DYNAMIC VEHICULAR LOADING OF THE NORTH FLOODWAY BRIDGES, 1975

by

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conducted for

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in cooperation with the

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Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

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PREFACE

This investigation was initiated on 1 July 1975 for the State Department of Highways and Public Transportation by Sam Cox, District 21 (Pharr) through Kenneth D. Hankins, D-10 Research (Austin) and Don W. McGowan, D-18, Maintenance and Operations Division (Austin). Field studies were conducted on 9 July 1975 by Center for Highway Research personnel, Dr. Clyde E. Lee, J. Leon Snider, and Randy Wallin, in cooperation with Mr. McGowan and personnel from the Resident Engineer's office in Raymondville.

Data reduction and analysis were performed at the Center by Dr. Hugh J. Williamson, Research Engineer Associate IV; J. Leon Snider, Technical Staff Assistant V; Randy Wallin, Computer Programmer I; Joe D. Word, Laboratory Research Assistant II; Steven H. Golding, Laboratory Research Assistant II; and other staff using facilities at the University and at the State Department of Highways and Public Transportation and computer programs developed through previous studies under the continuing Cooperative Highway Research Program. Professor Emeritus Phil M. Ferguson, Department of Civil Engineering, The University of Texas at Austin, contributed freely of his observations to the study, also.

This is the third report dealing with bridge roughness that has been prepared for the State Department of Highways and Public Transportation during the past two years. All these investigations are direct examples of applying the results of research in solving field design, maintenance, and operational problems that are of immediate concern and of long-range interest.

Photographs were provided by Don W. McGowan.

SUMMARY

The half-mile-long twin reinforced concrete bridges on U.S. 77 which cross the North Floodway above Harlingen, Texas, were opened to traffic in 1974. The 12-in. deck has developed an undulating longitudinal profile with sags of 1/4 to 1/2 inch in most of the 25-ft spans; thus the riding quality is impaired, vehicles using the highway are subject to extra wear, and dynamic loads in excess of the static weight of traffic are induced.

In this study, a computer simulation technique was used to investigate the complex interaction between the existing road surface profile and two representative trucks in order to assess the magnitude and placement of the potentially large dynamic wheel loads on the bridge structure. Maximum wheel forces from 50 to 100 percent greater than static wheel weight were predicted for the heavy vehicles operating at speeds between 40 and 55 mph.

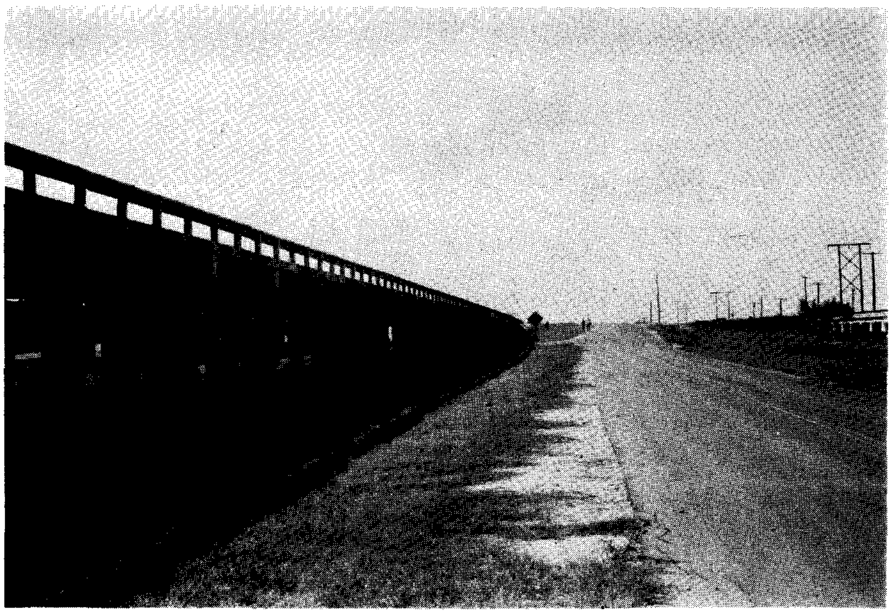
Since these large dynamic loads occur in the normal speed range for traffic, smoothing the surface profile with an overlay is recommended. Speed-zoning can be used for temporary alleviation, but effective enforcement will be very difficult on this particular highway.

DYNAMIC VEHICULAR LOADING OF THE
NORTH FLOODWAY BRIDGES, 1975

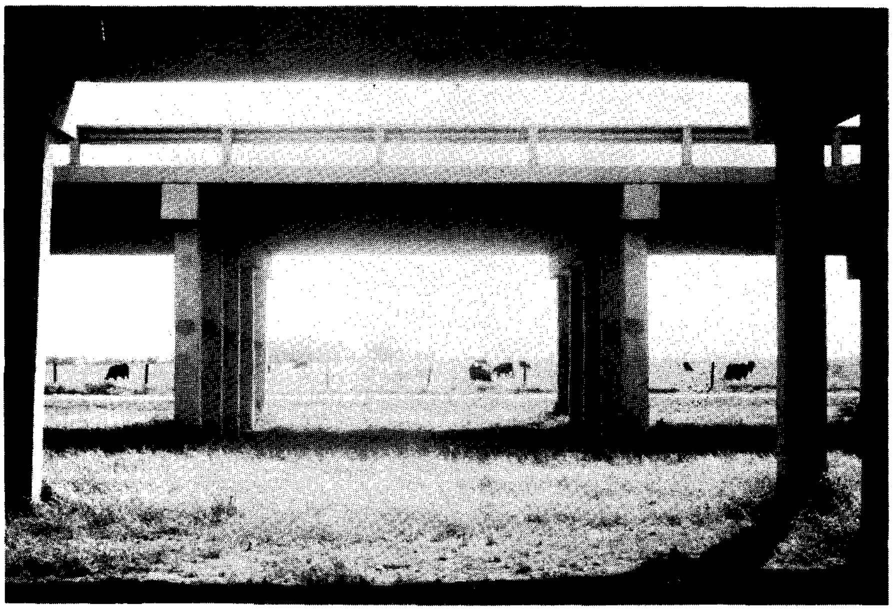
The North Floodway Bridges are located on U.S. 77 in Cameron County some 8 miles north of Harlingen, Texas. Twin bridges, each approximately one-half mile long, carry two lanes of traffic in each direction about 15 ft above the floor of the broad, shallow floodway. The 44-ft wide deck of each bridge, which includes the two 12-ft traffic lanes, a 12-ft right shoulder, and a 6-ft left shoulder, consists of a 12-in. reinforced concrete slab supported on 106 five-column concrete bents spaced nominally at 25-ft intervals (see photos). The slab is dowelled to each bent cap, and armored expansion joints are provided between the 200-ft, 125-ft, or 80-ft deck units. A 20-ft approach slab is added at the ends of both bridges.

These structures, which were opened to traffic in 1974, were constructed with a nominal 1/4-in. upward longitudinal camber in each span, but recent profile surveys show that virtually all spans now have sag of this magnitude or greater (see Appendix A). In some cases, elevation differences of up to an inch or more in a 30-ft longitudinal distance exist. This undulating profile extends over the full length of both bridges and is consistent in the transverse direction. The longitudinal profile of each shoulder is quite similar to the profile of each traffic lane; therefore, traffic loading up to now has apparently not had additional detrimental effects on the structure. Because of this similarity in the longitudinal profiles of all the adjacent lanes, and since the bridges are less than two years old, the cause of the sagging profile is probably related more to construction technique or to concrete shrinkage and creep than to traffic loading.

This irregular, undulating surface profile, regardless of its cause, forces the wheels of vehicles crossing the bridges to translate vertically, and at certain speeds, the sprung mass (body) of some vehicles is caused to bounce, roll, and pitch severely. Under critical conditions when the vertical movements of the vehicle are reinforced by each wave in the deck profile, large dynamic wheel forces are produced. Previous research (Refs 1, 2, 3, and 4)



View of southbound Harlingen North Floodway Bridge showing arrangement of bents.



View of northbound Harlingen North Floodway Bridge showing deck and support structure.

has shown that the resulting wheel impact forces can be more than twice the corresponding static wheel weight. These severe dynamic loads need to be minimized in order to prolong the service life of the structure, prevent excessive wear or damage to vehicles using the bridges, and provide acceptable riding quality.

There are several possible approaches to reducing or minimizing the magnitude of dynamic loading caused by traffic operating on a rough surface profile. An obvious solution is to smooth the profile. Sometimes, this is undesirable or economically unfeasible. Speed control, when practical, tends to reduce the effects of a rough surface and offers temporary alleviation if properly enforced. Or, load-zoning can be applied in extreme cases to restrict the magnitude of permissible static vehicle weight.

The Center for Highway Research at The University of Texas at Austin was asked to analyze the nature and magnitude of dynamic loading that is resulting from mixed traffic using the North Floodway Bridges and to suggest possible remedial measures for controlling the loads. Through previous research, computer simulation techniques which describe the complex dynamic behavior of various types of vehicles traveling at different speeds over defined surface profiles have been developed. Equipment for measuring the essential characteristics of the road profile was available at the Center, and experience in using the computer models was available. Therefore, the requested study was undertaken.

Field Measurements

Longitudinal profile measurements in each wheel path of each lane of interest on the bridges were required as input data to the computer simulation program. The General Motors Road Surface Dynamics Profilometer operating under Center for Highway Research Study No. 3-8-71-156 was used to obtain these data on 9 July 1975. Preliminary tests showed that the best speed for the profilometer to operate was at 20 mph. Profile waves up to 100 ft long can be measured without significant distortion at this speed. It was found that the profilometer vehicle pitched and oscillated excessively at 40 mph, a speed at which somewhat longer waves could be measured. A cursory analysis of the preliminary profile data and observation of traffic using the bridges indicated that waves 100 ft or shorter would be of primary interest; therefore, all profile measurements were made at 20 mph.

Profiles of the full length of each bridge were plotted and examined visually to determine zones which included profile characteristics that are likely to cause large impact loads. While it is possible to run the simulated vehicles over the full length of the bridges, this was deemed unnecessary and wasteful. Three sections, each about 350 ft long, were selected to be representative of profile patterns that would probably cause large dynamic loads. The first section, shown in Fig 1, includes about 125 ft of the approach in the right traffic lane of the southbound bridge plus the first 125 ft of the adjoining deck at the north end. This section includes a long wave on the approach pavement, a sudden drop of about an inch onto the 20-ft approach slab, and a series of 25-ft waves in the first few spans of the bridge.

The second section, shown in Fig 6, includes parts of two 200-ft deck units near the center of the northbound bridge and contains the repeating sawtooth pattern of 25-ft waves that are 1/4 to 1/2-inch in amplitude found throughout the length of both bridges. The third section (see Fig 12) includes about 150 ft of the north end of the northbound bridge, the tilted 20-ft approach (departure) slab, and some 150 ft of pavement just off the bridge in the right traffic lane. An elevation difference of 1-1/2 inches has developed in the 25-ft zone beyond the approach slab. All these profile plots show only the left wheel path, but measurements were made and used in the simulation model for both wheel paths. Visual examination and statistical analysis of the relationship between right and left wheel path profiles indicated great similarity; therefore, only the left wheel path profiles have been illustrated in these figures. Plots of the right wheel path profiles in the outside lane of both bridges are given in Appendix A.

In addition to the profile measurements, live-load deflection measurements were made near the middle of a 200-ft unit of the southbound bridge. A dial indicator with a least reading of 0.0001 inch was supported from the ground under the outside traffic lane and allowed to contact the bottom surface of the deck slab midway between two bents. The maximum live-load deflection observed was approximately 0.060 inch. The dead-load-only reading of the dial changed about 0.007 inch in the 4-hr period beginning at 11:00 A.M. Deflections of this magnitude can be assumed to have negligible effects on the dynamic behavior of vehicles on the bridge.

Vehicle Simulation

Although a wide variety of vehicles uses the bridges, critical dynamic loading is most likely to result from a few truck configurations. Grain and other agricultural produce are primary products hauled by the large trucks in the area. A single-unit two-axle dual-rear-tire vehicle (Type 2D; see Plate A) was chosen as representative of smaller trucks, and a five-axle articulated tractor-semi-trailer (Type 3S-2; see Plate B) unit was selected to represent the larger trucks. The parameters needed to characterize these vehicles were available from previous research and are summarized in Table 1.

Other types of vehicles such as mobile homes, cars towing camping trailers, and pickups with covers may experience adverse riding conditions on these bridges at certain speeds, but since these lighter vehicles will probably not create critical dynamic loads they were not included in this study. Further analysis of the effects of bridge roughness on ride quality is highly desirable, however.

The speed limit on U.S. 77 is normally 55 mph, and the bridges are expected to accommodate at least this speed. Observation of traffic and recent test rides over the North Floodway bridges by engineers in District 21 initiated the installation of advisory speed signs at 45 mph early in July 1975. Vehicle speeds between 20 mph and 60 mph were therefore used in the simulation study.

Analysis

Mathematical models of the two vehicles described above were, by computer simulation, "driven" over the selected sections of the bridges at various speeds. Tire forces that would result from the vehicles interacting with the surface profile were plotted and examined.

Figures 2 through 5 show the wheel forces predicted from the simulated vehicles operating over the profile shown in Fig 1. At 40 and 45 mph the dynamic rear wheel force of the 2D type truck varied from its static weight of 7,000 pounds by as much as 5,000 pounds (70 percent), and a similar percentage variation in wheel force for the rear axles of the 3S-2 type vehicle was produced at 50-55 mph (see Figs 4 and 5). The dynamic wheel force variations were found to be less than this at lower and higher speeds and the plots are therefore not included in this report.

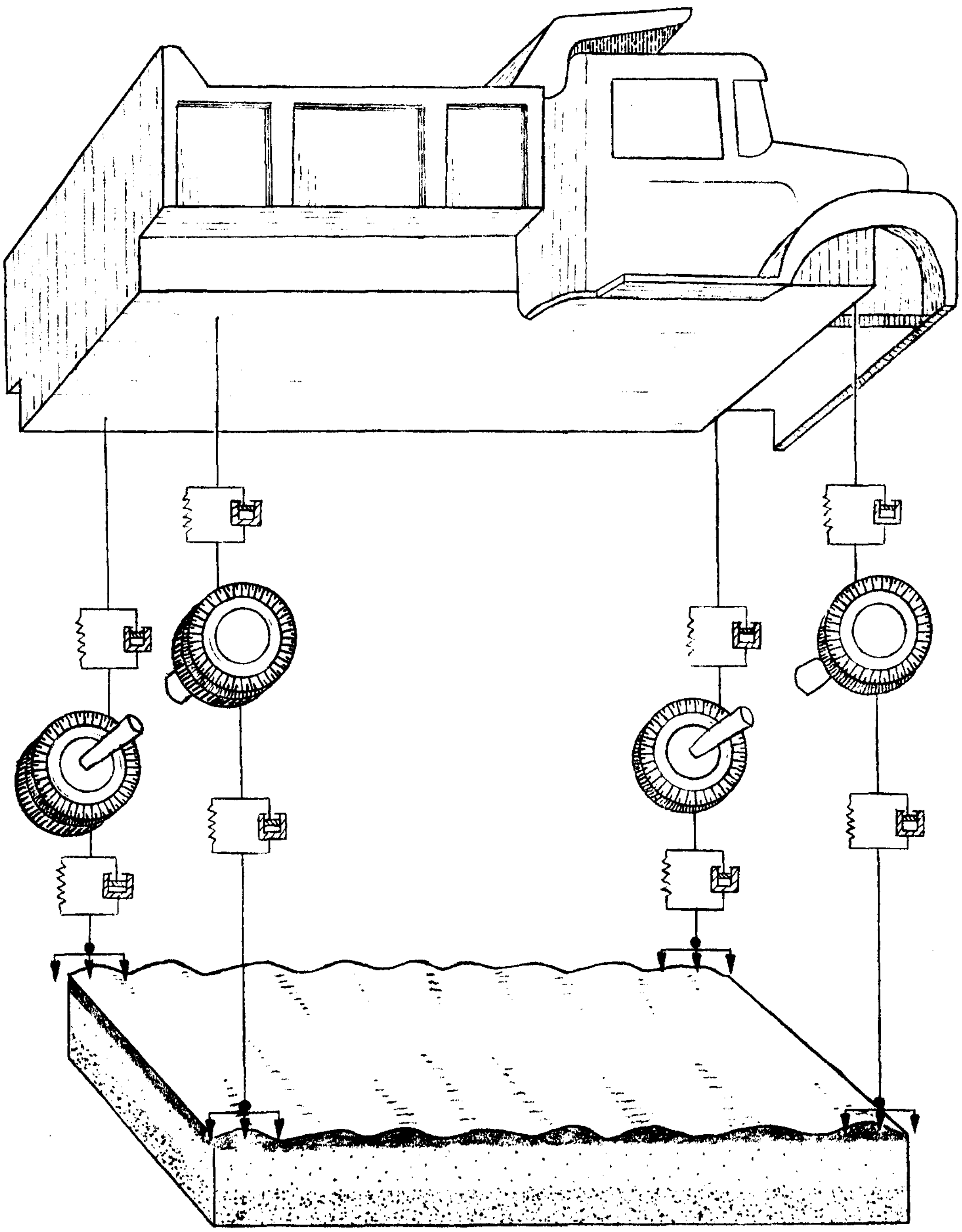


Plate A. Schematic diagram of single-unit two-axle dual tire (Type 2D) vehicle model.

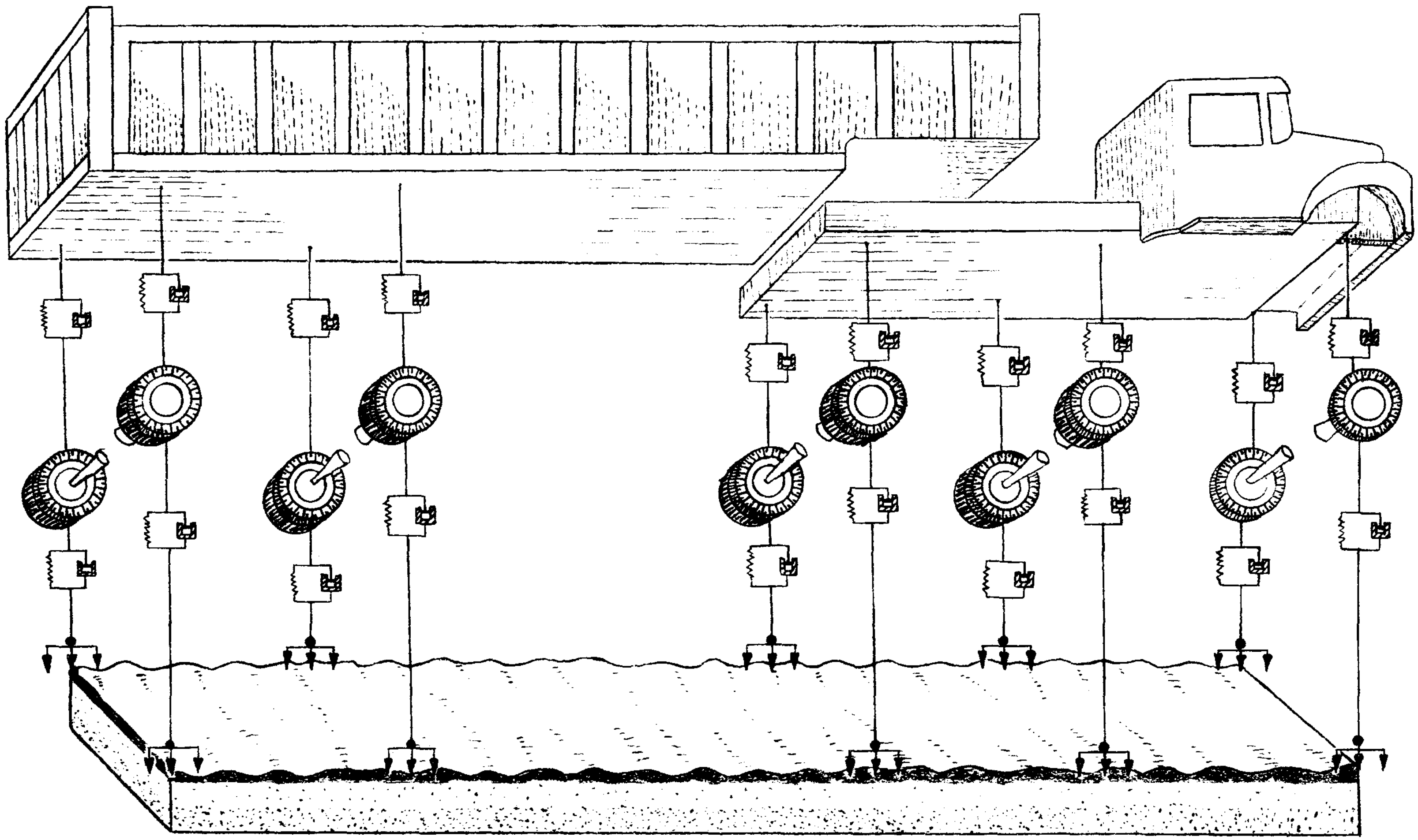


Plate B. Schematic diagram of five-axle tractor-semi-trailer articulated (Type 3S-2) vehicle model.

TABLE 1. VEHICLE CHARACTERISTICS

I. Two axle single unit (2D)

Body Mass	47.91	(lb-sec^2) / in.
Tread Width		
Axle 1	74.0	in.
Axle 2	70.0	in.
Axle Spacing	153.0	in.
Wheel Weights		
1 Right	3139	lb.
1 Left	3012	lb.
2 Right	7780	lb.
2 Left	7103	lb.
Suspension System		
Spring Stiffness		
Axle 1 Right and Left	535	lb/in.
Axle 2 Right and Left	3750	lb/in.
Damping		
Axle 1 Right and Left	5	percent of critical
Axle 2 Right and Left	3	percent of critical
Tires		
Stiffness		
Axle 1 Right and Left	4000	lb/in.
Axle 2 Right and Left (Duals)	8000	lb/in.
Damping		
Axle 1 Right and Left	2	percent of critical
Axle 2 Right and Left	2	percent of critical

(continued)

TABLE 1. (continued)

II. Five axle articulated (3S-2)		
Cab Mass	40.5	$(\text{lb-sec}^2) / \text{in.}$
Trailer Mass	143	$(\text{lb-sec}^2) / \text{in.}$
Tread Width		
Axle 1	77.0	in.
Axle 2	71.0	in.
Axle 3	71.0	in.
Axle 4	73.0	in.
Axle 5	73.0	in.
Axle Spacing		
Axle 1-2	147.0	in.
Axle 1-3	196.0	in.
Axle 1-4	472.0	in.
Axle 1-5	523.0	in.
Wheel Weights		
1 Right	6000	lb.
1 Left	6000	lb.
2 Right	8500	lb.
2 Left	8500	lb.
3 Right	8500	lb.
3 Left	8500	lb.
4 Right	8500	lb.
4 Left	8500	lb.
5 Right	8500	lb.
5 Left	8500	lb.

(continued)

TABLE 1. (continued)

Suspension System		
<u>Spring Stiffness</u>		
Axle 1 Right and Left	2000	lb/in.
Axle 2 Right and Left	6000	lb/in.
Axle 3 Right and Left	6000	lb/in.
Axle 4 Right and Left	6000	lb/in.
Axle 5 Right and Left	6000	lb/in.
<u>Damping</u>		
Axle 1 Right and Left	4.5	percent of critical
Axle 2 Right and Left	3.0	percent of critical
Axle 3 Right and Left	3.0	percent of critical
Axle 4 Right and Left	1.5	percent of critical
Axle 5 Right and Left	1.5	percent of critical
Tires		
<u>Stiffness</u>		
Axle 1 Right and Left	4500	lb/in.
Axle 2 Right and Left	8000	lb/in.
Axle 3 Right and Left	8000	lb/in.
Axle 4 Right and Left	7500	lb/in.
Axle 5 Right and Left	7500	lb/in.
<u>Damping</u>		
Axle 1 Right and Left	0.01	percent of critical
Axle 2 Right and Left	0.50	percent of critical
Axle 3 Right and Left	0.50	percent of critical
Axle 4 Right and Left	0.25	percent of critical
Axle 5 Right and Left	0.25	percent of critical

The dynamic wheel forces expected to result from the simulated vehicles operating on the profile shown in Fig 6 are illustrated in Figs 7 through 11. The repeating 25-ft waves in the bridge profile cause the 2D type truck to oscillate most severely at 40 mph and produce dynamic wheel forces up to about 50 percent greater than static wheel weight (see Figs 7 and 8). This profile induced the greatest dynamic effects in the front axle and the trailer axles of the 3S-2 type vehicle at 50 mph (see Figs 9, 10, and 11). In Fig 10, it can be noted that the rearmost wheel almost leaves the surface and causes downward loads nearly double the static wheel weight. At 45 and 55 mph, the oscillations of the vehicle are not in phase with the waves in the profile, damping occurs, and resulting dynamic wheel forces do not reach this same magnitude.

Dynamic forces caused by the profile shown in Fig 12 are expected to be quite large since traffic moves off the upward-tilted approach (departure) slab and vehicle wheels fall into a 1-1/2-inch depression. Figures 13 and 14 show the predicted wheel forces for the 2D type vehicle running at 45 and at 60 mph. The unsprung mass (wheels and axles) oscillate at a frequency of about 10 to 12 Hz, as is typical, and the sprung mass (body and load) translates at about 2.5 Hz. The severe oscillations of the undercarriage caused by the step-off bump damp out in about 40 ft, and the sprung mass goes through about three cycles before its oscillations are damped. Wheel forces range from about zero to some 70 percent greater than static wheel weight. It is interesting that the peak predicted forces occur at 45 mph rather than at 60 mph in this case. Oscillations of the sprung and unsprung masses were in proper phase with each other and with the profile to cause a severe peak load in the first cycle of the vehicle oscillation beyond the large profile depression.

Individual wheel loads are of concern when considering local stress conditions in the bridge deck, but the magnitude and position of the gross dynamic load on a particular span must also be accounted for in design. The gross dynamic force on the span is simply the accumulation of all the wheel forces at a given instant. Figure 15 shows a plot of the gross dynamic load that results from the 3S-2 vehicle operating over the sawtooth deck profile near the middle of the northbound bridge (see Fig 6). The dynamic forces produced by the simulated 80,000 pound vehicle varied more than 30 percent from the static weight of the truck. This variation is of the same order as the impact factor that is normally applied in the structural design of major bridge elements.

Conclusion and Recommendations

The maximum dynamic wheel loads resulting from the simulation of two representative trucks crossing the North Floodway bridges occurred in the speed range between 40 and 55 mph. At these speeds, the undulating profile which includes a repeating pattern of 25-ft waves excited the vehicles in the range of 2.35 to 3.23 Hz, and caused the various vehicle components (sprung masses, unsprung masses, tires, and suspension) to react in such a way that the higher frequency oscillations of the undercarriage (around 12 Hz) were added to the oscillations of the body/load masses (around 3 Hz) at critical times to produce quite large dynamic wheel loads (50 to 100 percent greater than static weight). This speed range is, of course, the normal operating range for traffic on the bridges, and remedial measures that will reduce the magnitude of dynamic loading are indicated.

Load-zoning nor speed zoning seems practical on this major highway; therefore, smoothing the surface with an overlay to remove the sags between bents and compensate for bent cap misalignment that may now exist appears to be the best solution. The regular pattern of 25-ft waves in the profile (see Appendix A) suggests that alignment of the bent caps is generally satisfactory, but that the deck has sagged since construction. Special attention to the finished grade of an overlay will be required in order to remove the waves of 1/4 to 1/2-inch amplitude. Consideration should also be given to whether sagging of the deck will continue.

In informal discussions with Professor Emeritus Phil M. Ferguson, he pointed out that the pattern of longitudinal reinforcing steel used in these bridge decks is efficient to resist bending moment but that it tends to cause sagging between supports when the concrete shrinks or creeps. That is, the volume change in the concrete due to these phenomena is resisted by the reinforcing steel. Heavier bottom steel at mid-span and heavier top steel over the supports restrains the concrete in these zones from shrinking or creeping as much as that in the respective top and bottom fibers of the slab where lighter reinforcing is used.

A supplementary design criteria based on tolerable deflection should probably be considered. Building codes for flat slabs set the minimum thickness of the slab as L/28 for both ends continuous unless special deflection checks are made. The 1-ft thick deck slab over the 25-ft supports satisfies this criteria, but the 30-ft spans exceed it slightly. Even though there may be no direct analogy between buildings and bridges, it is interesting to note that the 30-ft spans which occur in the middle of the 80-ft units do not meet the criteria and that each of these longer spans has more sag than is generally seen in the 25-ft spans (see Appendix A, pp A-2, A-4, A-7, A-8, A-11, A-13, A-16, and A-17). Perhaps this observation can be noted for reference in future design of bridges of this type.

This study has dealt primarily with dynamic loading of the bridge structure by traffic, but consideration of the riding quality and of the effects of road roughness on vehicles is needed. At least one truck has already experienced severe damage (frame of the trailer collapsed) while traveling on one of these bridges, and complaints about a rough ride have been voiced recently. Smoothing the riding surface will solve these problems.

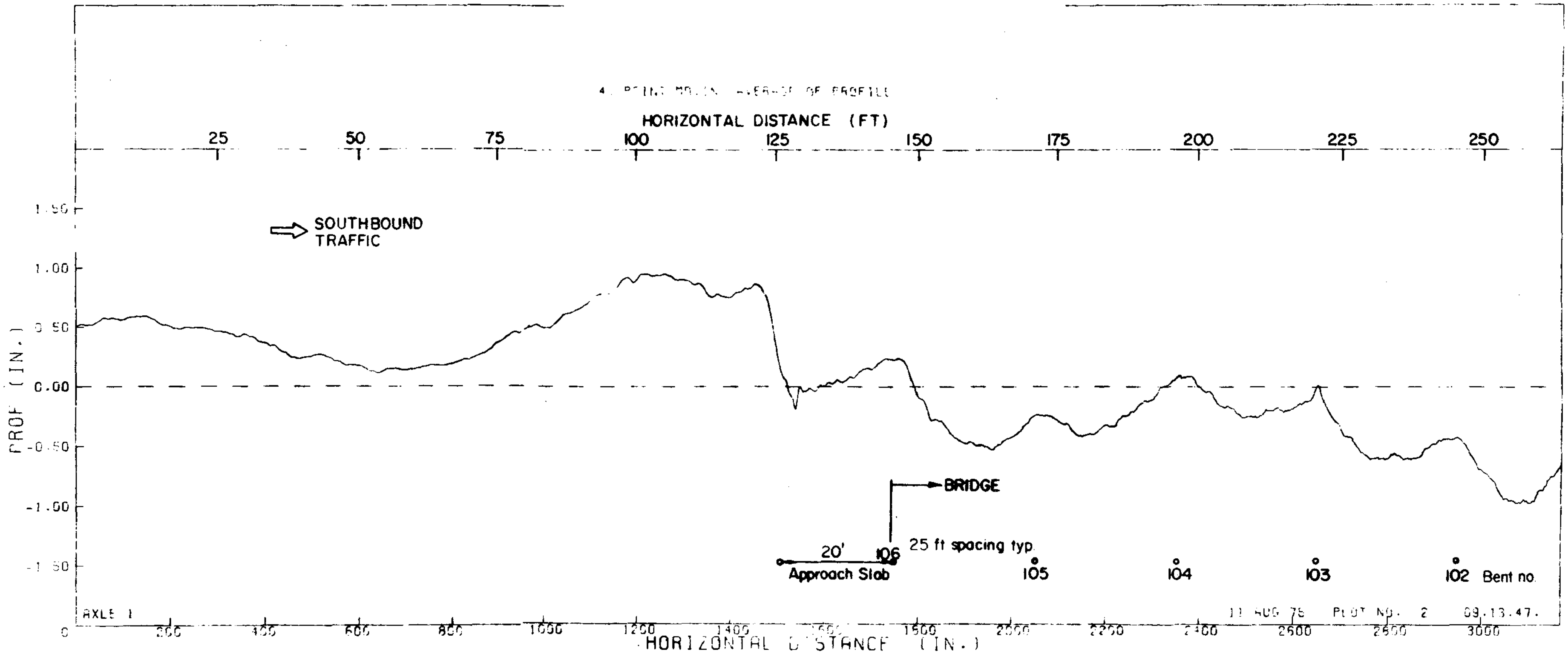


Fig 1. Profile of left wheel path of outside lane, southbound bridge, bent no.'s 106 to 102.

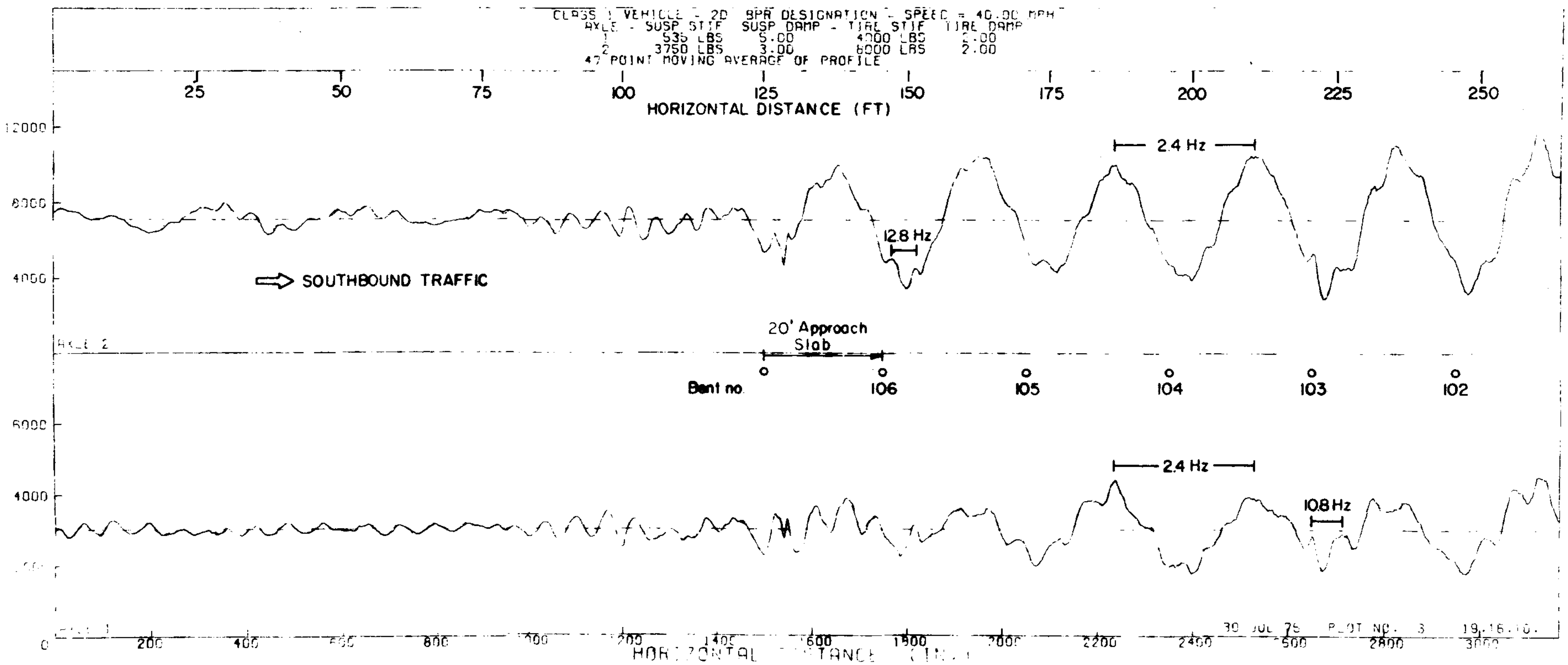


Fig 2. Left wheel forces on outside lane, southbound bridge, bent no.'s 106 to 102, 2-D truck at 40 mph.

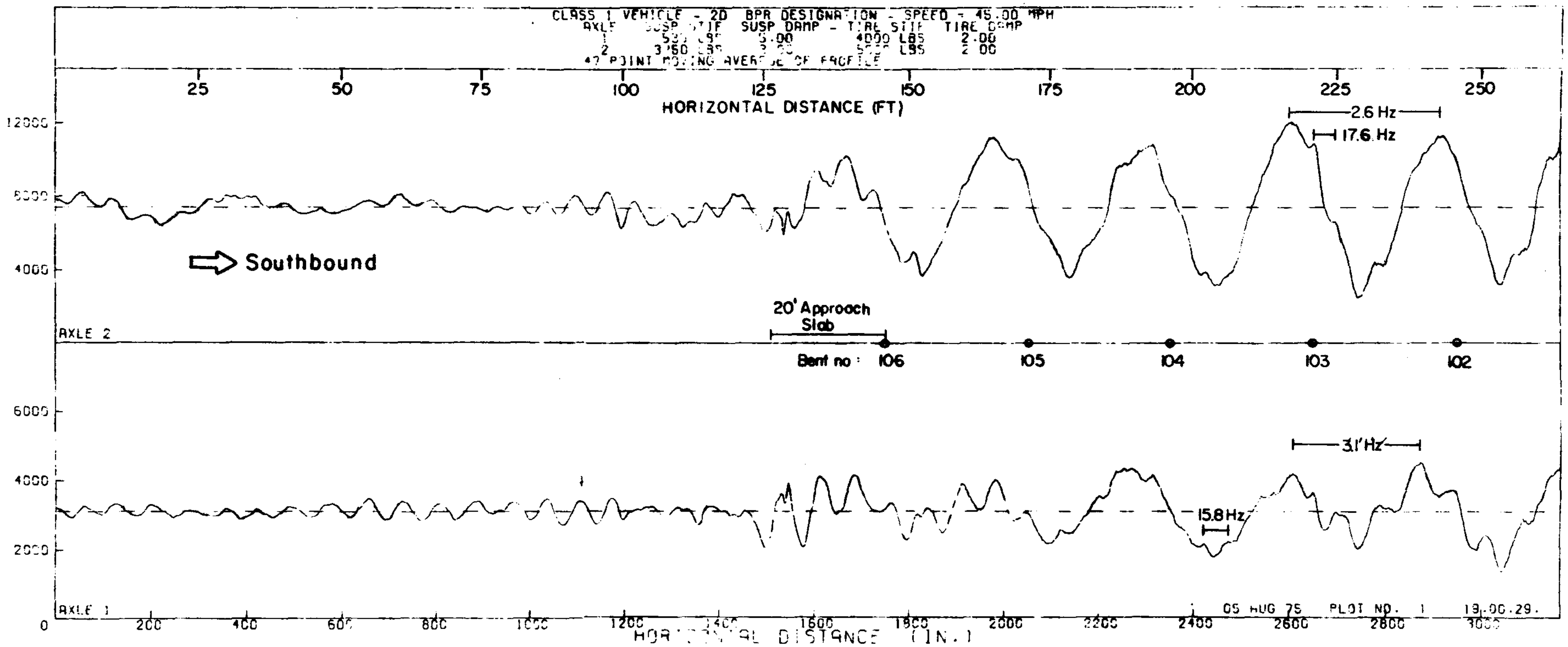


Fig 3. Left wheel forces on outside lane, southbound bridge, bent no.'s 106 to 102, 2-D truck at 45 mph.

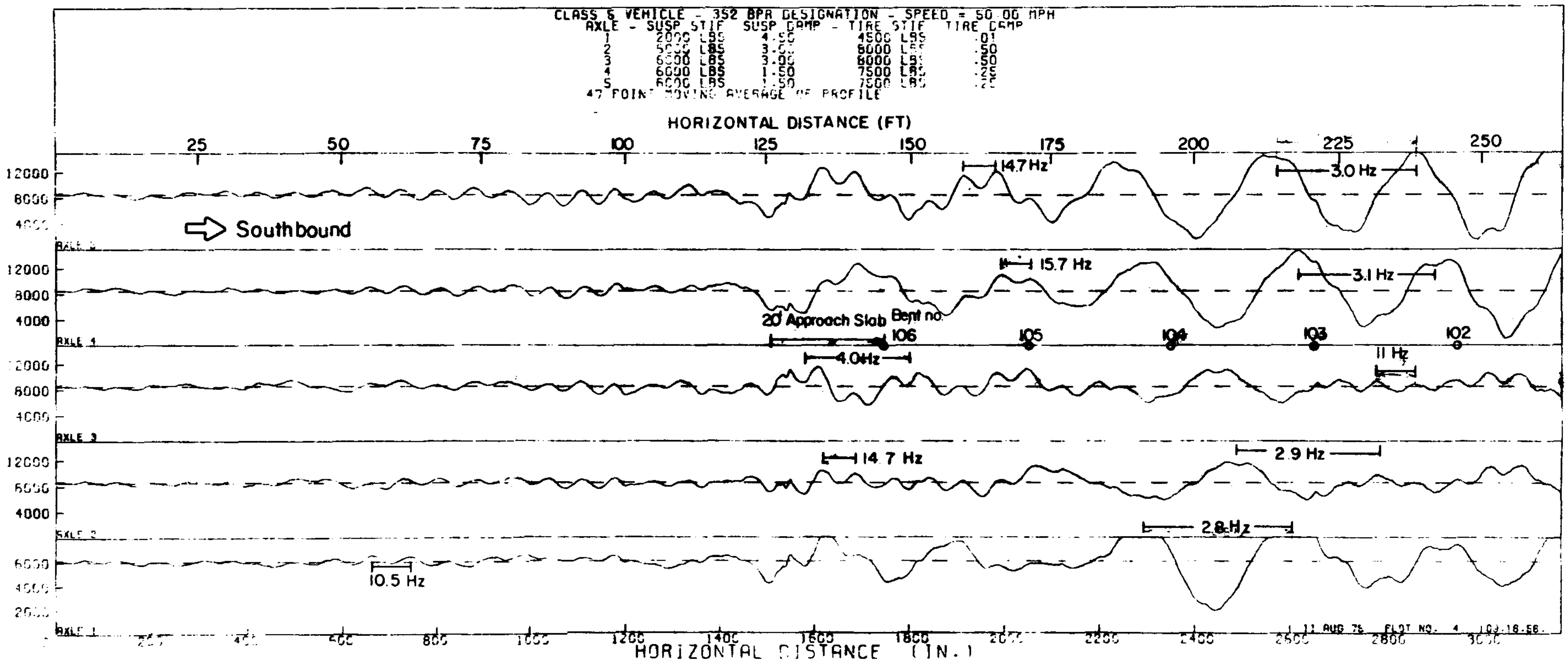


Fig 4. Left wheel forces on outside lane, southbound bridge, bent no.'s 106 to 102, 3S-2 truck at 50 mph.

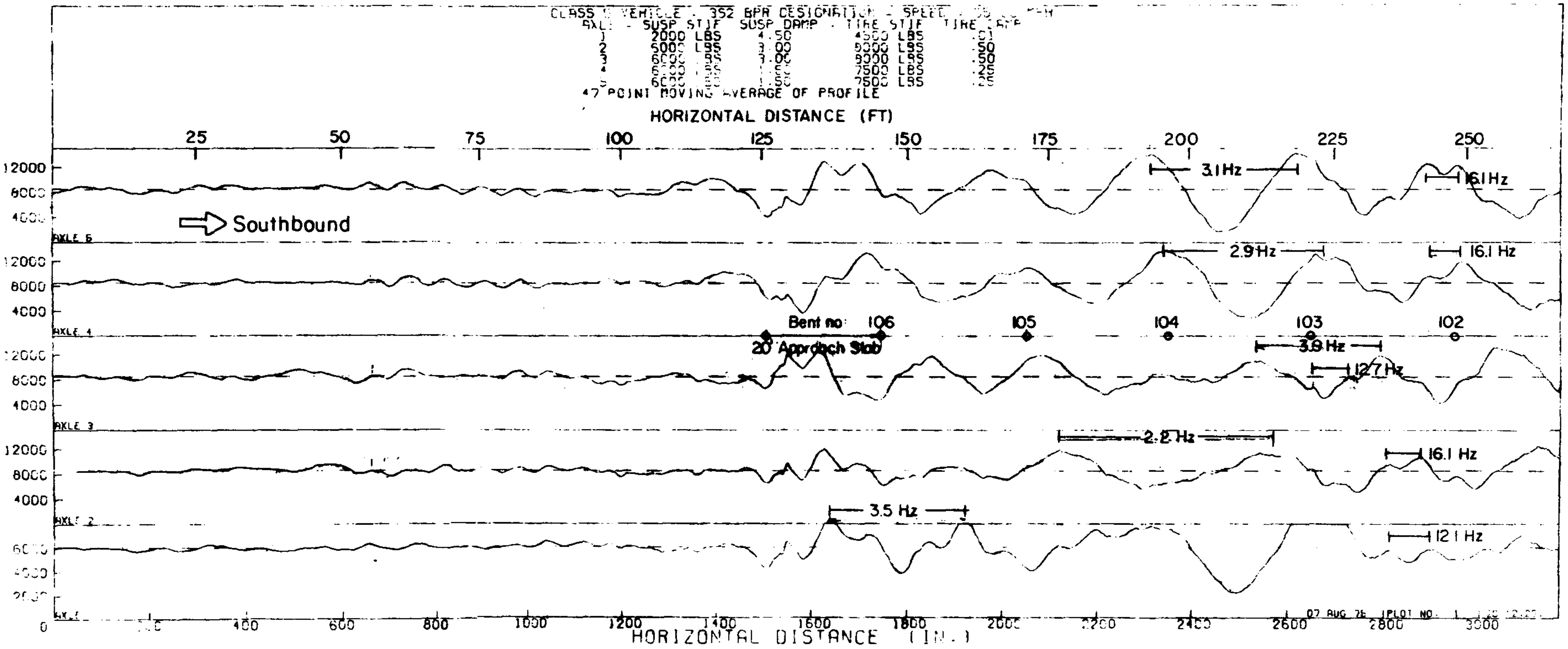


Fig 5. Left wheel forces on outside lane, southbound bridge, bent no.'s 106 to 102, 3S-2 truck at 55 mph.

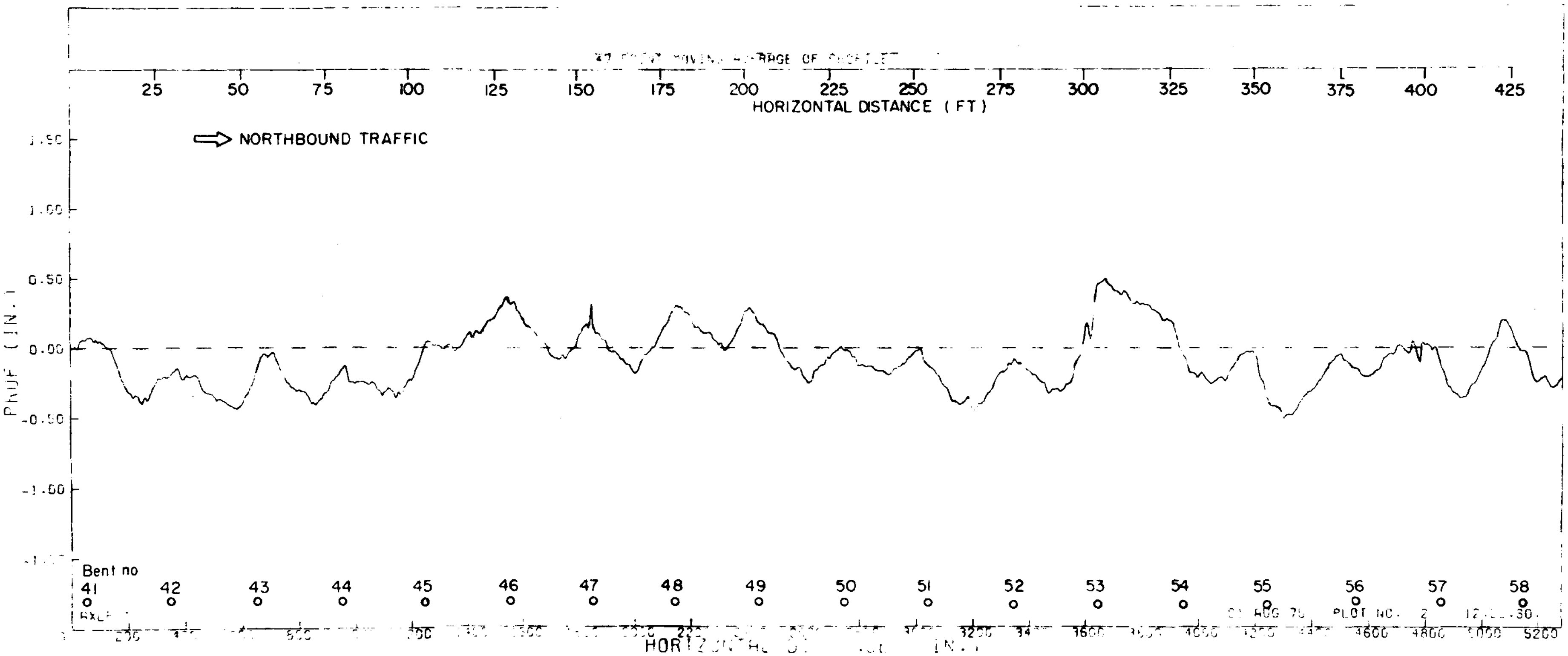


Fig 6. Profile of left wheel path of outside lane, northbound bridge, bent no.'s 41 to 58.

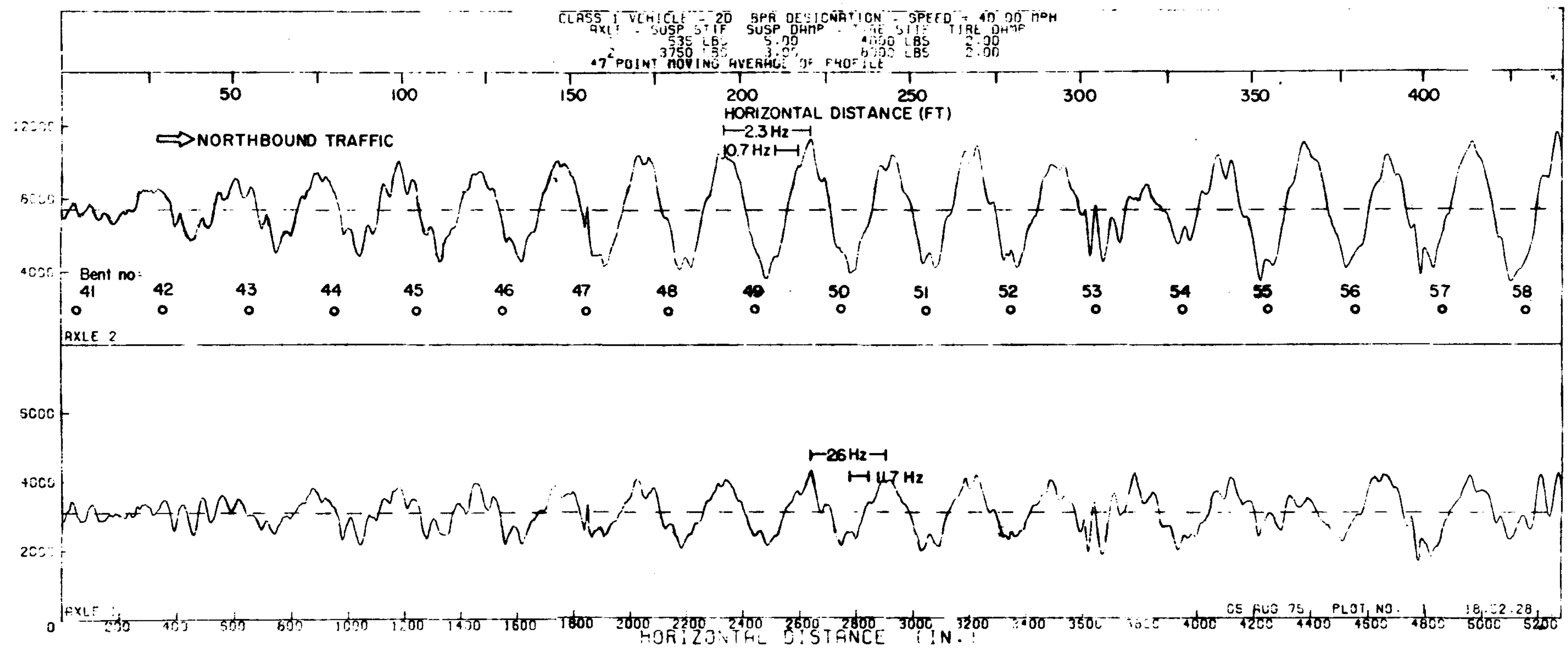


Fig 7. Left wheel forces on outside lane, northbound bridge, bent no.'s 41 to 58, 2-D truck at 40 mph.

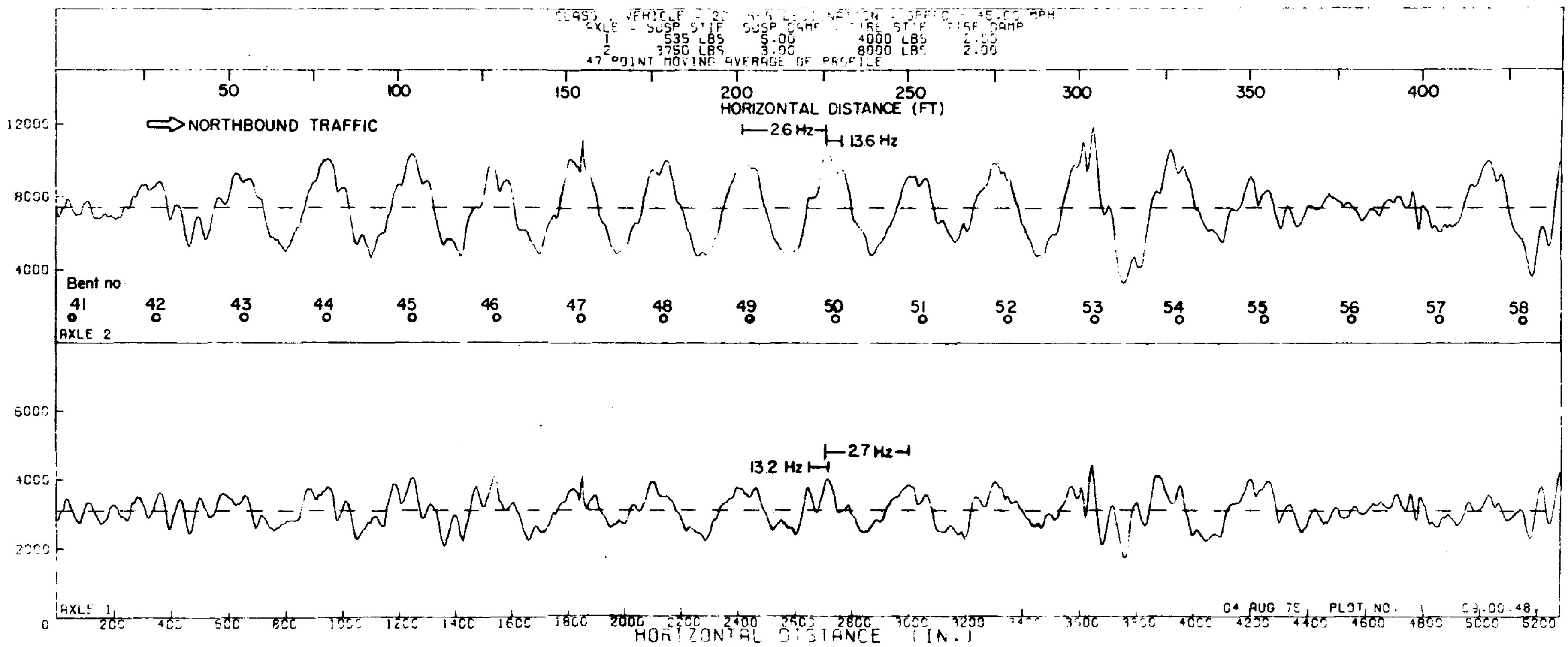


Fig 8. Left wheel forces on outside lane, northbound bridge, bent no.'s 41 to 58, 2-D truck at 45 mph.

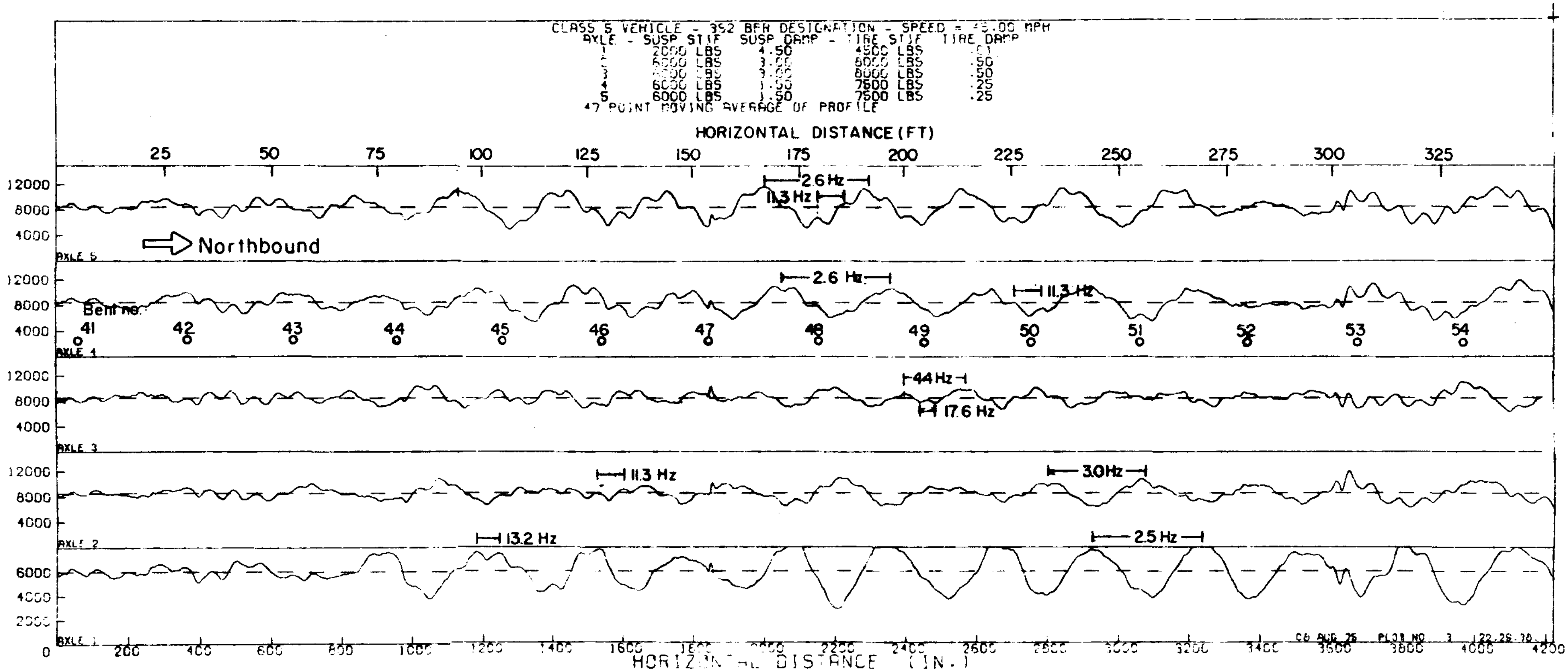


Fig 9. Left wheel forces on outside lane, northbound bridge, bent no.'s 41 to 54, 3S-2 truck at 45 mph.

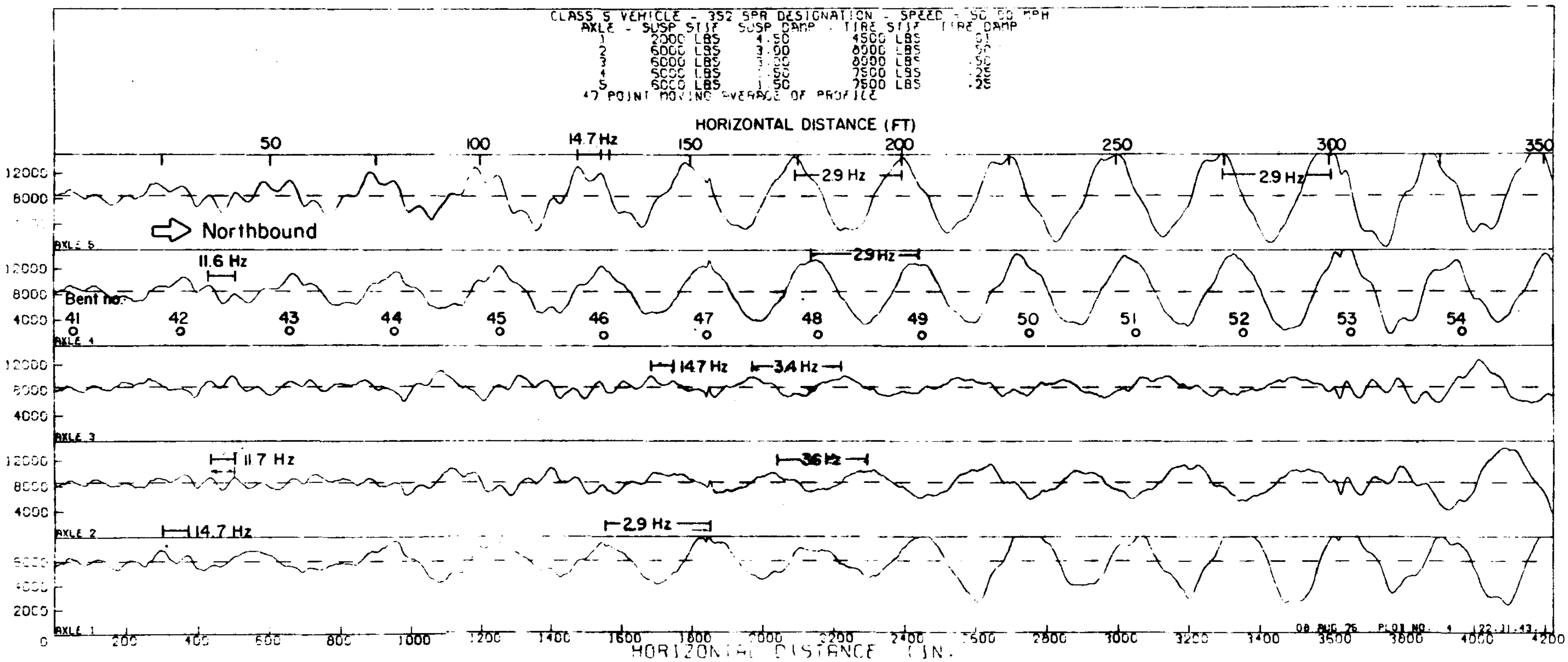


Fig 10. Left wheel forces on outside lane, northbound bridge, bent no.'s 41 to 54, 3S-2 truck at 50 mph.

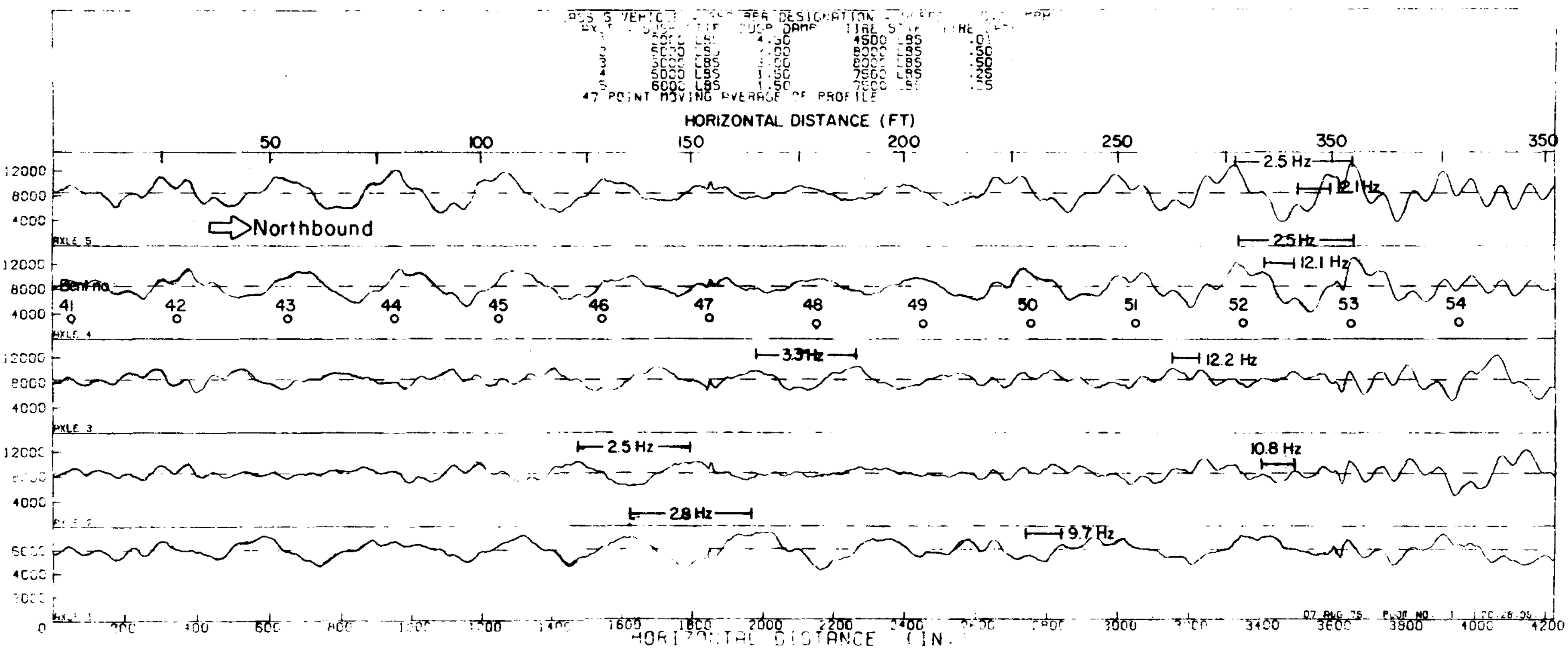


Fig 11. Left wheel forces on outside lane, northbound bridge, bent no.'s 41 to 54, 3S-2 truck at 55 mph.

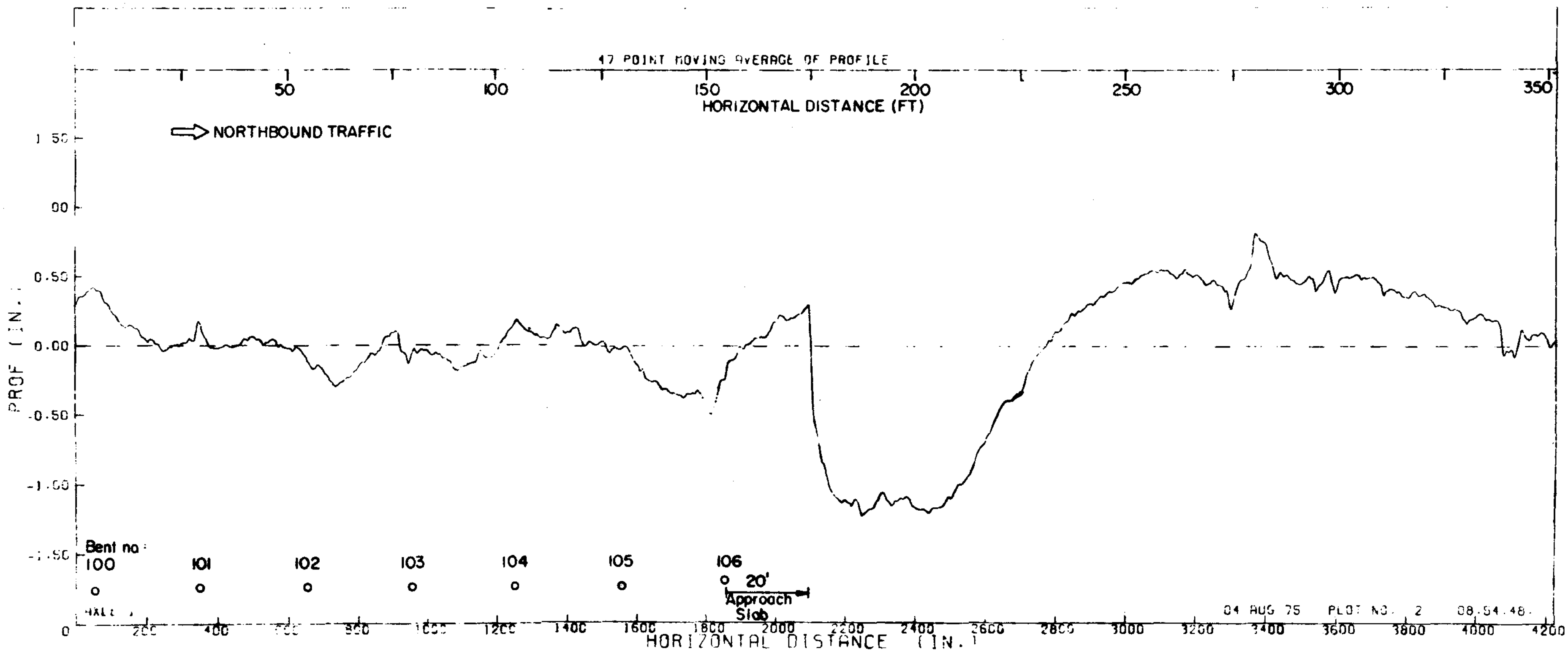


Fig 12. Profile of left wheel path of outside lane, northbound bridge, bent no.'s 100 to 106.

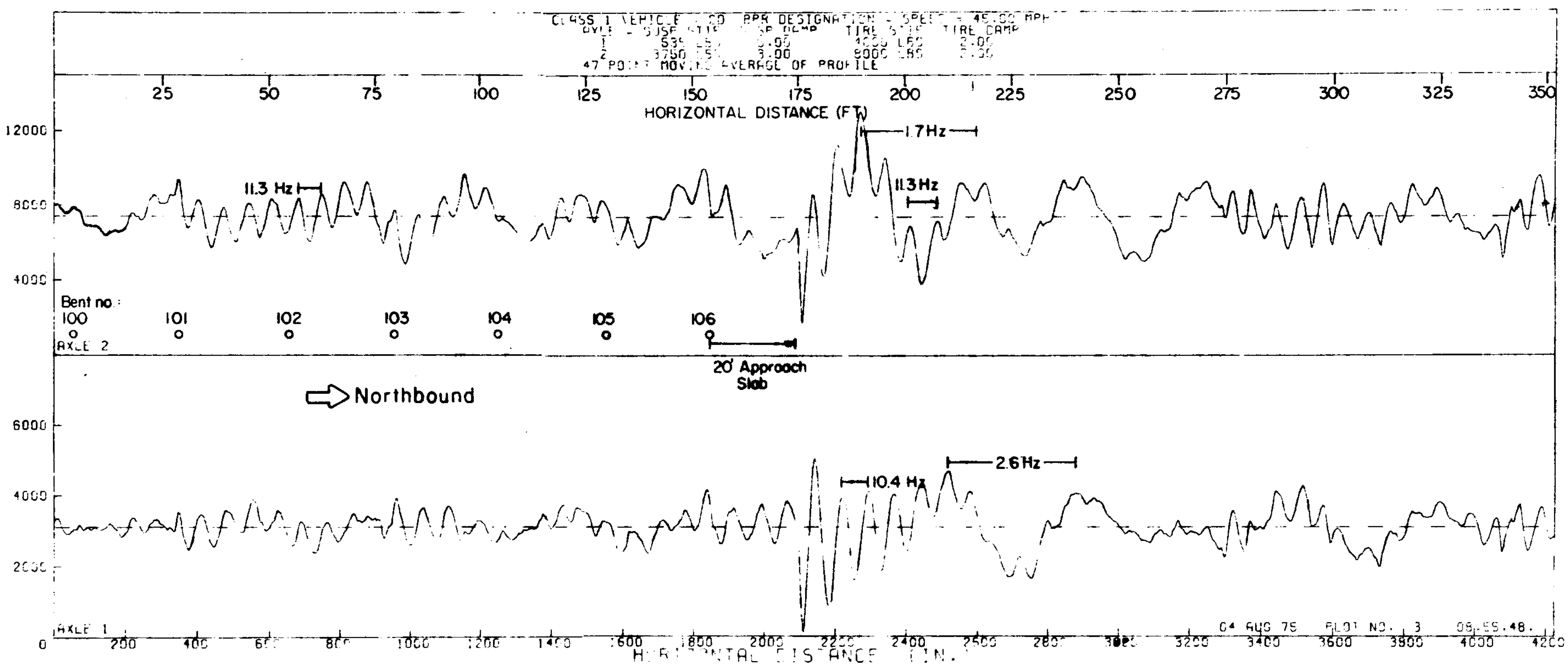


Fig 13. Left wheel forces on outside lane, northbound bridge, bent no.'s 100 to 106, 2-D truck at 45 mph.

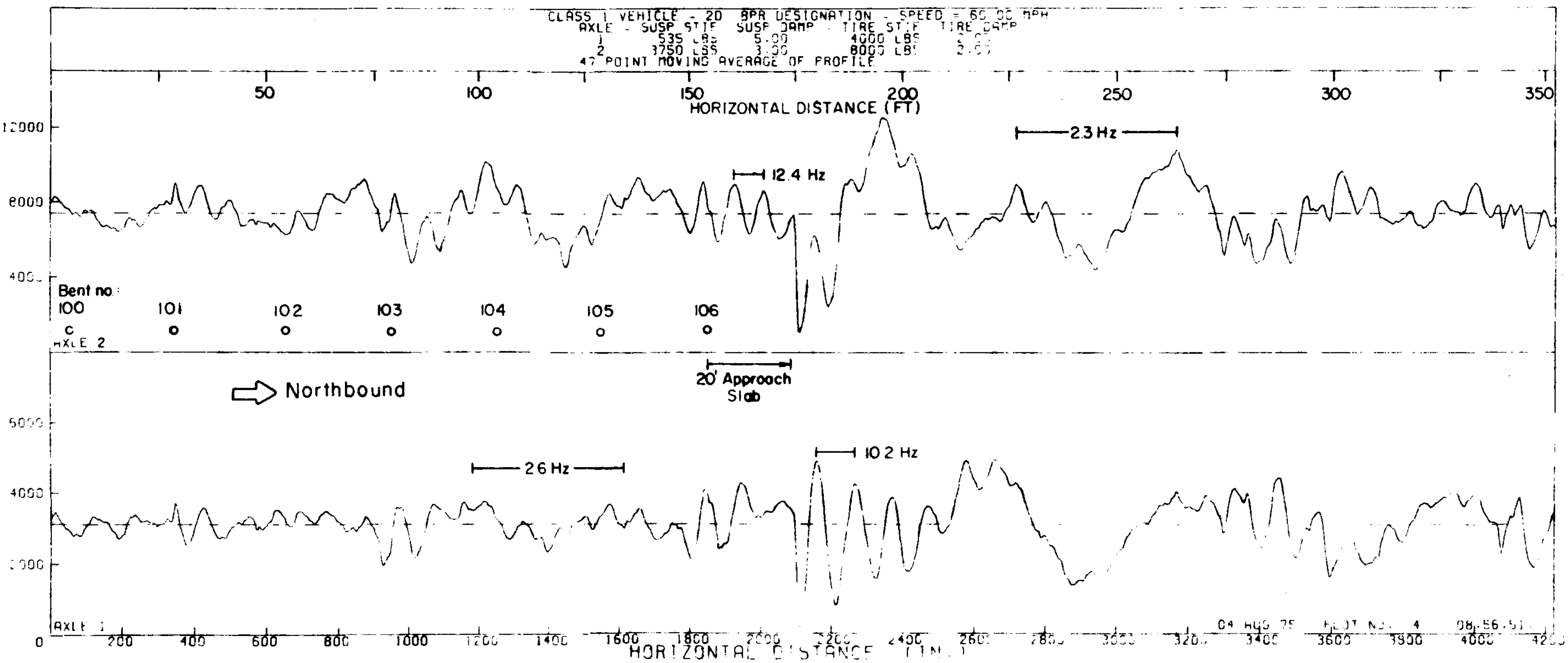


Fig 14. Left wheel forces on outside lane, northbound bridge, bent no.'s 100 to 106, 60 mph.

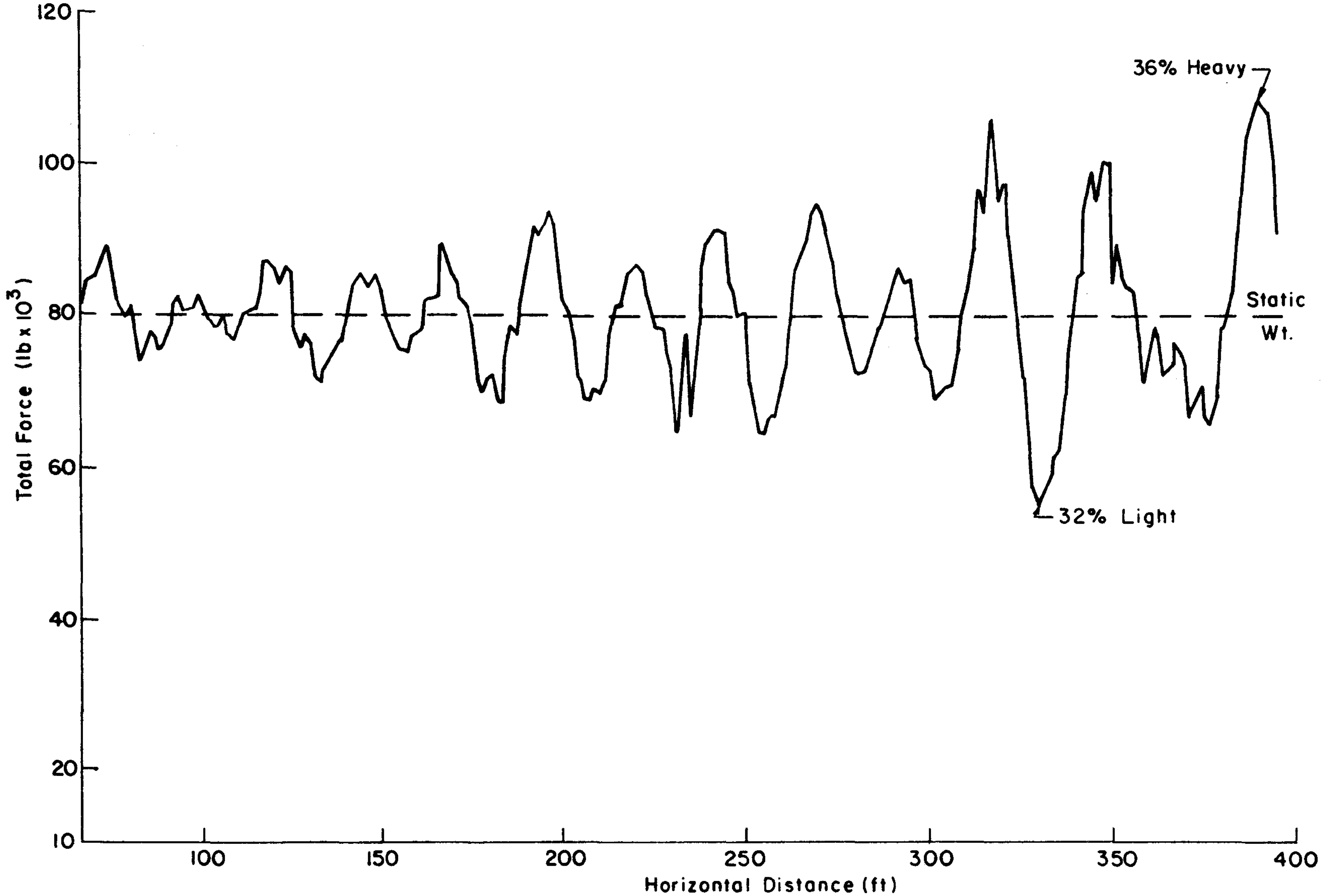


Fig 15. Total force exerted on bridge deck by 3S-2 vehicle traveling 50 mph over profile of Fig 6.

REFERENCES

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3. Lee, Clyde E., and Randy Machemehl, "Speed-Zoning to Reduce Dynamic Loads on the Port Isabel Causeway," Center for Highway Research, The University of Texas at Austin, March 1974.

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APPENDIX A

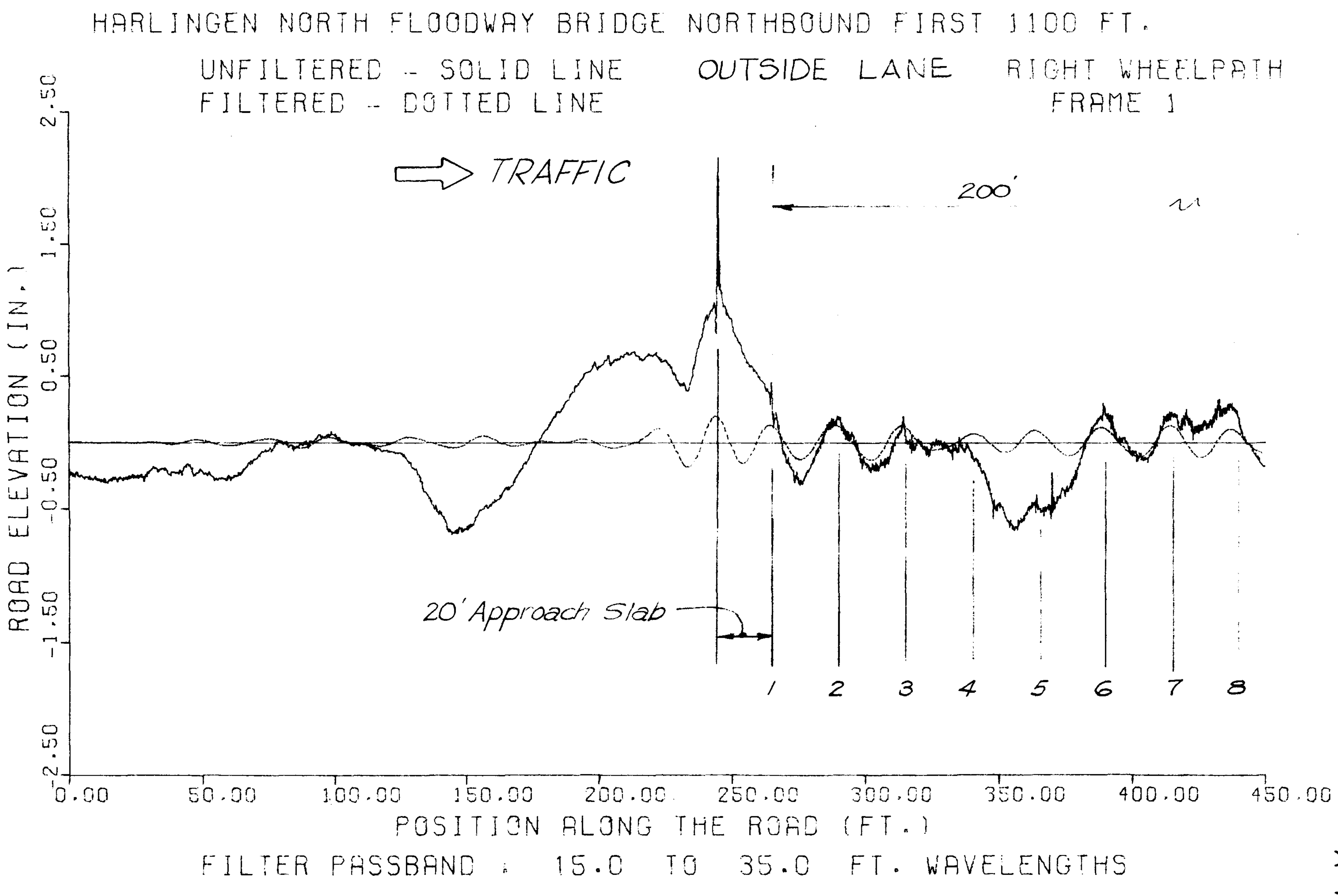
APPENDIX A

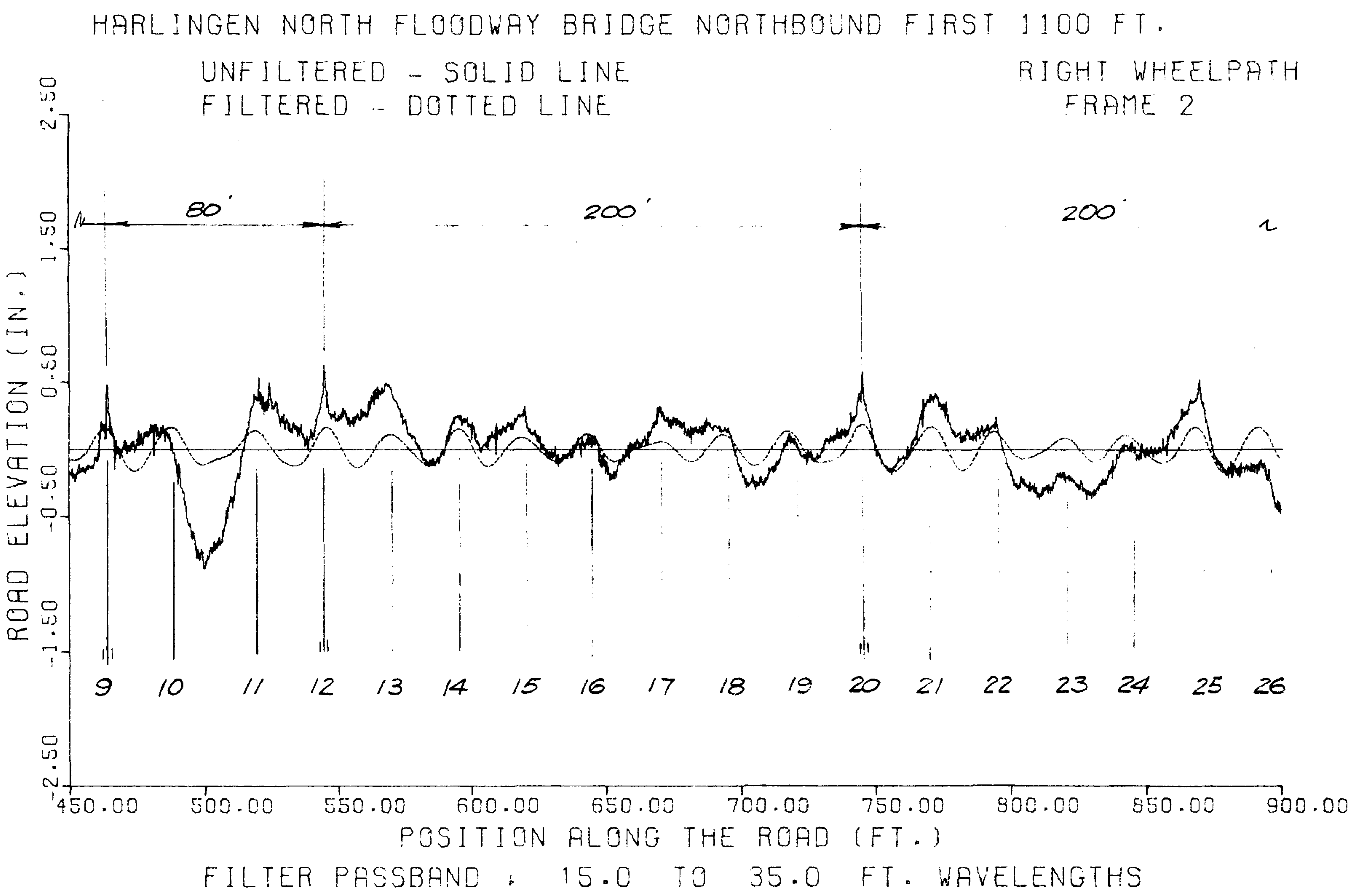
The General Motors Road Surface Dynamics Profilometer operating under Center for Highway Research Study No. 3-8-71-156 was operated at a speed of 20 mph over the North Floodway bridges near Harlingen, Texas on 9 July 1975 to obtain the information presented herein. The analog records that were recorded on magnetic tape were subsequently converted to digital form by frequent sampling so that digital computer programs could be used in analysis. The digitized raw data points have been plotted and connected by a solid line in the following figures. For analysis purposes, each bridge was considered in three 11,000-ft sections since standard computer programs were available to operate in this format. The data were further separated into shorter frames for convenience in presentation.

Most of the 1-1/4-inch-wide armored expansion joints show on the plots as sharp spikes since the small road-following wheel of the profilometer dropped into the joint under its 300-lb force. Approximate bent locations are shown in the figures along with the length of the slab units between expansion joints.

Because of certain limitation in the profilometer equipment, the road elevations are not absolute with respect to a horizontal plane. Relative elevations, however, over distances less than about 100 ft are fairly accurate. Patterns of roughness can thus be determined and allowances made for the fact that longer waves in the profile are not included precisely in the raw data.

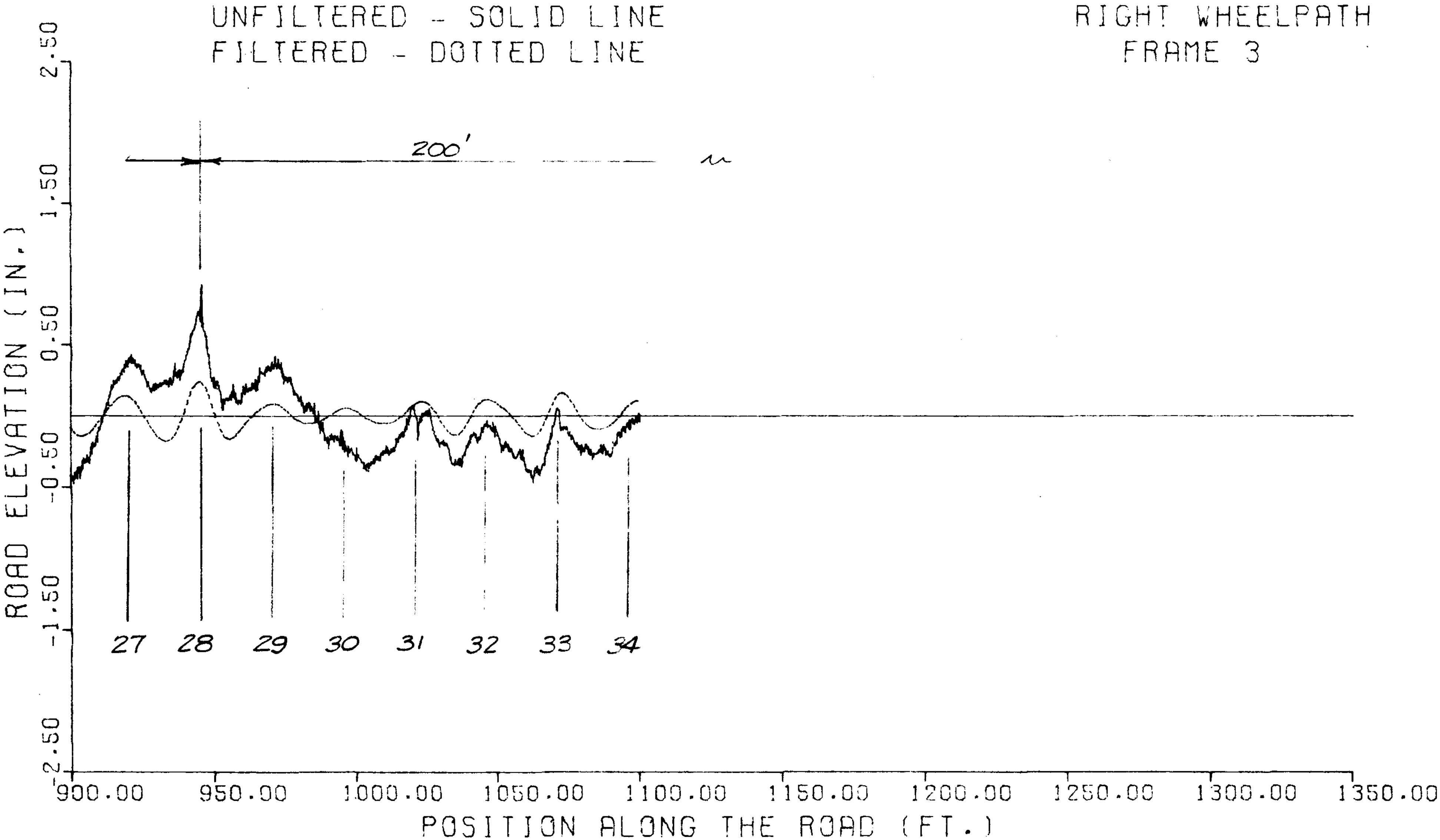
A special digital filtering program was used on the bridge profile data to determine a running average of the amplitude of wavelengths between 15 and 35 ft. The dotted line in the figures shows the amplitude of the waves in this range. A large portion of the total roughness of the bridge decks is accounted for by these wavelengths, and a definite pattern of repeating waves exists.


HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND FIRST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
OUTSIDE LANE
RIGHT WHEELPATH
FRAME 1
TRAFFIC
200'
20' Approach Slab
1
2
3
4
5
6
7
8
ROAD ELEVATION (IN.)
2.50
1.50
0.50
-0.50
-1.50
-2.50
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
400.00
450.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS



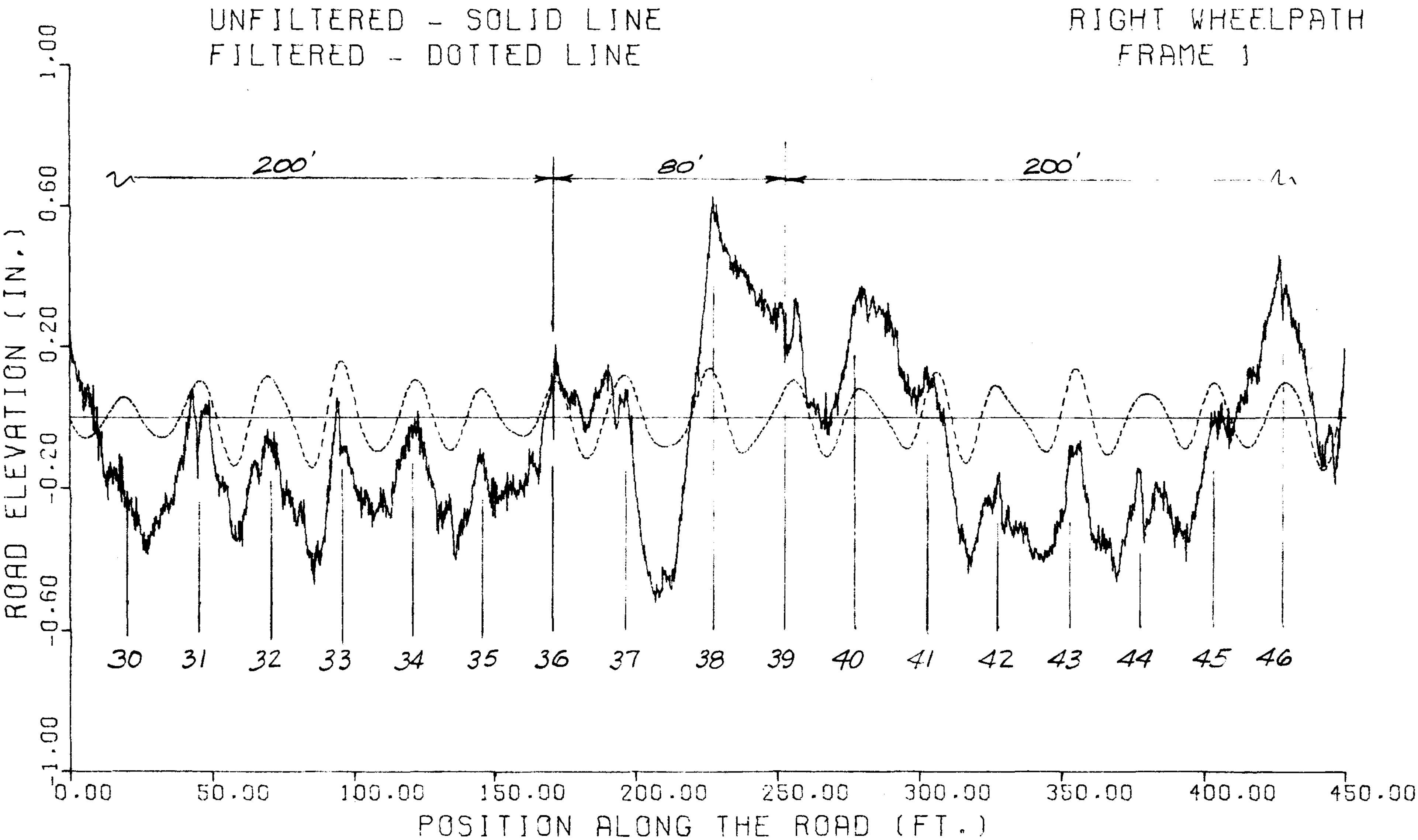
HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND FIRST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 2
80'
200'
200'
ROAD ELEVATION (IN.)
2.50
1.50
0.50
-0.50
-1.50
-2.50
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
450.00
500.00
550.00
600.00
650.00
700.00
750.00
800.00
850.00
900.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND FIRST 1100 FT.

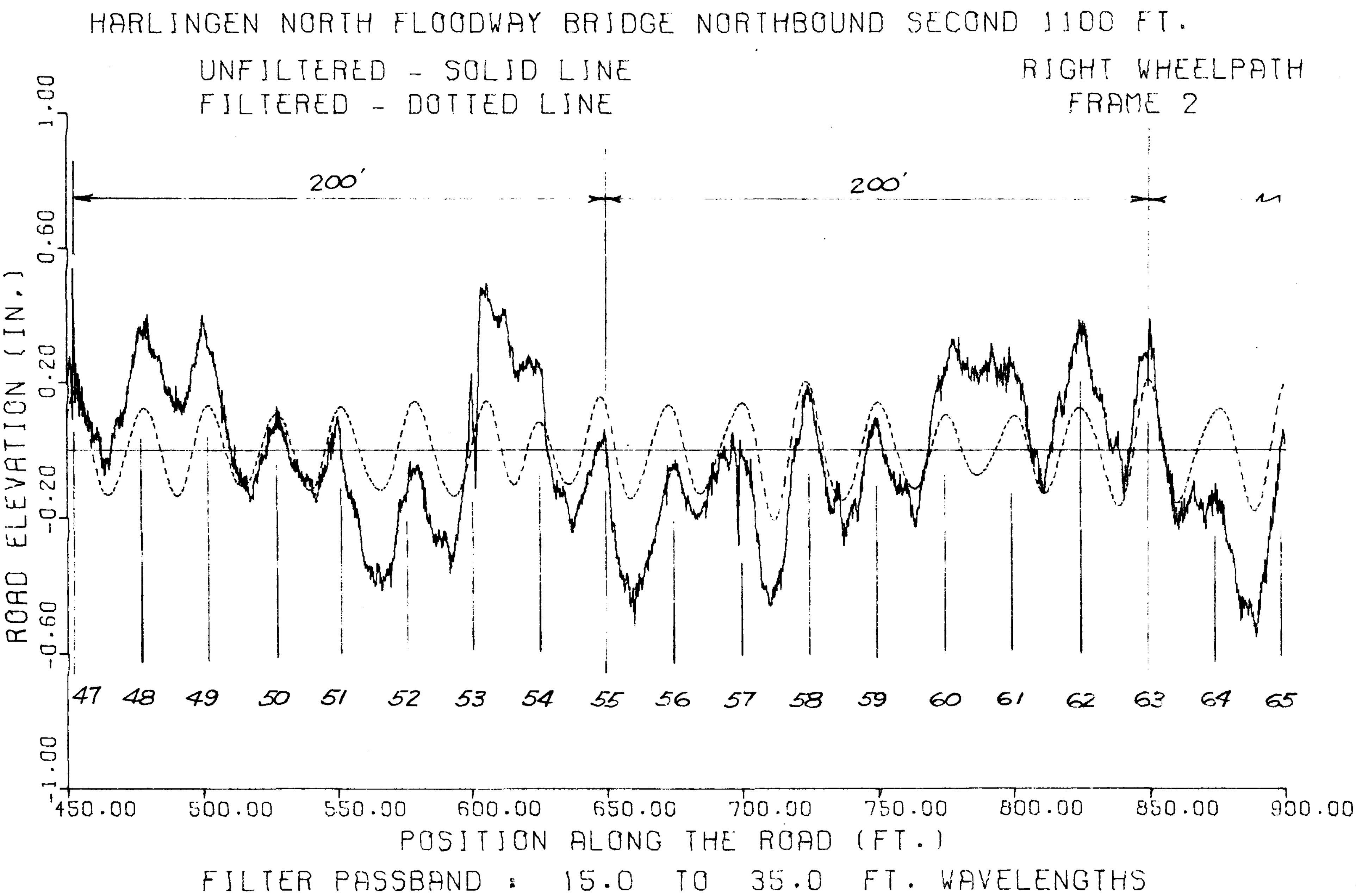


FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

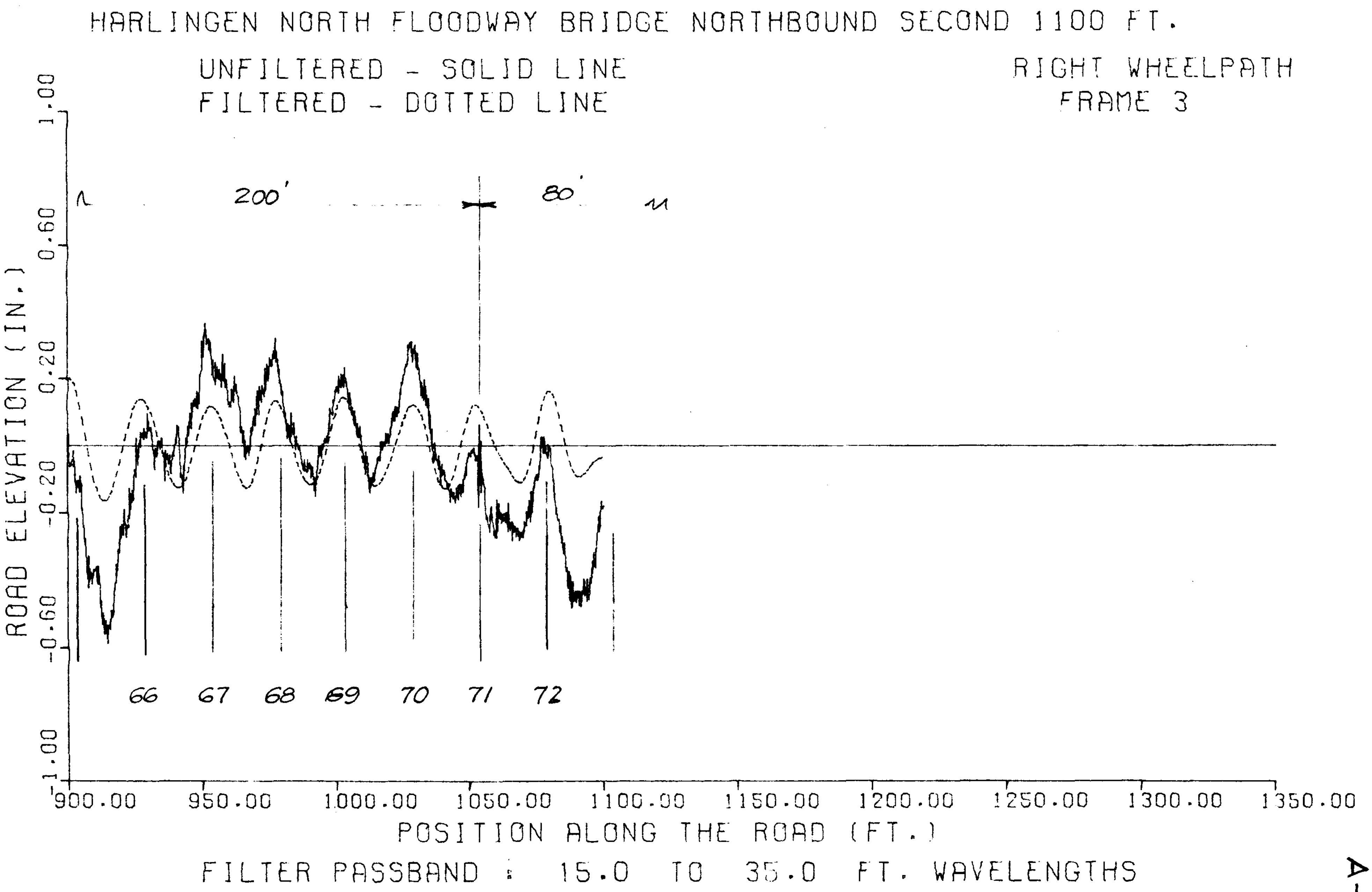
HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND SECOND 1100 FT.



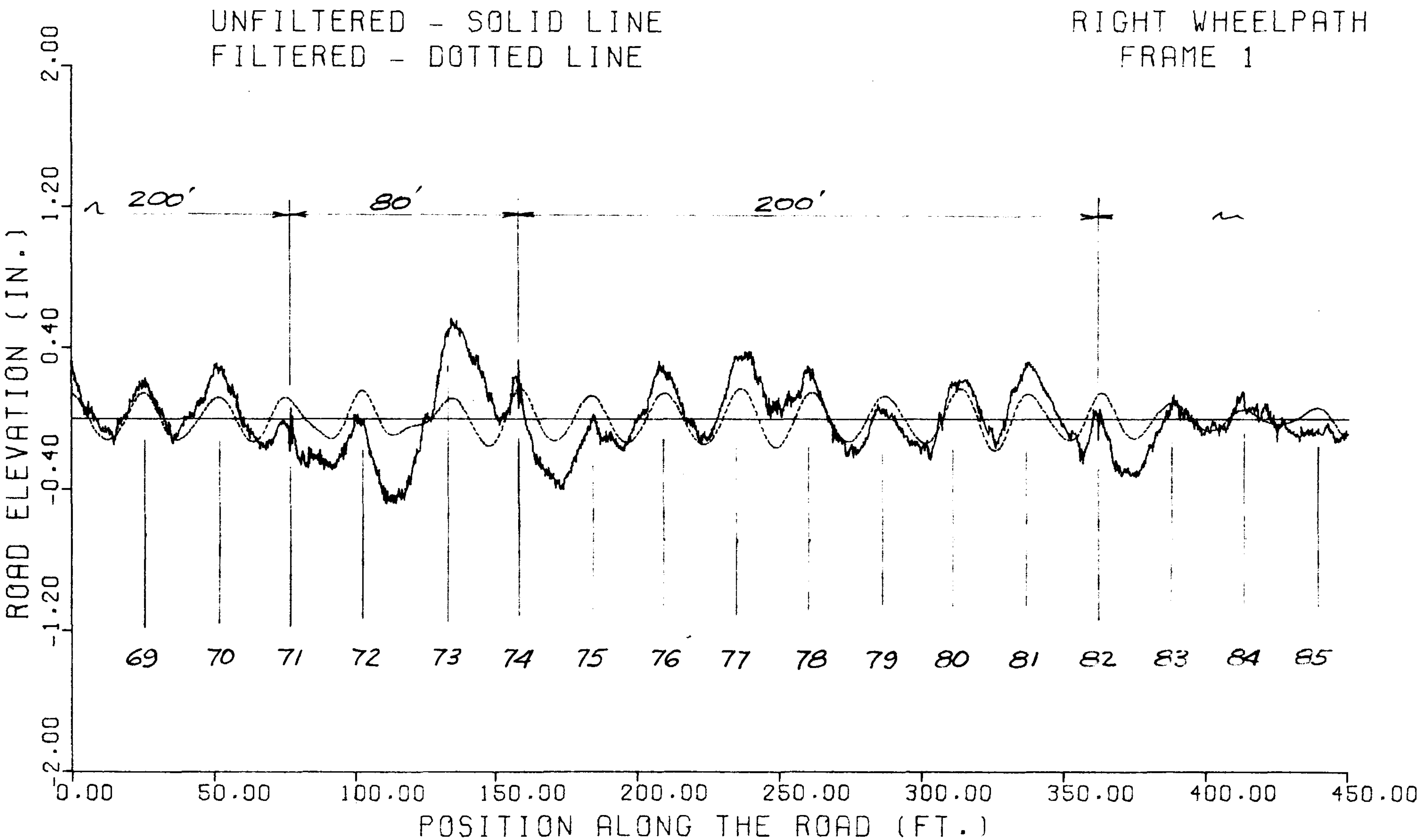
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS



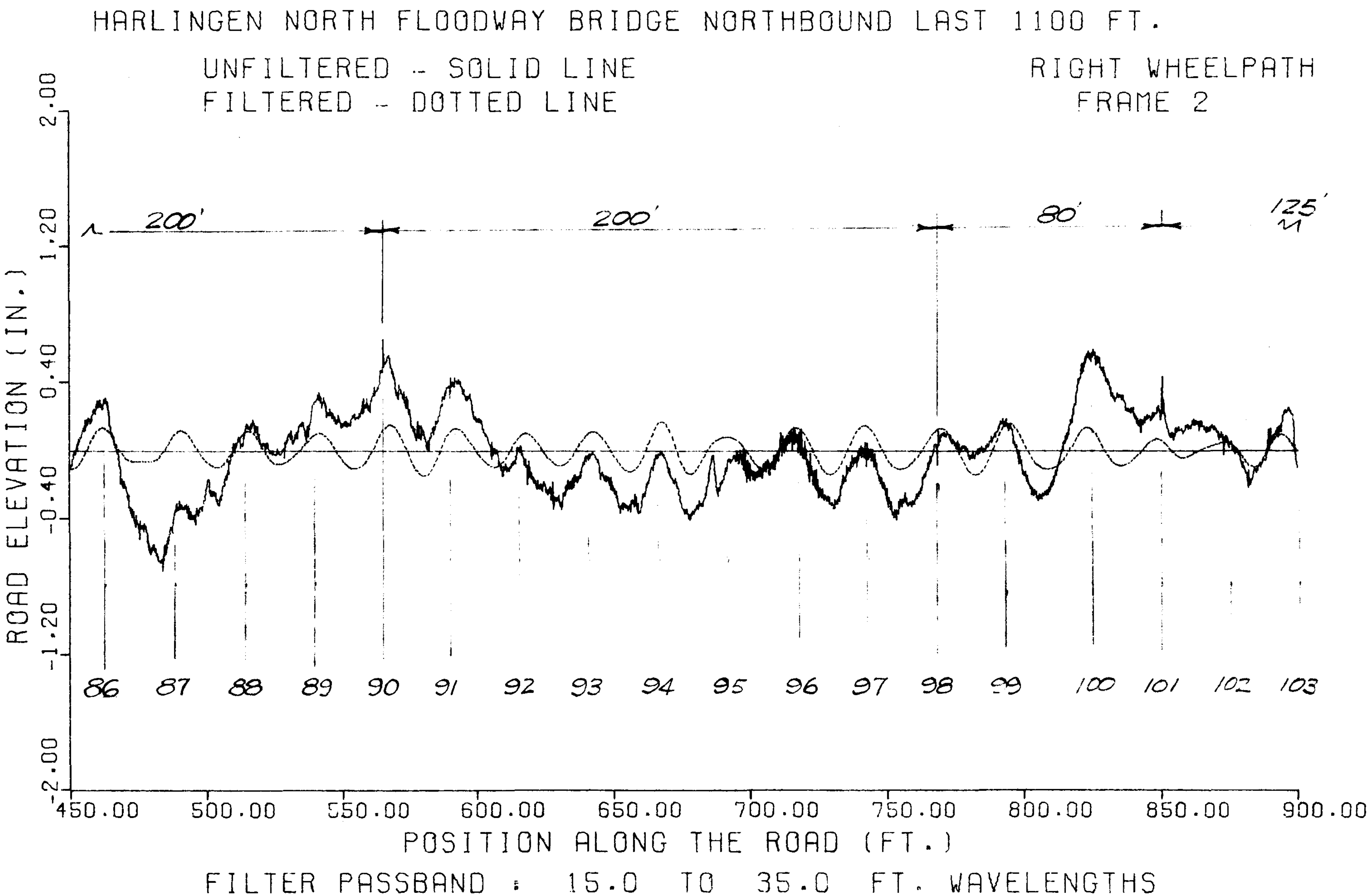
HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND SECOND 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 2
200'
200'
ROAD ELEVATION (IN.)
1.00
0.60
0.20
-0.20
-0.60
-1.00
47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65
450.00 500.00 550.00 600.00 650.00 700.00 750.00 800.00 850.00 900.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS


HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND SECOND 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 3
ROAD ELEVATION (IN.)
1.00
0.60
0.20
-0.20
-0.60
-1.00
200'
80'
66
67
68
69
70
71
72
900.00
950.00
1000.00
1050.00
1100.00
1150.00
1200.00
1250.00
1300.00
1350.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

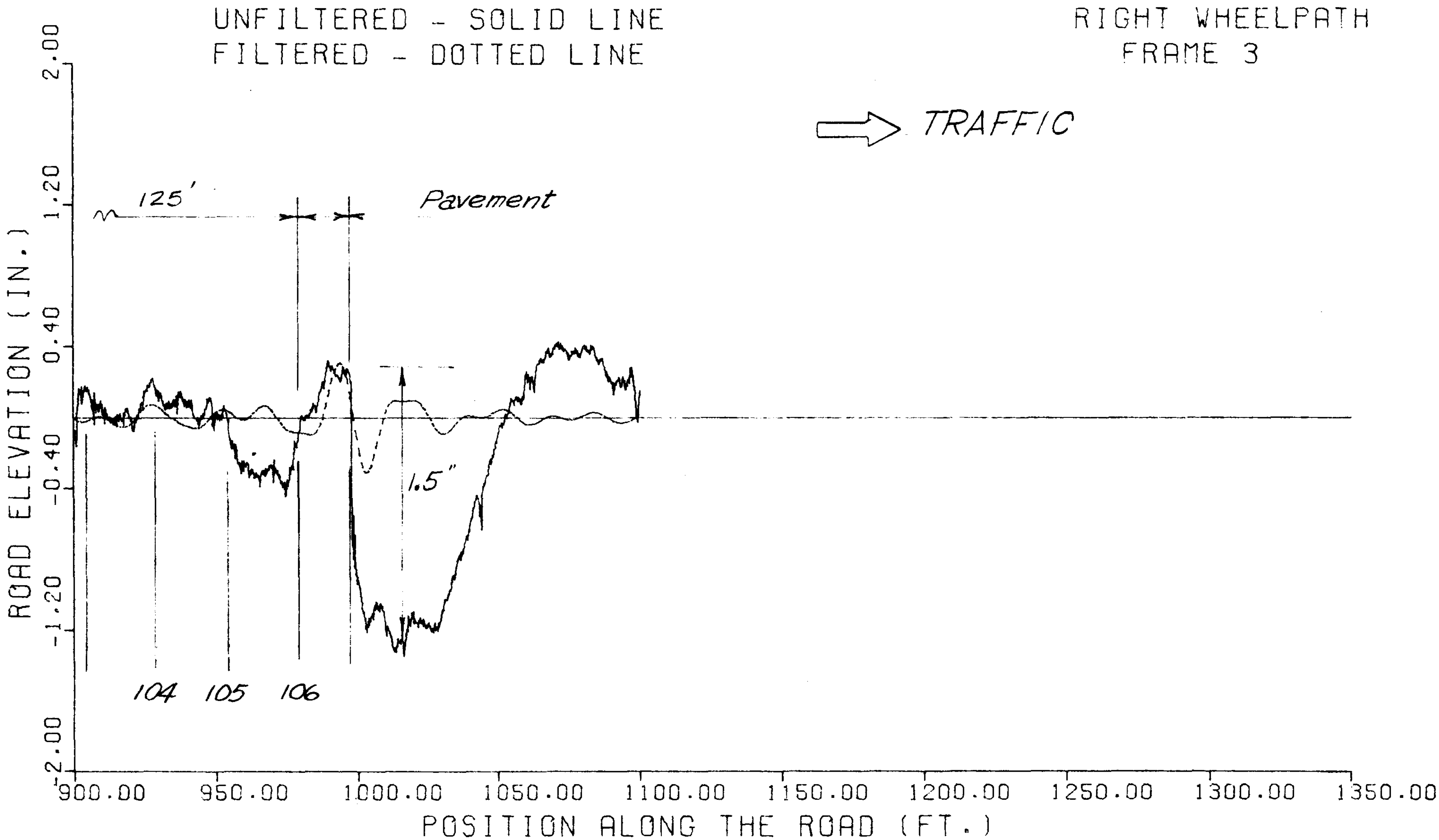
HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND LAST 1100 FT.



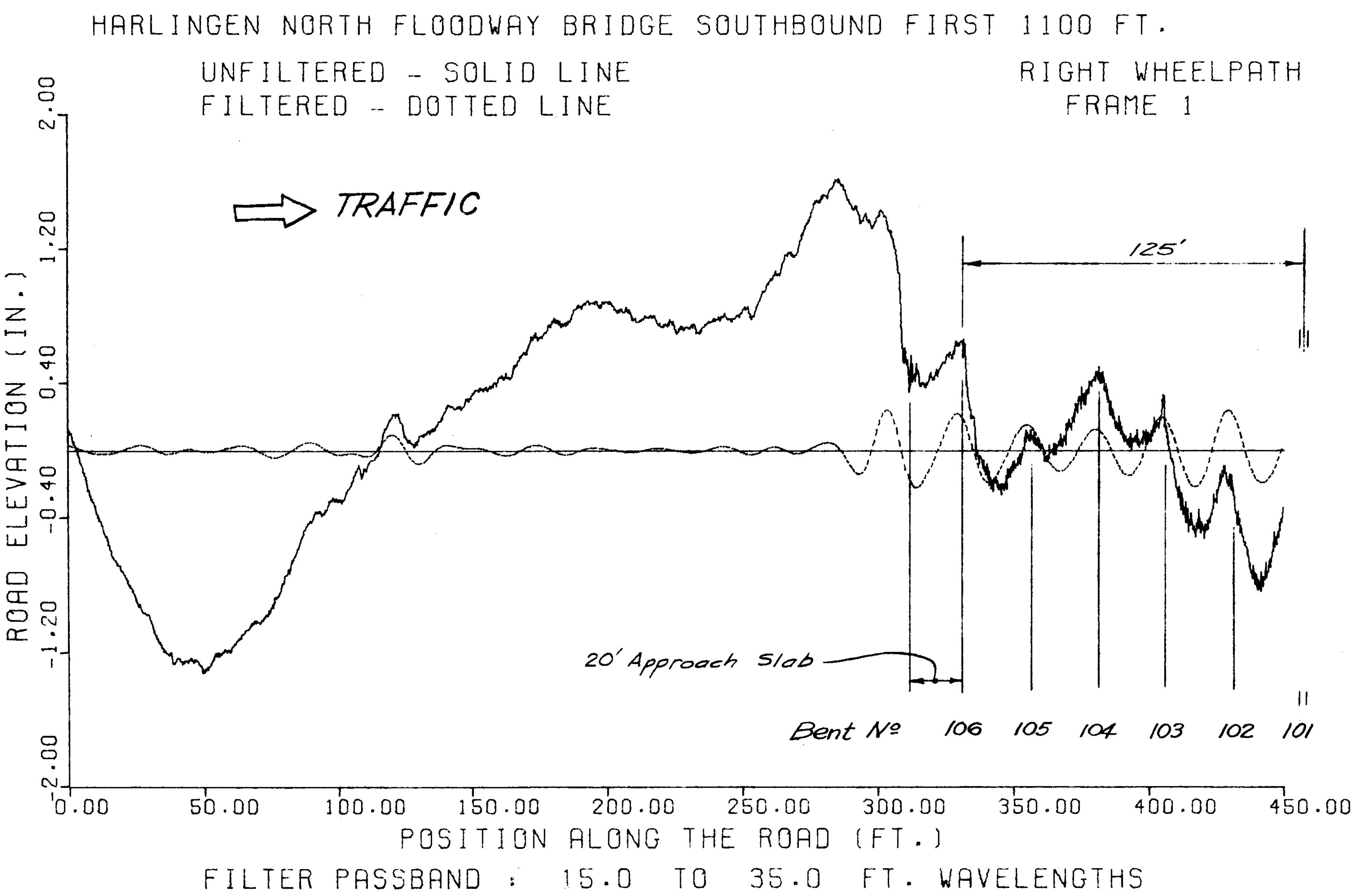
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS


HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND LAST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 2
ROAD ELEVATION (IN.)
2.00
1.20
0.40
-0.40
-1.20
-2.00
200'
200'
80'
125'
86
87
88
89
90
91
92
93
94
95
96
97
98
99
100
101
102
103
450.00
500.00
550.00
600.00
650.00
700.00
750.00
800.00
850.00
900.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

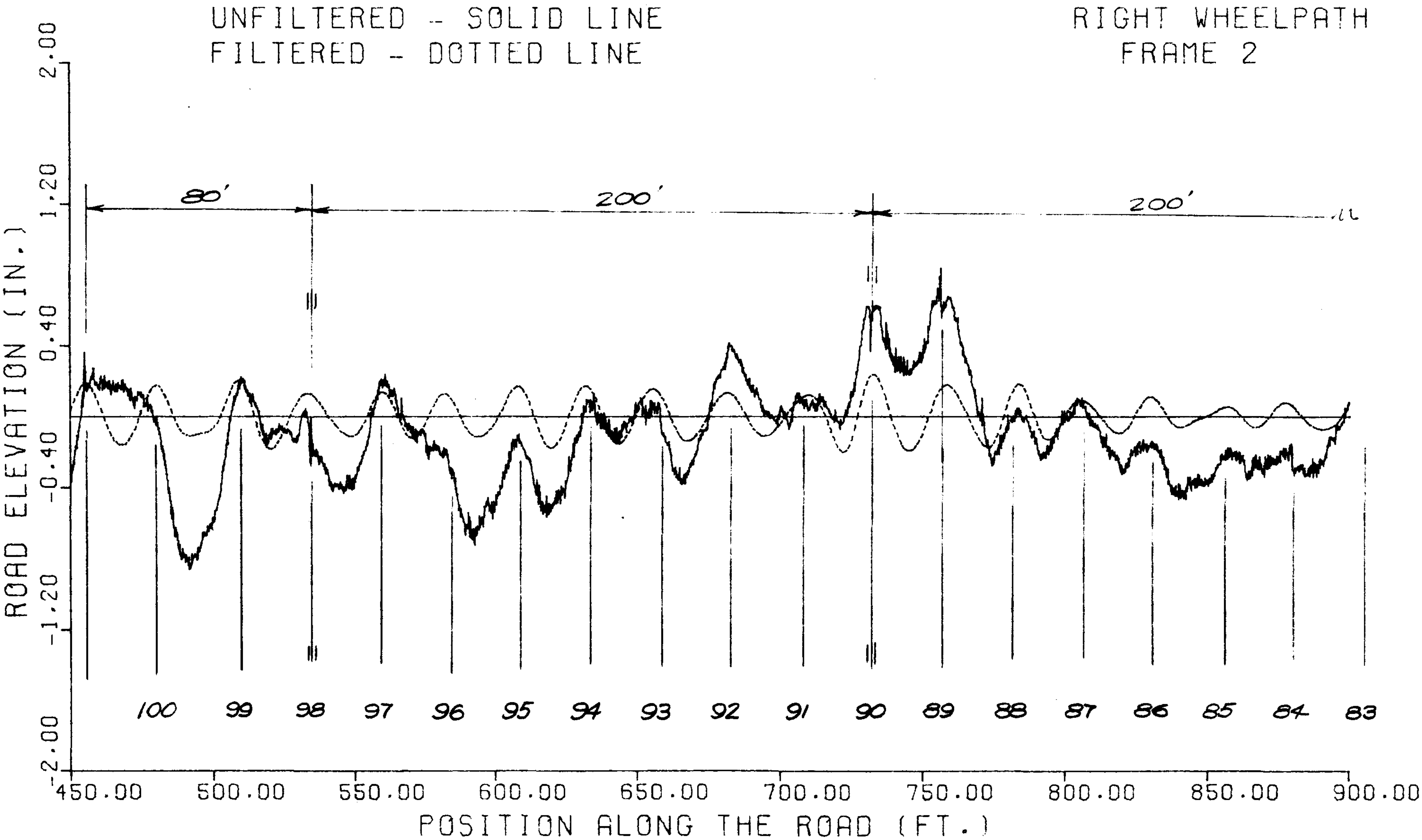
HARLINGEN NORTH FLOODWAY BRIDGE NORTHBOUND LAST 1100 FT.



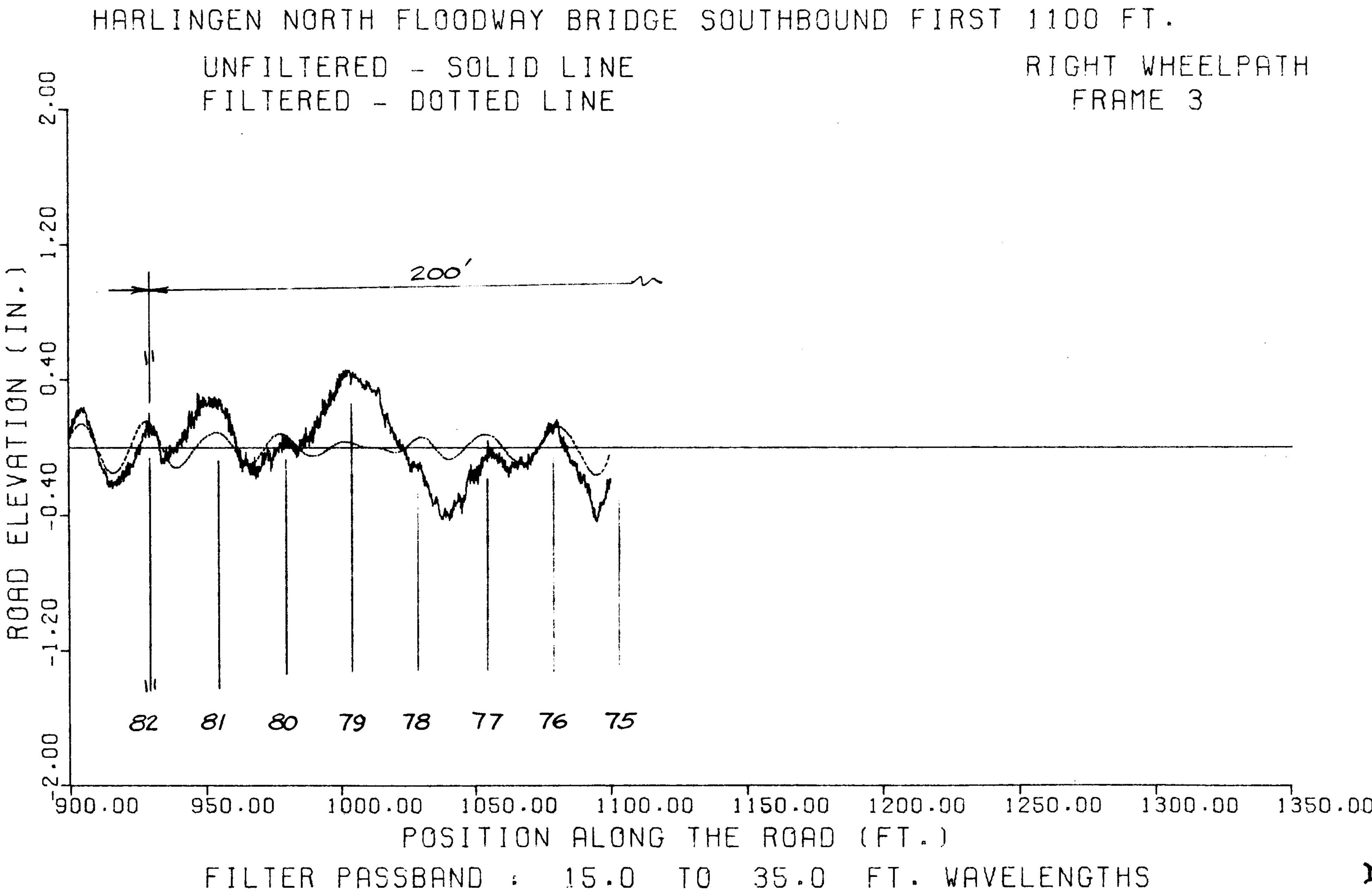
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS


HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND FIRST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED -- DOTTED LINE
RIGHT WHEELPATH
FRAME 1
TRAFFIC
125'
20' Approach Slab
Bent № 106 105 104 103 102 101
ROAD ELEVATION (IN.)
2.00
1.20
0.40
-0.40
-1.20
-2.00
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
400.00
450.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND FIRST 1100 FT.

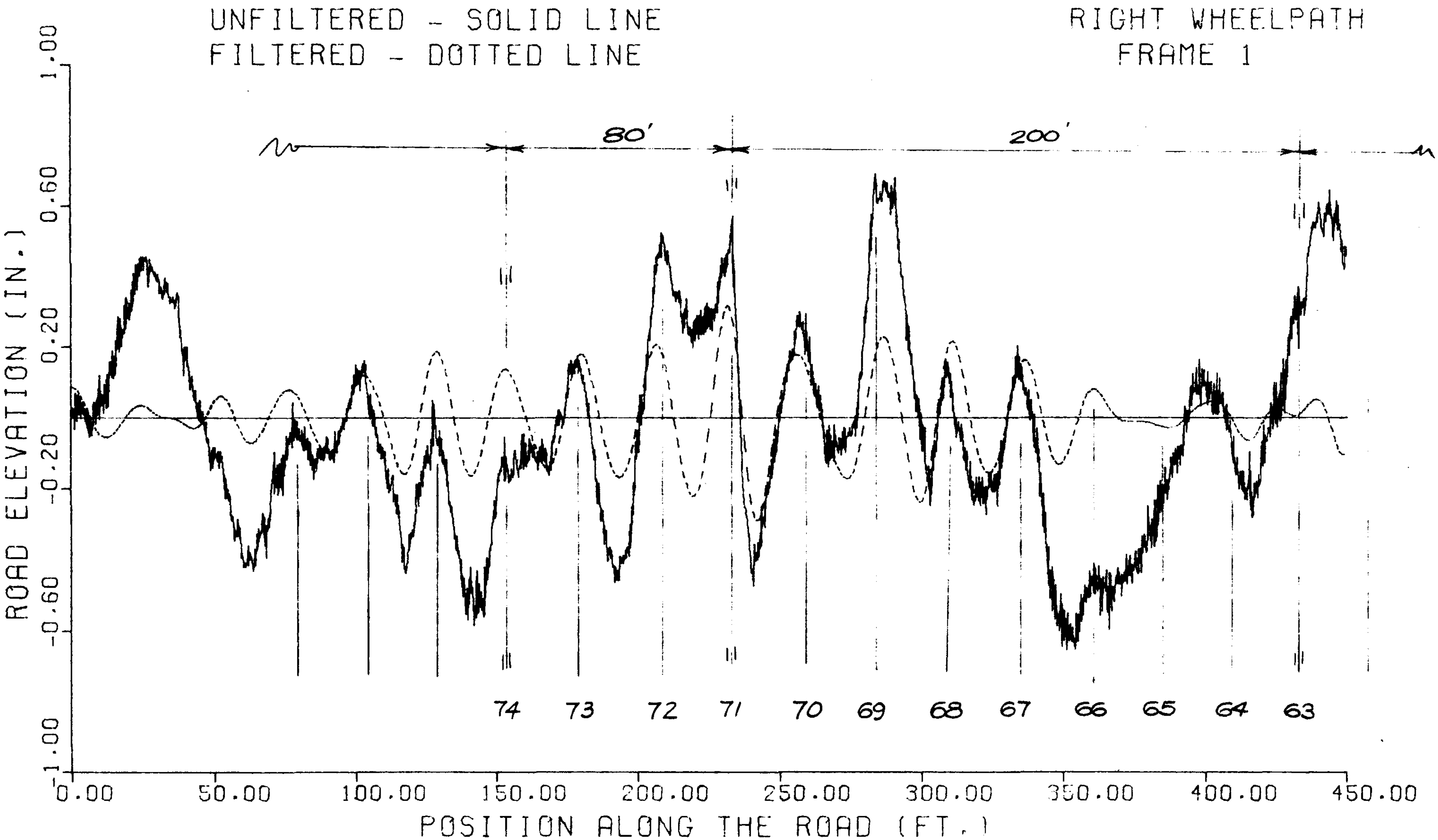


FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS



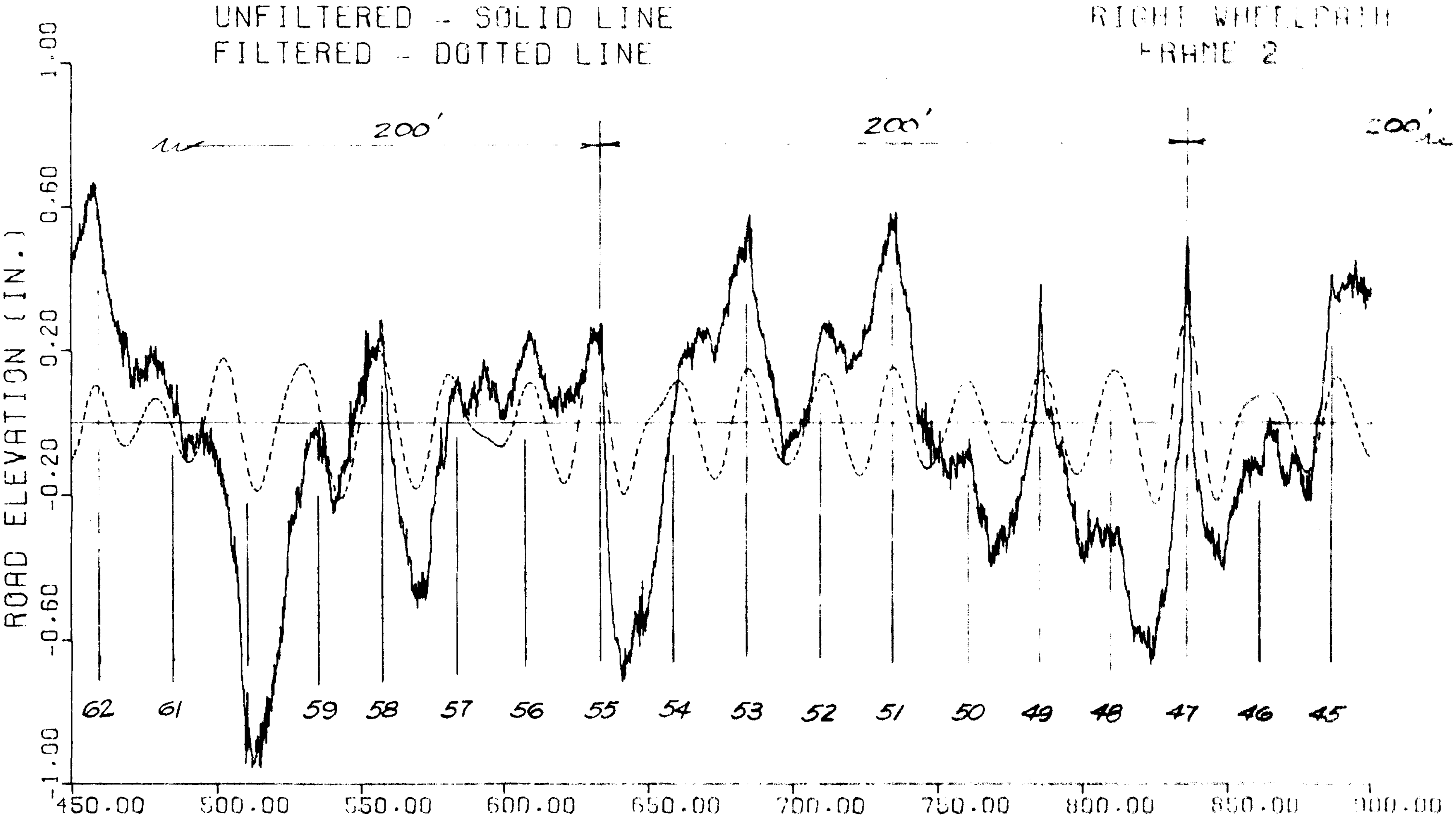
HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND FIRST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 3
ROAD ELEVATION (IN.)
2.00
1.20
0.40
-0.40
-1.20
-2.00
200'
82
81
80
79
78
77
76
75
900.00
950.00
1000.00
1050.00
1100.00
1150.00
1200.00
1250.00
1300.00
1350.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND SECOND 1100 FT.

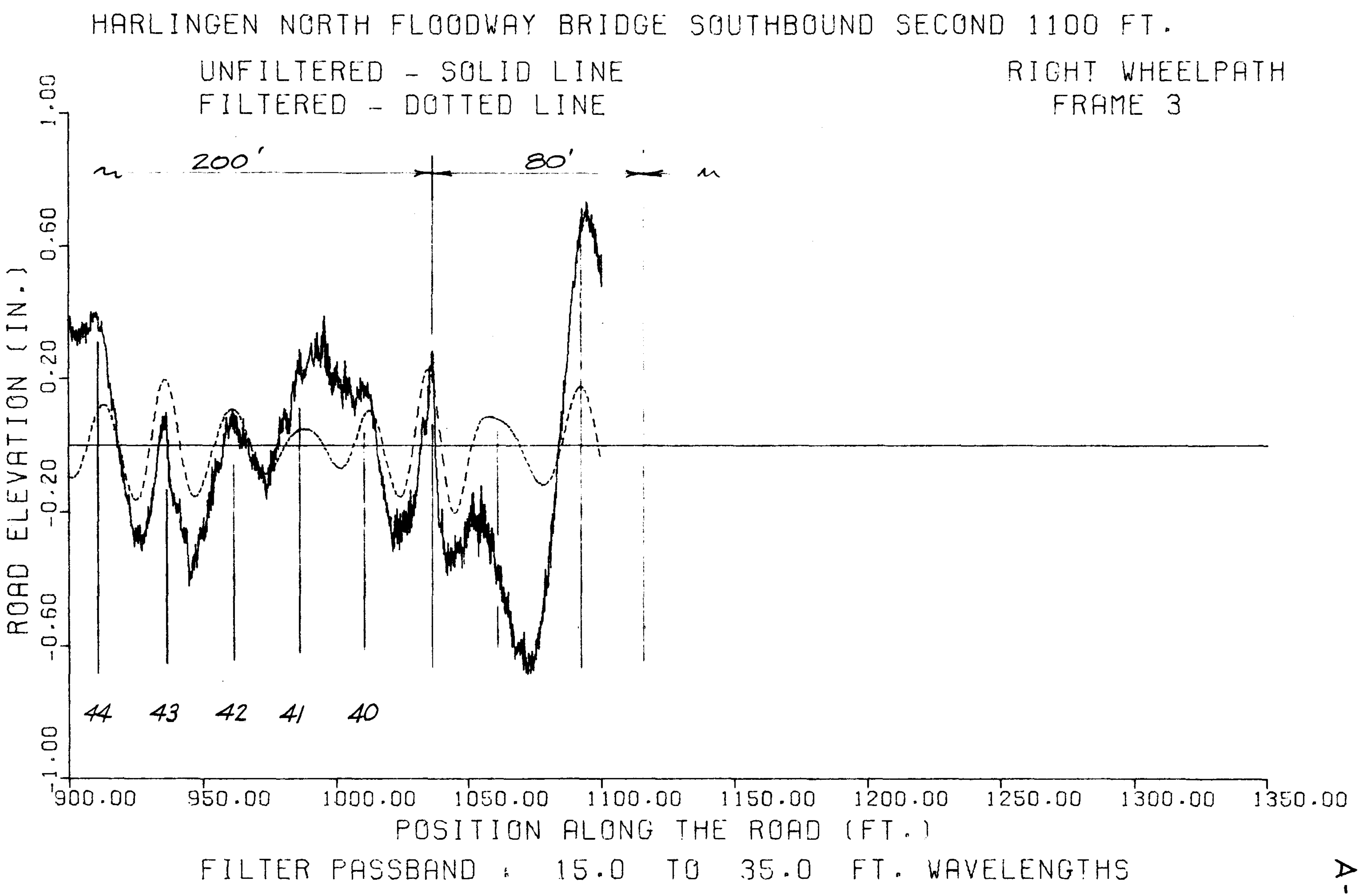


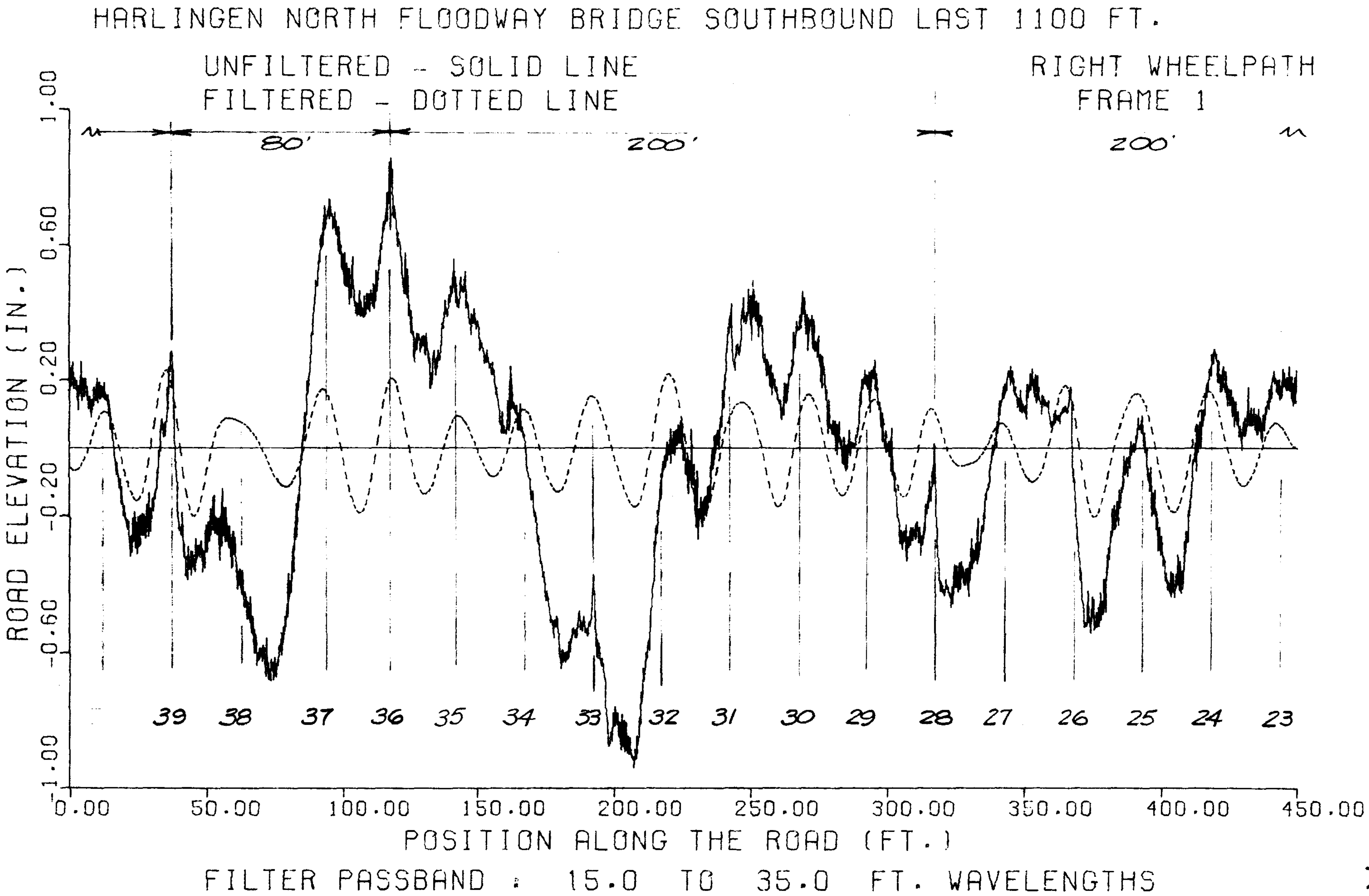
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND SECOND 1100 FT.

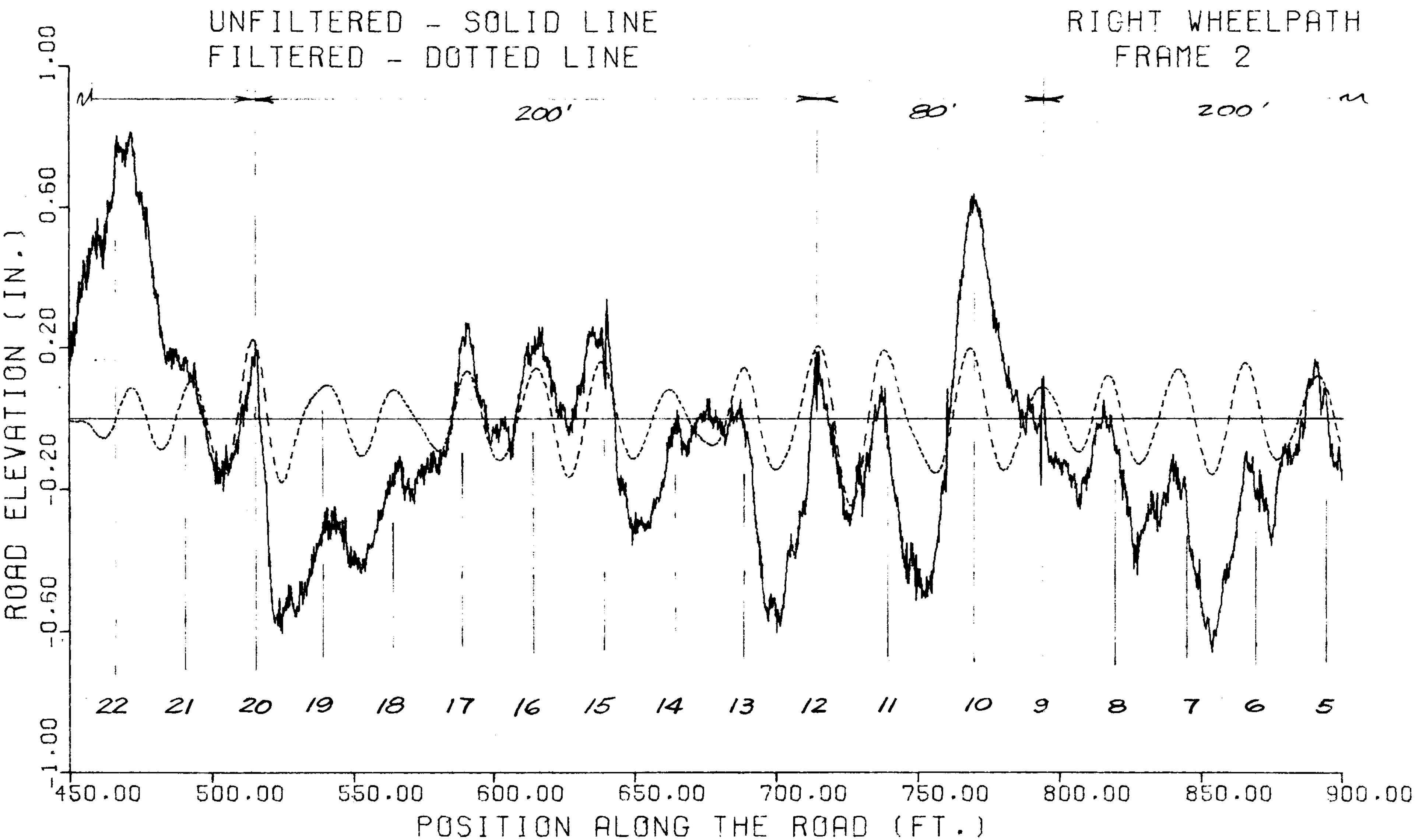


FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

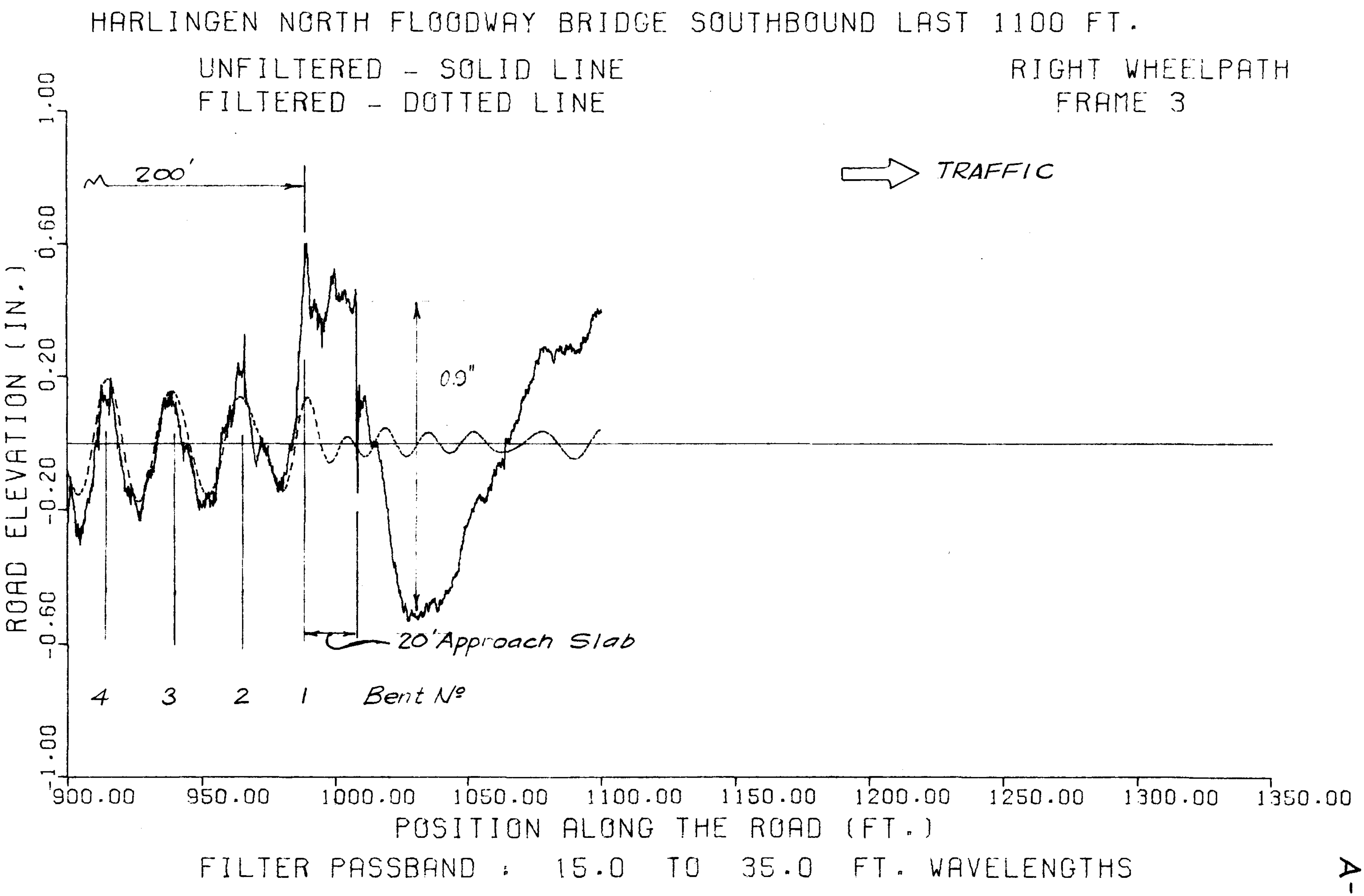

HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND SECOND 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 3
200'
80'
44
43
42
41
40
ROAD ELEVATION (IN.)
1.00
0.60
0.20
-0.20
-0.60
-1.00
900.00
950.00
1000.00
1050.00
1100.00
1150.00
1200.00
1250.00
1300.00
1350.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS


HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND LAST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 1
80'
200'
200'
ROAD ELEVATION (IN.)
1.00
0.60
0.20
-0.20
-0.60
-1.00
39 38 37 36 35 34 33 32 31 30 29 28 27 26 25 24 23
0.00 50.00 100.00 150.00 200.00 250.00 300.00 350.00 400.00 450.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND LAST 1100 FT.



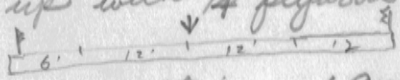
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS


HARLINGEN NORTH FLOODWAY BRIDGE SOUTHBOUND LAST 1100 FT.
UNFILTERED - SOLID LINE
FILTERED - DOTTED LINE
RIGHT WHEELPATH
FRAME 3
200'
TRAFFIC
0.9"
20' Approach Slab
4
3
2
1
Bent №
ROAD ELEVATION (IN.)
1.00
0.60
0.20
-0.20
-0.60
-1.00
900.00
950.00
1000.00
1050.00
1100.00
1150.00
1200.00
1250.00
1300.00
1350.00
POSITION ALONG THE ROAD (FT.)
FILTER PASSBAND : 15.0 TO 35.0 FT. WAVELENGTHS

Wed 23 Jun 76

0720 - Leon Snider, David Johnson, & CEL departed Austin in profilometer.
1130 - Lunch @ Robstown/Corpus Christi
1400 - Arrived at North Floodway bridges - Lupe Camargo, Jack Trammel, et al.

(a) a seal coat (AC 5) had been applied full width on both bridges on Fri 18 Jun 76; some bleeding in outside lanes by Wednesday - aggregate & brooming.

(b) string line (nylon) stretched tight between high points over bents and blocked up with 1/4" plywood blocks at mid span low points; 

Type F Mix.

(c) old Barber Greene machine sensing string w/ automatic control laid 16 ft "level-up" on NB inside lane Wed. A.M.; steel 3-wheel roller + pneumatic.

(d) new Barber Greene w/ 34' 3-section ski laid outside NB w/ automatic control; rolled.

(e) new Barber Greene machine sensing string w/ auto control laid 16 ft "level-up" on SB inside lane Wed P.M.

(f) new machine w/ 3-section ski on inside "level-up" was used for 16 ft outside lane level-up, but manual over-ride was attempted to fill low spots marked by string-line crew after seeing profilometer chart.

(g) profilometer charts on both lanes NB were provided by 1530 hrs. old machine used to dump mix in piles on outside lane NB & blade came in to spread mix where marked by string-line crew based on profilometer charts (blade began at 1830 hrs about Bent #37); second "level-up" w/ new machine up to Bent #37 using ski & manual control at selected spots.

(h) blade completed spot leveling NB outside, then inside SB, inside NB, outside SB. profiles after first blade work.

(j) more blade work Thur AM early

Type D Mix

(k) new machine using 2 wheels on booms w/ wire ≈40' long began 18 ft pass going SB on NB inside lane & 6' shoulder about 1000 hrs against traffic to save turn around time (wheels near center of bridge, auto control + slope 2%)

(l) after lunch new machine traveled around and began 18' inside lanes going w/ traffic (wire on rt side of machine) - profiled, finished NB inside after lunch)

(m) changed machine width to 11'-6" by about 1630 hrs and laid NB outside w/ joint tracking controller + slope; ran profile of SB inside + 1/2 mile of pavement about 1730 hrs, while machine worked NB outside lane

(n) machine completed SB outside 11'-6" and about half the outside SB 11'-6" shoulder Thur evening by 2000 hrs.

(o) machine laid NB outside shoulder & finished SB outside shoulder Fri AM. - profiles of SB outside lane & NB outside lane 0800 → 0900 Fri (NOPE ←)

(p) profile extended about 1/2 mi south of bridge on SB outside lane

(q) profiled about 1 mile on U.S. 77 in inside lane just north of Raymondville to show very smooth flexible pavement

WED 23 JUN 76

TBI-X

(a) seal coat on Fri 18 Jun 76

1530 hrs. ① S end of SB after level up. (b) level up on string NB old machine } 16' left side A.M.

SB new machine }

② "

(3) view of SB bridge looking N. (c) ski on first 16' pass w/ new machine P.M. { steel roller

also 16' { pneumatic

(4) " " NB " " "

from Bent 9 after second pass level up

(5) tearing & 3' from rt edge second level @ Bnt 30 (close up)

(6) " corrected after bent 36

1845 (7) bladed patch Bents# 37-38 outside NB after machine level (profilometer)

EJOIRE HOYERE FORSAGGE - CONTRACTOR PARTNER

RALPH