Report No. UMTA-GA-06-0010-81-1

INVESTIGATION OF DESIGN STANDARDS FOR URBAN RAIL TRANSIT ELEVATED STRUCTURES

B. W. Harrington, Jr.

June 1981

Final

Prepared for

URBAN MASS TRANSIT ADMINISTRATION

Office of Rail & Construction Technology

Washington, D.C. 20590

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
UMTA-GA-06-0010-81-1		

4. Title and Subtitle Investigation of Design Standards for Urban Rail Transit Elevated Structures	5. Report Date June 1981
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
9. Performing Organization Name and Address HARRINGTON, GEORGE AND DUNN, P.C. 1401 Peachtree Street, N.E., Suite 120 Atlanta, Georgia 30309	10. Work Unit No. (TRAIS)
	11. Contract or Grant No. DOT-UT-90013
12. Sponsoring Agency Name and Address **U.S. Department of Transportation** **Urban Mass Transportation Administration** **400 Seventh Street, S.W.** **Washington, D. C. 20590**	13. Type of Report and Period Covered Final Report
	14. Sponsoring Agency Code UTD-30

15. Supplementary Notes

16. Abstract

The purpose of this report was to review the various criteria for design of elevated rail transit structures currently in use by selected transit properties and to compile them in one source. The criteria was to be reviewed to determine the differences and similarities between them. An evaluation was to be made as to the feasibility of a uniform industry wide criteria for design of elevated transit structures similar to the AASHTO Standard Specifications for Highway Bridges.

This was accomplished through a library search, review of selected design criteria and designs completed using these criteria and personal interviews. A great deal of information was gathered and reviewed in this process.

Generally it was concluded that many similarities exist between criteria currently in use. The differences noted were minor and generally were related to unique operational considerations, state of the art at time of criteria development, and type of transit cars in use on the system. Because of the number of similarities between the reviewed criteria, it was concluded that a uniform industry wide criteria for design of elevated transit structures could be developed. However, it should be flexible enough for accommadating unique conditions that would occur throughout the country.

17. Key Words	18. Distribution Statement
Design Criteria Elevated Transit Structures	**Available to the Public through the National Technical Information Service, Springfield, Virginia 22161.**

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	**Unclassified**	47	

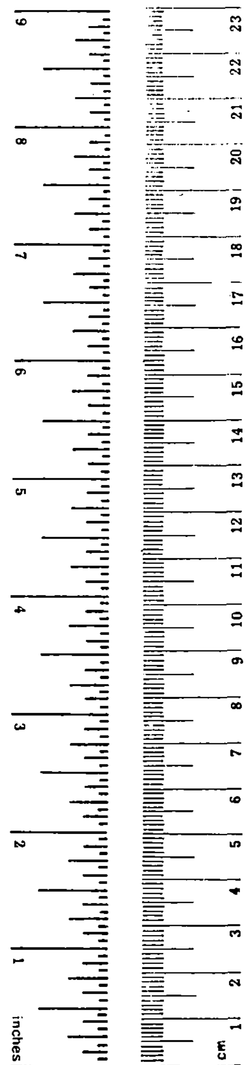
Form DOT F 1700.7 (8-72) *Reproduction of completed page authorized*

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

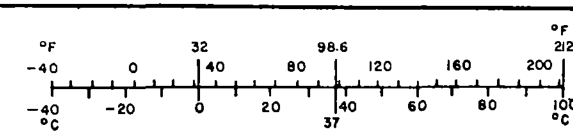
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



ACKNOWLEDGEMENT

This final report was prepared for the Urban Mass Transportation Administration by HARRINGTON, GEORGE and DUNN, P. C. We wish to thank the Office of Rail and Construction Technology, the American Public Transit Association and the transit industry in general for their assistance in providing valuable information and critiques of the content of this document. We also wish to express our appreciation to William Shepherd and William McEwen for their significant contributions to this report. This valuable informational interchange and the interest shown in the project highlight the sincere interest the transit industry has in providing emerging systems and all elements of the industry with a reference and comparison of the design criteria for elevated rail transit structures in use today.

CONTENTS

1.0 INTRODUCTION

During the past decade or more, mass transit or public transportation has received an increase in attention as a viable transportation mode within highly urbanized areas. This increase in recognition of the mass transit mode of moving people, in large measure, came about due to congestion of cars on urban highways and streets. At about the same time as automobile congestion was becoming critical, organized opposition to construction of urban freeways and requirements for environmental evaluation of construction all but stopped cities from relieving their transportation problems. It was these events that prompted urban communities to look to alternative transportation modes.

In most urban areas the central business district forms the hub with densely populated subcenters radiating out from this hub. Since these sub-centers, whether they are industrial, commerical or residential in nature, must be interconnected with the hub, interconnecting transportation systems are necessary for mobility. This led transportation planners to look at alternative transportation sytems that would not only interconnect these centers but move masses of people efficiently and at a reasonable cost. Where conditions were right, these planners recommended mass transit systems, one of which is the high speed steel wheel on steel rail system. Other types, such as people movers, light rail system and busses in exclusive rights-of-way, have been recommended and adopted for use in various situations. Only the steel wheel on steel rail high speed system is discussed herein.

It would appear that with the number of existing steel wheel on steel rail systems in operation in urban areas when other cities were doing initial planning, a lot of the existing design criteria could have been used on these proposed systems. Some of the data and information from these systems could have been and was used on the newer ones. However, a great deal of data used in the initial design and planning of these existing systems was dated circa 1900 and would not be applicable today.

Normally, one would expect design criteria for a new rail transit system to be substantially the same as that used on the existing sytems with, perhaps, some updating. However, as previously noted, rail rapid transit criteria existing in the early 1950's, in large measure, dated back to the period prior to World War I. Therefore, new criteria was written reflecting changes in public attitudes, land use policies, materials, and design technology. That part of the old standards which was found to still apply was reused.

Cost of major urban mass transportation systems must be controlled and held within reasonable limits. If this is to be accomplished, strong leadership in defining more productive and cost effective designs and construction procedures is required. The major capital cost of any rapid transit system is its construction. However, the character of design has a siginificant effect on the construction cost. The impact of design on construction is evident in highway bridge work. Over the years, criteria for design have been developed and are generally consistent throughout the country. This consistency in functional requirements has not limited the freedom of design but it has brought about a uniformity that is understandable and acceptable to all associated with the highway bridge field. The end result has been competitive and reasonable prices for construction of these projects.

Presently no such consistency or uniformity of criteria exist within the transit industry. It is the purpose of this report to review existing criteria and structure types for elevated transit structures and to determine the feasibility of industry wide guidelines for their design.

2.0 SUMMARY AND CONCLUSIONS

This study was undertaken with the overall intent to look at the design criteria being used on aerial girders, the difference between them and the possible impacts the design criteria might have on overall cost.

Any such comparison must keep in mind the fact that the first rail rapid transit systems in the United States were designed at the turn of the century. Few changes in design practice occurred between the two World Wars. Following World War II major improvements in methods and materials impacted all fields of endeavor. Many cities had grown in size and demand for mobility to warrant consideration of rail rapid transit also increased.

As new rapid transit systems began to be developed, it became apparent that there was not a uniform design criteria totally appropriate to elevated transit structures. The only logical step was to develop one. The need for a new criteria applicable to transit elevated structures was recognized when BART-San Francisco, the first new system, was being developed. The criteria developed for existing elevated transit structures did not address the type structure or vehicle being conceived for the new systems. With the passage of time and experience gained at BART, the need to develop new criteria for subsequent systems, such as WMATA-Washington D.C. Changes in the variou design codes as well as materials technology since the development of the BART criteria had to be recognized.

The process of developing totally new criteria for each new transit system continued and, as a result, each of the newer rapid transit systems have their own structural design criteria for elevated structures. As each of these criteria was reviewed, compared with each other and with standard codes and criteria, e.g. AASHTO Standard Specifications for Bridges and AREA Manual for Railway Engineering, it was found that there were differences as well as similarities. However, the similarities far outnumbered the differences and those differences, especially between the criteria of the various transit systems, were minor. The major differences between criteria developed by the newer transit systems were as follows:

- Use of Load Factor Design
- Seismic or highwind loading criteria
- Derailment Loading Criteria
- Point of application of some horizontal live loads

Seismic loading criteria, and very high wind loading are unique to a particular area. Where the derailment loading in one criteria appeared to be higher than used in other criteria, it was found to be dictated by an operational policy. The use of Load Factor Design in the various criteria was found to vary from not being allowed to its use being mandatory. The variation of point of application of the horizontal load did not appear to have any significant effect on the actual design load applied to the structure due to the variation in application of horizontal impact factors. In one criteria where the horizontal load was applied high, no impact was added; however, in criteria where it was applied low, impact was added.

It should be noted that vehicle dead weight differs measurably system to system. This variation appears to be diminishing, however, as vehicles tend to become standardized in size and components.

It became obvious that even though the criteria differed, the differences are not of major significants to overall cost of the structure. Probably the most significant difference that may affect cost was the variation in use of Load Factor Design.

When all of the most recent developed criteria are compared to AASHTO a great deal of similarity is found, especially with the design of concrete and steel structures. The significant differences are basically in the vertical loadings and the distribution of the loads within the superstructure. AREA however, even though applicable for rail type structures is probably somewhat too restrictive for transit structures due to the great difference in loadings. There is a great deal of material in the AREA that is applicable and should be considered. Generally, each of the criteria reviewed from the various transit properties indicates they are continuing to evolve based on new findings and research. However, other than these evolutionary changes, a very strong similarity exist between all transit criteria and the AASHTO

Standard Specifications for Bridges.

Based on the research and studies carried out in developing this report the following conclusions were drawn:

- A uniform structural design criteria for use on aerial girders for transit structures is practical and can be developed provided it allows for the unique environmental conditions, vehicle weights and operational policies of each system.
- More research is required in the area of impact loading and vehicle - structure interaction.
- It is impractical to attempt any industry wide standardization of an aerial girder or any of its components.
- The small differences existing in currently accepted design criteria plays a minor role in determining final cost of the structure.

3.0 OVERVIEW OF EXISTING SYSTEMS

3.1 HISTORY OF ELEVATED SYSTEMS

High speed rapid transit systems using steel wheels on rails can generally be categorized into two broad types, old and new. This distinct classification can be attributed to the fact that the older transit systems designed and constructed their elevated sections prior to World War II. Since that initial burst of design and construction of elevated sections prior to World War II, very little has been done to extend or replace those sections. In recent years the only significant design or construction of elevated rapid transit structures has been associated with new systems. The differences, as described later, between the older and the new elevated structures is significant.

In an article written in 1915 entitled, "Building the New Rapid Transit System of New York City - Design of the New Elevated Railway Lines", (Ref. 1), a very detailed explanation of the parameters to be considered when placing columns for the elevated structure within the streets is given. From this article, it is evident that other than basic structural integrity cost was the key factor in the decision process related to structure design. Designs similar to those described in the above article can be found in most cities having these older systems. Philadelphia, Boston and Chicago have similar type elevated structures. Figure 3-1 is a typical arrangement. The columns are generally spaced 26 to 31 feet apart at each bent with the 26 foot spacing generally used. The philosophy behind this 26 foot transverse spacing of columns was that on a 60 feet wide roadway, two lines of traffic could pass between the curb and the column. As can be seen from Figure 3-1, it may have been feasible in 1915 to get two lines of traffic within the 15'-6" between the curb and the column, but not today.

These older systems had their superstructure designed to provide a girder under each rail, or as close to this as possible. This was done to eliminate as much as possible any stresses in the deck or ties other than direct compression. Typical stringer spans between transverse bents were 70 feet. The stringers were designed as simple spans with fixed and expansion bearings at opposite ends and loaded with actual wheel or axle loads from the cars. The transverse bents were designed as simple span cross girders with knee bracing to provide transverse rigidity. This knee bracing along with special details at the base were assumed to provide fixity at the top and bottom of the columns in the transverse direction.

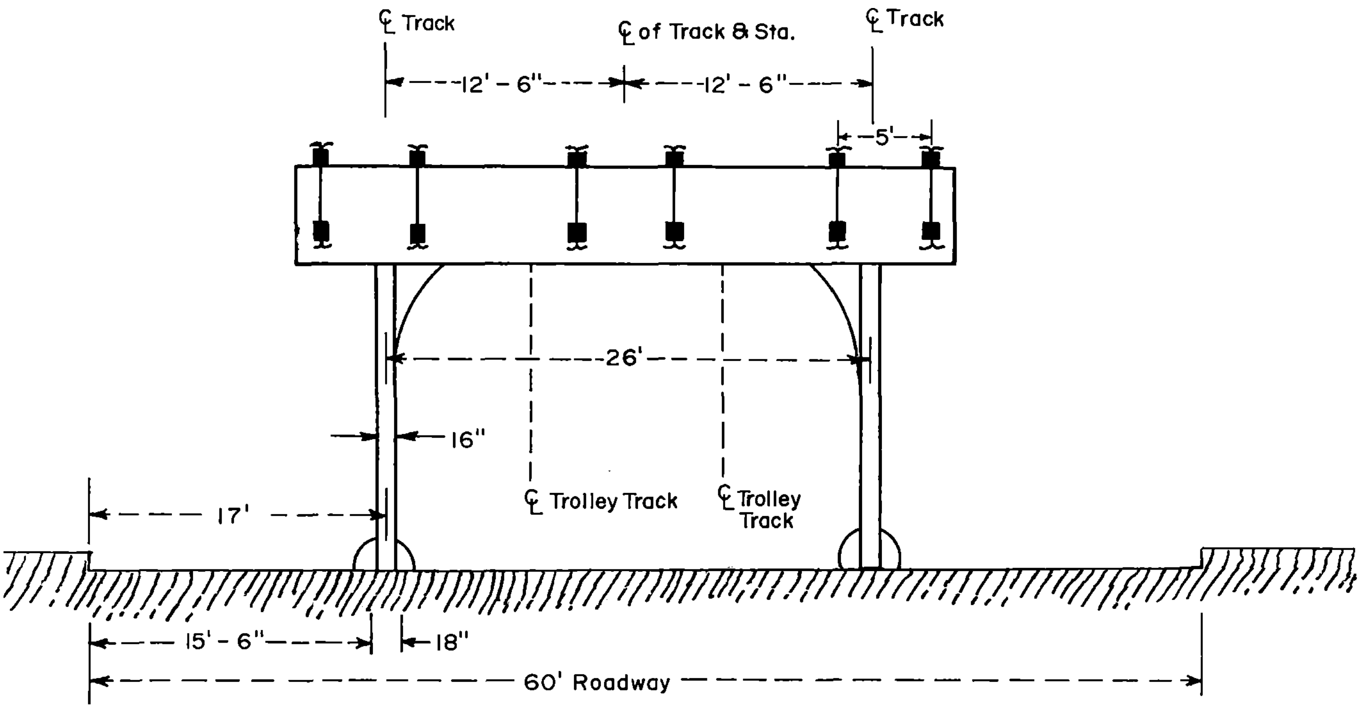


FIGURE 3-1 LOCATION OF COLUMNS ADOPTED FOR WIDE STREETS

Recently, there has been a great deal of consideration given to the impact of the older elevated structures on their environment. In those cities where the elevated structure has been placed within the street, they have effectively created longitudinal boundaries as well as contributed to the deterioration of neighborhoods. In general, wherever older elevated structures of considerable length exist, the buildings adjacent to them are in a state of disrepair or in some cases abandoned. Most of these buildings, if over one story, are unoccupied above the first story. In New York City, a community adjacent to an elevated line had undergone serious deterioration. The elevated line was removed and the area is now a thriving viable community.

Transit operation on the older elevated structures are also extremely noisy. Noise mitigation was a consideration in their design which revolved around whether to use ballasted track on concrete decks to reduce noise. However, cost was a significant factor and in New York City, the ballasted track and concrete deck was abandoned in favor of open floors to reduce the initial cost.

3.2 RECENT ELEVATED SYSTEMS

A considerable period of time elapsed between the design and construction of the earlier elevated systems and the new systems of today, i.e., San Francisco and Atlanta. During this period there was a tremendous growth of technology applicable to vehicles, communications and especially to structural analysis and behavior and construction materials.

As the various urban areas considering mass transit utilizing high speed rail lines passed beyond the feasibility study phase, horizontal and vertical alignments were established. Invariably, extensive elevated sections resulted from these studies.

The engineers planning these new systems realized that significant changes in construction materials, technology and an increased understanding of the behavior of structures had occurred in the near half century since the design and construction of the earlier elevated systems. Earlier designers did not fully consider designing for passenger comfort or the interaction of the transit car and the structure, or for continuously welded rail. These and many other concepts, technologies and construction materials, such as prestressed concrete, or even reinforced concrete for other than foundation or deck materials, were either not available or not considered by the earlier designers. A New York transit system design criteria, circa 1913, made no mention of concrete except as a deck or foundation material. Also, a full appreciation of the significance of fatigue was also not available in 1913. It became apparent at that time that the criteria for design of elevated transit structures available would not be applicable for new systems and structures.

There were design criteria for elevated structures available to the engineers planning the new transit systems. These were American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges (AASHTO) and the American Railroad Engineers Association's Manual for Railway Engineering (AREA). However, there were enough differences in the character and loading of transit structures that neither of these criteria could be used verbatim. The wheel spacings for the Cooper Railroad loading did not correspond to that generally found on the transit cars nor was the impact criteria appropriate for the suspension and drive systems used on transit cars. The same differences were generally true for the AASHTO Standard Specifications for Highway Bridges; the loading conditions were different and could be more accurately predicted than for highway structures. AASHTO impact factors were also not totally applicable. There was not one single design specification available that was totally applicable to elevated transit structures.

This technology growth plus experience, which is an extremely important factor, is reflected in the designs and construction materials and methods used in recent systems. As noted previously, concrete was considered for foundation and possibly as a deck material only for the earlier systems. Now, when reviewing the more recent systems, there are variations in superstructures from steel plate girder and concrete deck to precast prestressed and segmentally constructed concrete box girders. The following is a very brief description of some of the various types of elevated girders that have been recently designed.

a. Steel plate girders and composite deck supporting one track.

b. Precast concrete AASHTO girders and composite deck supporting one track.

c. Single steel or concrete box girder with composite precast and tranversely prestressed decks supporting two tracks.

d. Single steel or concrete box girders with composite precast concrete decks supporting one track.

e. Single concrete box girder with monolithic deck supporting one track.

f. Precast-prestressed concrete double tee beams supporting one track.

g. Single precast concrete box girder and deck constructed by segmental methods.

It is apparent when comparing the above designs to those circa 1919, which utilizied transverse riveted steel girders supporting longitudinal built up riveted steel stringers, that a great deal of advancement in both construction materials and structural analysis methods has taken place.

In the following sections an attempt will be made to identify the criteria used together with some of the underlying philosophies that have gone into some of these designs.

4.0 COMPARISON OF DESIGN CRITERIA FOR ELEVATED TRANSIT STRUCTURES

4.1 INTRODUCTION

The purpose of this study has been to review the design criteria for elevated structures developed in recent years along with the structures resulting from these criteria and attempt to determine if it is feasible to work toward developing this, we have visited several transit systems and have talked with engineers involved with the design of these structures. A great deal of data was collected including design criteria and drawings. This data along with data collected during a literature search was reviewed and is presented in the following sections.

4.2 REVIEW OF EXISTING DESIGN CRITERIA FOR AERIAL STRUCTURES

Design criteria from the cities listed below has been obtained for review and comparison. In order to provide a visual means of comparison, bar charts have been prepared that will show each of the systems and the variation in criteria used for various design considerations. We have included for comparison, the criteria for the same design considerations as presented by AASHTO. AREA, and ACI Committee 443 Report "Analysis and Design of Reinforced Concrete Bridge Structures". The following is a list of the cities with transit systems which were studied along with the abbreviations used in this report.

Atlanta	MARTA
San Francisco	BART
Washington	WMATA
New York	MTA
Philadelphia-Lindenwold	PATCO
Baltimore	MTA-Baltimore
Miami	MIAMI

As this data was reviewed, the differences in the various criteria became apparent. Improvements in the criteria as each subsequent criteria was developed was also evident. The variations in design requirements due to climatic conditions in various cities, the type transit car used and to some extent operational requirements, were all evident during the review.

4.2.1 GENERAL

The criteria prepared by all of the various transit authorities and ACI 443 make reference to the ACI Building Code Requirements for Reinforced Concrete (ACI-318), the AISC Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, and the AASHTO Standard Specifications for Highway Bridges. Some, but not all, make reference to the AREA Manual for Railway Engineering. If a generalization can be made from the material reviewed, it could be said that the older systems, i.e., Chicago, Philadelphia, and New York, did refer to AREA while the newer systems, Atlanta, Miami, Baltimore, etc., did not. PATCO, although contemporary with BART for design, was generally required to adhere to accepted railroad practices, i.e. AREA, because of interstate operations. The lack of reliance on the old railroad standards for new systems can probably be attributed to the increased understanding of structures under stress and an increase in confidence by the engineer in his ability to accurately predict transit loads. When loads can be predicted or determined with a high degree of reliability less conservatism is appropriate in the design. When considering the high ratio of live load to dead load found in railroad loading and the accuracy with which these loads can be predicted, the conservatism of the AREA Manual is prudent. Transit loading, however, does not have this same high ratio of live to dead load and the transit live load can be predicted more accurately and therefore does not warrant the conservatism of the AREA Manual.

4.2.2 TRANSIT CAR

There are as many types of transit cars as there are transit systems. There can also be various type transit cars using the same tracks within one system, e.g., New York and Cleveland. The empty weights of the various cars differ considerably, depending primarily on the size and year of manufacturer. The weights of the cars surveyed vary from a low of 58,000 pounds to as high as 90,000 pounds (See figure 4.1). The average empty weight is approximately 69,000 pounds.

4.2.3 TRANSIT LIVE LOADS

The live loads specified in the various criteria include both the empty car weight and the crush loading weight of the passengers. This crush load weight due to passengers is dependent upon the capacity of the car and the average weight assigned each passenger.

This average weight per person is generally accepted to range from 150 to 165 pounds, which can amount to a 10% variation. Since the capacity of the transit car can be determined fairly accurately, the design crush load is fairly predictable. The chart, figure 4-2 shows the variation in design crush loads for the various system. The center to center truck distances for all cars examined range between 70 to 75% of the coupled length of the car. Axle spacings on each truck range from a low of six feet six inches to eight feet six inches maximum. Seven feet to seven feet six inches appears to be the average axle spacing, especially on the newer cars. In most cases, the criteria examined did not make specific references to loadings due to work trains, e.g., ballast cars.

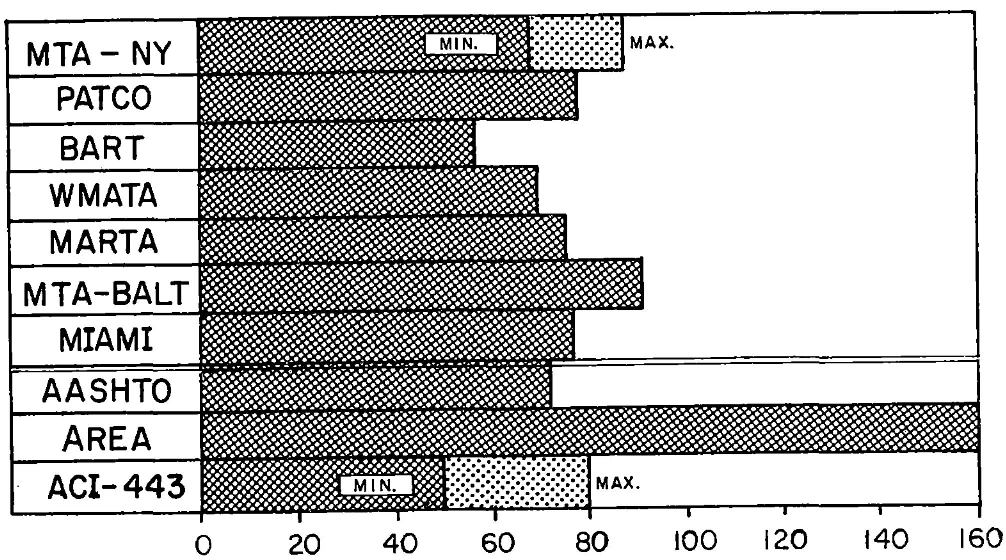


FIGURE 4-1 VARIATION OF EMPTY CAR WEIGHT (KIPS).

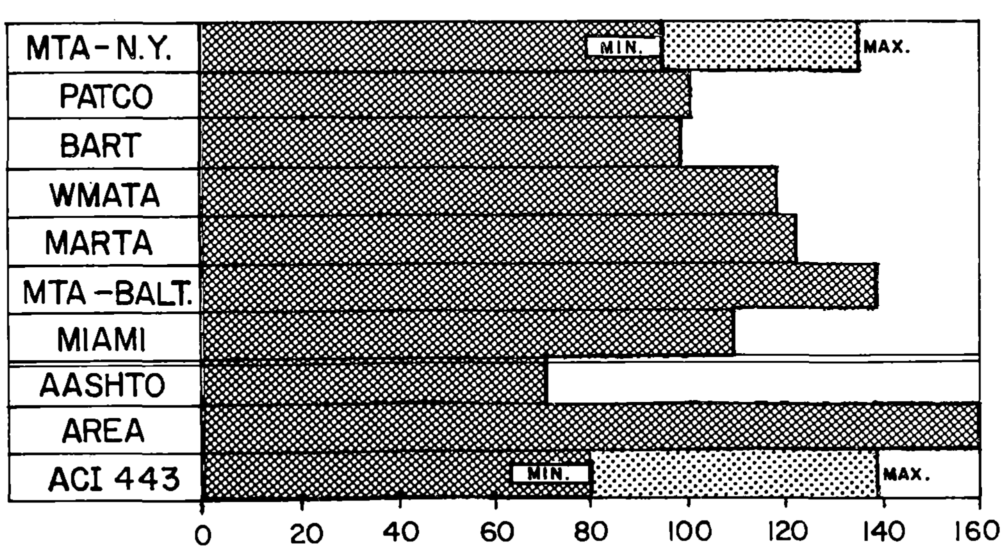


FIGURE 4-2 VARIATION OF DESIGN CRUSH LOAD (KIPS).

4.2.4 IMPACT

Generally, impact formulas specified for use in design of elevated transit structures are taken from that specified by AASHTO. This formula for impact, I, is

$$I = \frac{50}{125+L} \quad - (1), \text{ Where L is the span length}$$

and relates impact to span length only. However, span length is only one of many factors that affects the increase in live load to which the structure is subjected due to the movement of the live load. Such items as car suspension system, relative masses of vehicle and structure, structure natural frequency and damping characteristics, jointed versus continuously welded rail, all contribute to impact forces. To analytically evaluate all of these variables would be an enormous task. It is because of this enormity that most transit criteria have resorted to the use of highly simplified but proven, at least on structures carrying trucks and trains, impact formula.

It should also be pointed out that the early developers of these impact formulas knew of the problems associated with fatigue but did not have the knowledge of fatigue now available. Therefore, these engineers used the impact factor as one means of accounting for reduced allowable stress resulting from fatigue. Even though the impact formula indicated by most of the criteria reviewed used the AASHTO formula or close to it, the variation was quite wide. The following is a sample of criteria used:

MTA-NY $\quad I = \dfrac{150-\frac{L}{6}}{450+L}$ $\quad$ L is the span length except where noted.

PATCO $\quad I = \dfrac{100}{5} + 40 - \dfrac{3L^2}{1600}$ $\quad$ (L less than 80') AREA (Steel)

$I = \dfrac{100}{5} + 16 + \dfrac{600}{L-30}$ $\quad$ (L more than 80') AREA (Steel)

$I = \dfrac{100L}{L+D}$ $\quad$ (L is live load, D is dead load) AREA (Conc.)

BART $\quad$ I = 40% and 60% for positive and negative bending respectively for girders 140 feet or less in length.

WMATA	I = 30% and 40% for positive and negative bending respectively for non ballasted structures and 30% for ballasted structures. Applicable to girders 150 feet or less in length.	
MARTA	$I = \frac{50}{125+L}$	(Maximum of 30%) AASHTO
MTA-BALT.	I = 30% and 40% for positive and negative bending respectively for non ballasted structures and 30% for ballasted structures. Applicable to girders 150 feet or less in length.	
MIAMI	$I = \frac{50}{L+125}$	(Maximum 30%)
AASHTO	$I = \frac{50}{125+L}$	(Maximum 30%)
AREA (Steel)	$I = \frac{100}{5} + 40 - \frac{3L^2}{1600}$	(L less than 80')
	$I = \frac{100}{5} + 16 - \frac{600}{L-30}$	(L more than 80')
AREA (Conc.)	$I = \frac{100L}{L+D}$	(L is live load, D is dead load)
ACI 443	In absence of specific data, use AASHTO ($I = \frac{50}{L+125}$)	

There have been studies undertaken by Biggs, et al, on highway bridges that concluded that the primary contributors to large bridge vibrations are the initial "bounce" of the vehicle on its own suspension systems as it enters the span, surface irregularities on the bridge itself, and the ratio of bridge to vehicle natural frequency. The effect of bridge to vehicle natural frequencies ratio on the dynamic deflection can be seen in Figure 4-3. In this figure, the ratio of maximum dynamic (y_m) to maximum static deflection (y_{st}) is plotted against the ratio of bridge to vehicle natural frequency, ω and ω_v respectively.

Plot of the ratio of maximum dynamic deflection (y_m) to maximum static deflection (y_{st}) verus the ratio of bridge to vechicle natural frequency.

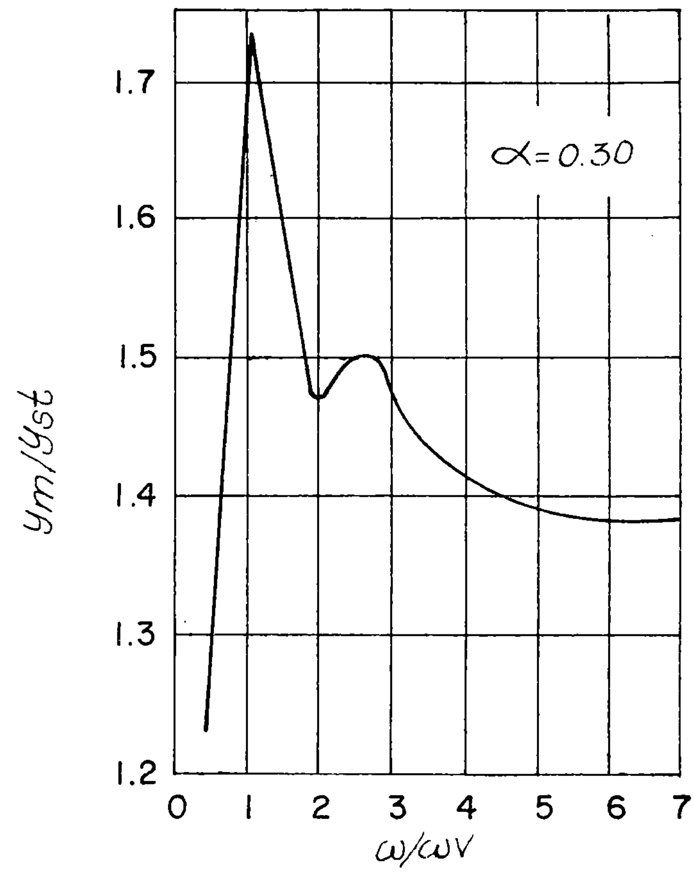


FIGURE 4-3

Plot of the ratio of maximum deflection to deflection for α, initial bounce, equal to 0.30 verus α, the initial bounce.

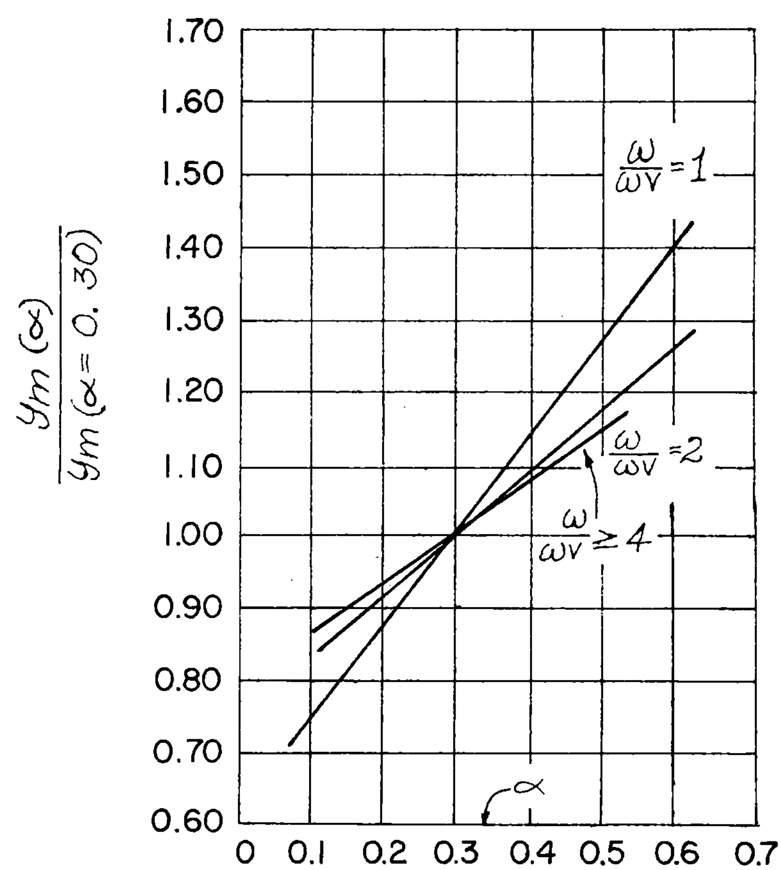


FIGURE 4-4

Figure 4-4 shows the effect of the initial "bounce" or initial vehicle oscillation on the dynamic deflection. If it is assumed that the vehicle natural frequency is 1.5 cps and

Where α = initial "bounce" or deflection

ω = circular natural frequency

α will be approximately equal to 0.20. Also asume that the natural frequency of the structure is twice that of the vehicle, which is what is generally stipulated by the reviewed criteria. Using these parameters,

$\alpha = .20$ and $\frac{\omega}{\omega_V} = 2.0$

Figure 4-3 yields $y_d/y_{st} = 1.47$ and

Figure 4-4 yields $y_d(\alpha)\ /y_d(\alpha = .30) = .91$

Therefore, the expected maximum dynamic deflection equals 1.47 x .91 = 1.338. Applying this same factor to the static bending moment provides a reasonable estimate of the dynamic moment on the beam. This relative simple exercise implies the reasonableness of the impact factors quoted in some of the different criteria for transit structures. From examining the graphs, it can be seen that if the initial "bounce" is reduced,the dynamic deflection will also be reduced. In transit structures riding on continuously welded rail that is properly maintained, this initial "bounce" should be significantly reduced. However, until such time as more data is available, it appears that AASHTO Impact values or constant impact values of 30 to 40% are reasonable.

4.2.5 TRANVERSE HORIZONTAL IMPACT

Transverse horizontal impact forces can result from variation in rail alignment, uneven wheel wear, lateral swaying of the transit car, and possibly from other uneven wearing or misalignments. The criteria reviewed generally went from one extreme to another. Either the criteria had a horizontal impact factor or it did not. Those criteria with horizontal impact differed in the percentage of vertical load, 10% versus 25%, and the point of application of the load. The criteria with a 10% factor applied the load at three feet six inches above the low rail and those with a 25% factor applied the load at the base of the rail. The wording of the criteria that used a 10% factor implied that by applying the load three feet six inches above the "top of the low rail" that this force may only be applicable within curved alignment. However, this is not the case and the load was in fact applied on both tangent and curved alignments.

4.2.6 CENTRIFUGAL FORCE

Most of the criteria reviewed have widely varying formulas for centrifugal force that essentially lead to the same loading. However, the difference appears to be the height above the top of the rail for application of the load. The following is a summary of the results of the various formulas when assuming a speed of 65 mph with a 2,450 feet radius. The force is a percentage of the vertical design load.

MTA - NY	CF = 13.2% of 2,000 pounds per foot @ five feet above top of rail
PATCO	CF = AREA formula
BART	CF = 18.3% @ three feet six inches above top of low rail
WMATA	CF = 11.9% @ five feet above top of rail
MARTA	CF = 17.5% @ three feet six inches above top of rail
MTA-BALT	CF = 11.9% @ five feet above top of rail
MIAMI	CF = 13.4% @ five feet two inches above top of rail
AASHTO	CF = 11.6% @ six feet above top of rail
AREA	CF = 11.6% @ six feet above top of rail
ACI 443	CF = 11.6% @ three feet four inches above top of rail

There is a 37% variation between the low and high for the centrifugal force and a 44% difference in height of applied load. When considering the combination of the centrifugal force and point of application, the difference amounts to 40% between the low and high valves. This force and its point of application for design of transit structures should be reasonably well defined since the variation in vehicles and speeds is very closely controlled and defined for each system. Even with the different vehicles for each system, there should not be as much variation of the force or its point of application indicated.

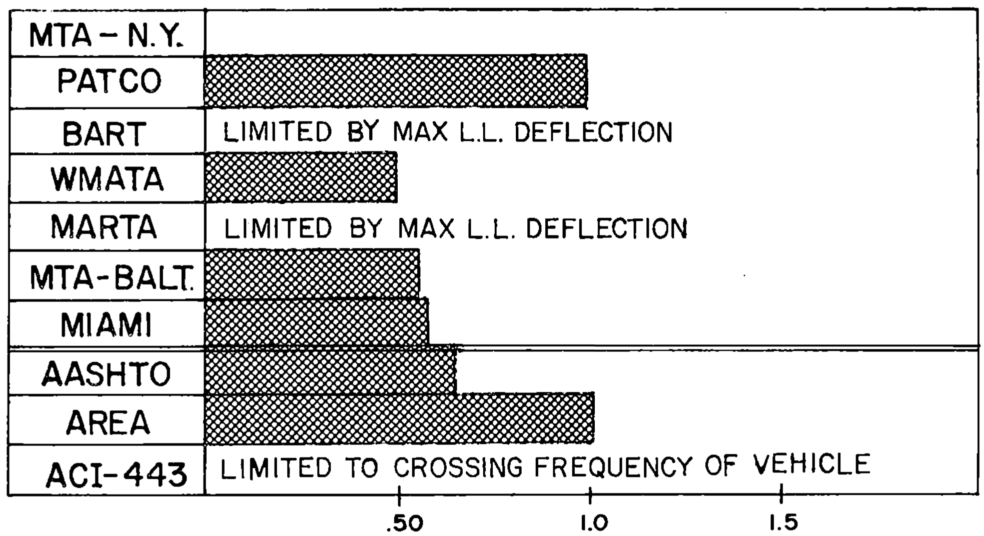
4.2.7 LONGITUDINAL FORCE

Longitudinal forces are generated primarily by acceleration and deceleration of the transit vehicles. It is usually specified as a percent of the live load and applied as some point above the top of the rail and acting in either direction. All of the criteria reviewed have approximately the same value specified; 10% to 15% of the live load. The difference however was in the point of application of the load. Most of the criteria indicated the load to be applied at the top of the rail. However, Miami's criteria indicated it should be applied at five feet two inches above the top of the rail. It would appear that if this force is a dynamic force created by an acceleration (or deceleration) that its point of application would be at the center of gravity of the loaded vehicle. The application of this load at the vehicle center of gravity would produce overturning forces acting at the trucks. Assuming average dimensions between trucks of 55 feet and five feet two inches to C.G. of vehicle from top of rail, the percent increase (decrease) in vertical load acting at the truck is only about 2.0%. This small increase (decrease) in wheel load implies that the point of application is not significant.

4.2.8 VIBRATION AND DEFLECTION CONTROL

Rider comfort and the prevention of interaction between the vehicle and structure is addressed in some manner by all of the criteria reviewed. The differences generally occur between railroad and highway criteria (AREA and AASHTO) and transit criteria. It appears that AREA and AASHTO have deflection criteria only. Most of the transit criteria have both a deflection and a vibration criteria. Also most tend to reduce the live load plus impact loading for deflection by a factor ranging from 75% to 89%. This factor is a means of recognizing that deflection should be based on normal loading rather than the loadings stipulated for crush load conditions. The allowable deflections stipulated by most criteria is limited by a factor of the span length, e.g., 1/640 of the span length. The chart shown in Figure 4-5 shows the variation in the allowable deflection factor combined with the percentage reduction of the live loads plus impact.

The AREA criteria was used as a base value of one. The effect of the actual loading has not been included in the chart due to the wide variation in loadings.



AREA Delfection Criteria ÷ System Deflection Criteria

FIGURE 4.5 DEFLECTION CRITERIA VARIATION

Vibration control is only addressed directly by criteria developed by BART and systems developed after BART. The criteria reviewed; BART, WMATA, MARTA, MIAMI and MTA-Baltimore all have the same criteria.

The unloaded natural frequency of the first mode of vibration for longitudinal girders shall not be less than 2.5 cycles per second. Further, no more than one span in a series of three consective spans shall have a first mode frequency less than three cycles per second.

A paper prepared by Bechtel Corporation for BART (Referecne 16) provided an indepth analysis of the problem of vehicle-structure interaction. In this paper they point out the inaccuracies of using highway and railroad impact factors for rapid transit structures. This paper appears to be the basis for the 2 1/2 cps frequency criteria for longitudinal tract girders of rapid transit structures.

4.2.9 DERAILMENT FORCES

Derailment forces are considered by some of the existing criteria and not by others. The omission of a derailment load is not an oversight but appears to the result of operational considerations and an assessment of the impact derailment loads will have on the structure under consideration. For example, the structure used by MARTA (Figure 4-6) which has a considerable torsional force acting on it under normal loading conditions, would be more susceptible to derailment loads than the WMATA single track structure (Figure 4-7). Operational considerations that could influence application of derailment loads are:

- Use of guard rails
- Vehicle undercarriage; i.e., will undercarriage hang up on rails if car derails and thereby limit its transverse excursion.
- Acceptance or non-acceptance of possibility of a damaged girder, and can service be maintained during repair.

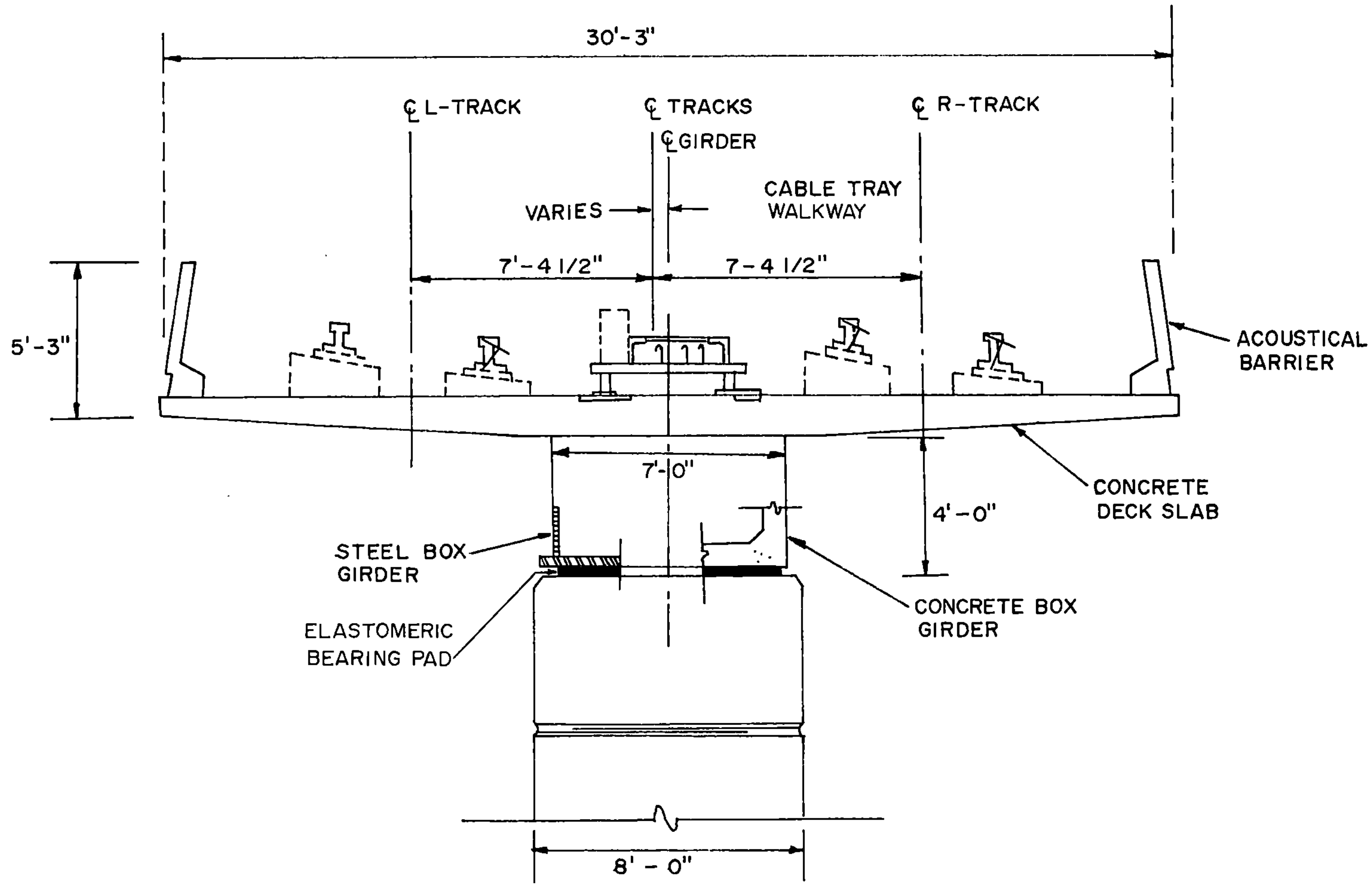
All of these considerations assume that if derailment does occur catastrophic failure or collapse is not probable.

MARTA, MIAMI, and ACI 443 were the only criteria reviewed that specifically addressed derailment. A summary is as follows:

- MARTA - Vehicle longitudinal axis set at three feet six inches from edge of deck. No impact added and 150% increase in allowable stresses.
- MIAMI - Vehicle longitudinal axis set a maximum of three feet from the track ℄ . 100% impact is added and 150% increase in allowable stresses.
- ACI 443 No specific value of location given. Indicates impact should not be added to vehicle load.

4.2.10 RAIL FORCES

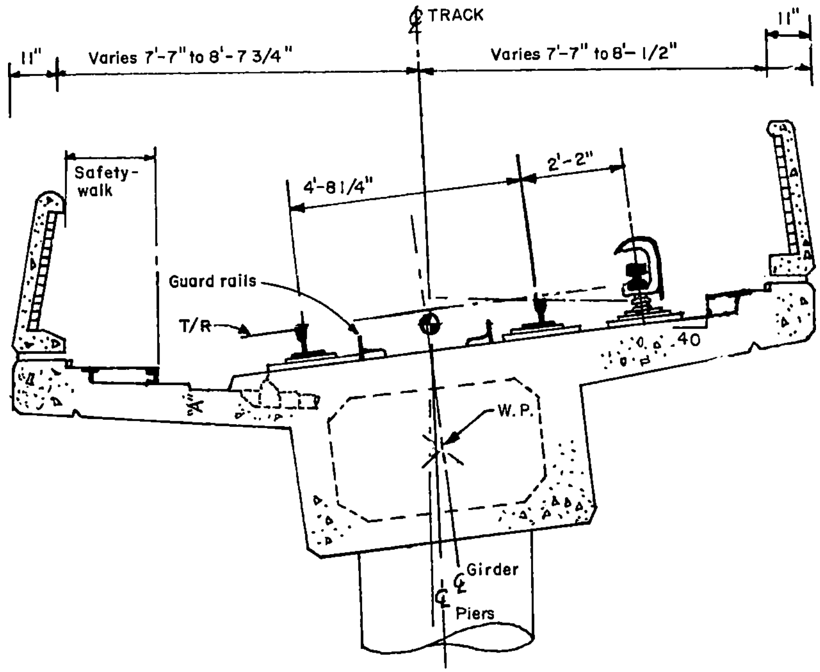
Forces in the rail, usually generated by temperature changes, can be significant when continuously welded rail (CWR) is used and it is terminated on the structure. The application and magnitude of this force is addressed in some form, either by specifying the loads due to terminating the CWR or



Typical Double Track Structure

MARTA

FIGURE 4-6



CROSS SECTION

SINGLE TRACK STRUCTURE

TILTED GIRDER

WMATA

FIGURE 4-7

specifying that CWR will not be terminated on aerial structures. In most cases, forces caused by terminating CWR on aerial structures is eliminated by designing vertical alignments to provide at-grade sections to anchor the rails off the structure. The magnitude of this force can vary and is dependent upon the size of rail used and the maximum temperature differentials expected. Acceleration or deceleration forces are also added to the temperature forces. The sum of these forces can amount to around 240 kips per rail if the rail is terminated on the structure.

The aerial structure criteria developed for the newer systems, BART and subsequent systems, include a longitudinal rail force associated with thermal stresses that is applied to the structure. There is some variation of the magnitude in the criteria reviewed. However, this force is dependent upon such items as the clamping force of rail fasteners, type girder bearings used, and flexibility of piers or bents, which can not be quantified totally by a criteria. The following is a summary of the criteria presented on this force:

BART	17 kips per rail applied to the first three piers adjoining any abutment or cross-over structure.
MARTA	21 kips per rail applied to the first three piers adjoining any abutment or cross-over structure.
WMATA	No specific value given. Requires designer to consider interaction of rail and structure.
MIAMI	Elastic analysis considering interaction of rail, fastener, bearings and substructure may be used. If not, the following is acceptable Symmetrical girders: Force = 0 Unsymmetrical girders: Force (kips) = 0.63L-8 Where L = span length in feet. Unsymmetrical girders are those with different bearing conditions or unsymmetrical arrangement of rail fasteners.
MTA-BALT.	Force = 0.65 x PxL where P = clamping force of rail fastener per linear foot; L = average span length of two adjacent spans.

4.2.11 FATIGUE

Over the years, fatigue has become recognized as a significant factor in the design of bridges and other structures subjected to repetitive loadings. Early engineers were aware of the problem but the available knowledge was not sufficient for them to accurately quantify its effects. Structures supporting rapid transit vehicles are perfect examples of fatigue loading. What is significantly different between highway loading and transit loading for fatigue design is that the loads and frequency of loading can be more accurately predicted for transit structures.

Most of the newer transit authorities, BART, MARTA, WMATA, etc., have elected to generally follow the AASHTO criteria for fatigue design. MIAMI has also modified its criteria by varying the load used for fatigue analysis based on the number of load cycles expected. The load is varied from 100% crush load to 89% crush load. The older systems have considered fatigue in their criteria, and it appears to be in conformance with AREA criteria.

4.2.12 LOAD FACTORS

Most of the criteria reviewed, especially from the newer systems, have elected to specify load factor design for their structures. In most cases load factor design has been applied to concrete structures only, however, it was allowed as an alternate method of design for steel structures by one system and required for both steel and concrete by another. All of the criteria reviewed have their load factors generally conforming to that stipulated by AASHTO. The only significant difference between the criteria reviewed was one criteria, MTA-Baltimore had an effective load factor of 1.7 times the earth forces for Group I loading while all others had a load factor of 1.3. The use of a 1.3 load factor for earth pressures is consistent with AASHTO and ACI 443. However, the ACI Building Code (ACI 318-77, Reference 2) required a load factor of 1.7 times earth loading.

Factors for service load design (working stress) are also provided for in most of the criteria. A summary of factors for Group I loading for both service load design and load factor design are shown in Table 4-1 and 4-2 respectively.

There are some differences in the various load factors used and also what loads should be included in a particular group. MARTA for example, specifies either centrifugal or longitudinal force to be used in Group I, while WMATA does not include longitudinal forces in Group I loads at all. The other variation that is obvious is the multipliers used for horizontally applied forces in Load Factor Design (Table 4-2). Most of the criteria reviewed, multiplied the horizontally applied forces by 1.3 while MIAMI multiplied them by 2.17. It appears to be some difference between the criteria reviewed as to what loads should be included in the various loading groups and what load factors should be applied to them.

TABLE 4-1 SERVICE LOAD DESIGN

CRITERIA	GROUP I LOADS	PERCENTAGE OF UNIT STRESS
MTA-NY	D + (L+I) + CF + (E+B+SF) (1)	100%
PATCO	Per AREA	100%
BART	D + (L+I) + (CF+LF) + (E+B+SF)	100%
WMATA	D + (L+I) + (CF+RF) + (E+B+SF)	100%
MARTA	D + (L+I) + (X) + (E+B) (X) = CF or LF	100%
MTA-BALT.	Service load design not allowed	---
MIAMI	D + (L+I) + (CF+LF+NF) + (E+B+SF)	100%
AASHTO	D + (L+I) + CF + (E+B+SF)	100%
AREA	D + (L+I) + CF	100%

Notes:

1. This combination was assumed. Allowable criteria did not indicate specific combination.
2. See Table 4-3 for explanation of load symbols.

TABLE 4-2 LOAD FACTOR DESIGN

CRITERIA	GROUP I LOADS
MTA-NY	
PATCO	AREA Criteria (1)
BART	1.5 D + 1.8 L + I + CF+LF) (2)
WMATA	1.3[D + 5/3 (L+I) + (CF+RF) + (E-B+SF)]
MARTA	1.3[D + 5/3 (L+I) + (X) + (E+B)] (X) = CF or LF
MTA-BALT.	1.3[D + 5/3 (L+I+RF) + (CF) + 4/3 E + (B+SF)]
MIAMI	1.3[D + 5/3 (L+I+LF+NF+CF) + (E+B+SF)]
AASHTO	1.3[D + 5/3 (L+I) + CF + (E+B+SF)]
AREA	Not provided for in edition reviewed
ACI 443	1.3[D + 5/3 (L+I) + CF + (E+B+SF)]

Notes:
1. Load factor design not apparent in criteria reviewed.
2. For Prestressed Concrete Girders and Earthquake Design.
3. See Table 4-3 for explanation of load symbols.

TABLE 4-3 LOAD SYMBOLS DEFINITIONS

D =	Dead Load	NF =	Horizontal Impact Force
L =	Live Load	RF =	Rolling Force
I =	Impact due to Live Load	E =	Earth Pressure
CF =	Centrifugal Force	B =	Buoyancy
LF =	Longitudinal Force	SF =	Stream Flow Pressure

5.0 CONSTRUCTION AND COST

OVERVIEW OF THE MAJOR COMPONENTS OF A RAPID TRANSIT AERIAL STRUCTURE

In order to accurately assess the cost of an elevated structure it is necessary to break it into its two major components, superstructure and substructure. The costs associated with elevated structures are significantly influenced by design and geographic location. Aspects relating to design are:

a. Accurate determination of loads and loading conditions.
b. Utilization of materials and construction methods.
c. Ease of construction.
d. Standardization of repetitive modes of construction.

The geographical aspects relating to cost are:

a. Availability of local cement.
b. Availability of local acceptable aggregates.
c. Distance from steel mills and fabrication shops.
d. Proximity of good labor market.
e. Working conditions as defined by weather, local ordinances and working restrictions.

The above brief list of those items affecting costs, though incomplete, tend to direct attention toward the geographical location as a major contributor to increases or decreases in the cost of a specific design.

An example of this would be that of a superstructure design limited to concrete in an area of excellent, inexpensive aggregate and local cement mills, and steel mills far away and inadequate local fabricator shops. If the design is simple, repetitive, and easily supproted from grade, it is reasonably suggested that concrete would be the more economical to construct.

This premise is supported by the relative cost index for labor and materials published in the Engineering News Record in December 1979.

Local ordinances and working restrictions appear in various forms. Cost resulting from them are often improperly attributed to structure construction cost. For example, the building of elevated structure and its supporting foundations in public streets restricts methods and times of construction which would not be present in a private right-of-way. Resulting increase cost should, perhaps, be better identified as "in lieu of right-of-way".

5.1 SUBSTRUCTURE

When elevated structure elements are standardized, so that economy of repetition can be realized, cost variables are then largely determined by site requirements. These are:

a. Foundation conditions (Spread vs. pile footings)

b. Ground clearance (Pier height)

c. Support structure spacing (Girder span length)

Figure 5-1 graphically shows the relative impact of the variation in span length, pier height, and foundation conditions. Generally, where extensive segments of elevated structures are used, costs associated with spacing of piers and foundations remain, more or less, constant due to reduction in their numbers as spacing is increased. Relative costs of foundations and piers, however, measurably increase with increased height of piers. Foundation costs also vary depending on the quality of the soil.

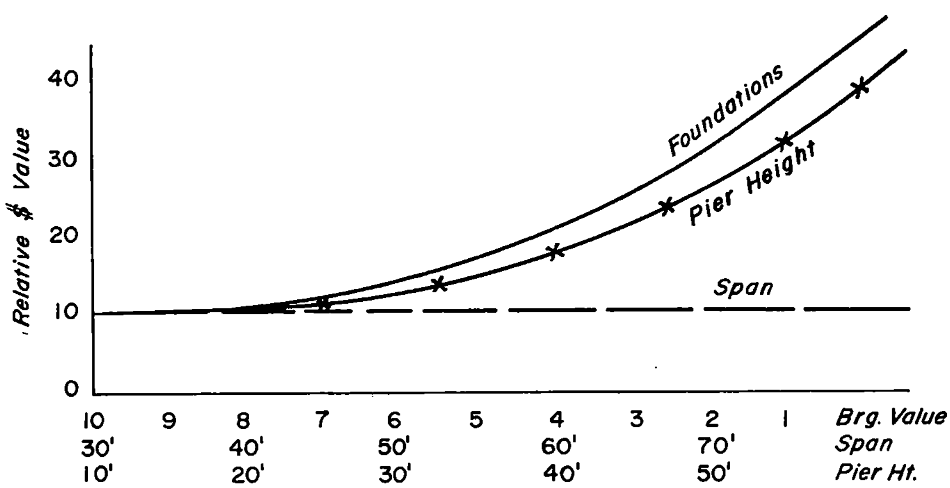


FIGURE 5-1

One factor that must be considered when making decisions for designs of structure elements for a major elevated section of a transit system is the future availability of a particular material. It must be kept in mind that the time between establishing a concept and construction can be anywhere from three to five or more years. The relationship between both availability and cost for concrete and steel can drastically change within that time frame. Construction methods and equipment can also experience significant change as can site access conditions.

Another factor that can significantly influence the cost is "buy American" contract clauses. Foreign suppliers, supported in some manner by favorable labor markets, tax considerations, or other considerations in their country, can provide very comparative prices to owners for such items as structural steel piles, etc. This requirement, or lack of it, must be known early in order to provide the designers with as much data to determine what material alternatives should be designed and which are not worth considering.

5.2 SUPERSTRUCTURE

The major area of cost variation in elevated structures due to design is in the area of the superstructure due to the large number of alternatives possible.

The design variations may include

a. Cast-in-place concrete structures.

b. Precast concrete structures.

c. Structural steel girders with cast-in-place deck slab.

d. Structural steel box section with either cast-in-place or precast deck.

c. Segmental precast concrete structures.

The above designs may result in relatively equal costs but only when we consider the geographical restraints noted previously.

It is imperative for economics in design, for the designer to be fully aware of relative costs of materials and labor in the project location prior to finalizing design. Occasionally, under ideal conditions, several designs must be provided to take full advantage of the economics prevailing in a given area. Any cost variation may be of such slight difference that the low bid may depend solely upon the risk the contractor is prepared to take.

To summarize, it is suggested that the superstructure costs will probably control the overall aerial structure cost. However, factors involved due to geographical location are possibly as important in the final cost for a given structure.

5.3 COST EXAMPLE

Typical costs - 80' spans - spread footings 18' high. (See Figure 5-2) Cost per linear feet of Aerial Structure - DBL Track, based on 1979 dollars.

Substructure	$ 416/Linear Feet
Superstructure	473/Linear Feet
Accoustical Barrier 2 sides	60/Linear Feet
Walkway	50/Linear Feet
	$ 999/Linear Feet

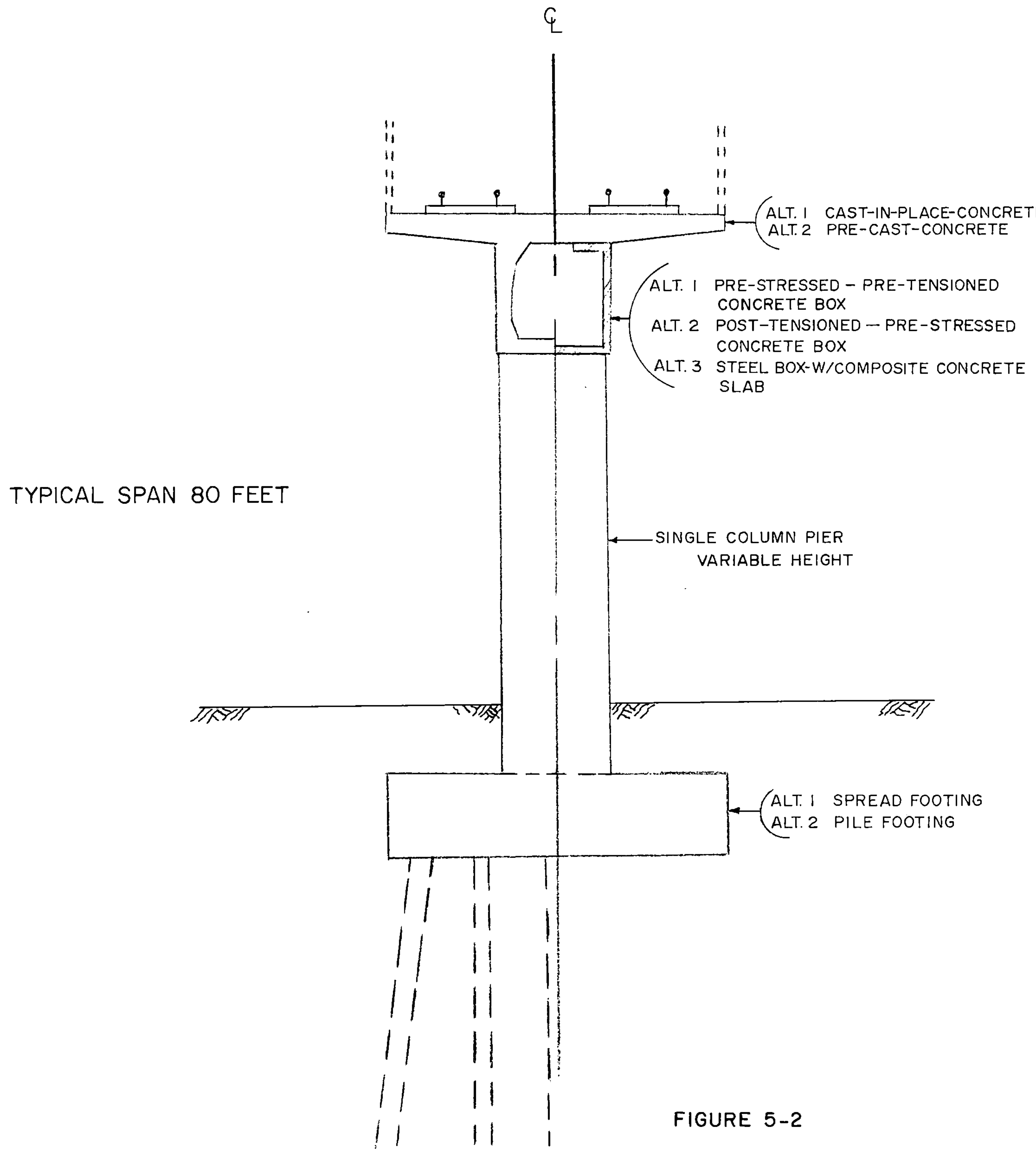


FIGURE 5-2

TYPICAL AERIAL STRUCTURE CONFIGURATION

The geographical location however may effect the overall cost by as much as 15% if alternative designs in either concrete or steel are not provided or encouraged. Along with the analysis of the materials market, a comparable study must be made to fully understand the effect of the labor market. Some locations will have extreme materials cost spread while the labor spread is reversed. The labor productivity must be analyzed and any design should incorporate labor reductions by simplification of fabrication.

Each design must be individually analyzed and costs must relate to materials, and labor components in that geographical location. Once a cost effective design has been developed for a specific area, that design should be retained only as long as it remains cost effective.

6.0 REVIEW OF SOME ELEVATED RAPID TRANSIT STRUCTURES

6.1 INTRODUCTION

The previous discussions have addressed the design criteria and their similarities and differences. It is obvious that there are differences of opinion as to what should be the basis for design. Review of the various structures designed in accordance with these criteria also finds obvious differences of opinion as to the application of the criteria. The type structures resulting from these criteria range from plate girders with concrete decks to precast-prestressed segmentally constructed structures. They have included single track, double track, triple track, and cross-over structures. It was also determined from the interviews held, that design consultants for elevated sections were both local and national firms. This implies that regardless of complexity or differences in design criteria, the resulting structures did not require unusual construction techniques. Which, in turn, probably resulted in reasonable bid prices for these structures.

The following paragraphs will briefly touch on a few of the newer standard structural superstructure sections designed under the provisions of the criteria reviewed. It should be pointed out that it is almost impossible, if not impractical, to review each of these designs and determine if a better design would have resulted from the use of a different criteria. Because of this, the review of these designs does not attempt to evaluate their merits or deficiencies, if any.

6.2 BALTIMORE

The criteria for design of the Baltimore System has been compared with others in Section 4 of this report. One thing that was particularly noticeable about the criteria used on the Baltimore system was the requirement to only use load factor design for both steel and concrete elevated structures. This trend toward use of the most current data available, as shown by the design criteria, is also evident in the contract drawings for their standard aerial structure. These drawings indicate that all line structures of steel box girders, shall be of unpainted A-588 Grade 50 steel.

This is the "weathering" type of steel that forms a self protective scale of an oxide that, once formed, prevents any further rusting of the steel. Even though this material is fairly commonplace, it is still noteworthy that the steel alternative was designed for its use. The use of the A-588 steel is probably tied to the fact that the steel box girders are not provided with access holes for inspection. There is historical data collected that provides the justification of not needing to paint the inside of steel box girders for non-weathering steel, nor th periodically inspect the interior.

The standard girder section for Baltimore is shown in Figure 6-1. This is a single track box structure. It was designed for alternates of steel with poured-in-place composite concrete deck and prestressed concrete box structure with both pretensioning and post-tensioning. The geometry of the structure is very similar to that of other systems having single track structures. The ends are notched at the bearings and the girders are super-elevated to help achieve the total superelevation required for curved alignments. The remaining required superelevation is provided by a second pour rail grout pad.

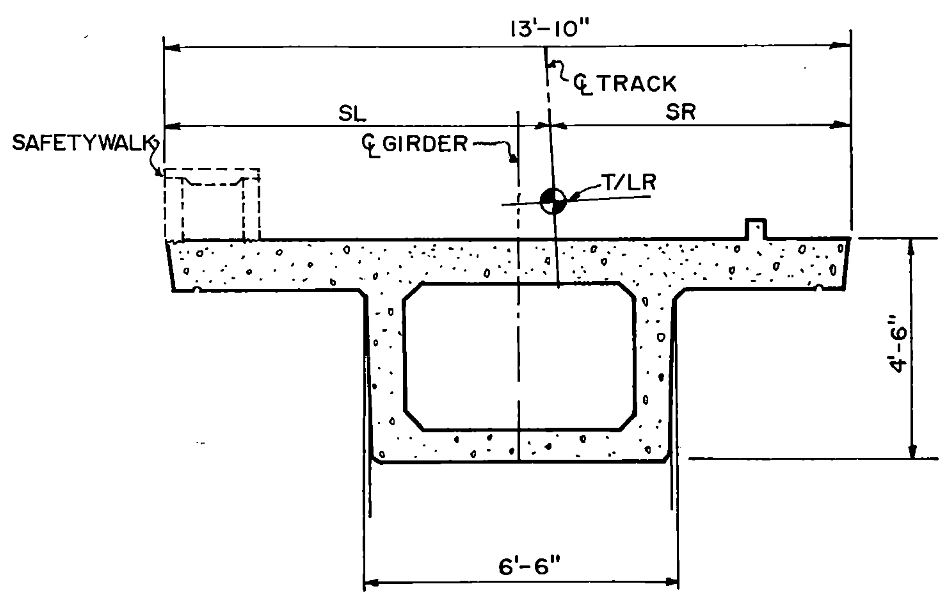


FIGURE 6-1

6.3 BART

The structure designed for the San Francisco Bay Area Rapid Transit District, BART, was the first "standard" structure designed for rapid transit systems using recent codes and materials. The structure designed was a single track box girder with a trapezoidal shape. See Figure 6-2.

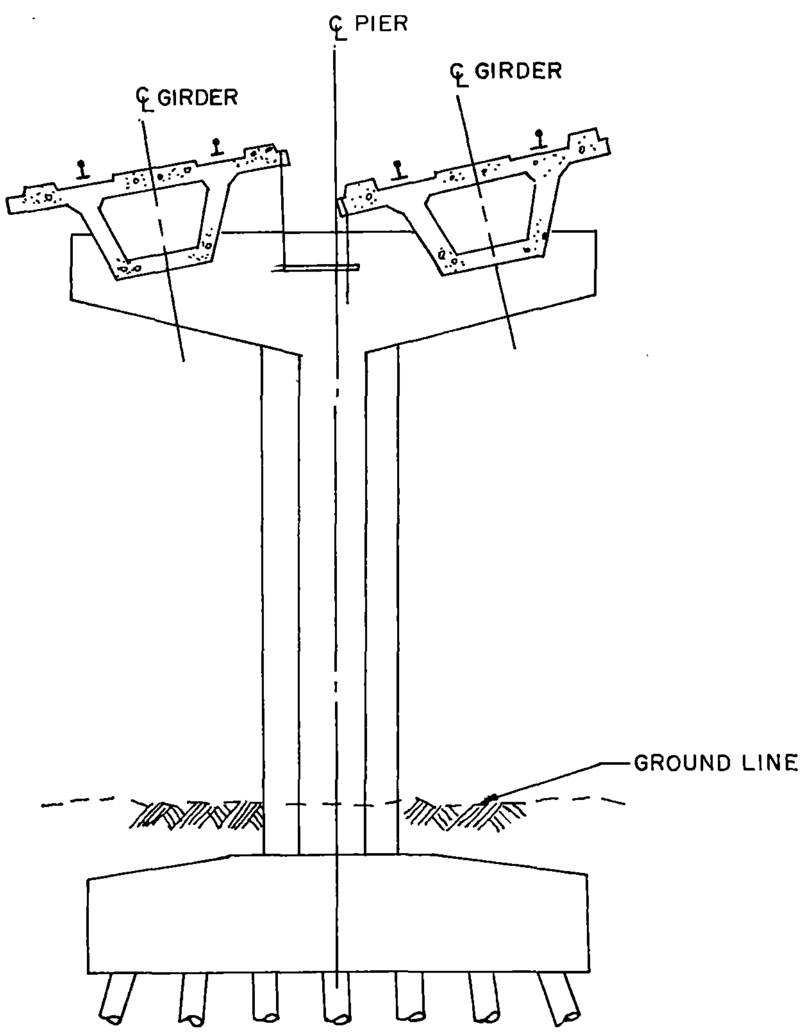


FIGURE 6-2

This structure was designed for both precast-prestressed concrete and structural steel. However, the precast-prestressed concrete units were used in most cases except where long spans dictated steel designs only. The piers were designed as hammerhead piers carrying two single track girders. Where the tracks widened to accommodate center platform stations one pier was provided under each girder.

The girder is superelevated to assist in obtaining the required superelevation for a curved alignment. Also, the girders are set on the bent caps and a cast-in-place closure strip is poured between the two girders, Figure 6-3.

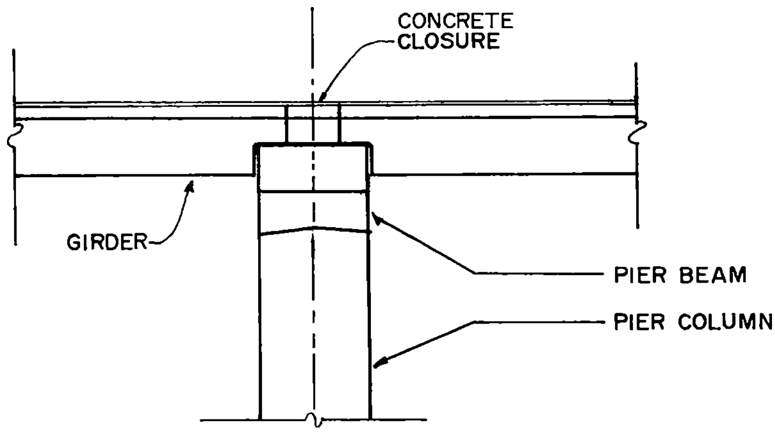


FIGURE 6-3

This closure pour engages dowels and horizontal shear keys in girders to resist lateral and longitudinal forces.

6.4 MARTA

The standard aerial girder section selected for the MARTA system, like Miami, is somewhat different than typical box girder or stringer sections generally used for elevated rail transit structures. The MARTA aerial girder has both precast concrete and steel box girder alternates and a precast deck slab which carries two tracks. The fact that the deck is carrying two tracks is unique within itself but in addition, it is mechanically connected to the girder to achieve composite action. See figure 6-4. Composite action was achieved through the use of high strength belts to fasten the deck to the girder. The tensioning of the bolt created a contact pressure between the two surfaces high enough to achieve composite action. The original concept was based on the shear-friction theory. Testing of this connection, under dynamic loading, was later performed by the Portland Cement Association Laboratories. The results proved the connection capable of transfering the composite loads for more than five million cycles.

BART and MARTA were the only two systems reviewed that had a seismic loading criteria. Even though MARTA's loading was in Zone II, it still required special shear keys to prevent movement detrimental to the structure. It should be pointed out that the Atlanta area straddles the line dividing seismic Zone I and II. Since most of federal agencies required their facilities in the area to be designed for Zone II requirements, the requirement was also applied to MARTA. The application of the seismic criteria apparently did not result in any undo hardship for design nor construction for the aerial structure.

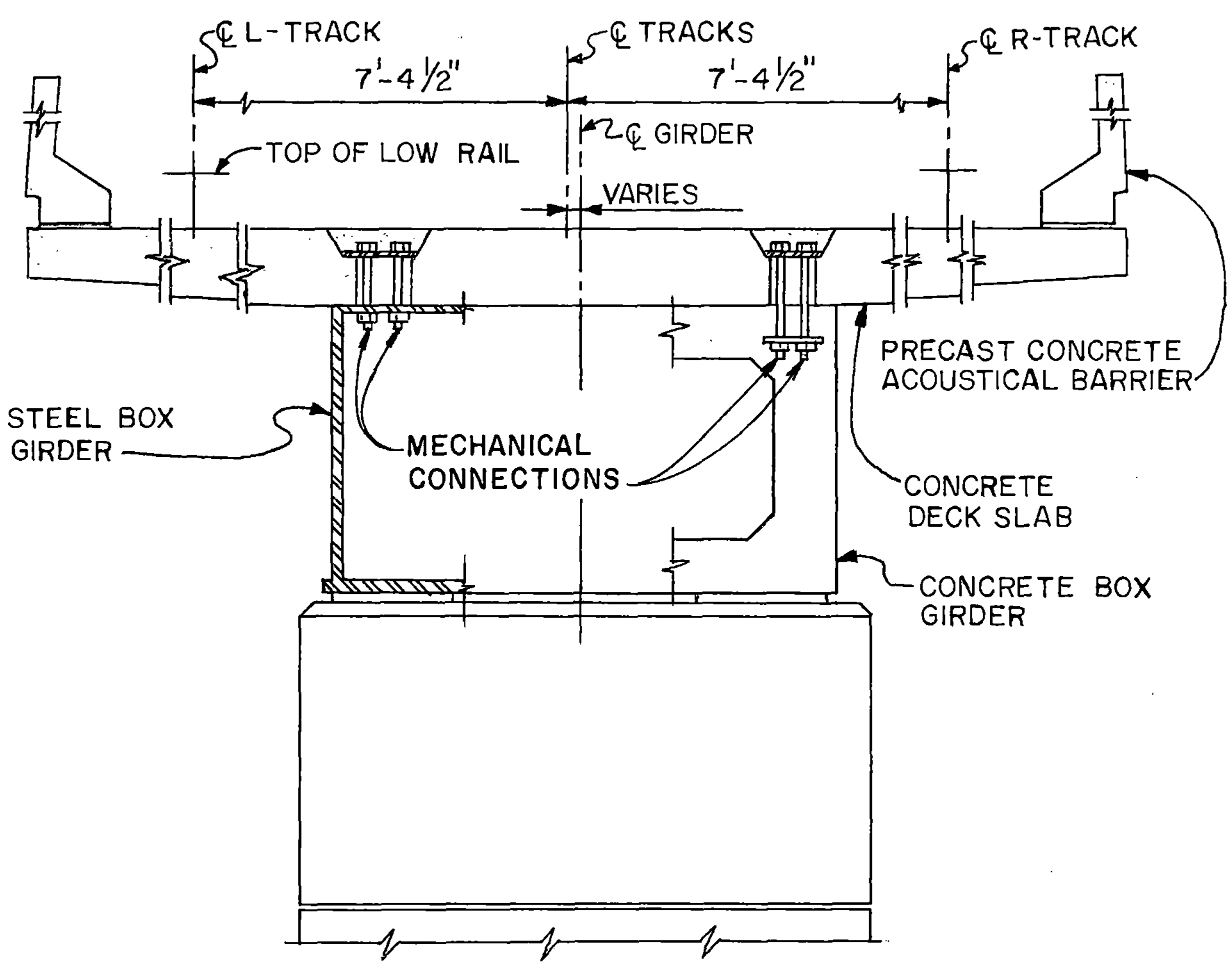


FIGURE 6-4

6.5 MIAMI

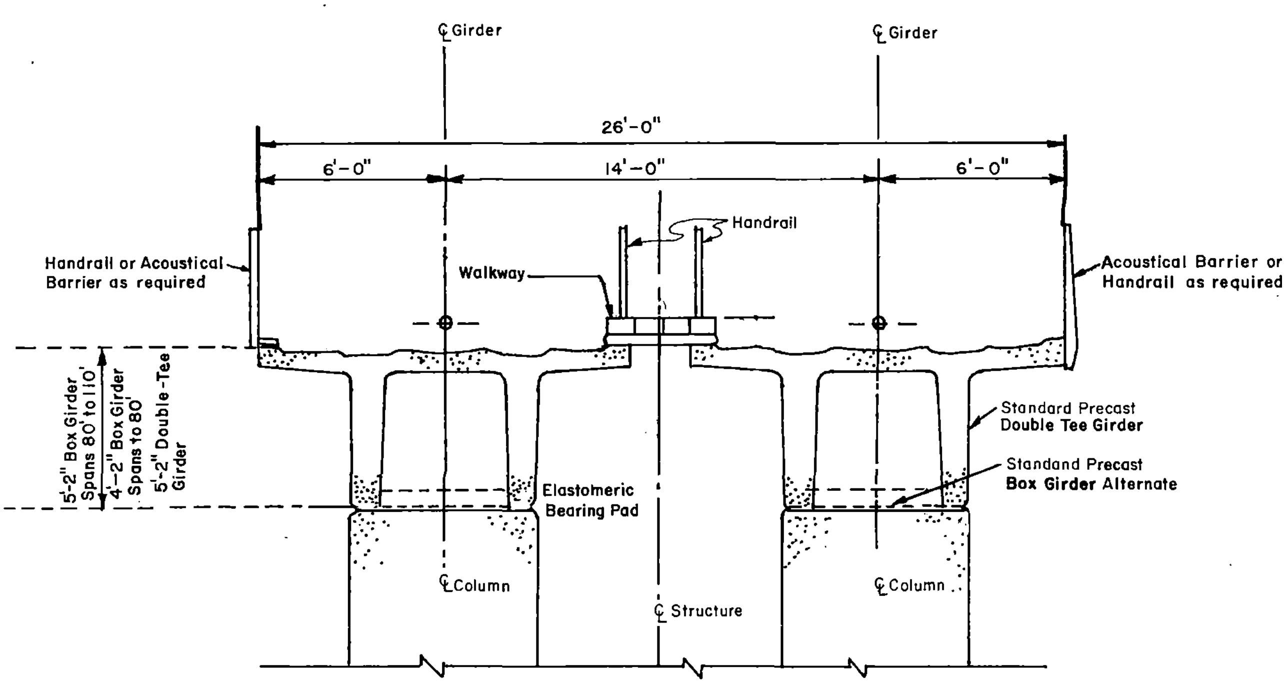
The standard aerial girder section, a double tee developed for the Miami-Dade County System is a very different concept for transit structures. The double tee cross section, however, is one that is very common and quite economical for building structures. Looking at other systems it is evident that the predominant cross-section is the single cell box girder, especially so on the new systems. The next choice appears to be well braced plate girder sections. Each of these latter two sections are quite capable of resisting torsional forces, especially the box section. At a first glance, it would appear that the unbraced double tee would not be adequate to resist torsional forces. However, it was proven by theoretical analysis and confirmed by full scale testing, including dynamic loading, to be capable of resisting the anticipated loading.

There are some positive points about the double tee that become evident after study. Concrete, and especially precast concrete, appeared to be the most likely construction material that would be used on the system due to geographical location and climate. Since the process and economics of precasting double tee's is a well proven fact, it was a reasonable step to adapt the standard aerial girder to that process. The entire girder could be poured in one step without the expense of costly interior formwork that would be needed with casting the box girders.

Miami is an area that is subject to very high winds, winds of hurricane force. A policy decision to discontinue train operations during periods of extraordinary wind velocities removed the need for otherwise costly and probably unrealistic loading conditions. By doing this the wind load on live load, which creates considerable trosional forces, was within range of the double tee. The higher hurricaneforce winds on the structure alone did not create torsional forces of any considerable amount and therefore the double tee was adequate for both loading conditions.

Also, the Miami criteria applied a very heavy penalty to the structure with its derailment criteria. In addition to setting the longitudinal axis of the care at derailment with a maximum possible eccentricity, one hundred percent impact was also added. This is a variation from both MARTA and ACI 443 which indicated impact is not to be applied. However after talking with the staff in Miami, it became clear that applicability of impact was used in an attempt to avoid the need for major repair of the structure in event of a derailment. Thus, system shutdown would not be required for other than clean up.

One fact that became evident during our discussions with the Miami staff is that; there evidently was a lot of interchange among operations planning people, equipment and facility designers during design. This apparently led to a criteria and a structure that met everyone's needs.



TYPICAL SECTION

MIAMI

FIGURE 6-5

6.6 SUMMARY

As in all fields of endeavor, progress in elevated transit structures has been made by innovation and application of methods and materials previously unused in the industry. Two obvious departures from previous practice are the double tee girder in Miami and the single steel box girder with composite concrete slab used by MARTA.

The motivation for using a particular elevated structure varies. Among the attributes listed for choosing particular girder/pier configurations are cost, speed of construction, aesthetics, space/clearance from other facilities, noise control, durability, and so on.

The successful accomplishment of all of the various design goals is not able to be judged until after a design has been installed for a number of years. The initial tests of aesthetics, noise control, and cost to construct have been successfully completed in most instances for the designs reviewed. The test of durability, or the structures ability to stand up under the tests of weather, repetitive loading, track maintenance, and the like, have yet to be reported for the newer systems. BART is the exception, having begun testing of the elevated structure in 1965.

Some distressing has been reported in the double tees used in Miami. At this writing, hairline cracks have appeared in the flanges. The cause of these cracks has not, as yet, been precisely determined and investigation is currently underway. The cracking has been judged not to diminish the structural capability of the girder. Repair is provided by sealing the cracks with an epoxy material to prevent moisture infiltration which might cause corrosion of reinforcing.

It is also reported that pier caps on the MARTA system have, in some instances, cracked. Also, storm drainage provisions from the deck at the pier leak and some evidence of corrosion exists.

The purpose in noting these conditons in newer structures is to identify the need for continued attention to design performance. Costs associated with developing a new facility are relatively easy to quantify. Long range costs of maintenance to preserve the structure in good condition are more difficult to accurately project. Past experience also suggests that the theoretical design life of a facility is much shorter than the actual useful life demanded in service. Durability of the structure should, therefore, be of equal concern as initial structural integrity to the designer. Knowledge of actual performance of these structures, correlated with their design criteria, would be of great value to designers of future elevated rail transit facilities.

REFERENCES

1. Lavis, Fred Building the Rapid Transit System of New York City, Engineering News, 1915.

2. American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-77) Detroit, Michigan, 1977.

3. Biggs, John M., Introduction to Structural Dynamics, McGraw-Hill, New York, 1964.

4. American Concrete Institute, Analysis and Design of Reinforced Concrete Bridge Structure (ACI 443), Detroit, Michigan, 1977.

5. American Iron and Steel Institute, Steel Structures for Mass Transit, Washington, D.C., Undated.

6. Parsons, Brinckerhoff-Tudor-Bechtel, San Francisco Bay Area Rapid Transit District Civil and Structural Design Criteria, Revised 1968.

7. Parsons, Brinckerhoff-Tudor-Bechtel, Metropolitan Atlanta Rapid Transit Authority Civil and Structural Design Criteria, April 1975.

8. Kaiser Transit Group, Metropolitan Dade County Transportation Improvement Program Compendium of Design Criteria, Volume III - Guideways Design Criteria, June 1978.

9. Daniel, Mann, Johnson, & Mendenhall/Kaiser Engineers, Mass Transit Administration Design Criteria Volume I, Baltimore, January 1976.

10. New York City Transit Authority Design Criteria, Chapter 3, New York City Transit Authority, New York, 1973 ed., 1934 ed., 1919 ed.

11. DeLeuw, Cather & Company, Washington Metropolitan Area Transit Authority Design Criteria, Washington, D.C., Undated.

12. Dorton, R. A. and Grovni, H. N., Review of Guideway Design Criteria in Existing Transit System Codes, Journal of the American Concrete Institute Volume 75, No. 4, April 1978.

13. American Association of State Highway Officials, Standard Specifications for Highway Bridges, 11th edition, 1973.

14. American Railway Engineering Association, Manual for Railway Engineering, 1972 edition.

15. American Public Transit Association, Rail Rapid Transit Systems, Technical Reference Report #1, Washington, D.C., Undated.

16. Bechtel Corporation, Scheffey, C. F. Dynamics of Vehicle-Structure Interaction Rapid Transit Structures, Bay Area Rapid Transit Authority, San Francisco.