mi to $5.4M/mi). From these data one may expect the barrier costs to range from less than ten percent of the system cost to as much as twenty percent. Further study of siting criteria (p. xx) will permit a better assessment of these costs.

5.2 BARRIER DAMAGE AND REPAIR COSTS

5.2.1 Methodology

The structural barrier designs and alternatives for each of the crash scenarios presented in this study have been assessed for probable maximum barrier damage sustained by a collision. The extent of barrier damage was based on interpretation of TBIP output displays. The output displays analyzed for each of the crash scenario incidents were for those runs which indicated the maximum impact forces as determined by equipment consist, speed, distance from centerline of the guideway to the barrier, and other parameters as defined in Chapter 3 of this report.

For purposes of preparing order-of-magnitude repair cost estimates for each crash scenario incident, the following must be determined: (1) length of barrier sections that require total replacement (critical lengths of wall failure as described in Chapter 4.1), (2) length of barrier sections that require minor repairs and restoration, and (3) length of barrier sections within the crash length that are not impacted by the vehicle and require neither total replacement nor minor repairs. The extent of probable barrier damage is therefore a function of length of a crash scenario incident and can be determined from interpretation of the TBIP display outputs which indicate the location of collision impacts and magnitudes of impact force.

Based on scaled measurements of TBIP output displays (e.g., Figure 3-4), the length of each crash scenario incident is defined as the distance between the initial and final impact points plus one-half of the critical wall failure length at each of these end points. Similarly, but more subjectively, the length of barrier sections that are not damaged can be estimated from scaled measurements. The difference in these measurements is the replacement/repair length. In the case of dual barrier scenarios, the replacement/repair lengths are determined separately for each wall. As indicated, the output displays assessed for each of the crash scenario incidents were those which showed maximum forces and, logically, would subject the barriers to greater damage.

For determining repair quantities, it has been assumed that 75% of the total damaged length determined as above would be totally replaced, and 25% would need only minor repairs. These quantities are reduced to linear meters (feet) of total replacement and square meters (feet) of minor repairs (measured in the vertical plane of the barrier wall), and cost estimates are based on the extension of unit prices for these repair elements. The estimated repair costs estimated herein for each of the scenarios represent a lump sum total of barrier replacement and repair costs and are stated in mid-1993 dollars.

5.2.2 Assumptions

In addition to the construction cost assumptions, the following assumptions have been used in the development of repair cost estimates:

- Estimated repair costs are for structural barrier elements only. No costs have been estimated for repair of guideway damage, superstructure damage on elevated structures, or other right-of-way infrastructure elements.

- Estimated repair costs are lump sum repair costs for each crash scenario incident, and include total replacement and minor repairs of barrier sections as required.

- Because of limited access, reduction in construction windows, requirements for demolition and removal of damaged barrier sections, and general reduction in repair efficiencies, the premium cost for total replacement has been estimated at an additional 50% of the previously estimated base unit costs for initial construction of barrier design and alternatives.

- Unit costs for minor repairs have been estimated at $81 per square meter ($7.50 per square foot) for concrete wall barriers and $108 per square meter ($10.00 per square foot) for structural steel barriers (square foot areas are vertical areas of barrier, i.e., length x height).

5.2.3 Estimated Repair Costs

Table 5-2 summarizes the repair costs for each barrier design alternative and crash scenario considered in this study. The elements of cost are provided in Appendix E to this report. Estimated repair costs have been rounded to the nearest five thousand dollars for each scenario.

As stated, the total repair cost for each scenario is composed of two separate elements: total replacement and minor repair costs. Minor repairs are generally assumed to include patching and/or shotcreting damaged surfaces in the case of concrete barriers, and straightening and painting in the case of structural steel members that may be reused without reducing the structural integrity of the barrier system. The costs for minor repairs are rather small when compared to the costs for total replacement. With the exception of Scenarios 11 through 13, minor repair costs represent a range of 2 to 4 percent of the total repair costs for each scenario. In the dual barrier alternatives of Scenarios 11 through 13, the minor repair cost component is in the range of 3 to 8 percent of the total repair costs of the alterations. This is indicative of the greater distance between impact points for the high speed equipment and proportionally less major damage to the barrier system.

Again, the precast concrete panel barrier alternatives are the least expensive, in terms of repair costs resulting from wall collision damage. The quick erection possible with precast concrete wall construction represents a marked advantage when repairs must be accomplished on operating facilities.

TABLE 5-2. SUMMARY OF INTRUSION BARRIER REPAIR COST ESTIMATES

At-Grade Barriers	Total Replacement/Repair Costs for Barrier Alternatives				
Scenario Number[1]	AG1	AG2	AG3	AG4	AG5
1	$150,000	$160,000	$165,000	$155,000	
2	$235,000	$250,000	$265,000	$265,000	
3	$160,000	$180,000	$210,000	$175,000	
4	$295,000	$195,000	$285,000	$305,000	
5	$160,000	$180,000	$215,000	$175,000	
6					$280,000
7					$490,000
8					$305,000
9					$315,000
10					$315,000
11	$180,000	$205,000	$265,000	$185,000	
12	$490,000	$555,000	$545,000	$625,000	
13	$1,025,000	$1,130,000	$1,025,000	$1,170,000	

Elevated Barriers

Scenario Number	EL1	EL2	EL3
14	$55,000	$90,000	$175,000
15	$175,000	$305,000	$490,000
16	$270,000	$480,000	$820,000
17	$320,000	$645,000	$775,000
18	$325,000	$650,000	$780,000

Highway Barriers

Scenario Number	HAG1	HAG2	HEL1	HEL2
19	$10,000.00	------	------	------
20	------	$50,000.00	------	------
21	------	------	$10,000.00	------
22	------	------	------	$50,000.00

[1] Refer to Table 2-1 for scenario list.

6. HAZARDS EVALUATION

6.1 VEHICLE DAMAGE ASSESSMENT

The accurate determination of vehicle damage and costs is a complicated, time-consuming problem, requiring finite element analyses and other such cost intensive techniques that are beyond the scope and objectives of this study. For this reason, costs have *not* been estimated for repair of vehicle damage.

An estimate of the damage anticipated for the train, however, may be obtained from the simulation by identifying the number of cars which strike the barrier(s), and the corresponding maximum impact force experienced by each car. From this data, the expected crush distance may be estimated, allowing an order of magnitude determination of the severity of damage.

The barrier/car interface is approximated in the TBIP code by a linear elastic spring. While this is not a very sophisticated approach, given the lack of knowledge about the constitutive properties of the car bodies and the required precision in barrier design practice, it is believed to be an acceptable model for estimating barrier design forces. The resulting predicted forces can be used to develop a qualitative, first-order estimate of the level of vehicular damage associated with each impact scenario. For purposes of this section, it is assumed that corner crushing of 300 to 600 mm (1 to 2 ft) or less is repairable, or minor, vehicle damage; while crushing of much more than this level is termed major damage, which may be more expensive or irreparable.

Figures 6-1 through 6-6 show the predicted maximum impact force sustained by each car impacting against each barrier for each of the HSR simulations selected as the design case. The damage sustained by the various cars can be determined by identifying those cars for which the force exceeds the value of 600 mm (2 feet) multiplied by the car's spring stiffness value.

The maximum impact force for the ICE cars impacting a single barrier ranges from 445 kN (100 kips) to nearly 1113 kN (250 kips). The assumed linear spring stiffness used in the simulation was 2481 kN/m (170 kips/ft). The anticipated structural damage to the car bodies is corner crushing of less than 300 mm (1 ft) (minor damage) on seven of the cars, with somewhat greater damage to one car. The last eight

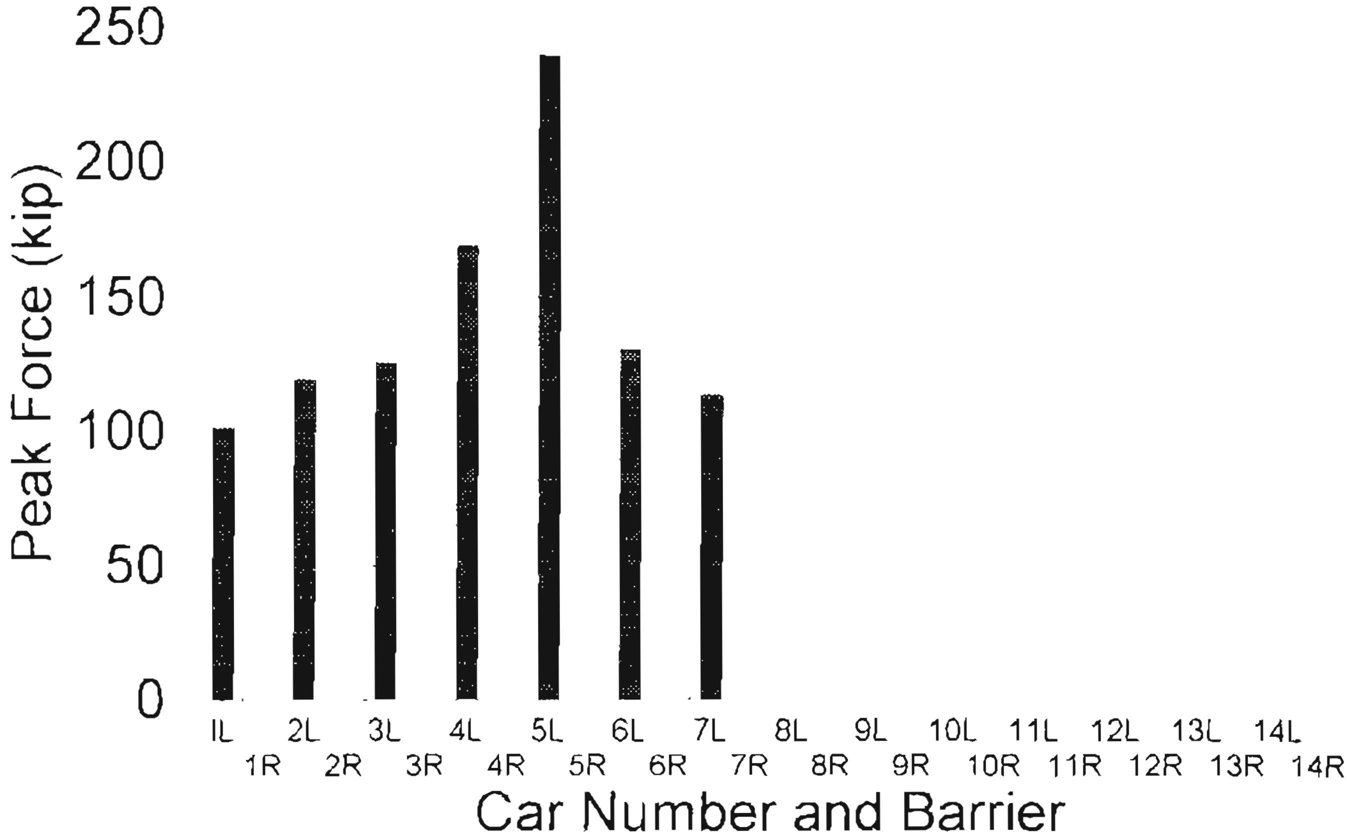

FIGURE 6-1. MAXIMUM BARRIER FORCES ON CARS
14-CAR ICE 120 KM/H (75 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET

Note: Metric Equivalent 1kip= 4.45 kN
1 ft= 0.305 m

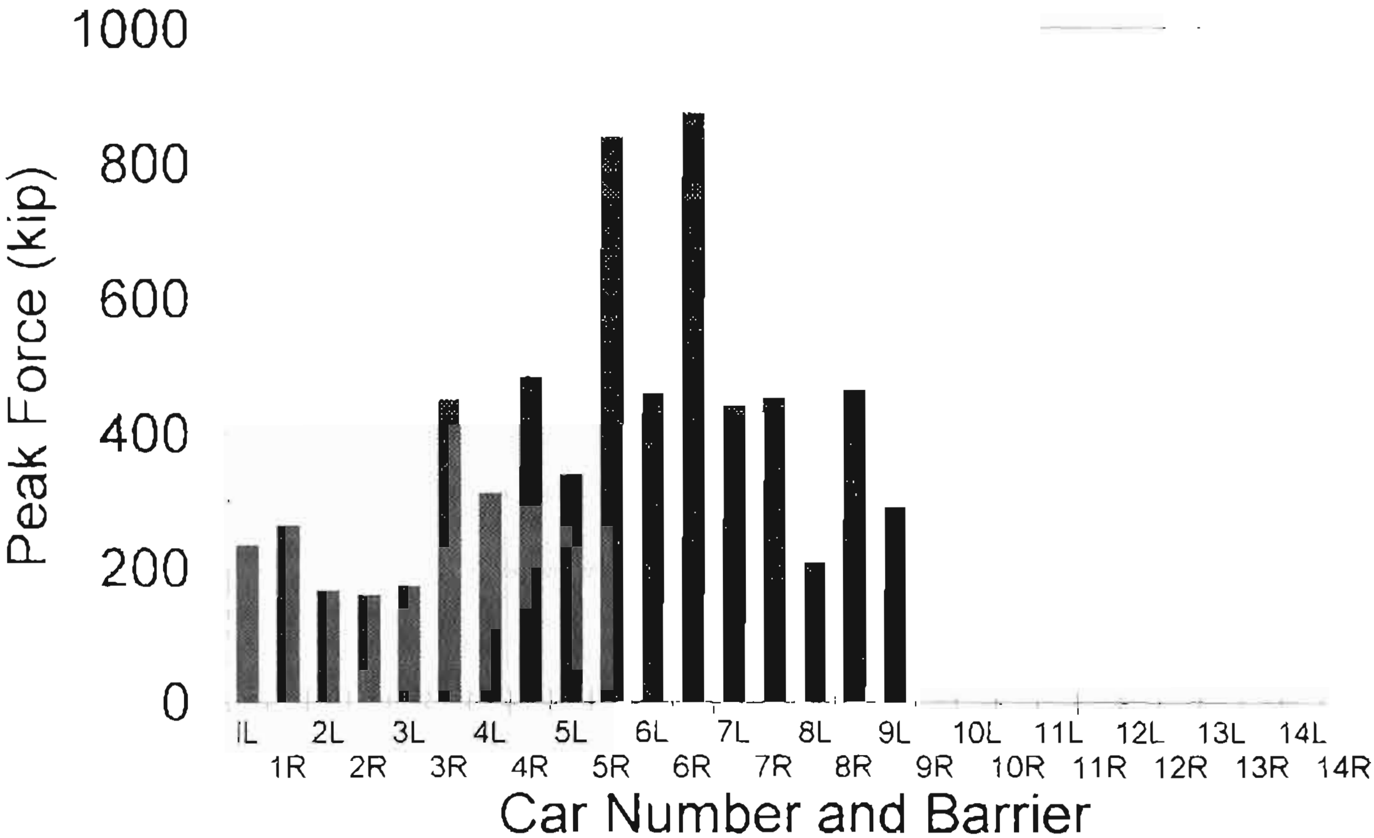

FIGURE 6-2. MAXIMUM BARRIER FORCES ON CARS
14-CAR ICE 160 KM/H (100 MPH) DUAL BARRIER, 2.74 M (9 FT) OFFSET

Note: Metric Equivalent 1 kip= 4.45 kN
1 ft= 0.305 m

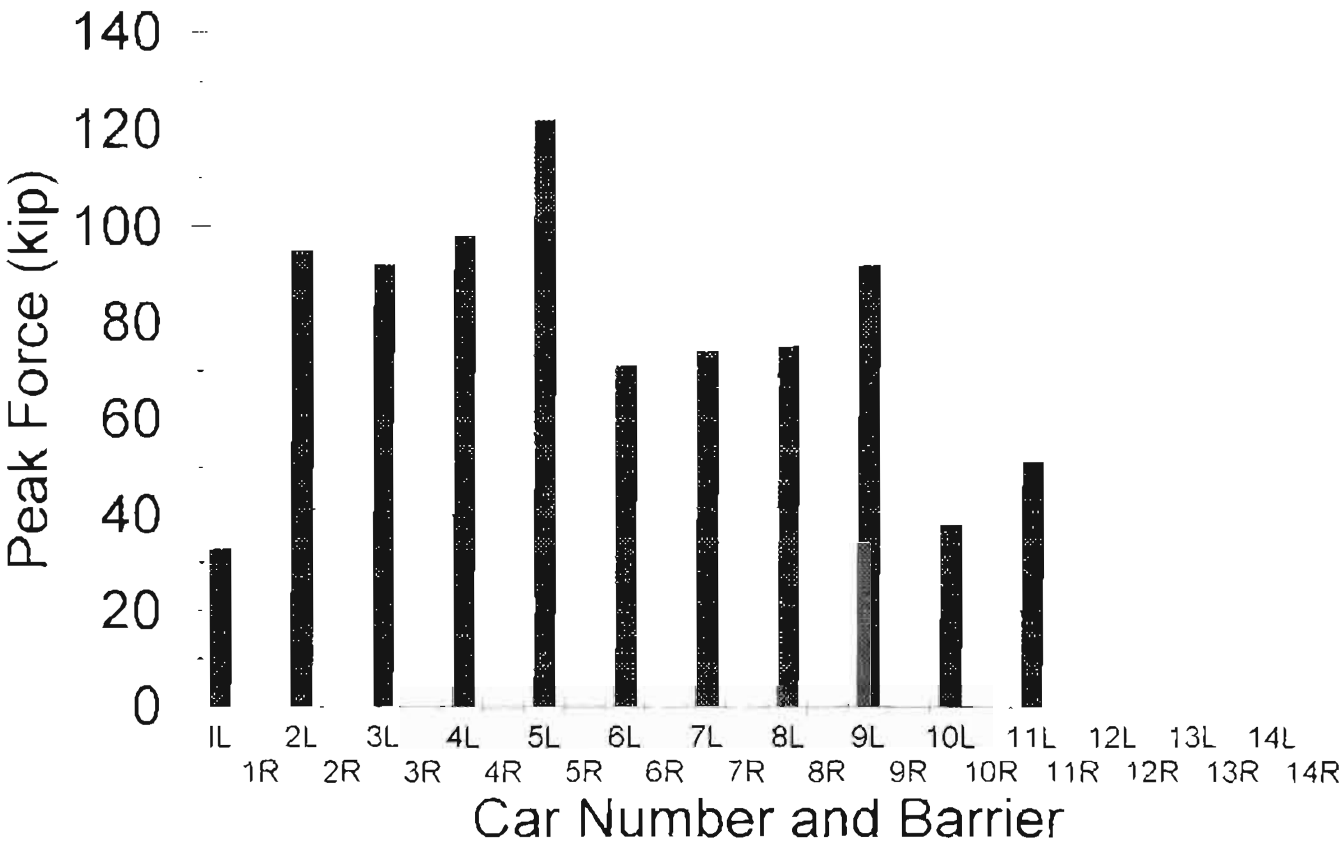

FIGURE 6-3. MAXIMUM BARRIER FORCES ON CARS
12-CAR TGV 160 KM/H (100 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET

Note: Metric Equivalent 1 kip= 4.45 kN
1 ft= 0.305 m

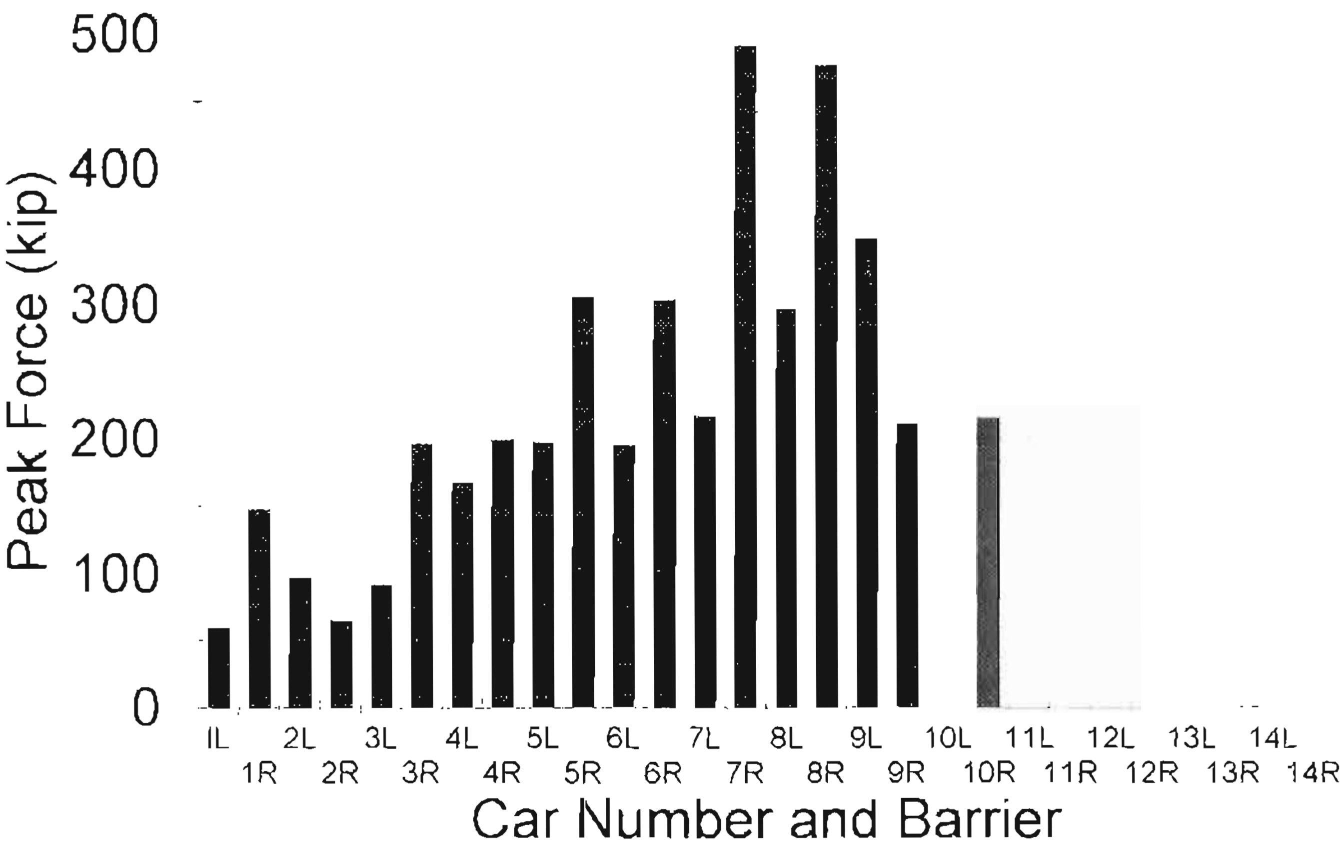

FIGURE 6-4. MAXIMUM BARRIER FORCES ON CARS
12-CAR TGV 160 KM/H (100 MPH) DUAL BARRIER, 2.74 M (9 FT) OFFSET

Note: Metric Equivalent 1 kip= 4.45 kN
1 ft= 0.305 m

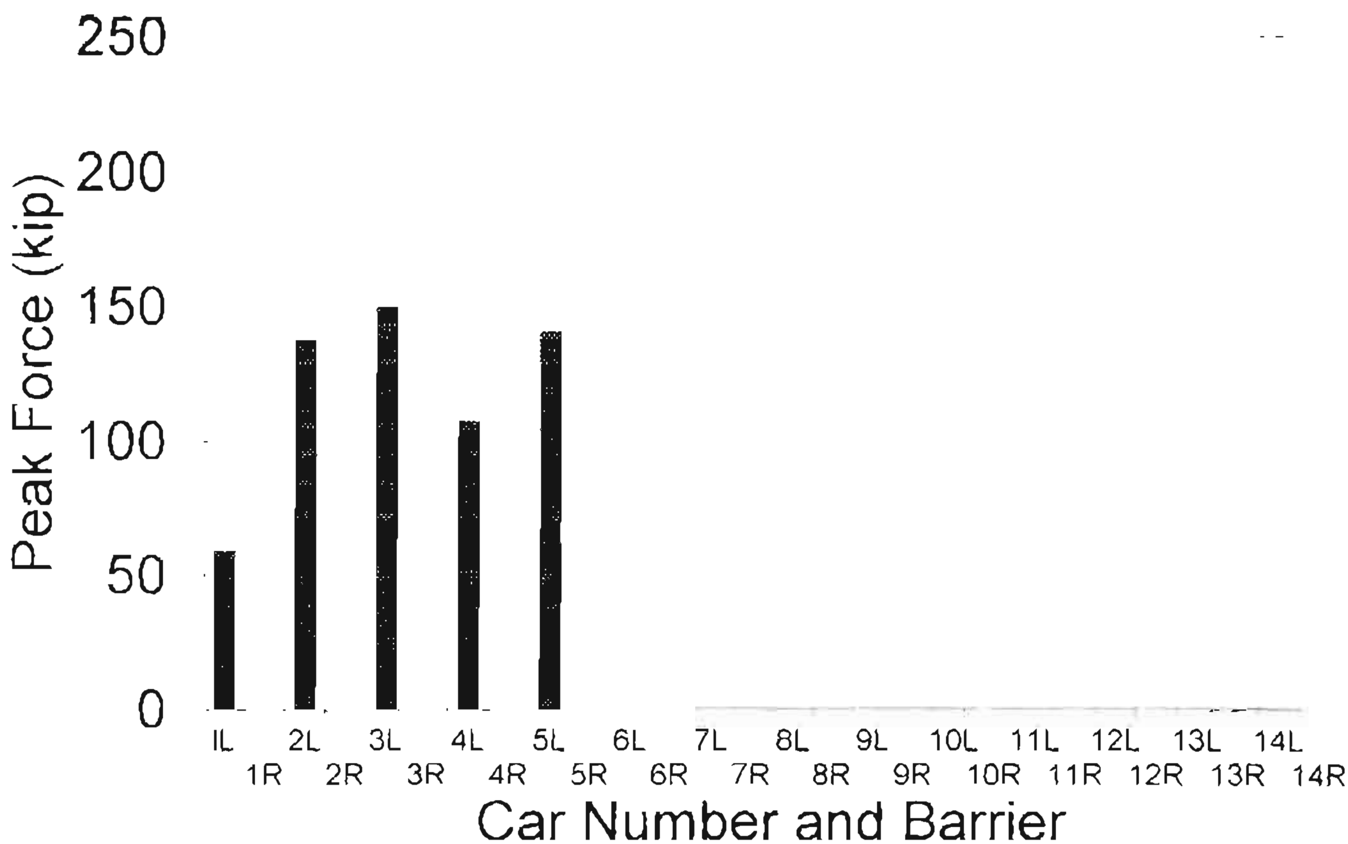

FIGURE 6-5. MAXIMUM BARRIER FORCES ON CARS
8-CAR MAGLEV 120 KM/H (75 MPH) SINGLE BARRIER, 3.35M (11 FT) OFFSET

Note: Metric Equivalent 1 kip= 4.45 kN
1 ft= 0.305 m

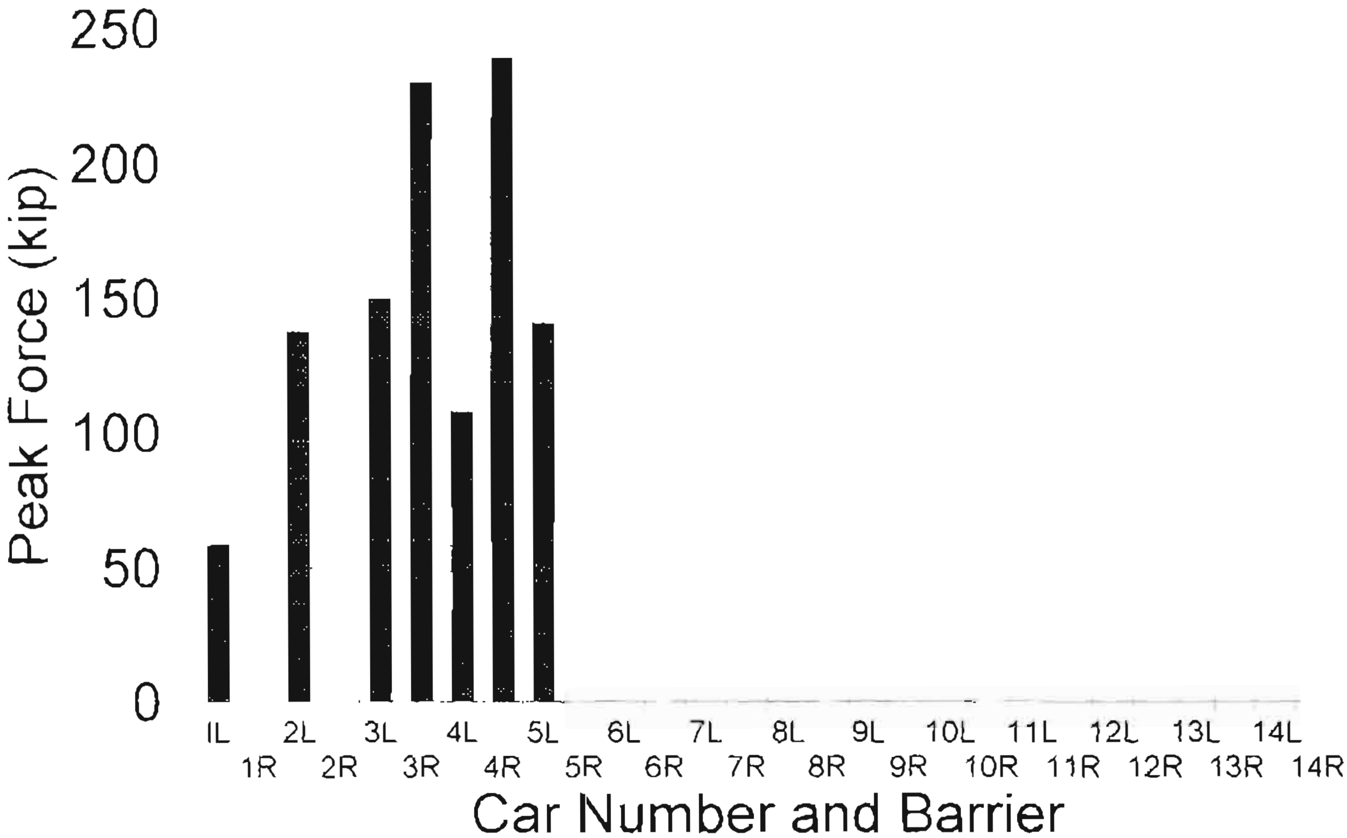

FIGURE 6-6. MAXIMUM BARRIER FORCES ON CARS
8-CAR MAGLEV 120 KM/H (75 MPH) DUAL BARRIER, 3.35 M (11 FT) OFFSET

Note: Metric Equivalent 1 kip= 4.45 kN
1 ft= 0.305 m

cars in the consist do not contact the barrier in this scenario. When dual barriers are involved, the forces and deformations are greater, with corner crushing deformations of more than 600 mm (2 ft) in six cars. Two of these six cars experienced forces of more than 3650 kN (800 kips), corresponding to an anticipated crush deformation on the order of 1.5 m (5 ft) (major damage).

In the single barrier scenario, all of the TGV cars experience forces expected to cause only minor damage. In the case of TGV cars and dual barriers, major damage is expected on three cars, and minor damage is expected to all other cars except the last, which does not impact either barrier in the scenario studied. The acceleration experienced by the last car is due to forces acting on the coupler and through the trucks, not due to barrier impact.

The first five cars in the eight-car Maglev consist are expected to sustain only minor damage when derailing in the presence of a single barrier offset 3.35 m (11 ft). When dual barriers are used, higher impact forces, and a second crushed corner, are predicted for two of the five cars, although the anticipated corner crush is still less than 600 mm (2 ft).

Major/minor vehicle damage is tabulated in Table 6-1 for a selected group of scenarios.

TABLE 6-1. SUMMARY OF ESTIMATED HSGGT VEHICLE DAMAGE

		Damaged Cars	
Vehicle	Barrier Type	Minor	Major
14 car ICE	Single	7	--
	Dual	3	6
12 car TGV	Single	11	--
	Dual	10	3
6 car Maglev	Single	5	--
	Dual	5	--

6.2 PASSENGER SAFETY ASSESSMENT

6.2.1 Introduction

Relationships between occupant safety and vehicular dynamics during a collision are extremely complex and difficult to quantify because they involve such important but widely varying factors as occupant physiology, size, seating position, degree of restraint, and compartment geometry and padding. Guidelines for evaluating vehicular impacts with roadside safety appurtenances are contained in the National Cooperative Highway Research Program (NCHRP) Report 350, "Recommended Procedures for the Safety Performance Evaluation of Highway Features" (Ross, et al. 1993) [15]. This document uses a simplified point mass, flail-space model for assessing risks to occupants within an impacting vehicle due to vehicular accelerations. For unrestrained conditions, two measures of risk are addressed: (1) the velocity at which a hypothetical occupant impacts a hypothetical interior surface, and (2) ridedown acceleration experienced by the occupant subsequent to contact with the interior surface.

The extent or severity of injury is primarily dependent on the occupant-to-compartment impact velocity and the intensity of forces to which the occupant is subjected thereafter. The occupant experiences essentially no absolute acceleration prior to impacting some part of the compartment interior. At occupant impact, the degree of injury sustained by the occupant is indicated by the magnitude of the occupant/compartment impact velocity which is determined by assuming the occupant moves as a free body across the compartment space. Following this impact, the occupant is assumed to remain in contact with the impacted surface and then directly experiences any subsequent accelerations imparted to the car. The maximum average acceleration occurring in any 10 millisecond (ms) period is used to evaluate occupant risk during this phase.

Threshold occupant impact velocity (OIV) and occupant ridedown acceleration have been determined from several sources including human volunteer testing, sled tests of animals, cadavers, and dummies, and automotive accident statistics. An attempt has been made to set the threshold values at a level equivalent to the American Association of Automotive Medicine Abbreviated Injury Scale (AIS) of 3 or less. AIS-3 classifies the resulting injury as severe but not life threatening.

Table 6-2 shows the recommended occupant risk values as adopted by NCHRP Report 350.

TABLE 6-2. RECOMMENDED OCCUPANT RISK VALUES [15]

Severity Measure	*Preferred Value*	*Maximum Acceptable Value*
Occupant Impact Velocity	9 m/s (30 ft/s)	12 m/s (39 ft/s)
Occupant Ridedown Acceleration (g)	15	20

A threshold value of 20 g is used for both the lateral and longitudinal directions. This value is considered survivable (i.e., AIS-3) for even long durations [25, 26, 27]. The design or preferred value is obtained by dividing the limit or threshold value by a factor of 1.33.

In order for the acceleration to produce occupant injury, it must have a minimum duration ranging from 0.007 to 0.04 sec., depending on the body component [25]. Thus, acceleration spikes of less than 0.007 sec. duration are not critical and are averaged from the pulse. An arbitrary duration of 0.010 sec. was selected as a convenient and somewhat conservative time base for averaging vehicle accelerations for occupant risk assessment.

6.2.2 Assessment of Passenger Risk

As the above recommendations reflect current practice in occupant risk analysis for occupants of automobiles involved in collisions and other highway accidents, they may not be appropriate for the analysis of risk to high speed rail passengers during derailments. Several factors contribute to the differences in the two types of events. First, the vehicle interior may be significantly different from that of a typical automobile. Because of this, different seating patterns, different treatment of interior surfaces, the presence or absence of occupant restraint systems, etc., will mean that the occupant impact velocity for passengers in an HSR vehicle may be significantly different from that of an automobile occupant. Second, the duration of the event causing the hazard is much longer for the HSR derailings than in a typical automobile collision event. For instance, consider the collision of an automobile with a roadside barrier. The duration of the portion of the event during which injuries are caused is short — on the order of 1 sec., compared with the duration of the events evaluated in this study which are on the order of 10 sec.

In spite of the differences between automobile roadside barrier collisions and the HSR derailing events studied here, the well established standards applied to highway vehicles are used in a first-order analysis of passenger risk. To accomplish this, the predicted acceleration histories of the cars in each derailing train have been determined from the TBIP runs that were used for barrier design. These runs represent the selection of variables (speeds, offsets, etc.) that produce maximum impact forces. The assumption made here is that the variables that produced the maximum barrier impact force would also produce the maximum vehicle accelerations. The resulting acceleration time histories are shown in Figures 6-7 and 6-8, for the 12-car ICE consist, at the speed and offset selected as being critical for barrier design. In these figures, the data plotted is the acceleration output at 10 ms intervals. The standard practice for automobile collision analysis calls for a 10 ms moving average, which cannot be generated from the 10 ms data represented. The 10 ms data can be used as an approximation of the 10 ms average data, however, for purposes of estimating maximum acceleration values and maximum barrier force values. Using the 10 ms data, the maximum resultant acceleration for each car is calculated and plotted in Figures 6-9 through 6-14.

From Figures 6-9 through 6-14, using the criteria listed above, the following conclusions may be drawn. For all single barrier cases studied, the peak accelerations for all cars do not exceed the 15 g recommended as maximum during the entire events. Dual barrier cases result in significantly higher peak accelerations, except for the Maglev cases, where the dual barrier collision is not appreciably different from the single barrier collision. The dual barrier, 12-car TGV model peak resultant car accelerations do not appreciably exceed 15 g, but 6 of the 12 cars experienced acceleration levels at approximately that value. The 14-car ICE model, when impacting dual barriers, experience significantly higher acceleration values. Nine of the fourteen cars experience peak accelerations above 15 g, and five experienced peak accelerations above 20 g. It is noted that these values are mass-center accelerations, and those passengers seated away from the mass center will experience greater or lesser values, depending on their location and the simultaneous magnitude of angular velocity and angular acceleration values for the car in question. The 14-car ICE dual barrier case represents the most critical scenario from the point of view of occupant safety, with potentially two-thirds of the passenger space experiencing acceleration levels which would be considered unacceptable by automobile collision standards.

To provide some insight into the question of how much increased hazard is represented by the presence of a barrier to a HSR consist in the event of a derailment, a simulation of a derailment of the 14-Car ICE studied above has been accomplished in the absence of a barrier. This has been accomplished by

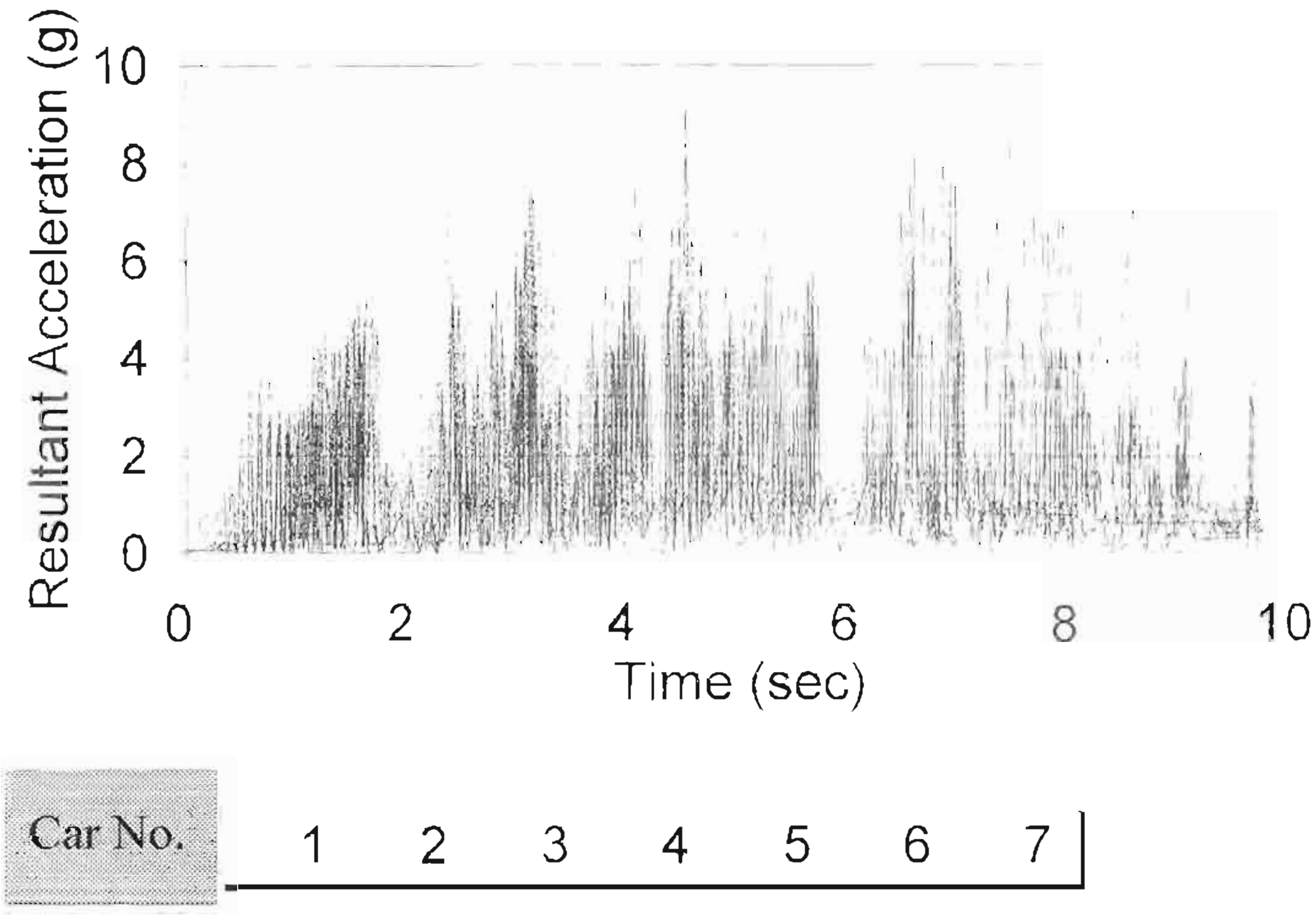

FIGURE 6-7. ACCELERATION OF CAR MASS CENTER
14-CAR ICE 120 KM/H (75 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET, CARS 1-7

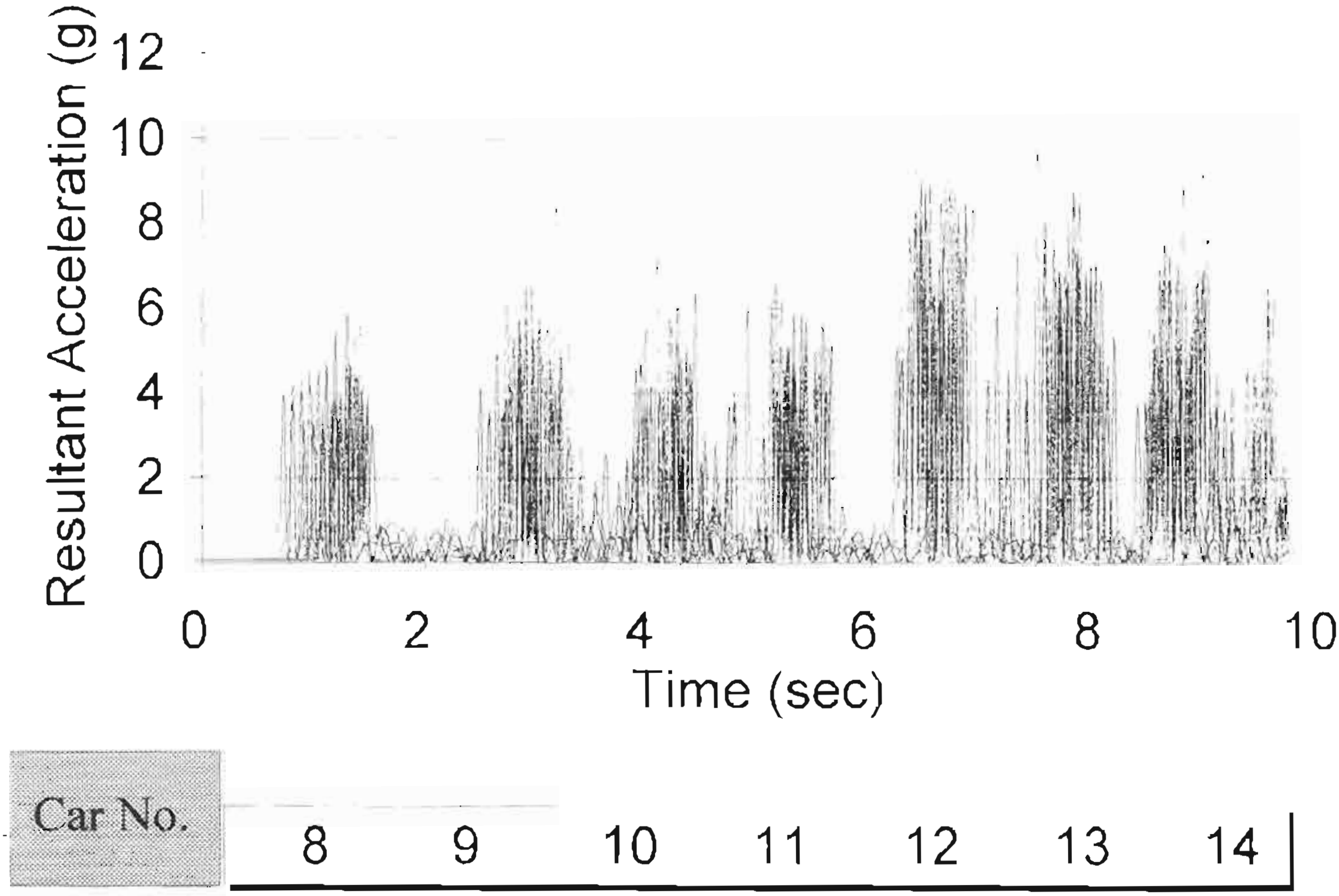

FIGURE 6-8. ACCELERATION OF CAR MASS CENTER
14-CAR ICE 120 KM/H (75 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET, CARS 8-14

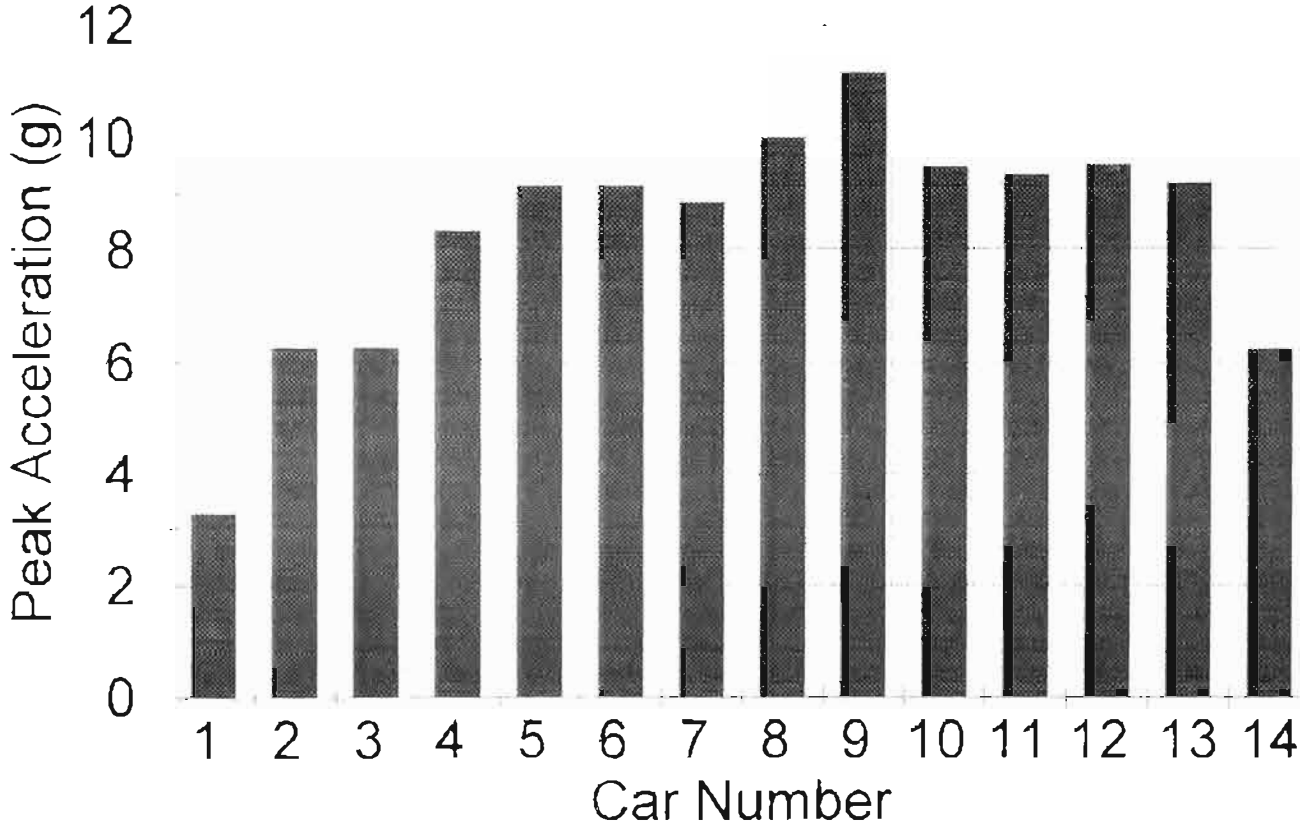

FIGURE 6-9. MAXIMUM RESULTANT ACCELERATION
14-CAR ICE 120 KM/H (75 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET

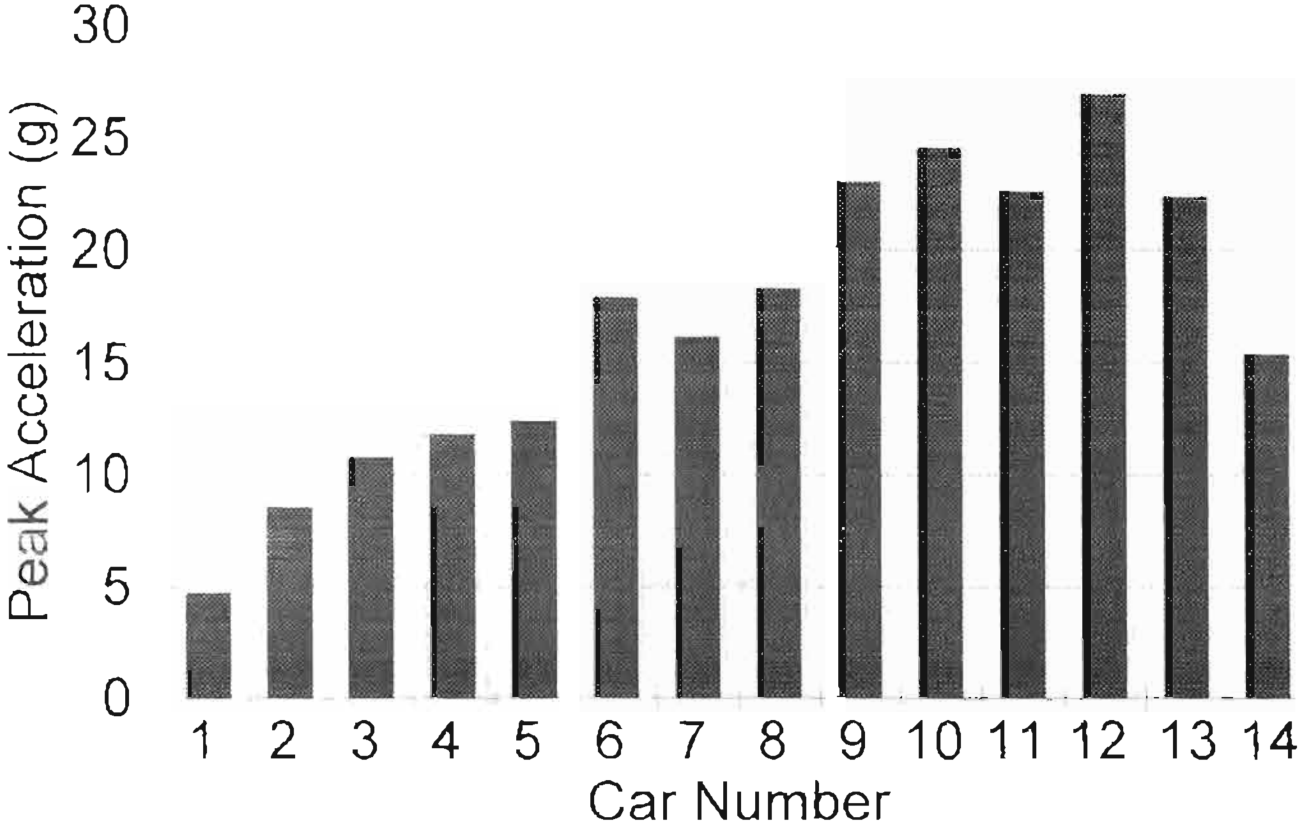

FIGURE 6-10. MAXIMUM RESULTANT ACCELERATION
14-CAR ICE 160 KM/H (100 MPH) DUAL BARRIER, 2.74 M (9 FT) OFFSET

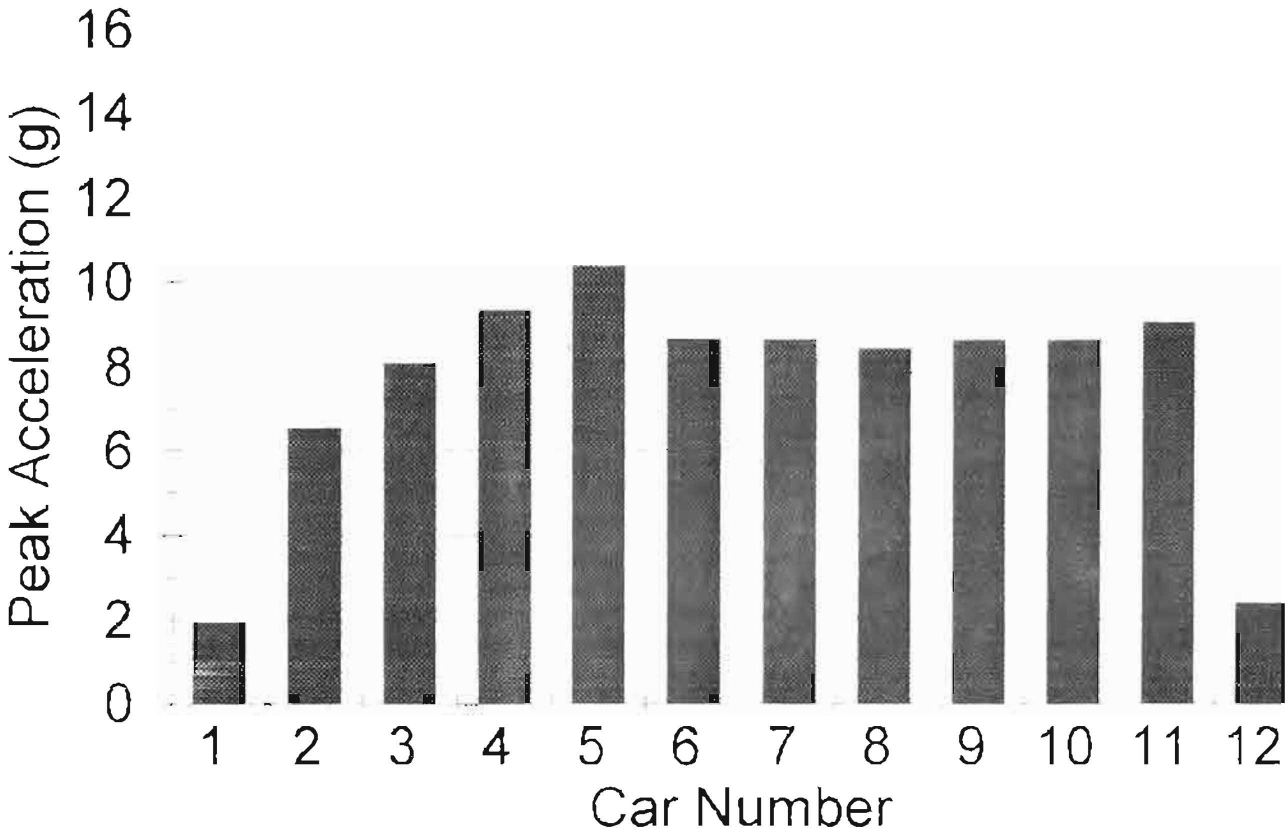

FIGURE 6-11. MAXIMUM RESULTANT ACCELERATION
12-CAR TGV 160 KM/H (100 MPH) SINGLE BARRIER, 2.74 M (9 FT) OFFSET

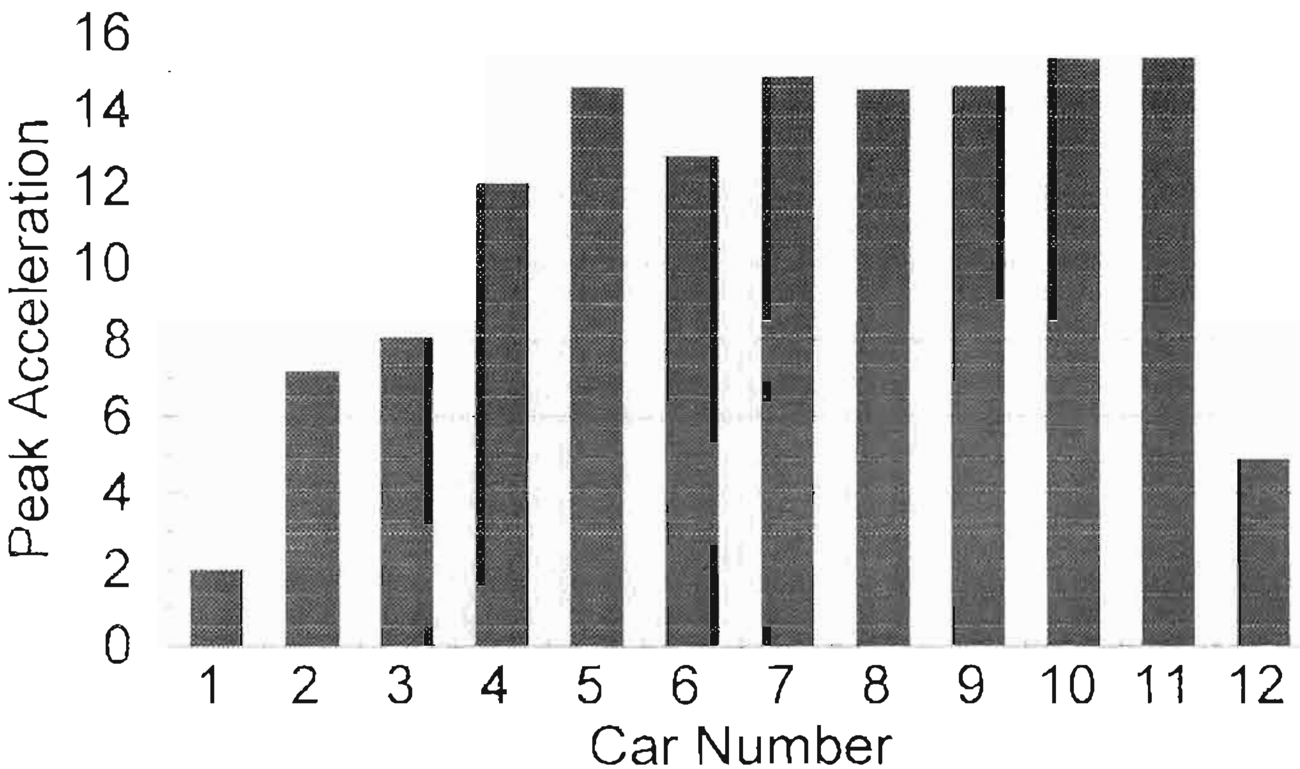

FIGURE 6-12. MAXIMUM RESULTANT ACCELERATION
12-CAR TGV 160 KM/H (100 MPH) DUAL BARRIER, 2.74 M (9 FT) OFFSET

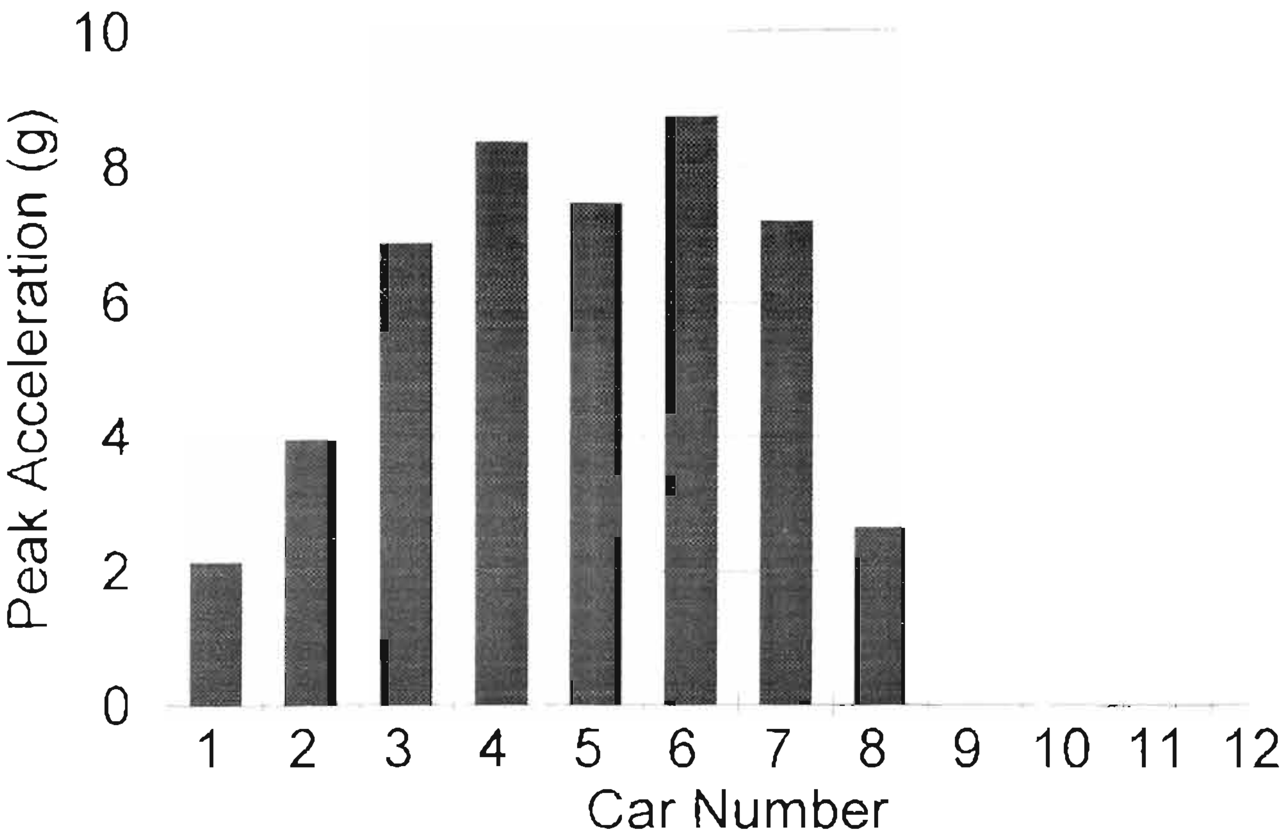

FIGURE 6-13. MAXIMUM RESULTANT ACCELERATION
8-CAR MAGLEV 120 KM/H (75 MPH) SINGLE BARRIER, 3.35 M (11 FT) OFFSET

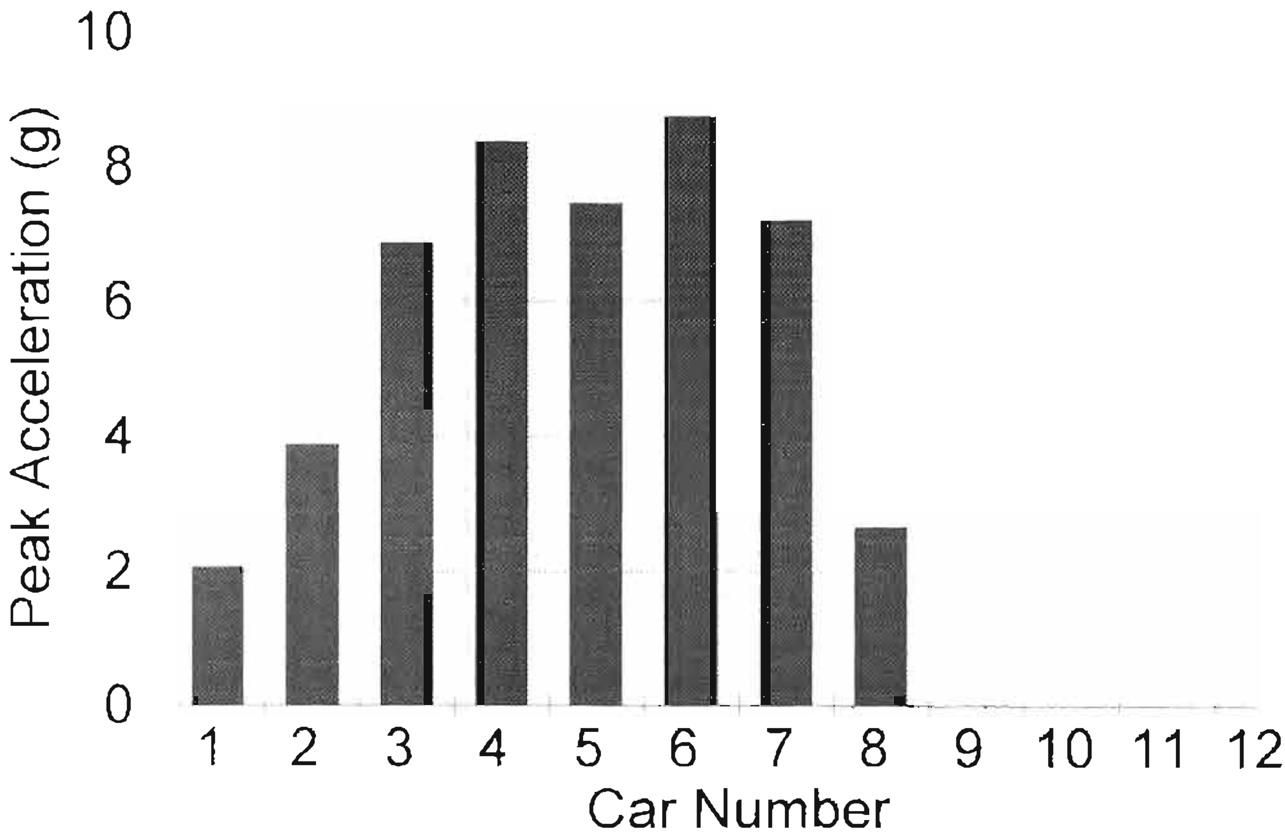

FIGURE 6-14. MAXIMUM RESULTANT ACCELERATION
8-CAR MAGLEV 120 KM/H (75 MPH) DUAL BARRIER, 3.35 M (11 FT) OFFSET

simply moving the barrier in the simulation to a distance great enough (30 m) from the track centerline so that the derailing vehicles do not interact with the barrier. The speed selected for this simulation is 160 km/h (100 mph), making this case identical to the dual-barrier ICE design case discussed above, except for the absence of the barriers. This case has been selected because of the high observed acceleration values in the presence of the dual barrier.

Figures 6-15 through 6-19 are selected results of this simulation. A comparison of Figure 6-15 with Figure 6-10 shows that the peak resultant accelerations are roughly doubled in the presence of the dual barrier. The maximum peak accelerations are still over 10 g in the absence of barriers, which is at first a curious result, given that the input coefficient of ground friction is only 1.0. An examination of Figures 6-16 through 6-19 leads to the observation that a lot of noise is present in the acceleration signal. A mean (direct current or DC) value of acceleration could be estimated from these figures which is not inconsistent with the 1.0 Coefficient of ground friction value. Noise of a similar nature is observed in automobile/barrier crash tests, because of "ringing," or excitation of higher frequency structural modes of the car structure, which is superimposed on the DC value in the measurements. The presence of the noise in the simulated signal is from different sources — excitation of various modes of vibration in the train structure. Structural damping was not modeled, so it is reasonable to expect that a more rigorous model might yield lower peak accelerations.

The acceleration values shown in the figures suggest that the presence of a barrier increases peak car accelerations by more than 100%. Based on engineering judgment, this is a reasonable prediction for the effect of the dual barrier. Since the single barrier simulation was carried out at 120 km/h (75 mph) (an initial speed which is believed to be critical for barrier loading for that case), a direct comparison of Figure 6-15 with Figure 6-9 is made more difficult. Still, it is reasonable to conclude that the presence of a single barrier, as in Figure 6-9, does not adversely affect peak car accelerations nearly as much as does the presence of dual barriers.

A summary of passenger safety assessment is given in Table 6-3. The conclusion to be drawn from these data is that the presence of crash barriers results in an increased acceleration that passengers must endure. Passenger safety is not compromised, however, based on automobile standards, except for dual barrier high speed rail installations, where accelerations exceed the threshold of 15 g.

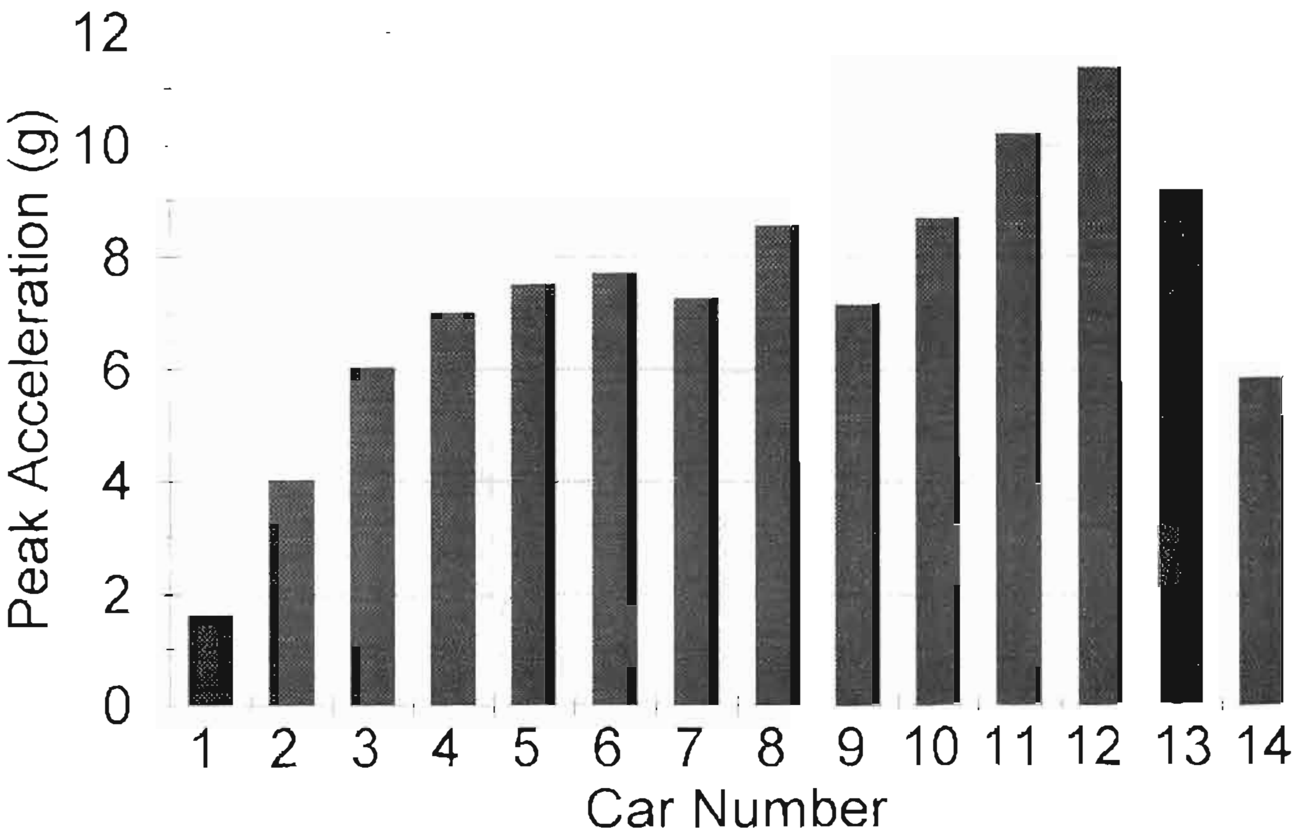

FIGURE 6-15. MAXIMUM RESULTANT ACCELERATION 14-CAR ICE 160 KM/H (100 MPH) NO BARRIER

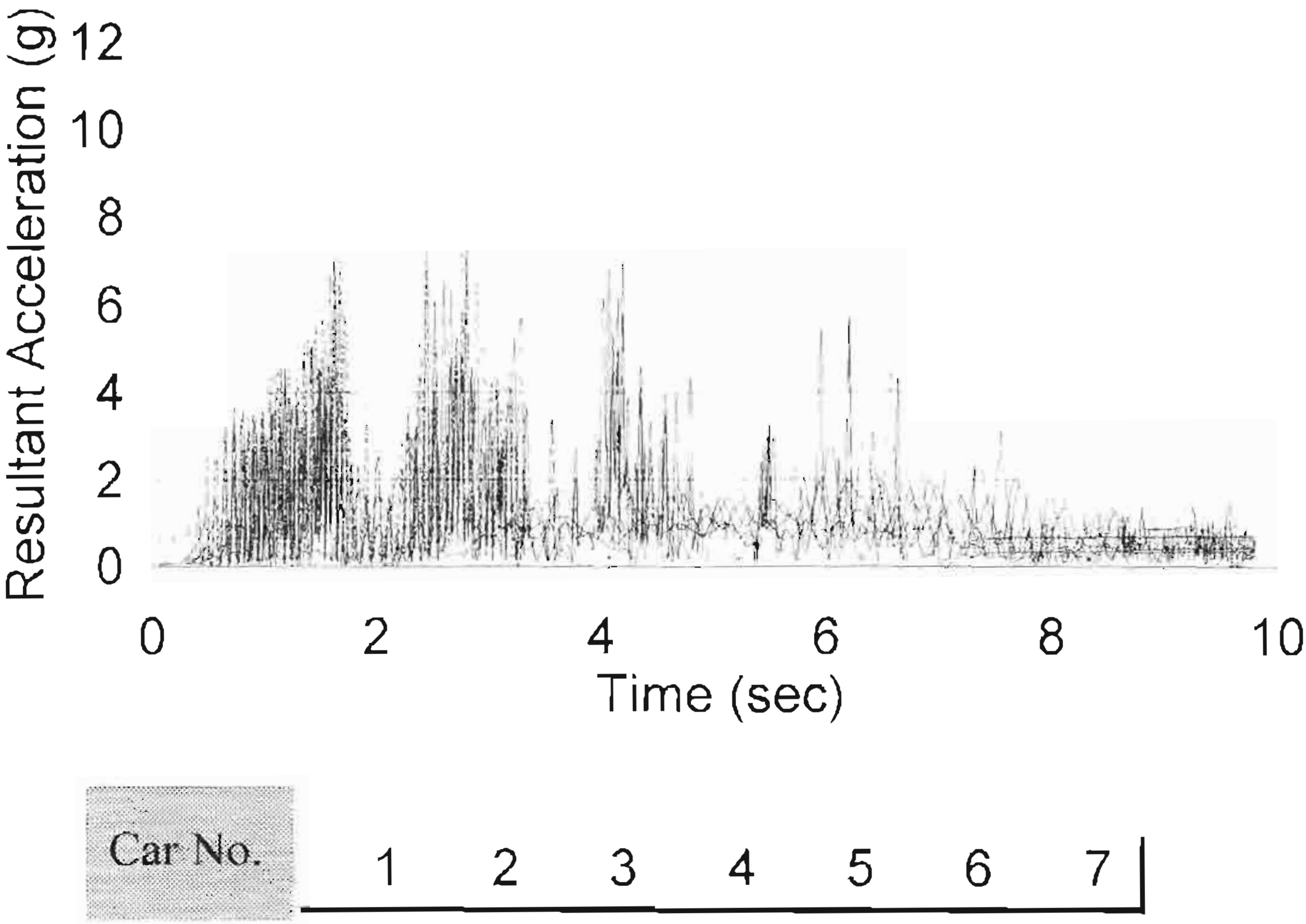

FIGURE 6-16. ACCELERATION OF CAR MASS CENTER
14-CAR ICE 160 KM/H (100 MPH) NO BARRIER, CARS 1-7

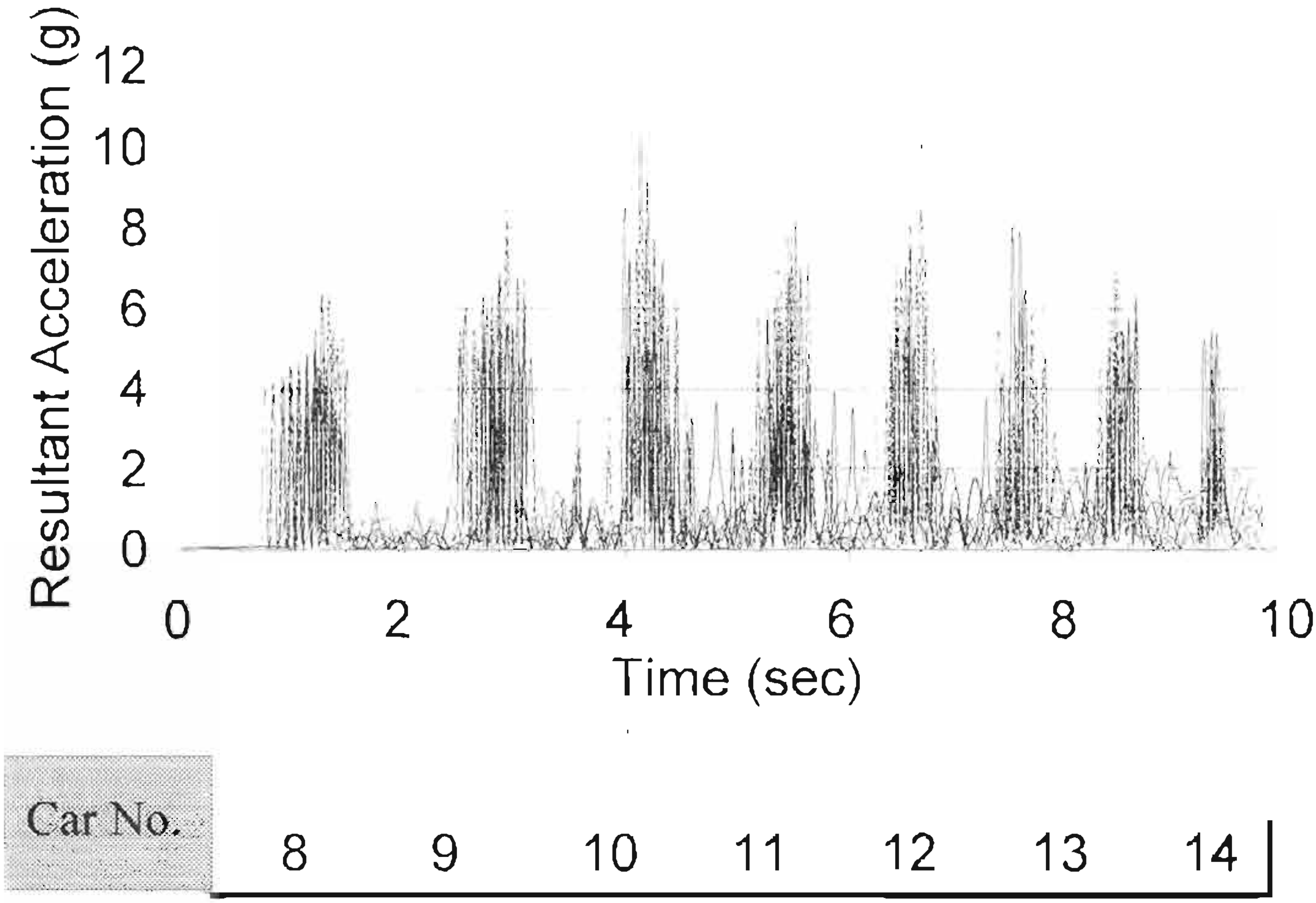

FIGURE 6-17. ACCELERATION OF CAR MASS CENTER
14-CAR ICE 160 KM/H (100 MPH) NO BARRIER, CARS 8-14

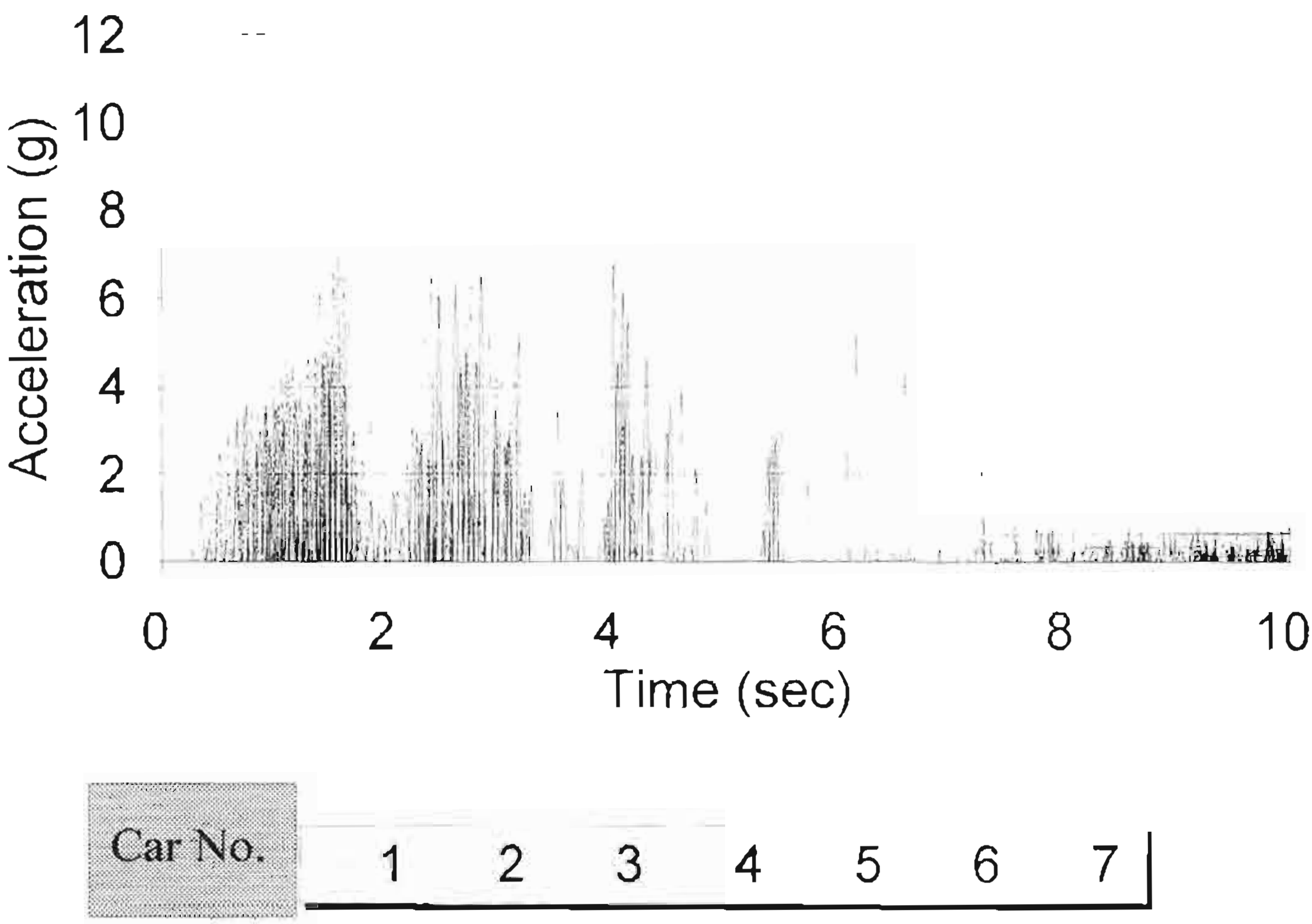

FIGURE 6-18. LONG ACCELERATION OF CAR MASS CENTER
14-CAR ICE 160 KM/H (100 MPH) NO BARRIER, CARS 1-7

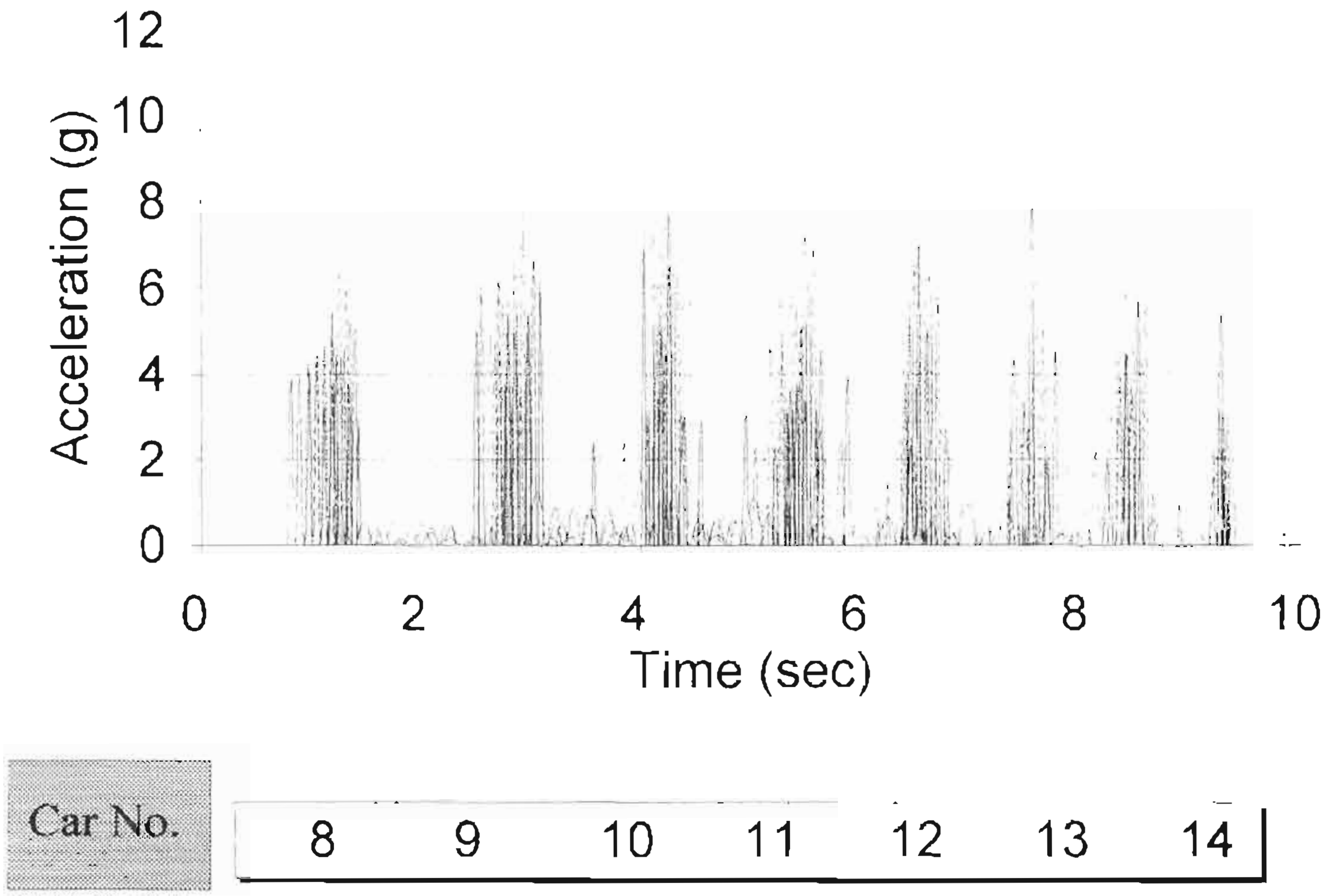

FIGURE 6-19. LONG ACCELERATION OF CAR MASS CENTER
14-CAR ICE 160 KM/H (100 MPH) NO BARRIER, CARS 8-14

TABLE 6-3. SUMMARY OF PASSENGER SAFETY ASSESSMENT

		Acceleration Experienced by Passengers	
Vehicle	Barrier Type	Cars Exceeding 15 g	Cars Exceeding 20 g
14-car ICE	Single	--	--
	Dual	9	5
12-car TGV	Single	--	--
	Dual	6	--
6-car Maglev	Single	--	--
	Dual	--	--

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 RECOMMENDED INTRUSION BARRIER TYPES

In this study, methods for the design of intrusion barriers have been developed, and barriers have been designed. Barrier costs have been estimated both in terms of construction cost and damage repair cost. The hazards to impacting vehicles and their passengers have been evaluated. The conclusion of the study is that the design and construction of effective intrusion barriers is feasible. It is clear that some types of barriers are more effective and feasible than others, and some should not be used at all.

Structural Barriers Structural intrusion barriers similar to the designs presented in Figures 4-6 through 4-31 are recommended. These barriers are feasible and recommended for all scenarios except where dual barriers would be used for railroad vehicles.

Dual Railroad Barriers: The forces estimated by the TBIP runs are so large for dual barriers containing conventional railroad vehicles that unreasonably large barriers result. They would only be required in elevated applications, such as on overhead railroad bridges. The derailment impact loads, however, would create large scale damage to the bridge superstructure and substructure. It is therefore recommended that these locations use some means other than intrusion barriers for reducing intrusion risk, such as speed restrictions, sensors, and increased maintenance procedures. Later studies on siting of barriers can make a better assessment after further study of the increased risks associated with this approach in comparison with the costs sited in this report. At this stage, however, the use of dual railroad barriers appears impractical.

Highway Barriers: Highway barriers capable of resisting 36,300 kg (80,000 pound) trucks, such as those shown in Figure 4-31 should be used.

Earthwork Barriers: Earthwork barriers such as berms and ditches are not recommended. They present an increased hazard for violent motion during derailment of high speed vehicles and do not prevent intrusion effectively.

Combination Barriers: Due to its high cost, the retaining wall combination barrier system shown in Figure 4-19 should only be used where grade differentials between the HSGGT system and adjacent transportation system require a retaining structure to maintain cut or fill requirements.

Alternative barrier types have been suggested in an attempt to provide the flexibility to take advantage of local practices and material availability. All of those listed will effectively perform the intended intrusion function. Careful consideration of many issues is critical to the selection of an intrusion barrier design that will be most effective, cost efficient, and appropriate to a specific site. The more obvious factors are costs (first cost, damage repair costs, and maintenance costs), speed of erection, right-of-way costs, environmental impacts, and ease of maintenance repairs from vehicle impact. Local issues notwithstanding, among all the alternate designs considered in this study, the recommended barrier system from an engineering, constructability and replacement/repair point of view is the precast concrete barrier system, with either precast concrete or steel foundation (AG-1 or AG-2). The precast alternates offer the following advantages:

- Lower construction cost, depending somewhat on geographical cost differences.

- Superior concrete quality compared to cast-in-place. Service life will be extended due to the increased protection of reinforcing steel offered by the higher quality, denser concrete.

- Less cracking and maintenance for concrete. Almost all of the elastic shortening, shrinkage, and creep will have taken place in the precast plant prior to construction.

- Quick erection. No formwork will be required. Only temporary lateral bracing of the columns and walls will be required until the connections between precast components have been completed.

- Quick repairs with minimal impact to operating facility. The modular nature of precast construction will lend itself to quick and inexpensive construction of repairs in the event of impact damage. This ease and speed of removing and replacing a portion of the barrier in the event of a train derailment or mishap is critical to restoring the service of all effected transportation systems in the shared right-of-way.

7.2 RECOMMENDED INTRUSION BARRIER DESIGN METHODOLOGY:

The recommended methodology for design of intrusion barriers is summarized as follows:

Establish Design Forces: For HSGGT and railroad vehicles a dynamic computer analysis such as the TBIP model described here should be used to estimate barrier impact forces. For highway vehicles, the methodology specified in the new AASHTO LRFD Bridge Design Specifications [13] (a draft of which is now under development) should be followed.

Design Barriers: After the barrier type has been selected, the barrier is designed. Performance Specifications are included in Appendix B for the purpose of establishing criteria for design. They can also be used to evaluate future barrier designs.

Construct Barriers: The barriers should be constructed according to local accepted practice, in accordance with the designs developed, and in accordance with governing local and national codes.

7.3 OTHER ISSUES TO BE CONSIDERED IN BARRIER DESIGN AND CONSTRUCTION

Beyond the technical issues described above, there are many others that must be addressed. Before a facility owner selects, designs, and constructs a barrier system it is important to consider the additional design, construction, and operational issues described below.

7.3.1 Design

- The actual soil conditions existing at the proposed site must be determined with a subsurface exploration program. Foundations must be designed to accommodate the in-situ soil conditions encountered.

- The extent of construction to be included in a specific project and the conditions which are to exist at the completion of that project must be defined.

- Environmental impacts must be considered and mitigated, especially in sensitive areas.

- The proximity of currently operating rights-of-way must be evaluated, and methods developed for continuing operations during construction.

- The calculations and construction documents produced by the designer should be reviewed to verify the adequacy of the design of the intrusion barrier system.

7.3.2 Construction and Operation

- Right-of-Way acquisition costs must be evaluated. Agreement must be reached on easements, rights, and responsibilities for inspection and maintenance of the intrusion barrier.

- Existing rights-of-way and structures (e.g. utilities, catenary wires and supports, bridges, culverts, retaining walls, buildings and trackwork) must be protected during intrusion barrier construction and the responsibility for repair of any damage must be established prior to the commencement of construction.

- Restrictions on construction activities must be established, including loads imposed on existing structures, access to intrusion barrier locations, and traffic detours.

- Insurance requirements must be determined and specified to the contractor.

- Administration and inspection personnel must have access to the construction site for the purpose of ascertaining that the work is proceeding in accordance with the contract documents.

- The reviewing agency must have the opportunity to review and comment on any change orders which have potential effects on intrusion barrier construction.

- The reviewing agency must have access to construction records, such as erection schedules, pile or caisson test results, driving logs, concrete test results, and other pertinent data affecting the intrusion barrier construction.

- The reviewing agency must have the right to review and approve the intrusion barrier construction procedures for construction adjacent to and above existing rights-of-way.

- Construction impacts requiring night and weekend work with limited working hours should be evaluated, in conjunction with the preparation of a detailed traffic maintenance plan.

7.4 CONCLUSIONS AND RECOMMENDATIONS FOR MINIMIZING INTRUSION HAZARDS

Certain conclusions can be drawn related to safety of different types of HSGGT systems, and recommendations can be made on ways to minimize intrusion hazards.

Intrusion Barriers: It is a foregone conclusion that constructing a barrier will reduce hazards associated with derailing HSGGT, or adjacent, vehicles. The barriers should effectively prevent intrusion that would result in a catastrophic accident. The passenger assessment discussed in Chapter 6, however, illustrates that there are hazards associated with the barrier itself. A derailed HSGGT vehicle, if fortunate enough to miss any adjacent vehicles, would experience higher accelerations with a barrier than without a barrier. Nevertheless, intrusion barriers clearly reduce, rather than increase hazards.

Conclusion: Intrusion barriers should be used to reduce intrusion hazards.

Dual Barriers: For all consists, dual barriers result in higher impact loads. High speed rail and freight consists impose extremely high forces onto dual barriers, and subject vehicles and passengers to strong forces and movements. In the case of high speed rail scenarios, vehicle accelerations are expected to exceed threshold limits for passenger safety currently accepted by the automobile industry. Barrier and vehicle damage in the event of a derailment are expected to be high for HSGGT, as are repair costs. Strengthening of bridge superstructures in elevated barriers would be significant and could add significantly to the cost of the installation. Maglev vehicles impart higher forces on dual barriers also, and the same observations hold true as above, except that they are not as serious.

Conclusion: The use of dual barriers should be avoided where possible for high speed rail consists and freight consists.

Number of Cars: It has been found that forces increase dramatically with longer consists.

Conclusion: Shorter consists should be used where feasible.

Vehicle Speed: Vehicles are found to impart higher impact loads to the barriers when traveling at lower derailment speeds, with the maximum load occurring at a derailment speed of 120 to 160 km/h (75 to 100 mph) for HSGGT consists, and 88 to 104 km/h (55 to 65 mph) for freight consists.

Conclusion: HSGGT guideways should be located such that barriers are not necessary in low design speed areas. In low speed areas, where possible, HSGGT guideways should be sited as far as practical (farther than 15 m) from adjacent corridors to eliminate the need for barriers. HSGGT consists should be operated at higher speeds where possible adjacent to barriers.

Barrier Offset Distance: Impact forces increase with increasing offset distances from the centerline of guideway up to a point, after which they decrease eventually to zero for large offset distances.

Conclusion: Avoid offsets where forces are maximum (from 2.74 m [9 feet] to 12 m [40 feet]). Instead, site barriers either very close to the guideway or very far from the guideway. Where barriers are located between two guideways, they should be located closest to the guideway that produces the highest forces.

7.5 RECOMMENDED FURTHER STUDY

7.5.1 Testing to Verify Assumptions

Further study is also needed to verify parameters used in the analysis and design of the barriers. In the current study, many of the parameters have necessarily been based on assumptions. Although reasonable values have been selected based on previous research in the automobile industry and elsewhere, the assumptions should be verified. An example is the assumed value of crush stiffness used in the TBIP program. This value has been extrapolated from results of tests performed on automobiles, trucks, and buses. Analysis indicates that variation of crush stiffness yields a wide variation in impact force. This and many other parameters could best be verified with crash testing or detailed analytical techniques that are outside of the scope of this study. The following techniques could be used:

1. *Full scale crash testing:* This testing would involve full scale crash testing of a high speed vehicle (ideally) against an instrumented barrier. This test would record actual forces generated in the impact from which stiffness values could be generated. While a full-scale crash testing approach would provide valuable information, total reliance on it would be an expensive proposition. If prohibitively expensive, testing of conventional freight or passenger trains would also provide useful information, perhaps at a lower cost than for high speed vehicles. Preliminary investigation indicates that performance of these tests for 40-car freight trains would cost approximately $1.5 M.

2. *Single car testing:* Impact tests of single cars against instrumented barriers can be used to better evaluate car crush stiffness and calibrate the TBIP models. Such tests are less expensive than full-scale consist tests and should be carried out first.

3. *Scale model testing:* Small scale (for example 1/8 full size) crash tests could be carried out as described above. The cost may be more reasonable and the results still instructive. There are

inaccuracies in the scale models, however, that must be accounted for analytically. This would introduce more uncertainty in the results than either of the above test methods, but it may prove to be a more cost effective approach.

4. *Analytical techniques:* Finite element analysis could be undertaken to determine force deformation characteristics of specific vehicles. Due to the uncertainties in failure modes, however, the results would be suspect.

These studies would provide additional information for FRA consideration in further evaluating safety issues in shared rights-of-way and would verify some of the unknowns that still exist after completion of this study.

7.5.2 Siting of Barriers

It is beyond the scope of this study to recommend where intrusion barriers should be used to minimize intrusion hazards. On the contrary, this study is intended to determine the physical requirements for intrusion barriers once the need for a barrier has been established. It should not be construed that the barriers developed in this study must be installed in all locations where high speed guided ground transportation systems are located adjacent to other transportation modes. The criteria for siting of intrusion barriers should be the subject of future studies.

Decisions must be made to determine in which locations, intrusion hazards warrant the cost of barriers. It may not be necessary to locate barriers at all locations on shared rights-of-way, as was assumed in the case study. More prudent siting criteria could reduce barrier installation costs significantly. High speed consists are designed and maintained to minimize derailments. Actual performance indicates a good track record. It may be more reasonable to locate protection-type intrusion barriers to exclude errant vehicles from high speed guideways at locations where there is a record of derailments of adjacent conventional trains, or errant highway vehicles. Containment of HSGGT vehicles provided by intrusion barriers may be necessary only at HSGGT terminals and in urban areas, but may be unnecessary in remote areas.

The costs associated with intrusion barriers are significant. They must be evaluated in combination with risks of intrusion in order to make decisions on where barriers are needed and where they are not. Structural intrusion resistance requirements could vary according to risk. AASHTO's use of warrants for highway design, along with performance levels which vary with traffic expectations, may be a reasonable approach to use in the siting of intrusion barriers.

7.5.3 Corridor-Specific Risk Analysis

A study of the siting of barriers should be performed that is specific to proposed corridors in order to more accurately determine where barriers are warranted. It should incorporate an evaluation of derailment risks associated with the specific equipment and guideway geometries to be used in the proposed high speed corridor. Considerations should include maintenance standards and maximum speeds. Also the operational environment, infrastructure condition, accident history and maintenance standards of any adjacent rail or highway corridors should be considered. This evaluation should be accomplished early in the development of the proposed corridor design because the cost and location of barriers may influence the final corridor geometry.

The risk analysis should consider all relevant aspects of the facility that affect the risk of intrusion in order that valid decisions can be made with respect to barrier placement. When assessing the need for barrier placement, an operating "profile" for the HSGGT facility and the adjacent transportation facility must be developed. Items of interest that make up this profile and influence the "risk" of operations and likely placement of barriers should be identified. Some examples are given below.

HSGGT Facility:

1. proposed operations
2. maximum operating speeds
3. guideway geometry
4. traffic type and mix
5. operating environment (weather conditions, trespassers, wildlife, etc.)
6. maintenance standards

Adjacent Transportation Facility:

1. traffic type and mix
2. maximum operating speeds
3. local geometry of adjoining easement

4. operating environment
5. infrastructure condition
6. maintenance standards
7. accident history
8. planned operations

APPENDIX A - LIST OF REFERENCES

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APPENDIX A - SUPPLEMENTAL READING LIST

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APPENDIX B - PERFORMANCE SPECIFICATIONS

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<u>Chapter A</u>

GENERAL

1. INTRODUCTION

The design criteria contained in this section have been utilized in the design of all structures or parts of structures for the HSGGT Intrusion barrier Study Project, including barrier structures, at-grade and elevated structures, and foundations.

In addition to requirements stated herein, the design of a structure owned and maintained by a particular agency, shall also be in accordance with standards utilized by that agency.

The values stated herein are shown in SI (metric) units and in U.S. Customary (English) units, with English units in parentheses.

2. CODES, MANUALS, AND SPECIFICATIONS

The following codes, manuals, and specifications shall be utilized in the structural design, unless otherwise specified herein. In case of conflicting provisions, the more restrictive shall govern unless justified by analysis or otherwise stated herein:

a. "Standard Specifications for Highway Bridges," Fourteenth Edition, 1989, including "AASHTO Interim Specifications, Bridges, 1991," of the American Association of State Highway and Transportation Officials, referred to in these criteria as "AASHTO Bridges."

b. "Manual for Railway Engineering," of the American Railway Association, Volumes 1 and 2, 1993 or latest edition, referred to in theses criteria as "AREA Manual."

c. "Building Code Requirements for Reinforced Concrete, ACI 318-89," of the American Concrete Institute, including its commentary, referred to in these criteria as "ACI 318-89."

d. "Analysis and Design of Reinforced Concrete Guideway Structures, ACI 358.1R-86," of the American Concrete Institute, referred to in these criteria as "ACI 358-86."

e. "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," Ninth Edition, 1989, of the American Institute of Steel Construction, referred to in these criteria as "AISC specifications."

3. DESIGN GUIDELINES

In addition to the Intrusion Barrier Design Study Report (the "Study Report") of which these specifications are a part, the design of the barrier wall structure shall use the guidelines and codes indicated herein:

- Plastic Design for Structural Steel.
- Yield-Line analysis for Reinforced Concrete.
- TTI Report (see List of References).
- WMATA Report (see List of References).
- AISC Specifications (Steel).
- ACI 318-89 (Reinforced Concrete).
- DIN 1072 (German Standard, see List of References).
- DIN 1075 (German Standard, see List of References).

Chapter B

MATERIALS

1. GENERAL

All materials shall conform to the applicable specifications and codes listed in Chapter A. If, in the opinion of the designer, significant economies can be achieved by the use of different materials than those specified in this section, while providing at least the same level of performance and durability, the designer may substitute alternate material standards after receiving written approval from the appropriate authority.

2. STRUCTURAL STEEL

Unless otherwise specified, structural steel shall conform to ASTM A36M (A36), Grade 250 (36), or to ASTM A588M (A588), Grade 345 (50).

High strength bolts and anchor bolts for structural steel connections shall conform to ASTM A325 (AASHTO M164) or to ASTM A490.

3. REINFORCED CONCRETE

Unless otherwise specified, concrete shall have a minimum specified compressive strength (f'c) of 27.5 MPa (4,000 psi) at 28 days.

All reinforcement shall be ASTM A615M (A615) or A706M (A706) Grade 400 (60).

4. PRECAST PRESTRESSED CONCRETE

Unless otherwise specified, concrete for prestressed members shall have a minimum specified compressive strength (f'c) of 41.2 MPa (6,000 psi) at 28 days, and a minimum compressive strength at time of initial prestress (f'ci) of 27.5 MPa (4,000 psi).

Prestressing reinforcement shall be high-strength steel wire, high-strength seven-wire strand, or high-strength alloy bars.

- High-strength steel wire shall conform to ASTM A421.

- High-strength seven-wire strand shall conform to the requirements of ASTM A416, Grade 1860 (270), including supplement for low relaxation strand.

- High-strength alloy bars shall conform to the requirements of ASTM A722. Bars with greater minimum ultimate strength but otherwise produced and tested in accordance with ASTM A722 may be used provided they have no properties that make them less satisfactory than the specified material and are approved by the appropriate authority.

Chapter C

LOADS

1. DEAD LOADS

Dead load shall consist of the weight of the complete structure and all material permanently fastened to and supported by it, including but not limited to, trackwork, barriers, walls, foundations, soil, water, and all other permanent loads.

2. LIVE LOADS

a. General

Live loads shall consist of any non-permanent loads, including the weight of the vehicle and the weight of the passengers, construction loads, and loads due to maintenance operations.

b. Vehicle Loads

For design purposes, the live loads applied to rail or guideway-supporting structures, such as bridge decks, shall take into account the axle loading and spacing, and car spacing for the equipment to be used on the facility. Data for some HSGGT vehicles are given in Table 3-1 of the Study Report. The number of cars in a consist shall be taken as that number which produces the most critical loading for the element under consideration, not to exceed the maximum number for which the system is designed. Critical loading shall be checked for axial, bending, shear and torsional stresses, deflections, and stability. Axle loads on track ties, or direct fixation are to be uniformly distributed longitudinally over not more than 2.29 meters (7.5 feet) when on 132-pound RE rail.

The weight of the loaded vehicles used for structural design should be based on "crush loading," i.e., the vehicle loaded with all seats full. The average passenger weight shall be taken as 75 kg (165 pounds); an average which includes provisions for luggage and other items.

3. DERAILMENT IMPACT FORCES ON VEHICLE INTRUSION BARRIERS

Impact forces resulting from derailment shall cause no collapse and no overturning of the barrier system or the elevated guideway structure. For elevated guideways, these forces may act simultaneously on the deck outside the limits of the tracks and on the vehicle restraint barriers.

Very little existing criteria address derailment loads for rail transit structures and railroad structures, and none for high speed rail. ACI 358 provides recommendations for transit vehicles of moderate speed [up to 160 km/h (100 mph)]. Since high speed rail service with speeds up to 483 km/h (300 mph) are considered in this project, the magnitude and line of action of derailment forces will be determined by a detailed analysis based on a two-dimensional computer program. Refer to section 3 (Study Methodology) of this Study Report.

Minimum design impact forces shall be as given in Table 3-3 of the Study Report.

4. VEHICLE IMPACT (OTHER THAN FROM DERAILMENT)

a. General

For design of those structures or structural elements listed below, the live loading shall be increased for dynamic, vibratory and impact effects from moving loads. These loads do not include horizontal impact forces from collision with a barrier.

Items to Which Impact Applies

- Superstructure, including steel or concrete supporting columns, legs of rigid frames, and generally those portions of the structure which extend down to the main foundation.

- The portion of concrete or steel piles above the ground line when they are rigidly connected to the superstructure as in rigid frames and continuous structures.

Items to Which Impact Does Not Apply

- Abutments, retaining walls, wall-type piers.
- Foundations and footings.
- Safety walks, stairways, station platforms or other pedestrian areas.

b. Vertical Impact Force

Impact considerations for bridges shall be in accordance with Article 3.8 of "AASHTO Bridges," or the AREA Manual. The impact factor shall be applied to the vehicle loading. Alternatively, vertical impact may be taken as 30 percent of the live load, with a 20 percent minimum for long spans.

c. Transverse Horizontal Impact Force

Provisions shall be made for a transverse horizontal impact force due to lateral swaying of the vehicle or due to rail misalignment and uneven wear of the wheels, equal to 10 percent of the vehicle loading. This force shall be applied horizontally in the vertical plane containing each axle and shall be assumed to act, normal to the track, through a point at least 3.0 feet (check center of gravity of vehicle) above the top of the low rail.

The transverse horizontal impact force and the centrifugal force shall be assumed to act simultaneously and are additive.

5. CENTRIFUGAL FORCE

In horizontal curves, a centrifugal force shall be applied horizontally to rail supporting structures, in the vertical plane containing each axle. The force shall be assumed to act through a line at the center of gravity of the particular vehicle under consideration. The magnitude of the centrifugal force shall be computed by the following formula:

$$CF = \frac{PV^2}{32.2R}$$

Where: CF = Centrifugal force in pounds

P = axle load in pounds

V = Velocity of train in feet per second

R = Radius of curvature in feet

The velocity shall be the maximum design velocity of the train except as limited by maximum allowable superelevation, grades, etc., of the track for the location of the structure.

This force is a radial force and shall be applied to the train as concentrated loads at the axle locations. The horizontal force component transmitted to the rails and supporting structure by an axle shall be concentrated at the rail having direct wheel-flange-to-rail-head contact.

Chapter D

BARRIER STRUCTURES

1. GENERAL

The barrier structure consists of a longitudinal wall made of precast or cast-in-place reinforced concrete or structural steel built for the purpose of withstanding the lateral impact forces imposed by a derailed vehicle. The wall must also be strong enough to redirect the vehicle, and high enough to prevent the vehicle from overturning and rolling over the wall.

The barrier structure shall consist of the following two systems:

Single-barrier: One barrier located on one side of the tracks between the two right-of-ways.

Dual-barrier: Two separate barriers, one located on each side of the dual tracks of the train consist under consideration. This type of barrier shall be used on elevated structures, in areas where shared right-of-way is located on both sides of the tracks under consideration, and as required by analysis or other operational requirements.

2. ANALYSIS AND DESIGN

The determination of impact forces caused by a derailed vehicle on a barrier wall is a very complex problem due to their highly indeterminate nature. The theoretical analysis of determining these forces shall be based on the Train-Barrier Impact Program (TBIP), a two-dimensional computer program as mentioned in Chapter 3 (Study Methodology) of this report, or a similar dynamic model.

The design of the barrier consists of determining the ultimate strength capacity of the various structural members to resist the total ultimate vehicle impact load. Failure mode analysis shall be used for the design of the barrier, refer to Chapter 3 (Study Methodology) of this report. The *yield-line theory* shall be used for concrete walls, and the *plastic theory* for structural steel walls.

Concrete walls: The total ultimate moment capacity of a free-standing concrete barrier wall is a function of the moment capacity of the localized beam at the top of the wall, the moment capacity of the wall below the beam, the cantilever moment capacity of the wall/column at the support, and the moment capacity of the supporting foundation.

Steel walls: The total ultimate moment capacity of a free-standing steel barrier wall is a function of the moment capacity of all the structural elements that must work together to produce the ultimate strength of the system, namely, the top beam, the posts, the base plate and the foundation. In order to determine the total ultimate vehicle impact load, all possibilities of failure modes shall be considered, including weak beam-strong post and strong beam-strong post systems.

3. DESIGN CONSIDERATIONS

Since derailment scenarios vary greatly by nature, the manner in which a vehicle impacts the wall varies accordingly. Rotational effects should be analyzed and additional consideration given to vertical that may result on the wall. These forces can be significant in the case of a structural steel beam and post system since weak-axis bending or biaxial bending may occur. The use of tubes and/or pipes may be the optimum structural members because of their strength in two directions.

In addition to resisting the horizontal impact forces normal to the barrier, the wall must also resist the resulting horizontal longitudinal forces as follows:

$$F_L = m\ F_N$$

where: F_L = Longitudinal impact force on barrier

m = Static coefficient of friction: 0.40 concrete to concrete, 0.25 concrete to steel

F_N = Normal impact force on barrier

The barrier wall finished surface shall be smooth and flush with the columns and at joints to prevent the vehicle from snagging or entangling with the barrier causing higher forces than predicted.

Elevated Structures:

The design of barrier walls located along the edges of a concrete deck slab on an elevated guideway structure must also take into account the rotation and capacity of the deck slab. If the deck slab is weak, it may control or limit the cantilever moment capacity. However, the yield-line and the plastic theories indicate that the total load capacity of the wall can be increased by strengthening the beam and wall/post by adding more longitudinal reinforcement or increasing the size, which in turn will increase the critical length of failure and engage more deck area. Nevertheless, the assumption that the deck can be reinforced if necessary to develop the strength of the barrier should be checked by taking into account the capacity of the deck.

4. DETAILS OF REINFORCEMENT

The lateral impact forces on the barrier wall tend to bend the wall into a dished shape surface in two directions, horizontally between supports and vertically as a cantilever at support points (wall/column or post). As a result, the wall must be reinforced in both directions. Torsion should also be considered and the reinforcement or connections properly detailed.

Since the yield-line theory takes into account the inelastic behavior of the concrete section (but the design of these sections are done using moment capacities based on ultimate strength method which are found by elastic analysis), it is recognized that the inelastic design is not consistent, although it is assumed safe and conservative. In order to achieve a failure mechanism or formation of plastic hinges, the concrete section must be able to rotate and deform considerably. Therefore, the sections should be lightly reinforced in order to achieve yielding of the reinforcement and avoid crushing of the concrete. Reinforcement limits should be confined to approximately 50 percent of maximum values allowed by ACI 318. In general, good practice will be to use smaller size bars at a smaller spacing, rather than large size bars at a larger spacing.

Since the impact force from a derailed vehicle occurs near the top of the wall, vertical reinforcement in that area should extend as far as cover will permit and bend around to ensure continuity and development of the reinforcement in tension.

Longitudinal reinforcement in columns, wall-columns, beams, and deep foundations shall be enclosed by spirals and hoops (or stirrups for beams) extending at least 6 feet beyond the developed length in tension, and spaced not more than 4 to 6 inches on center with a minimum spiral steel (or stirrup) of 1/2 inch diameter. The ends of stirrups (hook part or lap) must overlap by the least dimension of the member or anchored outside the impacted layer.

Where a column merges into a foundation pile or caisson, the reinforcement shall be extended into the foundation far enough to achieve at least a lap splice with the foundation reinforcement. Judgment shall be used in determining the extent; however, the intent is to be rather conservative on the detailing with the foundation system in order to insure a stronger foundation than the wall/column/post system and achieve fixity as well as ductility between the two members during plastic hinge formation.

In summary, care shall be exercised in reinforcing and detailing members and their connections to ensure continuity, ductility, and linkage of all members acting together, with the intent of providing a monolithically behaved and stable barrier system.

Chapter E

FOUNDATIONS

1. GENERAL

Design of foundations shall be in accordance with "AASHTO Bridges" for Bridges, and "ACI 318-89" for concrete.

The Types of foundations include:

- Spread Footing
- Drilled Caissons
- Piles: concrete or steel

Foundation capacity and lateral resistance of deep foundations are to be determined in conjunction with a study of the in-situ subsurface conditions at the proposed site performed under the direction of a qualified Geotechnical Engineer. Sufficient soil borings shall be taken to allow the analysis of results and development of parameters for the foundation design.

A deep foundation shall be used when a shallow foundation cannot be designed to carry the applied loads safely and economically. It shall also be used where scour, erosion, or settlement may occur. and the soil conditions permit its use, even though the bearing capacity of the soil is sufficient to make practical the use of shallow foundations.

2. DEEP FOUNDATIONS

a. Design Allowance for Installation Tolerance

Design should allow for an accidental construction misplacement of the center of gravity of the foundation, the lesser of three inches or 5% of the caisson/pile diameter in any direction.

b. Lateral Resistance

Primary consideration shall be given to the ability of piles or drilled caissons to resist lateral loads. A Geotechnical Engineer shall be consulted to determine the point of fixity below grade, and the caisson/pile designed as a reinforced concrete column to develop the required capacity to resist lateral loads in bending. The reinforcing steel shall be continuous and shall extend sufficiently below the plane where the soil provides adequate lateral restraint.

c. Procedures and Sequence of Installation

Any limitations on construction operations inherent in the design considerations and assumptions shall be noted on the contract drawings and referenced in the special provisions. These are especially important along existing structures or shared rights-of-way.

d. Drilled Caissons

Caissons shall include those members constructed with or without a temporary steel casing, removed during concreting operations, under slurry, or alternative methods of temporary ground support. Minimum reinforcement in caissons shall be as specified in "ACI 318-89" Chapters 10.8 and 10.9 and its commentary.

e. Piles

Steel or concrete piles shall have adequate capacity to accommodate driving stresses.
Pile splices are not recommended. However, when absolutely necessary and upon approval by the engineer, they shall be adequate to develop the full driving capacity and ultimate moment capacity of the pile. The web and flanges of steel piles shall be spliced by full penetration butt welds.

Piles subject to uplift shall be provided with adequate anchorage, such as studs welded to the pile, or reinforcement passed through the section to resist the design uplift load. The bond between the H-pile steel surface and the surrounding concrete shall not be included when evaluating uplift capacity. The factor of safety against uplift shall be 1.25.

3. SPREAD FOOTING AND PILE FOOTING

Analysis and design shall conform to "AASHTO Bridges," the AREA Manual, and as modified herein. It is recognized that compliance with the criteria specified above may result in undesirably thin footings. To ensure adequate footing thickness, the minimum thickness of footing to support a barrier structure shall be determined from the following formula:

$D \geq 2 + L/6$

Where: D = For spread footing: Thickness of concrete in feet, from top of footing to bottom of footing.

D = For pile or Drilled Caisson Footing: Thickness of concrete, in feet, from top of pile cap to top of pile or drilled caisson.

L = Horizontal distance, in feet, from face of wall at top of footing, to adjacent edge of footing.

In no case, however, shall the total concrete thickness of a footing be less than 2'-6" for a spread footing or less than 3'-0" for a pile or drilled caisson footing.

Bottom of footing shall not be less than 4'-0" below finish subgrade.

APPENDIX C - SIMULATION RESULTS

The results of the numerous TBIP analyses are summarized in the attached tables.

The first analyses were run to determine the affect of various parameters on resulting impact force. A parametric study was undertaken using the ICE vehicle to help understand the affect of vehicle speed at derailment, car and barrier stiffness, ground and barrier friction, barrier location, braking coefficient, coupler properties, number of cars in a consist, initial derailing angle, and number of barriers. The TBIP program was run for various permutations of these parameters, and the results are tabulated in the following tables for the "ICE" vehicle. This information is also shown graphically in Figures 3-8 through 3-15. The parametric study results are described in detail in Section 3.1.3.1, along with an explanation of some of the observed trends.

With the insight gained in the parametric study, additional runs were made for the remaining scenarios and other vehicles. Numerous analyses were made to determine maximum forces for each scenario. The results of these runs are included in the attached tables for the "TGV," "Maglev," and "Freight" vehicles.

The forces calculated as described above, and shown in the attached tables were then used to design the intrusion barriers. The methodology used to design the barriers is described in Section 4.1. The designs are shown in Figures 4-7 through 4-31.

ICE TRAIN

TABLE 1. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE SPEED

NO. OF CARS	SPEED (MPH)	BARRIER OFFSET DISTANCE		
		12 FEET	14 FEET	16 FEET
11	50	180.1	119.4	245.6
	75	189.4	246.6	281.6
	100	206.3	192.9	230
	150	139.2	169.4	245.6
	*200	94	139.2	260.9
12	50	180	210	245.6
	75	185.6	238.5	290
	100	236.4	261.7	316.6
	150	177.5	183.6	240.3
	*200	128.6	114.8	213.9
14	50	200.3	167.9	268.7
	75	270.9	305.3	315.5
	100	209.8	286.8	302.8
	150	186.8	195.2	269.3
	*200	194.3	218	170.3

% DIFFERENCE FROM BASE RUN		
92	-14	-6
101	77	8
119	39	-12
48	22	-6
0	0	0
40	83	15
44	108	36
84	128	48
38	60	12
0	0	0
3	-23	58
39	40	85
8	32	78
-4	-10	58
0	0	0

a) Initial Derailing Angle = 0.05 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 280/170 kips/ft
d) Ground Friction = 1.00

* Base Run

ICE TRAIN

TABLE 2. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE POWER/COACH STIFFNESS

NO. OF CARS	POWER/COACH STIFFNESS (KIPS/FT)	BARRIER OFFSET DISTANCE			% DIFFERENCE FROM BASE RUN		
		12 FEET	14 FEET	16 FEET			
11	*280/170	94	139.2	260.9	0	0	0
	500/300	125.9	174.7	336.1	34	26	29
	1000/600	184.4	240.9	451.2	96	73	73
	5000/3000	421.8	525.4	927.2	349	277	255
	10000/6000	568.5	740.4	1287.2	505	432	393
12	*280/170	128.6	114.8	231.9	0	0	0
	500/300	176.8	184.8	288.8	37	61	25
	1000/600	268.9	251	380	109	119	64
	5000/3000	581.7	532.8	788.1	352	364	240
	10000/6000	825.8	745.1	1098.7	542	549	374
14	*280/170	194.3	218	170.3	0	0	0
	500/300	257.5	272.6	271.7	33	25	60
	1000/600	357.1	380	358.4	84	74	110
	5000/3000	765.2	797	834.6	294	266	390
	10000/6000	1078.3	1127.2	1220.4	455	417	617

a) Speed = 200 rnph
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Ground Friction = 1.00

• **Base Run**

ICE TRAIN

TABLE 3. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE GROUND FRICTION

NO. OF CARS	GROUND FRICTION	BARRIER OFFSET DISTANCE			% DIFFERENCE FROM BASE RUN		
		12 FEET	14 FEET	16 FEET	12 FEET	14 FEET	16 FEET
11	0.25	23.6	23.6	20.8	-75	-83	-92
	0.5	69.1	79	78.3	-26	-43	-70
	*1.00	94	139.2	260.9	0	0	0
	1.5	217.9	226.6	308.9	132	63	18
	2	303.1	270	371.2	222	94	42
12	0.25	30.5	27.2	31.7	-76	-76	-86
	0.5	78.7	67.2	111.4	-39	-41	-52
	*1.00	128.6	114.8	231.9	0	0	0
	1.5	218.1	320.2	277.4	70	179	20
	2	293	349.4	366.1	128	204	58
14	0.25	38.2	40	29	-80	-82	-83
	0.5	96.2	98	83.8	-50	-55	-51
	*1.00	194.3	218	170.3	0	0	0
	1.5	298.9	266.7	323.9	54	22	90
	2	368.8	377.4	427	90	73	151

a) Speed = 200 mph
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 280/170 kips/ft

*** Base Run**

ICE TRAIN

TABLE 4. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE BRAKING FRICTION COEFFICIENT

SPEED MPH	BRAKING FRICTION COEFFICIENT (%)	BARRIER OFFSET DISTANCE		
		12 FEET	14 FEET	16 FEET
100	*5.5	209.8	286.2	302.8
	5.8	208.3	306.5	311
	7.2	229.1	264.1	286.1
*200	*5.5	194.3	218.7	170.2
	5.8	193	214.3	170.6
	7.2	187.6	210.7	177.6

a) Number of Cars = 14
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 280/170 kips/ft

• Base Run

ICE TRAIN

TABLE 5. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE OFFSET DISTANCE
(GROUND FRICTION = 1.00)

BARRIER OFFSET DISTANCE (FT)	SPEED (MPH)	
	100	***200**
8	153	124.7
9	239**	
10	184.7	181.2
12	209.8	194.3
14	286.8	218
16	302.8	170.3
18	367.1	221.6
20	418.3	296.4
24	421.1	180.9
28	430.2	165.5
32	366	92.9
40	259.3	0

a) Number of Cars = 14
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 280/170 kips/ft
e) Ground Friction = 1.00

*** Base Run**
**** 75 MPH**

ICE ***TRAIN***

TABLE 6. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE COUPLER MOMENT COEFFICIENT

TYPE OF COUPLER (COUPLER MOM. COEFF.)	SPEED (MPH)	
	100	200
EE (-0.415, 0.985, -0.336)	296.5	219.1
*EF (-0.702, 1.670. -0.720)	286.8	218
FF (-1.529, 3.837, -2.048)	282.6	221.5

a) Number of Cars = 14
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 2801170 kips/ft
e) Ground Friction = 1.00
f) Barrier Offset Distance = 14 ft

* Base Run

ICE TRAIN

TABLE 7. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE OFFSET DISTANCE

NO. OF CARS	GROUND FRICTION	BARRIER OFFSET DISTANCE		
		12 FEET	14 FEET	16 FEET
11	1.5	299.4	303.5	387.7
12	1.5	294.4	323.6	408.7
14	1.5	398.3	473.7	444.9

a) Speed = 75 mph
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 2801170 kipslft

ICE ***TRAIN***

TABLE 8. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE OFFSET DISTANCE
(GROUND FRICTION = 0.50)

BARRIER OFFSET DISTANCE (FT)	SPEED (MPH)	
	100	200
8	99.7	88.9
10	150.5	72.9
12	130.8	96.2
14	163.4	98
16	180.6	83.8
18	194.7	126.5
20	202.2	135.9
24	205.4	57
28	147.4	0
32	127.4	0
40	0	0

a) Number of Cars = 14
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 280/170 kips/ft
e) Ground Friction = 0.50

ICE TRAIN

TABLE 9. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE TRIPLE BARRIER OFFSET DISTANCE
(GROUND FRICTION = **1.00)**

TRIPLE BARRIER OFFSET DISTANCE (FT, FT)	SPEED (MPH)				
	50	75	100	150	*200
8, 8	103.8	114.4	119.3	140.1	127.2
10, 10	128.6	221	211.8	234.6	187.2
12, 12	225.4	253.4	316.6	384	304.9
14, 14	296.5	375.3	510.8	689.2	598.4
16, 16	349.1	539	615.1	1302.3	752.3
18, 18	330.3	593.6	838.4	2044.4	1455.4
20, 20	476.2	760.4	1445.7	2600	2194.7
24, 24	581.8	1342.1	3122	3159.5	3508.1
28, 28	928.5	2175.4	2942.1	2690.6	1724.6
32, 32	980.8	1863.3	2180.3	1621.9	289.6
40, 40	354	418.6	348.7	90.9	0

a) Number of Cars = 14
b) Initial Derailing Angle = 0.05 rad
c) Vehicle-Barrier Friction = 0.40
d) Power/Coach Stiffness = 2801170 kipslfl
e) Ground Friction = 1.oo

*** Base Run**

ICE TRAIN

TABLE 10. MAXIMUM IMPACT FORCE (KIPS)

DUAL BARRIER VARIABLE OFFSET DISTANCE

BARRIER DISTANCE	SPEED (MPH)				
NEAR, FAR (FT, FT)	50	75	100	150	200
8, 23	275.5	374.5	537.6	471.4	210.6
9, 24			874.6	654.9	
10, 25	349.9	673.9	874.5	982.2	225.5
12 , 27	432.6	675.8	1076.7	1238.4	335.6
14, 29	519	745.6	1256.2	1560.9	375
16, 31	580.1	1309.9	2241.7	1487.3	495.5
18, 33	848.9	1564.7	2631.9	1991.8	839
20, 35	995.8	1731.3	2541.9	2027.7	1049.9
24, 39	911.9	1920.4	2138.6	1900.7	712.2

TGV TRAIN

TABLE I - MAXIMUM IMPACT FORCE (KIPS)

VARIABLE OFFSET DISTANCE AND SPEED

NUMBER OF CARS	BARRIER OFFSET DISTANCE	SPEED (MPH)					
		50	75	100	150	*200	320
10	8	49.6			65.6		
	10	74.4	94.6	81.6	78	56.96	56.2
	12	100.1	159.9	131.37	95.8	104.53	30.6
	14	117	125.9	155.57	137.2	115.37	44.9
	16	146.9	202.5	157.7	119	84.87	13.2
	18	185.1	223.4	171.65	110	96.4	9.8
	20	207.8	177.8	180.56	103.9	71.39	0
	24	205.9	155.1	120.89	72	41.86	0
	28	101.6	121.7	56.2	0	0	0
	32	0	6	0		0	0
	40	0	0	0		0	0
12	8	54.6	92.5	101	81.5	75.8	
	9		89.4	121.8			
	10	92.5	128.8	117.7	117.1	92.88	101
	12	115.1	166.7	132.14	115.5	95.67	95.9
	14	126	168.6	165.59	125.3	125.67	40
	16	180.5	220	169.09	112.7	99.74	55.7
	18	189.3	217.2	169.53	93.8	77.36	25.7
	20	205.2	242	165.08	107.4	92.52	36.6
	24	220.4	231.2	182.6	74.2	42.96	32.9
	28	146.4	170.1	176.14	47.5	0	0
	32	0	68.4	108.78	0	0	0
	40		0	0		0	0

a) Initial Derailing Angle = 0.05 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 2451103 kips/ft
d) Ground Friction = 1.00

*** Base Run**

TGV TRAIN

TABLE 2. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE DUAL BARRIER OFFSET DISTANCE AND SPEED
(15 FT TRACK SPACING)

NUMBER OF CARS	NEAR BARRIER DISTANCE	SPEED (MPH)				
		50	75	100	150	200
10	8	54.48	165.31	178.42	68	50.63
	10	69.59	151.88	253.76	78	56.96
	12	104.58	321.87	262.17	96	104.53
	14	134.66	287.65	237.7	137	115.37
	16	132.27	333.32	330.75	119	84.87
	18	134.39	223.4	270.16	111	96.4
	20	139.38	177.8	180.56	104	71.39
	24	102.05	155.1	120.89	72	41.86
	28	0	121.7	56.2	0	0
	32	0	6	0	0	0
	40	0	0	0		0
12	8	99.54	354.04	479.35	145	68.53
	9			492		
	10	93.57	407.22	536.84	293	92.88
	12	165.74	413.62	602.26	237	95.67
	14	183.63	472.53	803.14	323	125.67
	16	165.15	591.47	775.18	279	99.74
	18	169.17	478.34	585.03	220	77.36
	20	151.96	249.71	587.78	181	92.52
	24	152.7	231.2	198.76	74	42.96
	28	85.18	170.1	176.14	48	0
	32	0	68.4	108.78	0	0
	40	0	0	0		0

a) Initial Derailing Angle = 0.05 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 245/103 kips/ft
d) Ground Friction = 1.00

TGV ***TRAIN***

TABLE 3. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE TRIPLE BARRIER OFFSET DISTANCE

(GROUND FRICTION = **1.00)**

NO. OF CARS	TRIPLE BARRIER OFFSET DISTANCE (FT, FT)	SPEED (MPH)				
		50	75	100	150	*200
10	8	58	71	86	65	57
	10	89	107	180	78	57
	12	139	232	268	157	105
	14	164	260	497	166	115
	16	195	455	445	129	85
	18	185	223	270	111	96
	20	286	633	650	104	71
	24	206	360	207	72	42
	28	102	122	56	0	0
	32	0	6	0		
	40		0			
12	8	67	85	95	82	76
	10	106	156	196	177	100
	12	158	247	295	232	149
	14	182	366	512	347	153
	16	255	511	834	286	135
	18	189	478	585	220	77
	20	374	812	961	365	100
	24	343	548	639	147	43
	28	146	204	477.9	116	0
	32	0	68	108.8	54	
	40		0	0	0	

a) Initial Derailing Angle = 0.05 rad

b) Vehicle-Barrier Friction = 0.40

c) Power/Coach Stiffness = 245/103 kips/ft

d) Ground Friction = 1.00

• Base Run

MAGLEV TRAIN

TABLE 1. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE SINGLE BARRIER OFFSET DISTANCE AND SPEED
(16.7 FT TRACK SPACING)

NUMBER OF CARS	BARRIER DISTANCE	SPEED (MPH)					
		50	75	100	150	200	300
6	11	35	24	46	26	28	21
	12	37	83	41	37	39	18
	14	62	82	67	45	33	17
	16	82	100	84	41	25	21
	18	92	117	101	42	43	20
	20	105	141	87	27	21	10
	24	128	143	91	0	0	0
	28	129	128	58	0		
	32	0	76	0			
	40	0	0				
8	11	113	164	102	31	28	28
	12	106	159	108	68	49	38
	14	155	191	129	68	59	34
	16	194	203	137	104	50	31
	18	250	249	169	84	38	34
	20	224	216	147	94	54	27
	24	247	252	169	59	36	0
	28	184	215	150	63	0	
	32	204	163	102	0		
	40	0	0	0			

a) Initial Derailing Angle = 0.05 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 1701170 kips/ft
d) Ground Friction = 1.oo

MAGLEV TRAIN

TABLE 2. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE DUAL BARRIER OFFSET DISTANCE AND SPEED

(16.7 FT TRACK SPACING)

NUMBER OF CARS	NEAR BARRIER DISTANCE	SPEED (MPH)					
		50	75	100	150	200	300
6	11	25.27	23.63	46.5	25.95	27.6	21.1
	12	37.24	83.27	41.3	37.49	38.9	18.4
	14	61.96	81.51	67.4	44.97	33.1	16.9
	16	82.32	99.55	84.5	41.13	20.5	20.7
	18	91.59	117.34	101.1	42.24	0	20.3
	20	104.63	141.35	86.6	27.11		9.9
	24	128.13	142.54	91.2	0		0
	28	128.72	127.81	58.1	0		
	32	0	76.34	0			
	40		0				
8	11	112.89	215.38	101.6	30.731	28.1	28
	12	105.5	258.17	108.3	67.54	48.6	38.2
	14	154.58	287.81	129.2	67.551	58.9	33.6
	16	193.58	360.31	136.5	103.709	50.1	31
	18	250.28	328.36	169.4	83.526	37.6	33.6
	20	223.9	321.24	147.2	83.584	54.5	26.8
	24	247.05	252.23	168.9	58.544	36.3	0
	28	183.69	215.48	150.1	63.472	0	
	32	0	162.77	101.6	0		
	40	0	0	0			

a) Initial Derailing Angle = 0.05 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 1701170 kipslfl
d) Ground Friction = 1.00

UNIFORM FREIGHT TRAIN

TABLE I - MAXIMUM IMPACT FORCE (KIPS)

VARIABLE SINGLE BARRIER OFFSET DISTANCE AND SPEED

(15 FT TRACK SPACING)

NUMBER OF CARS	BARRIER OFFSET DISTANCE	SPEED (MPH)			
		35	55	65	80
61	8	428	995	888	822
	9		829		
	10	356	882	655	709
	12	525	1643	1001	762
	14	342	853	804	692
	16	339	542	848	1007
	18	346	422	839	995
	20	446	469	834	890
	24	350	722	1034	864
	28	572	520	969	711
	32	438	466	902	543
	40	0	348	787	507

a) Initial Derailing Angle = 0.02
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 120 kips/ft
d) Ground Friction = 1.00

UNIFORM FREIGHT TRAIN

TABLE 2. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE DUAL BARRIER OFFSET DISTANCE AND SPEED
(15 FT TRACK SPACING)

NUMBER OF CARS	NEAR BARRIER DISTANCE	SPEED (MPH)			
		35	55	65	80
61	8	2170	2341	2248	2063
	9			2117	
	10	1673	1904	1883	1922
	12	1410	2043	1644	1562
	14	1247	1397	1368	1329
	16	1056	1052	1066	1072
	18	830	813	792	801
	20	563	602	541	589
	24	371	473	509	433
	28	182	232	372	275
	32	0	51	215	0
	40	0	0	0	0

a) **Initial Derailing Angle** = **0.02**
b) **Vehicle-Barrier Friction** = **0.40**
c) **Power/Coach Stiffness** = **120 kips/ft**
d) **Ground Friction** = **1.00**

MIXED FREIGHT TRAIN

TABLE 1. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE SINGLE BARRIER OFFSET DISTANCE AND SPEED
(15 FT TRACK SPACING)

NUMBER OF CARS	BARRIER OFFSET DISTANCE	SPEED (MPH)			
		35	55	65	80
18	8	188	214	230	189
	9		241		
	10	213	284	243	264
	12	330	335	281	278
	14	265	338	327	293
	16	286	335	267	316
	18	313	325	313	292
	20	290	355	314	275
	24	338	240	474	261
	28	359	244	326	301
	32	280	102	265	286
	40	272	0	0	0

a) Initial Derailing Angle = 0.02 rad
b) Vehicle-Barrier Friction = 0.40
c) Power/Coach Stiffness = 120 kipslfl
d) Ground Friction = 1.00

MIXED FREIGHT TRAIN

TABLE 2. MAXIMUM IMPACT FORCE (KIPS)

VARIABLE DUAL BARRIER OFFSET DISTANCE AND SPEED
(15 FT TRACK SPACING)

NUMBER OF CARS	NEAR BARRIER DISTANCE	SPEED (MPH)			
		35	55	65	80
18	8	556	1215	1639	1894
	9				1929
	10	832	1481	1313	1918
	12	1008	1505	1330	1774
	14	855	955	1082	1312
	16	557	719	1158	1136
	18	440	844	760	949
	20	450	583	542	762
	24	395	384	324	407
	28	234	180	98	131
	32	0	0	0	0
	40	0	0	0	0

a) Initial Derailing Angle = 0.02 rad
b) Vehicle-Bamer Friction = 0.40
c) Power/Coach Stiffness = 120 kips/ft
d) Ground Friction = 1.00

APPENDIX D - DESIGN CALCULATIONS

Calculation Design Criteria Summary

A. PROPOSED

These calculations cover the design of a structural rigid barrier wall built to withstand the impact loads generated by a derailed high-speed train colliding with the wall at speeds ranging from 50 mph to 300 mph. The calculations include various types of barrier wall structures which consist of the following structural elements:

> Longitudinal rectangular wall located parallel to the tracks or guideway structure; either a concrete wall with columns as required, or a structural steel beam and post system with a solid steel plate on the track side.
>
> Foundation which supports the wall structure and consists of the following: Precast concrete pile, steel HP pile, drilled concrete caisson, steel pipe pile, and retaining wall footing.

Since the primary load is a lateral impact load acting near the top of the barrier wall, this load is transferred by the wall/beam in bending to the column/foundation which in turn transfers it to the surrounding soil. Therefore, stability of the complete barrier structure is relied on the later (rather than vertical) resistance of the soil and is governed by either the yield strength of the foundation or by the passive resistance of the soil.

The structure design of the barrier structure, as outlined in the ***Interim*** *Study* ***Report*** under "Prototype Design" and mode analysis, i.e., yield-line theory for concrete, and plastic theory for steel. For additional information, refer to the above cited report.

B. LOADING

The barrier impact loads are given by TTI (Texas Transportation Institute) and are obtained from a two-dimensional computer program called TBIP (Train-Barrier Impact Program). The impact loads are a function of many parameters (see report) primarily dependent on the vehicle physical properties and characteristics, as well as the number of barriers (single or double), barrier distance form tracks, barrier stiffness, and barrier/vehicle friction.

C. CODES (See Criteria)

Since this effort is a feasibility study and not site specific, the A.R.E.A. Manual, AASHTO and ACI & AISC codes and standards will be used as appropriate.

D. DESIGN CASES/BARRIER TYPES

Three cases are considered with alternate designs developed for each case.

Case I: Train Barrier At-Grade - Free Standing Wall Barrier

- AG1 - Precast Concrete Barrier & Foundation
- AG2 - Precast Concrete Barrier & Steel Foundation
- AG3 - Cast In Place Concrete Barrier & Foundation
- AG4 - Structural Steel Barrier & Foundation
- AG5 - Retaining Wall Barrier

Case II: Train Barrier on Elevated Structure

- ELI - Concrete Parapet Wall Barrier
- EL2 - Cast In Place Concrete Parapet Wall Barrier
- EL3 - Structural Steel Barrier

Case III: Highway Barriers

- HAG1 - At Grade Cast In Place Concrete Barrier - Van Truck
- HAG2 - At Grade Cast In Place Concrete Barrier - Tank Truck
- HEL1 - Elevated Cast In Place Concrete Barrier - Van Truck
- HEL2 - Elevated Cast In Place Concrete Barrier - Tank Truck

E. SOIL DATA

It is intended that designs be representative of normal soil conditions. The following values (representative of average soil) are assumed for cohesionless and cohesive soils:

Allowable Bearing Capacity	q_a	= 4.0 ksf
Cohesion Strength	c	= 1.50 ksf
Average Effective Soil Unit Weight	g	= 110 pcf
Angle of Internal Friction	f	= 3 0

F. DESIGN ASSUMPTION/PHILOSOPHY

Since train behavior during derailment is extremely' complex and variable, the theoretical impact loads obtained from the TBIP program are approximate at best. Therefore, certain design assumptions are necessary with the objective of simplifying the calculations and not exercising great accuracy and detail that is normally associated with the typical design calculations. This design philosophy is justified since this design effort is a feasibility study with the overall purpose of developing a barrier design that will not collapse under train impact. The following assumptions and design considerations are made:

1. Impact load (F,) is applied 6" below the top of the barrier wall even though the actual hard point (floor level) is at least 1' below the top of the wall.

2. Longitudinal component (F,) if impact load is ignored since it is a secondary load and wall is stiff enough longitudinally.

3. Vertical component (F_u) of impact load: The barrier structure is assumed to have adequate strength in this direction to resist the load; however, the design will be checked for the case of the steel barrier.

4. Three-dimensional effects must be accounted for and added to the impact load. A value of 20% of impact load is assumed reasonable to cover uncertainties associated w/3D.

5. The wall structure is designed primarily for flexure and shear, and is then checked for deflections.

6. Flexural and shear reinforcement is detailed, as well as typical connections and details.

7. Material Strengths:

C.I.P. concrete	f'_c	= 4,000 psi
Precast Concrete	f'_c	= 6,000 psi, (4000 at release)
Reinforcement	f_y	= 60 ksi
Prestressing Steel	f_{pu}	= 270 ksi
Structural Steel	f_y	= 50 ksi (general)
	f_v	= 46 ksi (tube)
	f_v	= 35 ksi (pipe), 36 ksi design

8. Since several train derailment scenarios are considered, the resulting impact loads are grouped and barrier designs developed based on a load schedule and barrier type format. Drawings are generated for each barrier type and design case. Member sizes, reinforcement and details shown on the drawings are tabulated in schedule form to represent each scenario.

G. DESIGN ELEMENTS

Structural design is performed for the following elements:

I. Concrete and Steel Barriers:

1. Overturning analysis to determine barrier height

II. Concrete Barriers (at Grade)

1. Design of longitudinal wall
2. Design of vertical wall/column at supports (foundation)
3. Design of foundation
4. Design of connections

III. Steel Barriers (at Grade)

1. Design of top beam
2. Design of post/column
3. Design of foundation
4. Design of secondary beams
5. Design of plate
6. Design of connections

IV. Concrete Retaining Wall Barrier (at Grade)

1. Design of wall
2. Design of footing

Concrete & Steel Parapet Wall Barrier (Elevated)

1. Design of cantilever wall (concrete)
2. Design of beam & post (steel)
3. Design of connections

Sample Design Calculations:

Barrier Type AG1 (At Grade Alternate 1)

Precast Concrete Wall with Precast Concrete Foundation

FOUNDATION DESIGN - AGI

CHART NUMBER	COLUMN SIZE (IN)	SQUARE PILES		REQ'D Mc	N MAX	Mn MAX	N ACTUAL	Mn ACTUAL
		SIZE (IN)	TYPE					
AG1 - 1B	16.00	20	SOLID	266	19.37	**493.33**	12	**305.69**
AG1 - 1C	18.00	22	SOLID	316	23.43	**656.63**	16	**448.35**
AG1 - 1D	24.00	30	HC	463	43.57	**1530.00**	20	**784.89**
AG1 - 2B	20.00	24	SOLID	523	27.89	**852.48**	20	**611.39**
AG1 - 2C	24.00	30	HC	673	43.57	**1530.00**	20	**784.89**
AG1 - 2D	24.00	30	HC	529	43.57	**1530.00**	20	**784.89**
AG1 - 3B	24.00	30	HC	869	43.57	**1530.00**	24	**941.87**
AG1 - 3C	28.00	36	HC	1560	62.75	**2643.84**	36	**1695.36**
AG1 - 3D	28.00	36	HC	1305	62.75	**2643.84**	36	**1695.36**
AG1 - 4B	32.00	36	HC	1870	62.75	**2643.84**	44	**2072.11**
AG1 - 4C	30.00	36	HC	1716	62.75	**2643.84**	40	**1883.74**
AG1 - 4D	32.00	36	HC	2195	62.75	**2643.84**	48	**2260.48**

FOUNDATION EMBEDMENT DESIGN

ALTERNATE AG1

Kp = 3.00	TAN^2 (45+PHI/2)
J = 0.14	DAMPING CONSTANT (sec/ft)
V= VARIES	IMPACT VELOCITY (fps)
LF= 1 + J V	DYNAMIC LOAD FACTOR
c = 1.50	COHESION STRENGTH (ksf)
q = 13.50	SOIL STRENGTH (ksf)
γ = 110.00	AVERAGE EFECTIVE SOIL UNIT WEIGHT (pcf)
ϕ = 30	ANGLE OF INTERNAL FRICTION (degrees)
Mc= VARIES	ULTIMATE MOMENT CAPACITY OF COLUMN
H= VARIES	DISTANCE FROM IMPACT FORCE TO TOP OF GRADE (BARRIER HT -6")

Pdynamic = Mc / H   Pstatic = P dynamic / L.F.

EQUATION FOR EMBEDMENT LENGTH IN COHESIVE SOILS: LE=(P/qB)+SQRT[2(P/qB)^2+(4HP/qB)]

EQUATION FOR THE EMBEDMENT LENGTH IN COHESIONLESS SOILS: LE^3=2P(H+LE)/(UNIT WT)B(Kp)

ALT. #	Mc (FT-K)	H (FT)	P dyn. (KIPS)	Impact Velocity (fps)	L.F.	P static (KIPS)	B WIDTH (FT)	EMBEDMENT LENGTH (FT)				
								Cohesive		Cohesionless		
								REQ'D	PROV.	REQ'D		PROV.
AG1/1B	266	7.50	35	9	2.26	16	1.67	**5.37**	**7.9**	**9.99**	10.0	**13.0**
AG1/1B	266	7.50	35	5	1.70	21	1.67	**6.35**	**9.0**	**11.25**	11.3	**14.4**
AG1/1C	316	7.50	42	9	2.26	19	1.83	**5.63**	**8.2**	**10.32**	10.3	**13.4**
AG1/1C	316	7.50	42	5	1.70	25	1.83	**6.67**	**9.3**	**11.62**	11.6	**14.8**
AG1/1D	463	7.50	62	9	2.26	27	2.50	**5.87**	**8.5**	**10.62**	10.6	**13.7**
AG1/1D	463	7.50	62	5	1.70	36	2.50	**6.96**	**9.7**	**11.97**	12.0	**15.2**
AG1/2B	523	7.50	70	8	2.12	33	2.00	**7.50**	**10.3**	**12.61**	12.6	**15.9**
AG1/2B	523	7.50	70	10	2.40	29	2.00	**6.96**	**9.7**	**11.97**	12.0	**15.2**
AG1/2B	523	7.50	70	14	2.96	24	2.00	**6.14**	**8.7**	**10.97**	11.0	**14.1**
AG1/2C	673	7.50	90	8	2.12	42	2.50	**7.64**	**10.4**	**12.77**	12.8	**16.0**
AG1/2C	673	7.50	90	10	2.40	37	2.50	**7.08**	**9.8**	**12.11**	12.1	**15.3**
AG1/2C	673	7.50	90	14	2.96	30	2.50	**6.24**	**8.9**	**11.10**	11.1	**14.2**
AG1/2D	561	7.50	75	8	2.12	35	2.50	**6.84**	**9.5**	**11.82**	11.8	**15.0**
AG1/2D	561	7.50	75	10	2.40	31	2.50	**6.35**	**9.0**	**11.22**	11.2	**14.3**
AG1/2D	561	7.50	75	14	2.96	25	2.50	**5.61**	**8.2**	**10.29**	10.3	**13.3**
AG1/3B	869	7.50	116	36	6.04	19	2.50	**4.78**	**7.3**	**9.19**	9.2	**12.1**
AG1/3C	1560	7.50	208	36	6.04	34	3.00	**6.04**	**8.6**	**10.86**	10.9	**13.9**
AG1/3D	1305	7.50	174	36	6.04	29	3.00	**5.44**	**8.0**	**10.06**	10.0	**13.1**
AG1/4B	1870	8.50	220	4	1.56	141	3.00	**15.43**	**19.0**	**20.12**	20.1	**24.1**
AG1/4B	1870	8.50	220	35	5.90	37	3.00	**6.67**	**9.3**	**11.46**	11.5	**14.6**
AG1/4C	1716	8.50	202	4	1.56	129	3.00	**14.56**	**18.0**	**19.39**	19.4	**23.3**
AG1/4C	1716	8.50	202	35	5.90	34	3.00	**6.34**	**9.0**	**11.07**	11.1	**14.2**
AG1/4D	2195	8.50	258	4	1.56	166	3.00	**17.22**	**20.9**	**21.59**	21.6	**25.8**
AG1/4D	1295	8.50	152	35	5.90	26	3.00	**5.38**	**7.9**	**9.86**	9.9	**12.9**

TABLE AG1 - 1A

ULTIMATE MOMENT CAPACITIES FOR PRECAST CONCRETE BARRIER WALL ON DISCRETE FOUNDATION

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 6	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE) = 4% GROSS AREA FOR COLUMN (1% MINIMUM REINF.)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

CH #	H (IN.)	BEAM Bb (IN.)	BEAM MAX. Asb (IN^2)	BEAM PROV. Asb (IN^2)	BEAM PROV. NO. DIA.	BEAM PROV. SPA. (IN.)	Mb (FT-K)	WALL Bw (IN.)	WALL MAX. Asw (IN^2)	WALL PROV. Asb (IN^2)	WALL PROV. NO. DIA.	WALL PROV. SPA. (IN.)	Mw (FT-K)	COLUMN Bc (IN.)	COLUMN MAX. Asc (IN^2)	COLUMN PROV. Asb (IN^2)	COLUMN PROV. NO. DIA.	COLUMN PROV. SPA. (IN.)	Mc (FT-K)
1B	16	24	4.20	2.00	2 # 9	16.0	**108**	66	11.55	7.62	6 # 10	11.6	**405**	16	10.24	3.16	4 # 8	12.0	**175**
1B	16	24	4.20	1.80	3 # 7	8.0	**98**	66	11.55	9.00	9 # 9	7.3	**474**	16	10.24	4.00	4 # 9	12.0	**217**
1B	16	24	4.20	2.37	3 # 8	8.0	**127**	66	11.55	7.62	6 # 10	11.6	**405**	16	10.24	6.32	8 # 8	6.0	**318**
1B	16	24	4.20	3.81	3 # 10	8.0	**198**	66	11.55	11.43	9 # 10	7.3	**591**	16	10.24	8.00	8 # 9	6.0	**380**
1B	16	24	4.20	4.00	4 # 9	5.3	**207**	66	11.55	12.70	10 # 10	6.4	**650**	16	10.24	10.16	8 # 10	6.0	**446**
1C	18	24	4.87	2.37	3 # 8	8.0	**148**	66	13.40	6.32	8 # 8	8.3	**396**	18	12.96	5.08	4 # 10	14.0	**316**
1C	18	24	4.87	3.16	4 # 8	5.3	**195**	66	13.40	7.90	10 # 8	6.4	**490**	18	12.96	6.24	4 # 11	14.0	**378**
1C	18	24	4.87	3.95	5 # 8	4.0	**241**	66	13.40	7.00	7 # 9	9.7	**437**	18	12.96	8.00	8 # 9	7.0	**464**
1C	18	24	4.87	5.00	5 # 9	4.0	**299**	66	13.40	9.00	9 # 9	7.3	**555**	18	12.96	10.16	8 # 10	7.0	**557**
1C	18	24	4.87	5.08	4 # 10	5.3	**303**	66	13.40	10.00	10 # 9	6.4	**612**	18	12.96	12.48	8 # 11	7.0	**641**
1D	24	24	6.89	3.00	3 # 9	8.0	**267**	66	18.94	7.00	7 # 9	9.7	**626**	24	23.04	4.00	4 # 9	20.0	**369**
1D	24	24	6.89	4.00	4 # 9	5.3	**351**	66	18.94	8.00	8 # 9	8.3	**712**	24	23.04	8.00	8 # 9	10.0	**703**
1D	24	24	6.89	3.81	3 # 10	8.0	**335**	66	18.94	9.00	9 # 9	7.3	**798**	24	23.04	10.16	8 # 10	10.0	**869**
1D	24	24	6.89	5.08	4 # 10	5.3	**440**	66	18.94	10.16	8 # 10	8.3	**896**	24	23.04	12.00	12 # 9	6.7	**1002**
1D	24	24	6.89	6.35	5 # 10	4.0	**541**	66	18.94	11.43	9 # 10	7.3	**1002**	24	23.04	12.48	8 # 11	10.0	**1036**

TABLE AGI - 1C

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS		
INPUT DATA		
F (KIPS)	= 200	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7 00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6"
Mb (FT-K)	= 299	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= 555	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT)
Mc (FT-K)	= 316	ULTIMATE MOMENT CAPACITY OF WALL /COL AT FOUNDATION (VERT)
B (FT)	= 1.80	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 45	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

ODD SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-1)(N+1)McL/H(2NL-Lt) + 4McB/H(2NL-Lt) + 0

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION Pc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
18	1	444	0	10	0	455	45	<	222	
18	2	206	0	5	45	256	45	<	103	
18	3	134	63	3	0	200	45	<	67	
18	4	99	47	2	45	193	45	<	50	
18	5	79	111	2	0	192	45	>	39	Rw<F, N.G.
15	1	551	0	13	0	564	45	<	276	
15	5	95	112	2	0	209	45	<	48	
15	3	162	64	4	0	230	45	<	81	
15	4	120	47	3	45	215	45	<	60	
15	5	95	112	2	0	209	45	<	48	
15	6	79	93	2	45	219	45	>	39	Rw>F,OK
12	1	725	0	17	0	742	45	<	363	
12	2	320	0	8	45	373	45	<	160	
12	3	206	65	5	0	275	45	<	103	
12	4	151	48	4	45	248	45	<	76	
12	5	120	113	3	0	236	45	<	60	
12	6	99	94	2	45	240	45	<	50	
12	7	85	160	2	0	246	45	>	42	Rw>F,OK

Shear Desian

Provide Shear Reinforcement as required in wall, top beam (upper 2'-0" portion of wall), column and foundation for all alternates, use ACI Seismic requirement and performance specifications.

I) At Grade Alternate AG1 (Precast)

$f'_c = 6000 \cdot psi$ $f_y = 60000$ psi

A) Scenarios 1.2.6. 7 F = 200 kio Table AG1-1C (Refer to Spreadsheets and Table 2-1)

1. TOP BEAM $B := 24 \cdot in$ $H := 18 \cdot in$ $d := H - 3.5 \cdot in$ $d = 14.5 \cdot in$

$V_u := 45 \cdot kip$ $P_c := V_u$

$\phi V_c := 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d$

$\phi V_c = 45.8 \cdot kip$

min. shear reinf. $A_v := \frac{50 \cdot B \cdot s}{f_y}$

$A_v := (2 \cdot in) \cdot (0.2 \cdot in^2)$ $S_{max} := \frac{0.4 \, in^4 \cdot f_y}{50 \cdot lbf \cdot B}$

$A_v = 0.4 \cdot in^2$ $S_{max} = 20 \cdot in$

$S_{max} \le \frac{d}{4}$ $\frac{d}{4} = 4 \cdot in$

use **#4 @ 4"** hoops

2. WALL $B := 7.5 \cdot ft - 2 \cdot ft$ $B = 66 \cdot in$ $H := 18 \cdot in$ $d := 14.5 \cdot in$

$V_u := 45 \cdot kip$ same as beam use min.

$S_{max} := \frac{0.4 \cdot in^4 \cdot f_y}{50 \cdot lbf \cdot B}$ $S_{max} = 7.27 \cdot in$

check deep beam requirement $l_n := 15 \cdot in - 1.5 \cdot in$ $d := 7 \cdot in$

$\frac{l_n}{d} = 2$ $2 \le 5$

$Av_{min} := \frac{0.0015 \cdot (18 \cdot in) \cdot (12 \cdot in)}{2 \cdot ft}$ $Av_{min} = 0.16 \cdot \frac{in^2}{ft}$

$S_{max} := \frac{62 \cdot in}{5}$ $S_{max} = 12 \cdot in$

use **#4 @ 4** hoops

3. COLUMN $B = 18\cdot in$ $H = 18\cdot in$ $d = H - 2.5\cdot in$ $d = 15.5\cdot in$

$V_u = 45\cdot kip$; Rely on only two legs out of four

$\phi V_c = 0.85\cdot\frac{\sqrt{lbf}}{in}\cdot(2)\cdot\sqrt{f'_c}\cdot B\cdot d$ $\phi V_c = 37\cdot kip$

$V_s = \frac{V_u - \phi V_c}{0.85}$ $V_s = 9.7\cdot kip$

$s = \frac{0.4\cdot in^2\cdot f_y\cdot d}{V_s}$ $s = 38\cdot in$

$\frac{d}{2} = 7.75\cdot in$

check hoop requirement #4 @ $\frac{d}{4} = 4\cdot in$

use **#4 @ 4"** hoops

4. PILE $B = 22\cdot in$ $H = 22\cdot in$ $d = H - 2.5\cdot in$ $d = 19.5\cdot in$

$V_u = 45\cdot kip$ By inspection minimum

$\phi V_c = 0.85\cdot\frac{\sqrt{lbf}}{in}\cdot(2)\cdot\sqrt{f'_c}\cdot B\cdot d$ $\phi V_c = 56\cdot kip$

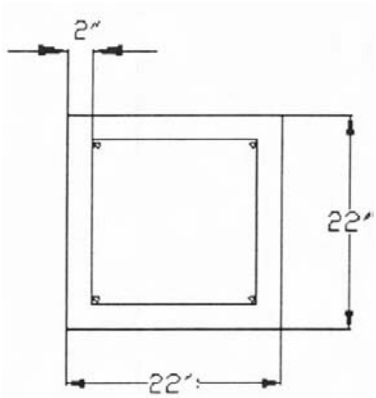
use spirals. Spiral Design:

$\rho_s = 0.45\cdot\left(\frac{A_g}{A_c} - 1\right)\cdot\left(\frac{f'_c}{f_y}\right)$

$A_g = H\cdot B$ $A_g = 484\cdot in^2$

$A_c = (18.5\cdot in)\cdot(18.5\cdot in)$ $A_c = 342\cdot in^2$

$\rho_s = 0.0187$



Try #4 Spiral:

$\rho_s = \frac{V_{sp}}{V_{core}}$

$\rho_s = \frac{18\cdot in\cdot 4\cdot 0.2\cdot in^2}{342.25\cdot in^2\cdot s}$

$s = 2.25\cdot in$ PITCH

use #4 spiral @ 2.25" pitch

Sample Design Calculations:

Barrier Type AG2 (At Grade Alternate 2)

Precast Concrete Wall ***with*** *Steel Pile Foundation*

FOUNDATION EMBEDMENT DESIGN

ALTERNATE AG2

Kp =	3.00	TAN^2 (45+PHI/2)
J =	0.14	DAMPING CONSTANT (sec/ft)
V=	VARIES	IMPACT VELOCITY (fps)
LF=	1 + J V	DYNAMIC LOAD FACTOR
c =	1.50	COHESION STRENGTH (ksf)
q =	13.50	SOIL STRENGTH (ksf)
γ =	110.00	AVERAGE EFECTIVE SOIL UNIT WEIGHT (pcf)
ϕ =	30	ANGLE OF INTERNAL FRICTION (degrees)
Mc=	VARIES	ULTIMATE MOMENT CAPACITY OF COLUMN
H=	VARIES	DISTANCE FROM IMPACT FORCE TO TOP OF GRADE (BARRIER HT -6")

Pdynamic = Mc / H   Pstatic = P dynamic / L.F.

EQUATION FOR EMBEDMENT LENGTH IN COHESIVE SOILS: LE=(P/qB)+SQRT[2(P/qB)^2+(4HP/qB)

EQUATION FOR THE EMBEDMENT LENGTH IN COHESIONLESS SOILS: LE^3=2P(H+LE)I(UNIT WT)B(Kp)

ALT. #	Mc (FT-K)	H (FT)	P dyn. (KIPS)	Impact Velocity (fps)	L.F.	P static (KIPS)	B WIDTH (FT)	EMBEDMENT LENGTH (FT) Cohesive REQ'D	Cohesive PROV.	Cohesionless REQ'D		Cohesionless PROV.
AG2/1B	252	7.50	34	9	2.26	15	0.83	7.89	10.7	13.04	13.0	16.3
AG2/1B	252	7.50	34	5	1.70	20	0.83	9.43	12.4	14.72	14.7	18.2
AG2/1C	308	7.50	41	9	2.26	18	1.00	7.98	10.8	13.16	13.2	16.5
AG2/1C	308	7.50	41	5	1.70	24	1.00	9.54	12.5	14.86	14.9	18.3
AG2/1C	311	7.50	41	9	2.26	18	0.84	8.95	11.8	14.24	14.3	17.7
AG2/1C	311	7.50	41	5	1.70	24	0.84	10.74	13.8	16.07	16.1	19.7
AG2/1D	471	7.50	63	9	2.26	28	0.86	11.56	14.7	16.87	16.9	20.6
AG2/1D	471	7.50	63	5	1.70	37	0.86	13.99	17.4	19.09	19.1	23.0
AG2/2B	546	7.50	73	8	2.12	34	1.09	11.32	14.4	16.62	16.6	20.3
AG2/2B	546	7.50	73	10	2.40	30	1.09	10.44	13.5	15.77	15.8	19.3
AG2/2B	546	7.50	73	14	2.96	25	1.09	9.13	12.0	14.41	14.4	17.8
AG2/2B	542	7.50	72	8	2.12	34	0.86	13.18	16.5	18.38	18.4	22.2
AG2/2B	542	7.50	72	10	2.40	30	0.86	12.13	15.3	17.43	17.5	21.2
AG2/2B	542	7.50	72	14	2.96	24	0.86	10.57	13.6	15.92	16.0	19.5
AG2/2C	608	7.50	81	8	2.12	38	1.23	11.26	14.4	16.58	16.6	20.2
AG2/2C	608	7.50	81	10	2.40	34	1.23	10.39	13.4	15.71	15.7	19.3
AG2/2C	608	7.50	81	14	2.96	27	1.23	9.08	12.0	14.37	14.4	17.8
AG2/2D	438	7.50	58	8	2.12	28	1.02	10.27	13.3	15.60	15.6	19.2
AG2/2D	438	7.50	58	10	2.40	24	1.02	9.48	12.4	14.79	14.8	18.3
AG2/2D	438	7.50	58	14	2.96	20	1.02	8.31	11.1	13.52	13.5	16.9
AG2/3B	808	7.50	108	36	6.04	18	1.24	6.91	9.6	11.91	11.9	15.1
AG2/3C	975	7.50	130	36	6.04	22	1.23	7.81	10.6	12.96	13.0	16.3
AG2/3D	1146	7.50	153	36	6.04	25	1.05	9.54	12.5	14.86	14.9	18.3
AG2/4B	3067	8.50	361	4	1.56	231	1.37	39.60	45.6	35.58	35.6	41.1
AG2/4B	3067	8.50	361	35	5.90	61	1.37	14.87	18.4	19.67	19.7	23.6
AG2/4C	3621	8.50	426	4	1.56	273	1.39	44.82	51.3	38.13	38.1	43.9
AG2/4C	3621	8.50	426	35	5.90	72	1.39	16.50	20.2	21.01	21.0	25.1
AG2/4D	2258	8.50	266	4	1.56	170	1.34	31.67	36.8	31.28	31.3	36.4
AG2/4D	2258	8.50	266	35	5.90	45	1.34	12.32	15.6	17.40	17.4	21.1

TABLE AG2 - 2A

ULTIMATE MOMENT CAPACITIES FOR PRECAST CONCRETE BARRIER WALL ON DISCRETE FOUNDATION

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
Fy (KSI)	= 50	YIELD STRENGTH OF STEEL COLUMNS
F'c (KSI)	= 6	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE) = 4% GROSS AREA FOR COLUMN (2% EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

		BEAM								WALL								COLUMN			
CH #	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	NO.		DIA.	SPA. (IN.)	Mb (FT-K)	Bw (IN.)	MAX. Asw (IN^2)	PROV. Asb (IN^2)	NO.		DIA.	SPA. (IN.)	Mw (FT-K)	SHAPE (HP or W)	Zx (IN^3)	d (IN)	Mc (FT-K)
2B	20	24	5.54	3.00	3	#	9	8.0	**213**	66	15.25	9.00	9	#	9	7.3	**636**				
	20	24	5.54	3.81	3	#	10	8.0	**267**	66	15.25	10.00	10	#	9	6.4	**702**	W10X100	130	11.10	**542**
	20	24	5.54	4.68	3	#	11	8.0	**323**	66	15.25	13.97	11	#	10	5.8	**959**	HP13X87	131	12.95	**546**
	20	24	5.54	5.08	4	#	10	5.3	**349**	66	15.25	14.04	9	#	11	7.3	**963**				
	20	24	5.54	6.24	4	#	11	5.3	**420**	66	15.25	15.60	10	#	11	6.4	**1061**				
2C	24	24	6.89	3.00	3	#	9	8.0	**267**	66	18.94	8.00	8	#	9	8.3	**712**				
	24	24	6.89	3.81	3	#	10	8.0	**335**	66	18.94	9.00	9	#	9	7.3	**798**				
	24	24	6.89	5.08	4	#	10	5.3	**440**	66	18.94	10.00	10	#	9	6.4	**882**	HP14X89	146	13.83	**608**
	24	24	6.89	6.35	5	#	10	4.0	**541**	66	18.94	11.00	11	#	9	5.8	**966**				
	24	24	6.89	7.80	5	#	11	4.0	**652**	66	18.94	13.97	11	#	10	5.8	**1210**				
2D	24	24	6.89	1.80	3	#	7	8.0	**162**	66	18.94	7.20	12	#	7	5.3	**643**				
	24	24	6.89	2.37	3	#	8	8.0	**212**	66	18.94	9.00	9	#	9	7.3	**798**				
	24	24	6.89	3.16	4	#	8	5.3	**280**	66	18.94	9.48	12	#	8	5.3	**838**	HP12X74	105	12.13	**438**
	24	24	6.89	4.00	4	#	9	5.3	**351**	66	18.94	10.00	10	#	9	6.4	**882**				
	24	24	6.89	4.68	3	#	11	8.0	**408**	66	18.94	14.04	9	#	11	7.3	**1216**				

TABLE AG2 - 2B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS

INPUT DATA

F (KIPS)	= 300	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6"
Mb (FT-K)	= 267	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= 702	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 542	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 1.09	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 77	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

ODD SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-1)(N+1)McL/H(2NL-Lt) + 4McB/H(2NL-Lt) + 0

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION Pc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
20	1	446	0	10	0	456	77	<	223	
20	2	208	0	5	77	290	77	<	104	
20	3	136	108	3	0	246	77	>	68	Rw<F, N.G.
16	1	578	0	13	0	591	77	<	289	
16	2	265	0	6	77	348	77	<	132	
16	3	172	109	4	0	284	77	<	86	
16	4	127	81	3	77	288	77	>	63	Rw<F, N.G.
15	1	625	0	14	0	638	77	<	312	
15	2	284	0	6	77	367	77	<	142	
15	3	184	109	4	0	297	77	<	92	
15	4	136	81	3	77	297	77	>	68	Rw<F, N.G.
14	1	679	0	15	0	694	77	<	339	
14	2	306	0	7	77	390	77	<	153	
14	3	198	110	4	0	312	77	<	99	
14	4	146	81	3	77	308	77	>	73	Rw>F,OK
10	1	1041	0	23	0	1064	77	<	521	
10	2	446	0	10	77	533	77	<	223	
10	3	284	113	6	0	403	77	<	142	
10	4	208	83	5	77	373	77	<	104	
10	5	164	196	4	0	364	77	<	82	
10	6	136	162	3	77	378	77	>	68	Rw>F,OK

Shear Design

Provide Shear Reinforcement as required in wall, top beam (upper 2'-0" portion of wall), column and foundation for all alternates, use ACI Seismic requirement and performance specifications.

II) At Grade Alternate AG2 (Precast Wall Steel Column and Pile)

f_c 6000 psi f_y 60000 psi Shear Design for Top Beam and Wall Only

A) Scenarios 1, 2, 6, 7 F = 200 kip Table AG2-1C

1. TOP BEAM B 24 in H 18 in d $H - 3.5$ in $d = 14.5$ in

V_u 44 kip same as AG1

#4 @ 4" Hoops Beam and Wall

2. COLUMN PILE W10X60

$V_u = 41$ kip d 10.22 in t_w 0.42 in

$f_p = \frac{V_u}{d \cdot t_w}$ $f_p = 10.25$ ksi

Fpu_{max} $0.6 \cdot Fpu$ (Horne, Plastic Theory of Stuctures p79)

$Fpu_{max} = 30$ ksi

$f_p < Fpu_{max}$

$0.5 \cdot Fpu = 15$ ksi

$f_p < 0.5 \cdot Fpu$

3. ENCASEMENT STEEL

Encasement steel is provided to ensure ductility and prevent crushing of the concrete. The steel member is designed to resist the total load

18"X18" Encasement

A_g (18 in)·(18 in)

$0.01 \cdot A_g = 3.2$ in^2 8-#6 (3.5 in^2)
Vert:

#4 @ 6" Ties

B) Scenario 3, 5, 8, 10, 11 F=300 kip Table AG2-2B (Refer to Spreadsheets and Table 2-1)

1. TOP BEAM AND WALL

[illegible] 20 in [illegible] kip

same as AG1

#4 @ 4" Hoops

2. COLUMN, PILE W10X100

$V_u = 77 \cdot kip$ $d = 11.1 \cdot in$ $t_w = 0.68 \cdot in$

$f_p = \frac{V_u}{d \cdot t_w}$ $f_p = 10.2 \cdot ksi$

$0.5 \cdot Fpu = 15 \cdot ksi$

$f_p < 0.5 \cdot fpu$

3. ENCASEMENT STEEL

Encasement steel is provided to ensure ductility and prevent crushing of the concrete. The steel member is designed to resist the total load

20"X20" Encasement

$A_g = (20 \cdot in) \cdot (20 \cdot in)$

$0.01 \cdot A_g = 4 \cdot in^2$ 8-#7 ($4.8 \cdot in^2$)

#4 @ 6" Ties

Sample Design Calculations:

Barrier Type AG3 (At Grade Alternate 3)

Precast Concrete Wall with Precast Concrete Foundation

FOUNDATION DESIGN - AG3

CHART NUMBER	COLUMN SIZE (IN)	CAISSON DIAMETER (IN)	REQ'D Mc	As MIN	STEEL # OF BARS	STEEL BAR SIZE	ACTUAL As	CENTER SPACING	ACTUAL Mc
AG3 - 1B	24	30	**235**	7.07	10	# 9	10.00	6.91	**460**
AG3 - 1C	24	30	**330**	7.07	10	# 9	10.00	6.91	**460**
AG3 - 1D	36	40	**299**	12.57	14	# 9	14.00	7.18	**940**
AG3 - 1E	36	40	**466**	12.57	14	# 9	14.00	7.18	**940**
AG3 - 2B	24	30	**380**	7.07	10	# 9	10.00	6.91	**460**
AG3 - 2C	24	30	**511**	7.07	10	# 9	10.00	6.91	**560**
AG3 - 2D	36	40	**653**	12.57	14	# 9	14.00	7.18	**920**
AG3 - 2E	36	40	**533**	12.57	14	# 9	14.00	7.18	**920**
AG3 - 3B	30	36	**1046**	10.18	14	# 11	21.84	6.16	**1180**
AG3 - 3C	30	40	**1243**	12.57	16	# 10	20.32	6.23	**1290**
AG3 - 3D	36	40	**1204**	12.57	16	# 10	20.32	6.23	**1290**
AG3 - 3E	36	40	**1429**	12.57	16	# 11	24.96	6.17	**1540**
AG3 - 4B	36	48	**1878**	18.10	18	# 11	28.08	6.88	**2000**
AG3 - 4C	36	48	**1778**	18.10	18	# 11	28.08	6.88	**2000**
AG3 - 4D	42	48	**2120**	18.10	20	# 11	31.20	6.20	**2200**
AG3 - 4E	42	48	**2165**	18.10	20	# 11	31.20	6.20	**2200**

FOUNDATION EMBEDMENT DESIGN

ALTERNATE AG3

Kp = 3.00	TAN^2 (45+PHI/2)
J = 0.14	DAMPING CONSTANT (sec/ft)
V= VARIES	IMPACT VELOCITY (fps)
LF= 1 + J V	DYNAMIC LOAD FACTOR
c = 1.50	COHESION STRENGTH **(ksf)**
q = 13.50	SOIL STRENGTH **(ksf)**
γ = 110.00	AVERAGE EFECTIVE SOIL UNIT WEIGHT (pcf)
φ = 30	ANGLE OF INTERNAL FRICTION (degrees)
Mc= VARIES	ULTIMATE MOMENT CAPACITY OF COLUMN
H= VARIES	DISTANCE FROM IMPACT FORCE TO TOP OF GRADE (BARRIER HT -6")

Pdynamic = Mc / H　　　Pstatic = P dynamic / L.F.

EQUATION FOR EMBEDMENT LENGTH IN COHESIVE SOILS: LE=(P/qB)+SQRT[2(P/qB)^2+(4HP/qB)]

EQUATION FOR THE EMBEDMENT LENGTH IN COHESIONLESS SOILS: LE^3=2P(H+LE)/(UNIT WT)B(Kp)

ALT. #	Mc (FT-K)	H (FT)	P dyn. (KIPS)	Impact Velocity (fps)	L.F.	P static (KIPS)	B WIDTH (FT)	EMBEDMENT LENGTH (FT) Cohesive REQ'D	Cohesive PROV.	Cohesionless REQ'D		Cohesionless PROV.
AG3/1B	235	7.50	31	9	2.26	14	2.50	**3.97**	**6.4**	**8.06**	8.1	**10.9**
AG3/1B	235	7.50	31	5	1.70	18	2.50	**4.67**	**7.1**	**9.03**	9.0	**11.9**
AG3/1C	330	7.50	44	9	2.26	19	2.50	**4.82**	**7.3**	**9.26**	9.3	**12.2**
AG3/1C	330	7.50	44	5	1.70	26	2.50	**5.68**	**8.3**	**10.39**	10.4	**13.4**
AG3/1D	299	7.50	40	9	2.26	18	3.33	**3.87**	**6.3**	**7.91**	7.9	**10.7**
AG3/1D	299	7.50	40	5	1.70	23	3.33	**4.55**	**7.0**	**8.88**	8.9	**11.8**
AG3/1E	466	7.50	62	9	2.26	27	3.33	**4.98**	**7.5**	**9.47**	9.5	**12.4**
AG3/1E	466	7.50	62	5	1.70	37	3.33	**5.88**	**8.5**	**10.66**	10.7	**13.7**
AG3/2B	380	7.50	51	8	2.12	24	2.50	**5.42**	**8.0**	**10.07**	10.1	**13.1**
AG3/2B	380	7.50	51	10	2.40	21	2.50	**5.05**	**7.6**	**9.57**	9.6	**12.5**
AG3/2B	380	7.50	51	14	2.96	17	2.50	**4.47**	**6.9**	**8.78**	8.8	**11.7**
AG3/2C	511	7.50	68	8	2.12	32	2.50	**6.46**	**9.1**	**11.38**	11.4	**14.5**
AG3/2C	511	7.50	68	10	2.40	28	2.50	**6.00**	**8.6**	**10.80**	10.8	**13.9**
AG3/2C	511	7.50	68	14	2.96	23	2.50	**5.31**	**7.8**	**9.90**	9.9	**12.9**
AG3/2D	653	7.50	87	8	2.12	41	3.33	**6.31**	**8.9**	**11.18**	11.2	**14.3**
AG3/2D	653	7.50	87	10	2.40	36	3.33	**5.86**	**8.4**	**10.61**	10.6	**13.7**
AG3/2D	653	7.50	87	14	2.96	29	3.33	**5.18**	**7.7**	**9.73**	9.7	**12.7**
AG3/2E	533	7.50	71	8	2.12	34	3.33	**5.59**	**8.2**	**10.28**	10.3	**13.3**
AG3/2E	533	7.50	71	10	2.40	30	3.33	**5.20**	**7.7**	**9.77**	9.8	**12.7**
AG3/2E	533	7.50	71	14	2.96	24	3.33	**4.61**	**7.1**	**8.97**	9.0	**11.9**
AG3/3B	1046	7.50	139	36	6.04	23	3.00	**4.78**	**7.3**	**9.20**	9.2	**12.1**
AG3/3C	1243	7.50	166	36	6.04	27	3.33	**4.98**	**7.5**	**9.47**	9.5	**12.4**
AG3/3D	1204	7.50	161	36	6.04	27	3.33	**4.88**	**7.4**	**9.35**	9.4	**12.3**
AG3/3E	1429	7.50	191	36	6.04	32	3.33	**5.40**	**7.9**	**10.02**	10.0	**13.0**
AG3/4B	1878	8.50	221	4	1.56	142	4.00	**12.77**	**16.0**	**17.80**	17.8	**21.6**
AG3/4B	1878	8.50	221	35	5.90	37	4.00	**5.65**	**8.2**	**10.20**	10.2	**13.2**
AG3/4C	1778	8.50	209	4	1.56	134	4.00	**12.32**	**15.6**	**17.39**	17.4	**21.1**
AG3/4C	1778	8.50	209	35	5.90	35	4.00	**5.47**	**8.0**	**9.98**	10.0	**13.0**
AG3/4D	2120	8.50	249	4	1.56	160	4.00	**13.83**	**17.2**	**18.77**	18.8	**22.6**
AG3/4D	2120	8.50	249	35	5.90	42	4.00	**6.06**	**8.7**	**10.71**	10.7	**13.8**
AG3/4E	2165	8.50	255	4	1.56	163	4.00	**14.03**	**17.4**	**18.95**	19.0	**22.8**
AG3/4E	2165	8.50	255	35	5.90	43	4.00	**6.13**	**8.7**	**10.81**	10.8	**13.9**

TABLE AG3 - 3A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE BARRIER WALL ON DISCRETE FOUNDATION

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE) = 4% GROSS AREA FOR COLUMN (2% EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

CH #	H (IN.)	BEAM Bb (IN.)	BEAM MAX. Asb (IN^2)	BEAM PROV. Asb (IN^2)	BEAM PROV. NO.	BEAM PROV. DIA.	BEAM PROV. SPA. (IN.)	Mb (FT-K)	WALL Bw (IN.)	WALL MAX. Asw (IN^2)	WALL PROV. Asb (IN^2)	WALL PROV. NO.	WALL PROV. DIA.	WALL PROV. SPA. (IN.)	Mw (FT-K)	COLUMN Bc (IN.)	COLUMN MAX. Asc (IN^2)	COLUMN PROV. Asb (IN^2)	COLUMN PROV. NO.	COLUMN PROV. DIA.	COLUMN PROV. SPA. (IN.)	Mc (FT-K)
3B	28	24	6.20	4.68	3	# 11	8.0	459	66	17.06	14.04	9	# 11	7.3	1366	30	16.8	8.89	7	# 10	3.7	876
	28	24	6.20	5.00	5	# 9	4.0	487	66	17.06	15.00	15	# 9	4.1	1451	30	16.8	10.16	8	# 10	3.1	984
	28	24	6.20	5.08	4	# 10	5.3	495	66	17.06	15.24	12	# 10	5.3	1472	30	16.8	10.92	7	# 11	3.7	1046
	28	24	6.20	6.24	4	# 11	5.3	595	66	17.06	14.04	9	# 11	7.3	1366	30	16.8	12.48	8	# 11	3.1	1170
	28	24	6.20	6.35	5	# 10	4.0	605	66	17.06	15.60	10	# 11	6.4	1503	30	16.8	14.04	9	# 11	2.8	1287
3C	32	24	7.26	4.74	6	# 8	3.2	549	66	19.97	12.70	10	# 10	6.4	1475	30	19.2	7.62	6	# 10	4.4	900
	32	24	7.26	5.00	5	# 9	4.0	577	66	19.97	14.04	9	# 11	7.3	1619	30	19.2	7.80	5	# 11	5.5	920
	32	24	7.26	6.24	4	# 11	5.3	708	66	19.97	15.00	15	# 9	4.1	1721	30	19.2	9.36	6	# 11	4.4	1084
	32	24	7.26	6.35	5	# 10	4.0	719	66	19.97	15.24	12	# 10	5.3	1746	30	19.2	10.92	7	# 11	3.7	1243
	32	24	7.26	7.62	6	# 10	3.2	847	66	19.97	15.60	10	# 11	6.4	1784	30	19.2	12.48	8	# 11	3.1	1394
3D	28	24	6.20	4.68	3	# 11	8.0	459	66	17.06	14.04	9	# 11	7.3	1366	36	20.16	6.24	4	# 11	9.3	645
	28	24	6.20	5.00	5	# 9	4.0	487	66	17.06	15.00	15	# 9	4.1	1451	36	20.16	8.89	7	# 10	4.7	893
	28	24	6.20	5.08	4	# 10	5.3	495	66	17.06	15.24	12	# 10	5.3	1472	36	20.16	10.16	8	# 10	4.0	1006
	28	24	6.20	6.24	4	# 11	5.3	595	66	17.06	14.04	9	# 11	7.3	1366	36	20.16	10.92	7	# 11	4.7	1072
	28	24	6.20	6.35	5	# 10	4.0	605	66	17.06	15.60	10	# 11	6.4	1503	36	20.16	12.48	8	# 11	4.0	1204
3E	32	24	7.26	3.00	3	# 9	8.0	356	66	19.97	14.04	9	# 11	7.3	1619	36	23.04	7.62	6	# 10	5.6	913
	32	24	7.26	3.16	4	# 8	5.3	375	66	19.97	15.00	15	# 9	4.1	1721	36	23.04	7.80	5	# 11	7.0	933
	32	24	7.26	3.81	3	# 10	8.0	447	66	19.97	15.24	12	# 10	5.3	1746	36	23.04	9.36	6	# 11	5.6	1104
	32	24	7.26	3.95	5	# 8	4.0	463	66	19.97	15.60	10	# 11	6.4	1784	36	23.04	10.92	7	# 11	4.7	1269
	32	24	7.26	4.00	4	# 9	5.3	469	66	19.97	18.72	12	# 11	5.3	2106	36	23.04	12.48	8	# 11	4.0	1429

TABLE AG3 - 3B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS

INPUT DATA

F (KIPS)	= 600	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY, Rw)
H (FT)	= 7.00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6")
Mb (FT-K)	= 605	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= 1503	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 1046	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 3.00	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 149	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

ODD SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-1)(N+1)McL/H(2NL-Lt) + 4McB/H(2NL-Lt) + 0

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION Pc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
25	1	752	0	40	0	792	149	<	376	
25	2	356	0	19	149	525	149	<	178	
25	3	233	206	12	0	452	149	>	117	Rw<F, N.G.
20	1	967	0	51	0	1018	149	<	483	
20	2	451	0	24	149	625	149	<	226	
20	3	294	208	16	0	518	149	>	147	Rw<F, N.G.
15	1	1354	0	72	0	1425	149	<	677	
15	2	615	0	33	149	797	149	<	308	
15	3	398	211	21	0	630	149	<	199	
15	4	294	156	16	149	615	149	>	147	RW>F,OK
12	1	1781	0	94	0	1875	149	<	891	
12	2	787	0	42	149	978	149	<	393	
12	3	505	214	27	0	746	149	<	253	
12	4	372	158	20	149	699	149	<	186	
12	5	294	374	16	0	684	149	>	147	RW>F,OK
10	1	2256	0	120	0	2376	149	<	1128	
10	2	967	0	51	149	1168	149	<	483	
10	3	615	217	33	0	865	149	<	308	
10	4	451	159	24	149	784	149	<	226	
10	5	356	378	19	0	753	149	<	178	
10	6	294	312	16	149	771	149	>	147	RW>F,OK

Shear Design

Provide Shear Reinforcement as required in wall, top beam (upper 2'-0" portion of wall), column and foundation for all alternates, use ACI Seismic requirement and performance specifications.

III) At Grade Alternate AG3

$f'_c = 4000 \cdot psi$ $f_y = 60000 \cdot psi$

A) Scenarios 1, 2, 6, 7 F = 200 kip Table AG3-1C (Refer to Spreadsheets and Table 2-1)

1 TOP BEAM $B = 24 \cdot in$ $H = 20 \cdot in$ $d = H - 4.5 \cdot in$ $d = 15.5 \cdot in$

$V_u = 47 \cdot kip$ (Beam and Wall)

$\phi V_c = 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d$

$\phi V_c = 40 \cdot kip$

By inspection min #4 @ $\frac{d}{4} = 3.875 \cdot in$ $\frac{d}{4} = 4 \cdot in$

use #4 @ 4" hoops

2. WALL   same   use #4 @ 4" hoops

3. COLUMN $B = 24 \cdot in$ $H = 20 \cdot in$ $d = H - 3.5 \cdot in$ $d = 16.5 \cdot in$

$V_u = 47 \cdot kip$ $\frac{d}{4} = 4.1 \cdot in$

use #4 @ 4" hoops

4. CAISSON $\phi = 30 \cdot in$ $A_g = 706.86 \cdot in^2$

$V_u = 47 \cdot kip$ equivalent Rect. Col. $B = \frac{A_g}{\phi \cdot 0.8}$ $B = 29.45 \cdot in$ $H = 24 \cdot in$

$D_s = \phi - 3 \cdot in - 0.5 \cdot in - \frac{.5 \cdot in}{2} \cdot 2$ $D_s = 21.5 \cdot in$ $\frac{2}{3} D_s = 14.333 \cdot in$

$d_c = \frac{H - \frac{2}{3} D_s}{2}$ $d_c = 4.833 \cdot in$

$d = H - d_c$ $d = 19.17 \cdot in$

$\phi V_c = 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d$ $\phi V_c = 60.7 \cdot kip$ min.

SPIRAL DESIGN

$$P_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\left(\frac{f'_c}{f_y}\right)$$

$A_g = H \cdot B$ $\qquad A_g = 706.86 \cdot in^2$

$A_c = \frac{\pi \cdot (H^2)}{4}$ $\qquad A_c = 452.4 \cdot in^2$

$P_s = 0.0169$

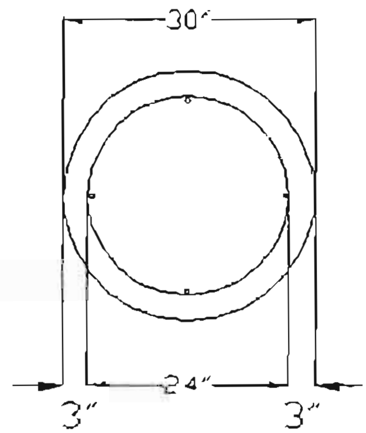
Try #4 Spiral:

$$P_s = \frac{4 \cdot A_s \cdot (H - d_b)}{s \cdot H^2}$$

$$P_s = \frac{(H - 0.5 \cdot in) \cdot 4 \cdot 0.2 \cdot in^2}{H^2 \cdot in^2 \cdot s}$$

$s = 2.0 \cdot in$ PITCH

use <u>#4 spiral @ 2.0"</u> pitch



B) <u>Scenarios 4, 9, 13 F = 1100 kip Table AG3-4B</u> (Refer to Spreadsheets and Table 2-1)

1 TOP BEAM $\quad B = 24 \cdot in \quad H = 40 \cdot in \quad d = H - 4.5 \cdot in \quad d = 35.5 \cdot in$

$V_u = 235 \cdot kip \qquad V_{beam} = 0.33 \cdot V_u \qquad V_{beam} = 78 \cdot kip$

$V_{wall} = 0.67 \cdot V_u \qquad V_{wall} = 157 \cdot kip$

$\phi V_c = 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d \qquad \phi V_c = 92 \cdot kip$

min. $\qquad \frac{d}{4} = 8.88 \cdot in$

use <u>#4 @ 6"</u> hoops

2. WALL $V_u := V_{wall}$ $V_u = 157 \cdot kip$

$B := 78 \cdot in$ $H := 40 \cdot in$ $d := 35.5 \cdot in$

$$s := \frac{0.4 \cdot in^2 \cdot f_y}{50 \cdot lbf \cdot B} \qquad s = 6.2 \cdot in$$

$$\phi V_c := 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f_c} \cdot B \cdot d \qquad \phi V_c = 298 \cdot kip$$

use #4 @ 6" hoops

3. COLUMN $B := 36 \cdot in$ $H := 40 \cdot in$ $d := H - 3.5 \cdot in$ $d = 36.5 \cdot in$

$V_u := 235 \cdot kip$

$$\phi V_c := 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f_c} \cdot B \cdot d \qquad \phi V_c = 141 \cdot kip$$

$V_s := 111 \cdot kip$

#4 $$s := \frac{0.4 \cdot in^2 \cdot f_y \cdot d}{V_s} \qquad s = 7.9 \cdot in$$

$\frac{d}{4} = 9.125 \cdot in$ use 6"

use <u>#4 @ 4"</u> hoops

4. CAISSON $\phi := 48 \cdot in$

Spiral Design $A_g := 1809 \cdot in^2$

$$A_c := \frac{\pi \cdot (H^2)}{4} \qquad A_c = 1257 \cdot in^2$$

$$P_s := 0.45 \cdot \left(\frac{A_g}{A_c} - 1\right) \cdot \left(\frac{f_c}{f_y}\right) \qquad P_s = 0.0092$$

$$P_s = \frac{4 \cdot A_s \cdot (H - d_b)}{s \cdot H^2}$$

$$P_s = \frac{(H - 0.5 \cdot in) \cdot 4 \cdot 0.2 \cdot in^2}{H^2 \cdot in^2 \cdot s}$$

$s := 2.0 \cdot in$ PITCH

use <u>#4 spiral @ 2.0"</u> pitch

Sample Desipn Calculations:

Barrier Type AG4 (At Grade Alternate 4)

Structural Steel Post and Rail with Steel Pile Foundation

FOUNDATION EMBEDMENT DESIGN

ALTERNATE AG4

Kp = 3.00	TAN^2 (45+PHI/2)
J = 0.14	DAMPING CONSTANT **(sedft)**
V= VARIES	IMPACT VELOCITY **(fps)**
LF= 1 + J V	DYNAMIC LOAD FACTOR
c = 1.50	COHESION STRENGTH (ksf)
q = 13.50	SOIL STRENGTH (ksf)
γ = 110.00	AVERAGE EFECTIVE SOIL UNIT WEIGHT (pcf)
ϕ = 30	ANGLE OF INTERNAL FRICTION **(degrees)**
Mc= VARIES	ULTIMATE MOMENT CAPACITY OF COLUMN
H= VARIES	DISTANCE FROM IMPACT FORCE TO TOP OF GRADE (BARRIER HT -6")

Pdynamic = Mc / H   Pstatic = P dynamic / L.F.

EQUATION FOR EMBEDMENT LENGTH IN COHESIVE SOILS: LE=(P/qB)+SQRT[2(P/qB)^2+(4HP/qB)]

EQUATION FOR THE EMBEDMENT LENGTH IN COHESIONLESS SOILS: LE^3=2P(H+LE)/(UNIT WT)B(Kp)

ALT. #	Mc (FT-K)	H (FT)	P dyn. (KIPS)	Impact Velocity (fps)	L.F.	P static (KIPS)	B WIDTH (FT)	EMBEDMENT LENGTH (FT) Cohesive REQ'D	Cohesive PROV.	Cohesionless REQ'D		Cohesionless PROV.
AG4/1A	221	7.50	29	9	2.26	13	1.00	**6.52**	**9.2**	**11.45**	11.5	**14.6**
AG4/1A	221	7.50	29	5	1.70	17	1.00	**7.75**	**10.5**	**12.89**	12.9	**16.2**
AG4/1B	395	7.50	53	9	2.26	23	1.17	**8.46**	**11.3**	**13.69**	13.7	**17.1**
AG4/1B	395	7.50	53	5	1.70	31	1.17	**10.14**	**13.2**	**15.47**	15.5	**19.0**
AG4/1C	670	7.50	89	9	2.26	40	1.50	**10.09**	**13.1**	**15.41**	15.4	**18.9**
AG4/1C	670	7.50	89	5	1.70	53	1.50	**12.15**	**15.4**	**17.42**	17.4	**21.2**
AG4/1D	464	7.50	62	9	2.26	27	1.33	**8.61**	**11.5**	**13.86**	13.9	**17.2**
AG4/1D	464	7.50	62	5	1.70	36	1.33	**10.32**	**13.3**	**15.66**	15.7	**19.2**
AG4/2A	395	7.50	53	8	2.12	25	1.17	**8.81**	**11.7**	**14.07**	14.1	**17.5**
AG4/2A	395	7.50	53	10	2.40	22	1.17	**8.15**	**11.0**	**13.36**	13.4	**16.7**
AG4/2A	395	7.50	53	14	2.96	18	1.17	**7.17**	**9.9**	**12.21**	12.2	**15.4**
AG4/2B	464	7.50	62	8	2.12	29	1.33	**8.96**	**11.9**	**14.25**	14.3	**17.7**
AG4/2B	464	7.50	62	10	2.40	26	1.33	**8.29**	**11.1**	**13.50**	13.5	**16.9**
AG4/2B	464	7.50	62	14	2.96	21	1.33	**7.29**	**10.0**	**12.37**	12.4	**15.6**
AG4/2C	819	7.50	109	8	2.12	52	1.50	**11.99**	**15.2**	**17.28**	17.3	**21.0**
AG4/2C	819	7.50	109	10	2.40	46	1.50	**11.05**	**14.2**	**16.38**	16.4	**20.0**
AG4/2C	819	7.50	109	14	2.96	37	1.50	**9.65**	**12.6**	**14.97**	15.0	**18.5**
AG4/3A	694	8.50	82	36	6.04	14	1.33	**5.91**	**8.5**	**10.55**	10.6	**13.6**
AG4/3B	897	7.50	120	36	6.04	20	1.67	**6.17**	**8.8**	**11.00**	11.0	**14.1**
AG4/3C	1472	7.50	196	36	6.04	32	1.83	**7.86**	**10.6**	**13.01**	13.0	**16.3**
AG4/4A	1349	8.50	159	4	1.56	102	1.67	**18.47**	**22.3**	**22.58**	22.6	**26.8**
AG4/4A	1349	8.50	159	35	5.90	27	1.67	**7.79**	**10.6**	**12.77**	12.8	**16.0**
AG4/4B	1898	8.50	223	4	1.56	143	2.00	**20.68**	**24.7**	**24.21**	24.2	**28.6**
AG4/4B	1898	8.50	223	35	5.90	38	2.00	**8.58**	**11.4**	**13.65**	13.7	**17.0**
AG4/4C	1671	8.50	197	4	1.56	126	2.33	**16.96**	**20.7**	**21.39**	21.4	**25.5**
AG4/4C	1671	8.50	197	35	5.90	33	2.33	**7.24**	**10.0**	**12.15**	12.2	**15.4**

TABLE AG4 - 4B

YIELD LINE EQUATIONS OF STRUCTURAL STEEL BEAM AND POST

INPUT DATA		DESCRIPTION
F (KIPS)	= 1100	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY, Rw)
H (FT)	= 8.00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HT-6")
Mpb (FT-K)	= 1898	PLASTIC MOMENT CAPACITY OF BEAM
Mpc (FT-K)	= 1898	PLASTIC MOMENT CAPACITY
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 237	POST/COLUMN CAPACITY = Mpc/H
L (FT)	= VARIES	POST CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + POST LOAD (B)

Rw = 16(Mpb)/(2NL-Lt) + (Mpc/H)x(N-1)

Z EQUATIONS			
Z EQUATIONS	T= 1.22	Z= D^3/6[1-(1-2t/D)^3]	
24 SCH 80	D= 24.00	Z=	632.77

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	POST LOAD (KIPS) (B)	BARRIER CAPACITY (KIPS) (Rw)	REACTION CHECK Pc (KIPS)	>=	(A)/2 (KIPS)	FINAL CHK
14	1	1321	0	1321	237	<	660	
14	2	596	237	833	237	<	298	
14	3	384	475	859	237	>	192	Rw<F, N.G.
14	4	284	712	996	237	>	142	
10	1	2025	0	2025	237	<	1012	
10	2	868	237	1105	237	<	434	
10	3	552	475	1027	237	<	276	
10	4	405	712	1117	237	>	202	Rw>F, OK
10	5	320	949	1269	237	>	160	
8	1	2761	0	2761	237	<	1381	
8	2	1125	237	1362	237	<	562	
8	3	706	475	1181	237	<	353	
8	4	515	712	1227	237	<	257	
8	5	405	949	1354	237	>	202	Rw>F, OK
8	6	334	1186	1520	237	>	167	
5	1	6075	0	6075	237	<	3037	
5	2	2025	237	2262	237	<	1012	
5	3	1215	475	1689	237	<	607	
5	4	868	712	1580	237	<	434	
5	5	675	949	1624	237	<	337	
5	6	552	1186	1739	237	<	276	
5	7	467	1424	1891	237	>	234	Rw>F, OK
5	8	405	1661	2066	237	>	202	

(IV) AT GRADE ALTERNATE AG4 (Structural Steel)

A) F=200 kip Table AG4-1D 16" dia. X 0.656 $A = 31.6 \cdot in^2$

$V_u = 66 \cdot kip$

$MV_u = \frac{2.0 \cdot V_u}{A}$

$MV_u = 4.2 \cdot ksi$ $MV_u \le 0.5 \cdot Fpu$

$MV_u \le 10.8 \cdot ksi$

B) F=300 kip Table AG4-2C 18" dia. X 0.937 $A = 50.2 \cdot in^2$

$V_u = 117 \cdot kip$

$MV_u = \frac{2.0 \cdot V_u}{A}$

$MV_u = 4.66 \cdot ksi$ OK

C) F=600 kip Table AG4-3C 22" dia. X 1.125 $A = 73.78 \cdot in^2$

$V_u = 210 \cdot kip$

$MV_u = \frac{2.0 \cdot V_u}{A}$

$MV_u = 5.7 \cdot ksi$ OK

D) F=1100 kip Table AG4-4B 24" dia. X 1.218 $A = 87.2 \cdot in^2$

$V_u = 237 \cdot kip$

$MV_u = \frac{2.0 \cdot V_u}{A}$

$MV_u = 5.4 \cdot ksi$ OK

EL3) F=2540 kip Table EL3-4B 36" dia. X 1.125 $A = 123.18 \cdot in^2$

$V_u = 456 \cdot kip$

$MV_u = \frac{2.0 \cdot V_u}{A}$

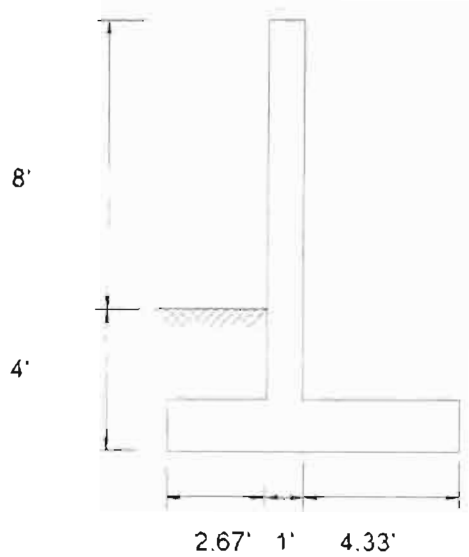
$MV_u = 7.4 \cdot ksi$ OK

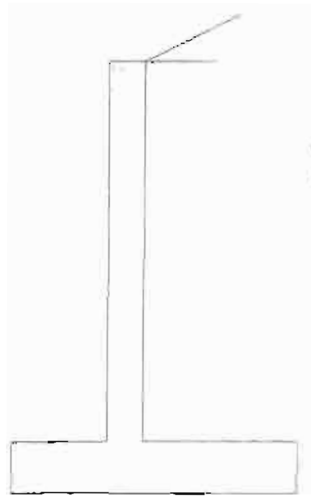
<u>Sample Design Calculations:</u>

Barrier Type AG5 (At Grade Alternate 5)

Cast In Place Concrete Retaining Wall Barrier

VNTSC Intrusion Barrier Study. Retaining Wall Design





SURCHARGE = 100 psf

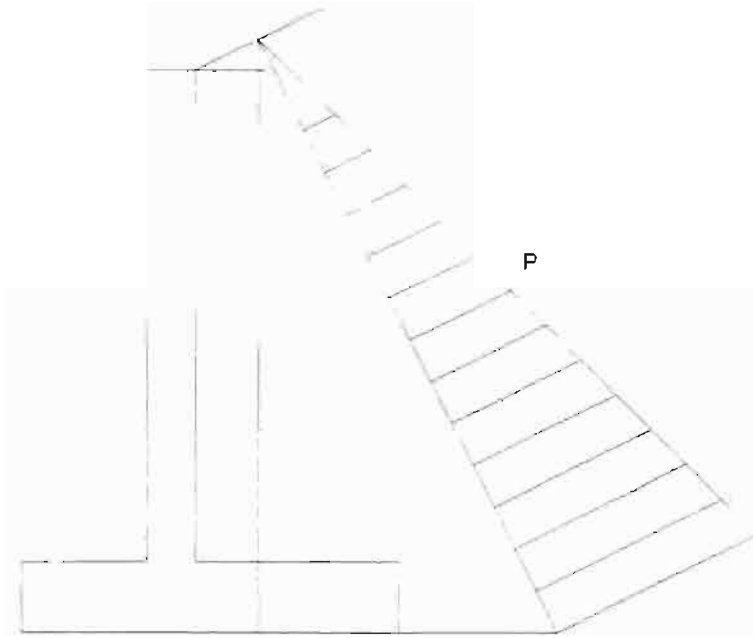
$\alpha = 26.57 \cdot deg$ $\quad Qs = 100 \cdot psf$ $\quad \gamma = 120 \cdot psf$

$\cos(\alpha) = 0.89$ $\quad H = 12 \cdot ft$ $\quad Ka = 0.30$

$\frac{1}{\cos(\alpha)} = 1.118$ $\quad L = 8 \cdot ft$

(flat soil) $\quad Pa = \frac{Ka \cdot \gamma \cdot H^2}{2}$

VNTSC Intrusion Barrier Study: Retaining Wall Design



$$H' = \frac{H}{\cos(\alpha)}$$

$$P' = \frac{1}{2} Ka \cdot \gamma \cdot (H')^2$$

$$P' = \frac{1}{2} Ka \cdot \gamma \cdot \left(\frac{H}{\cos(\alpha)}\right)^2$$

$$P = \cos(\alpha) \cdot P'$$

Horizontal Component of P' = P

P' = sloped soil
P = flat soil

$$P = \cos(\alpha) \cdot \left(\frac{1}{2}\right) \cdot (Ka \cdot \gamma) \cdot \left(\frac{H}{\cos(\alpha)}\right)^2$$

P = 2898.07 ·lbf

$$P = \frac{Ka \cdot \gamma}{2} \cdot \frac{H^2}{\cos(\alpha)^2} \cdot \cos(\alpha)$$

$$P = \frac{Pa}{\cos(\alpha)}$$

P = 2898.07 ·lbf

Or simply adjust the constant Ka, the new multiplying constant

$$Ka' = \frac{Ka}{\cos(\alpha)}$$

Ka' = 0.34

$$P = \frac{Ka' \cdot \gamma \cdot H^2}{2}$$

P = 2898.07 ·lbf

P with flat soil would be 2592 lbs.

SURCHARGE

$$Ps = Qs \cdot H \cdot Ka' \cdot (1 \cdot ft)$$

Ps = 402.51 ·lbf

Check on Footing Length:

$$Wx := ((H \cdot \gamma) + (Qs \cdot 1 \cdot ft))$$

$$Wx = 1540 \cdot psf \cdot ft$$

$$M := P \cdot (4 \cdot ft) + Ps \cdot (6 \cdot ft)$$

$$M = 14007.33 \cdot lbf \cdot ft$$

$$X := \sqrt{\frac{2 \cdot M}{Wx}}$$

$$X = 4.27 \cdot ft$$ OK

OVERTURNING

$$Fh := P + Ps$$

$$Fh = 3301 \cdot lbf$$

$$M := 14004 \cdot lbf \cdot ft$$

RIGHTING

LOADS

$W1 := (150 \cdot pcf) \cdot ((\cdot ft) \cdot (1.5 \cdot ft) \cdot (1 \cdot ft)$	$W1 = 1800 \cdot lbf$
$W2 := (150 \cdot pcf) \cdot (1 \cdot ft) \cdot (11 \cdot ft)\ (I \cdot A)$	$W2 = 1650 \cdot lbf$
$W3 := \gamma \cdot (4.33 \cdot ft) \cdot (11 \cdot ft) + (4.33 \cdot ft) \cdot Qs \cdot (1 \cdot ft)$	$W3 = 6148.6 \cdot lbf$
$W4 := \gamma \cdot (2.5 \cdot ft) \cdot (2.67 \cdot ft)$	$W4 = 801 \cdot lbf$
$Vt := W1 + W2 + W3 + W4$	$Vt = 10399.6 \cdot lbf$

MOMENTS

$M1 = W1 \cdot (4 \cdot ft)$ — $M1 = 7200 \cdot lbf \cdot ft$

$M2 = W2 \cdot (3.17 \cdot ft)$ — $M2 = 5230.5 \cdot lbf \cdot ft$

$M3 = W3 \cdot (5.84 \cdot ft)$ — $M3 = 35907.82 \cdot lbf \cdot ft$

$M4 = W4 \cdot (1.34 \cdot ft)$ — $M4 = 1073.34 \cdot lbf \cdot ft$

$Mt = M1 + M2 + M3 + M4$ — $Mt = 49411.66 \cdot lbf \cdot ft$

F.O.S. for Overturning: 2.0

$$FOS = \frac{Mt}{M'}$$

$FOS = 3.53$ $\quad 3.53 \geq 2.0$ OK

F.O.S. for Sliding: 1.5

$\mu = \tan(32 \cdot deg)$

$\mu = 0.62$ use $\mu = 0.6$

$$FOS = \frac{Vt \cdot \mu}{Fh}$$ $FOS = 1.89$ $\quad 1.89 \geq 1.5$ OK

FOOTING SOIL PRESSURES

$x = \frac{Mt - M'}{Vt}$ — $x = 3.4 \cdot ft$ — $A = L \cdot (1 \cdot ft)$

$e = \frac{L}{2} - x$ — $e = 0.6 \cdot ft$ from the centerline — $s = \frac{(1 \cdot ft) \cdot L^2}{6}$

$M = Vt \cdot e$ — $M = 6190.74 \cdot lbf \cdot ft$

$Qmax = \frac{Vt}{A} + \frac{M}{s}$ — $Qmax = 1880.33 \cdot psf$

$Qmin = \frac{Vt}{A} - \frac{M}{s}$ — $Qmin = 719.57 \cdot psf$

Design of Stem

$h := 10.5 \cdot ft$

$H1 := \frac{Ka' \cdot \gamma \cdot h^2}{2} \cdot (1.7)$ $H1 = 3772.02 \cdot lbf$

$H2 := Ka' \cdot Qs \cdot h \cdot (1.7) \cdot (1 \cdot ft)$ $H2 = 598.73 \cdot lbf$

$Hs := H1 + H2$ $Hs = 4370.75 \cdot lbf$

Moments:

$Mu := H1 \cdot (3.5 \cdot ft) + H2 \cdot (5.25 \cdot ft)$

$Mu = 16.35 \cdot kip \cdot ft$

$b := 12 \cdot in$ $d := (12 \cdot in) - 2 \cdot in - \left(\frac{6}{16} \cdot in\right)$ $d = 9.62 \cdot in$ $f_y := 60000 \cdot psi$ $f'_c := 4000 \cdot psi$

$Ku := \frac{Mu \cdot \left(12000 \cdot \frac{lbf \cdot in}{kip \cdot ft}\right)}{b \cdot d^2}$ $db := \frac{6}{8} \cdot in$

$Ku = 176.44 \cdot psi$ $\rho := 0.0036$

$As := \rho \cdot b \cdot d$

$As = 0.42 \cdot in^2$ #6 @ 12" (soil face)

$l_d := \frac{0.03 \cdot \frac{in}{\sqrt{lbf}} \cdot (db) \cdot f_y}{\sqrt{f'_c}}$ $l_d = 21.35 \cdot in$

$l_d \cdot (1.31) = 27.96 \cdot in$

28.0 in
2'-6" beyond cutoff

Moment 2'-6"
above base of stem

= 8 ft. from surface

CUTOFF

$q_v = Ka' \cdot \gamma \cdot (8 \cdot ft)$ $\qquad q_v = 322.01 \cdot \frac{lbf}{ft}$

$q_h = Ka' \cdot Qs \cdot (1 \cdot ft)$ $\qquad q_h = 33.54 \cdot \frac{lbf}{ft}$

$H1a = \frac{q_v \cdot (8 \cdot ft)}{2} \cdot (1.7)$ $\qquad H1a = 2189.65 \cdot lbf$

$H2a = q_h \cdot (8 \cdot ft) \cdot (1.7)$ $\qquad H2a = 456.18 \cdot lbf$

$Mu = H1a \cdot \left(\frac{8}{3} \cdot ft\right) + H2a \cdot \left(\frac{8}{2} \cdot ft\right)$

$Mu = 7.66 \cdot kip \cdot ft$

$Ku_1 = \frac{Mu \cdot \left[12000 \cdot \left(\frac{lbf \cdot in}{kip \cdot ft}\right)\right]}{b \cdot d^2}$ $\qquad \rho = 0.0016$

$Ku_1 = 82.73 \cdot psi$

$As = \rho \cdot b \cdot d$

$As = 0.18 \cdot in^2$

5 @ 12"
(100% lapped on # 6's)

DEVELOPEMENT LENGTH - - STEM REINF INTO FOOTING

required $l_d = 21.35 \cdot in$

allowable $l_d = (18 \cdot in) - (3 \cdot in) - \left(\frac{6}{8} \cdot in + \frac{5}{8} \cdot in\right)$ $l_d = 13.63 \cdot in$

$13.63 < 21.35$

Cover = 13.63" Therefore use standard hooks

$$Lhb := \frac{\left(1200 \cdot \frac{\sqrt{lbf}}{in}\right) \cdot \left(\frac{6}{8} \cdot in\right)}{\sqrt{f'_c}}$$

$Lhb = 14.23 \cdot in$

$$Ldh := Lhb \cdot 0.7 \cdot \left(\frac{0.433 \cdot in^2}{0.440 \cdot in^2}\right)$$

$Ldh = 9.8 \cdot in$ $9.8 \le 13.6$ OK

Shear in Stem

$Vu := H1 - H2$

$Vu = 4370.75 \cdot lbf$

$$\phi Vc := \left(0.85 \cdot \frac{\sqrt{lbf}}{in}\right) \cdot 2 \cdot \sqrt{f'_c} \cdot b \cdot d$$

$\phi Vc = 12.42 \cdot kip$ 12.42 kip is less than 4.4 kip OK

Temp and Shrinkage

$$Ash := \frac{0.002 \cdot b \cdot h}{2 \cdot ft}$$

$Ash = 0.14 \cdot \frac{in^2}{ft}$ Ea. Face # 4 @ 12"

Shear at Cutoff Point

ACI requires shear at cutoff point to be less than or equal to $\frac{2}{3}\phi V_c$

$$\frac{2}{3}\phi V_c = 8.28\cdot kip$$

AND:

$$8.28 \geq 4.37$$

There is no need to worry about Vu at cutoff point which will be less than Vu at base

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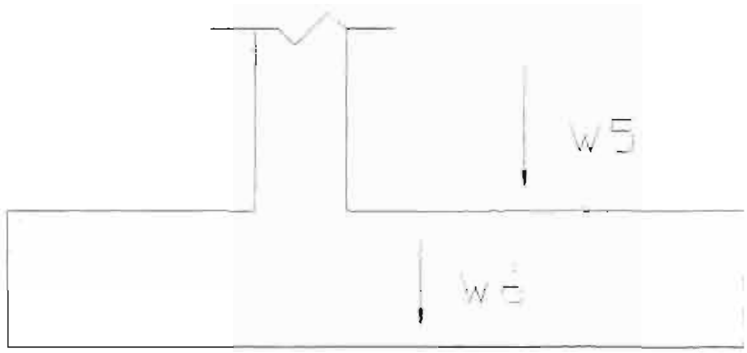
Design of Heel



$$W5 = W3$$

$$W5 = 6148.6\cdot lbf$$

$$W6 = (150\ psf)\cdot(1.5\ ft)\cdot(4.33\ ft)$$

$$W6 = 974.25\cdot lbf$$

$$Wt = W5 + W6$$

$$Wt = 7122.85\cdot lbf$$

$Q_{max} = H1a$ $\qquad Q_{max} = 2189.65\cdot lbf$

$Q_{min} = H2a$ $\qquad Q_{min} = 456.18\cdot lbf$

54% across from Qmin

$$Q = 899.0\cdot lbf$$

$Q_{ave} = \dfrac{Q + Q_{min}}{2}$ $\qquad Q_{ave} = 677.59\cdot lbf$

$Q_{ave}\cdot(4.33\cdot ft) = 2933.96\cdot lbf\cdot ft$ $\qquad 2933.96 \le 7123$ $\quad$ OK

Temp and Shrinkage in Heel

$\dfrac{0.0018\cdot(12\cdot in)\cdot(18\cdot in)}{2} = 0.194\cdot in^2$ $\quad$ use 0.20 $\qquad$ # 5 @ 12"

Top and Bottom Longitudinal

Neglect upward soil pressure and use a load factor or 1.4 for shear and moment since soil and concrete make up the load

$Shear := Wt \cdot (1.4)$

$Shear = 9.97 \cdot kip$

$\phi Vc = 12.42 \cdot kip$ $\quad 12.42 \geq 10$ $\quad$ OK

Moment:

$Mu := ([illegible]1 \cdot kip) \cdot \left(\frac{3}{2} \cdot ft\right)$ $\quad Mu = 9.6 \cdot kip \cdot ft$

$$Ku := \frac{Mu \, [illegible] \, 12000 \cdot \frac{lbf \cdot in}{kip \cdot ft}}{b \cdot d^2}$$

$Ku = 103.63 \cdot psi$ $\quad \rho := 0.0020$

$As := 0.0020 \cdot (12 \cdot in) \cdot (9.5 \cdot in)$ $\quad As = 0.23 \cdot in^2$

$As_{min} := 0.0018 \cdot (12 \cdot in) \cdot (12 \cdot in)$ $\quad As_{min} = 0.26 \cdot in^2$

5 @ 12" NO BENDS

TABLE AG5 - 3A

ULTIMATE MOMENT CAPACITIES FOR CONCRETE BARRIER WALL

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF AREA: MAX. LIMIT IS FOR DUCTILITY 0.011Bd FOR BEAM AND WALL (EACH FACE), 0.0033Bd FOR COLUMN
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

		BEAM						WALL							COLUMN					
	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. SPA. (IN.)	Mb (FT-K)	Bw (IN.)	MAX. Asw (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. DIA.	PROV. SPA. (IN.)	Mw (FT-K)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. SPA. (IN.)	Mc (FT-K)
A	18	24	3.56	3.56			**195**	108	16.04	16.04				**880**	12	0.58	0.58			**36**
B	18	24	3.56	1.80	3 # 7	8.0	**104**	108	16.04	4.84	11 #	6	10.0	**285**	12	0.58	0.44	# 6 @	12	**28**
C	18	24	3.56	1.80	3 # 7	8.0	**104**	108	16.04	5.40	9 #	7	12.5	**317**	12	0.58	0.44	# 6 @	12	**28**
D	18	24	3.56	1.80	3 # 7	8.0	**104**	108	16.04	6.60	11 #	7	10.0	**385**	12	0.58	0.44	# 6 @	12	**28**
E	18	24	3.56	1.80	3 # 7	8.0	**104**	108	16.04	7.11	9 #	8	12.5	**413**	12	0.58	0.44	# 6 @	12	**28**
F	18	24	3.56	2.37	3 # 8	8.0	**135**	108	16.04	4.84	11 #	6	10.0	**285**	12	0.58	0.44	# 6 @	12	**28**
G	18	24	3.56	2.37	3 # 8	8.0	**135**	108	16.04	5.40	9 #	7	12.5	**317**	12	0.58	0.44	# 6 @	12	**28**
H	18	24	3.56	2.37	3 # 8	8.0	**135**	108	16.04	6.60	11 #	7	10.0	**385**	12	0.58	0.44	# 6 @	12	**28**
I	18	24	3.56	2.37	3 # 8	8.0	**135**	108	16.04	7.11	9 #	8	12.5	**413**	12	0.58	0.44	# 6 @	12	**28**
J	18	24	3.56	3.00	3 # 9	8.0	**167**	108	16.04	4.84	11 #	6	10.0	**285**	12	0.58	0.44	# 6 @	12	**28**
K	18	24	3.56	3.00	3 # 9	8.0	**167**	108	16.04	5.40	9 #	7	12.5	**317**	12	0.58	0.44	# 6 @	12	**28**
L	18	24	3.56	3.00	3 # 9	8.0	**167**	108	16.04	6.60	11 #	7	10.0	**385**	12	0.58	0.44	# 6 @	12	**28**
M	18	24	3.56	3.00	3 # 9	8.0	**167**	108	16.04	7.11	9 #	8	12.5	**413**	12	0.58	0.44	# 6 @	12	**28**

TABLE AG5 - 3B

YIELD LINE EQUATIONS OF CONCRETE WALL

INPUT DATA		DESCRIPTION
F (KIPS)	= 1100	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 10.50	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K/')	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD ≈ Rw/Wc

CRITICAL LENGTH (L) = Lt/2 + SQR [$(Lt/2)^2$ + 8H(Mb + Mw)/Mc]

BARRIER CAPACITY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)

= [16(Mb+Mw)/(2L-Lt)] + $2McL^2$/H(2L-Lt)

	Mb (FT-K)	Mw (FT-K)	Mc (FT-K/')	L (FT)	BEAM LOAD (KIPS) (A)	WALL LOAD (KIPS) (B)	BARRIER CAPACITY Rw (KIPS)	Lr (FT)	Bc (FT)	END CHECK EWc (KIPS)	>	(A)/2 (KIPS)	Pr * L (KIPS)	SOIL & BARRIER CAPACITY	CAP. CHK
A	195	880	36	52.3	173	191	364	104.6	26.2	91	>	86	1349	1712	OK
B	104	285	28	36.7	91	105	196	73.5	18.4	49	>	46	947	1143	OK
C	104	317	28	38.1	95	109	204	76.2	19.0	51	>	47	982	1186	OK
D	104	385	28	40.8	102	116	218	81.7	20.4	55	>	51	1053	1271	OK
E	104	413	28	41.9	105	119	224	83.9	21.0	56	>	52	1081	1305	OK
F	135	285	28	38.0	95	109	203	76.1	19.0	51	>	47	981	1184	OK
G	135	317	28	39.4	98	112	210	78.7	19.7	53	>	49	1015	1225	OK
H	135	385	28	42.0	105	119	225	84.0	21.0	56	>	53	1083	1308	OK
I	135	413	28	43.1	108	122	230	86.1	21.5	58	>	54	1111	1341	OK
J	167	285	28	39.4	98	112	211	78.8	19.7	53	>	49	1016	1226	OK
K	167	317	28	40.7	102	116	217	81.3	20.3	54	>	51	1048	1266	OK
L	167	385	28	143.2	108	123	231	86.5	21.6	58	>	54	1115	1346	OK
M	167	413	28	144.31	111	125	237	88.5	22.1	59	>	56	1141	1378	OK

V) AT GRADE ALTERNATE AG5   (Retaining Wall)

A) Scenarios 6, 7, 8, 10, 11   F = 300 kip   Table AG5-2B (Case C)

1. TOP BEAM $B = 24 \cdot in$ $H = 12 \cdot in$ $d = H - 4.5\,in$ $d = 7.5 \cdot in$

$V_u = 16 \cdot kip$ $V_{beam} = 0.22 \cdot V_u$ $V_{beam} = 4 \cdot kip$

$$\phi V_c = 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d \qquad \phi V_c = 19 \cdot kip$$

use   #4 @ 6"   hoops max

B) Scenario 9   F = 1100 kip   Table AG5-3B (Case C)

1. TOP BEAM $B = 24 \cdot in$ $H = 18 \cdot in$ $d = H - 4.5\,in$ $d = 13.5 \cdot in$

$V_u = 51 \cdot kip$ $V_{beam} = 0.25 \cdot V_u$ $V_{beam} = 13 \cdot kip$

$$\phi V_c = 0.85 \cdot \frac{\sqrt{lbf}}{in} \cdot (2) \cdot \sqrt{f'_c} \cdot B \cdot d \qquad \phi V_c = 35 \cdot kip$$

use   #4 @ 6"   hoops

Sample Design Calculations:

Barrier Type EL1 (Elevated Alternate 1)

Precast Concrete Wall

TABLE ELI - 1A

ULTIMATE MOMENT CAPACITIES FOR PRECAST CONCRETE BARRIER WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 6	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM. WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.014Bd FOR BEAM AND WALL (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

	H (IN.)	BEAM								WALL								COLUMN					
		Bb (IN.)	MAX. Asb (IN^2)	PROV.					Mb (FT-K)	Bw (IN.)	MAX. Asw (IN^2)	PROV.					Mw (FT-K)	Bc (IN.)	MAX. Asc (IN^2)	PROV.			Mc (FT-K)
				Asb (IN^2)	NO.		DIA.	SPA. (IN.)				Asb (IN^2)	NO.		DIA.	SPA. (IN.)				Asb (IN^2)	NO.	SPA. (IN.)	
A	12	24	2.86	2.86					100	72	8.57	8.57					301	12	1.60	1.60			63
B	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.20	# 7 @	6	48
C	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.19	# 8 @	8	48
D	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.20	# 9 @	10	48
E	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.27	# 10 @	12	51
F	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.20	# 7 @	6	48
G	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.19	# 8 @	8	48
H	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.20	# 9 @	10	48
I	12	24	2.86	2.40	4	#	7	5.3	85	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.27	# 10 @	12	51
J	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.20	# 7 @	6	48
K	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.19	# 8 @	8	48
L	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.20	# 9 @	10	48
M	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	7.90	10	#	8	7.1	279	12	1.60	1.27	# 10 @	12	51
N	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.20	# 7 @	6	48
O	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.19	# 8 @	8	48
P	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.20	# 9 @	10	48
Q	12	24	2.86	2.37	3	#	8	8.0	84	72	8.57	8.00	8	#	9	9.1	282	12	1.60	1.27	# 10 @	12	51

TABLE EL1 - 1B

YIELD LINE EQUATIONS OF CONCRETE WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
F (KIPS)	= 300	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.50	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K/')	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD = Rw/Wc

CRITICAL LENGTH (L) = Lt/2 + SQR [(Lt/2)² + 8H(Mb + Mw)/Mc]

BARRIER CAPACITY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)

= [16(Mb+Mw)/(2L-Lt)] + 2McL²/H(2L-Lt)

	Mb (FT-K)	Mw (FT-K)	Mc (FT-K/')	L (FT)	BEAM LOAD (KIPS) (A)	WALL LOAD (KIPS) (B)	BARRIER CAPACITY Rw (KIPS)	Lr (FT)	Bc (FT)	END CHECK EWc (KIPS)	>	(A)/2 (KIPS)	CAP. CHK
A	100	301	63	22.2	163	210	372	44.5	11.1	93	>	81	OK
B	85	279	48'	24.0	136	172	308	47.9	12.0	77	>	68	OK
C	85	279	48	24.1	135	170	306	48.2	12.0	76	>	68	OK
D	85	279	48	24.0	136	172	308	47.9	12.0	77	>	68	OK
E	85	279	51	23.4	139	177	317	46.8	11.7	79	>	70	OK
F	85	282	48	24.1	136	172	309	48.1	12.0	77	>	68	OK
G	85	282	48	24.2	136	171	307	48.4	12.1	77	>	68	OK
H	85	282	48	24.1	136	172	309	48.1	12.0	77	>	68	OK
I	85	282	51	23.5	140	178	318	47.0	11.8	80	>	70	OK
J	84	279	48	23.9	136	172	307	47.9	12.0	77	>	68	OK
K	84	279	48	24.1	135	170	305	48.1	12.0	76	>	67	OK
L	84	279	48	23.9	136	172	307	47.9	12.0	77	>	68	OK
M	84	279	51	23.4	139	177	316	46.8	11.7	79	>	70	OK
N	84	282	48	24.0	136	172	308	48.1	12.0	77	>	68	OK
O	84	282	48	24.2	136	171	306	48.3	12.1	77	>	68	OK
P	84	282	48	24.0	136	172	308	48.1	12.0	77	>	68	OK
Q	84	282	51	23.5	140	178	318	47.0	11.7	79	>	70	OK

1. Vertical Bars Outside Face

Provide 50% of vertical bars inside face for ductility

Scenerio	F (kip	Inside Face	Outside Face
		EL1	EL1
14	300	#7 @ 6"	#7 @ 12"
15	600	#9 @ 6"	#9 @ 12"
16	1100	#10 @8"	#9 @ 12"
17, 18	2540	#10 @5"	#10 @ 10"

2. Shear Desian

Top Beam: B=24"

Scenario	F (kip	Table	EL1								
		EL1	H (in)	D (in)	Vu (kip)	Vbeam (kip)	Φ Vc (kip)		Vu (kip)	Vs (kip)	Smax (in)
14	300	1A(B)	12	8.5	77	18	27	<	18	0	4
15	600	2A(J)	16	12.5	160	32	40	<	32	0	6
16	1100	3A(F)	24	20.5	282	80	65	<	80	18 min	6
17, 18	2540	4A(C)	40	36.5	641	111	115	>	111	0	6

Spacing lesser of dl2 or 6" for ductility

Sample Design Calculations:

Barrier Type EL2 (Elevated Alternate 2)

Cast In Place Concrete Wall

TABLE EL2 - 2A

ULTIMATE MOMENT CAPACITIFS FOR C.I.P. CONCRETE BARRIER WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

$M = 0.9AsFy(d-a/2)$, $a = AsFy/(0.85F'cB)$

		BEAM								WALL								COLUMN					
	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	NO.		DIA.	SPA. (IN.)	Mb (FT-K)	Bw (IN.)	MAX. Asw (IN^2)	PROV. Asb (IN^2)	NO.		DIA.	SPA. (IN.)	Mw (FT-K)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	NO.	SPA. (IN.)	Mc (FT-K)
A	18	24	3.56	3.56					**195**	72	10.69	10.69					**586**	12	1.91	1.91			**113**
B	18	24	3.56	2.40	4	#	7	5.3	**136**	72	10.69	10.27	13	#	8	5.3	**566**	12	1.91	1.91	# 10 @	8	**112**
C	18	24	3.56	2.40	4	#	7	5.3	**136**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.58	# 8 @	6	**95**
D	18	24	3.56	2.40	4	#	7	5.3	**136**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.91	# 10 @	8	**112**
E	18	24	3.56	2.40	4	#	7	5.3	**136**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.58	# 8 @	6	**95**
F	18	24	3.56	2.40	4	#	7	5.3	**136**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.91	# 10 @	8	**112**
G	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.27	13	#	8	5.3	**566**	12	1.91	1.58	# 8 @	6	**95**
H	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.27	13	#	8	5.3	**566**	12	1.91	1.91	# 10 @	8	**112**
I	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.58	# 8 @	6	**95**
J	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.91	# 10 @	8	**112**
K	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.58	# 8 @	6	**95**
L	18	24	3.56	3.16	4	#	8	5.3	**175**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.91	# 10 @	8	**112**
M	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.27	13	#	8	5.3	**566**	12	1.91	1.58	# 8 @	6	**95**
N	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.27	13	#	8	5.3	**566**	12	1.91	1.91	# 10 @	8	**112**
O	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.58	# 8 @	6	**95**
P	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.00	10	#	9	7.1	**552**	12	1.91	1.91	# 10 @	8	**112**
Q	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.58	# 8 @	6	**95**
R	18	24	3.56	3.00	3	#	9	8.0	**167**	72	10.69	10.16	8	#	10	9.1	**560**	12	1.91	1.91	# 10 @	8	**112**

TABLE **EL2 - 2B**

YIELD LINE EQUATIONS OF CONCRETE WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
F (KIPS)	**= 600**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.50	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K/')	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD = Rw/Wc

CRITICAL LENGTH (L) = Lt/2 + SQR [(Lt/2)² + 8H(Mb + Mw)/Mc]

BARRIER CAPACITY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)

= [16(Mb+Mw)/(2L-Lt)] + 2McL²/H(2L-Lt)

	Mb (FT-K)	Mw (FT-K)	Mc (FT-K/')	L (FT)	BEAM LOAD (KIPS) (A)	WALL LOAD (KIPS) (B)	BARRIER CAPACITY Rw (KIPS)	Lr (FT)	Bc (FT)	END EWc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
A	195	586	113	23.1	304	388	**692**	46.1	11.5	**173**	>	**152**	**OK**
B	136	566	112	22.0	288	372	**660**	44.1	11.0	**165**	>	**144**	**OK**
C	136	552	95,	23.5	262	333	**595**	47.0	11.8	**149**	>	**131**	**NG**
D	136	552	112	21.8	285	369	**654**	43.7	10.9	**164**	>	**142**	**OK**
E	136	560	95	23.6	264	334	**598**	47.3	11.8	**149**	>	**132**	**NG**
F	136	560	112	22.0	286	371	**657**	43.9	11.0	**164**	>	**143**	**OK**
G	175	566	95	24.3	272	342	**614**	48.6	12.1	**154**	>	**136**	**OK**
H	175	566	112	22.6	296	380	**675**	45.1	11.3	**169**	>	**148**	**OK**
I	175	552	95	24.1	270	340	**610**	48.2	12.1	**152**	>	**135**	**OK**
J	175	552	112	22.4	293	377	**670**	44.8	11.2	**168**	>	**146**	**OK**
K	175	560	95	24.2	271	341	**612**	48.4	12.1	**153**	>	**135**	**OK**
L	175	560	112	22.5	295	379	**673**	45.0	11.2	**168**	>	**147**	**OK**
M	167	566	95	24.2	271	341	**612**	48.4	12.1	**153**	>	**135**	**OK**
N	167	566	112	22.4	294	378	**672**	44.9	11.2	**168**	>	**147**	**OK**
O	167	552	95	24.0	268	339	**607**	48.0	12.0	**152**	>	**134**	**OK**
P	167	552	112	22.3	291	376	**667**	44.5	11.1	**167**	>	**146**	**OK**
Q	167	560	95	24.1	269	340	**610**	48.2	12.1	**152**	>	**135**	**OK**
R	167	560	112	22.4	293	377	**670**	44.8	11.2	**168**	>	**146**	**OK**

TABLE EL2 - 1A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE BARRIER WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

	H (IN.)	BEAM Bb (IN.)	BEAM MAX. Asb (IN^2)	BEAM PROV. Asb (IN^2)	BEAM PROV. NO.		BEAM PROV. DIA.	BEAM PROV. SPA. (IN.)	Mb (FT-K)	WALL Bw (IN.)	WALL MAX. Asw (IN^2)	WALL PROV. Asb (IN^2)	WALL PROV. NO.		WALL PROV. DIA.	WALL PROV. SPA. (IN.)	Mw (FT-K)	COLUMN Bc (IN.)	COLUMN MAX. Asc (IN^2)	COLUMN PROV. Asb (IN^2)	COLUMN PROV. NO.	COLUMN PROV. SPA. (IN.)	Mc (FT-K)
A	14	24	2.51	2.51					97	72	7.52	7.52					290	12	1.39	1.39			59
B	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.20	# 7 @	6	52
C	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.19	# 8 @	8	51
D	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.20	# 9 @	10	52
E	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.27	# 10 @	12	55
F	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.20	# 7 @	6	52
G	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.19	# 8 @	8	51
H	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.20	# 9 @	10	52
I	14	24	2.51	2.40	4	#	7	5.3	93	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.27	# 10 @	12	55
J	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.20	# 7 @	6	52
K	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.19	# 8 @	8	51
L	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.20	# 9 @	10	52
M	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.11	9	#	8	8.0	276	12	1.39	1.27	# 10 @	12	55
N	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.20	# 7 @	6	52
O	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.19	# 8 @	8	51
P	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.20	# 9 @	10	52
Q	14	24	2.51	2.37	3	#	8	8.0	92	72	7.52	7.00	7	#	9	10.7	272	12	1.39	1.27	# 10 @	12	55

TABLE EL2 - 1B

YIELD LINE EQUATIONS OF CONCRETE WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
F (KIPS)	**= 300**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.50	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K/')	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD = Rw/Wc

CRITICAL LENGTH (L) = Lt/2 + SQR [$(Lt/2)^2$ + 8H(Mb + Mw)/Mc]
BARRIER CAPACITY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)
= [16(Mb+Mw)/(2L-Lt)] + $2McL^2$/H(2L-Lt)

	Mb (FT-K)	Mw (FT-K)	Mc (FT-K/')	L (FT)	BEAM LOAD (KIPS) (A)	WALL LOAD (KIPS) (B)	BARRIER CAPACITY Rw (KIPS)	Lr (FT)	Bc (FT)	END EWc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
A	97	290	59	22.5	155	200	**355**	44.9	11.2	**89**	>	**78**	**OK**
B	93	276	52	23.3	142	181	**323**	46.6	11.7	**81**	>	**71**	**OK**
C	93	276	51	23.4	141	179	**321**	46.8	11.7	**80**	>	**71**	**OK**
D	93	276	52	23.3	142	181	**323**	46.6	11.7	**81**	>	**71**	**OK**
E	93	276	55	22.8	146	187	**332**	45.6	11.4	**83**	>	**73**	**OK**
F	93	272	52	23.2	141	180	**321**	46.4	11.6	**80**	>	**71**	**OK**
G	93	272	51	23.3	140	179	**319**	46.6	11.7	**80**	>	**70**	**OK**
H	93	272	52	23.2	141	180	**321**	46.4	11.6	**80**	>	**71**	**OK**
I	93	272	55	22.7	145	186	**331**	45.4	11.3	**83**	>	**72**	**OK**
J	92	276	52	23.3	142	181	**322**	46.5	11.6	**81**	>	**71**	**OK**
K	92	276	51	23.4	141	179	**320**	46.8	11.7	**80**	>	**70**	**OK**
L	92	276	52	23.3	142	181	**322**	46.5	11.6	**81**	>	**71**	**OK**
M	92	276	55	22.8	145	186	**332**	45.5	11.4	**83**	>	**73**	**OK**
N	92	272	52	23.2	141	180	**321**	46.3	11.6	**80**	>	**71**	**OK**
O	92	272	51	23.3	140	179	**319**	46.6	11.6	**80**	>	**70**	**OK**
P	92	272	52	23.2	141	180	**321**	46.3	11.6	**80**	>	**71**	**OK**
Q	92	272	55	22.6	145	186	**330**	45.3	11.3	**83**	>	**72**	**OK**

1. Vertical Bars Outside Face

Provide 50% of vertical bars inside face for ductility

Scenerio	F (kip)	Inside Face	Outside Face
		EL2	EL2
14	300	#7 @ 6"	#7 @ 12"
15	600	#8 @ 6"	#8 @ 12"
16	1100	#9 @ 6"	#9 @ 12"
17, 18	2540	#10 @ 5"	#10 @ 10"

2. Shear Desian

Top Beam: B=24"

Scenario	F (kip)	Table	EL2								
		EL2	H (in)	D (in)	Vu (in)	Vbeam (kip)	Φ Vc (kip)		Vu (kip)	Vs (kip)	Smax (in)
14	300	1A(B)	14	9.5	81	21	24	>	21	0	4
15	600	2A(G)	18	13.5	154	37	35	<	37	0	6
16	1100	3A(J)	24	19.5	281	81	50	<	81	36	6
17, 18	2540	4A(D)	40	35.5	646	99	92	<	99	0	6

Spacing lesser of d/2 or 6" for ductility

Sample D alculations:

Barrier Type EL3 (Elevated Alternate 3)

Structural Steel Post and Railing

TABLE EL3 - 4B

YIELD LINE EQUATIONS OF STRUCTURAL STEEL BEAM AND POST

INPUT DATA		DESCRIPTION
F (KIPS)	**= 2540**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 9.00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HT-6")
Mpb (FT-K)	= 4106	PLASTIC MOMENT CAPACITY OF BEAM
Mpc (FT-K)	= 4106	PLASTIC MOMENT CAPACITY
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 456	POST/COLUMN CAPACITY = Mpc/H
L (FT)	= VARIES	POST CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + POST LOAD (B)
Rw = 16(Mpb)/(2NL-Lt) + (Mpc/H)x(N-1)

Z EQUATIONS	T=	1.13	Z= D^3/6[1-(1-2t/D)^3]	
36 SCH ?	D=	36.00	Z=	1368.77

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	POST LOAD (KIPS) (B)	BARRIER CAPACITY (KIPS) (Rw)	REACTION CHECK Pc (KIPS)	>=	(A)/2 (KIPS)	FINAL CHK
15	1	2628	0	**2628**	**456**	<	**1314**	
15	2	1195	456	**1651**	**456**	<	**597**	
15	3	773	913	**1685**	**456**	>	**386**	**Rw<F, N.G.**
15	4	571	1369	**1940**	**456**	>	**286**	
12	1	3458	0	**3458**	**456**	<	**1729**	
12	2	1528	456	**1984**	**456**	<	**764**	
12	3	981	913	**1893**	**456**	<	**490**	
12	4	722	1369	**2091**	**456**	>	**361**	**Rw<F, N.G.**
12	5	571	1825	**2396**	**456**	>	**286**	
9	1	5054	0	**5054**	**456**	<	**2527**	
9	2	2119	456	**2576**	**456**	<	**1060**	
9	3	1341	913	**2253**	**456**	<	**670**	
9	4	981	1369	**2349**	**456**	<	**490**	
9	5	773	1825	**2598**	**456**	>	**386**	**Rw>F, OK**
9	6	638	2281	**2919**	**456**	>	**319**	
5	1	13140	0	**13140**	**456**	<	**6570**	
5	2	4380	456	**4836**	**456**	<	**2190**	
5	3	2628	913	**3541**	**456**	<	**1314**	
5	4	1877	1369	**3246**	**456**	<	**939**	
5	5	1460	1825	**3285**	**456**	<	**730**	
5	6	1195	2281	**3476**	**456**	<	**597**	
5	7	1011	2738	**3748**	**456**	<	**505**	
5	8	876	3194	**4070**	**456**	>	**438**	**Rw>F, OK**
5	9	773	3650	**4423**	**456**	>	**386**	

Sample Design Calculations:

Barrier Type HAG1 (Highway At Grade Alternate 1)

Cast In Place Concrete Barrier - Van Truck

FOUNDATION DESIGN - HAG1 & HAG2

CHART NUMBER	COLUMN SIZE (IN)	CAISSON DIAMETER (IN)	REQ'D Mc	As MIN	STEEL			ACTUAL As	CENTER SPACING	ACTUAL Mc
					# OF BARS	BAR SIZE				
HAG-1	18	24	**260**	4.52	6	#	9	6.00	8.38	**230**
HAG-1	18	24	**260**	4.52	7	#	9	7.00	7.18	**280**
HAG-1	24	28	**356**	6.16	10	#	9	10.00	6.28	**440**
HAG-2	24	28	**356**	6.16	10	#	9	10.00	6.28	**440**

FOUNDATION EMBEDMENT DESIGN

ALTERNATES HAG1 & HAG2

Kp = 3.00	TAN^2 (45+PHI/2)
J = 0.14	DAMPING CONSTANT (sec/ft)
V= VARIES	IMPACT VELOCITY (fps)
LF= 1 + J V	DYNAMIC LOAD FACTOR
c = 1.50	COHESION STRENGTH (ksf)
q = 13.50	SOIL STRENGTH (ksf)
γ = 110.00	AVERAGE EFECTIVE SOIL UNIT WEIGHT (pcf)
ϕ = 30	ANGLE OF INTERNAL FRICTION (degrees)
Mc= VARIES	ULTIMATE MOMENT CAPACITY OF COLUMN
H= VARIES	DISTANCE FROM IMPACT FORCE TO TOP OF GRADE (BARRIER HT -6")

Pdynamic = Mc / H   Pstatic = P dynamic / L.F.

EQUATION FOR EMBEDMENT LENGTH IN COHESIVE SOILS: LE=(P/qB)+SQRT[2(P/qB)^2+(4HP/qB)]

EQUATION FOR THE EMBEDMENT LENGTH IN COHESIONLESS SOILS: LE^3=2P(H+LE)/(UNIT WT)B(Kp)

ALT. #	Mc	H (FT)	P dyn. (KIPS)	Impact Velocity (fps)	L.F.	P static (KIPS)	B WIDTH (FT)	EMBEDMENT LENGTH (FT)				
								Cohesive		Cohesionless		
								REQ'D	PROV.	REQ'D		PROV.
HAG1	260	4.17	62	0	1.00	62	2.00	**9.32**	**12.3**	**15.49**	15.5	**19.0**
HAG1	260	4.17	62	0	1.00	62	2.00	**9.32**	**12.3**	**15.49**	15.5	**19.0**
HAG1	260	4.17	62	0	1.00	62	2.00	**9.32**	**12.3**	**15.49**	15.5	**19.0**
HAG2	356	7.50	47	0	1.00	47	2.33	**8.57**	**11.4**	**13.80**	13.8	**17.2**
HAG2	356	7.50	47	0	1.00	47	2.33	**8.57**	**11.4**	**13.80**	13.8	**17.2**
HAG2	356	7.50	47	0	1.00	47	2.33	**8.57**	**11.4**	**13.80**	13.8	**17.2**

TABLE HAG1 - A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE HIGHWAY BARRIER WALL

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd FOR BEAM AND WALL (EACH FACE) = 4% GROSS AREA FOR COLUMN (2% EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

BEAM									WALL									COLUMN								
H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	PROV. NO.		PROV. DIA.	PROV. SPA (IN.)	. Mb (FT-K)	H (IN.)	Bw (IN.)	MAX. Asw (IN^2)	PROV. Asb (IN^2)	PROV. NO.		PROV. DIA.	PROV. SPA. (IN.)	Mw (FT-K)	H (IN.)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	PROV. NO.		PROV. DIA.	PROV. SPA. (IN.)	Mc (FT-K)
11	12	0.86	1.32	3	#	6	2.0	33	14	20	2.09	1.76	4	#	6	4.0	69	18	18	6.48	4.74	6	#	8	2.0	260
11	12	0.86	0.88	2	#	6	4.0	23	14	20	2.09	1.80	3	#	7	6.0	71	18	18	6.48	3.95	5	#	8	2.5	223
11	12	0.86	1.20	2	#	7	4.0	30	14	20	2.09	2.37	3	#	8	6.0	**90**	18	18	6.48	2.64	6	#	6	2.0	157
11	12	0.86	1.80	3	#	7	2.0	42	14	20	2.09	2.00	2	#	9	12.0	78	18	18	6.48	6.00	6	#	9	2.0	312
11	12	0.86	2.37	3	#	8	2.0	51	14	20	2.09	2.54	2	#	10	12.0	**96**	18	18	6.48	7.62	6	#	10	2.0	**369**

TABLE HAG1 - B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS

INPUT DATA

F (KIPS)	= 124	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY, Rw)
H (FT)	= 4.00	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6"
Mb (FT-K)	= 33	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= 69	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 260	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 2.50	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 65	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION Pc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
15	1	68	0	26	0	94	65	>	34	Rw<F, N.G.
15	2	31	0	12	65	108	65	>	15	Rw<F, N.G.
15	3	20	92	8	0	119	65	>	10	Rw<F, N.G.
15	4	15	68	6	65	153	65	>	7	Rw>F,OK
12	1	89	0	34	0	123	65	>	45	Rw<F, N.G.
12	2	39	0	15	65	120	65	>	20	Rw<F, N.G.
12	3	25	93	10	0	128	65	>	13	Rw>F,OK
12	4	19	69	7	65	159	65	>	9	Rw>F,OK
10	1	113	0	43	0	156	65	>	57	Rw>F,OK
10	2	48	0	19	65	132	65	>	24	Rw>F,OK
10	3	31	95	12	0	137	65	>	15	Rw>F,OK
10	4	23	69	9	65	166	65	>	11	Rw>F,OK
10	5	18	164	7	0	189	65	>	9	Rw>F,OK
8	1	154	0	59	0	213	65	<	77	
8	2	63	0	24	65	152	65	>	31	Rw>F,OK
8	3	39	97	15	0	151	65	>	20	Rw>F,OK
8	4	29	71	11	65	175	65	>	14	Rw>F,OK
8	5	23	166	9	0	198	65	>	11	Rw>F,OK
5	1	339	0	130	0	469	65	<	170	
5	2	113	0	43	65	221	65	>	57	Rw>F,OK
5	3	68	104	26	0	198	65	>	34	Rw>F,OK
5	4	48	74	19	65	206	65	>	24	Rw>F,OK
5	5	38	173	14	0	225	65	>	19	Rw>F,OK

Sample Design Calculations:

Barrier Type HAG2 (Highway At Grade Alternate 2)

Cast In Place Concrete Barrier - Tank Truck

TABLE HAG2 - 1A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE BARRIER WALL ON DISCRETE FOUNDATION

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL. AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

	BEAM									WALL			COLUMN								
	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	PROV. NO.		PROV. DIA.	PROV. SPA. (IN.)	Mb (FT-K)	H (IN.)	Bw (IN.)	Mw (FT-K)	H (IN.)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	PROV. NO.		PROV. DIA.	PROV. SPA. (IN.)	Mc (FT-K)
	16	21	2.89	3.95	5	#	8	3.3	**193**	0	0	**0**	8	60	3.63	4.80	8	#	7	7.4	**276**
	16	21	2.89	3.95	5	#	8	3.3	**193**	0	0	**0**	8	60	3.63	4.80	8	#	7	7.4	**276**
	16	21	2.89	3.95	5	#	8	3.3	**193**	0	0	**0**	8	60	3.63	4.80	8	#	7	7.4	**276**
	16	21	2.89	3.95	5	#	8	3.3	**193**	0	0	**0**	8	60	3.63	4.80	8	#	7	7.4	**276**
	16	21	2.89	3.95	5	#	8	3.3	**193**	0	0	**0**	8	60	3.63	4.80	8	#	7	7.4	**276**

TABLE HAG2 - 1B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS

INPUT DATA		
F (KIPS)	= 175	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 3.50	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6")
Mb (FT-K)	= 193	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K	= 0	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 276	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 2.50	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 79	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: $Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H$

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION CHECK Pc (KIPS)	>	(A)/2 (KIPS)	CAP. CHK
10	1	210	0	53	0	262	79	<	105	
10	2	90	0	23	79	191	79	>	45	Rw>F,OK
10	3	57	115	14	0	186	79	>	29	Rw>F,OK
10	4	42	84	11	79	215	79	>	21	Rw>F,OK
10	5	33	199	8	0	241	79	>	17	Rw>F,OK
10	6	27	165	7	79	278	79	>	14	Rw>F,OK
10	7	23	280	6	0	310	79	>	12	Rw>F,OK
10	8	20	244	5	79	348	79	>	10	Rw>F,OK
10	9	18	360	5	0	383	79	>	9	Rw>F,OK
10	10	16	324	4	79	423	79	>	8	Rw>F,OK
10	11	15	440	4	0	458	79	>	7	Rw>F,OK
10	12	13	403	3	79	498	79	>	7	Rw>F,OK
10	13	12	520	3	0	535	79	>	6	Rw>F,OK
10	14	11	482	3	79	575	79	>	6	Rw>F,OK
10	15	11	599	3	0	612	79	>	5	Rw>F,OK
10	16	10	561	3	79	652	79	>	5	Rw>F,OK
10	17	9	678	2	0	690	79	>	5	Rw>F,OK
10	18	9	640	2	79	730	79	>	4	Rw>F,OK
10	19	8	757	2	0	768	79	>	4	Rw>F,OK
10	20	8	719	2	79	808	79	>	4	Rw>F,OK

TABLE HAG2 - 2A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE HIGHWAY BARRIER WALL

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

BEAM									WALL									COLUMN								
H	Bb	MAX.	PROV.					Mb	H	Bw	MAX.	PROV.					Mw	H	Bc	MAX.	PROV.					Mc
(IN.)	(IN.)	Asb (IN^2)	Asb (IN^2)	NO.		DIA.	SPA. (IN.)	(FT-K)	(IN.)	(IN.)	Asw (IN^2)	Asb (IN^2)	NO.		DIA.	SPA. (IN.)	(FT-K)	(IN.)	(IN.)	Asc (IN^2)	Asb (IN^2)	NO.		DIA.	SPA. (IN.)	(FT-K)
15	21	2.66	2.37	3	#	8	6.5	**112**	17	27	4.01	4.00	4	#	9	6.3	**219**	24	24	3.83	3.95	5	#	8	4.0	**356**
15	21	2.66	2.37	3	#	8	6.5	**112**	17	27	4.01	4.00	4	#	9	6.3	**219**	24	24	3.83	3.95	5	#	8	4.0	**356**
15	21	2.66	2.37	3	#	8	6.5	**112**	17	27	4.01	4.00	4	#	9	6.3	**219**	24	24	3.83	3.95	5	#	8	4.0	**356**
15	21	2.66	2.37	3	#	8	6.5	**112**	17	27	4.01	4.00	4	#	9	6.3	**219**	24	24	3.83	3.95	5	#	8	4.0	**356**
15	21	2.66	2.37	3	#	8	6.5	**112**	17	27	4.01	4.00	4	#	9	6.3	**219**	24	24	3.83	3.95	5	#	8	4.0	**356**

TABLE HAG2 - 2B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS		
INPUT DATA		
F (KIPS)	**= 175**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.50	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6"
Mb (FT-K)	= 112	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= 219	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 356	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 2.00	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 48	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]

EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION Pc (KIPS)	>	CHECK (A)/2 (KIPS)	CAP. CHK
10	1	362	0	25	0	**387**	**48**	**<**	**181**	
10	2	155	0	11	48	**213**	**48**	**<**	**77**	
10	3	99	69	7	0	**175**	**48**	**<**	**49**	
10	4	72	51	5	48	**176**	**48**	**>**	**36**	**Rw>F,OK**
10	5	57	120	4	0	**181**	**48**	**>**	**29**	**Rw>F,OK**
10	6	47	99	3	48	**197**	**48**	**>**	**24**	**Rw>F,OK**
10	7	40	169	3	0	**212**	**48**	**>**	**20**	**Rw>F,OK**
10	8	35	147	2	48	**232**	**48**	**>**	**17**	**Rw>F,OK**
10	9	31	217	2	0	**250**	**48**	**>**	**15**	**Rw>F,OK**
10	10	28	195	2	48	**272**	**48**	**>**	**14**	**Rw>F,OK**
10	11	25	265	2	0	**292**	**48**	**>**	**13**	**Rw>F,OK**
10	12	23	243	2	48	**315**	**48**	**>**	**12**	**Rw>F,OK**

Sample Design Calculations:

Barrier Type HEL1 (Highway Elevated Alternate 1)

Cast In Place Concrete Barrier - Van Truck

TABLE HELI - A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE HIGHWAY BARRIER WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-4.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-3.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY
		= 0.011Bd FOR BEAM AND WALL (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB

	BEAM									WALL									COLUMN						
	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	Asb (IN^2)	PROV. NO.		DIA	SPA (IN.)	Mb (FT-K)	H (IN.)	Bw (IN.)	MAX. Asw (IN^2)	Asb (IN^2)	PROV. NO.		DIA.	SPA (IN.)	Mw (FT-K)	H (IN.)	Bc (IN.)	MAX. Asc (IN^2)	Asb (IN^2)	PROV. NO.	SPA (IN.)	Mc (FT-K)
A	11	12	0.86	0.86					23	14	20	2.09	2.09					81	18	12	1.91	1.91			113
B	11	12	0.86	1.33	3	#	6	2.0	33	14	20	2.09	0.44	1	#	6	0.0	18	18	12	1.91	0.47	# 5 @	8	30
C	11	12	0.86	1.33	3	#	6	2.0	33	14	20	2.09	0.44	1	#	6	0.0	18	18	12	1.91	0.47	# 5 @	8	30
D	11	12	0.86	1.33	3	#	6	2.0	33	14	20	2.09	0.88	2	#	6	12.0	36	18	12	1.91	0.47	# 5 @	8	30

TABLE HELI - B

YIELD LINE EQUATION8 OF CONCRETE WALL ON BRIDGE DECK		
INPUT DATA		DESCRIPTION
F (KIPS)	= **124**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 4.00	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-W)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD = Rw/Wc

CRITICAL LENGTH (L) = $Lt/2 + SQR[(Lt/2)^2 + 8H(Mb + Mw)/Mc]$

BARRIER CAPACIIY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)

= $[16(Mb+Mw)/(2L-Lt)] + 2McL^2/H(2L-Lt)$

	Mb (FT-K)	Mw (FT-K)	Mc (FT-Kf)	L (FT)	BEAM LOAD (KIPS) (A)	WALL LOAD (KIPS) (B)	BARRIER CAPACITY Rw (KIPS)	Lr (FT)	Bc (FT)	END CHECK EWc (KIPS)	>	(A)/2 (KIPS)	CAP. CHK
A	33	81	113	8.5	139	339	477	[illegible]	[illegible]	**119**	>	**69**	**OK**
B	33	18	30	10.4	52	101	**153**	20.7	5.2	**38**	>	**26**	OK
C	33	18	30,	10.4	52	101	**153**	20.7	5.2	**38**	>	**26**	OK
D	33	36	30	11.5	**61**	109	**170**	23.0	5.7	**43**	>	**31**	**OK**

Sample Design Calculations:

Barrier Type HEL2 (Highway Elevated Alternate 2)

Cast In Place Concrete Barrier - Tank Truck

TABLE HEL2 - 1A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE BARRIER WALL ON DISCRETE FOUNDATION		
INPUT DATA		**DESCRIPTION**
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.5	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.5	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA: MAX. LIMIT IS FOR DUCTILITY = 0.011Bd (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

BEAM							WALL			COLUMN						
H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	PROV. NO. # DIA.	PROV. SPA. (IN.)	Mb (FT-K)	H (IN.)	Bw (IN.)	Mw (FT-K)	H (IN.)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	PROV. NO. # DIA.	PROV. SPA. (IN.)	Mc (FT-K)
16	21	2.89	3.95	5 # 8	3.3	193	0	0	0	8	60	3.63	4.80	8 # 7	7.4	276
16	21	2.89	3.95	5 # 8	3.3	193	0	0	0	8	60	3.63	4.80	8 # 7	7.4	276
16	21	2.89	3.95	5 # 8	3.3	[illegible]	[illegible]	[illegible]	[illegible]	8	60	3.63	4.80	8 # 7	7.4	276
[illegible]	[illegible]	2.89	3.95	5 # 8	3.3	193	0	0	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	21	2.89	3.95	5 # 8	3.3	193	0	0	0	8	60	3.63	4.80	8 # 7	7.4	276

TABLE HEL2 - 1B

YIELD LINE EQUATIONS OF CONCRETE WALL ON DISCRETE FOUNDATIONS		
		INPUT DATA
F (KIPS)	= 175	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 3.50	DISTANCE FROM T/FOUNDATION TO IMPACT FORCE (BARRIER HEIGHT-6")
Mb (FT-K)	= 193	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K	= 0	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K)	= 276	ULTIMATE MOMENT CAPACITY OF WALL /COL. AT FOUNDATION (VERT.)
B (FT)	= 2.50	WIDTH OF FOUNDATION
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
Pc (KIPS)	= 79	POST/COLUMN CAPACITY = Mc/H
L (FT)	= VARIES	FOUNDATION CENTERLINE SPACING
N (#)	= VARIES	NUMBER OF SPANS IN FAILURE MECHANISM

BARRIER CAPACITY (Rw) = BEAM LOAD(A) + COLUMN LOAD[INTERIOR (B)+END (C)+MIDDLE (D)]
EVEN SPANS: Rw = 16(Mb+Mw)/(2NL-Lt) + (N-2)NMcL/H(2NL-Lt) + 4McB/H(2NL-Lt) + Mc/H

L (FT)	N (#)	BEAM LOAD (KIPS) (A)	COLUMN LOAD (KIPS) INTERIOR (B)	END (C)	MIDDLE (D)	BARRIER CAPACITY Rw (KIPS)	REACTION CHECK Pc (KIPS)	>	(A)/2 (KIPS)	CAP. CHK
10	1	210	0	53	0	**262**	**79**	**<**	**105**	
10	2	90	0	23	79	**191**	**79**	**>**	**45**	**Rw>F,OK**
10	3	57	115	14	0	**186**	**79**	**>**	**29**	**Rw>F,OK**
10	4	42	84	11	79	**215**	**79**	**>**	**21**	**Rw>F,OK**
10	5	33	199	8	0	**241**	**79**	**>**	**17**	**Rw>F,OK**
10	6	27	165	7	79	**278**	**79**	**>**	**14**	**Rw>F,OK**
10	7	23	280	6	0	**310**	**79**	**>**	**12**	**Rw>F,OK**
10	8	20	244	5	79	**348**	**79**	**>**	**10**	**Rw>F,OK**
10	9	18	360	5	0	**383**	**79**	**>**	**9**	**Rw>F,OK**
10	10	16	324	4	79	**423**	**79**	**>**	**8**	**Rw>F,OK**
10	11	15	440	4	0	**458**	**79**	**>**	**7**	**Rw>F,OK**
10	12	13	403	3	79	**498**	**79**	**>**	**7**	**Rw>F,OK**
10	13	12	520	3	0	**535**	**79**	**>**	**6**	**Rw>F,OK**
10	14	11	482	3	79	**575**	**79**	**>**	**6**	**Rw>F,OK**
10	15	11	599	3	0	**612**	**79**	**>**	**5**	**Rw>F,OK**
10	16	10	561	3	79	**652**	**79**	**>**	**5**	**Rw>F,OK**
10	17	9	678	2	0	**690**	**79**	**>**	**5**	**Rw>F,OK**
10	18	9	640	2	79	**730**	**79**	**>**	**4**	**Rw>F,OK**
10	19	8	757	2	0	**768**	**79**	**>**	**4**	**Rw>F,OK**
10	20	8	719	2	79	**808**	**79**	**>**	**4**	**Rw>F,OK**

TABLE HEL2 - 2A

ULTIMATE MOMENT CAPACITIES FOR C.I.P. CONCRETE HIGHWAY BARRIER WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
Fy (KSI)	= 60	YIELD STRENGTH OF REINFORCEMENT
F'c (KSI)	= 4	CONCRETE COMPRESSIVE STRENGTH
H (IN.)	= VARIES	BEAM, WALL, AND COLUMN THICKNESS/DEPTH
B (IN.)	= VARIES	BEAM, WALL, AND COLUMN EFFECTIVE WIDTH
d (IN.)	= H-3.0	EFFECTIVE DEPTH FOR BEAM & WALL
	= H-2.0	EFFECTIVE DEPTH FOR COLUMN
As (IN^2)	= VARIES	REINF. AREA. MAX. LIMIT IS FOR DUCTILITY
		= 0.011Bd FOR BEAM AND WALL (EACH FACE)
M (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM/WALL/COLUMN

M= 0.9AsFy(d-a/2), a=AsFy/(0.85F'cB)

	BEAM								WALL								COLUMN						
	H (IN.)	Bb (IN.)	MAX. Asb (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. DIA.	PROV. SPA. (IN.)	Mb (FT-K)	H (IN.)	Bw (IN.)	MAX. Asw (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. DIA.	PROV. SPA. (IN.)	Mw (FT-K)	H (IN.)	Bc (IN.)	MAX. Asc (IN^2)	PROV. Asb (IN^2)	PROV. NO.	PROV. SPA. (IN.)	Mc (FT-K)
A	11	21	1.85	1.85				[illegible]	13	27	2.97	2.97				[illegible]	17	12	1.96	1.96			[illegible]
B	11	21	1.85	1.33	3	# 6	6.5	44	13	27	2.97	0.88	2	# 6	18.0	38	17	12	1.96	0.86	# 6 @	6	67
C	11	21	1.85	1.33	3	# 6	6.5	44	13	27	2.97	0.88	2	# 6	18.0	38	17	12	1.96	0.86	# 6 @	6	67
D	11	21	1.85	1.33	3	# 6	6.5	44	13	27	2.97	0.88	2	# 6	18.0	38	17	12	1.96	0.86	# 6 @	6	67
E	11	21	1.85	1.33	3	# 6	6.5	44	13	27	2.97	0.88	2	# 6	18.0	38	17	12	1.96	0.86	# 6 @	6	67
F	11	21	1.85	1.33	3	# 6	6.5	44	13	27	2.97	0.88	2	# 6	18.0	38	17	12	1.96	0.86	# 6 @	6	67

TABLE HEL2 - 2B

YIELD LINE EQUATIONS OF CONCRETE WALL ON BRIDGE DECK

INPUT DATA		DESCRIPTION
F (KIPS)	**= 175**	MAXIMUM IMPACT FORCE (<= BARRIER CAPACITY , Rw)
H (FT)	= 7.50	DISTANCE FROM T/SLAB TO IMPACT FORCE (BARRIER HEIGHT - 6")
Mb (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF BEAM AT TOP OF WALL (LONGIT.)
Mw (FT-K)	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (LONGIT.)
Mc (FT-K/')	= VARIES	ULTIMATE MOMENT CAPACITY OF WALL (VERT.)
Wc (K/FT)	= Mc/H	WALL CAPACITY PER LINEAL FOOT OF WALL
EWc (KIPS)	= VARIES	END WALL CAPACITY = Wc(Bc)
Bc (K/FT)	= VARIES	WIDTH OF END WALL = (Lr - L)/2
Lt (FT)	= 5	LENGTH OF DISTRIBUTED IMPACT LOAD
L (FT)	= VARIES	CRITICAL LENGTH OF WALL FAILURE
Lr (FT)	= VARIES	TOTAL LENGTH OF WALL RESISTING IMPACT LOAD = Rw/Wc

CRITICAL LENGTH (L) = $Lt/2 + SQR[(Lt/2)^2 + 8H(Mb + Mw)/Mc]$

BARRIER CAPACITY (Rw) = BEAM LOAD (A) [TOP BEAM + HORIZ. WALL] + VERT. WALL LOAD (B)

$= [16(Mb+Mw)/(2L-Lt)] + 2McL^2/H(2L-Lt)$

	Mb (FT-K)	**Mw (FT-K)**	**Mc (FT-K/')**	**L (FT)**	**BEAM LOAD (KIPS) (A)**	**WALL LOAD (KIPS) (B)**	**BARRIER CAPACITY Rw (KIPS)**	**Lr (FT)**	**Bc (FT)**	**END EWc (KIPS)**	**>**	**CHECK (A)/2 (KIPS)**	**CAP. CHK**
A	60	121	121	12.3	147	248	**396**	24.6	6.2	**99**	**>**	**74**	**OK**
B	44	38	57	12.2	68	116	**185**	24.4	6.1	**46**	**>**	**34**	**OK**
C	44	38	57	12.2	68	116	**185**	24.4	6.1	**46**	**>**	**34**	**OK**
D	44	38	57	12.2	68	116	**185**	24.4	6.1	**46**	**>**	**34**	**OK**
E	44	38	57	12.2	68	116	**185**	24.4	6.1	**46**	**>**	**34**	**OK**
F	44	38	57	12.2	68	116	**185**	24.4	6.1	**46**	**>**	**34**	**OK**

Sample Desipn Calculations:

Deflections

Crash barrier deformations due to both elastic and plastic deflections.

Materials considered: steel and;
concrete.

Assumptions: foundation is rigid and unyielding;
barrier is continuous;
post deformations are critical;

Deformations determined are those that occur just prior to failure.
The total number of posts that can yield is limited by the total plastic post deformation.

I GENERAL PROCEDURE

A) Analyze member to determine internal stresses and strains for both the elastic and the plastic portions of the member.

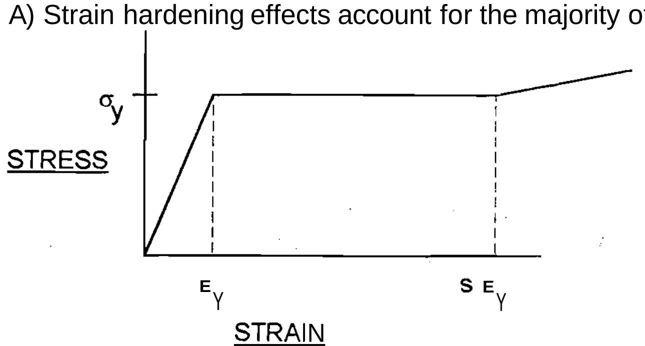
B) Determine the length of the member that undergoes plastic deformations.

C) Using the curvature diagram along the length of the member, then the total deflection can be computed by the moment-area method.

II STEEL FRAMED BARRIER

Reference: "Inelastic Beams Under Moment Gradient," by Lay, Maxwell G. and Galambos, Theodore V. , Journal of the Structural Division, ASCE, ST 1, p.381 - 391, February 1967.

A) Strain hardening effects account for the majority of the plastic deformations.



1) ε_y : strain at yield;
2) $s\varepsilon_y$: strain hardening effect.
3) For A36 steel, $s=11.5$ and E_s = 29,000,000 psi based on experimental data;
4) Length of plastic hinge: $l_p = \tau L$ and $\tau = 1 - (M_{ps}/M_o)$ where $M_{ps} \approx 0.94\ M_p$ and M_o (max) $\approx \frac{1}{2}(\sigma_u/\sigma + 1)\ M_p$ and M_o (max) $\approx 1.30\ M_p$ for A36 steel.
Therefore: τ = 0.27%

B) Local buckling also effects the length of the plastic hinge:
1) $l_p = \tau L = 2.65\ r_y$ or
2) $l_p = \tau L = 1.42(t/w)(A_w/A_f)^{1/4}$ b so that for a pipe or tube post then $l_p \approx 1.42$ b

C) Support Spacing
1) Maximum unsupported length for A36 steel:
= 70 r_y for simply supported beams and
= 90 r_y for continuous beams.

D) Shear limits:

1) Maximum shear capacity: $V_s = A_w (\sigma_u + \sigma_y) / 2\sqrt{3}$ and

2) For combined shear and flexure then $V_{max} = (M_o - M_{ps}) / \tau L = 0.36\ Mp / \tau L$

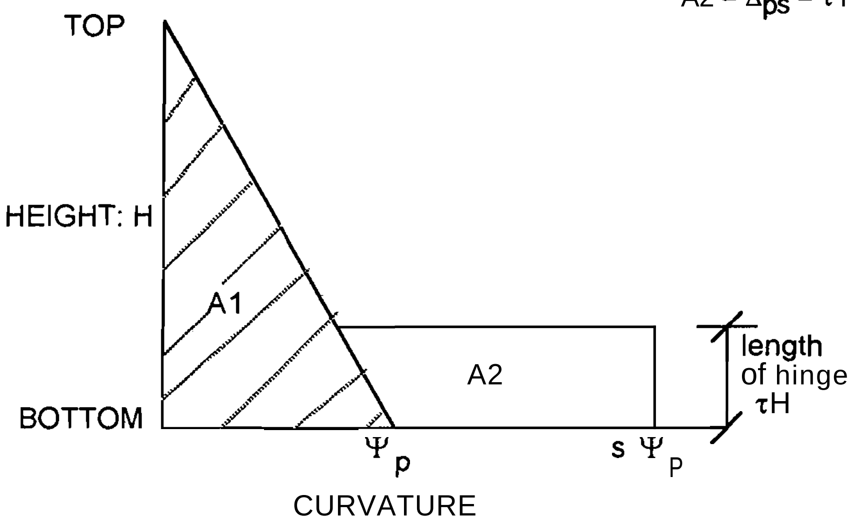
Ref.: "Plastic Theory of Structures"', by M.R. Horne, 1979, 2nd edition, Pergamon Press, p.79.

3) For $V_{design} \leq ½ (0.6\ F_y\ d\ t_w)$ then shear does not effect the plastic moment capacity.

E) Curvature analysis based on members cross-section for a cantilever member:

$A1 = \Delta_p = 1/3\ (\psi_p)H^2$ and

$A2 = \Delta_{ps} = \tau H\ (s-1)(\psi_p)(H - \tau H/2)$



MOMENT-AREA DIAGRAM

where $\psi_p = 2\varepsilon_y / d = M_p / EI$ and the curvature ductility ratio: $\mu_c = s\psi_p / \psi_p = s$
or $\mu_c = \theta_h / \theta_y$ where $\theta_y = M_p\ H / 2EI$ and $\theta_h = 2.84\varepsilon_y(\beta-1)(b/d)(t_f/t_w)\ (A_w/A_f)^{1/4}\ (1 + V_1/V_2)$

F) Deformation ductility factor

1) $\Delta_{total} = \Delta_p + \Delta_{ps} = \mu_d\ \Delta_p$ where $\mu_d = 1 + 3(\mu_c - 1)\tau(1-0.5\tau)$

<u>Example using structural steel post:</u> AG4, F = 300 KIPS

For a steel column consisting of 18" O schedule 80 pipe that is seven (7) feet high. Therefore: A = 50.2 in.2

I = 1834 in.4

S_y = 203.8 in.3 and M_p = 819 ft-kips

r_y = 6.04 in.

The elastic hinge rotation for a cantilever is: $\theta_y = M_p\ H / (2EI) = 0.005794$ radians

The elastic deflection: $\Delta_y = M_p H^2 / (3EI) = \{819.0 \times 12 (7 \times 12)^2\} / (3 \times 29000 \times 1834) = 0.435''$

The inelastic hinge rotation is: $\theta_h = 2.84\varepsilon_y(\beta-1)(b/d)(t_f/t_w)(A_w/A_f)^{1/4}(1 + V_1/V_2)$

For a round or square tube section then the following assumptions are made:

$b/d = 1$, $t_f/t_w = 1$, $A_w/A_f = 1$, also the maximum value for $V_1/V_2 = 1$

then $\theta_h = 2.84\varepsilon_y(\beta-1)(1+1)$

For A36 steel then $\varepsilon_y = 36000 / 29000000 = 0.00124$ and $\beta = 11.5$

and then $\theta_h = 0.07404$ (radians).

For a cantilever height of seven (7) feet then the curvature ductility ratio is:

$\mu_c = \theta_h / \theta_y = 0.07404 / 0.005794 = 12.8$ as a check against $\mu_c = s = 11.5$.

The length of the plastic hinge : $l_p = \tau H = 0.276 H = 0.276 (7 \times 12) = 23.2''$

$\tau = 1 - (M_{ps} / M_o) = 1 - (0.94M_p/1.30M_p) = 0.276 = l_p/H$

or Check: $l_p = 1.42 b \approx 1.42(18) = 25.56''$ and $l_p/H = 0.304$ <u>GOVERNS</u>

and $l_p = 2.65r_y = 2.65(6.04) = 16''$

The displacement ductility ratio is $\mu_d = (\Delta_y + \Delta_p) / \Delta_y = 1 + 3(\mu_c-1)l_p/H(1-0.5l_p/H)$

$\mu_d = 1 + 3(12.8-1)\ 0.304\ (1-0.5(.304)) = 10.13$

Therefore $\Delta_{total} = \mu_d\Delta_y = 10.13(0.435) = 4.40''$ maximum for this cantilever.

G) Deflection analysis for beam member

1) model as a member that is simply supporterd at ends with a concentrated load at midspan;

2) deflection at yield: $\Delta_y = \dfrac{M_p L^2}{12EI}$ and rotation at yield: $\theta_y = \dfrac{M_p L}{12EI}$

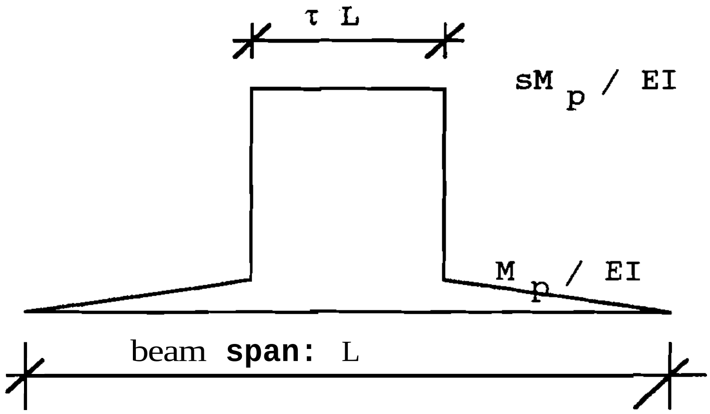
3) Inelastic deflection : $\Delta_p = \theta_h(\frac{1}{4}L - \frac{1}{8}\tau L)$

where: $\theta_h = 0.07404$ for A36 steel
$\tau = 0.276$ or $1.42b/L$ or $2.65r_y/L$
L = beam span
$lp = \tau L$

4) curvature ductility ratio: $\mu_c = \theta_h / \theta_y$

5) displacement ductility ratio: $\mu_d = 1 - 6(\mu_c - 1)\frac{l_p}{L}(1 - \frac{l_p}{L})$

6) Total beam deflection: A„„ $= \mu_d \Delta_y = \Delta_y + \Delta_p$



<u>MOMENT AREA DIAGRAM FOR SIMPLY SUPPORTED BEAM WITH CONCENTRATED LOAD AT MIDSPAN</u>

<u>Example: </u>AG4, F=300 KIPS

For beam span 19 feet and using 18"diameter shcedule 80 pipe then

τ L= 0.276 (19x12) =63" or 1.42(18)=26" or 2.65(6.04) = 16" and L = 228"
then θ_h = 0.07404 and θ_y = 819x12(19x12) / (4x29000x1834) = 0.0105328

μ_c = 0.07404 / 0.0105328 = 7.0 and l_p / L = 631228 = 0.276

Δ_y = 819x12(19x12)^2/(12~29000~183÷)0.80"

μ_d = 1 + 6(7.0 - 1)0.276(1-0.276) = 8.23

Therefore Δ_{total} = 8.23 (0.80) = 6.58"

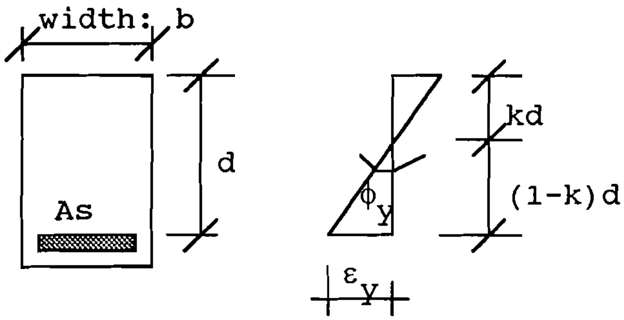
H) Total barrier deflection

1) Summation of post and beam deflections

2) For the previous examples:
$\Delta_{barrier}$ = 4.40 + 6.58 = 10.98"

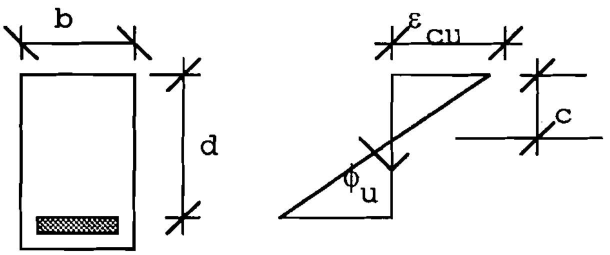
III <u>REINFORCED CONCRETE FRAMED BARRIER</u>
Reference: "Handbook of Concrete Engineering," edited by Mark Fintel, Van Nostrand Reinhold Co., 1974, p. 232 - 241.
A) Curvature analysis based on members cross-section $\mu_C = \phi_u / \phi_y$

For a member with tension reinforcing only:



$$\phi_y = \frac{\varepsilon_y}{d(1-k)} = \frac{f_y}{E_s\,(1-k)d}$$

<u>YIELD CURVATURE</u> ϕ_Y



$$\phi_u = \frac{\varepsilon_{cu}}{c}$$

<u>ULTIMATE CURVATURE</u> ϕ_u

The curvature ductility ratio is:

$$\mu_c = \frac{\varepsilon_{cu}(0.85\beta_1 f'_c)E_s(1+\rho n-\sqrt{2\rho n+\rho^2 n^2})}{\rho f_y^{\,2}}$$

And for ε_C = 0.003 and E_S = 29,000,000 psi then:{ACI 318 10.2.3}

The recommended limits for μ_C to be greater than **5** which corresponds to a reinforcing ratio of $\rho < 0.5\ \rho_b$ to ensure sufficient ductility. {Refer to Table **8-2**}

B) Compression reinforcing tends to increase the members ductility. $\rho' = A'_S / bd$

1) ductility ratio:

$$\mu_c = \frac{\phi_u}{\phi_y} = \frac{\varepsilon_{cu} d(1-k)E_s}{c f_y}$$

2) depth of compression block

$$c = \frac{(\rho-\rho')f_y d}{0.85 f'_c \beta_1}$$

3) value of k

$$k = \sqrt{\{(\rho+\rho')^2 n^2 + 2[\rho+(\rho' d'/d)]n\}} - (\rho+\rho')n$$

4) Must check compression reinforcing to see if it has yielded or not

C) The maximum concrete strain can be increased by using confining stirrups. (ρ_S)

1) $\varepsilon_{cu} = 0.0015\{1 + 150\rho_S + (0.7 - 10\rho_S)d/c\}$ or;

2) $\varepsilon_{cu} = 0.003 + 0.02b/L + (\rho_S f_y/138)^2$

D) Length of plastic hinge: $l_p \approx 0.5\ h$

1) $l_p = d/4$ if $v_u < v_c$

2) $l_p = 2d/3$ if $v_u > v_c$

E) Young's Modules of Elasticity

1) ACI 318 : $E_c = 57000\sqrt{f'_c}$ psi

F) Transformed cracked moment of inertia: I_{cr}

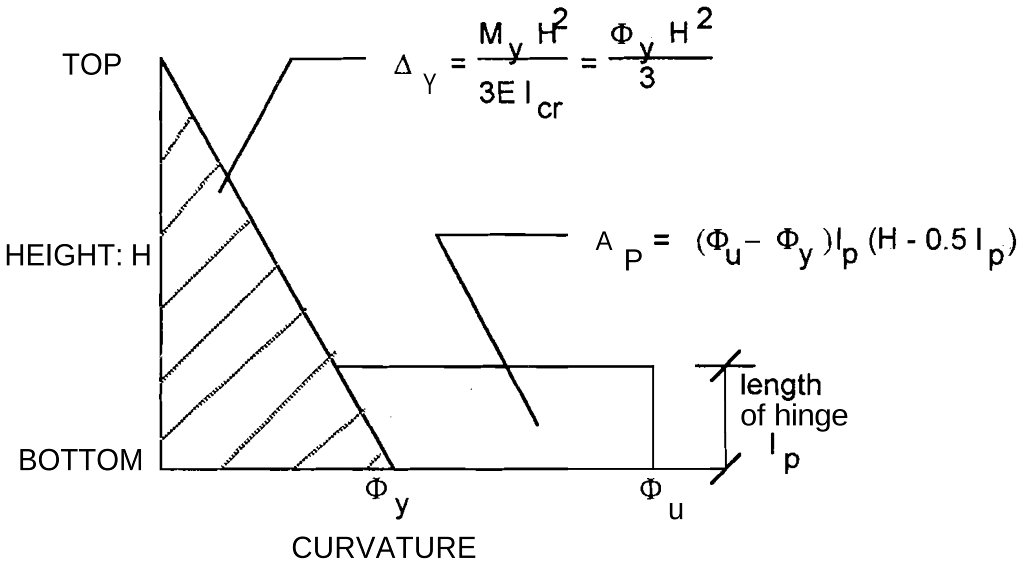
1) For doubly reinforced sections;

2) $n = E_S / E_C$

3) Locate neutral axis: $x = kd$

4) Therefore $I_{cr} = bx^3/3 + (n-1)A'_S(x-d')^2 + nA_S(d-x)^2$

G) Curvature Diagram For a Cantilever Member: $\Phi_i = M_i / EI_{cr}$



MOMENT-AREA DIAGRAM

H) Total displacement for cantilever member:

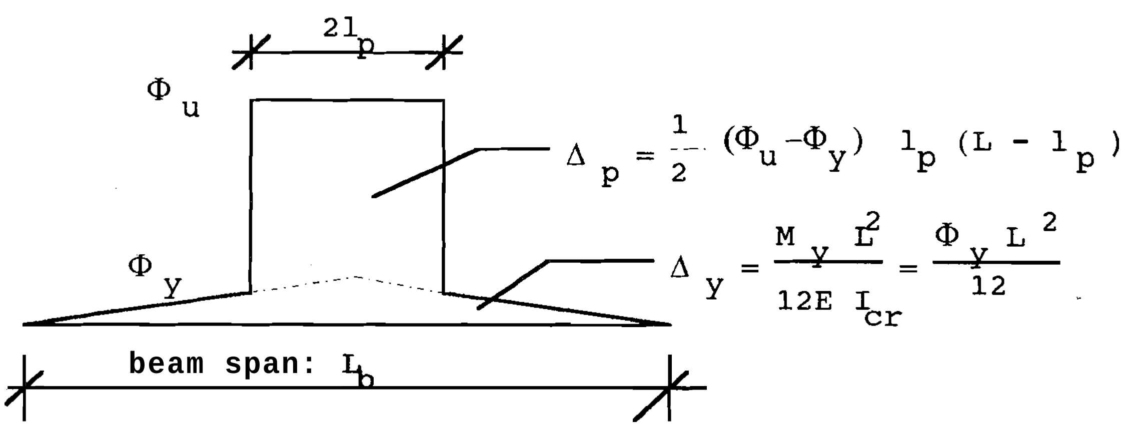
1)

$$\Delta_{total} = \Delta_y + \Delta_p = \mu_d \Delta_y$$

$$\mu_d = 1 + 3(\mu_c - 1)\frac{l_p}{H}(1 - 0.5\frac{l_p}{H})$$

I) Curvature diagram for beam member:

1) Model as simply supported member with concentrated load at midspan. Hinge forms at midspan of beam.



MOMENT AREA DIAGRAM FOR SIMPLY SUPPORTED BEAM WITH CONCENTRATED LOAD AT MIDSPAN

J) Determine curvature ductility ratio: μ_d

$$\mu_d = 1 + 6(\mu_c - 1)l_p/L(1-l_p/L)$$

K) Total deflection at centerline af beam member.

$$\Delta_{total} = \Delta_y + \Delta_p = \mu_d \Delta_y$$

L) Total barrier deflection:

1) Summation of post and beam deflection.

Reinforced Concrete Plastic Deformation Calculations For a Cantilever Post:
(AG3, F=300 kips, **WALL** THICKNESS = **22")**

Height of cantilever: $H := 84$ inches

Concrete compressive strength $fc := 4000$ psi

Reinforcing yield strength: $fy := 60000$ psi $fs := 24000$

Young's Modulus; steel $Es := 29000000$ psi

Young's Modulus; concrete $Ec := 57000 \cdot \sqrt{fc}$ $Ec = 3.605 \cdot 10^{6}$ psi

width $b := 24$ inches

thickness $h := 22$ $d := h - 3.5$ $d = 18.5$ inches

$n := \frac{Es}{Ec}$ $n = 8.044$

Tension reinf $As := 4.00$ in^2 $\rho := \frac{As}{b \cdot d}$ $\rho = 0.009$

Compression reinf. $Asc := 2.0$

$dc := 3.5$

$\beta 1 := 0.85$ $\rho c := \frac{Asc}{b \cdot d}$ $\rho h := 0.008$

Depth of compression block at ultimate strain: c

$c := (\rho - \rho c) \cdot fy \frac{d}{0.85 \cdot fc \cdot \beta 1}$ $c = 1.73$ in. Check c with cover to compression steel to see if compression steel is stressed.

Depth of compression block at first yield: kd

$k := \sqrt{(\rho + \rho c)^2 \cdot n^2 + 2 \cdot \left(\rho + \rho c \cdot \frac{dc}{d}\right) \cdot n} - (\rho + \rho c) \cdot n$ $k = 0.304$ $j := 1 - \frac{k}{3}$ $j = 0.899$

$k \cdot d = 5.627$ inches

Moment at first yield: $My := \frac{As \cdot fs \cdot j \cdot d}{12000}$ $My = 132.994$ **ft.kips**

•

Ultimate moment: $Mu := 380$ **ft.kips**

Ultimate concrete strain: $\varepsilon c := 0.003 + 0.02 \frac{b}{H} + \left(\rho h \cdot \frac{fy}{138000}\right)^2$ $\varepsilon c = 0.009$

Estimate length of plastic hinge $lp := 0.5 \cdot h$ $lp = 11$ inches

Determine curvature ductility ratio:

$$\mu c := \varepsilon c \cdot d \cdot (1 - k) \cdot \frac{Es}{c \cdot fy} \qquad \mu c = 31.382$$

Determine displacement ductility ratio:

$$\phi y := \frac{fy}{Es \cdot (1 - k) \cdot d} \qquad \phi y = 1.607 \cdot 10^{-4} \text{ radians / inch}$$

Displacement at first yield: $\Delta y := \frac{\phi y \cdot H^2}{3}$ $\Delta y = 0.378$ inches

$$\mu d := 1 + 3 \cdot (\mu c - 1) \cdot \frac{lp}{H} \cdot \left(1 - 0.5 \cdot \frac{lp}{H}\right) \qquad \mu d = 12.154$$

Total deflection $\Delta t := \mu d \cdot \Delta y$ $\Delta t = 4.595$ inches

Reinforced Concrete Plastic Deformation Calculations For a Cantilever Post:
(AG3, F=300 kips, WALL THICKNESS = 22")

Height of cantilever: $H = 84$ **inches**

Concrete compressive strength $fc = 4000$ psi

Reinforcing yield strength: $fy = 60000$ psi $fs = 24000$

Young's Modulus; steel $Es = 29000000$ psi

Young's Modulus; concrete $Ec = 57000 \cdot \sqrt{fc}$ $Ec = 3.605 \cdot 10^6$ psi

width $\mathbf{b} = 24$ inches

thickness $h = 22$ $\mathbf{d} = h - 3.5$ $\mathbf{d} = 18.5$ inches

$n := \frac{Es}{Ec}$ $n = 8.044$

Tension reinf $As := 4.00$ in^2 $p = \frac{As}{b \cdot d}$ $p = 0.009$

Compression reinf. $Asc := 2.0$

$dc := 3.5$

$\beta 1 := 0.85$ $\rho c := \frac{Asc}{b \cdot d}$ $\rho h = 0.008$

Depth of compression block at ultimate strain: c

$c = (\rho - \rho c) \cdot fy \cdot \frac{d}{0.85 \cdot fc \cdot \beta 1}$ $c = 1.73$ in. Check c with cover to compression steel to see if compression steel is stressed.

Depth of compression block at first yield: kd

$k := \sqrt{(\rho + \rho c)^2 \cdot n^2 + 2 \cdot \left(\rho + \rho c \cdot \frac{dc}{d}\right) \cdot n} - (\rho + \rho c) \cdot n$ $k = 0.301$ $j := 1 - \frac{k}{3}$ $j = 0.899$

$k \cdot d = 5.627$ inches

Moment at first yield: $\mathbf{My} = \frac{As \cdot fs \cdot j \cdot d}{12000}$ $\mathbf{My} = 132.994$ **ft.kips**

Ultimate moment: $\mathbf{Mu} = 380$ **ft.kips**

Ultimate concrete strain: $\varepsilon c = 0.003 + 0.02 \cdot \frac{b}{H} + \left(\rho h \frac{fy}{138000}\right)^2$ $\varepsilon c = 0.009$

Estimate length of plastic hinge $lp = 0.5 \cdot h$ $lp = 11$ inches

Determine curvature ductility ratio:

$$\mu c = \varepsilon c \cdot d \cdot (1 - k) \cdot \frac{Es}{c \cdot fy} \qquad \mu c = 31.382$$

Determine displacement ductility ratio:

$$\phi y = \frac{fy}{Es \cdot (1 - k) d} \qquad \phi y = 1.607 \cdot 10^{-4} \text{ radians / inch}$$

Displacement at first yield: $\Delta y := \frac{\phi y \cdot H^2}{3}$ $\Delta y = 0.378$ inches

$$\mu d := 1 + 3 \cdot (\mu c - 1) \cdot \frac{lp}{H} \cdot \left(1 - 0.5 \cdot \frac{lp}{H}\right) \qquad \mu d = 12.154$$

Total deflection $\Delta t = \mu d \cdot \Delta y$ $\Delta t = 4.595$ inches

APPENDIX E - COST ESTIMATE CALCULATIONS

Cost Estimate Calculations:
Barrier Type AG1 (At Grade Alternate 1)

Precast Concrete Wall with Precast Concrete Foundation

TABLE 3 - BARRIER DATA SCHEDULE

AT GRADE ALTERNATES - AG1, AG2

SCENARIO NO.	UNITS	AG1 TO AG2 WALL HEIGHT HT TYP	AG1 TO AG2 WALL HEIGHT HT AG5	AG1 WALL SIZE T	AG1 COLUMN SIZE B x T	AG1 FOUNDATION SIZE D	AG1 FOUNDATION SPA. L	AG1 FOUNDATION EMBED LE	AG2 WALL SIZE T	AG2 COLUMN SIZE [ENCAS.] B x T	AG2 COLUMN SIZE [STEEL] SIZE	AG2 COLUMN SIZE [STEEL] As	AG2 FOUNDATION (STEEL) SIZE	AG2 FOUNDATION SPA. L	AG2 FOUNDATION EMBED LE
1, 2	METRIC	2.29	2.44	457	457 x 457	559	4.57	4.57	457	457.2 x 457	W 254 x 27.2	11355	W 254 x 27	4.57	6.10
6, 7	US	7.50	8.00	18	18 x 18	22	15.0	15.0	18	18 x 18	W 10 x 60	17.6	W 10 x 60	15.0	20.0
3, 5, 8	METRIC	2.29	2.44	508	508 x 508	610	4.27	4.88	508	508 x 508	W 254 x 45.3	18968	W 254 x 45	4.27	7.01
10, 11*	US	7.50	8.00	20	20 x 20	24	14.0	16.0	20	20 x 20	W 10 x 100	29.4	W 10 x 100	14.0	23.0
12	METRIC	2.29	0.00	711	711 x 711	914	4.57	4.27	711	711.2 x 711	W 356 x 59.8	25032	W 356 x 60	4.27	5.18
	US	7.50	0.00	28	28 x 28	36	15.0	14.0	28	28 x 28	W 14 x 132	38.8	W 14 x 132	14.0	17.0
4, 9, 13*	METRIC	2.59	2.74	762	762 x 762	914	2.74	7.32	711	711.2 x 711	W 356 x 193	80645	W 356 x 193	4.27	13.41
	US	8.50	9.00	30	30 x 30	36	9.0	24.0	28	28 x 28	W 14 x 426	125.0	W 14 x 426	14.0	44.0

AT GRADE ALTERNATES - AG3, AG4, AG5

SCENARIO NO.	UNITS	AG3 TO AG5 WALL HEIGHT HT TYP	AG3 TO AG5 WALL HEIGHT HT AG5	AG3 WALL SIZE T	AG3 COLUMN SIZE B x T	AG3 FOUNDATION SIZE D	AG3 FOUNDATION SPA. L	AG3 FOUNDATION EMBED LE	AG4 WALL SIZE T	AG4 BEAM SIZE D	AG4 RAIL SIZE D	AG4 COLUMN SIZE D	AG4 FOUNDATION SIZE D	AG4 FOUNDATION SPA. L	AG4 FOUNDATION EMBE LE	AG5 WALL SIZE T	AG5 FOUNDATION WIDTH D*	AG5 FOUNDATION DEPTH LE
1, 2	METRIC	2.29	2.44	508	610 x 508	762	4.88	4.27	406	406	305	406	406	4.57	6.10	305	2.44	457
6, 7	US	7.50	8.00	20	24 x 20	30	16.0	14.0	16	16	12	16	16	15.0	20.0	12	8.0	18
3, 5, 8	METRIC	2.29	2.44	558.8	610 x 559	762	3.66	4.27	457	457	356	457	457	5.79	6.40	305	2.44	457
10, 11*	US	7.50	8.00	22	24 x 22	30	12.0	14.0	18	18	14	18	18	19.0	21.0	12	8.0	18
12	METRIC	2.29	0.00	711.2	762 x 711	914.4	4.57	3.96	559	559	406	559	559	4.57	5.18	0	0.00	0
	US	7.50	0.00	28	30 x 28	36	15.0	13.0	22	22	16	22	22	15.0	17.0	0	0	0
4, 9, 13*	METRIC	2.59	2.74	1016	914 x 1016	1219	3.66	6.71	610	610	457	610	610	3.05	8.84	457	2.74	610
	US	8.50	9.00	40	36 x 40	48	12.0	22.0	24	24	18	24	24	10.0	29.0	18	9.0	24

ELEVATED ALTERNATES - EL1, EL2, EL3

SCENARIO NO.	UNITS	EL1 TO EL3 WALL HEIGHT HT	EL1 WALL SIZE T	EL2 WALL SIZE T	EL3 WALL SIZE T	EL3 BEAM SIZE D	EL3 RAIL SIZE D	EL3 COLUMN SIZE D	EL3 COLUMN SPA L
14	METRIC	2.44	305	356	457	457	356	457	4.57
	US	8.00	12	14	18	18	14	18	15.0
15	METRIC	2.44	406	457	559	559	406	559	4.57
	US	8.00	16	18	22	22	16	22	15.0
16	METRIC	2.74	610	610	610	610	457	610	2.44
	US	9.00	24	24	24	24	18	24	8.0
17,18	METRIC	2.90	1016	1016	914	914	508	914	2.74
	US	9.50	40	40	36	36	20	36	9.0

UNITS:

HT = METERSFEET
T = MILIMETERSIINCHES
B = MILIMETERS/INCHES
*D = MILIMETERSIINCHESFOR AG1 TO AG4, METERSFEET FOR AG5
L = METERSIFEET
LE = METERSIFEET
STEEL SIZE = MILIMETERSxKILOGRAMS/INCHESxPOUNDS
As = SQUARE MILIMETERS/SQUARE INCHES

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Note: Refer to Table 2-1 for Description of Scenario Numbers

AG1 - COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS: FOUNDATION: SIZE	$/UNIT	L	LE	N	TL	$/HL	COLUMNS: B	T	$/UNIT	L	N	A	HT	V	$/HL
1, 2	METRIC	558.8	95.12	4.57	4.57	220	1,005	95.56	457	457	988	4.57	220	0.21	2.29	105	103.76
6, 7	US	22	29.00	15	15	353	5,295	29.08	18.0	18.0	756	15	353	2.25	7.5	221	31.59
3, 5, 8	METRIC	609.6	102.66	4.27	4.88	235	1,148	117.83	508	508	988	4.27	235	0.26	2.29	139	137.21
10, 11	US	24	31.30	14	16	378	6,050	35.87	20.0	20.0	756	14	378	2.78	7.5	292	41.78
12	METRIC	914.4	153.83	4.57	4.27	220	938	144.23	711	711	988	4.57	220	0.51	2.29	254	251.07
	US	36	46.90	15	14	353	4,942	43.90	28.0	28.0	756	15	353	5.44	7.5	534	76.44
4, 9, 13	METRIC	914.4	153.83	2.74	7.32	365	2,674	411.34	762	762	988	2.74	365	0.58	2.59	550	543.42
	US	36	46.90	9	24	588	14,104	125.28	30.0	30.0	756	9	588	6.25	8.5	1,156	165.56

FOUNDATION
SIZE IN MILIMETERS/INCHES
L= SPACING IN METERS (FEET)
N= PILES PER KILOMETER (MILES)
LE= EMBEDDED LENGTH IN METERS (+6") (FEET)
TL= TOTAL PILE LENGTH IN METERS (FEET)
$/UNIT= DOLLARS PER LINEAR METER (FOOT)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

COLUMNS
B= COLUMN WIDTH IN MM (FEET)
T= COLUMN DEPTH IN MM (FEET)
L= SPACING IN METERS (FEET)
N= COLUMNS PER KILOMETER (MILES)
A= COLUMN AREA IN SQ. METERS (SQ. FEET)
HT= COLUMN HEIGHT IN METERS (FEET)
V= VOLUME IN CUBIC METERS (CUBIC YARD)
$/UNIT= $ PER CUBIC METER (CUBIC YARD)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

SCENARIO NO.	UNITS	MATERIALS: WALL PANELS: T	$/UNIT	N	PL	V	$/HL	GROUT: N COL	$/COL	$/HL	EXPENDABLES: N COL	$/COL	$/HL	MISC.: N COL	$/COL	$/HL	TOTAL $/HL
1, 2	METRIC	457.2	261	219	4.12	941	245.99	220	50.00	10.98	220	10.00	2.20	220	100.00	21.97	480.45
6, 7	US	18.00	200	352	13.50	1,980	75.00	353	50.00	3.34	353	10.00	0.67	353	100.00	6.69	146.37
3, 5, 8	METRIC	508.0	261	234	3.76	1,023	267.54	235	50.00	11.76	235	10.00	2.35	235	100.00	23.53	560.22
10, 11	US	20.00	200	377	12.33	2,153	81.57	378	50.00	3.58	378	10.00	0.72	378	100.00	7.16	170.67
12	METRIC	711.2	261	219	3.86	1,373	359.04	220	50.00	10.98	220	10.00	2.20	220	100.00	21.97	789.49
	US	28.00	200	352	12.67	2,890	109.47	353	50.00	3.34	353	10.00	0.67	353	100.00	6.69	240.50
4, 9, 13	METRIC	762.0	261	364	1.98	1,426	372.89	365	50.00	18.27	365	10.00	3.65	365	100.00	36.54	1,386.13
	US	30.00	200	587	6.50	3,001	113.68	588	50.00	5.57	588	10.00	1.11	588	100.00	11.13	422.33

WALL PANELS
T= WALL THICKNESS IN METERS (FEET)
$/UNIT= $ PER CUBIC METER (CUBIC YARD)
N= PANELS PER KILOMETER (MILE)
PL= PANEL LENGTHS
V= VOLUME IN CUBIC METERS (CUBIC YARDS)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

MISCELLANEOUS ITEMS
$ PER METER (FOOT)
YHL PER METER (FOOT)

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Note: Refer to Table 2-1 for Description of Scenario Numbers

AG1 - COST ESTIMATE

SCENARIO NO.	UNITS	LABOR: DRIVE PILES P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	SET COLUMNS D	C.S.	H.R	C.C	$/HL
1, 2	METRIC	8	80%	6.4	220	34.32	8	34.30	2,195	75.35	34.32	7	38.45	2,153	73.90
6, 7	US	8	80%	6.4	353	55.16	8	34.30	2,195	22.93	55.16	7	38.45	2,153	22.49
3, 5, 8	METRIC	8	80%	6.4	235	36.76	8	34.30	2,195	80.70	36.76	7	38.45	2,153	79.16
10, 11	US	8	80%	6.4	378	59.08	8	34.30	2,195	24.56	59.08	7	38.45	2,153	24.09
12	METRIC	8	80%	6.4	220	34.32	8	34.30	2,195	75.35	34.32	7	38.45	2,153	73.90
	US	8	80%	6.4	353	55.16	8	34.30	2,195	22.93	55.16	7	38.45	2,153	22.49
4.9.13	METRIC	6	80%	4.8	365	76.13	8	34.30	2,195	167.13	76.13	7	38.45	2,153	163.93
	US	6	80%	4.8	588	122.43	8	34.30	2,195	50.90	122.43	7	38.45	2.153	49.93

SCENARIO NO.	UNITS	LABOR: GROUT COLUMNS D	C.S	H.R	C.C	$/HL	SET WALL PANELS W.P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	TOTAL $/HL
1, 2	METRIC	34.32	4	32.65	1,045	35.86	18	80%	14.4	219	15.19	10	38.60	3088	46.89	232.00
6, 7	US	55.16	4	32.65	1,045	10.91	18	80%	14.4	352	24.44	10	38.60	3088	14.30	70.64
3, 5, 8	METRIC	36.76	4	32.65	1,045	38.41	18	80%	14.4	234	16.27	10	38.60	3088	50.24	248.51
10, 11	US	59.08	4	32.65	1,045	11.69	18	80%	14.4	377	26.19	10	38.60	3088	15.32	75.67
12	METRIC	34.32	4	32.65	1,045	35.86	18	80%	14.4	219	15.19	10	38.60	3088	46.89	232.00
	US	55.16	4	32.65	1,045	10.91	18	80%	14.4	352	24.44	10	38.60	3088	14.30	70.64
4, 9, 13	METRIC	76.13	4	32.65	1,045	79.55	18	80%	14.4	364	25.31	10	38.60	3088	78.15	488.76
	US	122.43	4	32.65	1,045	24.23	18	80%	14.4	587	40.74	10	38.60	3088	23.83	148.88

DRIVE PILES. SET COLUMNS, GROUT COLUMN & WALL PANELS

P= PILES PER DAY
EF= EFFICIENCY FACTOR. %
R= RATE. PILES PER DAY
N= NUMBER OF PILES PER KILOMETER (MILE)
D= DURATION, DAYS

CS= **CREW SIZE**
HR= HOURLY RATE. $ PER HOUR
CC= CREW COST. $
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)
W.P.= WALLS SET PER DAY

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG1 - COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT: PILE DRIVING D	EC	TC	SET COLUMNS D	EC	TC	GROUT COLUMNS D	EC	TC	SET PANELS D	EC	TC	MISC. & SMALL TOOLS D	EC	TC	TOTAL
1, 2	METRIC	34.32	1,910	65.56	34.32	800	27.46	34.32	145	4.98	15.19	2,208	33.53	34.32	125	4.29	135.81
6, 7	US	55.16	1,910	19.95	55.16	800	8.36	55.16	145	1.51	24.44	2,208	10.22	55.16	125	1.31	41.35
3, 5, 8	METRIC	36.76	1,910	70.22	36.76	800	29.41	36.76	145	5.33	16.27	2,208	35.92	36.76	125	4 60	145.48
10, 11	US	59.08	1,910	21.37	59.08	800	8.95	59.08	145	1.62	26.19	2,208	10.95	59.08	125	1.40	44.30
12	METRIC	34.32	1,910	65.56	34.32	800	27.46	34.32	145	4.98	15.19	2,208	33.53	34.32	125	4.29	135.81
	US	55.16	1,910	19.95	55.16	800	8.36	55.16	145	1,51	24.44	2,208	10.22	55.16	125	1.31	41.35
4, 9, 13	METRIC	76.13	1,910	145.42	76.13	800	60.91	76.13	145	11.04	25.31	2,208	55.88	76.13	125	9.52	282.76
	US	122.43	1,910	44.29	122.43	800	18.55	122.43	145	3.36	40.74	2,208	17.04	122.43	125	2.90	86.14

EQUIPMENT COSTS
D= DURATION OF WORK
EC= EQUIPMENT COST IN $/DAY
TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	MISCELLANEOUS ITEMS: FP	RPLI	CMD	1	2	3	TOTAL
1, 2	27,968	21,753	31,075	0	0	0	80.80
6, 7	45,000	35,000	50,000				24.62
3, 5, 8	29,832	21,753	37,290	0	0	0	88.88
10, 11	48,000	35,000	60,000				27.08
12	27,968	21,753	40,398	0	0	0	90.12
	45,000	35,000	65,000				27.46
4, 9, 13	46,613	21,753	71,473	0	0	0	139.84
	75,000	35,000	115,000				42.61

SCENARIO NO.		TOTAL COST SUMMARY FOR AG1: MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
1, 2	$/M	480.45	232.00	80.80	135.81	929.06	185.81	1,114.87
6, 7	$/FOOT	146.37	70.64	24.62	41.35	282.98	56.60	339.57
3, 5, 8	$/M	560.22	248.51	88.88	145.48	1,043.09	208.62	1,251.71
10, 11	$/FOOT	170.67	75.67	27.08	44.30	317.72	63.54	381.27
12	$/M	789.49	232.00	90.12	135.81	1,247.42	249.48	1,496.90
	$/FOOT	240.50	70.64	27.46	41.35	379.95	75.99	455.94
4, 9, 13	$/M	1,386.13	488.76	139.84	282.76	2,297.49	459.50	2,756.99
	$/FOOT	422.33	148.88	42.61	86.14	699.96	139.99	839.96

FP = FLAGGING PROTECTION
RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
CMD= ,CONTRACTORS MOB & DEMO
$/LM= DOLLARS PER LINEAR METER
$/LF= DOLLARS PER LINEAR FOOT

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type AG2 (At Grade Alternate 2)

Precast Concrete Wall with Steel Pile Foundation

AG2 - COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS: STEEL PILES FOUNDATION SIZE			$/UNIT	L	N	LE	TL	WT	K	$/HL	STEEL COLUMNS SIZE			As	$/UNIT	N	HT	WT	K	$/HL
1, 2	METRIC	W	254 x	27	0.62	4.57	220	6.10	1,339	89	120	73.83	W	254 x	27.2	11354.8	0.62	220	2.29	89.2	44.8	27.68
6, 7	US	W	10	x 60	0.28	15.0	353	20.0	7,060	60	80	22.46	W	10	x 60	17.6	0.28	353	7.5	60	30.1	8.42
3, 5, 8	METRIC	W	254 x	45	0.62	4.27	235	7.01	1,650	149	245	151.56	W	254 x	45.3	18967.7	0.62	235	2.29	148.7	80.0	49.40
10, 11	US	W	10	x 100	0.28	14.0	378	23.0	8,697	100	165	46.12	W	10	x 100	29.4	0.28	378	7.5	100	53.7	15.04
12	METRIC	W	356 x	60	0.62	4.27	235	5.18	1,219	196	239	147.87	W	355.6 x	59.8	25032.2	0.62	235	2.29	196.3	105.6	65.21
	US	W	14	x 132	0.28	14.0	378	17.0	6,428	132	161	45.00	W	14	x 132	38.8	0.28	378	7.5	132	70.9	19.85
4, 9, 13	METRIC	W	356 x	193	0.62	4.27	235	13.41	3,156	634	2,000	######	W	355.6 x	193.0	80645.0	0.62	235	2.59	633.5	386.2	238.53
	US	W	14	x 426	0.28	14.0	378	44.0	16,638	426	1,342	375.87	W	14	x 426	125.0	0.28	378	8.5	426	259.3	72.61

FOUNDATION

- $/UNIT= DOLLARS PER KILOGRAM (POUND)
- L= SPACING IN METERS (FEET)
- N= PILES PER KILOMETER (MILES)
- LE= EMBEDDED LENGTH IN METERS (+6") (FEET)
- TL= TOTAL PILE LENGTH IN METERS (FEET)
- WT= WEIGHT IN KILOGRAM PER METER (POUND/FOOT)
- K= TOTAL WEIGHT PER HORIZONTAL LENGTH IN KILOGRAM PER METER (POUND PER FOOT)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL COLUMNS

- As= CROSS SEC. AREA IN SO. MILIMETERS (SO. INCHES)
- $/UNIT= DOLLARS PER KILOGRAM (POUND)
- L= SPACING IN METERS (FEET)
- N= COLUMNS PER KILOMETER (MILES)
- HT= COLUMN HEIGHT IN METERS (FEET)
- WT= WEIGHT IN KILOGRAM PER METER (POUND/FOOT)
- K= TOTAL WEIGHT PER HORIZONTAL LENGTH IN KILOGRAM PER METER (POUND PER FOOT)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

SCENARIO NO.	UNITS	MATERIALS: CONCRETE ENCASEMENT B	T	N	HT	V	$/UNIT	$/HL	BASE PLATES N	$/UNIT	$/HL	WALL PANELS $/UNIT	N	PL	V	$/HL	MISC. N COL	$/COL	$/HL	TOTAL $/HL
1, 2	METRIC	457	457	220	2.29	99.3	419	41.56	220	75	16.48	262	219	4.1	941	246.14	220	120	26.36	432.04
6, 7	US	18	18	353	7.50	208.6	320	12.64	353	75	5.01	200	352	13.5	1,980	75.00	353	120	8.02	131.57
3, 5, 8	METRIC	508	508	235	2.29	128.6	419	53.84	235	100	23.53	262	234	3.8	1,023	267.70	235	120	28.23	574.27
10, 11	US	20	20	378	7.50	270.3	320	16.38	378	75	5.37	200	377	12.3	2,153	81.57	378	120	8.59	173.08
12	METRIC	711	711	235	2.29	258.7	419	108.26	235	100	23.53	262	234	3.6	1,355	354.53	235	120	28.23	727.64
	US	28	28	378	7.50	543.6	320	32.94	378	100	7.16	200	377	11.7	2,852	108.02	378	120	8.59	221.58
4, 9, 13	METRIC	711	711	235	2.59	259.2	419	108.51	235	100	23.53	262	234	3.6	1,536	401.81	235	120	28.23	2,035.74
	US	28	28	378	8.50	544.8	320	33.02	378	100	7.16	200	377	11.7	3,232	122.43	378	120	8.59	619.69

CONCRETE ENCASEMENT

- B= COLUMN WIDTH IN MILIIMETERS (INCHES)
- T= COLUMN WIDTH IN MILIIMETERS (INCHES)
- N= COLUMNS PER KILOMETER (MILES)
- HT= COLUMN HEIGHT IN METERS (FEET)
- V= VOLUME OF CONCRETE ENCASEMENT IN CUB. METERS (CUB. YARDS)
- $/UNIT = $/CUB. METER ($/CY)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

WALL PANELS

- $/UNIT= $ PER CUBIC METER (CUBIC YARD)
- N= PANELS PER KILOMETER (MILE)
- PL= PANEL LENGTHS
- V= VOLUME IN CUBIC METERS (CUBIC YARD)
- $/HL= $PER HORIZONTAL LENGTH IN METERS (FEET)

MISCELLANEOUS ITEMS

- $= $PER KILOMETER
- $/HL= $ PER MILE

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Note: Refer to Table 2-1 for Description of Scenario Numbers

AG2 - COST ESTIMATE

LABOR																
SCENARIO NO.	UNITS	DRIVE PILES									SET STEEL COLUMNS					
		P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	C	D	C.S.	H.R	C.C	$/HL
1, 2	METRIC	16	80%	12.8	220	17.16	8	34.30	2,195	37.67	15	14.64	6	38.30	1,838	26.92
6, 7	US	16	80%	12.8	353	27.58	8	34.30	2,195	11.47	15	23.53	6	38.30	1,838	8.19
3, 5, 8	METRIC	15	80%	12.0	235	19.61	8	34.30	2,195	43.04	15	15.69	6	38.30	1,838	28.84
10, 11	US	15	80%	12.0	378	31.51	8	34.30	2,195	13.10	15	25.21	6	38.30	1,838	8.78
12	METRIC	16	80%	12.8	235	18.38	8	34.30	2,195	40.35	15	15.69	6	38.30	1,838	28.84
	US	16	80%	12.8	378	29.54	8	34.30	2,195	12.28	15	25.21	6	38.30	1,838	8.78
4, 9, 13	METRIC	10	80%	8.0	235	29.41	8	34.30	2,195	64.56	15	15.69	6	38.30	1,838	28.84
	US	10	80%	8.0	378	47.27	8	34.30	2,195	19.65	15	25.21	6	38.30	1,838	8.78

LABOR																	
SCENARIO NO.	UNITS	SET CONCRETE ENCASEMENT						SET WALL PANELS									TOTAL
		E	D	C.S	H.R	C.C	$/HL	W.P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	$/HL
1, 2	METRIC	3	73.22	9	33.20	2,390	175.03	18	80%	14.40	219	15.19	10	38.60	3,088	46.89	313.44
6, 7	US	3	117.67	9	33.20	2,390	53.27	18	80%	14.40	352	24.44	10	38.60	3,088	14.30	95.42
3, 5, 8	METRIC	3	78.43	9	33.20	2,390	187.48	18	80%	14.40	234	16.27	10	38.60	3,088	50.24	338.43
10, 11	US	3	126.05	9	33.20	2,390	57.07	18	80%	14.40	377	26.19	10	38.60	3,088	15.32	103.04
12	METRIC	3	78.43	9	33.20	2,390	187.48	18	80%	14.40	234	16.27	10	38.60	3,088	50.24	335.74
	US	3	126.05	9	33.20	2,390	57.07	18	80%	14.40	377	26.19	10	38.60	3,088	15.32	102.22
4, 9, 13	METRIC	3	78.43	9	33.20	2,390	187.48	18	80%	14.40	234	16.27	10	38.60	3,088	50.24	359.95
	US	3	126.05	9	33.20	2,390	57.07	18	80%	14.40	377	26.19	10	38.60	3,088	15.32	109.59

DRIVE PILES, SET COLUMNS, GROUT COLUMN & WALL PANELS

P= PILES PER DAY
EF= EFFICIENCY FACTOR, %
R= RATE, PILES PER DAY
N= NUMBER OF PILES PER KILOMETER (MILE)
D= DURATION, DAYS
E=, ENCASEMENT PER DAY

C= COLUMNS PER DAY
CS= CREW SIZE
HR= HOURLY RATE, $ PER HOUR
CC= CREW COST, $ PER DAY
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)
W.P.= WALLS SET PER DAY

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG2 - COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT: PILE DRIVING D	EC	TC	SET COLUMNS D	EC	TC	COLUMN ENCASEMENT D	EC	TC	SET PANELS D	EC	TC	MISC. & SMALL TOOLS D	EC	TC	TOTAL
1, 2	METRIC	17.16	1,910	32.78	14.64	1,160	16.99	73.22	860	62.97	15.19	2,208	33.53	73.22	125	9.15	155.42
6, 7	US	27.58	1,910	9.98	23.53	1,160	5.17	117.67	860	19.17	24.44	2,208	10.22	117.67	125	2.79	47.32
3, 5, 8	METRIC	19.61	1,910	37.45	15.69	1,160	18.20	78.43	860	67.45	16.27	2,208	35.92	78.43	125	9.80	168.82
10, 11	US	31.51	1,910	11.40	25.21	1,160	5.54	126.05	860	20.53	26.19	2,208	10.95	126.05	125	2.98	51.40
12	METRIC	18.38	1,910	35.11	15.69	1,160	18.20	78.43	860	67.45	16.27	2,208	35.92	78.43	125	9.80	166.48
	US	29.54	1,910	10.69	25.21	1,160	5.54	126.05	860	20.53	26.19	2,208	10.95	126.05	125	2.98	50.69
4, 9, 13	METRIC	29.41	1,910	56.17	15.69	1,160	18.20	78.43	860	67.45	16.27	2,208	35.92	78.43	125	9.80	187.55
	US	47.27	1,910	17.10	25.21	1,160	5.54	126.05	860	20.53	26.19	2,208	10.95	126.05	125	2.98	57.10

EQUIPMENT COSTS

D= DURATION OF WORK

EC= EQUIPMENT COST IN $/DAY

TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	UNITS	MISCELLANEOUS ITEMS: FP	RPLI	CMD	1	TOTAL
1, 2	METRIC	46,613	24,860	27,968	0	99.44
6, 7	US	75,000	40,000	45,000		30.30
3, 5, 8	METRIC	46,613	24,860	31,075	0	102.55
10, 11	US	75,000	40,000	50,000		31.25
12	METRIC	46,613	24,860	37,290	0	108.76
	US	75,000	40,000	60,000		33.14
4, 9, 13	METRIC	46,613	24,860	71,473	0	142.95
	US	75,000	40,000	######		43.56

FP = FLAGGING PROTECTION

RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE

CMD= CONTRACTORS MOB 6 DEMO

$/LM= DOLLARS PER LINEAR METER

$/LF= DOLLARS PER LINEAR FOOT

TOTAL COST SUMMARY FOR AG2: SCENARIO NO.		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
1, 2	$/M.	432.04	313.44	99.44	155.42	1,000.34	200.07	1,200.40
6, 7	$/FOOT	131.57	95.42	30.30	47.32	304.61	60.92	365.54
3, 5, 8	$/M.	574.27	338.43	102.55	168.82	1,184.07	236.81	1,420.89
10, 11	$/FOOT	173.08	103.04	31.25	51.40	358.77	71.75	430.53
12	$/M.	727.64	335.74	108.76	166.48	1,338.63	267.73	1,606.36
	$/FOOT	221.58	102.22	33.14	50.69	407.63	81.53	489.16
4, 9, 13	$/M.	######	359.95	142.95	187.55	2,726.19	545.24	3,271.43
	$/FOOT	619.69	109.59	43.56	57.10	829.94	165.99	995.93

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type AG3 (At Grade Alternate 3)

Precast Concrete Wall with Precast Concrete Foundation

AG3 - COST ESTIMATE

MATERIALS															
SCENARIO NO.	UNITS	CONCRETE CAISSONS FOUNDATION							CAST-IN-PLACE CONCRETE COLUMNS						
		D	$/UNIT	L	N	LE	TL	$/HL	B	T	$/UNIT	N	HT	V	$/HL
1, 2	METRIC	762	88.6	4.88	206	4.27	879	77.87	610	508	458	206	2.29	145.9	66.74
6, 7	US	30	27.0	16.0	331	14.0	4,634	23.70	24	20	350	331	7.50	306.5	20.32
3, 5, 8	METRIC	762	88.6	3.66	274	4.27	1,171	103.70	610	559	458	274	2.29	213.7	97.76
10, 11	US	30	27.0	12.0	441	14.0	6,174	31.57	24	22	350	441	7.50	449.2	29.77
12	METRIC	914	124.6	4.57	220	3.96	871	108.52	762	711	458	220	2.29	272.2	124.54
	US	36	38.0	15.0	353	13.0	4,589	33.03	30	28	350	353	7.50	572.0	37.92
4, 9, 13	METRIC	1219	209.9	3.66	274	6.71	1,840	386.26	914	1016	458	274	2.59	660.5	302.18
	US	48	64.0	12.0	441	22.0	9,702	117.60	36	40	350	441	8.50	1,388.3	92.03

FOUNDATION
D= CAISSONS DIAMETER IN INCHES (MM)
$/UNIT= DOLLARS PER METER (FOOT)
L= SPACING IN METERS (FEET)
N= CAISSONS PER KILOMETER (MILES)
LE= EMBEDDED LENGTH IN METERS (+6") (FEET)
TL= TOTAL CAISSONS LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

CAST-INPLACE CONCRETE COLUMNS
B= COLUMN WIDTH IN MILIMETERS (INCHES)
T= COLUMN WIDTH IN MILIMETERS (INCHES)
$/UNIT= DOLLARS PER CUB. METER (CUBIC YARD)
L= SPACING IN METERS (FEET)
N= COLUMNS PER KILOMETER (MILES)
HT= COLUMN HEIGHT IN METERS (FEET)
V= VOLUME OF CONCRETE IN CUB. METERS (CY)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

MATERIALS												
SCENARIO NO.	UNITS	WALL PANELS							MISC.			TOTAL
		HT	T	$/UNIT	N	PL	V	$/HL	L.S.	$	$/HL	$/HL
1, 2	METRIC	2.29	508	360	205	4.3	1,016	365.61	0	0	0	510.21
6, 7	US	7.5	20	275	330	14.0	2,139	111.40	0	0	0	155.41
3, 5, 8	METRIC	2.29	559	360	274	3.0	1,069	384.42	0	0	0	585.89
10, 11	US	7.5	22	275	441	10.0	2,246	116.97	0	0	0	178.32
12	METRIC	2.29	711	360	220	3.8	1,361	489.71	0	0	0	722.77
	US	7.5	28	275	353	12.5	2,860	148.96	0	0	0	219.90
4, 9, 13	METRIC	2.59	1016	360	274	2.7	1,982	712.96	0	0	0	1,401.39
	US	8.5	40	275	441	9.0	4,165	216.93	0	0	0	426.56

WALL PANELS
HT= PANEL HEIGHT IN METERS (FEET)
T= WALL THICKNESS IN METERS (FEET)
$/UNIT= $ PER CUBIC METER (CUBIC YARD)
N= PANELS PER KILOMETER (MILE)
PL= PANEL LENGTHS
V= VOLUME IN CUBIC METERS (CUBIC YARDS)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

MISCELLANEOUS ITEMS
$= $ PER KILOMETER
$/HL= $ PER MILE

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG3 - COST ESTIMATE

LABOR																	
SCENARIO NO.	UNITS	INSTALL CAISSONS									SET CAST-IN-PLACE CONCRETE COLUMNS AND PANELS						TOTAL
		C	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	V	C.D.	D	C.S	H.R	C.C	$/HL
1, 2	METRIC	5	80%	4.0	206	51.50	7	30.00	1,680	86.52	1,162	19	60.8	13	32.40	3,370	204.78	291.30
6, 7	US	5	80%	4.0	331	82.75	7	30.00	1,680	26.33	2,445	25	97.8	13	32.40	3,370	62.42	88.75
3, 5, 8	METRIC	5	80%	4.0	274	68.58	7	30.00	1,680	115.22	1,282	19	67.1	13	32.40	3,370	225.95	341.17
10, 11	US	5	80%	4.0	441	110.25	7	30.00	1,680	35.08	2,695	25	107.8	13	32.40	3,370	68.80	103.88
12	METRIC	5	80%	4.0	220	54.92	7	30.00	1,680	92.26	1,634	23	71.2	13	32.40	3,370	239.86	332.12
	US	5	80%	4.0	353	88.25	7	30.00	1,680	28.08	3,432	30	114.4	13	32.40	3,370	73.01	101.09
4, 9, 13	METRIC	4	80%	3.2	274	85.73	7	30.00	1,680	144.03	2,643	31	86.4	13	32.40	3,370	290.99	435.02
	US	4	80%	3.2	441	137.81	7	30.00	1,680	43.85	5,553	40	138.8	13	32.40	3,370	88.60	132.45

INSTALL CAISSONS, COLUMNS. & WALL PANELS
C= CAISSONS PER DAY
EF= EFFICIENCY FACTOR, %
R= RATE. CAISSONS PER DAY
N= NUMBER CAISSONS PER KILOMETER (MILE)
D= DURATION. DAYS

CS= CREW SIZE
HR= HOURLY RATE, $ PER HOUR
CC= CREW COST, $ PER DAY
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)
V= TOTAL VOLUME OF CONCRETE IN CY/CM
C.D.= VOLUME OF CONCRETE PLACED PER DAY

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG3 - COST ESTIMATE

EQUIPMENT											
SCENARIO NO.	UNITS	INSTALL CAISSONS			SET COLUMNS AND PANELS			MISC. & SMALL TOOLS			TOTAL
		D	EC	TC	D	EC	TC	D	EC	TC	$/HL
1, 2	METRIC	51.50	2,139	110.16	60.77	910	55.30	60.77	78	4.74	170.20
6, 7	US	82.75	2,139	33.52	97.81	910	16.86	97.81	78	1.44	51.83
3, 5, 8	METRIC	68.58	2,139	146 70	67.05	910	61.02	68.58	78	5.35	213.07
10, 11	US	110.25	2,139	44.66	107.80	910	18.58	110.25	78	1.63	64.87
12	METRIC	54.92	2,139	117.47	71.18	910	64.78	71.18	78	5.55	187.80
	US	88.25	2,139	35.75	114.40	910	19.72	114.40	78	1.69	57.16
4, 9, 13	METRIC	85.73	2,139	183.37	86.36	910	78.59	86.36	78	6.74	268.70
	US	137.81	2,139	55.83	138.83	910	23.93	138.83	78	2.05	81.81

EQUIPMENT COSTS
D= DURATION OF WORK
EC= EQUIPMENT COST IN $/DAY
TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	UNITS	MISCELLANEOUS ITEMS						
		FP	RPLI	CMD	1	2	3	TOTAL
1, 2	METRIC	37,290	21,753	31,075	0	0	0	90.12
6, 7	US	60,000	35,000	50,000				27.46
3, 5, 8	METRIC	40,398	21,753	37,290	0	0	0	99.44
10, 11	US	65,000	35,000	60,000				30.30
12	METRIC	42,884	21,753	40,398	0	0	0	105.03
	US	69,000	35,000	65,000				32.01
4, 9, 13	METRIC	52,828	21,753	80,796	0	0	0	155.38
	US	85,000	35,000	130,000				47.35

FP= FLAGGING PROTECTION
RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
CMD= CONTRACTORS MOB & DEMO
$/LM= DOLLARS PER LINEAR METER
$/LF= DOLLARS PER LINEAR FOOT

TOTAL COST SUMMARY FOR AG3								
SCENARIO NO.		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
1.2	$/M.	510.21	291.30	90.12	170.20	0.00	0.00	0.00
6,7	UFOOT	155.41	88.75	27.46	51.83	0.00	0.00	0.00
3,5,8	$/M.	585.89	341.17	99.44	213.07	3,865.68	773.14	4,638.82
10, 11	$/FOOT	178.32	103.88	30.30	64.87	3,520.78	704.16	4.224.93
12	$/M.	722.77	332.12	105.03	187.80	3,936.71	787.34	4,724.06
	$/FOOT	219.90	101.09	32.01	57.16	3,542.27	708.45	4,250.73
4, 9, 13	$/M.	1,401.39	435.02	155.38	268.70	3,941.58	788.32	4,729.89
	$/FOOT	426.56	132.45	47.35	81.81	3,543.69	708.74	4,252.43

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type AG4 (At Grade Alternate 4)

Structural Steel Post and Rail with Steel Pile Foundation

AG4 - COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS: STEEL PIPE PILES FOUNDATION							STEEL COLUMNS						BASE PLATES				
		D	$/UNIT	L	N	LE	TL	$/HL	D	$/UNIT	N	HT	TL	$/HL	WT	$/UNIT	N	K	$/HL
1, 2	METRIC	406	111.19	4.57	220	6.10	1,339	148.93	406	111.19	220	2.29	502	55.85	148	0.88	220	32,539	28.73
6, 7	US	16	33.90	15.0	353	20.0	7,060	45.33	16	33.90	353	7.5	2,648	17.00	327	0.40	353	115,431	8.74
3, 5, 8	METRIC	457	176.46	5.79	174	6.40	1,112	196.17	457	176.46	174	2.29	397	70.06	148	0.88	174	25,720	22.71
10, 11	US	18	53.80	19.0	279	21.0	5,857	59.68	18	53.80	279	7.5	2,092	21.31	327	0.40	279	91,199	6.91
12	METRIC	559	259.45	4.57	220	5.18	1,139	295.39	559	259.45	220	2.29	502	130.32	148	0.88	220	32,539	28.73
	US	22	79.10	15.0	353	17.0	6,001	89.90	22	79.10	353	7.5	2,648	39.66	327	0.40	353	115,431	8.74
4, 9, 13	METRIC	610	306.68	3.05	329	8.84	2,909	892.08	610	306.68	329	2.59	853	261.47	148	0.88	329	48,735	43.03
	US	24	93.50	10.0	529	29.0	15,341	271.66	24	93.50	529	8.5	4,497	79.63	327	0.40	529	172,983	13.10

FOUNDATION
D= PIPE DIAMETER IN MM (INCHES)
$/UNIT= DOLLARS PER METER (FOOT)
L= SPACING IN METERS (FEET)
N= PILES PER KILOMETER (MILES)
LE= EMBEDDED LENGTH IN METERS (-6") (FEET)
TL= TOTAL PILE LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL COLUMNS
D= PIPE DIAMETER IN MM (INCHES)
$/UNIT= WLLARS PER METER (FOOT)
L= SPACING IN METERS (FEET)
N= PILES PER KILOMETER (MILES)
HT= COLUMN HEIGHT IN, METERS (FEET)
TL= TOTAL PILE LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

BASE PLATES
WT= WEIGHT IN KILOGRAMS (PUONDS)
$/UNIT= DOLLARS PER KILOGRAM (POUND)
N= PLATES PER KILOMETER (MILES)
K= TOTAL WEIGHT IN KILOGRAMS (POUNDS)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

SCENARIO NO.	UNITS	MATERIALS: STEEL PIPE BEAM				STEEL PIPE RAILING (INT.& BOT.)				STEEL FENDER SYSTEM						MISC.			TOTAL
		T	$/UNIT	TL	$/HL	T	$/UNIT	TL	$/HL	TH	$/UNIT	WT	HT	K	$/HL	L.S.	$	$/HL	$/HL
1, 2	METRIC	40.6	111.19	1,000	111.19	30.48	51.99	2,000	103.98	12.7	0.62	7833	2.29	227	140.59	1	32.90	32.90	622.18
6, 7	US	16	33.90	5,280	33.90	12.0	15.85	10,560	31.70	0.50	0.28	490	7.5	153	42.88	1	10.03	10.03	189.58
3, 5, 8	METRIC	45.7	176.46	1,000	176.46	35.56	65.44	2,000	130.87	12.7	0.62	7833	2.29	227	140.59	1	26.01	26.01	762.88
10, 11	US	18	53.80	5,280	53.80	14.0	19.95	10,560	39.90	0.50	0.28	490	7.5	153	42.88	1	7.93	7.93	232.40
12	METRIC	55.9	259.45	1,000	259.45	40.64	85.61	2,000	171.22	12.7	0.62	7833	2.29	227	140.59	1	32.90	32.90	1,058.59
	US	22	79.10	5,280	79.10	16.0	26.10	10,560	52.20	0.50	0.28	490	7.5	153	42.88	1	10.03	10.03	322.51
4, 9, 13	METRIC	61.0	306.68	1,000	306.68	45.72	108.24	2,000	216.48	12.7	0.62	7833	2.59	258	159.34	1	49.30	49.30	1,928.39
	US	24	93.50	5,280	93.50	18.0	33.00	10,560	66.00	0.50	0.28	490	8.5	174	48.59	1	15.03	15.03	587.52

STEEL PIPE RAILING (TOP)
T= PIPE DIAMETER IN CM (INCHES)
$/UNIT= DOLLARS PER METER (FOOT)
TL= TOTAL LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL FENDER SYSTEM
TH= PLATE THICKNESS IN MM (INCHES)
$/UNIT= DOLLARS PER KILOGRAM (POUNC)
WT= WEIGHT IN KILOGRAM PER CUBIC METER (POUNDS PER CUBIC FOOT)
HT= FENDER HEIGHT IN METERS (FEET)
K= WEIGHT PER HORIZONTAL LENGTH IN KILOGRAMS PER METER (POUNDS PER FOOT)

MISCELLANEOUS ITEMS
$= $PER LINEAR METER/LINEAR FOOT
$/HL= $PER LINEAR METER/LINEAR FOOT

6/2/94

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG4 - COST ESTIMATE

LABOR																	
SCENARIO	UNITS	DRIVE PILES									SET STEEL COLUMNS						
NO.		P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	C	D	C.S.	H.R	C.C	$/HL
1, 2	METRIC	14	80%	11.2	220	19.61	8	34.30	2,195	43.05	15	14.64	6	38.30	1,838	26.92
6, 7	US	14	80%	11.2	353	31.52	8	34.30	2,195	13.10	15	23.53	6	38.30	1,838	8.19
3, 5, 8	METRIC	12	80%	9.6	174	18.09	8	34.30	2,195	39.70	15	11.58	6	38.30	1,838	21.28
10, 11	US	12	80%	9.6	279	29.05	8	34.30	2,195	12.08	15	18.59	6	38.30	1,838	6.47
12	METRIC	14	80%	11.2	220	19.61	8	34.30	2,195	43.05	15	14.64	6	38.30	1,838	26.92
	US	14	80%	11.2	353	31.52	8	34.30	2,195	13.10	15	23.53	6	38.30	1,838	8.19
4, 9, 13	METRIC	10	80%	8.0	329	41.13	8	34.30	2,195	90.28	15	21.93	6	38.30	1,838	40.32
	US	10	80%	8.0	529	66.13	8	34.30	2,195	27.49	15	35.27	6	38.30	1,838	12.28

LABOR															
SCENARIO	UNITS	INSTALL PIPE RAILS							INSTALL FENDER SYSTEM						
NO.		N	R	D	C.S	H.R	C.C	$/HL	N	R	D	C.S	H.R	C.C	$/HL
1, 2	METRIC	219	6	36.44	9	36.20	2,606	94.99	219	6	36.44	6	38.30	1,838	67.00
6, 7	US	352	6	58.67	9	36.20	2,606	28.96	352	6	58.67	6	38.30	1,838	20.43
3, 5, 8	METRIC	173	6	28.77	9	36.20	2,606	74.99	173	6	28.77	6	38.30	1,838	52.89
10, 11	US	278	6	46.32	9	36.20	2,606	22.86	278	6	46.32	6	38.30	1,838	16.13
12	METRIC	219	6	36.44	9	36.20	2,606	94.99	219	6	36.44	6	38.30	1,838	67.00
	US	352	6	58.67	9	36.20	2,606	28.96	352	6	58.67	6	38.30	1,838	20.43
4, 9, 13	METRIC	328	6	54.67	9	36.20	2,606	142.48	328	6	54.67	6	38.30	1,838	100.50
	US	528	6	88.00	9	36.20	2,606	43.44	528	6	88.00	6	38.30	1,838	30.64

LABOR									
SCENARIO	UNITS	PAINTING STRUCTURAL STEEL							TOTAL
NO.		N	R	D	C.S.	H.R.	C.C.	$/HL	$/HL
1, 2	METRIC	219	8	27.3	3	30.95	742.80	20.30	252.27
6, 7	US	352	8	44.0	3	30.95	742.80	6.19	76.87
3, 5, 8	METRIC	173	8	21.6	3	30.95	742.80	16.03	204.90
10, 11	US	278	8	34.7	3	30.95	742.80	4.89	62.43
12	METRIC	219	8	27.3	3	30.95	742.80	20.30	252.27
	US	352	8	44.0	3	30.95	742.80	6.19	76.87
4, 9, 13	METRIC	328	8	41.0	3	30.95	742.80	30.45	404.04
	US	528	8	66.0	3	30.95	742.80	9.29	123.14

DRIVE PILES. SET COLUMNS, GROUT COLUMN & WALL PANELS

P= PILES PER DAY
EF= EFFICIENCY FACTOR. %
R= RATE, PILES PER DAY
N= NUMBER OF PILES PER KILOMETER (MILE)
D= DURATION, DAYS
CS= CREW SIZE
HR= HOURLY RATE, $ PER HOUR
CC= CREW COST, $ PER DAY
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)
W.P.= WALLS SET PER DAY

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG4 - COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT: PILE DRIVING D	PILE DRIVING EC	PILE DRIVING TC	SET COLUMNS D	SET COLUMNS EC	SET COLUMNS TC	INSTALL PIPE RAILS D	INSTALL PIPE RAILS EC	INSTALL PIPE RAILS TC	INSTALL FENDER SYSTE D	INSTALL FENDER SYSTE EC	INSTALL FENDER SYSTE TC	PAINTING D	PAINTING EC	PAINTING TC	MISC. & SMALL TOOLS D	MISC. & SMALL TOOLS EC	MISC. & SMALL TOOLS TC	TOTAL
1, 2	METRIC	19.61	1,910	37.46	14.64	1,160	16.99	36.44	1402	51.10	36.44	1,402	51.10	27.33	200	5.47	36.44	150	5.47	167.57
6, 7	US	31.52	1,910	11.40	23.53	1,160	5.17	58.67	1402	15.58	58.67	1,402	15.58	44.00	200	1.67	58.67	150	1.67	51.06
3, 5, 8	METRIC	18.09	1,910	34.55	11.58	1,160	13.43	28.77	1402	40.34	28.77	1.402	40.34	21.58	200	4.32	28.77	150	4.32	137.28
10, 11	US	29.05	1,910	10.51	18.59	1,160	4.08	46.32	1402	12.30	46.32	1,402	12.30	M.74	200	1.32	46.32	150	1.32	41.82
12	METRIC	19.61	1,910	37.46	14.64	1,160	16.99	36.44	1402	51.10	36.44	1,402	51.10	27.33	200	5.47	36.44	150	5.47	167.57
	US	31.52	1.910	11.40	23.53	1,160	5.17	58.67	1402	15.58	58.67	1.402	15.58	44.00	200	1.67	58.67	150	1.67	51.06
4,9,13	METRIC	41.13	1,910	78.55	21.93	1.160	25.44	54.67	1402	76.64	54.67	1,402	76.64	41.00	200	8.20	54.67	150	8.20	273.68
	US	66.13	1,910	23.92	35.27	1,160	7.75	88.00	1402	23.37	88.00	1,402	23.37	66.00	200	2.50	88.00	150	2.50	83.40

EQUIPMENT COSTS
D= DURATION OF WORK
EC= EQUIPMENT COST IN $/DAY
TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	UNITS	MISCELLANEOUS ITEMS: FP	RPLI	CMD	1	TOTAL
1, 2	METRIC	37,290	21,753	34,804	0	93.85
6, 7	US	60,000	35,000	56,000		28.60
3, 5, 8	METRIC	27,968	21,753	39,155	0	88.88
10, 11	US	45,000	35,000	63,000		27.08
12	METRIC	37,290	21,753	45,370	0	104.41
	US	60,000	35,000	73,000		31.82
4, 9, 13	METRIC	46,613	21,753	59,043	0	127.41
	US	75,000	35,000	95,000		38.83

FP = FLAGGING PROTECTION
RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
CMD= CONTRACTORS MOB & DEMO
$/LM= DOLLARS PER LINEAR METER
$/LF= DOLLARS PER LINEAR FOOT

TOTAL COST SUMMARY FOR AG4

SCENARIO NO.		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
1, 2	$/M.	622.18	252.27	93.85	167.57	1,135.86	227.17	1,363.04
6, 7	$/FOOT	189.58	76.87	28.60	51.06	346.11	69.22	415.33
3, 5, 8	$/M.	762.88	204.90	88.88	137.28	1,193.94	238.79	1,432.72
10, 11	$/FOOT	232.40	62.43	27.08	41.82	363.74	72.75	436.49
12	$/M.	1,058.59	252.27	104.41	167.57	1,582.85	316.57	1,899.41
	$/FOOT	322.51	76.87	31.82	51.06	482.27	96.45	578.72
4, 9, 13	$/M.	1,928.39	404.04	127.41	273.68	2,733.51	546.70	3,280.21
	$/FOOT	587.52	123.14	38.83	83.40	832.88	166.58	999.45

Note: Refer to Table 2-1 for Description of Scenario Numbers

<u>Cost Estimate Calculations:</u>

Barrier Type AG5 (At Grade Alternate ***5)***

Cast In Place Concrete Retaining Wall Barrier

AG5 - COST ESTIMATE

MATERIALS																			
SCENARIO	CAST-IN-PLACE CONCRETE RETAINING WALL									STEEL SHEETING			STRUCTURAL BACKFILL			GRANULAR BACKFILL			TOTAL
NO	HT	T	D	LE	FOOT	STE	OTAL	$/UNIT	$/HL	HT	$/UNIT	$/HL	V	$/UNIT	$/HL	V	$/UNIT	$/HL	$/HL
1, 2,	2.44	305	2.44	457	1.12	0.98	2.09	327	683.30	4.57	37.65	172.20	6.3	13.07	82.00	9.0	19.61	177.12	1,114.62
6, 7	8.00	12	8.00	18	0.44	0.39	0.83	250	208.33	15.00	3.50	52.50	2.5	10.00	25.00	3.60	15.00	54.00	339.83
3, 5, 8,	2.44	305	2.44	457	1.12	0.98	2.09	327	683.30	4.57	37.65	172.20	6.3	13.07	82.00	9.0	19.61	177.12	1,114.62
10, 11	8.00	12	8.00	18	0.44	0.39	0.83	250	208.33	15.00	3.50	52.50	2.5	10.00	25.00	3.60	15.00	54.00	339.83
12																			
4, 9, 13	2.74	457	2.74	610	1.67	1.53	3.21	327	######	4.57	37.65	172.20	6.3	13.07	82.00	10.3	19.61	201.72	1,503.65
	9.00	18	9.00	24	0.67	0.61	1.28	250	319.44	15.00	3.50	52.50	2.5	10.00	25.00	4.10	15.00	61.50	458.44

CAST-IN-PLACE CONCRETE RETAINING WALL

HT= HEIGHT OF THE WALL ABOVE GRADE IN METERS (FEET)
T= THICKNESS OF THE STEM **IN MILIMETERS** (INCHES)
D= WIDTH OF THE FOOTING IN METERS (FEET)
LE= THICKNESS OF THE FOOTING IN **MILIMETERS** (INCHES)
V FOOT= VOLUME OF CONCR. IN FOOTING IN CUB. METERSIMETER (CUB. **YARD/FO**
V STEM= VOLUME OF CONCR IN STEM IN CUB. **METERS/METER** (CUB. **YARD/FOOT)**
TOTAL V= TOTAL VOLUME OF CONCR. IN CUB. METERSIMETER **(CUB.YARD/FOOT)**
$/UNIT= DOLLARS PER CUB METER (CUBIC YARD)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL SHEETING

HT= HEIGHT OF SHEETING IN METERS (FEET)
$/UNIT= **$ PER** SQ. METER OF SHEETING (SQ. FOOT)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STRUCTURAL AND GRANULAR BACKFILL

V= VOLUME OF BACKFILL IN CUB. **M** PER L.M. (CUB. Y PER L.F.)
$/UNIT= $ PER CUB. METER (CUB YARD)
$/HL= **$ PER** HORIZONTAL LENGTH IN METERS (FEET)

Note: Refer to Table 2-1 for Description of Scenario Numbers

AG5 - COST ESTIMATE

LABOR											
SCENARIO NO	CAST-IN-PLACE CONCRETE RETAINING WALL						STEEL SHEETING				
	FOOTING			STEM							
	V	R	$/HL	V	R	$/HL	HT	R	D	DR	$/HL
1, 2,	1.12	117.19	130.68	0.98	189.54	184.95	4.57	70	0.066	2,500	164.00
6, 7	0.44	89.65	39.84	0.39	#####	56.39	15.00	750	0.020	2,500	50.00
3, 5, 8,	1.12	117.19	130.68	0.98	189.54	184.95	4.57	70	0.066	2,500	164.00
10, 11	0.44	89.65	39.84	0.39	#####	56.39	15.00	750	0.020	2,500	50.00
12											
4, 9, 13	1.67	117.19	196.02	1.53	189.54	290.64	4.57	70	0.066	2,500	164.00
	0.67	89.65	59.77	0.61	#####	88.61	15.00	750	0.020	2,500	50.00

CAST-INPLACE CONCRETE RETAINING WALL

V= VOLUME OF CONCRETE IN CUB. METER (CUB. FOOT)

R= RATE IN $ PER CUB. METER (CUB. FOOT)

$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL SHEETING

A=LxHT AREA OF SHEETING IN SQ. METERS/METR, SQ. FEET/FOOT

R= RATE IN SQ. METERS (SQ. FEET) PER DAY

D= TOTAL DAYS

DR= DAILY RATE IN $ PER DAY

$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

LABOR											
SCENARIO NO	STRUCTURAL EXCAVATION			STRUCTURAL BACKFI			GRANULAR BACKFILL			TOTAL
	V	R	$/HL	V	R	$/HL	V	$/UNIT	$/HL	$/HL
1, 2,	8.28	7.19	59.53	6.27	3.27	20.50	9.03	1.96	17.71	577.37
6, 7	3.30	5.50	18.15	2.50	2.50	6.25	3.60	1.50	5.40	176.03
3, 5, 8,	8.28	7.19	59.53	6.27	3.27	20.50	9.03	1.96	17.71	577.37
10, 11	3.30	5.50	18.15	2.50	2.50	6.25	3.60	1.50	5.40	176.03
12										
4, 9, 13	8.28	7.19	59.53	6.27	3.27	20.50	9.03	1.96	17.71	748.40
	3.30	5.50	18.15	2.50	2.50	6.25	3.60	1.50	5.40	228.18

STRUCTURAL EXCAVATION AND BACKFILLING

V= VOLUME OF BACKFILL IN CUB. M PER L.M. (CUB. Y PER L.F.)

R= $ PER CUB. METER (CUB YARD)

$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

Note: Refer to Teble 2-1 for Description of Scenario Numbers

AG5 - COST ESTIMATE

SCENARIO NO	EQUIPMENT																		
	CAST-IN-PLACE CONCRETE RETAINING WALL						STEEL SHEETING			STRUCTURAL EXCAVATION			STRUCTURAL BACKFILL			GRANULAR BACKFILL			TOTAL
	FOOTING			STEM															
	V	E.C.	$/HL	V	E.C.	$/HL	D	E.D.	$/HL	V	E.C.	$/HL	V	E.C.	$/HL	V	E.C.	$/HL	$/HL
1, 2,	1.12	44.44	49.56	0.98	50.85	49.62	0.066	2,030	133.17	8.28	9.80	81.18	6.27	4.12	25.83	9.03	4.12	37.20	376.55
6, 7	0.44	34.00	15.11	0.39	38.90	15.13	0.020	2,030	40.60	3.30	7.50	24.75	2.50	3.15	7.88	3.60	3.15	11.34	114.80
3, 5, 8,	1.12	44.44	49.56	0.98	50.85	49.62	0.066	2,030	133.17	8.28	9.80	81.18	6.27	4.12	25.83	9.03	4.12	37.20	376.55
10, 11	0.44	34.00	15.11	0.39	38.90	15.13	0.020	2,030	40.60	3.30	7.50	24.75	2.50	3.15	7.88	3.60	3.15	11.34	114.80
12																			
4, 9, 13	1.67	44.44	74.34	1.53	50.85	77.97	0.066	2,030	133.17	8.28	9.80	81.18	6.27	4.12	25.83	10.29	4.12	42.36	434.85
	0.67	34.00	22.67	0.61	38.90	23.77	0.020	2,030	40.60	2.50	7.50	18.75	3.60	3.15	11.34	4.10	3.15	12.92	130.04

EQUIPMENT COSTS
- V= VOLUME OF CONCRETE IN CUB. METERS (CUB. FEET)
- E.C.= EQUIPMENT COST IN $/DAY
- $/HL= TOTALCOST FOR EQUIPMENT PER METER (FOOT)

STEEL SHEETING
- D= TOTAL DAYS
- E.D.= EQUIPMENT COST PER DAY
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STRUCTURAL EXCAVATION AND BACKFILLING
- V= VOLUME OF BACKFILL IN CUB. M PER L.M. (CUB. Y PER L.F.)
- E.C.= EQUIPMENT COST PER CUB. METER (CUB YARD)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

SCENARIO NO	MISCELLANEOUS ITEMS						
	FP	RPLI	CMD	1	2	3	TOTAL
1, 2,	40,398	21,753	65,258				127.41
6, 7	65,000	35,000	105,000				38.83
3, 5, 8,	40,398	21,753	65,258				127.41
10, 11	65,000	35,000	105,000				38.83
12							
4, 9, 13	43,505	21,753	68,365				133.62
	70,000	35,000	110,000				40.72

- FP = FLAGGING PROTECTION
- RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
- CMD= CONTRACTORS MOB & DEMO
- $/LM= DOLLARS PER LINEAR METER
- $/LF= DOLLARS PER LINEAR FOOT

SCENARIO NO	TOTAL COST SUMMARY FOR AG5							
		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
1, 2,	$/M.	1,114.62	577.37	127.41	376.55	2,195.95	439.19	2,635.14
6, 7	$/FOOT	339.83	176.03	38.83	114.80	669.50	133.90	803.40
3, 5, 8,	$/M.	1,114.62	577.37	127.41	376.55	2,195.95	439.19	2,635.14
10, 11	$/FOOT	339.83	176.03	38.83	114.80	669.50	133.90	803.40
12	$/M.							
	$/FOOT							
4, 9, 13	$/M.	1,503.65	748.40	133.62	434.85	2,820.53	564.11	3,384.63
	$/FOOT	458.44	228.18	40.72	130.04	857.39	171.48	1,028.86

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type EL1 (Elevated Alternate 1)

Precast Concrete Wall

EL1 - COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS								TOTAL
		WALL PANELS					MISC.			
		T	HT	YUNIT	V	$/HL	L.S	$/UNIT	$/HL	$/HL
14	METRIC	305	2.44	261	0.743	194.36	1	16.40	16.40	210.76
	US	12	8.00	200	0.296	59.26	1	5.00	5.00	64.26
15	METRIC	406	2.44	261	0.991	259.14	1	16.40	16.40	275.54
	US	16	8.00	200	0.395	79.01	1	5.00	5.00	84.01
16	METRIC	610	2.74	261	1.673	437.30	1	16.40	16.40	453.70
	US	24	9.00	200	0.667	133.33	1	5.00	~~5.00~~	~~138.33~~
17.18	METRIC	1016	2.90	261	2.943	769.33	1	16.40	16.40	785.73
	US	40	9.50	200	1.173	234.57	1	5.00	5.00	239.57

WALL PANELS

T= **WALL THICKNESS** IN **MILIMETERS** (INCHES)

V= VOLUME OF CONCRETE IN CUBIC METER PER METER (CUBIC YARD PER FOOT)

$/UNIT= $ PER CUBIC METER (CUBIC YARD)

$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

MISCELLANEOUS ITEMS

$ PER METER (FOOT)

6/3/94

Note: Refer to Table 2-1 for Description of Scenario Numbers

EL1 - COST ESTIMATE

SCENARIO NO.	UNITS	LABOR					
		R	D	C.S.	H.R.	C.C.	$/HL
14	METRIC	73.2	0.01367	10	38.60	3088	42.20
	US	240.0	0.00417	10	38.60	3088	12.87
15	METRIC	73.2	0.01367	10	38.60	3088	42.20
	US	240.0	0.00417	10	38.60	3088	12.87
16	METRIC	73.2	0.01367	10	38.60	3088	42.20
	US	240.0	0.00417	10	38.60	3088	12.87
17,18	METRIC	73.2	0.01367	10	38.60	3088	42.20
	US	240.0	0.00417	10	38.60	3088	12.87

LABOR

R= RATE IN METERS (FEET) PER DAY WITH 80% EFFICIENCY

D= DAYS PER FOOT (METER)

C.S.= CREW SIZE

H.R.= HOURLY RATE

C.C.= CREW COST PER DAY

Note: Refer to Table 2-1 for Description of Scenario Numbers

EL1 - COST ESTIMATE

EQUIPMENT COST								
SCENARIO NO.	UNITS	SET PANELS			MISC. & SMALL TOOLS			TOTAL
		D	EC	TC	D	EC	TC	
14	METRIC	0.01367	2,208	30.18	0.0137	125	1.71	31.88
	US	0.00417	2,208	9.20	0.0042	125	0.52	9.72
15	METRIC	0.01367	2,208	30.18	0.0137	125	1.71	31.88
	US	0.00417	2,208	9.20	0.0042	125	0.52	9.72
16	METRIC	0.01367	2,208	30.18	0.0137	125	1.71	31.88
	US	0.00417	2,208	9.20	0.0042	125	0.52	9.72
17,18	METRIC	0.01367	2,208	30.18	0.0137	125	1.71	31.88
	US	0.00417	2,208	9.20	0.0042	125	0.52	9.72

EQUIPMENT COSTS

D= DURATION OF WORK

EC= EQUIPMENT COST IN $/DAY

TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

MISCELLANEOUS ITEMS								
SCENARIO NO.	UNITS	FP	RPLI	CMD	1	2	3	TOTAL $/HL
14	METRIC	27,968	21,753	37,290	0	0	0	87.01
	US	45,000	35,000	60,000				26.52
15	METRIC	29,832	21,753	40,398	0	0	0	91.98
	US	48,000	35,000	65,000				28.03
16	METRIC	27,968	21,753	43,505	0	0	0	93.23
	US	45,000	35,000	70,000				28.41
17,18	METRIC	37,290	21,753	49,720	0	0	0	108.76
	US	60,000	35,000	80,000				33.14

FP = FLAGGING PROTECTION PER KILOMETER (PER MILE)

RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE PER KILOMETER (PER MILE)

CMD= CONTRACTORS MOB 8 DEMO PER KILOMETER (PER MILE)

$/LM= DOLLARS PER LINEAR METER

$/LF= DOLLARS PER LINEAR FOOT

SCENARIO NO.		TOTAL COST SUMMARY FOR EL1						
		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
14	$/M	210.76	42.20	87.01	31.88	371.85	74.37	446.23
	$/FOOT	64.26	12.87	26.52	9.72	113.36	22.67	136.03
15	$/M	275.54	42.20	91.98	31.88	441.61	88.32	529.93
	$/FOOT	84.01	12.87	28.03	9.72	134.63	26.93	161.56
16	$/M	453.70	42.20	93.23	31.88	621.02	124.20	745.22
	$/FOOT	138.33	12.87	28.41	9.72	189.33	37.87	227.20
17,18	$/M	785.73	42.20	108.76	31.88	968.58	193.72	1,162.29
	$/FOOT	239.57	12.87	33.14	9.72	295.30	59.06	354.36

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type EL2 (Elevated Alternate 2)

Cast In Place Concrete Wall

EL2 - COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS								
		WALL PANELS					MISC.			TOTAL
		HT	T	$/UNIT	V	$/HL	L.S.	$	$/HL	$/HL
14	METRIC	2.44	356	327	0.867	283.61	0	0	0	283.61
	US	8.00	14	250	0.346	86.42	0	0	0	86.42
15	METRIC	2.44	457	327	1.115	364.64	0	0	0	364.64
	US	8.00	18	250	0.444	111.11	0	0	0	111.11
16	METRIC	2.74	610	327	1.673	546.97	0	0	0	546.97
	US	9.00	24	250	0.667	166.67	0	0	0	166.67
17.18	METRIC	2.90	1016	327	2.943	962.26	0	0	0	962.26
	US	9.50	40	250	1.173	293.21	0	0	0	293.21

WALL PANELS

HT= PANEL HEIGHT IN METERS (FEET)
T= WALL THICKNESS IN MILIMETERS (INCHES)
$/UNIT= $ PER CUBIC METER (CUBIC YARD)
V= VOLUME IN CUBIC METERS PER METER (CUBIC YARDS PER FOOT)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

6/3/94

Note: Refer to Table 2-1 for Description of Scenario Numbers

EL2 - COST ESTIMATE

LABOR								
SCENARIO NO.	UNITS	SET CAST-IN-PLACE CONCRETE COLUMNS AND PANELS						
		V	C.D.	D	C.S	H.R	C.C	$/HL
14	METRIC	0.867	19	0.0453	13	32.40	3.370	152.81
	US	0.346	25	0.0138	13	32.40	3,370	46.59
15	METRIC	1.115	19	0.0583	13	32.40	3,370	196.47
	US	0.444	25	0.0178	13	32.40	3,370	59.90
16	METRIC	1.673	19	0.0875	13	32.40	3,370	294.71
	US	0.667	25	0.0267	13	32.40	3,370	89.86
17.18	METRIC	2.943	19	0.1539	13	32.40	3,370	518.47
	US	1.173	25	0.0469	13	32.40	3,370	158.08

INSTALL CAST-IN-PLACE WALL PANELS

C.D = VOLUME OF CONCRETE PLACED PER DAY

D= DURATION, DAYS

CS= CREW SIZE

HR= HOURLY RATE, $ PER HOUR

CC= CREW COST, $ PER DAY

$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

Note: Refer to Table 2-1 for Description of Scenario Numbers

EL2 - COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT						
		SET PANELS			MISC. & SMALL TOOLS			TOTAL
		D	EC	TC	D	EC	TC	$/HL
14	METRIC	0.0453	2,139	97.00	0.0453	78	3.54	100.54
	US	0.0138	2,139	29.58	0.0138	78	1.08	30.65
15	METRIC	0.0583	2,139	124.72	0.0583	78	4.55	129.27
	US	0.0178	2,139	38.03	0.0178	78	1.39	39.41
16	METRIC	0.0875	2,139	187.08	0.0875	78	6.82	193.90
	US	0.0267	2,139	57.04	0.0267	78	2.08	59.12
17,18	METRIC	0.1539	2,139	329.12	0.1539	78	12.00	341.12
	US	0.0469	2,139	100.35	0.0469	78	3.66	104.01

SCENARIO NO.	UNITS	MISCELLANEOUS ITEMS						
		FP	RPLI	CMD	1	2	3	TOTAL
14	METRIC	37,290	21,753	31.075	0	0	0	90.12
	US	60,000	35,000	50,000				27.46
15	METRIC	40,398	21,753	37,290	0	0	0	99.44
	US	65,000	35,000	60,000				30.30
16	METRIC	42,884	21,753	40,398	0	0	0	105.03
	US	69,000	35,000	65,000				32.01
17,18	**METRIC**	52,828	21,753	80,796	0	0	0	155.38
	US	85,000	35,000	130,000				47.35

FP= FLAGGING PROTECTION
RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
CMD= CONTRACTORS MOB & DEMO
$/LM= DOLLARS PER LINEAR METER
$/LF= DOLLARS PER LINEAR FOOT

SCENARIO NO.	UNITS	MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
14	$/M.	283.61	152.81	90.12	100.54	627.08	125.42	752.50
	$/FOOT	86.42	46.59	27.46	30.65	191.13	38.23	229.35
15	$/M.	364.64	196.47	99.44	129.27	789.82	157.96	947.79
	$/FOOT	111.11	59.90	30.30	39.41	240.73	48.15	288.88
16	$/M.	546.97	294.71	105.03	193.90	1,140.61	228.12	1,368.73
	$/FOOT	166.67	89.86	32.01	59.12	347.65	69.53	417.18
17,18	$/M.	962.26	518.47	155.38	341.12	1,977.22	395.44	2,372.66
	$/FOOT	293.21	158.08	47.35	104.01	602.65	120.53	723.17

TOTAL COST SUMMARY FOR EL2 (table title above)

Note: Refer to Table 2-1 for Description of Scenario Numbers

Cost Estimate Calculations:

Barrier Type EL3 (Elevated Alternate 3)

Structural Steel Post and Railing

EL3 - COST ESTIMATE

MATERIALS																	
SCENARIO NO.	UNITS	STEEL COLUMNS							BASE PLATES					STEEL PIPE BEAM			
		D	L	$/UNIT	N	HT	TL	$/HL	WT	$/UNIT	N	K	$/HL	T	$/UNIT	TL	$/HL
14	METRIC	457	4.57	176.46	220	2.44	536	94.54	148	0.88	220	32,539	28.73	457	176.46	1,000	176.46
	US	18	15.00	53.80	353	8.0	2,824	28.77	327	0.40	353	115,431	8.74	18	53.80	5,280	53.80
15	METRIC	559	4.57	259.45	220	2.44	536	139.01	202	0.88	220	44,282	39.10	559	259.45	1,000	259.45
	US	22	15.00	79.10	353	8.0	2,824	42.31	445	0.40	353	157,085	11.90	22	79.10	5,280	79.10
16	METRIC	610	2.44	306.68	411	2.74	1,128	345.86	231	0.88	411	94,953	83.84	610	306.68	1,000	306.68
	US	24	8.00	93.50	661	9.0	5,949	105.35	510	0.40	661	337,110	25.54	24	93.50	5,280	93.50
17,18	METRIC	914	2.74	433.29	365	2.90	1,058	458.61	566	0.88	365	206,933	182.72	914	433.29	1,000	433.29
	US	36	9.00	132.10	588	9.5	5,583	139.68	1,250	0.40	588	734,583	55.65	36	132.10	5,280	132.10

STEEL COLUMNS

D= PIPE DIAMETER IN MM (INCHES)
$/UNIT= DOLLARS PER METER (FOOT)
L= SPACING IN METERS (FEET)
N= PILES PER KILOMETER (MILES)
HT= COLUMN HEIGHT IN METERS (FEET)
TL= TOTAL PILE LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

BASEPLATES

WT= WEIGHT IN KILOGRAMS (POUNDS)
$/UNIT= DOLLARS PER KILOGRAM (POUND)
N= PLATES PER KILOMETER (MILES)
K= TOTAL WEIGHT IN KILOGRAMS (POUNDS)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

STEEL PIPE BEAM AND RAILING

T= PIPE DIAMETER IN MM (INCHES)
$/UNIT= DOLLARS PER METER (FOOT)
TL= TOTAL LENGTH IN METERS (FEET)
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

MATERIALS															
SCENARIO NO.	UNITS	STEEL PIPE RAILING (INT.& BOT.)				STEEL FENDER SYSTEM						MISC.			TOTAL
		T	$/UNIT	TL	$/HL	TH	$/UNIT	WT	HT	K	$/HL	L.S.	$	$/HL	$/HL
14	METRIC	356	65.44	2,000	130.87	12.7	0.62	7833	2.44	243	149.97	1	36.24	36.24	616.82
	US	14	19.95	10,560	39.90	0.50	0.28	490	8.0	163	45.73	1	11.05	11.05	188.00
15	METRIC	406	85.61	2,000	171.22	12.7	0.62	7833	2.44	243	149.97	1	37.23	37.23	795.96
	US	16	26.10	10,560	52.20	0.50	0.28	490	8.0	163	45.73	1	11.35	11.35	242.59
16	METRIC	457	108.24	2,000	216.48	12.7	0.62	7833	2.74	273	168.71	1	43.13	43.13	1,164.71
	US	18	33.00	10,560	66.00	0.50	0.28	490	9.0	184	51.45	1	13.15	13.15	354.99
17.18	METRIC	508	140.22	2,000	280.44	12.7	0.62	7833	290	288	178.09	1	45.59	45.59	1,578.74
	US	.20	4275	10,560	85.50	0.50	0.28	490	9.5	194	54.31	1	13.90	13.90	. 481.14

STEEL FENDER SYSTEM

TH= PLATE THICKNESS IN MM (INCHES)
$/UNIT= DOLLARS PER KILOGRAM (POUND)
WT= WEIGHT IN KILOGRAM PER CUBIC METER (POUNDS PER CUBIC FOOT)
HT= FENDER HEIGHT IN METERS (FEET)
K= WEIGHT PER HORIZONTAL LENGTH IN KILOGRAMS PER METER (POUNDS PER FOOT)

MISCELLANEOUS ITEMS

$= $PER LINEAR METER/LINEAR FOOT
$/HL= $PER LINEAR METERILINEAR FOOT

Note: Refer to Table 2-1 for Description of Scenario Numbers

EL3 - COST ESTIMATE

LABOR														
SCENARIO NO.	UNITS	SET STEEL COLUMNS						INSTALL PIPE RAILS						
		C	D	C.S.	H.R	C.C	$/HL	N	R	D	C.S	H.R	C.C	$/HL
14	METRIC	15	14.64	6	38.30	1,838	26.92	219	6	36.44	9	36.20	2,606	94.99
	US	15	23.53	6	38.30	1,838	8.19	352	6	58.67	9	36.20	2,606	28.96
15	METRIC	15	14.64	6	38.30	1,838	26.92	219	6	36.44	9	36.20	2,606	94.99
	US	15	23.53	6	38.30	1,838	8.19	352	6	58.67	9	36.20	2,606	28.96
16	METRIC	15	27.40	6	38.30	1,838	50.37	410	6	68.33	9	36.20	2,606	178.10
	US	15	44.07	6	38.30	1,838	15.34	660	6	110.00	9	36.20	2,606	54.30
17,18	METRIC	15	24.36	6	38.30	1,838	44.79	364	6	60.74	9	36.20	2,606	158.31
	US	15	39.18	6	38.30	1,838	13.64	587	6	97.78	9	36.20	2,606	48.27

LABOR																
SCENARIO NO.	UNITS	INSTALL FENDER SYSTEM							PAINTING STRUCTURAL STEEL							TOTAL
		N	R	D	C.S	H.R	C.C	$/HL	N	R	D	C.S.	H.R.	C.C.	$/HL	$/HL
14	METRIC	219	6	36.44	6	38.30	1,838	67.00	219	8	27.3	3	30.95	742.80	20.30	209.21
	US	352	6	58.67	6	38.30	1,838	20.43	352	8	44.0	3	30.95	742.80	6.19	63.77
15	METRIC	219	6	36.44	6	38.30	1,838	67.00	219	8	27.3	3	30.95	742.80	20.30	209.21
	US	352	6	58.67	6	38.30	1,838	20.43	352	8	44.0	3	30.95	742.80	6.19	63.77
16	METRIC	410	6	68.33	6	38.30	1,838	125.62	410	8	51.3	3	30.95	742.80	38.07	392.17
	US	660	6	110.00	6	38.30	1,838	38.30	660	8	82.5	3	30.95	742.80	11.61	119.55
17,18	METRIC	364	6	60.74	6	38.30	1,838	111.67	364	8	45.6	3	30.95	742.80	33.84	348.61
	US	587	6	97.78	6	38.30	1,838	34.04	587	8	73.3	3	30.95	742.80	10.32	106.27

C= COLUMNS PER DAY
CS= CREW SIZE
HR= HOURLY RATE. $ PER HOUR
CC= CREW COST, $ PER DAY
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

W.P.= WALLS SET PER DAY

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Note: Refer to Table 2-1 for Description of Scenario Numbers

EL3 - COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT															
		SET COLUMNS			INSTALL PIPE RAILS			INSTALL FENDER SYSTEM			PAINTING			MISC. & SMALL TOOLS			TOTAL
		D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	
14	METRIC	14.64	1,160	16.99	36.44	1402	51.10	36.44	1,402	51.10	27.33	200	5.47	36.44	150	5.47	130.11
	US	23.53	1,160	5.17	58.67	1402	15.58	58.67	1,402	15.58	44.00	200	1.67	58.67	150	1.67	39.66
15	METRIC	14.64	1,160	16.99	36.44	1402	51.10	36.44	1,402	51.10	27.33	200	5.47	36.44	150	5.47	130.11
	US	23.53	1,160	5.17	58.67	1402	15.58	58.67	1,402	15.58	44.00	200	1.67	58.67	150	1.67	39.66
16	METRIC	27.40	1,160	31.78	68.33	1402	95.80	68.33	1,402	95.80	51.25	200	10.25	68.33	150	10.25	243.89
	US	44.07	1,160	9.68	110.00	1402	29.21	110.00	1,402	29.21	82.50	200	3.13	110.00	150	3.13	74.35
17,18	METRIC	24.36	1,160	28.26	60.74	1402	85.16	60.74	1,402	85.16	45.56	200	9.11	60.74	150	9.11	216.80
	US	39.18	1,160	8.61	97.78	1402	25.96	97.78	1,402	25.96	73.33	200	2.78	97.78	150	2.78	66.09

EQUIPMENT COSTS

D= DURATION OF WORK

EC= EQUIPMENT COST IN $/DAY

TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	UNITS	MISCELLANEOUS ITEMS				
		FP	RPLI	CMD	1	TOTAL
14	METRIC	31,075	21,753	37,290	0	90.12
	US	50,000	35,000	60,000		27.46
15	METRIC	31,075	21,753	37,290	0	90.12
	US	50,000	35,000	60,000		27.46
16	METRIC	37,290	21,753	43,505	0	102.55
	US	60,000	35,000	70,000		31.25
17,18	METRIC	40,398	21,753	46,613	0	108.76
	US	65,000	35,000	75,000		33.14

FP = FLAGGING PROTECTION

RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE

CMD= CONTRACTORS MOB & DEMO

$/LM= DOLLARS PER LINEAR METER

$/LF= DOLLARS PER LINEAR FOOT

TOTAL COST SUMMARY FOR EL3								
SCENARIO NO.		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
14	$/M.	616.82	209.21	90.12	130.11	1,046.27	209.25	1,255.52
	$/FOOT	188.00	63.77	27.46	39.66	318.89	63.78	382.67
15	$/M.	795.96	209.21	90.12	130.11	1,225.41	245.08	1,470.49
	$/FOOT	242.59	63.77	27.46	39.66	373.48	74.70	448.18
16	$/M.	1,164.71	392.17	102.55	243.89	1,903.31	380.66	2,283.98
	$/FOOT	354.99	119.55	31.25	74.35	580.13	116.03	696.16
17,18	$/M.	1,578.74	348.61	108.76	216.80	2,252.91	450.58	2,703.50
	$/FOOT	481.14	106.27	33.14	66.09	686.64	137.33	823.96

Note: Refer to Table 2-1 for Description of Scenario Numbers

<u>Cost Estimate Calculations:</u>

Highway Barriers

HIGHWAY BARRIERS -- COST ESTIMATE

SCENARIO NO.	UNITS	MATERIALS: FOUNDATION							C-I-P CONCRETE WALL				PLATES/POST							
		SIZE	$/UNIT	L	LE	N	TL	$/HL	A	V	$/UNIT	$/HL	T	H	W	L	N	WT	$/UNIT	$/HL
19	METRIC	609.6	55.76	3.05	5.95	329	1,956	109.06	0.31	315.0	261	82.35	25	330	152	2.54	790	7,922	3.31	26.21
	US	24	17.00	10	19.5	529	10,316	33.21	3.39	662.7	200	25.10	1	13.00	6.00	8.33	1,270	28,083	1.50	7.98
20	METRIC	711.2	75.44	3.05	5.64	329	1,856	139.99	0.63	627.4	261	164.03	0	0	0	0.00	0	0	0.00	0.00
	US	28	23.00	10	18.5	529	9,787	42.63	6.75	1320.0	200	50.00	0	0.00						
21	METRIC	0.0	0.00	0.00	0.00	0	0	0.00	0.31	315.0	261	82.35	25	330	152	2.54	790	7,922	3.31	26.21
	US	0	0.00	0	0	0	0	0.00	3.39	662.7	200	25.10	1	13.00	6.00	8.33	1,270	28,083	1.50	7.98
22	METRIC	0.0	0.00	0.00	0.00	0	0	0.00	0.50	503.5	261	131.63	0	0	0	0.00	0	0	0.00	0.00
	US	0	0.00	0	0	0	0	0.00	5.42	1059.3	200	40.12	0	0.00						

FOUNDATION
SIZE IN MILIMETERS/INCHES
- L= SPACING IN METERS (FEET)
- N= PILES PER KILOMETER (MILES)
- LE= EMBEDDED LENGTH IN METERS (+6") (FEET)
- TL= TOTAL PILE LENGTH IN METERS (FEET)
- $/UNIT= DOLLARS PER LINEAR METER (FOOT)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

CONCRETE WALLS
- A= WALL AREA IN SQUARE METERS (SQUARE FEET)
- $/UNIT $ PER CUBIC METER (CUBIC YARD)
- V= VOLUME IN CUBIC METERS PER KILOMETER (CUBIC YARDS PER MILE)
- $/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)

PLATE POSTS
- T= THICKNESS IN MM (IN)
- H= HEIGHT OF PLATE IN MM (IN)
- W= WIDTH OF PLATE IN MM (IN)
- L= PLATE SPACING IN METERS (FEE
- N= NUMBER OF PLATES PER KILO-METER (MILE)
- WT= TOTAL WEIGHT IN KILOGRAMS (POUNDS)
- $/UNIT= $ PER KILOGRAM ($ PER POUND)

SCENARIO NO.	UNITS	WALL/POST								PIPE BEAM		C-I-P CONCRETE BEAM					TOTAL
		T	H	W	L	N	V	$/UNIT	$/HL	SIZE OF ELLIPS	$/HL	W	H	V	$/UNIT	$/HL	
19	METRIC					0				203 x 124	$98.40						316.02
	US					0				8.000 x 4.875	$30.00						96.29
20	METRIC	203	533	1.52	3.05	329	54	261.44	14.21			0.405	0.533	0.216	327	$70.68	388.91
	US	8	21	5.00	10.00	529	114	200.00	4.33			1.330	1.750	0.086	250	$21.55	118.51
21	METRIC	0	0	0	0.00	0	0	0.00	0.00	203 x 124	$98.40						206.96
	US	0	0.00			0				8.000 x 4.875	$30.00						63.08
22	METRIC	203	533	1.52	3.05	329	54	261.44	14.21			0.405	0.533	0.216	327	$70.68	216.52
	US	'8	21	5.00	10.00	529	114	200.00	4.33			1.330	1.750	0.086	250	$21.55	66.00

WALUPOST
- T= THICKNESS IN MM (IN)
- H= HEIGHT OF PLATE IN MM (IN
- W= WIDTH OF PLATE IN METERS (FEET)
- L= PLATE SPACING IN METERS (FEET)
- N= NUMBER OF PLATES PER KILOMETER (MILE)
- V= TOTAL VOLUME IN CUBIC METERS (CUBIC YARDS)
- $/UNIT= $ PER CUBIC METER (CUBIC YARD)

CONCRETE BEAM
- W= WIDTH IN METERS (FEET)
- H= HEIGHT IN METERS (FEET)
- V= VOLUME IN CUBIC METERS PER METER (CUBIC YARDS PER FOOT)

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Note: Refer to Table 2-1 for Description of Scenario Numbers

HIGHWAY BARRIERB -- COST ESTIMATE

LABOR																							
SCENARIO	UNITS	INSTALL CAISSONS									SET CAST-IN-PLACE CONCRETE WALLS							SET PLATES/ POST					
NO.		C	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	V	C.D.	D	C.S	H.R	C.C	$/HL	P	D	C.S	H.R	C.C	$/HL
19	METRIC	5	80%	4.0	329	82.25	7	30.00	1,680	138.18	315	19	16.5	13	32.40	3,370	55.50	20	39	3	38.30	919	36.29
	US	5	80%	4.0	529	132.25	7	30.00	1,680	42.08	663	25	26.5	13	32.40	3,370	16.92	20	63	3	38.30	919	11.05
20	METRIC	5	80%	4.0	329	82.25	7	30.00	1,680	138.18	627	19	32.8	13	32.40	3,370	110.54	0	0	0	0.00	0	0.00
	US	5	80%	4.0	529	132.25	7	30.00	1,680	42.08	1,320	25	52.8	13	32.40	3,370	33.70	0	0	0	0.00	0	0.00
21	METRIC	0	0%	0.0	0	0.00	0	0.00	0	0.00	315	19	16.5	13	32.40	3,370	55.50	20	39	3	38.30	919	36.29
	US	0	0%	0.0	0	0.00	0	0.00	0	0.00	663	25	26.5	13	32.40	3,370	16.92	20	63	3	38.30	919	11.05
22	METRIC	0	0%	0.0	0	0.00	0	0.00	0	0.00	503	19	26.3	13	32.40	3,370	88.71	0	0	0	0.00	0	0.00
	US	0	0%	0.0	0	0.00	0	0.00	0	0.00	1,059	25	42.4	13	32.40	3,370	27.04	0	0	0	0.00	0	0.00

LABOR																									
SCENARIO	UNITS	SET WALUWST									SET PIPE BEAM							SET CONCRETE BEAM							TOTAL
NO.		W.P	EF	R	N	D	C.S.	H.R.	C.C.	$/HL	N	R	D	C.S	H.R	C.C	$/HL	V	C.D.	D	C.S	H.R	C.C	$/HL	
19	METRIC	0	0%	0.0	0	0.00	0	0.00	0	0.00	393	6	65.46	6	38.30	1,838	120.34	0.000		0.00000	0	0.00	0	0.00	314.02
	US	0	0%	0.0	0	0.00	0	0.00	0	0.00	633	6	105.48	6	38.30	1,838	36.72	0.000		0.00000	0	0.00	0	0.00	95.72
20	METRIC	18	80%	14.4	329	22.85	10	38.30	3064	70.00	0	0	0.00	0	0.00	0	0.00	0.216	15	0.01414	13	32.40	3.370	47.63	296.36
	US	18	80%	14.4	529	36.74	10	38.30	3064	21.32	0	0	0.00	0	0.00	0	0.00	0.086	20	0.00431	13	32.40	3.370	14.52	90.30
21	METRIC	0	0%	0.0	0	0.00	0	0.00	0	0.00	393	6	65.46	6	38.30	1,838	120.34	0.000	0	0.00000	0	0.00	0	0.00	175.84
	US	0	0%	0.0	0	0.00	0	0.00	0	0.00	633	6	105.48	6	38.30	1,838	36.72	0.000	0	0.00000	0	0.00	0	0.00	53.64
22	METRIC	18	80%	14.4	329	22.85	10	38.30	3064	70.00	0	0	0.00	0	0.00	0	0.00	0.216	15	0.01414	13	32.40	3,370	47.63	136.34
	US	18	80%	14.4	529	36.74	10	38.30	3064	21.32	0	0	0.00	0	0.00	0	0.00	0.086	20	0.00431	13	32.40	3,370	14.52	41.56

INSTALL CAISSONS , WALLS AND CONCRETE BEAMS

C= CAISSONS PER DAY
EF= EFFICIENCY FACTOR, %
R= RATE, CAISSONS PER DAY
N= NUMBER CAISSONS PER KILOMETER (MILE)
D= DURATION, DAYS

W.P.= WALLS SET PER DAY
CS= CREW SIZE
HR= HOURLY RATE. $ PER HOUR
CC= CREW COST. $
$/HL= $ PER HORIZONTAL LENGTH IN METERS (FEET)
C.D.= VOLUME OF CONCRETE PLACED PER DAY

P= PLATES PER DAY

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Note: Refer to Table 2-1 for Description of Scenario Numbers

HIGHWAY BARRIERS -- COST ESTIMATE

SCENARIO NO.	UNITS	EQUIPMENT																					
		INSTALL CAISSONS			SET WALLS			SET PLATES/ POST			SET WALL/POST			SET PIPE BEAM			SET CONCRETE BEAM			MISC. & SMALL TOOLS			TOTAL
		D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	D	EC	TC	
19	METRIC	82.25	2,139	175.93	16.47	2,208	36.37	39.48	1,000	39.48	0.00	0	0.00	65.46	1402		0.00000	0	0	82.25	125	10.28	262.06
	US	132.25	2,139	53.58	26.51	2,208	11.09	63.49	1,000	12.02	0.00	0	0.00	105.48	1402		0.00000	0	0.00	132.25	125	3.13	79.82
20	METRIC	82.25	2,139	175.93	32.81	2,208	72.44	0.00	0	0.00	22.85	2,208	50.45	0.00	0		0.01414	1,000	14.14	82.25	125	10.28	323.23
	US	132.25	2.139	53.58	52.80	2,208	22.08	0.00	0	0.00	36.74	2.208	15.36	0.00	0		0.00431	1,000	4.31	132.25	125	3.13	98.46
21	METRIC	0.00	0	0.00	16.47	2,208	36.37	39.48	1,000	39.48	0.00	0	0.00	65.46	1402		0.00000	0	0.00	65.46	125	8.18	84.03
	US	0.00	0	0.00	26.51	2,208	11.09	63.49	1,000	12.02	0.00	0	0.00	105.48	1402		0.00000	0	0.00	105.48	125	2.50	25.61
22	METRIC	0.00	0	0.00	26.33	2,208	58.13	0.00	0	0.00	22.85	2,208	50.45	0.00	0		0.01414	1,000	14.14	26.33	125	3.29	126.00
	US	0.00	0	0.00	42.37	2,208	17.72	0.00	0	0.00	36.74	2,208	15.36	0.00	0		0.00431	1,000	4.31	42.37	125	1.00	38.39

EQUIPMENT COSTS
D= DURATION OF WORK
EC= EQUIPMENT COST IN $/DAY
TC= TOTAL COST FOR EQUIPMENT PER METER (FOOT)

SCENARIO NO.	MISCELLANEOUS ITEMS						
	FP	RPLI	CMD	1	2	3	TOTAL
19	27,968	21,753	31,075	0	0	0	80.80
	45,000	35,000	50,000				24.62
20	29,832	21,753	37,290	0	0	0	88.88
	48,000	35,000	60,000				27.08
21	27,968	21,753	21,753	0	0	0	71.47
	45,000	35,000	35,000				21.78
22	46,613	21,753	27,968	0	0	0	96.33
	75,000	35,000	45,000				29.36

SCENARIO NO.	UNITS	TOTAL COST SUMMARY						
		MAT	LABOR	MISC ITEMS	EQUIP	SUB	CONT @ 20%	TOTAL
19	$/M	316.02	314.02	80.80	262.06	972.89	194.58	1,167.47
	$/FOOT	96.29	95.72	24.62	79.82	296.45	59.29	355.74
20	$/M	388.91	296.36	88.88	323.23	1,097.38	219.48	1,316.86
	$/FOOT	118.51	90.30	27.08	98.46	334.35	66.87	401.22
21	$/M	206.96	175.84	71.47	84.03	538.30	107.66	645.96
	$/FOOT	63.08	53.64	21.78	25.61	164.11	32.82	196.93
22	$/M	216.52	136.34	96.33	126.00	575.20	115.04	690.24
	$/FOOT	66.00	41.56	29.36	38.39	175.32	35.06	210.38

FP = FLAGGING PROTECTION
RPLI= RAILROAD PROTECTIVE LIABILITY INSURANCE
CMD= CONTRACTORS MOB 8 DEMO
$/LM= DOLLARS PER LINEAR METER
$/LF= DOLLARS PER LINEAR FOOT

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Note: Refer to Table 2-1 for Description of Scenario Numbers