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Adequacy of Three Highway Structures in Southern Nevada for Spent Fuel Transportation

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Abstract

The transportation of spent nuclear fuel across Nevada to the proposed repository site in the southern part of the state could significantly affect the infrasrtructure system in Nevada.

One of the major classes of structures affected by the nuclear waste transportation is highway bridges and culverts.

This report presents the results of the research work conducted during 1993. The study includes the evaluation of three bridge structures, G-965, B-976 and H-1020. The three structures are located on a potential nuclear waste transportation route in southern Nevada.

Where bridge upgrading is needed, retrofitting procedures are suggested and the associated cost estimates are presented.

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Chapter 1

Introduction

1.1 Introductory Remarks

The transportation of spent nuclear fuel across Nevada to the proposed repository site could significantly affect the infrastructure in Nevada. It is, therefore, essential that the potential impact be carefully studied to insure that a safe and reliable network of paved roads and railroads will be available.

One of the major classes of structures affected by the nuclear waste transport is highway bridges and culverts. These structures are generally complex, and the evaluation of their safety and performance involves a large number of factors. The wide variety of construction types, configurations, functions, etc. further complicates the treatment of bridge systems. The task of investigating the adequacy of the existing highway bridges in Nevada to carry the nuclear waste load must take into consideration the current condition of the bridges as well as the anticipated additional loads.

This report presents the results of the research conducted by the Civil Engineering Department at the University of Nevada, Reno, during 1993. The study includes the evaluation of specific bridges on a potential nuclear waste transportation route. Where bridge upgrading is needed, retrofitting procedures are suggested and the associated cost estimates are presented.

1.2 Objective and Scope

The primary objective of this study was to select some appropriate bridges and conduct a rather comprehensive study of their condition, structural capacity and repair cost, if any.

Before the above objectives are addressed, a data base for the existing highway bridges needs to be established for a selected route which is likely to be used to transport the spent fuel. In this case, the selected route extends from the Utah-Nevada border southward to the proposed repository site. This route includes Alternate US 93 from Wendover at the Utah state line to Lages, US 93 from Lages to Ely, US 6 from Ely to Tonopah, and then US 95 from Tonopah to the proposed repository site at Amargosa Valley. An alternate path, considered as a possible segment of the route in this study, includes Nevada State Route (SR) 318 from the junction of US 6 to US 93 south of Hiko, US 93 to I-15, I-15 to the junction of Craig Road north of Las Vegas, Craig Road between I-15 and US 95, and finally, US 95 to Amargosa Valley. A microcomputer data base for all the bridges and culverts on this route is available at the Nevada Department of Transportation (NDOT).

Due to the shortage of the available time, this study was limited to three bridge structures on the selected route, namely: a T-girder bridge (G-965), a rigid frame concrete box structure (H-1020), and a three-barrel concrete culvert (B-976). The research team conducted a site inspection for those bridges where deficiencies were noted and recorded according to the guidelines presented by Reference 11. The study also included structural analyses to compare the available and the required load carrying capacities of the bridges when they are subjected to the loads generated by the transportation of the spent nuclear fuel. The structural analyses included finite element models of the bridge structures. The scope of the finite element models was limited to gravity loads.

As a final segment of this study, rehabilitation alternatives were reviewed and the retrofitting costs were estimated.

Chapter 2

Field Inspection and Appraisal

2.1 Introduction

One of the primary objectives of this study was to select bridge systems that are on the proposed nuclear waste shipping route and carry out a relatively detailed evaluation of those bridges. The selection of the bridges, in addition to their description and the results of field inspection, are presented in this chapter.

2.2 Selection of Bridges for Detailed Study

For this phase of the study, the potential nuclear waste shipping route from the Utah border at Wendover south to the proposed repository site at Amargosa Valley, as shown in Figure 2.1, was examined.

The route includes US 93A from Wendover to Lages, US 93 from Lages to Ely, and US 6 from Ely to the junction of Nevada State Route (SR) 318. At this point two alternative routes may be selected: (A) SR 318 to US 93 south of Hiko, US 93 to I-15, I-15 to the junction of Craig Road north of Las Vegas, Craig Road between I-15 and US 95, and finally US 95 to Amargosa Valley; (B) west on US 6 to Tonopah and then south on US 95 to Amargosa Valley.

A study of the data base for bridge structures showed that 75 bridge-size (span $\geq$ 20') structures exist on this route (Table 2.1), of which 61 are concrete culverts, 3 are steel culverts, 1 is steel girder bridge and 10 are concrete bridges of various types as shown in Table 2.2. The distribution of these structures along the different route segments is summarized in Table 2.3 through Table 2.5.

The steel girder bridge (G-132) is located near Wendover and carries US 93A. Because this bridge has been recently replaced with a new structure, it was decided not to study it at this time. The 10 remaining bridge systems consist of 5 pairs of structures at 5 locations along alternative route A. Each pair comprises two identical bridges that carry traffic in opposite directions. One pair (I-1075 N & S) carries US 95 at the Mercury interchange. The other 4 pairs (G-958 N & S, G-961 N & S, G-965 N & S and I-969 N & S) carry I-15 between the junction of US 93 and Craig Road northeast of Las Vegas. Because the physical aspects and the age of I-1075 N & S are close to those of I-969 N & S, it was decided to exclude I-1075 N & S from this study, and consequently, from the preliminary site inspection. Thus, of all the bridges along the route considered in this study, only those that carry I-15 were selected for further investigation.

Although the evaluation of bridges can be more involved than that for culverts, it is realized that, in terms of safety and transport of nuclear waste, culverts are as important as bridges. Moreover, culverts constitute about 85% of the bridge-size structures along the selected route in this study. Therefore, it was important in this project to include the analysis of some culverts. Three concrete culvert-type structures, of which two are twin structures, were selected for inspection during the preliminary site visit. The selection was primarily based on providing different types of concrete culvert structures: a multi-barrel box culvert with skew angle of 30 degrees (structure B-976) and a single-barrel box frame with no skew (structures H-1020 N & S). Similar to the selected bridges, these structures also carry I-15 northeast of Las Vegas.

Because of the limited time of this study, it was decided to analyze one bridge and two culverts of different features. On February 25, 1993, the research team visited the pre-selected structures. A general inspection was conducted to uncover signs of major distress, if any, that would make any one of those structures a more favorable candidate for further analysis.

In general, all four pairs of bridges revealed some minor deficiencies which were primarily related to serviceability. Since the physical conditions of the bridges were similar, it was decided that the bridge type should be the criterion for selecting one bridge for further analysis. Having analyzed slab and box-girder bridges in previous studies [9,12], it was agreed that bridge G-965 (N & S), a twin concrete T-girder type bridge, will be considered for further analysis. Since the two inspected culvert structures were of different configurations, it was decided that both structures, B-976 and H-1020 (N & S), would also be considered for further analysis in this study. A detailed discussion of the observations made during the site inspection is presented in Section 2.5.

2.3 General Description of the Selected Bridges

2.3.1 Description of G-965 (N & S); Concrete T-Girder

G-965 (N & S) is located in Clark County and carries I-15 over Union Pacific Railroad (UPRR) spur north of Las Vegas, Nevada. The bridge facility is a pair of identical, but separate, bridges that carry traffic in opposite directions (North bound and South bound). Each bridge is a continuous three-span reinforced concrete structure with an overall span length of 174 feet. The bridge deck is simply supported at the abutments and rigidly connected to the bents at the two interior supports.

The structures were built in 1963. Each deck is a 42-foot wide cast-in-place concrete T-girder system. The deck consists of six equally spaced girders and a 7-inch thick slab. The girder depth varies gradually from 6′-6″ and 4′-0″ at the interior and exterior supports, respectively, to 2′-9″ at mid-spans between the supports. The deck is skewed at an angle of 20 degrees and carries two traffic lanes heading in the same direction. Bridges G-965 N & S are shown in Figure 2.2.

At each abutment, The superstructure rests on six elastomeric pads. Each abutment bears on eleven concrete piles. The piles were driven to a minimum bearing of 30 tons each. At the interior supports, each bent cap is supported by three 3-foot diameter concrete circular columns. Single footings are provided to carry the loads transmitted by the columns.

The structure was designed for the American Association of State Highway and Transportation Officials (AASHTO) HS-20 loading [2]. The concrete was specified to have a minimum strength of 3,000 psi at 28 days. Reinforcing steel was specified as "intermediate grade" (f_y = 50,000 psi) with an allowable stress of 20,000 psi.

2.3.2 Description of B-976; Three-Barrel Culvert

B-976 is located in Clark County and carries I-15 over Vandenberg Wash northeast of Las Vegas, Nevada. The bridge facility was built in 1970 and is a three-barrel reinforced concrete culvert that carries four traffic lanes, two in each direction.

The structure is 45 feet long, 245 feet wide, and is skewed at an angle of 30 degrees with respect to the road centerline. A cross section of B-976 is shown in Figure 2.3.

The culvert is a "triple 12′ x 10′" which means that each one of the three barrels has a clear space that is 12 feet wide and 10 feet high. The exterior and interior walls are 8-inch thick each. The top slab is 12-inch thick and is about 6 feet below final grade. The invert slab is 12.5-inch thick.

The structure was designed to carry AASHTO HS-20 loading. The minimum concrete compressive strength was specified as 3,000 psi at 28 days. The reinforcing steel was specified as "intermediate grade" with an allowable stress of 20,000 psi.

2.3.3 Description of H-1020 (N & S); One-Barrel Box Frame

H-1020 N and H-1020 S are located in Clark County and carry I-15 in opposite directions over Navy Road northeast of Las Vegas, Nevada. The two structures are identical and each carry two traffic lanes heading in the same direction. Each bridge is a reinforced concrete single-barrel box frame. Each frame is 16 feet long and 38 feet wide. Both structures have no skew.

H-1020 (N or S) has a clear interior space of 14′-0″ width and 14′-0″ height. A uniform thickness of 11 inches is used throughout the box, i.e for the walls, top slab and invert slab. The top slab is about 2.5 feet below final grade level. A cross section of H-1020 is shown in Figure 2.4.

The structure was designed to carry AASHTO HS-20 loading. The minimum concrete compressive strength was specified as 3,000 psi at 28 days. The reinforcing steel was specified as "intermediate grade" with an allowable stress of 20,000 psi.

2.4 National Bridge Inventory and Recent Inspection Data

Part of the background investigation was to examine the National Bridge Inventory data and the bridge inspection data maintained by the Nevada Department of Transportation (NDOT). The NDOT inspection program is designed so that each bridge is inspected at least every 2 years. An in-depth inspection, using a "cherry picker"-type bridge inspection vehicle is performed every 6 years. Annual or special inspections are performed as warranted. G-965, B-976 and H-1020 structures were last inspected in early 1991.

2.4.1 Structure G-965 (N & S)

Listed are the important facts concerning G-965.

- The structure is located in NDOT District I on I-15 at milepost CL-53.66, northeast of Las Vegas, Nevada.
- I-15 has a "mainline" designated level of service and is a defense highway. The functional classification for I-15 is "principal arterial—interstate."
- The detour length is zero miles since traffic can be channeled to one of the twin bridges to bypass the other.
- The bridge is owned and maintained by the State of Nevada.
- The average daily traffic volume was 5342 vehicles in 1989, of which 27 percent was truck traffic. The average daily traffic is expected to increase to 14,523 vehicles by the year 2011.

- The approach roadway width and the deck roadway width are both 38 feet, and carry two lanes of traffic in the same direction. The overall structure width is 43 feet.
- The structure consists of three spans; there are no approach spans. The bridge's overall length is 174 feet. Each exterior span is approximately 52 feet long. The interior span is 74 feet long.
- The bridge supports are skewed at 20 degrees to the centerline of the deck.
- Traffic safety features meet currently acceptable standards.
- The operating rating, i.e., normal overload rating, is 59 tons maximum, based on AASHTO HS load configuration. The normal inventory rating is 36 tons maximum, based on HS load configuration.
- The structure is open to traffic with no restrictions, and has no posted load limit.
- The bridge has no fracture critical components.
- No future improvements are pending or have been proposed.
- The bridge has an asphalt concrete wearing surface, and has no other protective system.
- The deck condition and superstructure condition ratings are both 8, indicating "very good condition—no problems noted."
- The substructure rating is 8, indicating "very good condition—no problems noted."
- The overall structural evaluation is 8.
- The sufficiency rating is 96.5.
- The bridge is neither "structurally deficient" nor "functionally obsolete."

2.4.2 Structure B-976

Listed are the important facts concerning B-976.

- The structure is located in NDOT District I on I-15 at milepost CL-49.51, north of Las Vegas, Nevada.
- I-15 has a "mainline" designated level of service and is a defense highway. The functional classification for I-15 is "principal arterial—interstate."
- The detour or bypass length is 3 miles.
- The bridge is owned and maintained by the State of Nevada.
- The average daily traffic volume was 10,105 vehicles in 1989, of which 30 percent was truck traffic. The average daily traffic is expected to increase to 30,000 vehicles by the year 2011.
- The approach roadway width is 38 feet in each direction. The structure carries four traffic lanes, two in each direction.
- The structure consists of three main spans; there are no approach spans. The bridge's overall length is 45 feet.
- The culvert has a skew of 30 degrees.
- Traffic safety features meet currently acceptable standards.
- The operating rating, i.e., normal overload rating, is 73 tons maximum, based on AASHTO HS load configuration. The normal inventory rating is 36 tons maximum, based on HS load configuration.
- The structure is open to traffic with no restrictions, and has no posted load limit.
- No future improvements are pending or have been proposed.

- The deck condition and superstructure condition ratings are both 8, indicating "very good condition—no problems noted."
- The overall structural evaluation is 8.
- The channel rating is 6, indicating "bank is beginning to slump."
- Scour conditions rating is 8, indicating that the "foundation is stable for calculated scour conditions."
- The sufficiency rating is 80.2.
- The culvert is neither "structurally deficient" nor "functionally obsolete."

2.4.3 Structure H-1020 (N & S)

Listed are important facts concerning H-1020.

- The structure is located in NDOT District I on I-15 at milepost CL-55.82, north of Las Vegas, Nevada.
- I-15 has a "mainline" designated level of service and is a defense highway. The functional classification for I-15 is "principal arterial—interstate."
- The detour or bypass length is 1 mile.
- The bridge is owned and maintained by the State of Nevada.
- The average daily traffic volume was 5342 vehicles in 1989, of which 26 percent was truck traffic. The average daily traffic is expected to increase to 9,500 vehicles by the year 2011.
- The approach roadway and the bridge widths are 38 feet and 43.3 feet, respectively. The structure carries two traffic lanes in the same direction. The overall structure width is 43.3 feet.

- The structure consists of one span; there are no approach spans. The bridge's overall length is 16 feet.
- The box frame is not skewed.
- Traffic safety features meet currently acceptable standards.
- The operating rating, i.e., normal overload rating, is 73 tons maximum, based on AASHTO HS load configuration. The normal inventory rating is 36 tons maximum, based on HS load configuration.
- The structure is open to traffic with no restrictions, and has no posted load limit.
- No future improvements are pending or have been proposed.
- The deck condition and superstructure condition ratings are both 8, indicating "very good condition—no problems noted."
- The overall structural evaluation is 8.
- The sufficiency rating is 92.6.
- The structure is neither "structurally deficient" nor "functionally obsolete."

2.5 Field Reconnaissance

The research team performed two field inspection trips, first on February 25, 1993, and then on June 11, 1993. The purpose of the first visit was to determine which bridge structures on the selected route should be considered for further analysis in this study. The candidate bridges (G-958-N&S, B-976, G-961 N&S, G-965 N&S, H-1020 N&S, and I-969 N&S) were inspected in a general way and the observed deficiencies were noted. Based on the preliminary observations, three bridge systems (G-965 N&S, B-976 and H-1020 N&S) were selected for further analysis. During the second visit, only the selected bridges were inspected again in greater detail. Based on the federal and state inspection guides [1], the inspection level performed by the researchers was more detailed than "routine bridge inspection" but not as comprehensive as a "detailed inspection". During the inspection, the actual bridge structures were first compared with the drawings in a general way to identify any visible discrepancy. No attempt was made to determine the actual dimensions, the location and amount of steel, or the insitu material properties. It was noted that there is general agreement between the drawings and the as-built bridges.

2.5.1 Inspection of Bridge G-965 N

Inspection of the structural condition and the traffic safety revealed the following:

Deck

The top of the deck was covered with asphalt material and could not be viewed from the top to check for cracks in the concrete. The bottom of the superstructure, however, was surveyed for cracks and other signs of distress by looking up at the bottom of the deck. Since no lifting equipment were available at the time of inspection, the deck bottom of the middle span was too high to be checked for cracks or other signs of distress, and thus, no crack survey was carried out for that part of the deck.

The bottom of the deck at both the north and the south spans had some minor pattern cracking. Thin shrinkage cracks which are formed in a random pattern are considered minor in this study. All accessible girders were observed to have thin, but noticeable, flexural cracks in the webs, spaced at 2 to 3 feet apart. The crack widths were not measured, but it was estimated that none of the visible cracks was greater than 0.016 inch in width. Thin full-height cracks were also visible in the outer diaphragms.

No marks of corrosion or signs of efflorescence (a white deposit caused by the crystallization of soluble chemicals carried to the surface by water) were detected on the underside of the deck.

In general, the slider bearings were centered and clear of debris. The second east bearing at the north abutment was 0.5 to 1.5 inches off center in the expansion direction as shown in Figure 2.5. At the south abutment, the first east pad under the bearing had a lateral crack.

Bents

The inspection of the bents was limited to the lower portions of the bent columns. The upper portions of the columns and the bent caps were inaccessible in the absence of proper equipment. The columns appeared to be in very good condition with no visible cracks or other deficiencies. Slight slope erosion near the columns was noted.

Abutments

The survey of the abutments revealed minor vertical cracks in the south end wing walls. Minor rust stains and thin flexural cracks (in the negative moment region) were observed at the south abutment.

Traffic Safety

The observations which are primarily related to traffic safety were as follows:

- The asphalt on the deck appeared to be in good condition. Minor ruts were observed in the right traffic lane.
- The expansion joints at the two ends of the bridge appeared to be in good condition when looked at from the top. No signs of water leakage through the joints were observed underneath the deck.
- The concrete barrier rails had many vertical cracks with some rust stains at some locations.
- The galvanized steel guardrails at the transition segments appeared to be in sound condition.
- The bridge approaches were in good condition except for some undermined guardrail posts at the east side of the south approach.

2.5.2 Inspection of Bridge G-965 S

Following a visual inspection similar to that of bridge G-965 N, the following observations were noted.

<u>Deck</u>

The top deck was covered with asphalt material and could not be surveyed for cracks. Surveying the bottom of the superstructure at the end spans revealed some typical flexural cracks in the bridge girders similar to those in G-965N. The cracks were narrow and the width of each crack was estimated to be less than 0.016 inch. A moderately wide crack of about 5 inches long was observed at the top of the second east girder of the north span. The deck bottom did not show any signs of corrosion or efflorescence. The bearings were in good shape except for one improperly set spacer at the west-most bearing of the north abutment.

Bents

The bent caps could not be inspected closely, but the columns appeared to be in very good condition. Minor erosion was noticed near the columns.

Abutments

The abutments were in very good condition except for minor vertical cracks in the north end wing walls.

Traffic Safety

- The asphalt on the deck appeared to be in good condition. Minor ruts were observed in the right traffic lane at midspan. The approach span had many patched potholes. Some minor flushing was noticed at the wheel tracks.
- The expansion joints at the two ends of the bridge appeared to be in good condition when looked at from the top. No signs of water leakage through the joints were observed underneath the deck.
- The concrete barrier rails had many vertical cracks with some rust stains at some locations.
- The galvanized steel guardrails at the transition segments appeared to be in sound condition.
- The bridge approach guardrail were in good condition but for a traffic damage in the right lane at about 150 feet before the bridge.

2.5.3 Inspection of Bridge B-976

The inspection of the three-barrel culvert covered the interior of each cell (walls and ceilings), in addition to the wing walls.

Two minor spalled concrete areas were noted at the east end of the interior walls as shown in Figure2.6.

The exterior wall of the south cell had a horizontal crack as well as some vertical cracks as shown in Figure 2.7. Those cracks were considered thin except for the first vertical crack from the east end where the measured crack width was about 0.03 inch. Diagonal cracks were noted at the top of the cell walls near the ends. At the south-east, north-west and south-west corners, the diagonal cracks extended into the wing walls and the crack width was large, ranging approximately between 0.1 inch to 0.2 inch (see Figure 2.8). The interior cell had narrow vertical cracks in the walls that continued across through the bottom of the top slab. Such cracks occurred at intervals ranging approximately between 6 feet to 10 feet.

Signs of efflorescence and traces of water leakage were visible in several spots at the bottom of the top slab. Minor erosion was observed near the top of the wing wall at the south-west.

2.5.4 Inspection of Bridges H-1020 N & S

Visual inspection of the north- and south- bound box-structures was limited to the inside of the barrels. The inspection revealed two continuous horizontal cracks throughout each wall of both structures. The horizontal cracks were located at approximately 4 feet and 8 feet above floor level. Vertical thin cracks, approximately presence of some wide cracks in the walls of culvert B-976 is an indication of deterioration.

The allowable crack width under service loads is related to an index which is specified in different codes. Because one of the primary reasons for crack width control is to protect the steel from corrosion, the environmental conditions to which the member is subjected to is an important factor. According to the ACI code [3], the crack width index factor, "Z", is limited to 145 for exterior exposure and 175 for interior exposure (controlled environment). These values have been slightly lowered and adopted by AASHTO [2], where the "Z" limits are 130 for severe exposure and 170 for moderate exposure.

The above "Z" values are related to the width of the crack. The crack width index of 130 and 170 correspond to crack widths of 0.0119 inch and 0.0155 inch, respectively. Many cracks in culvert B-976 exceed the upper crack limit of 0.0155 inch.

Based on the field inspection, it is expected that the NDOT ratings for the superstructure of bridge B-976 would be reduced. The superstructure rating of "8" would reduce to "7" because of the cracks, efflorescence marks and rust stains. For G-965 (N & S) and H-1020 (N & S), the NDOT rating of "8" for both the superstructure and the substructure seem to be appropriate and no change is expected for the current bridge conditions.

Combined Sufficiency Rating = $S_1 + S_2 + S_3 - S_4 = 80.2\%$. This figure agrees with the rating calculated by NDOT.

Based on the site inspection, the rating of the structural evaluation (Item #67) might reduce from 8 to 7. This reduction will not affect the sufficiency rating. The updated average daily traffic of 12590 (1992 figures) would reduce S_3 from 10.2 to 9.6. The revised sufficiency rating will reduce to 79.6%.

3.2.3 Sufficiency Rating for H-1020

1- Structural Adequacy and Safety

$A = B = C = D = E = F = G = H = 0$

$AIT = 36 \times 1.00 = 36$

$I = (36 - 36)^{1.5} \times 0.2778 = 0$

$S_1 = 55 - 0 = 55$

2- Serviceability and Functional Obsolescence

Rating Reduction, $J = 5$

Width of Roadway Insufficiency: $G = H = 0$

Vertical Clearance Insufficiency, $I = 0$

$S_2 = 30 - 5 = 25$

3- Essentiality for Public Use

$A = 0.4$

$B = 2$

$S_3 = 15 - (0.4 + 2) = 12.6$

4- Special Reduction

$S_1 + S_2 + S_3 = 92.6 > 50$, therefore, S_4 applies.

$A = 0$

$B = 0$

$C = 0$

$S_4 = 0 + 0 + 0 = 0$

Combined Sufficiency Rating $= S_1 + S_2 + S_3 - S_4 = 92.6\%$. This figure agrees with the rating calculated by NDOT.

The updated average daily traffic (ADT) of 6,295 (1992 figures) would reduce S_3 from 12.6 to 12.5. All other rating categories would not be affected. Thus, the revised sufficiency rating would reduce to 92.5%.

3.3 Structural Analysis

For the evaluation of the structural adequacy of each bridge, the following steps were taken: (1) The actual capacity of the bridge was first identified by reviewing the bridge design notes, if available, and by calculating the load carrying capacity according to current specifications [2], (2) the vehicle types that can potentially be used for the nuclear waste transportation were identified, and (3) an analysis of the bridge was performed to identify the strength requirements for the transportation of the spent fuel. These strength requirements were then compared to the available structural capacity of the bridge.

The load carrying capacities of the three structures were determined based on information presented in the bridge design notes [5] and approved drawings. All three bridges were designed using the "working stress method" and according to AASHO standard specifications effective at the time. In the case of B-976 and H-1020, no design notes were available for review. Based on required criteria, the two culvert-type structures were selected from predesigned set of structures. The loading was based on AASHO HS-20 load which is similar to the HS-20 load used in the current AASHTO specifications [2]. A typical HS-20 truck is shown in Figure 3.1. However, the current AASHTO method of distributing the load to the different structural elements of a bridge deck is different from that specified two and three decades ago.

Based on necessity and cost considerations, overweight truck shipments for transporting spent fuel from reactors to repository facilities have been considered as an attractive option for a long time. An analysis related to several factors involved in the development of an overweight truck cask are presented in Reference 6. Table 3.1 shows four cask/vehicle systems which include the current legal-weight cask (LWT_1), the current overweight cask (OWT_1), the future legal-weight cask (LWT_2) and the future overweight cask (OWT_2). A representative vehicle/trailer configuration that would potentially in the future accommodate the 40-ton cask of OWT_2 is shown in Figure 3.2. It consists of seven axles and it is a 4-3 configuration which includes four axles attached to the vehicle and three axles attached to the trailer. When the 40-ton cask is placed on the trailer, the axle weights are 11,000 pounds on the steering axle, 51,000 pounds on the tridem main vehicle axle and 54,000 pounds on tridem trailer axle. The gross weight of the vehicle is 116,000 pounds. Typical tridem and tandem wheel configurations are shown in Figure 3.3.

The OWT_2, which is described above, and the AASHTO's HS-20 loading were chosen as the critical configurations for loading the bridges and checking their structural adequacy for carrying the loads.

According to the latest AASHTO method of load distribution, a load type may be applied either as a truck load (concentrated loads) or as a lane load (distributed and concentrated loads). The loading arrangement which produces the most critical stress values is considered for design. In the case of nuclear waste transportation vehicle loads, only truck loads can be considered in structural analyses since no alternative lane loads are available yet (as is the case with H and HS loads).

A proposed alternate design method for the future is the Load and Resistance Factor Design Method (LRFD). In this method, the load and capacity reduction factors and load combinations are based on statistical considerations such that equally critical structures would have similar factors of safety. This method has not been adopted yet by AASHTO for bridge construction. However, a draft of the proposed LRFD bridge specifications [10] is under review by participants in the work of the National Cooperative Highway Research Program. In this study, it was felt that it would be appropriate to apply the LRFD approach, in addition to the current ultimate strength method, in checking the safety of the bridges under the applied loads. Due to time limitation, however, the LRFD method was applied only to check flexure and shear in the girders of bridge G-965.

3.3.1 Analysis of Bridge G-965

Based on the bridge design notes, G-965 was modeled and analyzed as a two-dimensional three-span continuous structure as shown in Figure 3.4. The bridge deck was considered simply supported at the abutments and rigidly connected to bents at the two interior supports. The bents had total fixities at the footings. The impact factor applied to flexure and shear was 1.28 for all girder spans. The critical design bending moments and shear forces at different locations along the spans are shown in Table 3.2.

In this study, a three-dimensional finite element model was used to analyze the bridge. The model was developed using transverse continuous beams, representing 4-foot wide slab strips, spanning between the six longitudinal girders. The girders, in turn, span between the end supports at the abutments and the bent caps. The variable depth of each girder was accounted for by dividing the girder into elements spanning between transverse beams. The depth of each girder element was considered as the average depth between the two transverse beams it spans. The finite element model, which was analyzed using the program IMAGES 3D [4], is shown in Figure 3.5.

The applied impact factors were 29% for positive moments and shear forces, and 27% for negative moments. The load distribution factors for the interior girders were found to be 0.750 and 0.625 for lane loads and truck loads, respectively. The impact and load distribution factors were based on the current AASHTO requirements for bridge construction [2].

Live load shear and bending moment influence lines for each girder were produced by applying a set of six unit loads, placed transversely on the girders along the skew of the bridge. The load set was moved at intervals of 1/10th the span length along the traffic direction. Such a load arrangement would represent a fully loaded deck. The influence line diagrams were then used in conjunction with the applied vehicle loads, adjusted for distribution factors and impact, to find the critical live load shear and bending moments along the girders under both load configurations, HS-20 and OWT_2. In the case of HS-20 load, the lane load and the truck load were both checked at all nodes to select the governing values. The highest live load shear forces and bending moments were found to occur at the first interior girder. These values (including impact) are listed in Table 3.3 and Table 3.4.

The dead load of the structure due to self-weight and asphalt paving was analyzed and the corresponding shear forces and bending moments are presented in Table 3.3 and Table 3.4. The load cases for thermal, longitudinal, wind on structure, and wind on moving loads were also checked, but were found non-critical when considering the load groups prescribed in Table 3.22.1 A of Reference 2. The most critical loading combination was found to be Load Group IA. The resulting ultimate shear forces and bending moments at the first interior girder are presented in Table 3.3 and Table 3.4. When these values are compared to the corresponding factored nominal capacities, it is found that the girder would be safe in shear and bending moment under the two different load types considered in this study.

The interior girder was also checked for flexure and shear according to the LRFD method. This method is presented in Reference 10 and is basically a multiple check of different "limit states". In this study, only the Strength Limit State, which parallels the ultimate condition in the strength method, was checked. The different factors used in this analysis were as follows:

- Limit State Factors:

  Ductility, η_D = 0.95

  Redundancy, η_R = 1.05

  Importance, η_I = 1.05

- Impact Factor = 0.5 (does not apply to lane load)
- Multiple Presence Factor, m = 0.85 (for 3 or more loaded lanes)
- Load Distribution Factors (Interior Girders):

  For Positive Moments = 0.68

  For Negative Moments = 0.65

  For Shear = 0.78

- Correction Factors for Skewed Bridges:

  For Moments = 0.78

  For Shear = 1.07

- Resistance Factor, ϕ, for Flexure and Shear = 0.9

Using the influence lines described before and the two different load configurations of HS-20 and OWT_2, the factored bending moments and shear forces were calculated for the first interior girder and are presented in Table 3.5 and Table 3.6. When these values are compared to the factored nominal capacities, it can be concluded that the girder is safe in shear and flexure.

Due to time constraints, the analysis of bridge G-965 is limited to the deck girders.

3.3.2 Analysis of Bridge B-976

In this study, the analysis of the three-barrel culvert was completed by analyzing the four different structural elements that make up the culvert: the top slab, the exterior walls, the interior walls, and the invert slab.

Top Slab

The top slab was modeled as a continuous three-span beam with pin or roller supports over the exterior walls (Figure 3.6). The model supports were based on the reinforcement details of the slab-wall connections which do not allow for moment transfer.

The vertical downward earth pressure on the slab was based on 6-foot deep fill as shown on the alignment profile. AASHTO specifications require that only truck loads be considered in the design and analysis of culvert structures. Each wheel load is transformed through the soil fill into a uniformly distributed load acting on a square area whose side is 1.75 the depth under the wheel load [2]. For multiple wheel loads whose distribution regions overlap, the overall load distribution area would be the zone confined within the outer envelop of the overlapping regions. In this region, a uniform load distribution is assumed. In the case of B-976, three loaded lanes were placed in each traffic direction. At a depth of 6 feet, the distribution areas resulting from loads placed in the opposite traffic directions do not overlap. Therefore, only one loaded direction was needed for the analysis, resulting in a load distribution area that is 38.5 feet wide and 10.5 feet long along each axle as shown in Figure 3.7. No impact factor was added since impact is neglected below a depth of 3 feet. However, a load reduction factor of 0.9 was introduced since three lanes were loaded simultaneously.

With the aid of influence line diagrams, the maximum live load shear forces and bending moments were found along a 1-foot wide strip of the slab. The analysis was completed for the two load types, HS-20 and OWT_2. The ultimate loads were then calculated and compared to the factored nominal capacities. The results, which are presented in Table 3.7 and Table 3.8, indicate that the culvert's top slab is safe under loads imposed by either the standard AASHTO HS-20 load or the future overweight transport vehicle.

Exterior Walls

The exterior walls were modeled as simply supported beams spanning between the top slab and the invert slab. The loads considered acting on the wall were the horizontal soil pressure as prescribed by AASHTO specifications [2] and the axial load coming from the loads on the slab.

Knowing that the existence of an axial load in the wall might either enhance or reduce its flexural capacity, the interaction diagram for a 1-foot wide strip of the wall was produced (Figure 3.8) and two load cases were checked: one with axial load and one without. The maximum axial live load was found with the aid of the influence line diagrams for the reaction at the end support of the slab.

The maximum factored bending moment (M_u) occurs at a depth of 7.05 feet and was calculated as 7.10 Kip-ft/ft. At the same depth, the factored axial load (P_u) was found to be 9.30 Kip/ft and 9.24 Kip/ft under HS-20 and OWT_2, respectively. To check for the case with axial load, the factored bending moment and axial load were increased by a factor of 1/0.70, resulting in 10.14 Kip-ft/ft and 13.29 Kip/ft for the bending moment and axial load, respectively. These values are within the interaction envelop. In the absence of any axial forces, the wall's factored nominal bending moment capacity would be 13.76 Kip-ft/ft which is greater than M_u. Therefore, the exterior walls are safe in flexure. The maximum shear force in the exterior wall occurs at the bottom support. The corresponding factored shear V_u was calculated as 2.7 Kip/ft. The factored nominal shear capacity of the wall is 6.7 Kip/ft. Therefore, it can be concluded that the wall is also safe in shear.

Interior Walls

Interior walls were modeled as axial members since the reinforcement details at the connections with the top slab and the invert slab do not allow for moment transfer.

The influence line diagram for the reaction at the interior support of the top slab was used to find the maximum possible axial load. It was found that the HS-20 truck load case controls and the corresponding factored axial load was calculated as 21.2 Kip/ft. This figure is much less than the nominal axial load capacity of the wall which is 144.2 Kip/ft. Hence, it can be concluded that the interior walls are safe under the two load configurations, HS-20 and OWT_2.

Invert Slab

Similar to the top slab, the invert slab was modeled as a continuous 3-span beam with pin supports at the exterior walls. The invert slab was analyzed by considering a 1-foot wide strip of the slab. The loads considered in the analysis were the dead load of the structure, the vertical load due to the soil fill on top of the structure, and the traffic live load.

To obtain the maximum live load effect, three truck loads were placed in each traffic direction. In the case of the HS-20 load case, the spacing between the truck rear axle and the trailer axle was kept at the minimum specified distance of 14 feet. This arrangement allowed the transfer of the full loads from both axles to the invert slab. The corresponding unfactored live load per 1-foot width was found as 0.04 Kip/ft. For the OWT_2 load case, the unfactored live load on the invert slab was also 0.04 Kip/ft.

The factored shear forces and bending moments were calculated and were found to be less than the corresponding factored nominal capacities of the invert slab as shown in Table 3.9 and Table 3.10. Therefore, the invert slab is safe under both load types.

3.3.3 Analysis of Bridge H-1020

The three-dimensional finite element model of H-1020 box frame is shown in Figure 3.9. The model is basically an array of beam elements that intersect at nodal points. Four cross sections, each having a thickness of 11 inches, were used to define the geometrical properties of the beam elements:

1- Interior Cross Beams: which are the beams that run along the short dimensions of the structure (15 ft) in the top slab, invert slab and walls, excluding the outermost beams at both ends of the structure. The geometrical properties of the interior cross beams were based on a 2-foot wide cross section.

2- Exterior Cross Beams: these beams are similar to the interior cross beams but are 1'-8" wide. The exterior cross beams are the outermost beams at both ends of the structure.

3- Longitudinal Middle Beams: which are the beams that run along the 38-foot width of the structure, at the middle of the top slab, invert slab and walls. The geometrical properties of these beams were based on a 7.5-foot strip width.

4- Longitudinal Corner Beams: which are located along the corners of the structure. The geometrical properties of these beams were derived from an L-shaped cross section. Each leg width was considered as 3.75 feet.

When the applied loads caused the invert slab to bend against the soil underneath, the model was pin supported at all nodal points in the invert slab. When the middle section of the invert slab had the tendency to bend upwards, however, the restraints at the interior nodes (middle of invert slab) were removed.

The vertical downward earth pressure was based on 2.5-foot deep fill as shown on the alignment profile. At this depth, the corresponding impact factor is 10%. Three lanes were loaded which required a load reduction factor of 0.90. For maximum live load bending moment, the wheel loads were placed above the middle of the top slab as shown in Figure 3.9. The concentrated wheel loads were then transformed into distributed loads according to the procedure described in Section 3.3.2. For a cross beam which happens to be directly under a wheel load, WL, the intensity of the distributed load was found to be 0.104 WL per foot applied for a distance of 4.375 feet. The balance of the wheel load was distributed to the two beams which are located on either side of the beam under the wheel load, at an intensity of 0.062 WL per foot applied for a distance of 4.375 feet.

The elements' bending moments and axial loads due to live load, dead load and earth pressure were obtained separately using the computer program IMAGES 3D. The results were then superimposed. The live load results were excluded whenever such results reduced the overall bending moment. The critical factored bending moments and the corresponding factored axial loads were calculated. These values were then increased by a factor of 1/0.70, as presented in Table 3.11, so that they could be compared to the corresponding nominal interaction envelops which are shown in Figure 3.10 through Figure 3.16. It was found that all sections were safe in flexure under the two load configurations, HS-20 and OWT_2.

The maximum shear force in the top slab was found to occur under the HS-20 load configuration. The wheel loads were placed in such a way that their corresponding distributed loads (described before) were totally supported by the top slab, starting at the slab edge and extending 4.375 feet over the slab. The factored shear force was calculated as 10.9 Kip/ft. The factored nominal shear capacity at the same section was calculated as 10.8 Kip/ft which is 0.9% less than the required factored capacity. Therefore, the top slab can be considered safe in shear.

Since joints are often the weakest links in a structural system, a major concern in this study was whether the knee joints at the corners of the box structure are capable of providing adequate resistance to the applied bending moments. The behavior of knee joints is thoroughly addressed in Reference 7. Depending on the sense of the applied moment, knee joints are divided into two types: joints subjected to "closing" moments and those subjected to "opening" moments (Figure 3.17). The behavior and moment resisting capacity in each joint type is greatly dependent on the reinforcement detail of the joint. Joints subjected to "closing" moments can be easily detailed to develop the moment capacity of adjoining members as is presented in Reference 7. A typical detail of a knee joint under "closing" moment is shown in Figure 3.18. The knee joints in the box structure of bridge H-1020 are subjected to "closing" moments at all times. The reinforcement details of those joints, as shown on the bridge construction drawings, were checked and were found to closely match the details recommended for the development of sufficient strength. Therefore, the joints were considered adequate.

3.4 Concluding Remarks

In this chapter, the geometric and structural adequacy of all three bridges, G-965, B-976 and H-1020, were reviewed. Two appraisal methods were used. The first method was to calculate updated sufficiency rating figures. The other method incorporated structural analyses of the bridges to check their structural safety under the loads imposed by the standard AASHTO HS-20 load and the future overweight transportation vehicles.

Based on visual inspection and the updated average daily traffic (ADT), it is expected that the sufficiency ratings of G-965, B-976 and H-1020 be slightly downgraded from 96.6, 80.2, and 92.6 to 96.5, 79.6 and 92.5, respectively. These revised figures indicate that the bridges are still in good condition and no major repair work is needed. The estimated quantities and costs of the suggested regular maintenance works for the bridges are presented in chapter 4 of this report.

The structural analyses revealed that all three bridge structures, G-965, B-976 and H-1020, would be safe under both load configurations, HS-20 and OWT_2. Because of time constraints, however, the analysis of G-976 was limited to the girders .

Chapter 4

Bridge Repair and Retrofit

4.1 Introductory Remarks

One of the major reasons for studying the adequacy of highway bridges to carry the additional traffic, generated by the spent fuel repository, is to determine the economic impact on existing bridges. The question to be addressed is what, if any, measures would need to be taken to upgrade the structural capacity and traffic safety features of highway bridges on the designated routes for the spent fuel traffic. The purpose of this chapter is to demonstrate the various methods and the associated costs needed to repair the reported deficiencies. Based on the structural analyses of the three bridge systems, it was found that no structural capacity upgrading is needed.

The unit prices used in this study for finding total cost estimates are primarily based on the data presented in Reference 8 adjusted for inflation at an annual rate of 5 percent. For some items, the unit costs were estimated based on educated guess and previous experience. The observed cracks which formed a pattern at the surveyed portions of the deck were assumed to exist throughout the deck. Thus, the surveyed quantities of those cracks include additional estimated quantities to allow for the unsurveyed locations.

4.2 Retrofitting of Bridge G-965

The necessary repair works for bridge G-965 can be placed into two groups:

(1) that related to traffic safety elements and (2) that related to the superstructure and substructure.

Traffic Safety Elements

The inspection of the bridge by the research team revealed only minor repair work is necessary in traffic safety elements. The deficiencies related to traffic safety were presented in Section 2.5.1 of this report. The unit costs of repairing those deficiencies are summarized in items 4, 5 and 6 of Table 4.1.

Super- and Sub-Structure Elements

The repair works needed for the super- and sub-structure elements involve the repair of cracks in the concrete and eroded soil at the bents foundations.

The cracks can be categorized into three main groups: (1) cracks that are equal to or greater than 0.016 inch in width, (2) cracks that are less than 0.016 inch in width and (3) pattern cracks and spalled concrete. The crack repair technique depends on the crack width and whether or not the crack is stable. From the field inspection (Section 2.5), it appears that the cracks are stable and the moisture penetration has not been extensive. Nonetheless, it is advisable that the reinforcing bars in the vicinity of the cracks be exposed and cleaned during the repair procedure. Moreover, none of the observed cracks seem to exceed 0.016 inch in width.

One technique can be followed in repairing those cracks that are less than 0.016 inch in width as well as pattern cracks and spalled concrete areas. The following steps are necessary in this method:

(a) Chip away the loose concrete adjacent to the crack to expose the reinforcing steel. The width of the chipped strip need not be wider than 4 inches.

(b) Clean the reinforcing bars and remove any rust layer, then cover the bars with epoxy coating.

(c) Apply a layer of bonding epoxy mortar to the exposed concrete surface.

(d) Place metal lath to cover the chipped away concrete area.

(e) Patch the exposed area with shrinkage compensating concrete and finish the surface.

The repair of the eroded areas involves the placement of rip-rap to fill the holes and prevent more erosion.

The quantities and costs of the different repair items in bridge G-965 are reflected in Table 4.1. Based on the bill of quantities presented in Table 4.1, the cost of the repair works needed for bridge G-965 would be about $ 96,000.

4.3 Retrofitting of Bridge B-976

The repair works needed for structure B-976 involve the repair of cracks, spalled concrete and eroded soil.

The repair of thin cracks (width < 0.016 inch) and spalled concrete can be performed according to the procedure described for bridge G-965 in Section 4.2.

A more extensive repair method is required for the cracks that are 0.016 inch or more in width. This method, which is adopted from Reference 9, involves the following steps:

(a) Remove the concrete in the vicinity of the crack to a width of 10 inches. Expose the reinforcing bars.

(b) Drill holes to be used in the placement of the additional U-shaped bars shown in Figure 4.1.

(c) Clean the existing steel bars the cover them with epoxy coating.

(d) Place the U-shaped bars as shown in Figure 4.1.

(e) Apply a layer of bonding epoxy mortar to the exposed concrete surface.

(f) Place metal lath and patch the crack with shrinkage compensating concrete, then finish the surface.

The eroded soil can be repaired by filling the holes with rip-rap to prevent more erosion from taking place.

The bill of quantities of the repair items and the associated costs are shown in Table 4.2. The overall cost of retrofitting B-976 is estimated to be about $ 48,400.

4.4 Retrofitting of Bridge H-1020

The retrofit works needed for H-1020 are limited to the repair of thin cracks and a few spalled concrete spots. The repair of these deficiencies can be done using the method described for bridge G-965 in Section 4.2.

The overall cost of retrofitting H-1020 is estimated at about $ 19,000. The quantities and unit prices are shown in Table 4.3.

4.5 Concluding Remarks

In this chapter, the necessary works and the associated costs needed to upgrade the conditions of bridges G-965, B-976 and H-1020 were presented. Note that the structural adequacy of the substructures were assumed to be sufficient.

Since all three bridge systems were found to possess adequate structural capacities, there would be no need for structural improvement. The only upgrading cost would be that incurred by the correction of the deficiencies which are the result of normal decay. Such upgrading works may not be considered urgent, but it is believed that the recommended repair works, if done, would prevent an accelerated deterioration of the bridges when they are subjected to higher traffic volume and axle loads. The cost of upgrading bridges G-965, B-976 and H-1020 were estimated at $ 96,000, $ 48,400 and $ 19,000, respectively.

Chapter 5

Summary and Conclusions

5.1 Summary

The transportation of spent nuclear fuel across Nevada to the proposed repository site in the southern part of the state could significantly affect the infrastructure system in Nevada. It is, therefore, essential that the potential impact be carefully studied to insure that a safe and reliable network of paved roads and railroads will be available.

One of the major classes of structures affected by the nuclear waste transportation is highway bridges and culverts. These structures are generally complex, and the evaluation of their safety and performance involves a large number of factors.

This report presents the results of the research conducted during 1993. The study included the evaluation of specific bridges on a potential nuclear waste transportation route. Where bridge upgrading is needed, retrofitting procedures were suggested and the associated cost estimates are presented.

The primary objective of this study was to select some appropriate bridges and conduct a rather comprehensive study of their condition, structural capacity and repair cost, if any.

Before the above objectives were addressed, a data base for the existing highway bridges had to be established for a selected route which is likely to be used to transport spent fuel. In this case, the selected route extends from the Utah-Nevada border southward to the proposed repository site. This route includes Alternate US 93 from

Wendover at the Utah state line to Lages, US 93 from Lages to Ely, US 6 from Ely to Tonopah, and then US 95 from Tonopah to the proposed repository site at Amargosa Valley. An alternate path, considered as a possible segment of the route in this study, includes Nevada State Route (SR) 318 from the junction of US 6 to US 93 south of Hiko, US 93 to I-15, I-15 to the junction of Craig Road north of Las Vegas, Craig Road between I-15 and US 95, and finally, US 95 to Amargosa Valley. A microcomputer data base for all the bridges and culverts on this route is available at the Nevada Department of Transportation (NDOT).

Due to the shortage of the available time, this study was limited to three bridge systems, two of which are culvert-type structures. The selected structures were G-965, B-976 and H-1020.

In Chapter 2, the procedure for selecting the bridges included in this study was presented. All three bridges were briefly described, then the results of the site inspection were reported. The site inspection revealed some deterioration.

In chapter 3, the geometric and structural adequacy of the bridges were reviewed. Two appraisal methods were employed. The first method was to calculate the updated sufficiency rating figures. The other method incorporated structural analyses to check the structural safety of the bridges under the loads imposed by the standard HS-20 load and the proposed future overweight transportation vehicles (OWT_2).

Chapter 4 presented the different retrofitting methods needed to upgrade the conditions of the bridges. The estimated retrofitting costs were also presented in Chapter 4.

5.2 Conclusions

The primary conclusions in this study are as follows:

- The sufficiency ratings given by the Nevada Department of Transportation (NDOT) for bridges G-965, B-976 and H-1020 are 96.5, 80.2 and 92.6, respectively. These figures are based on the inspection conducted in 1991. Based on this study, however, it is expected that the current conditions of G-965, B-976 and H-1020 would slightly lower the sufficiency ratings to 96.5, 79.6 and 92.5, respectively. These revised figures indicate that the bridges are still in good condition and no major repairs are needed.
- The structural analyses, conducted in this study, revealed that all three bridge systems would be safe under the loads imposed by the standard HS-20 load as well as the "future overweight cask" truck/trailer (OWT_2). Hence, no structural capacity upgrading is needed for any of the three bridge systems.
- The estimated costs of repairing the reported deficiencies in bridges G-965, B-976 and H-1020 are $ 96,000, $ 48,400 and $ 19,000, respectively.

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Table 2.4 Bridges on Route Segment B

Route	Bridge Type	Number
US 6	Concrete Culvert	4
US 95	Concrete Culvert	10

Table 2.5 Bridges on Route Segment C

Route	Bridge Type	Number
US 93A	Concrete Culvert Steel Culvert Steel Girder Bridge	2 1 1
US 6	Concrete Culvert	1

Table 3.1 Cask/Vehicle Types

	Cask Type	Payload	Loaded Cask Weight (pounds)	GVW (pounds)	Axle Configuration and Weights
LWT_1	Current Legal Weight (NLI-1/2)	1 PWR/ 2 BWR	45,000	74,500	(3-2) 10-32.5-32
LWT_2	Future Legal Weight (25 ton)	2 PWR/ 5 BWR	50,500	79,000	(3-2) 11-34-34
OWT_1	Current Overweight (TN-8/9)	3 PWR/ 7 BWR	78,000	111,000	(4-3) 12-47-52
OWT_2	Future Overweight (40 ton)	4 PWR/ 10 BWR	80,000	116,000	(4-3) 11-51-54

Source: Battelle Analysis

Table 3.2 Shear Forces and Bending Moments Considered in the Original Design of the Girders of Bridge G-965

Position	Shear Force (Kip)	Bending Moment (Kip-ft)
A	80	
0.1 AB	63	318
0.2 AB	47	494
0.3 AB	30	572
0.4 AB	30	575, -48
0.5 AB	46	495, -168
0.6 AB	62	338, -312
0.7 AB	79	110, -528
0.8 AB	95	-756
0.9 AB	111	-1043
BA	128	-1445
BC	130	-1445
0.1 BC	110	-913
0.2 BC	89	-482
0.3 BC	68	366, -182
0.4 BC	48	572
0.5 BC	27	642

Table 3.3 Bending Moment Values for the First Interior Girder of Bridge G-965

Pt. on Span	M_{DL} (Kip-ft)	HS-20 Moment (Kip-ft)		OWT_2 Moment (Kip-ft)		ϕM_n (Kip-ft)
		$M_{(LL+I)}$	M_u	$M_{(LL+I)}$	M_u	
0.1 AB	101.9	125.4	491.1	119.4	473.9	2150
0.3 AB	194.3	249.4	965.9	252.5	974.7	1653
0.5 AB	117.7	229.7 -136.8	810.0 -238.2	231.1 -102.0	814.0 -138.7	1537 -634
0.7 AB	-142.2	101.7 -210.3	106.0 -786.3	124.1 -146.7	170.1 -604.4	2086 -1903
0.9 AB	-599.7	-452.6	-2074.0	-388.6	-1891.0	-3312
B	-827.7	-716.2	-3124.3	-642.4	-2913.3	-4217
0.1 BC	-532.1	-486.4	-2082.8	-479.5	-2063.1	-3312
0.3 BC	5.0	158.1 -169.2	458.7 -477.4	146.5 -118.3	425.5 -331.8	2142 -824
0.5 BC	182.1	246.8	942.6	236.1	912.0	1627

Table 3.4 Shear Force Values for the First Interior Girder of Bridge G-965

Pt. on Span	V_{DL} (Kip)	HS-20 Shear (Kip)		OWT_2 Shear (Kip)		ϕV_n (Kip)
		$V_{(LL+I)}$	V_u	$V_{(LL+I)}$	V_u	
@ d from A	17.93	33.29	118.52	31.19	112.51	139.3
0.1 AB	15.72	31.50	110.53	29.67	105.29	117.1
0.3 AB	0.93	18.90	55.26	18.12	53.03	88.3
0.5 AB	14.24	24.30	88.01	26.70	94.86	126.0
0.7 AB	28.28	35.78	139.09	37.47	143.93	153.9
0.9 AB	45.01	44.30	185.21	44.05	184.50	246.1
B	49.6	46.18	196.55	46.70	198.04	302.3
0.1 BC	42.33	42.79	177.41	43.62	179.78	250.1
0.3 BC	20.43	31.51	116.68	33.32	121.85	140.3
0.5 BC	4.34	18.70	59.12	19.24	60.67	93.6

Table 3.5 LRFD Bending Moment Values for the First Interior Girder of Bridge G-965

Pt. on Span	HS-20 Moment (Kip-ft)		OWT_2 Moment (Kip-ft)		ϕM_n (Kip-ft)
	$M_{(LL+I)}$	Factored M	$M_{(LL+I)}$	Factored M	
0.3 AB	340.3	773.9	289.4	652.6	1653
0.7 AB	-273.3	609.9	-174.6	-433.5	-1903
B	-889.3	2449.5	805.0	-2199.0	-4217
0.5 BC	357.9	795.8	283.0	639.1	1627

Table 3.6 LRFD Shear Force Values for the First Interior Girder of Bridge G-965

Pt. on Span	HS-20 Shear (Kip)		OWT_2 Shear (Kip)		ϕV_n (Kip)
	$V_{(LL+I)}$	Factored V	$V_{(LL+I)}$	Factored V	
@ d from A	53.3	118.6	56.2	100.2	141.7
0.1 AB	47.3	105.7	49.9	88.8	118.4
0.3 AB	28.3	51.8	30.5	42.8	88.2
0.5 AB	42.4	94.3	46.6	82.27	94.7
0.7 AB	60.4	145.0	63.7	124.0	155.8
0.9 AB	72.6	188.7	76.3	163.2	218.5
B	77.6	203.7	80.8	175.4	278.0
0.1 BC	74.1	187.9	77.8	161.8	222.8
0.3 BC	55.0	124.9	58.3	106.4	140.1
0.5 BC	31.9	62.6	35.2	53.8	92.3

Table 3.7 Maximum Bending Moment Values per Foot Width of the Top Slab of B-976

Pt. on Span	M_{DL} (Kip-ft)	M_{Earth} (Kip-ft)	HS-20 Moment (Kip-ft)		OWT_2 Moment (Kip-ft)		ϕM_n (Kip-ft)
			$M_{(LL+I)}$	M_u	$M_{(LL+I)}$	M_u	
0.4 AB	1.93	9.25	3.52	22.18	5.56	26.61	35.10
B	-2.41	-11.56	-3.51	-25.78	-3.82	-26.45	-40.45
0.5 BC	0.60	2.89	2.59	10.16	4.37	14.03	35.10

Table 3.8 Maximum Shear Force Values per foot width of the Top Slab of B-976

Pt. on Span	V_{DL} (Kip)	V_{Earth} (Kip)	HS-20 Shear (Kip)		OWT_2 Shear (Kip)		ϕV_n (Kip)
			$V_{(LL+I)}$	V_u	$V_{(LL+I)}$	V_u	
@d from A	0.65	3.10	1.09	7.24	0.98	7.00	11.24
@ d on BA	1.03	4.94	1.53	11.08	1.38	10.75	11.24
@ d on BC	0.84	4.03	1.43	9.44	1.24	9.03	11.24

Table 3.9 Maximum Bending Moment Values per foot width of the Invert Slab of B-976

Pt. on Span	M_{DL} (Kip-ft)	M_{Earth} (Kip-ft)	HS-20 Moment (Kip-ft)		OWT_2 Moment (Kip-ft)		ϕM_n (Kip-ft)
			$M_{(LL+I)}$	M_u	$M_{(LL+I)}$	M_u	
0.4 AB	5.27	9.25	0.51	19.98	0.51	19.98	36.98
B	-6.59	-11.56	-0.64	-24.98	-0.64	-24.98	-42.67
0.5 BC	1.65	2.89	0.16	6.25	0.16	6.25	36.98

Table 3.10 Maximum Shear Force Values per foot width of the Invert Slab of B-976

Pt. on Span	V_{DL} (Kip)	V_{Earth} (Kip)	HS-20 Shear (Kip)		OWT_2 Shear (Kip)		ϕV_n (Kip)
			$V_{(LL+I)}$	V_u	$V_{(LL+I)}$	V_u	
@d from A	1.76	3.10	0.17	6.69	0.17	6.69	11.80
@ d on BA	2.81	4.94	0.27	10.66	0.27	10.66	11.80
@ d on BC	2.29	4.03	0.22	8.69	0.22	8.69	11.80

Table 3.11 Critical Bending Moments[1] and Axial Loads[2] per Foot Width in Structure H-1020

Location	HS-20		OWT_2	
	$M_u/0.7$	$P_u/0.7$	$M_u/0.7$	$P_u/0.7$
Top Slab, Mid Span	28.4	5.9	31.6	6.4
Top Slab, @ Support	27.6	5.91	33.8	6.51
Wall, Top	28.3	11.6	34.4	15.5
Wall, Middle	5.9	13.2	4.4	17.1
with no Live Load	8.6	9.0	8.6	9.0
Wall, Bottom	8.7	12.7	7.1	19.0
with no Live Load	11.6	10.9	11.6	10.9
Invert Slab, Mid Span	6.5	7.1	6.5	7.1
Invert Slab, @ Support	11.6	7.1	11.6	7.1

1 Units are in Kip-ft/ft.
2 Units are in Kip/ft.

Table 4.1 Cost Estimate of Retrofitting Bridge G-965

Item Description	Unit	Qty	Unit Price	Ref	Mult	Total ($)
1. Deck						
1.1 Repair flexural cracks	L.ft	1450	32.00	8	1.41	65,424
1.2 Repair Pattern cracks	ft^2	108	39.00	8	1.41	5,939
2. Abutments						
2.1 Gracks	L.ft	57	32	8	1.41	2,572
2.2 Bearing devices	no.	3	1500	Est.	1.00	4,500
3. Slope Erosion at Bents	no.	6	500	Est.	1.00	3,000
4. Repair Minor Ruts in Asphalt	L.ft	1	5000	Est.	1.00	5,000
5. Fix Damaged Guardrail Posts	Item	1	1500	est.	1.00	1,500
6. Fix Cracks in Rail Barrier	item	1	6000	est.	1.00	6,000

Total = $ 93,935

Table 4.2 Cost Estimate of Retrofitting Bridge B-976

Item Description	Unit	Qty	Unit Price	Ref	Mult	Total ($)
1. Cracks < 0.016" in Width	L.ft.	1020	32	8	1.41	46,022
2. Cracks ≥ 0.016" in Width	L.ft	29	41	8	1.41	1,676
3. Spalled Concrete	Item	1	200	Est.	1.00	200
4. Slope Erosion	Item	1	500	Est.	1.00	500

Total = $ 48,398

Table 4.3 Cost Estimate of Retrofitting Bridge H-1020

Item Description	Unit	Qty	Unit Price	Ref	Mult	Total ($)
1. Thin Cracks	L.ft.	414	32	8	1.41	18,680
2. Spalled Concrete	Item	1	200	Est.	1.00	200

Total = $ 18,880

Figures

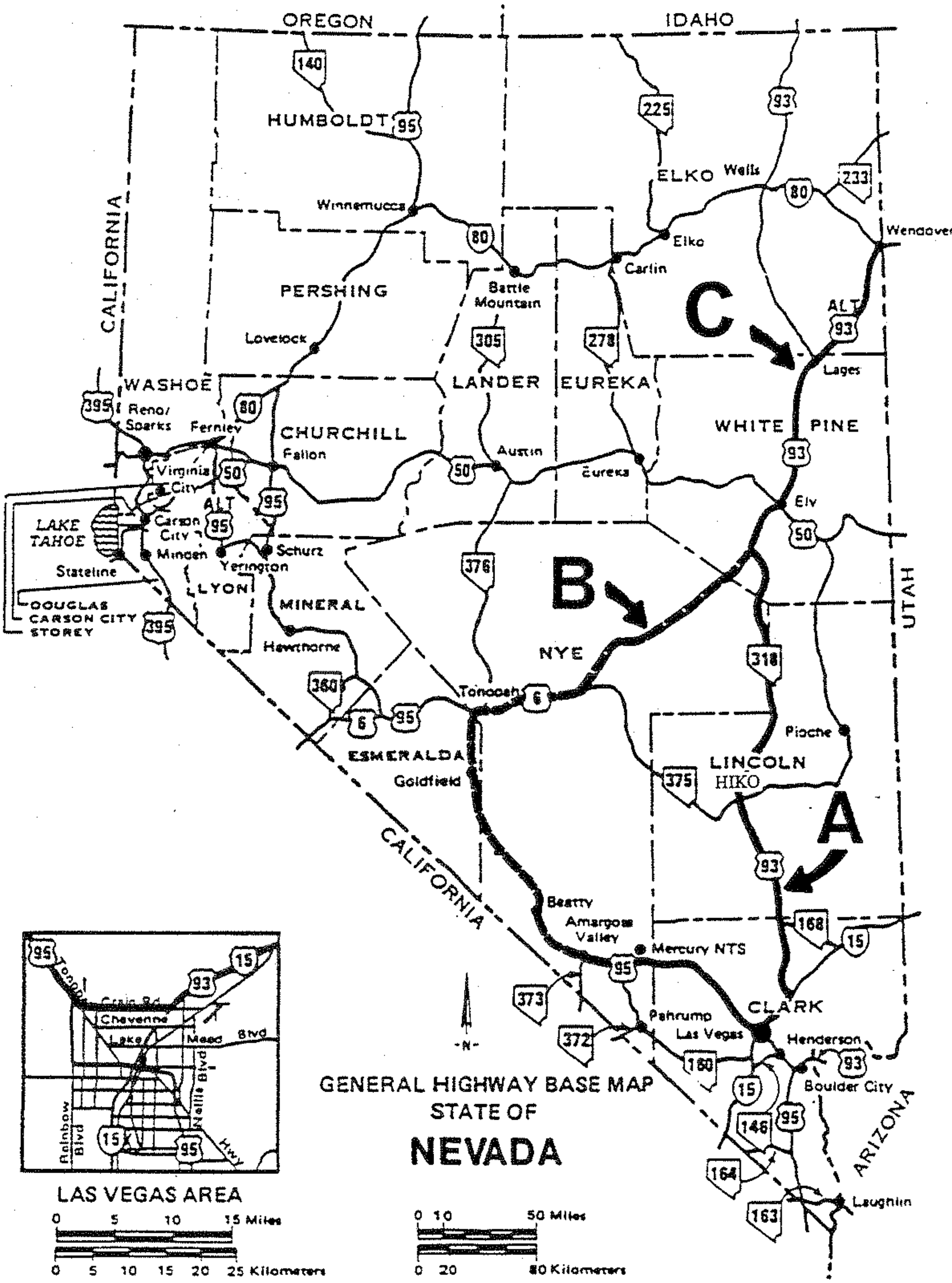


Figure 2.1 Map of the Route between Wendover and Amargosa Valley

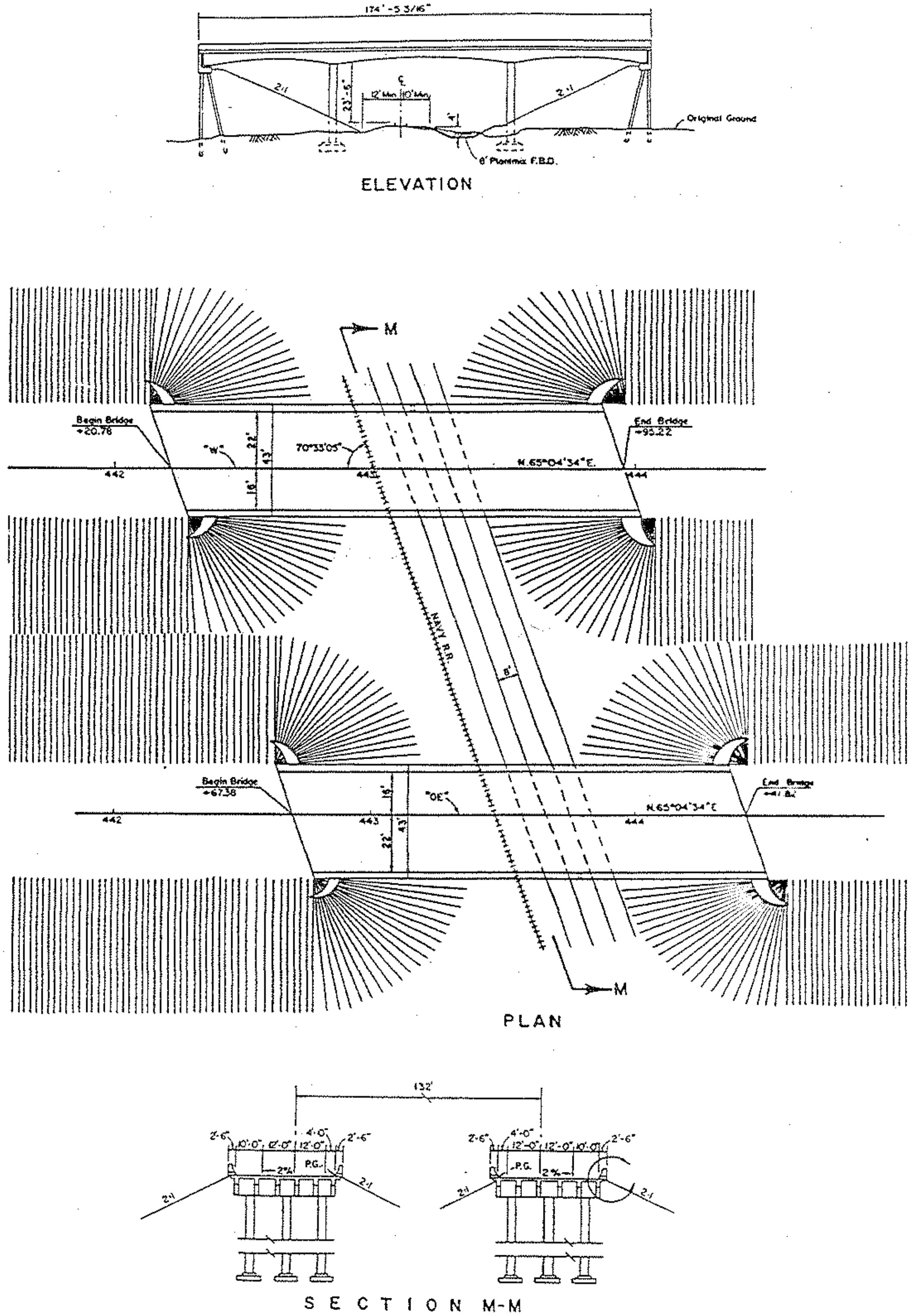


Figure 2.2 Plan and Elevation of Bridge G-965

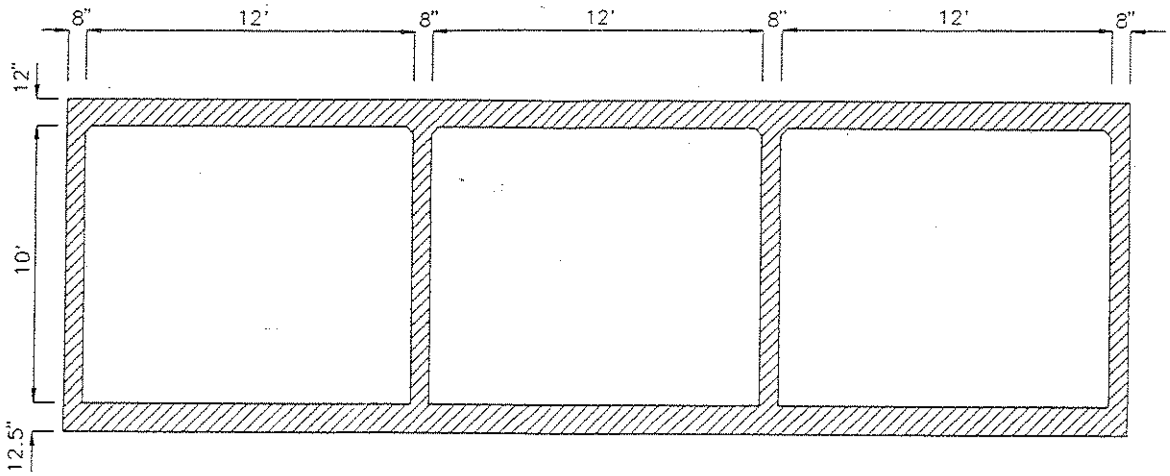


Figure 2.3 Cross Section of Bridge B-976

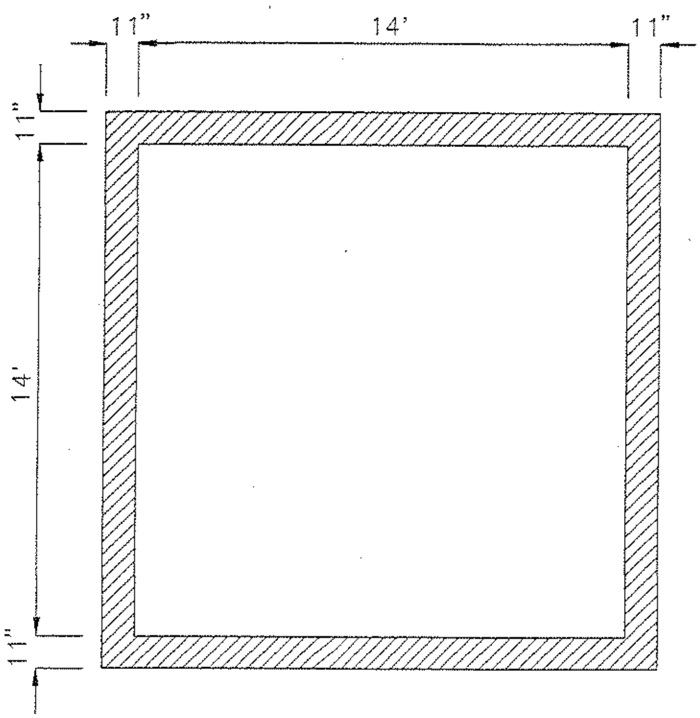


Figure 2.4 Cross Section of Bridge H-1020

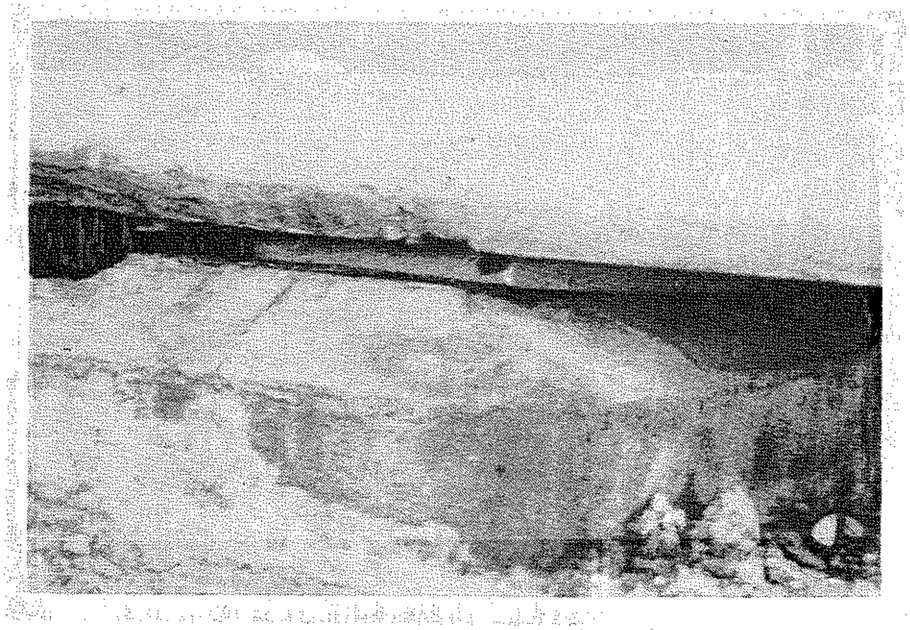


Figure 2.5 Off-Center Slider Bearing in Bridge G-965 N

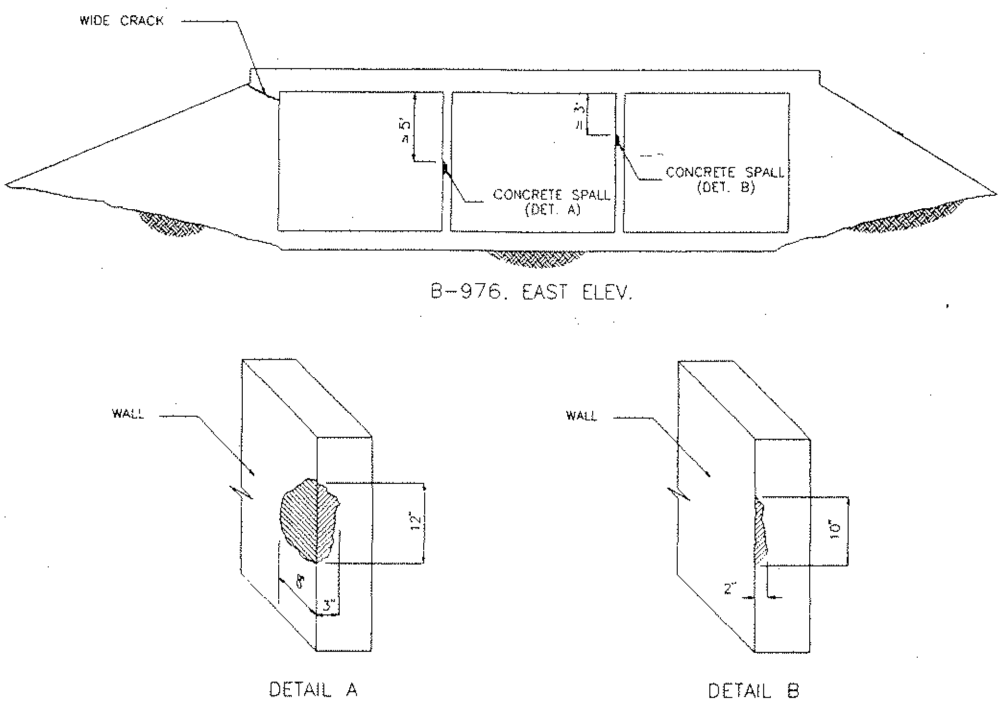


Figure 2.6 Spalled Concrete at the East End of B-976

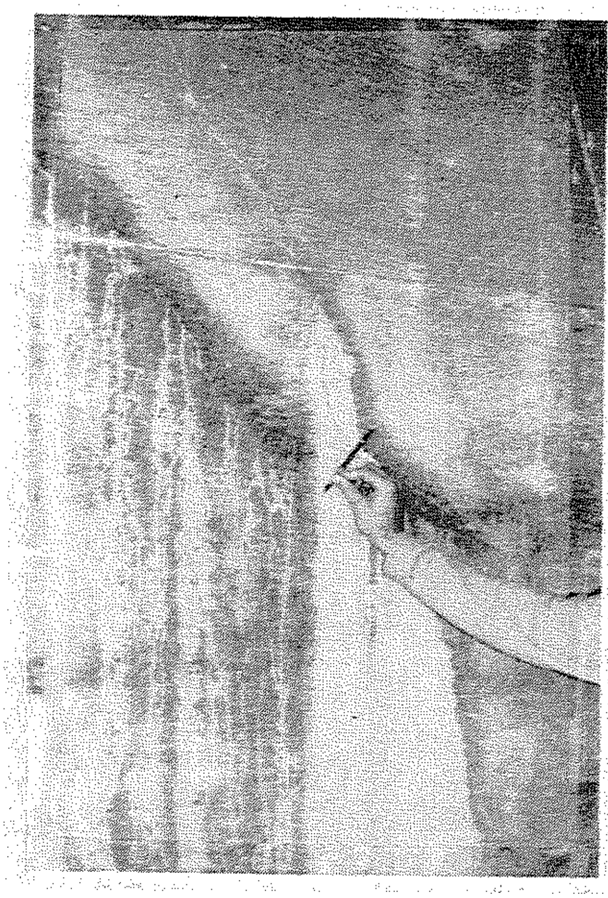


Figure 2.7 Vertical Crack in the Exterior Wall of B-976

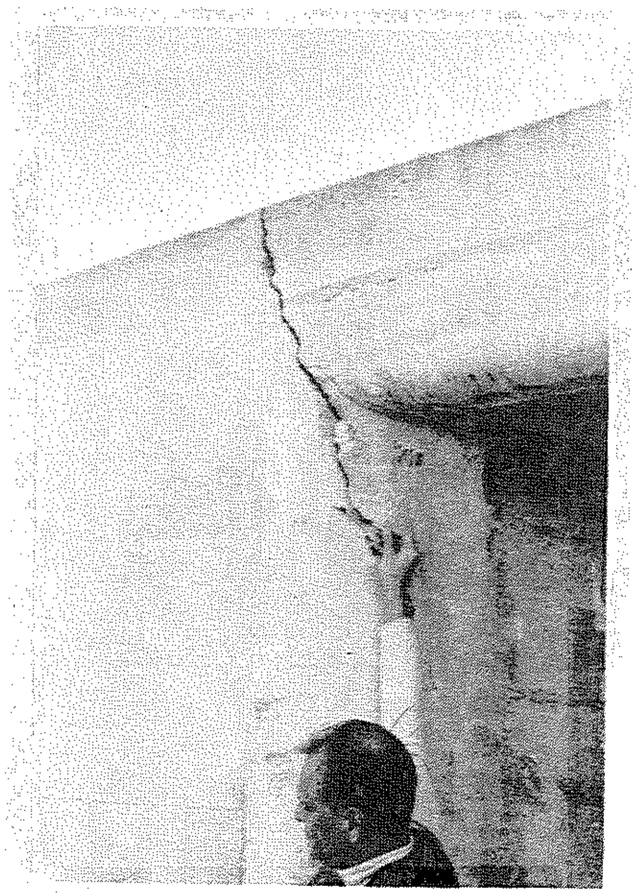


Figure 2.8 Wide Crack at the South-East Corner of B-976

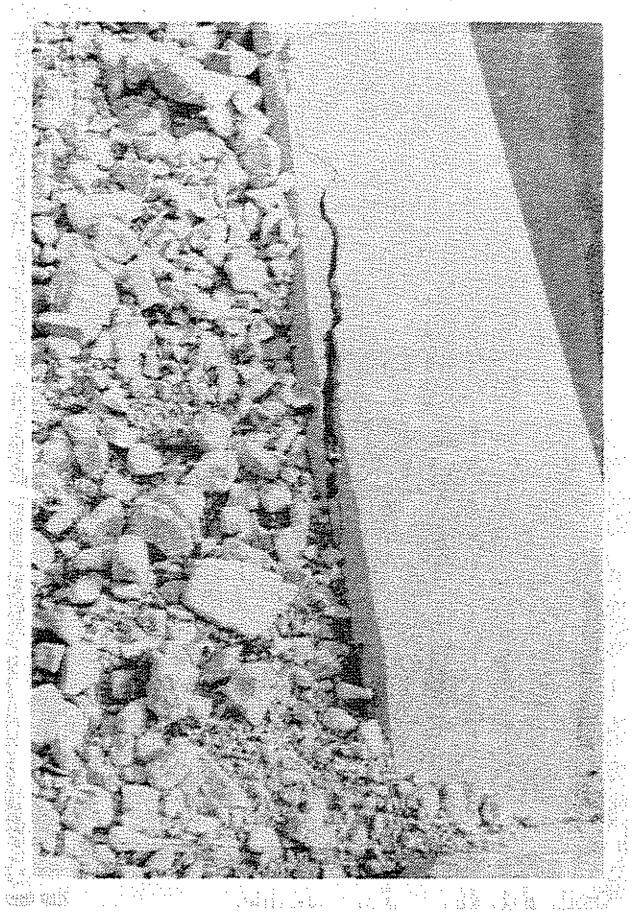


Figure 2.9 Concrete Spalling In the Wing Wall of H-1020

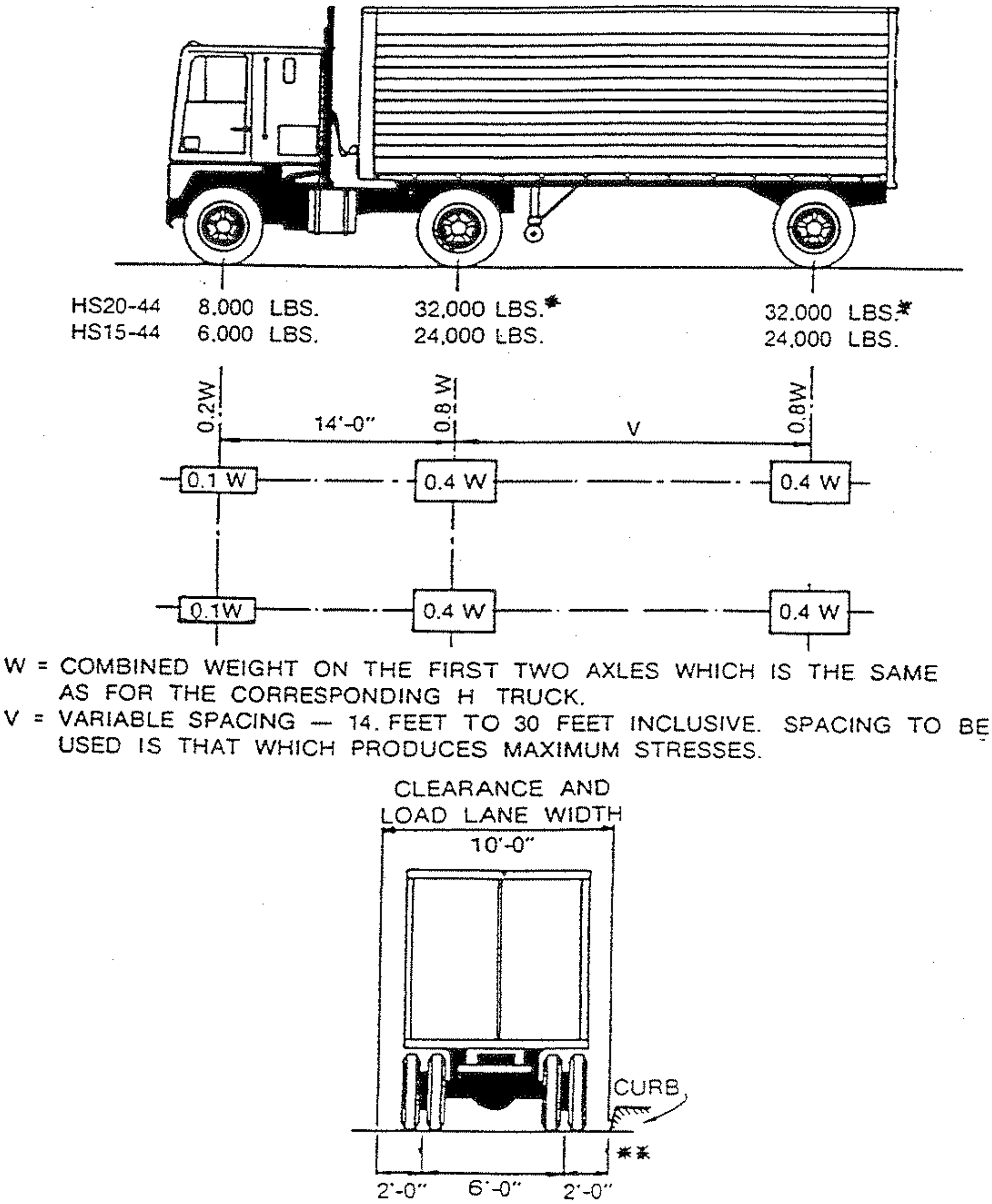


Figure 3.1 HS-20 Truck Configuration and Loads

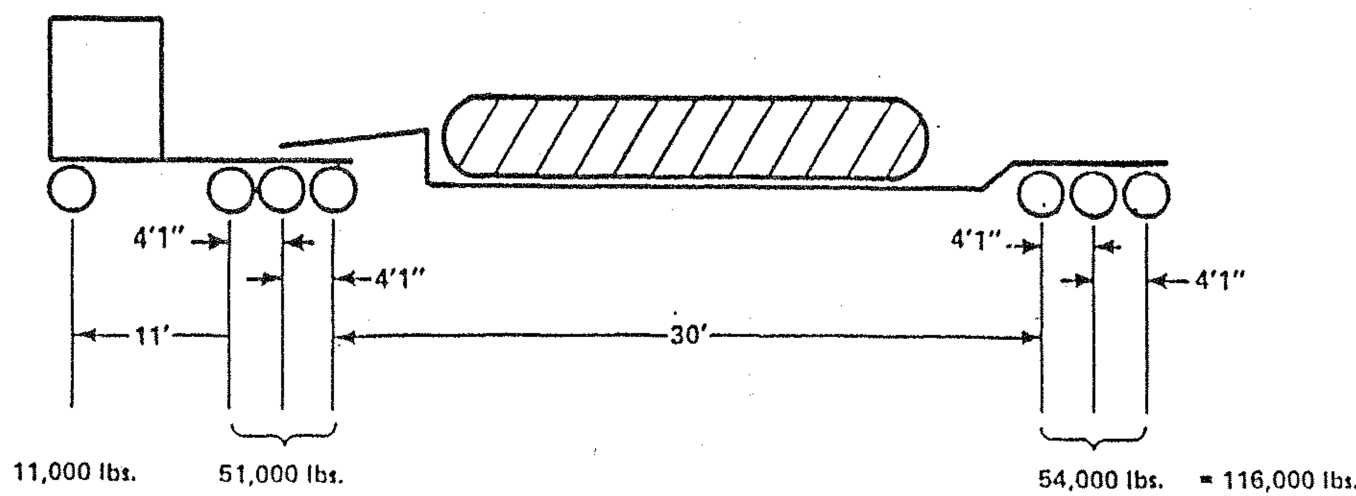


Figure 3.2 Represeniative Trailer and Axle Configurationfor OWT_2 Truck

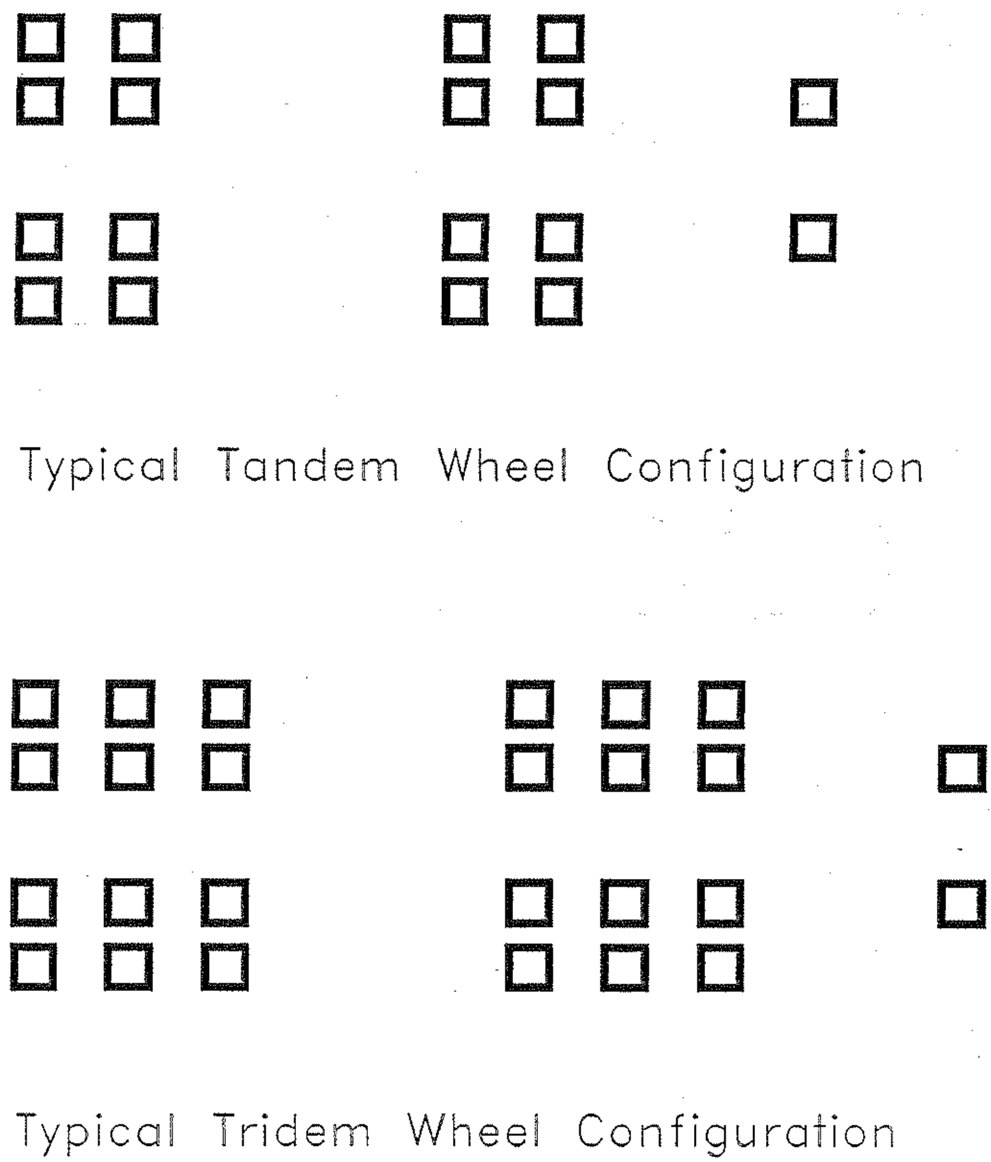


Figure 3.3 Typical Tridem and Tandem Wheel Configurations

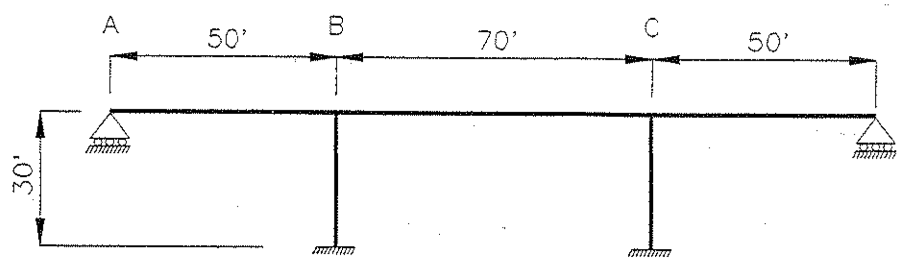


Figure 3.4 2-D Model of Bridge G-965

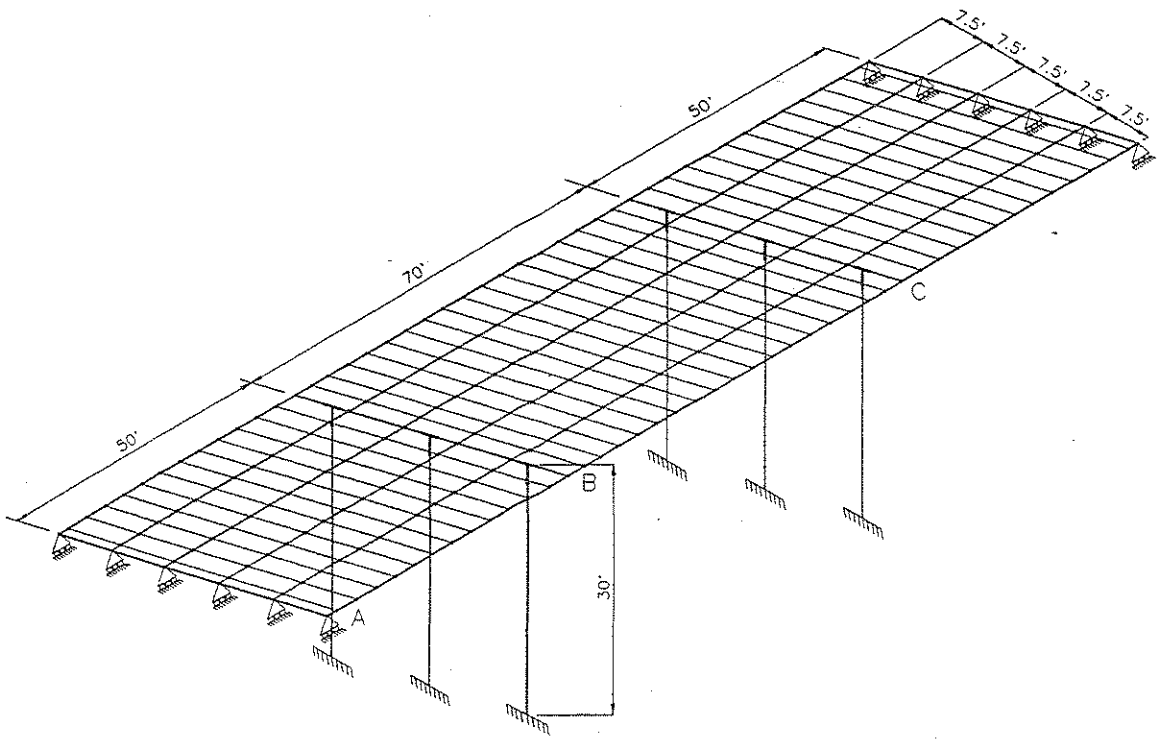


Figure 3.5 The Finite Element Model of Bridge G-965

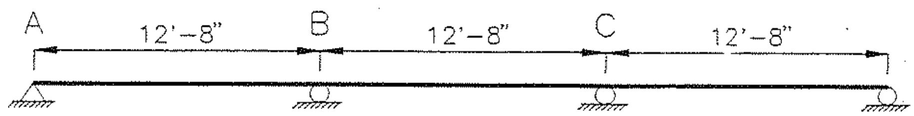
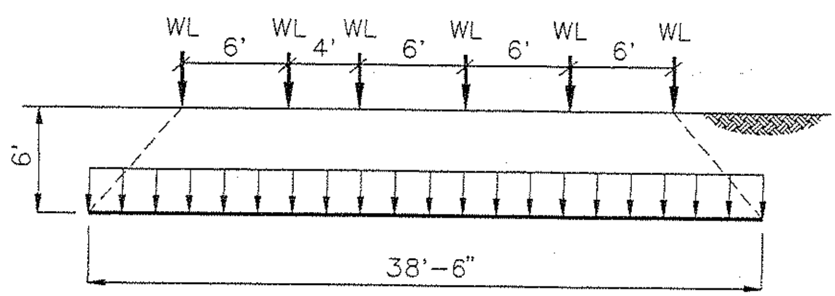


Figure 3.6 Continuous Beam Model of the Top Slab of B-976



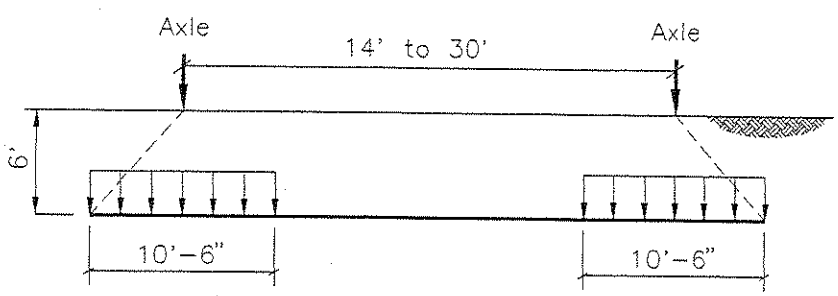


Figure 3.7 Wheel Load Distibution through a 6-Foot Deep Fill

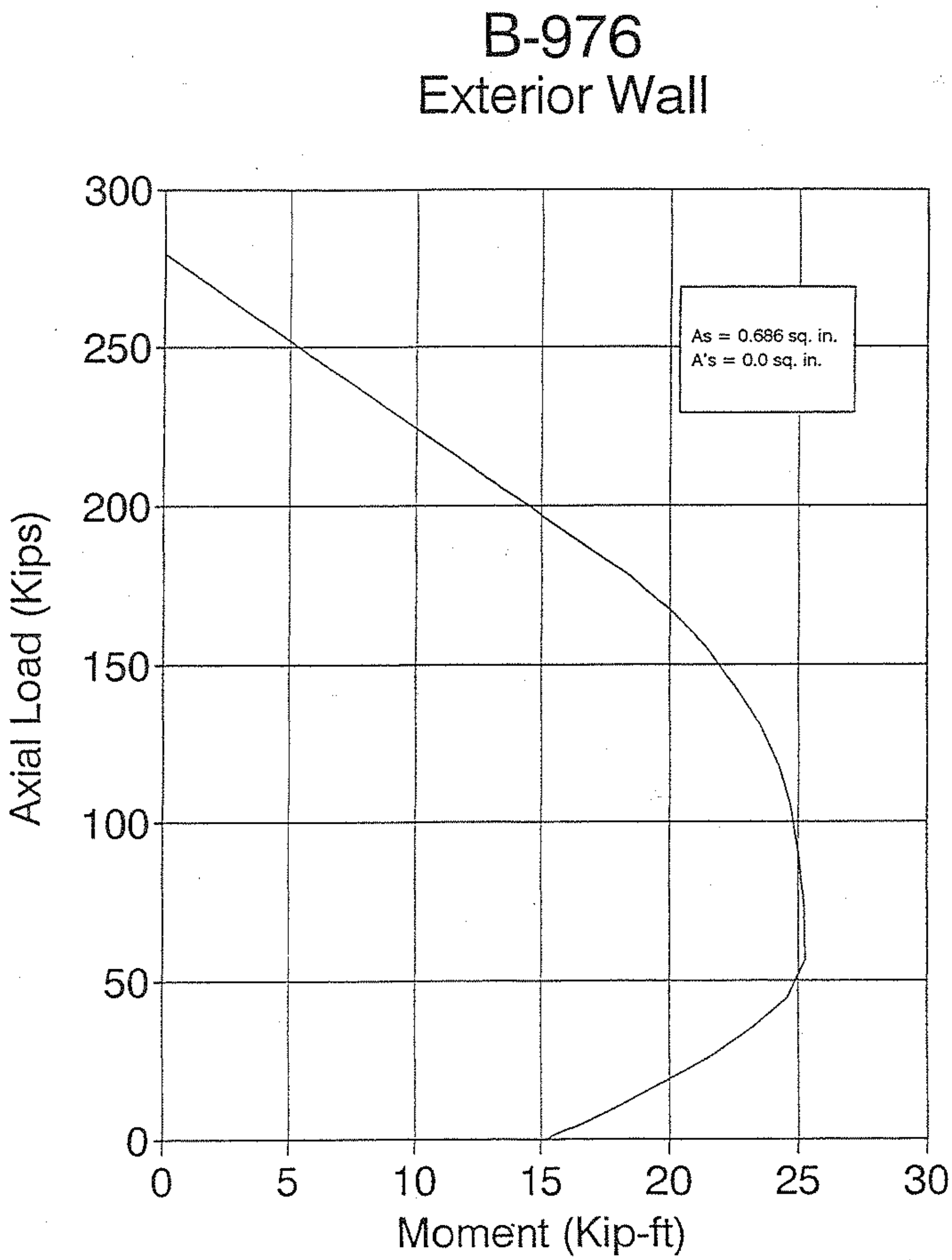


Figure 3.8 Nominal Moment Interaction Diagram of the Exterior Walls of Bridge B-976

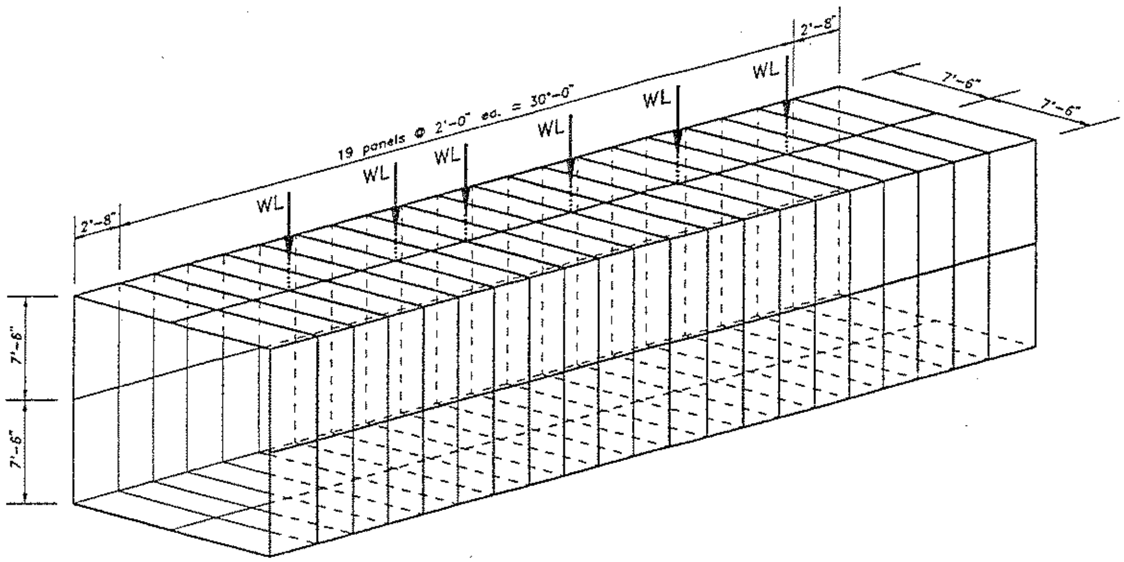


Figure 3.9 The Finite Element Model of Bridge H-1020

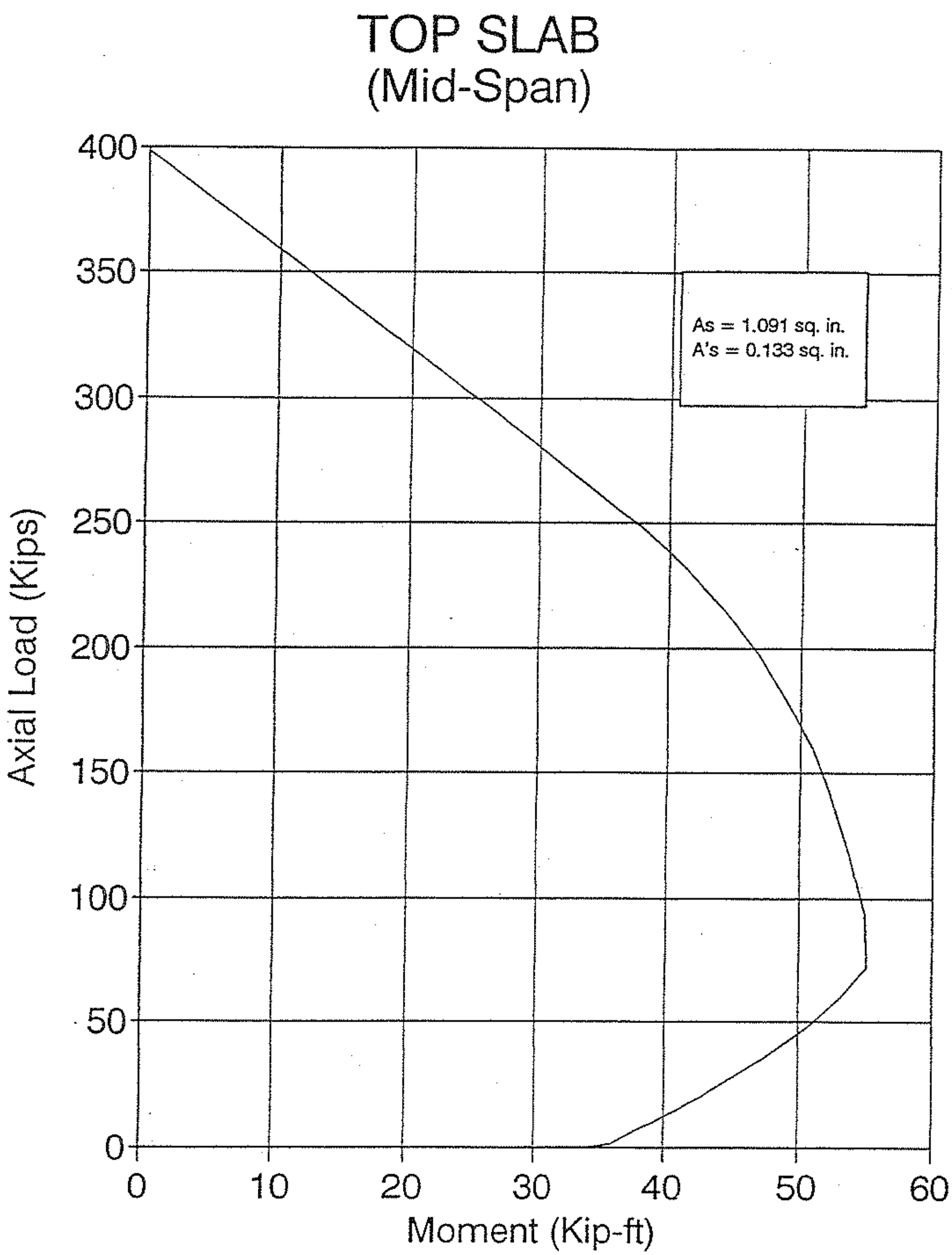


Figure 3.10 Nominal Moment Interaction Diagram at the Mid Span of the Top Slab of H-1020

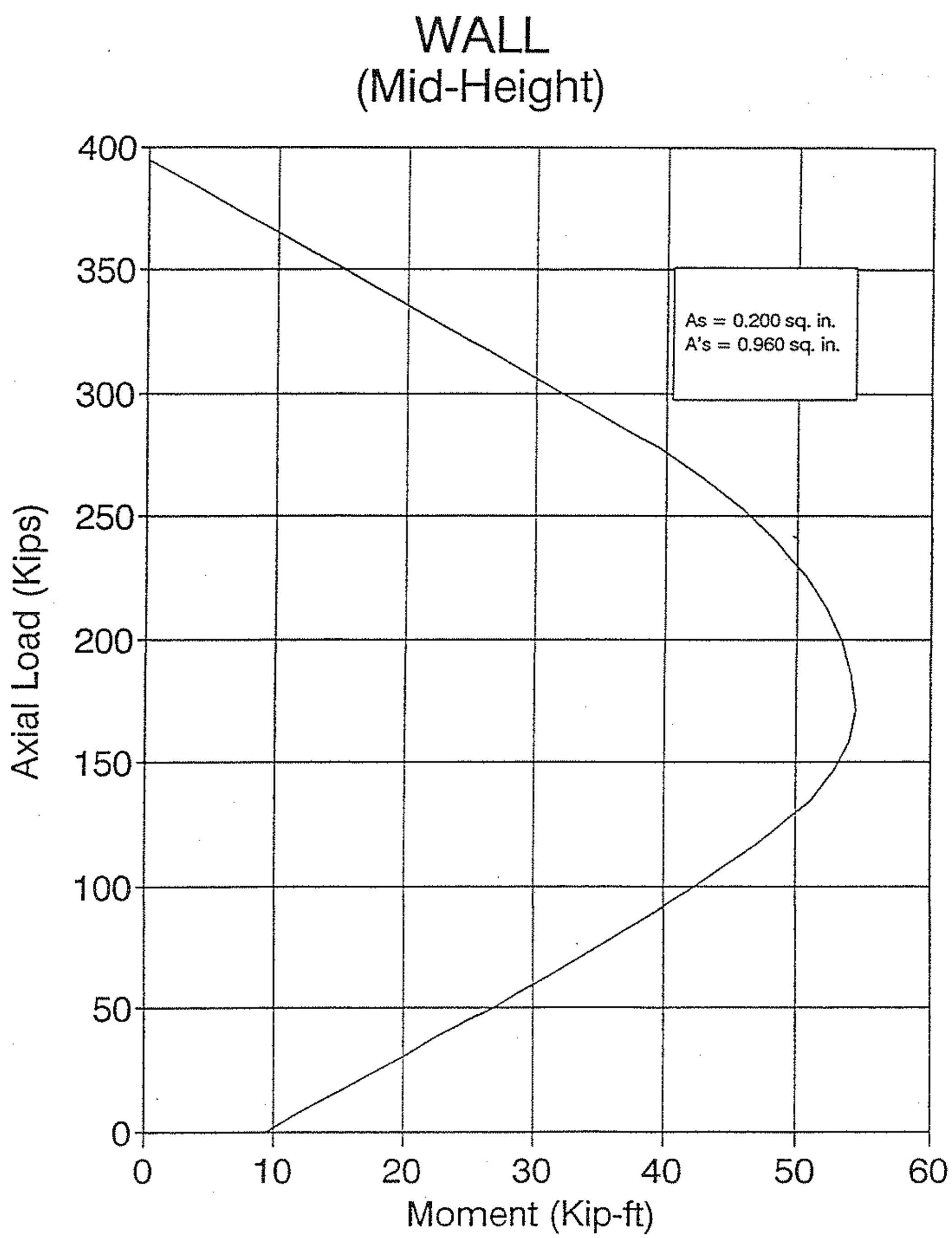


Figure 3.13 Nominal Moment Interaction Diagram at the Mid Height of the Wall of H-1020

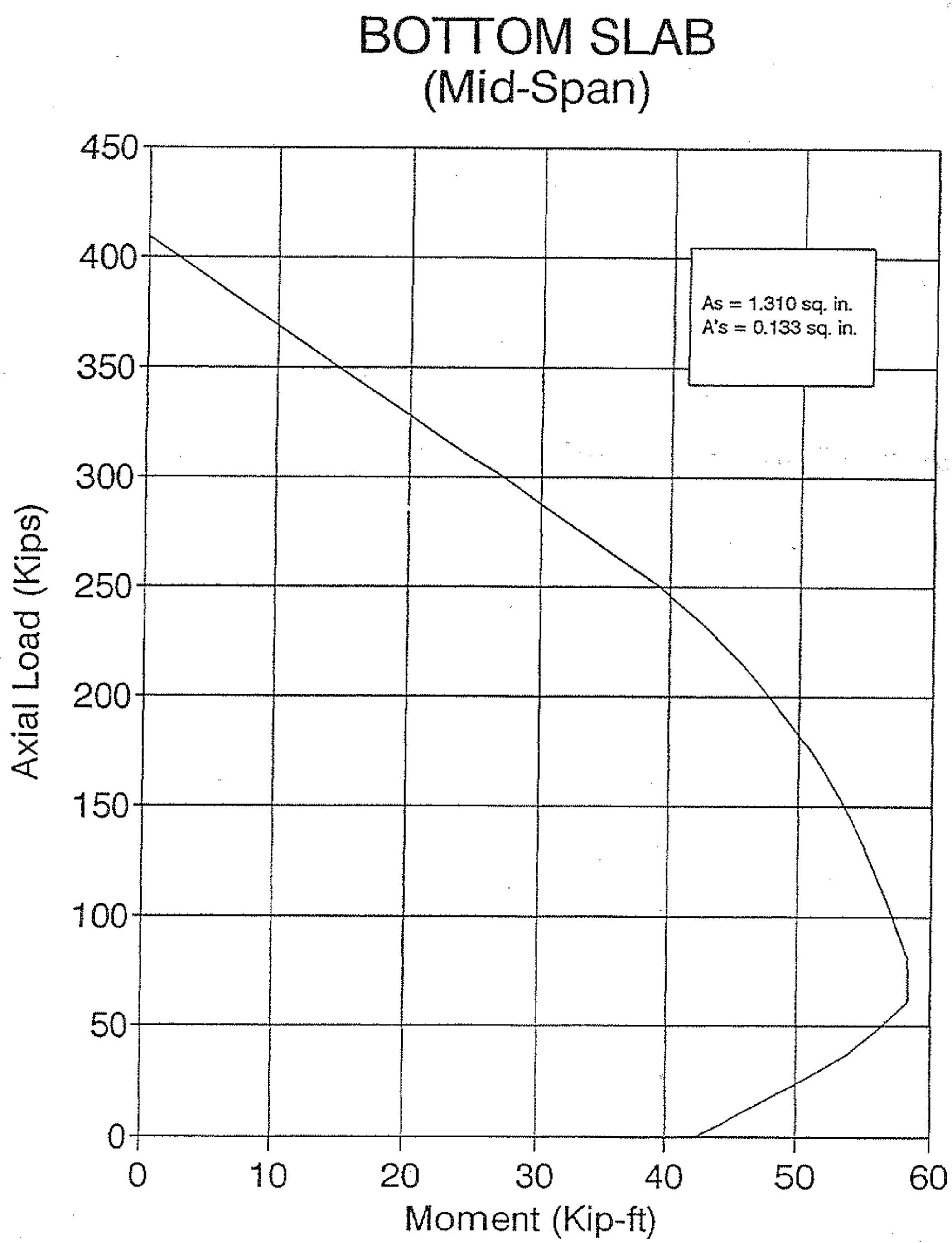


Figure 3.15 Nominal Moment Interaction Diagram at the Mid Span of the Invert Slab of H-1020

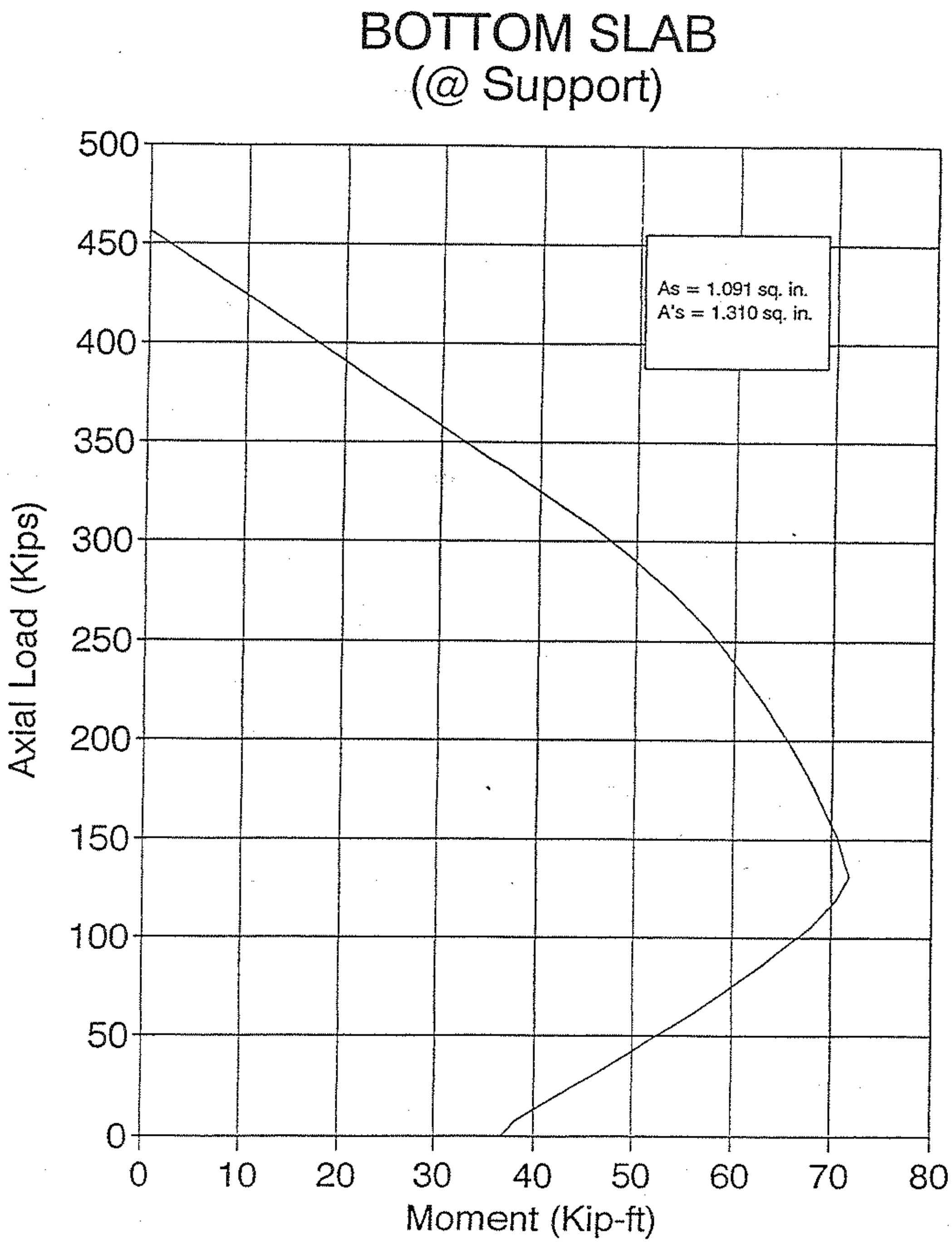


Figure 3.16 Nominal Moment Interaction Diagram at the Support of the Invert Slab of H-1020

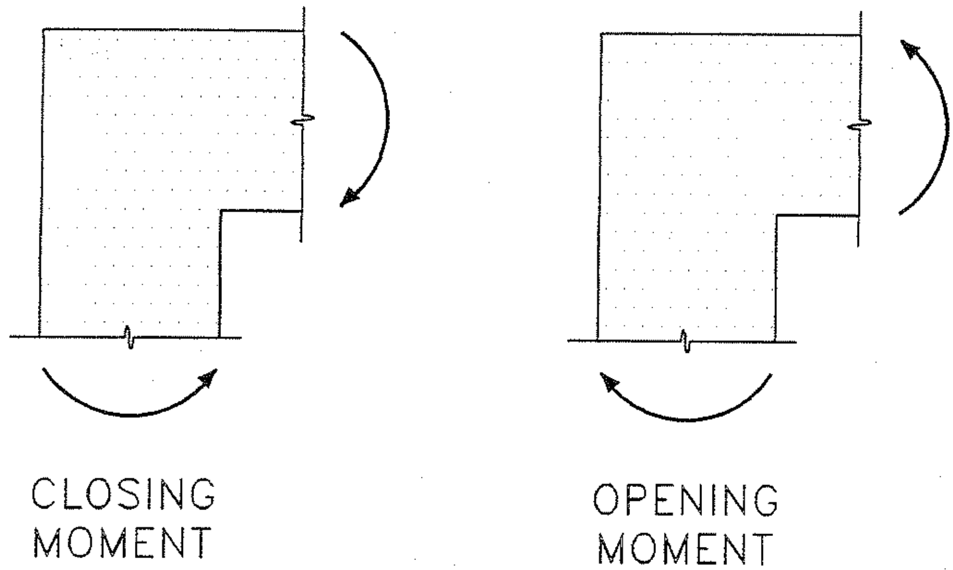


Figure 3.17 Closing and Opening Moments in Knee Joints

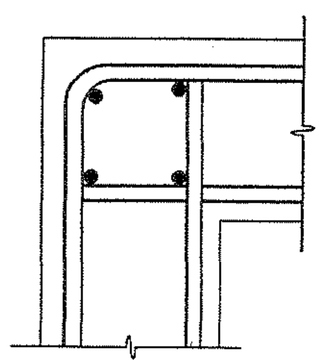


Figure 3.18 Suggested Detail of Knee Joint under Closing Moment

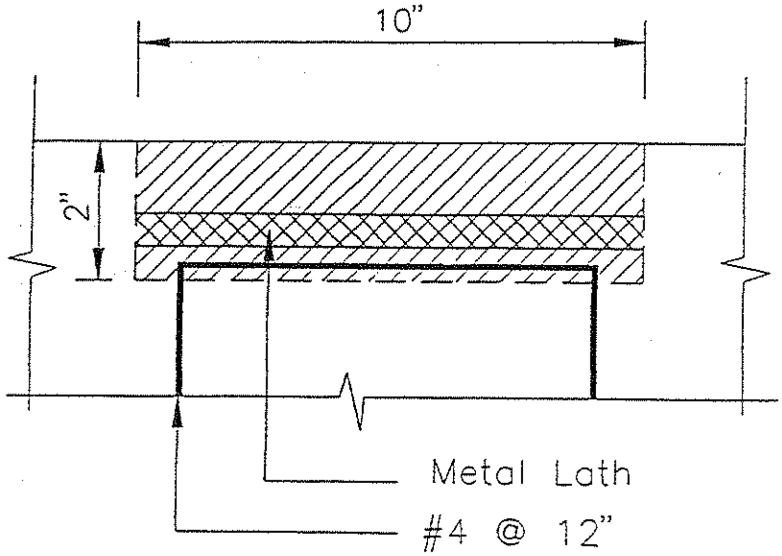
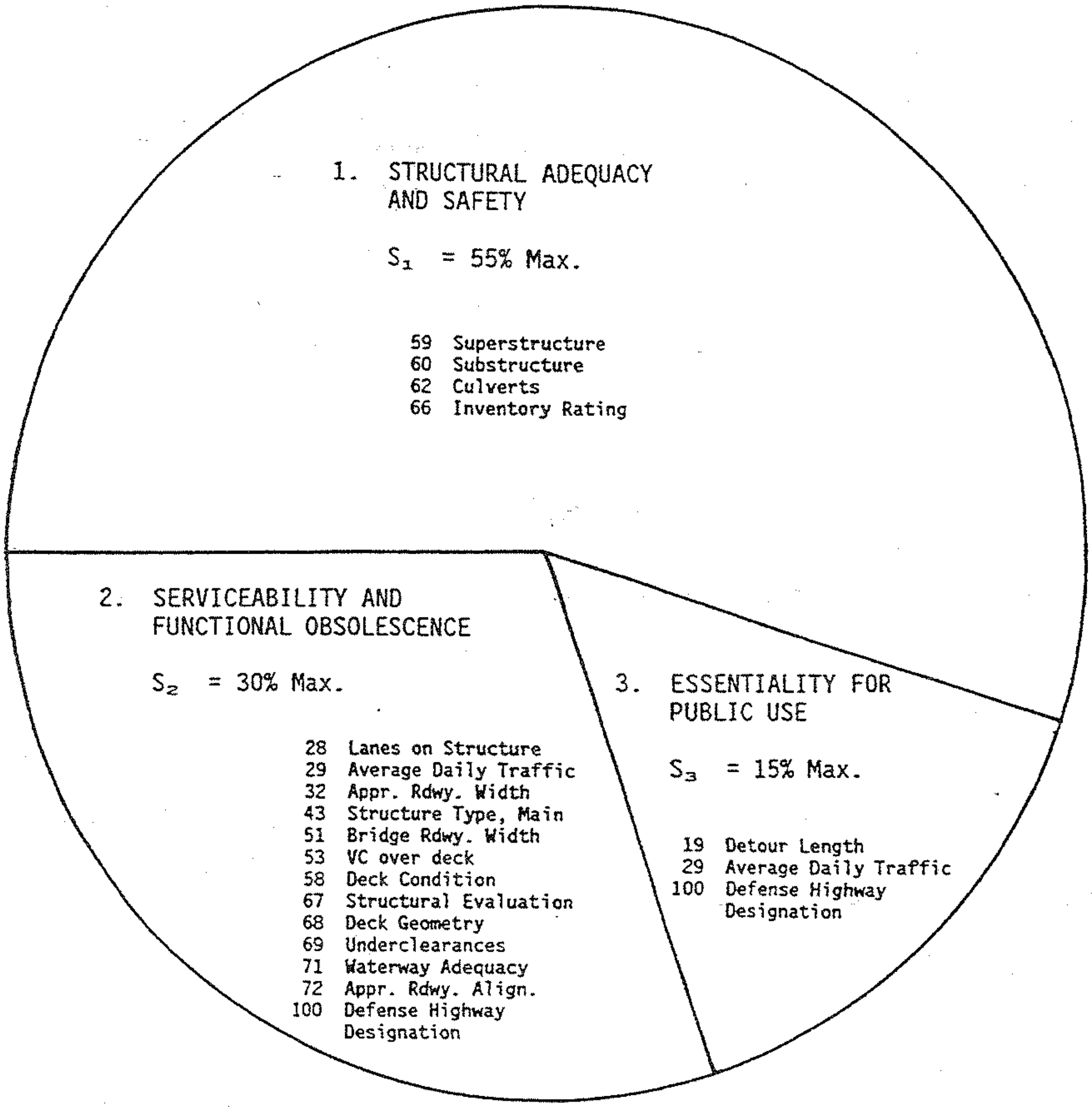


Figure 4.1 Repair of Cracks ≥ 0.016"

Appendix A
Sufficiency Rating Formula



4. SPECIAL REDUCTIONS

S_4 = 13% Max.

19 Detour Length
36 Traffic Safety Features
43 Structure Type, Main

SUFFICIENCY RATING $= S_1 + S_2 + S_3 - S_4$

Sufficiency Rating shall not be less than 0% nor greater than 100%

Sufficiency Rating Formula

1. Structural Adequacy and Safety (55% maximum)

   a. Only the lowest code of Item 59, 60, or 62 applies.

If #59 (Superstructure Rating) or #60 (Substructure Rating) is	≤ 2	then	A = 55%
	= 3		B = 40%
	= 4		C = 25%
	= 5		D = 10%
If #59 and #60 = N and #62 (Culvert Rating) is	≤ 2	then	E = 55%
	= 3		F = 40%
	= 4		G = 25%
	= 5		H = 10%

   b. Reduction for Load Capacity:

      (1) Calculate AIT (Adjusted Inventory Tonnage) as follows:

      If the 1st digit of #66 = 1, AIT = the 2nd & 3rd digits x 1.56;

      If the 1st digit of #66 = 2, AIT = the 2nd & 3rd digits x 1.00;

      If the 1st digit of #66 = 3, AIT = the 2nd & 3rd digits x 1.56;

      If the 1st digit of #66 = 4, AIT = the 2nd & 3rd digits x 1.01;

      If the 1st digit of #66 = 5, AIT = the 2nd & 3rd digits x 0.77;

      If the 1st digit of #66 = 6, AIT = the 2nd & 3rd digits x 0.67;

      If the 1st digit of #66 = 9, AIT = the 2nd & 3rd digits x 1.00;

      (2) Calculate using the following formulas or use Figure 2:

      $I = (36 - AIT)^{1.5} \times 0.2778$

      If $(36 - AIT) \leq 0$, then $I = 0$

      "I" shall not be less than 0% nor greater than 55%.

$S_1 = 55 - (A + B + C + D + E + F + G + H + I)$

S_1 shall not be less than 0% nor greater than 55%.

2. Serviceability and Functional Obsolescence (30% maximum)

a. Rating Reductions (13% maximum)

If #58 (Deck Condition) is	≤ 3 $= 4$ $= 5$	then	A = 5% A = 3% A = 1%
If #67 (Structural Evaluation) is	≤ 3 $= 4$ $= 5$	then	B = 4% B = 2% B = 1%
If #68 (Deck Geometry) is	≤ 3 $= 4$ $= 5$	then	C = 4% C = 2% C = 1%
If #69 (Underclearances) is	≤ 3 $= 4$ $= 5$	then	D = 4% D = 2% D = 1%
If #71 (Waterway Adequacy) is	≤ 3 $= 4$ $= 5$	then	E = 4% E = 2% E = 1%
If #72 (Approach Road Alignment) is	≤ 3 $= 4$ $= 5$	then	F = 4% F = 2% F = 1%

J = (A + B + C + D + E + F)

J shall not be less than 0% nor greater than 13%.

b. Width of Roadway Insufficiency (15% maximum)

Use the sections that apply:
- (1) applies to all bridges;
- (2) applies to 1-lane bridges only;
- (3) applies to 2 or more lane bridges;
- (4) applies to all <u>except</u> 1-lane bridges.

Also determine X and Y:

X (ADT/Lane) = #29 (ADT) $\div$ first 2 digits of #28 (Lanes)

Y (Width/Lane) = #51 (Bridge Rdwy. Width) $\div$ first 2 digits of #28

(1) Use when the last 2 digits of #43 (Structure Type) are not equal to 19 (Culvert):

If (#51 + 2 Ft.) < #32 (Approach Roadway Width) G = 5%

(2) For 1-lane bridges only, use Figure 3 or the following:

If the first 2 digits of #28 (Lanes) are equal to 01 and

$Y < 14$	then	$H = 15\%$
$Y \geq 14 < 18$		$H = 15\left(\frac{18-Y}{4}\right)\%$
$Y \geq 18$		$H = 0\%$

(3) For 2 or more lane bridges. If these limits apply, do not continue on to (4) as no lane width reductions are allowed.

If the first 2 digits of #28 = 02 and $Y \geq 16$, $H = 0\%$

If the first 2 digits of #28 = 03 and $Y \geq 15$, $H = 0\%$

If the first 2 digits of #28 = 04 and $Y \geq 14$, $H = 0\%$

If the first 2 digits of #28 $\geq$ 05 and $Y \geq 12$, $H = 0\%$

(4) For all <u>except</u> 1-lane bridges, use Figure 3 or the following:

If $Y < 9$ and $X > 50$	then	$H = 15\%$
$Y < 9$ and $X \leq 50$		$H = 7.5\%$
$Y \geq 9$ and $X \leq 50$		$H = 0\%$

If $X > 50$ but ≤ 125 and

$Y < 10$	then	$H = 15\%$
$Y \geq 10 < 13$		$H = 15\left(\frac{13-Y}{3}\right)\%$
$Y \geq 13$		$H = 0\%$

If $X > 125$ but ≤ 375 and

$Y < 11$	then	$H = 15\%$
$Y \geq 11 < 14$		$H = 15\left(\frac{14-Y}{3}\right)\%$
$Y \geq 14$		$H = 0\%$

If $X > 375$ but ≤ 1350 and

$Y < 12$	then	$H = 15\%$
$Y \geq 12 < 16$		$H = 15\left(\frac{16-Y}{4}\right)\%$
$Y \geq 16$		$H = 0\%$

If $X > 1350$ and

$Y < 15$	then	$H = 15\%$
$Y \geq 15 < 16$		$H = 15\ (16-Y)\%$
$Y \geq 16$		$H = 0\%$

G + H shall not be less than 0% nor greater than 15%.

c. Vertical Clearance Insufficiency - (2% maximum)

If #100 (Defense Highway Designation) > 0 and

#53 (VC over Deck) $\geq$ 1600	then	$I = 0\%$
#53 $<$ 1600		$I = 2\%$

If #100 = 0 and

#53 $\geq$ 1400	then	$I = 0\%$
#53 $<$ 1400		$I = 2\%$

$$S_2 = 30 - \left[J + (G + H) + I\right]$$

S_2 shall not be less than 0% nor greater than 30%.

3. Essentiality for Public Use (15% maximum)

a. Determine:

$$K = \frac{S_1 + S_2}{85}$$

b. Calculate:

$$A = \frac{\#29\ (ADT) \times \#19\ (Detour\ Length)}{200{,}000 \times K} \times 15$$

"A" shall not be less than 0% nor greater than 15%.

c. Defense Highway Designation:

If #100 is > 0	then	B = 2%
If #100 = 0	then	B = 0%

$S_3 = 15 - (A + B)$

S_3 shall not be less than 0% nor greater than 15%.

4. Special Reductions (Use only when $S_1 + S_2 + S_3 \geq 50$)

a. Detour Length Reduction, use Figure 4 or the following:

$A = (\#19)^4 \times (5.205 \times 10^{-8})$

"A" shall not be less than 0% nor greater than 5%.

b. If the 2nd and 3rd digits of #43 (Structure Type, Main) are equal to 10, 12, 13, 14, 15, 16, or 17; then

B = 5%

c.

If 2 digits of #36 (Traffic Safety Features)	= 0	C = 1%
If 3 digits of #36	= 0	C = 2%
If 4 digits of #36	= 0	C = 3%

$S_4 = A + B + C$

S_4 shall not be less than 0% nor greater than 13%.

Sufficiency Rating = $S_1 + S_2 + S_3 - S_4$

List of CCEER Publications

Report No.	Publication
CCEER-84-1	Saiidi, M., and R. A. Lawver. "User's manual for LZAK-C64, a computer program to implement the Q-model on Commodore 64." *Report number CCEER-84-1.* Reno: University of Nevada, Department of Civil Engineering. January 1984.
CCEER-84-2	Douglas, B. M., and T. Iwasaki. "Proceedings of the first USA-Japan bridge engineering workshop," held at the Public Works Research Institute, Tsukuba, Japan. *Report number CCEER-84-2.* Reno: University of Nevada, Department of Civil Engineering. April 1984.
CCEER-84-3	Saiidi, M., J. D. Hart, and B. M. Douglas. "Inelastic static and dynamic analysis of short R/C bridges subjected to lateral loads." *Report number CCEER-84-3.* Reno: University of Nevada, Department of Civil Engineering. July 1984.
CCEER-84-4	Douglas, B. "A proposed plan for a national bridge engineering laboratory." *Report number CCEER-84-4.* Reno: University of Nevada, Department of Civil Engineering. December 1984.
CCEER-85-1	Norris, G. M., and P. Abdollaholiaee. "Laterally loaded pile response: Studies with the strain wedge model." *Report number CCEER-85-1.* Reno: University of Nevada, Department of Civil Engineering. April 1985.
CCEER-86-1	Ghusn, G. E., and M. Saiidi. "A simple hysteretic element for biaxial bending of R/C columns and implementation in NEABS-86." *Report number CCEER-86-1.* Reno: University of Nevada, Department of Civil Engineering. July 1986.
CCEER-86-2	Saiidi, M., R. A. Lawver, and J. D. Hart. "User's manual of ISADAB and SIBA, computer programs for nonlinear transverse analysis of highway bridges subjected to static and dynamic lateral loads." *Report number CCEER-86-2.* Reno: University of Nevada, Department of Civil Engineering. September 1986.
CCEER-87-1	Siddharthan, R. "Dynamic effective stress response of surface and embedded footings in sand." *Report number CCEER-87-1.* Reno: University of Nevada, Department of Civil Engineering. June 1987.
CCEER-87-2	Norris, G., and R. Sack. "Lateral and rotational stiffness of pile groups for seismic analysis of highway bridges." *Report number CCEER-87-2.* Reno: University of Nevada, Department of Civil Engineering. June 1987.
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