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A WIND TUNNEL STUDY OF THE AERODYNAMIC STABILITY OF THE PROPOSED PAPINEAU BRIDGE

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A WIND TUNNEL STUDY OF THE AERODYNAMIC STABILITY OF THE PROPOSED PAPINEAU BRIDGE

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NOTATION

D depth of bridge, ft.

J moment of inertia per unit length, slugs-ft.

m mass per unit length, slugs/ft.

n frequency, c.p.s.

N natural frequency, cps

R Reynolds Number

V velocity, ft/sec.

Y vibration amplitude, ft.

α wind angle

δ damping logarithmic decrement

ν air viscosity, ft^2/sec.

ρ air density, slugs/ft^3

Subscripts

F flexure

F.S. full-scale

M model scale

T torsion

1.0 INTRODUCTION

The proposed bridge is to cross the Riviere des Prairies joining the City of Montreal to the City of Laval. The bridge is of the cable-stayed type with an orthotropic deck. It has six traffic lanes, a width of 92 ft., a centre span of 790 ft. and two side spans of 295 ft. The bridge is shown in Fig. 1 and Fig. 2 and the model wind tunnel installation is shown in Fig. 3. The model was constructed in the Low Speed Aerodynamics Section workshop to a scale of 1:48 from a drawing supplied by Gendron, Lefebvre and Associes.

The tests were conducted in a 7 ft. x 7 ft. closed working section. The model was a rigid sectional model corresponding in length to 336 ft. of the bridge centre span. It was suspended across the working section by an external mounting system in which helical springs were used to restrain torsional and vertical flexural motions. Electromagnetic damping was used to augment the natural damping of the suspension system when required. The suspension system is described in Fig. 4. The dynamic characteristics of the model were scaled from data for the full-scale bridge that were provided by the customer.

The tests were conducted with and without a groundboard. The groundboard was installed at the equivalent of 25 feet below the bridge to simulate the height above water of the proposed installation. Some tests were conducted at other bridge heights to determine the importance of this variable.

In order to assess the effect of snow piled against the side hand rails, tests were done with the hand rails blocked by tape.

The model was tested over a range of speeds corresponding to full-scale values from 0 to 125 m.p.h. and at several wind angles.

Measurements were made of the vibration amplitudes and frequencies.

Properties of the full-scale bridge are given below:

Deck width	92 ft.
Box width	34 ft.
Depth	12 ft.
Centre span	790 ft.
Weight/unit length	8900 lbs/ft.
Inertia/unit length	1.7×10^5 slugs ft.
Radius of gyration	24.6 ft.
N_F	0.519 c.p.s.
N_T	1.01 c.p.s.
N_T/N_F	1.95

2.0 DYNAMIC SIMILARITY

The use of a rigid "sectional" model, as opposed to a complete bridge model, is now commonly accepted* and the

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* D. E. J. Walshe, "The Use of Models to Predict the Oscillatory Behaviour of Suspension Bridges in Wind." Proceedings of Symposium on Wind Effects on Buildings and Structures, National Physical Laboratory, England 1963.

displacements measured in the wind tunnel represent the corresponding movements of the bridge at the point of maximum displacement. For a first mode vibration this would represent, therefore, the movement at mid-span.

In order to achieve correspondence between model and full-scale behaviour, it is necessary that certain conditions for dynamic similarity be satisfied. This requirement can be achieved if there is equality between model and full scale of the following non-dimensional parameters:

a) $\frac{V}{N_D D}$, $\frac{V}{N_T D}$

b) $\frac{J}{\rho D^4}$, $\frac{m}{\rho D^2}$

c) δ_F, δ_T

It is further required that the model centre of gravity and the axis of torsional movement correspond to the full-scale bridge. Finally, for correct simulation of coupled torsion-flexure, it is necessary to have the ratio N_F/N_T the same for model and full-scale.

When the foregoing conditions are satisfied, the equivalent full-scale velocities can be calculated on the basis of the equality of the dimensionless velocities, $\frac{V}{ND}$, thus

$$V_{F.S.} = V_M \cdot \frac{D_{F.S.}}{D_M} \cdot \frac{N_{F.S.}}{N_M}$$

The full-scale amplitude is obtained by multiplying the model amplitude by the linear scale. Thus

$$Y_{F.S.} = Y_M \cdot \frac{D_{F.S.}}{D_M}$$

Correct extrapolation to full-scale presupposes knowing a correct value of the damping of the full-scale bridge at zero wind velocity. This is a difficult property for the bridge engineer to predict and is undoubtedly a critical factor in predicting full-scale amplitude from wind tunnel test results.

3.0 AERODYNAMIC SIMILARITY

The model was tested in a steady uniform airstream whereas the real wind is a boundary layer wherein velocity increases with altitude and the flow is turbulent.

It is reasonable to ignore the velocity gradient since the change of velocity over the vertical depth of the bridge is small.

There will be some response of the bridge to the wind turbulence; however the energy of the turbulence will be mostly at frequencies considerably lower than the natural frequencies of the bridge. A typical value for the peak in the wind energy spectrum will be at a value of $n/V = 0.0005$ cycles/ft.* For a velocity of $V = 40$ ft/sec. (27.3 m.p.h.), the peak will occur at a frequency of 0.02 c.p.s. which is considerably lower than the bridge frequencies. Consequently, the wind energy at the bridge natural frequencies will be very small.

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*A. G. Davenport, "The Relationship of Wind Structure to Wind Loading". Vol. I, Proceedings of Symposium on Wind Effects on Buildings and Structures, National Physical Laboratory, England 1963.

It has been common practice to neglect these variables in wind tunnel testing of bridges. The primary goal of the tests is to determine the degree of aerodynamic stability of the bridge and it is unlikely that the presence of turbulence would worsen the stability. Excitation due to eddy shedding might, in fact, be reduced by turbulence.

Experience of other investigators is that findings in uniform steady flow wind tunnel tests can be extrapolated to full-scale with a reasonable degree of confidence.

The aerodynamic similarity parameter, the Reynolds Number, $R = \frac{VD}{\nu}$, does not duplicate full scale values in these tests. However, for sharp-edged shapes such as we are concerned with, the effects of the Reynolds Number will be minimized.

4.0 TEST PROCEDURE

Accelerometers were mounted on the suspension system to indicate torsional and flexural motions. The recorded outputs were analyzed to give frequency, mode and amplitude. The velocity was increased slowly from zero to a maximum speed of 125 m.p.h. (full-scale value) and recordings were made at velocities where significant motion was observed.

The natural frequencies of the model suspension were 7.5 c.p.s. in flexure and 14.6 c.p.s. in torsion which corresponds to 0.519 c.p.s. and 1.01 c.p.s. respectively for the full-scale bridge. Thus the ratio $\frac{N_{F.S.}}{N_M}$ is equal to 0.0692 and

$$V_{F.S.} = V_M \cdot \frac{48}{1} \cdot 0.0692 = 3.32\ V_M$$

$$Y_{F.S.} = 48\ Y_M$$

5.0 ACCURACY

The interpretation of amplitude from the accelerometer recordings was checked for accuracy by comparing results in six instances to amplitudes measured directly at the bridge model. It was found that the discrepancy between the two methods was consistently less than 15 percent.

6.0 PRESENTATION OF RESULTS

The results are presented as plots of full-scale amplitude against full-scale velocity in Figs. 5(a) to 5(r). On the figures the flexural motions are distinguished from torsional motions by using circles for flexural data points, diamonds for torsion and triangles for cases of combined flexure and torsion.

7.0 DISCUSSION OF RESULTS

7.1 Effect of Groundboard

Tests were done with the without the groundboard installed. It was found that the groundboard had the effect of increasing the amplitude of vibration. A typical effect is demonstrated by comparing Fig. 5(a) and Fig. 5(1) which are zero wind angle results. The amplitudes of Fig. 5(1) with the groundboard installed are several times those of Fig. 5(a) without the groundboard. This is in spite of a higher value of the damping logarithmic decrement. The velocities at which vibrations occur are unchanged by the presence of the groundboard.

The importance of the groundboard cannot be overstressed and it is reasonable to extrapolate to full-scale only those results that were obtained with the groundboard installed.

The need for a groundboard was not recognized until tests were done at small positive wind angles and consequently there is a significant amount of information available for tests done without the groundboard. However, this has been included for comparison and can be used to assess the effects of wind angle and damping.

The specified groundboard height of 25 ft. was tested under several conditions, (Figs. 5(l) to 5(r)) but some tests were done at other angles to determine the sensitivity to height. The effect of height on the peak flexural amplitude in the low velocity range 20 to 40 m.p.h. is shown in Fig. 6. The effect is less severe for the handrail blocked case.

7.2 Effect of Handrail Blockage

It was anticipated that snow piled against the handrails would have the effect of increasing the aerodynamic bluffness of the bridge and thus the wind-excited amplitudes. This condition was simulated by blocking the handrails with tape.

This was not significant in the low velocity range below 40 m.p.h. At high velocities the amplitudes were increased by about 50 percent in several instances. With the groundboard at 25 ft. and at zero wind angle, the handrail blockage at 80 m.p.h. increased amplitude by about 100 percent.

Generally this effect does not appear to be as serious as anticipated and except under very severe and unexpected wind conditions, it is scarcely significant.

7.3 Effect of Wind Angle

Under certain topographical situations the wind can have the effect of coming down on a bridge or blowing up from beneath. This effect was simulated by testing the bridge over a range of bridge angles between -2-1/2° and +5° where positive angles represent the case where the wind is blowing up from below.

It was found that the wind angle was insignificant in the low velocity range but could increase the amplitudes by as much as 300 percent at very high velocities. The wind angle did not have a serious effect on amplitude with the handrail blocked.

The particular topography at the bridge site is reasonably flat and wind angles other than zero would not be expected. This, together with the fact that it only affects amplitude at unusually high velocities, suggests that this is not a serious factor.

7.4 Effect of Damping

Without added damping from the electromagnets, the logarithmic decrement was about 0.04 in both torsion and flexure when the groundboard was out. The installation of the groundboard appeared to have the effect of increasing the logarithmic decrement to about 0.05.

Several tests were done with the electromagnets in use. The corresponding values of the logarithmic decrement were: $\delta_F = 0.06$ and $\delta_T = 0.07$ without the groundboard and $\delta_F = 0.08$ and $\delta_T = 0.1$ with the groundboard installed.

This range of damping had only a small effect on the amplitudes. The lowest values used are comparable to the lowest values expected for the full-scale bridge. If the completed bridge has higher values of damping, it would be expected that the amplitudes would be lower than predicted on the basis of these tests.

8.0 CONCLUSIONS

1. Both flexural and torsional oscillations were observed and under certain circumstances they occurred simultaneously. However, even at very high wind speeds (up to 120 m.p.h. full-scale) there was no evidence of the dangerous flutter or galloping vibrations where amplitudes can increase without limit. All vibrations resemble those that arise as a result of eddy shedding and are therefore amplitude limited.

2. Amplitudes greater than the full-scale equivalent of 2-1/2 in. were not observed except at high velocities (greater than 40 m.p.h.) for all tested conditions. These include the simulation of snow by blocking the handrail, wind angles from -2-1/2° to +5° and low damping values ($\delta = 0.04$).

For the case of the groundboard at 25 ft. and zero wind angle (Fig. 5(1)) amplitudes of 2 in. are exceeded only between 70 and 100 m.p.h. and reach in that range a value of 3-1/2 in. Under the same conditions but with the handrail blocked, the amplitude at low velocities remains below 2 in. but rise as high as 8 in. in the 70 to 100 m.p.h. range.

3. The groundboard generally increased the level of excitations of the bridge. Therefore amplitudes predicted from tests done with the groundboard installed at the equivalent of a 25 ft. full-scale height should be the most meaningful.

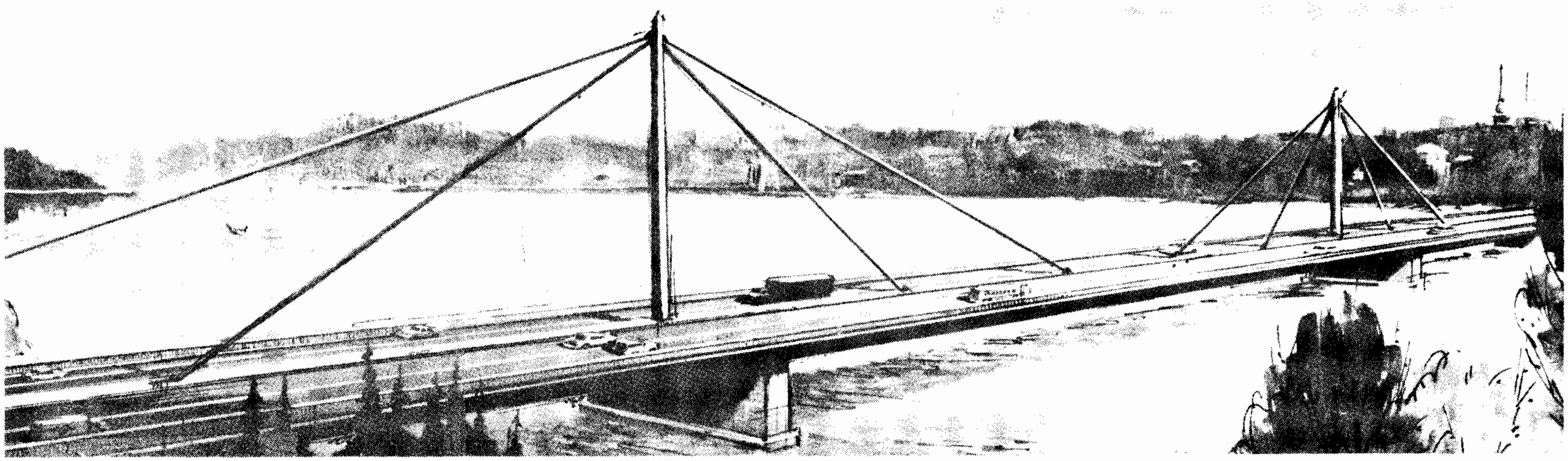
4. The simulations of snow piled against the handrail by blocking the handrail with tape increased the vibration amplitudes generally. However, serious consequences of this effect are restricted to very high velocities (above 50 m.p.h.).

5. With positive wind angles where the wind rises upward in the neighbourhood of the bridge, the amplitudes increase. However, under these conditions, the amplitudes still remain below 2 in. at low velocities whereas higher amplitudes occur in the high velocity range.

The very low height of the bridge above the water and the absence of large hills near the bridge would suggest that it is unlikely to be exposed to a serious problem arising from positive wind angles.

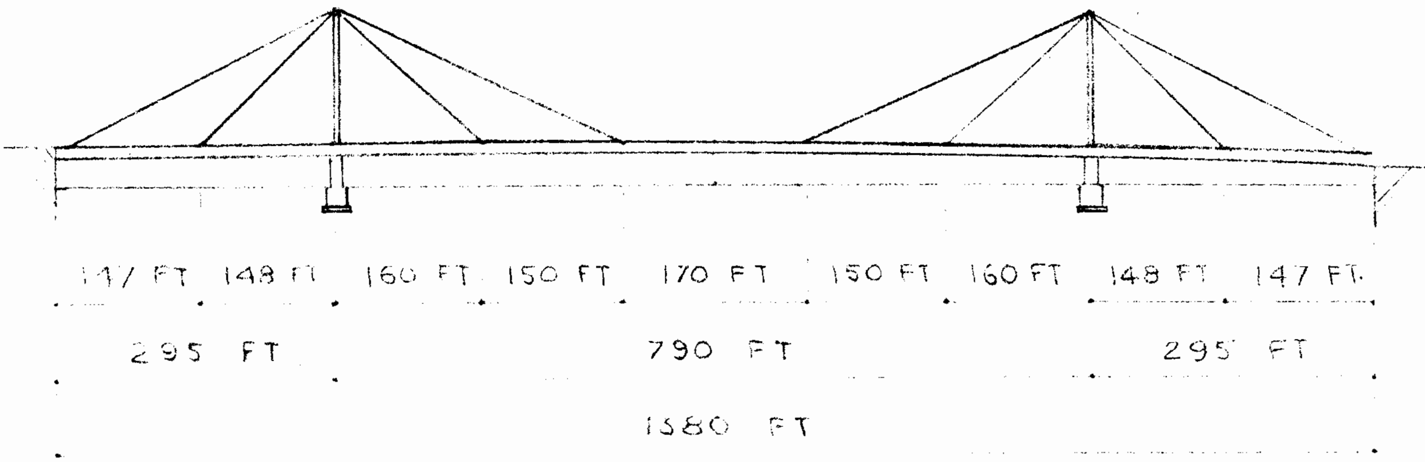
6. The damping values used should cover the range to be anticipated with the full-scale bridge. The higher damping values decreased amplitudes but were insufficient to eliminate the vibrations.

FIG 1

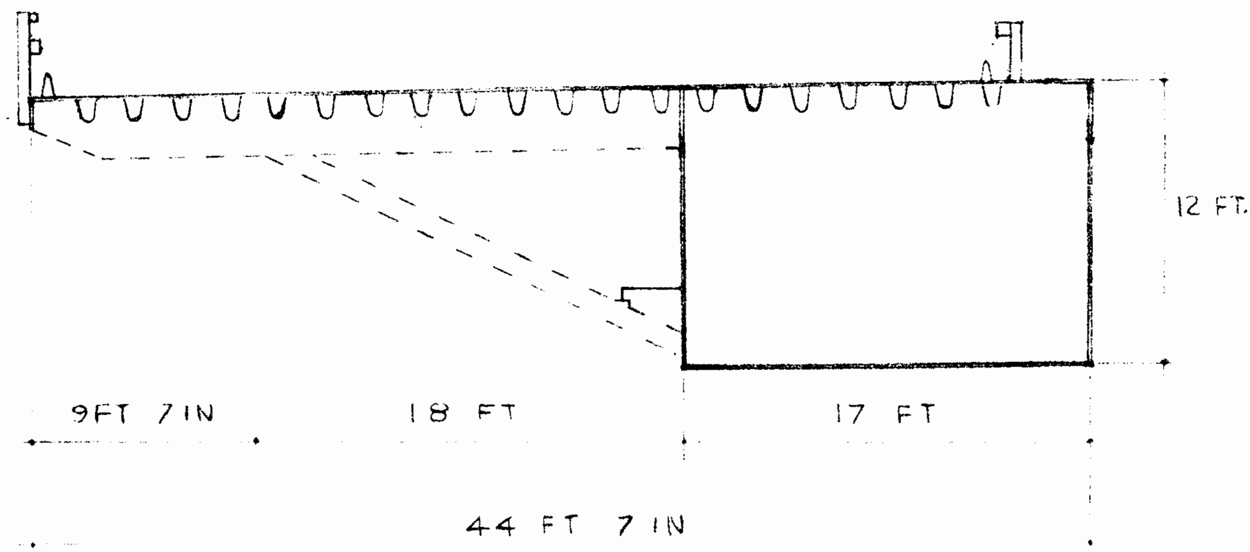


ARTIST'S DRAWING OF PROPOSED PAPINEAU BRIDGE

FIG 2



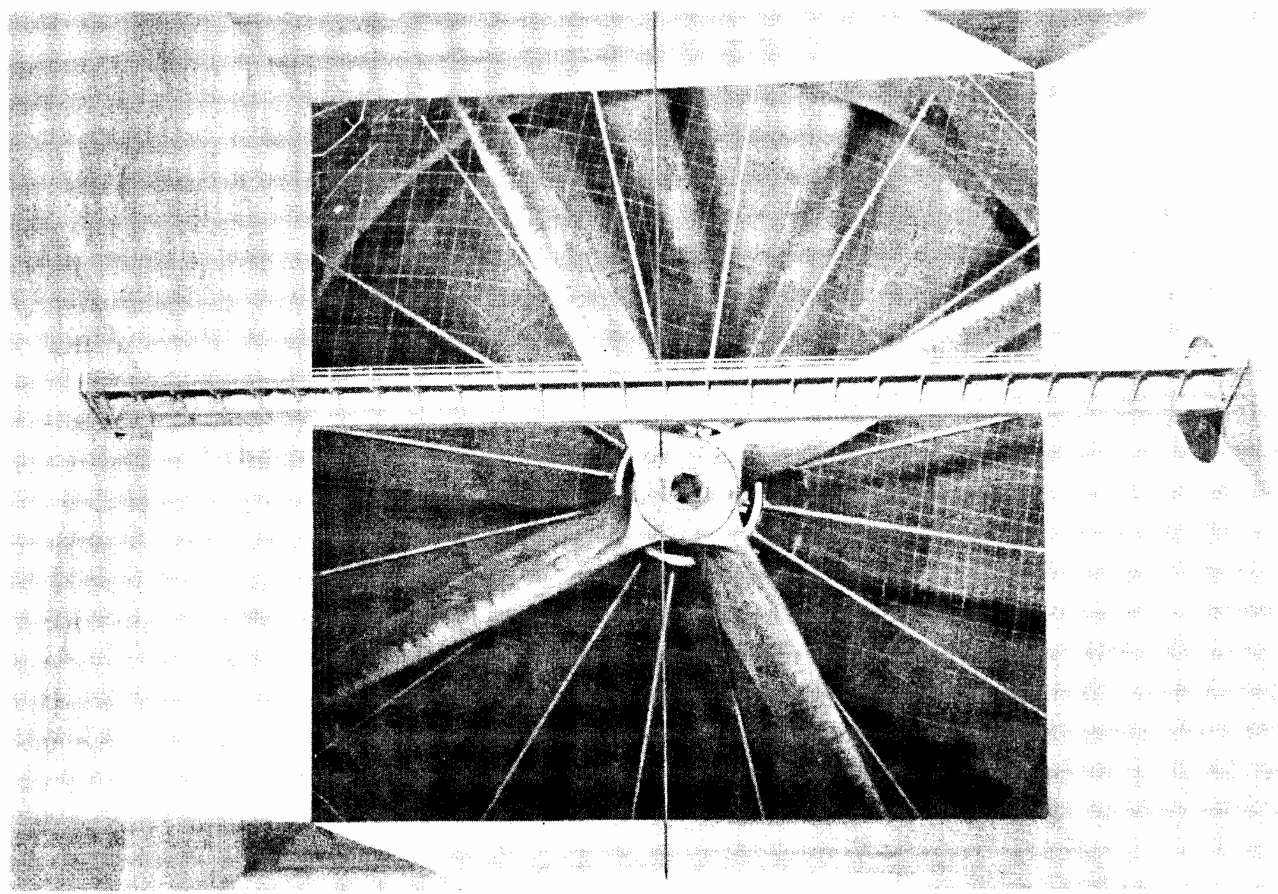
SCALE : 1-IN = 200 FT



SCALE : $\frac{1}{8}$ IN. = 1-FT.

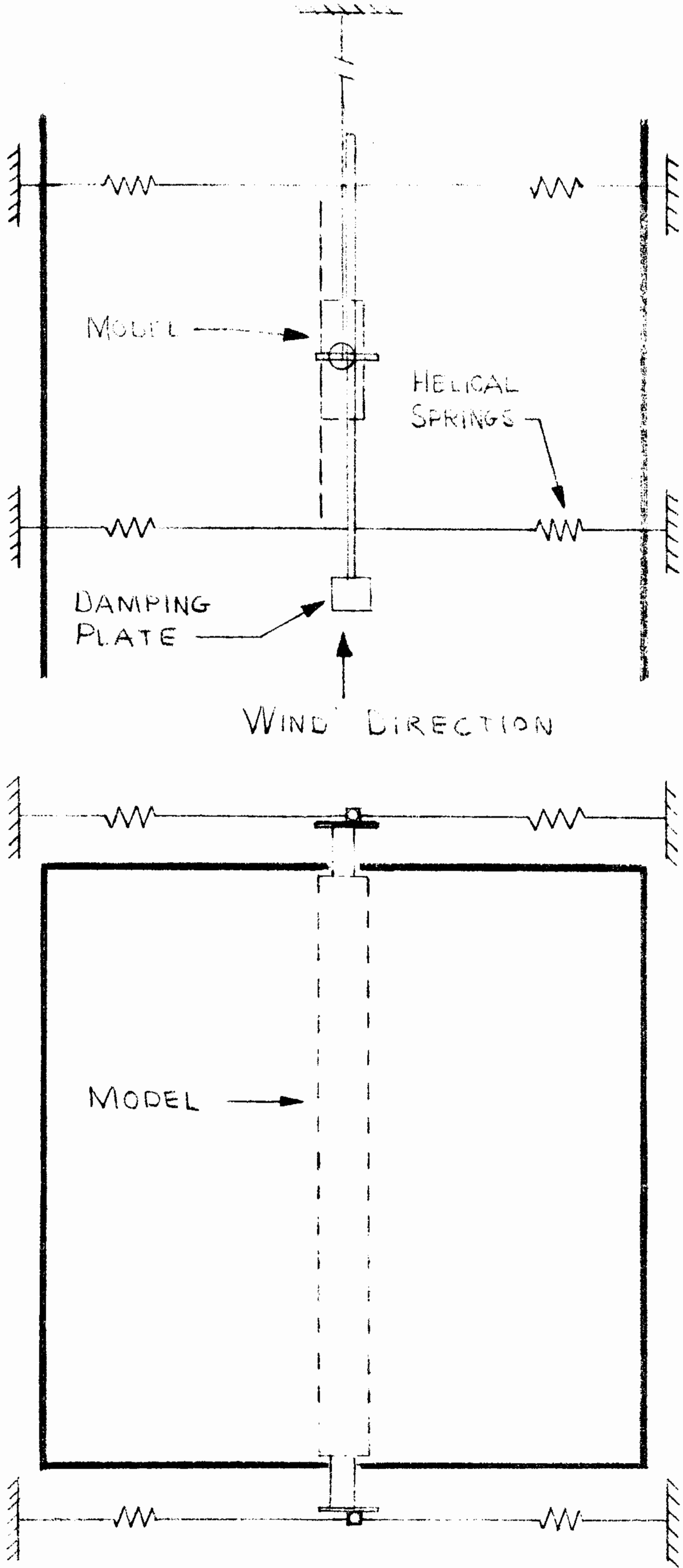
PAPINEAU BRIDGE FULL SCALE DIMENSIONS

FIG 3



PAPINEAU BRIDGE WIND TUNNEL MODEL INSTALLATION

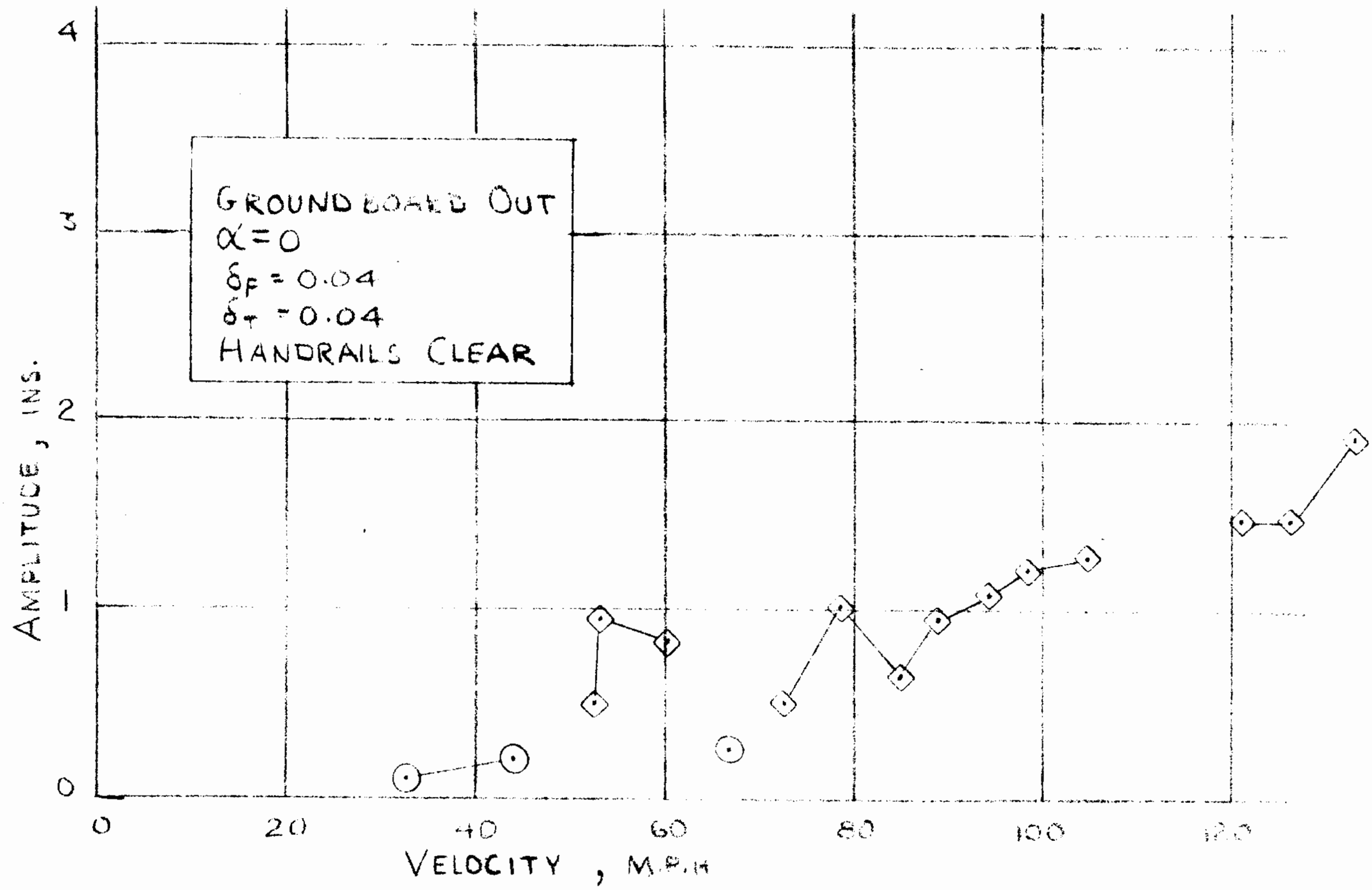
FIG 4



ELEVATION

PLAN

MODEL SUSPENSION SYSTEM



○ FLEXURE
◇ TORSION – AMPLITUDE AT BRIDGE EDGE
△ MIXED FLEXURE AND TORSION

FIGURE 5a – VIBRATION AMPLITUDE

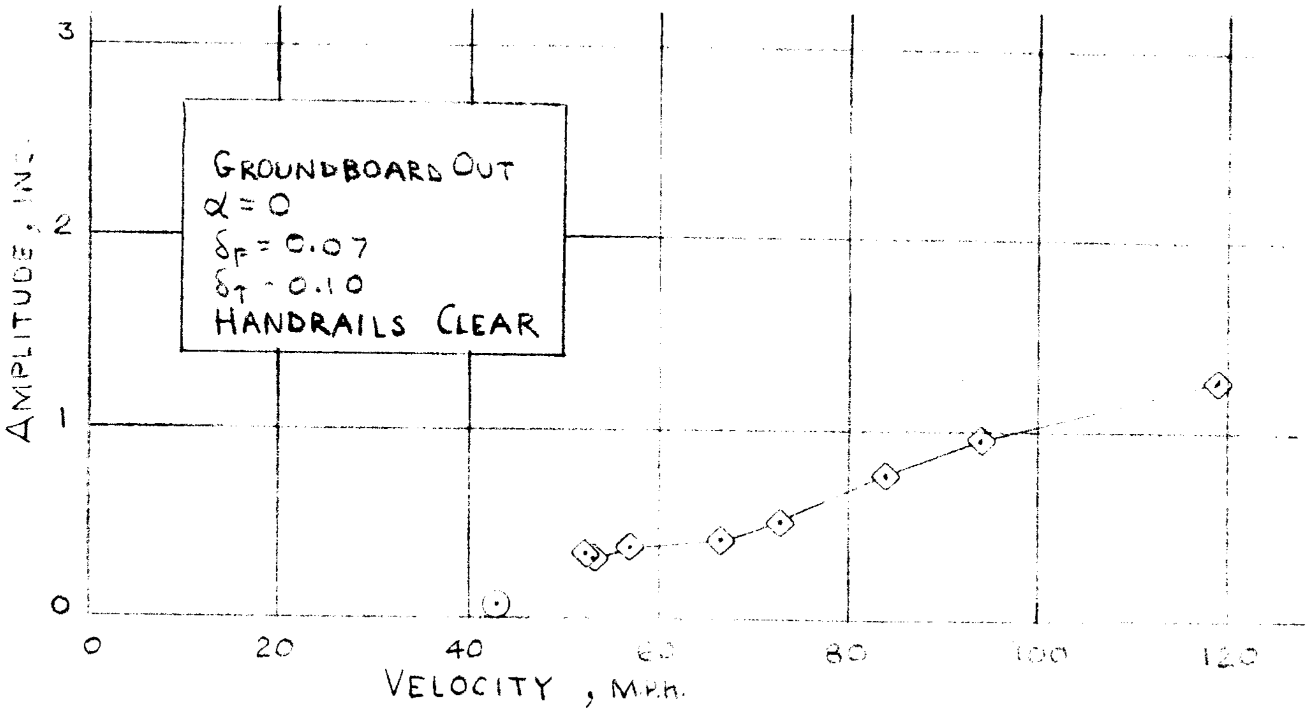


FIG 5 b

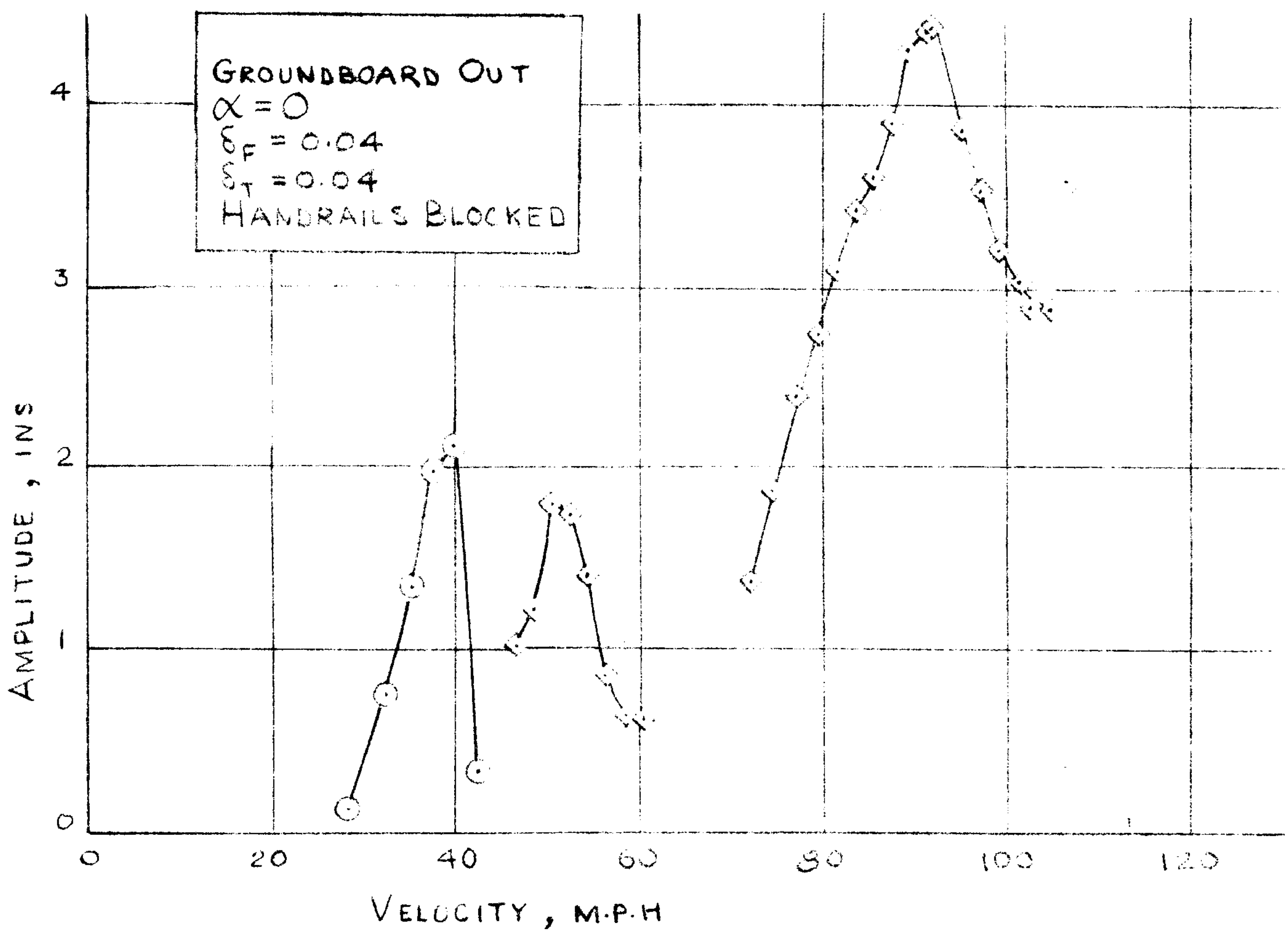


FIG 5 c

VIBRATION AMPLITUDE

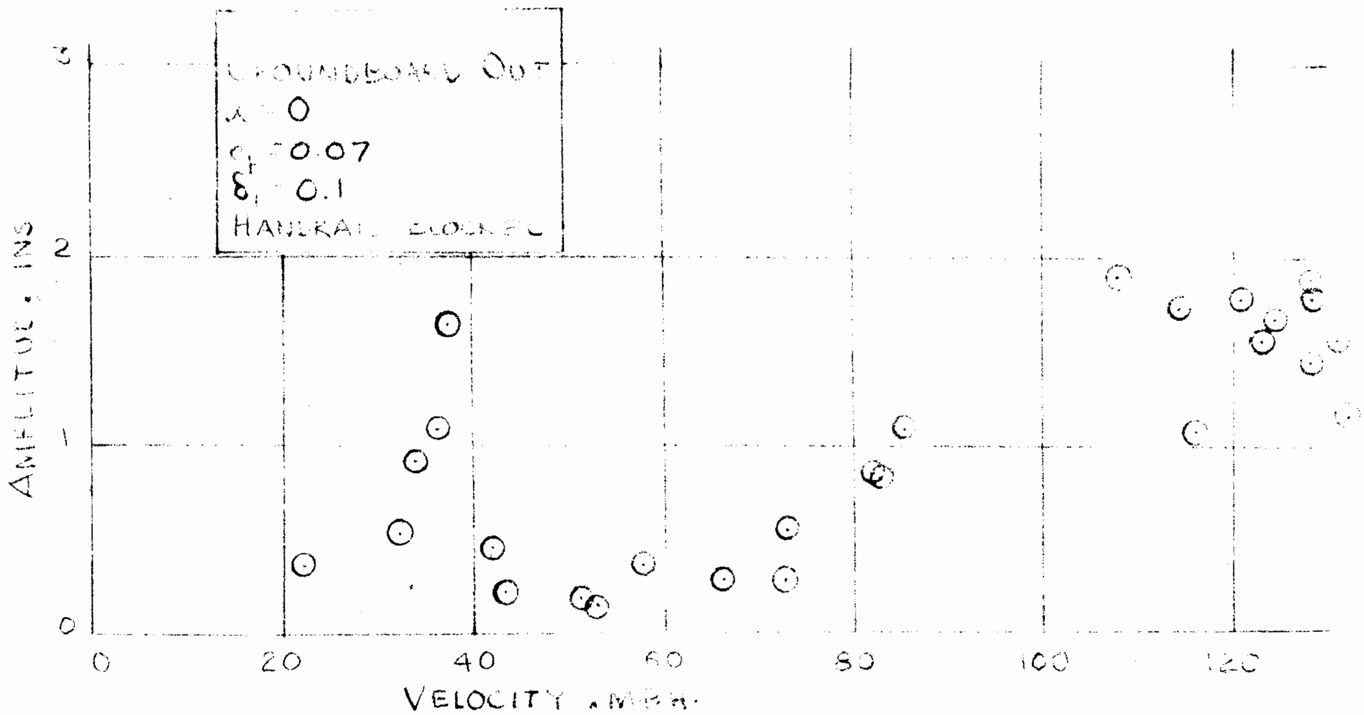


Fig 5 d

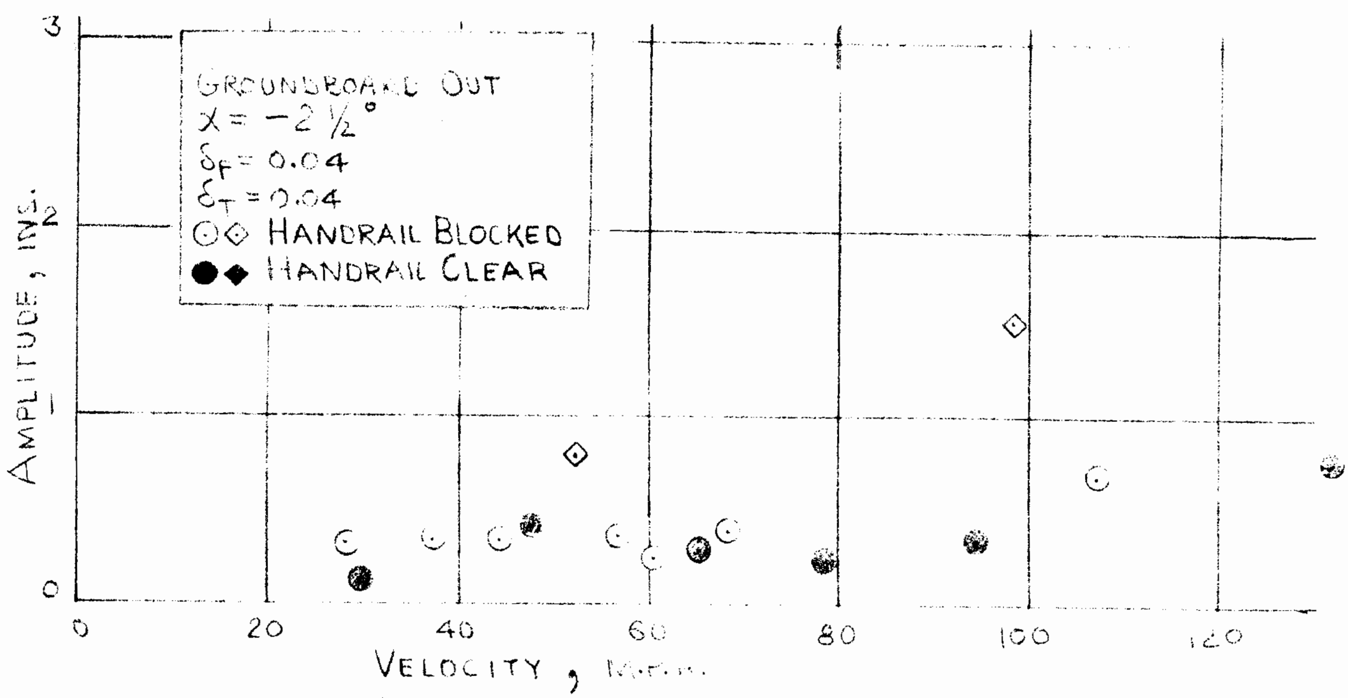


Fig 5 e

Vibration Amplitude

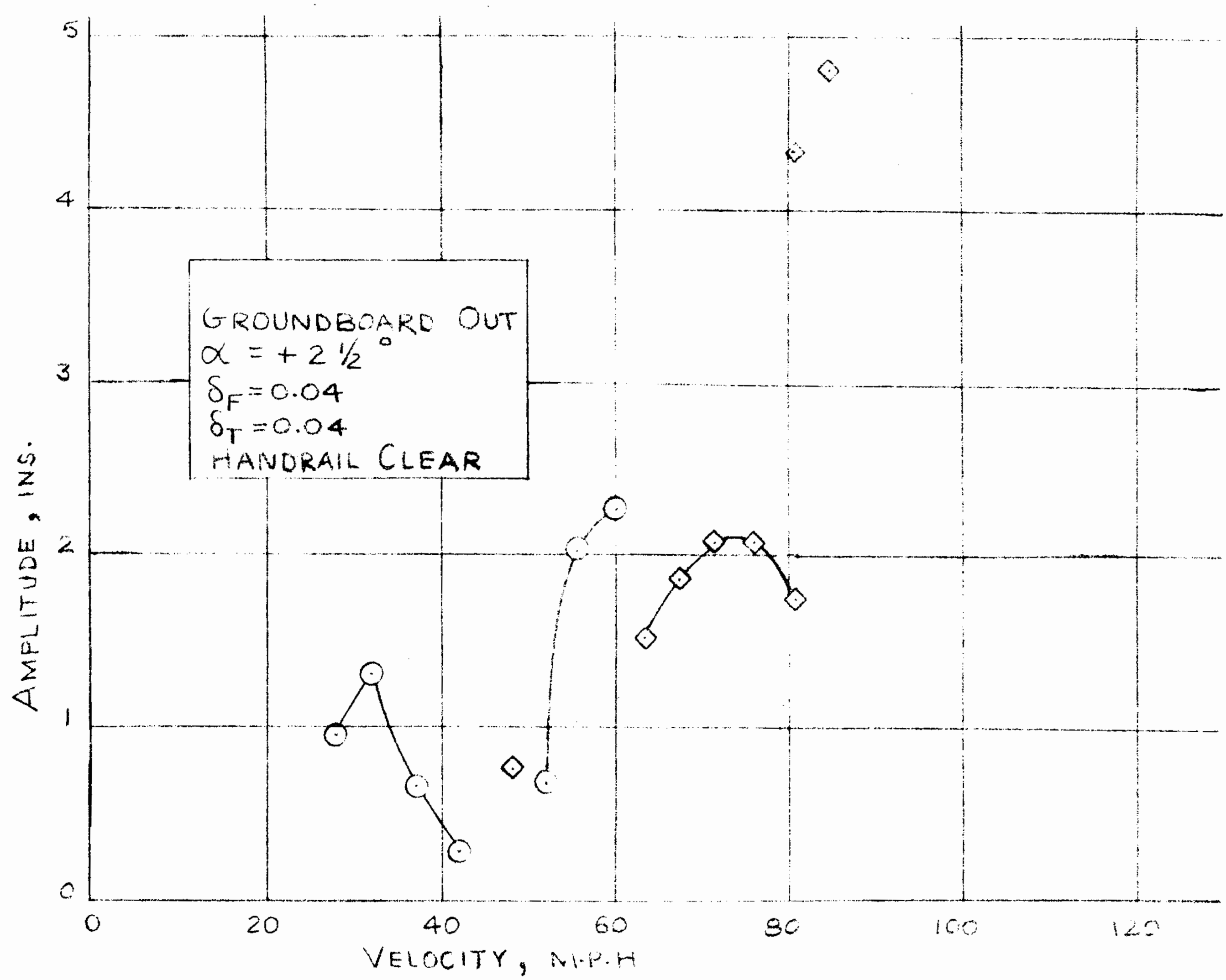


FIGURE 5 f – VIBRATION AMPLITUDE

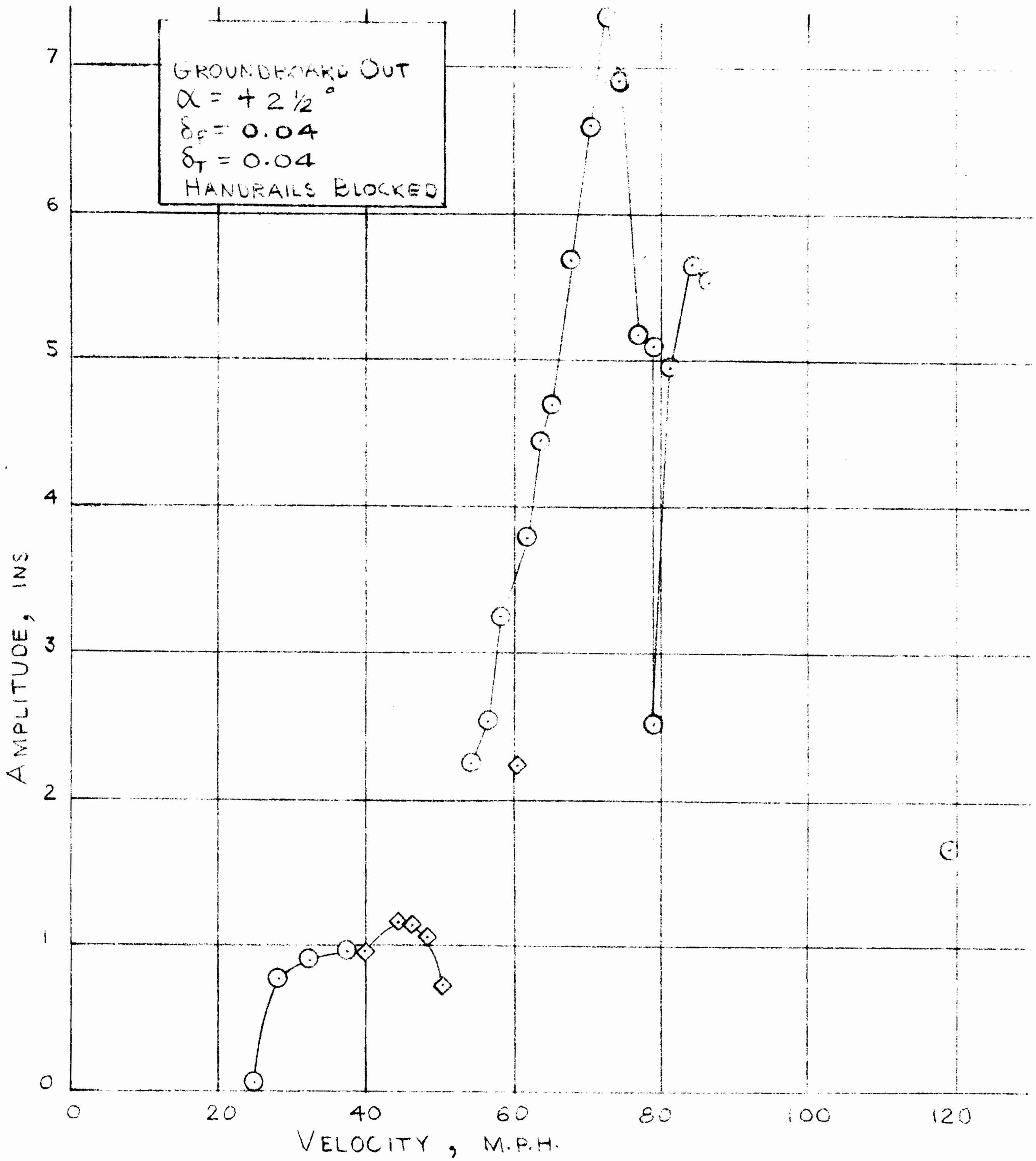


FIGURE 5g – VIBRATION AMPLITUDE

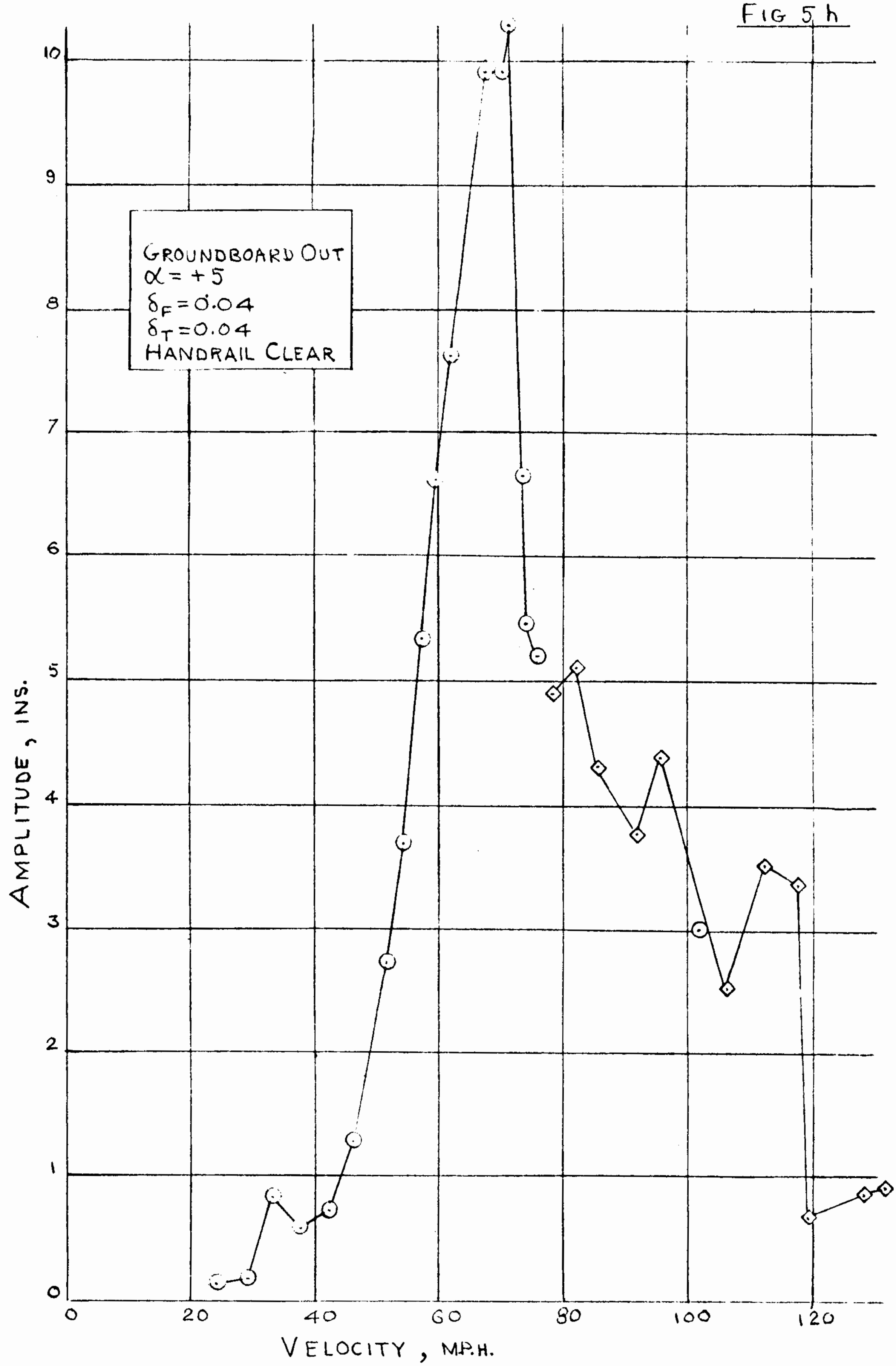


Figure 5 h – Vibration Amplitude

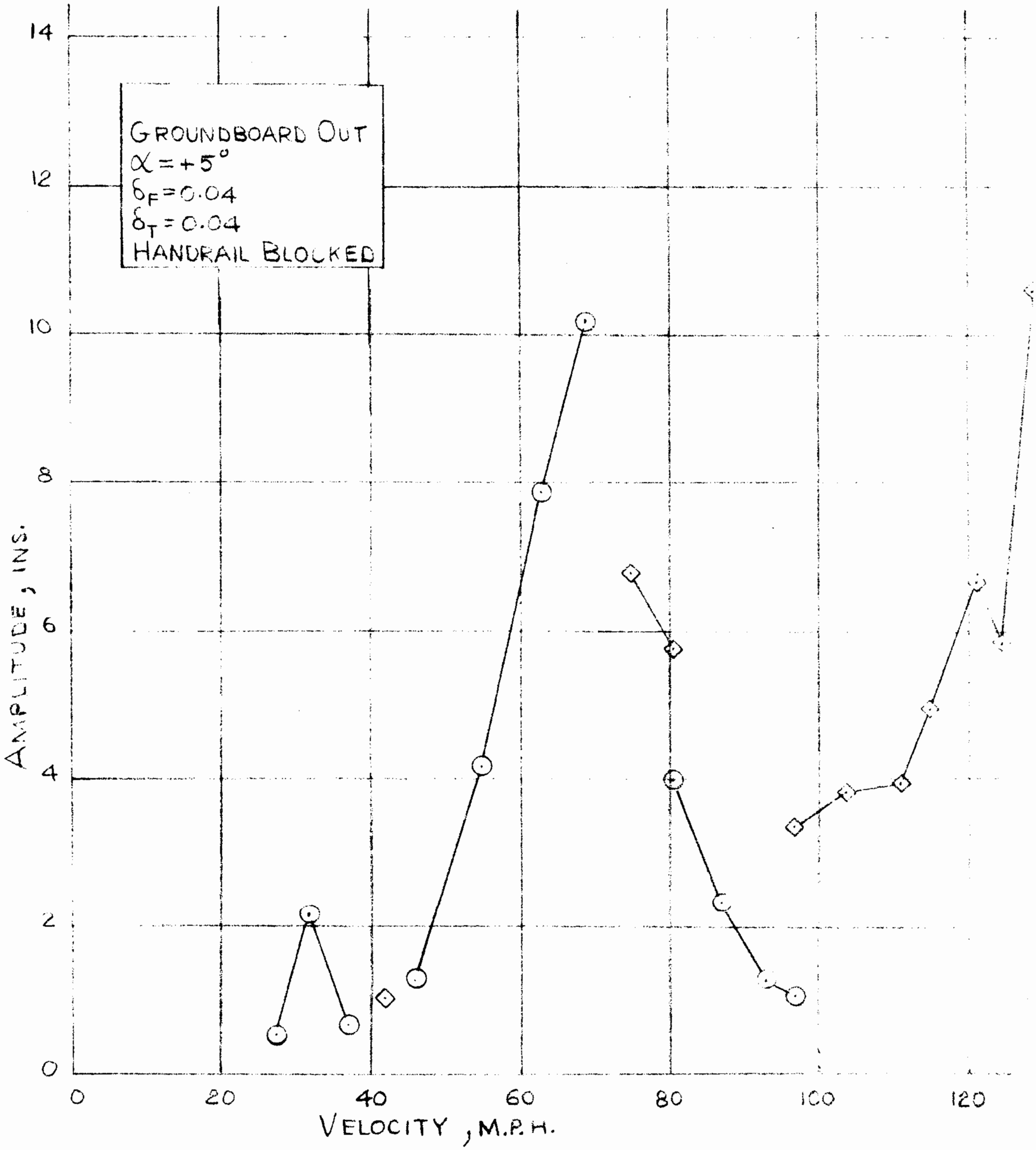


FIGURE 5 i – VIBRATION AMPLITUDE

Fig 5 j,k

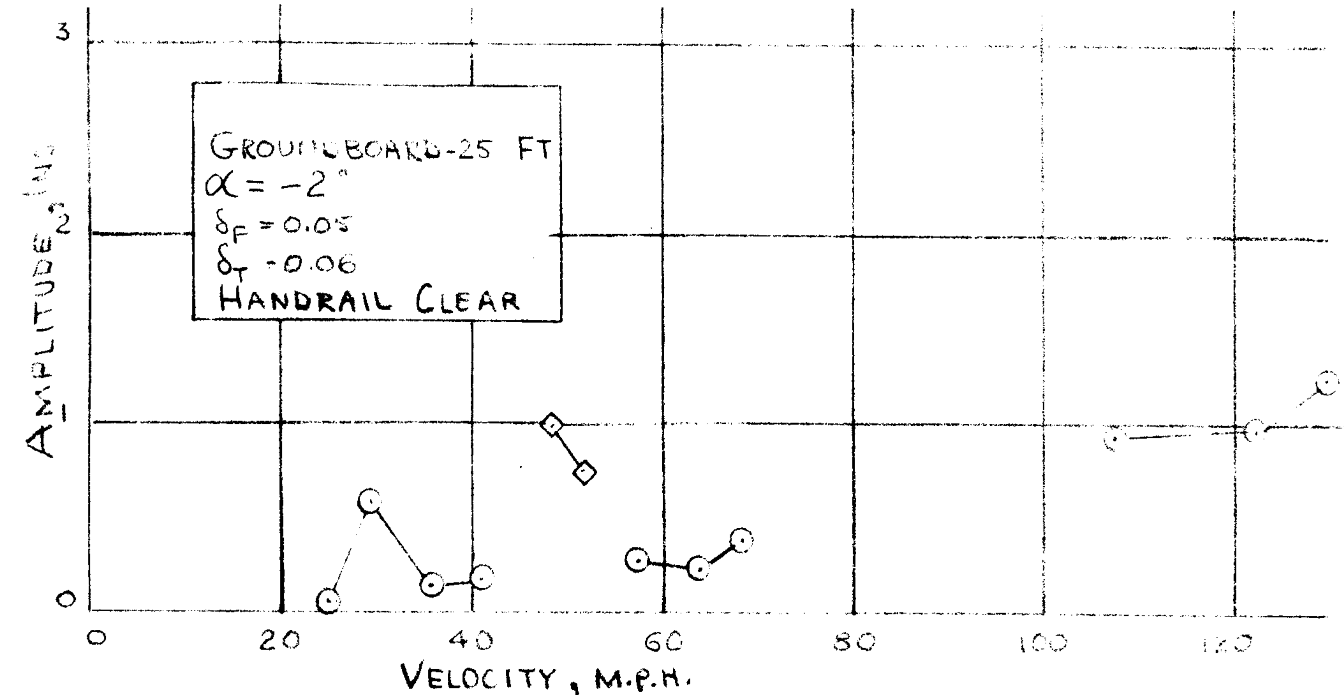


Fig 5 j

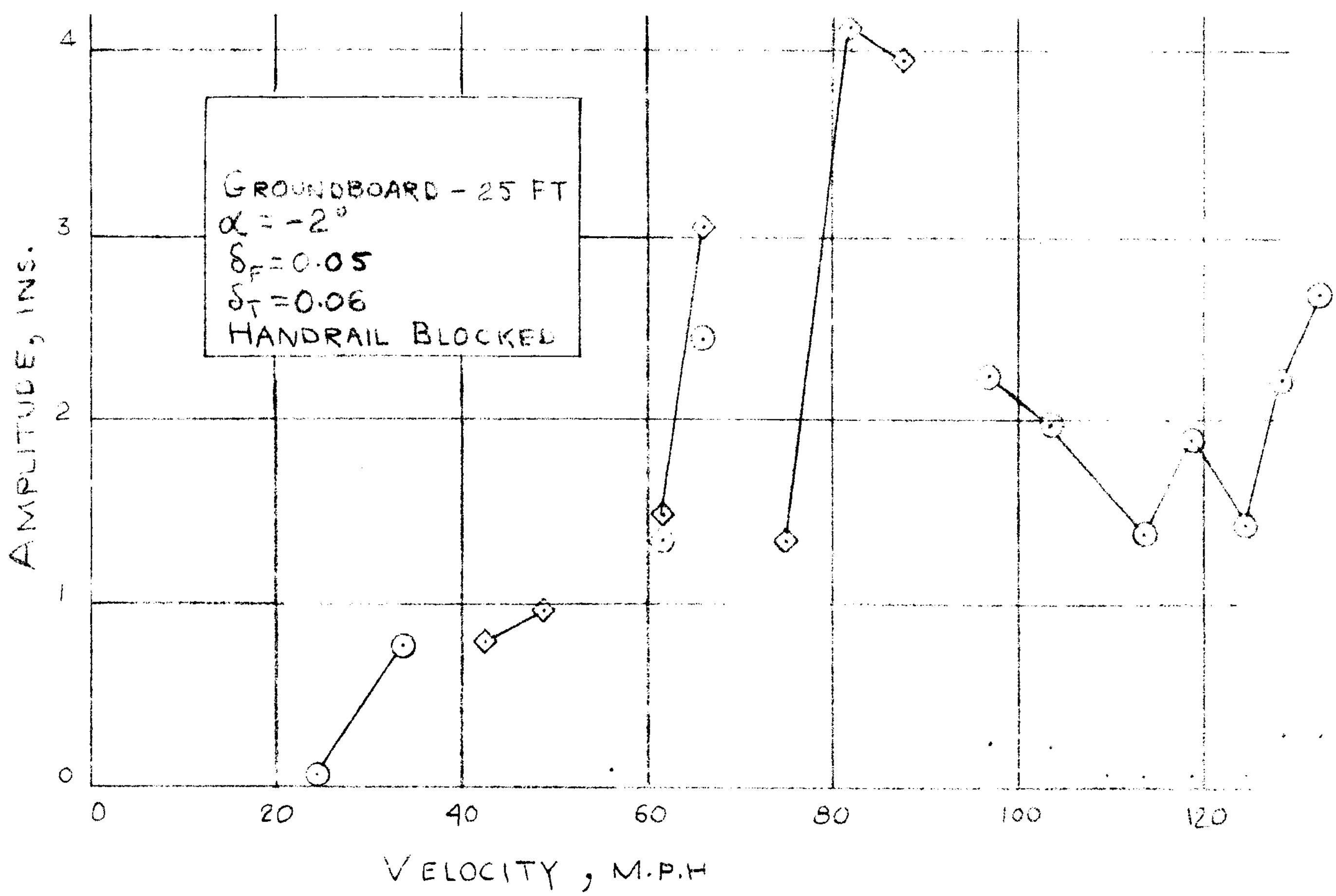


Fig 5 K

VIBRATION AMPLITUDE

Fig 5-1, m

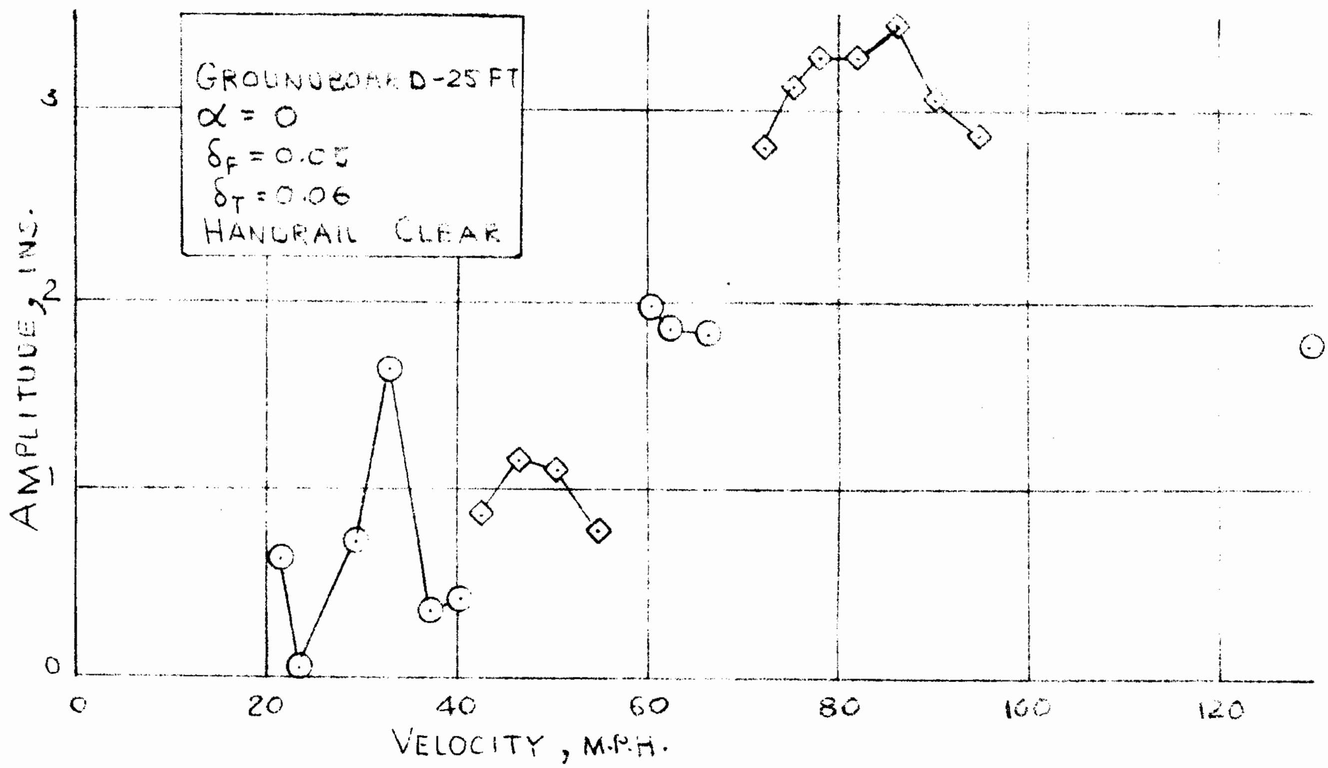


Fig 5-1

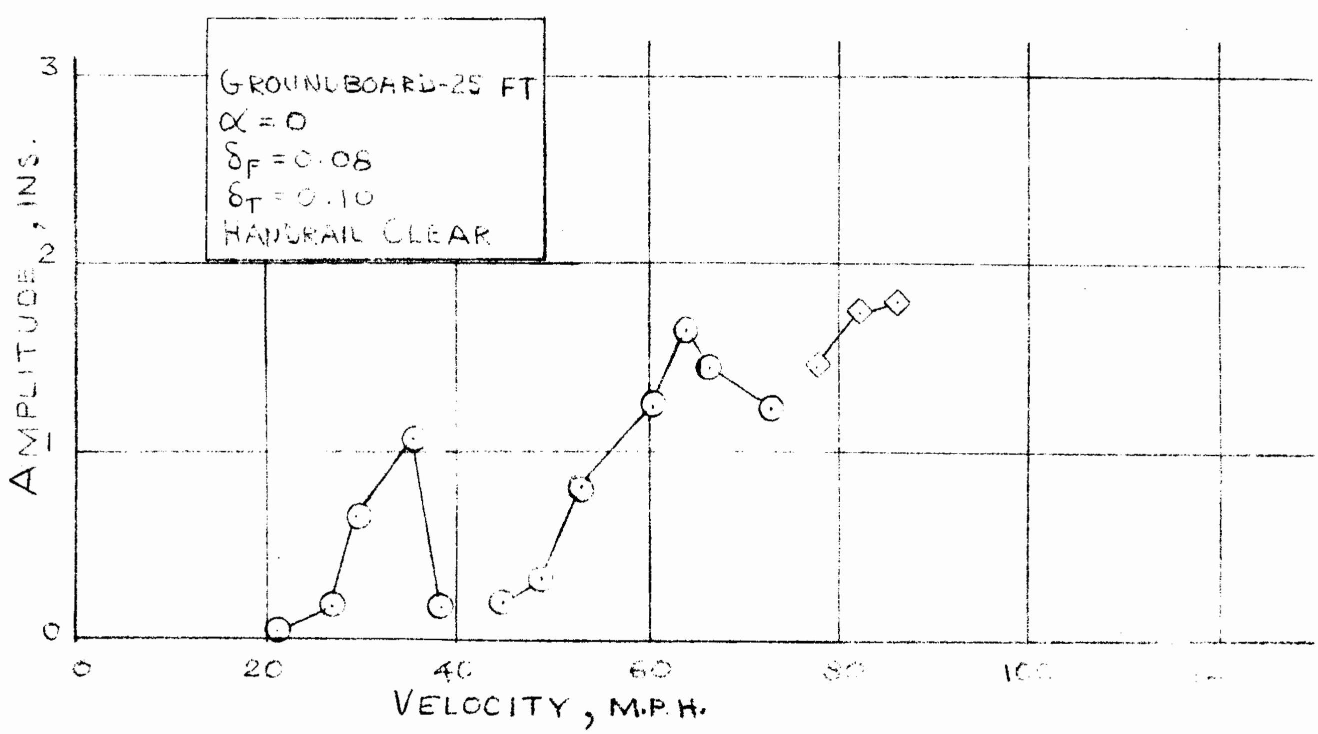


Fig 5 m

Vibration Amplitude

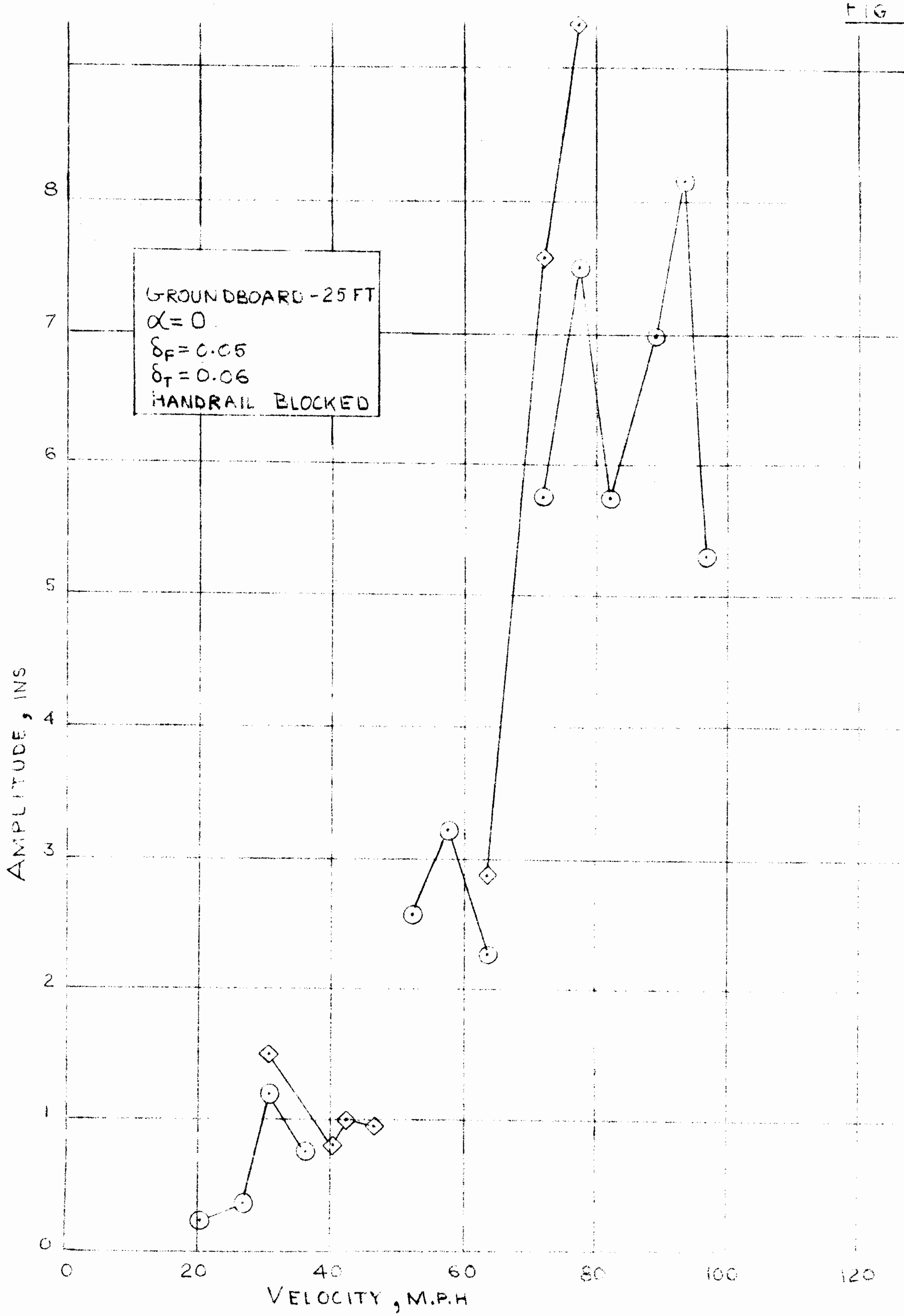


FIGURE 5 n - VIBRATION AMPLITUDE

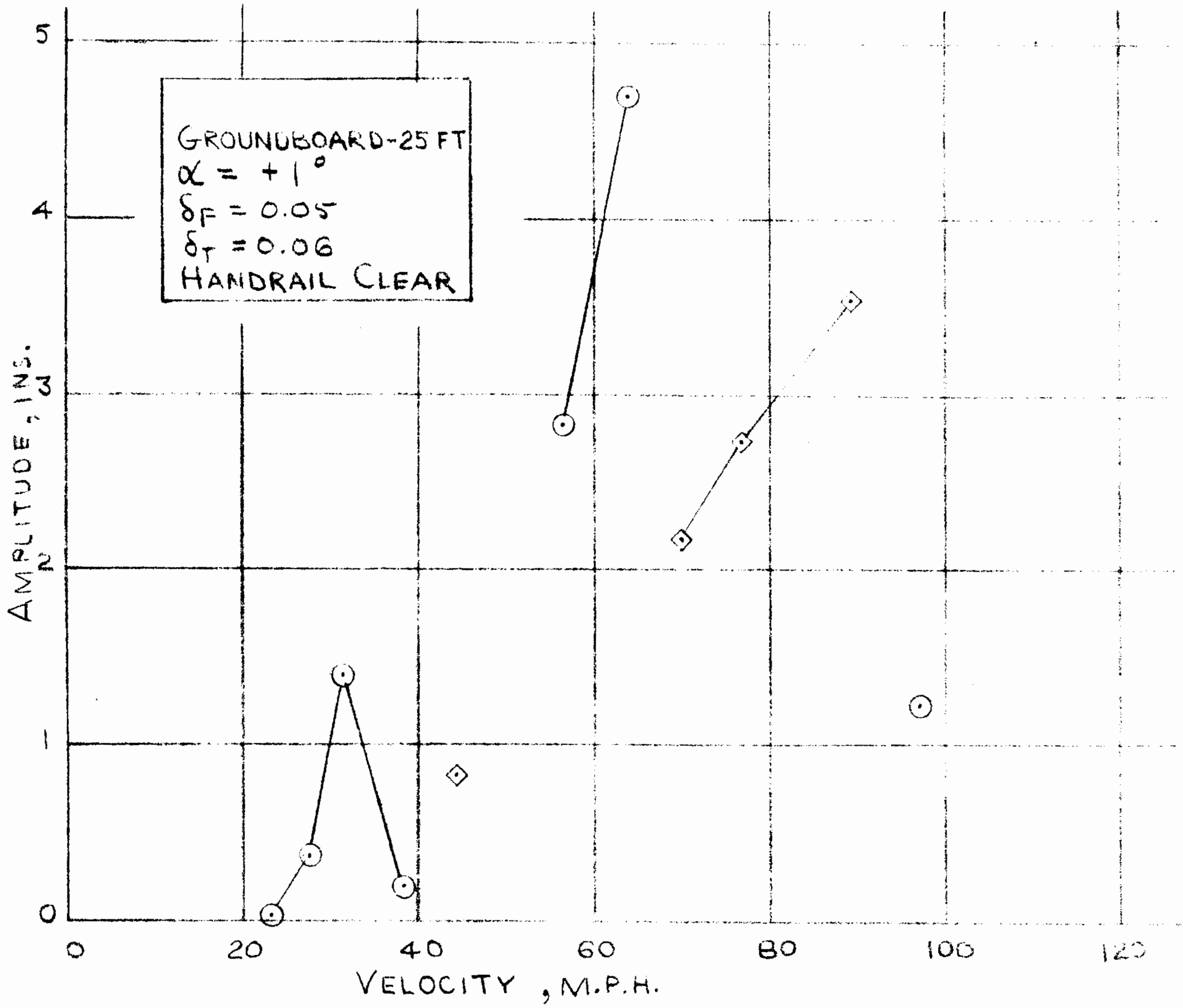


FIGURE 5-O - VIBRATION AMPLITUDE

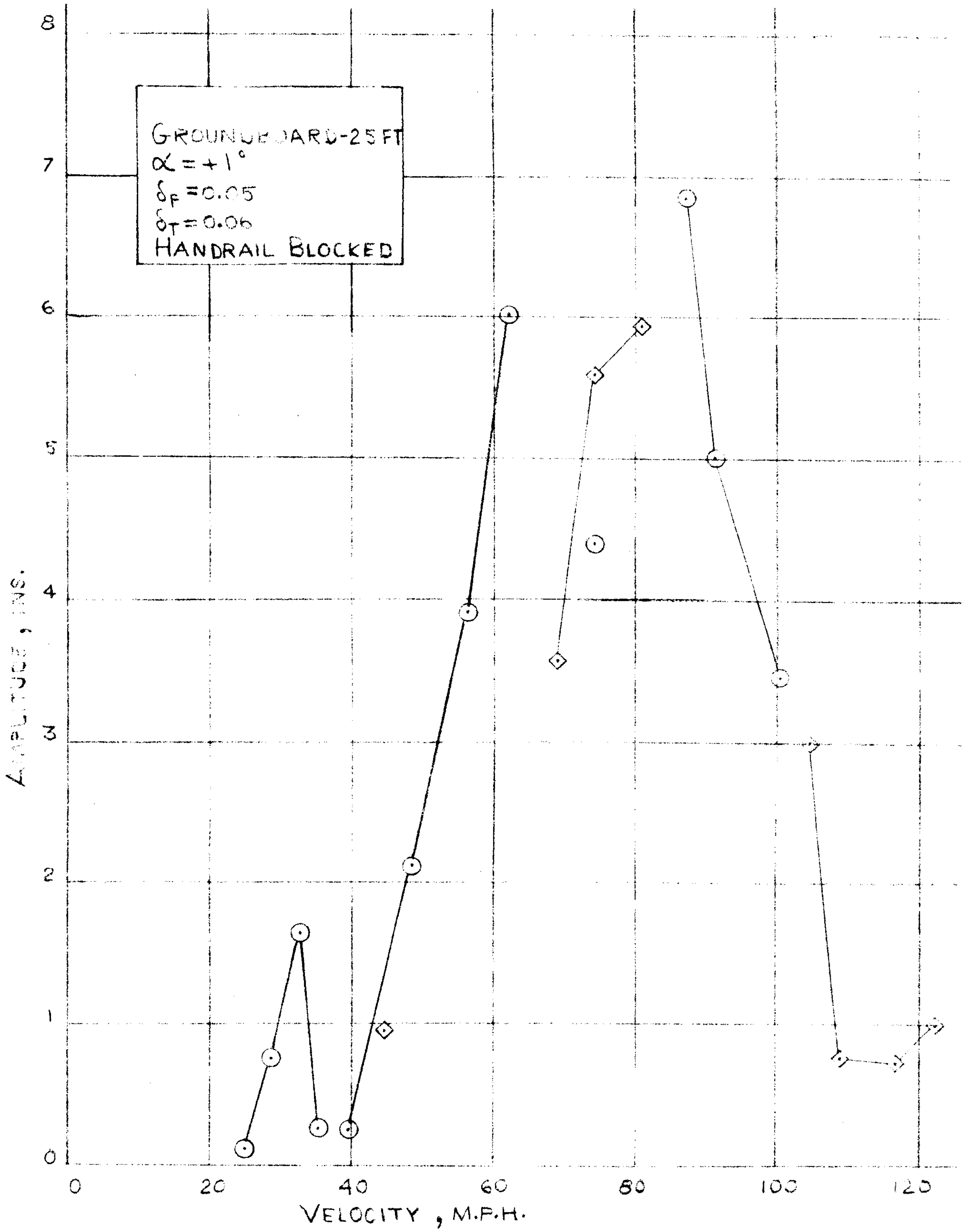


Figure 5 p - Vibration Amplitude

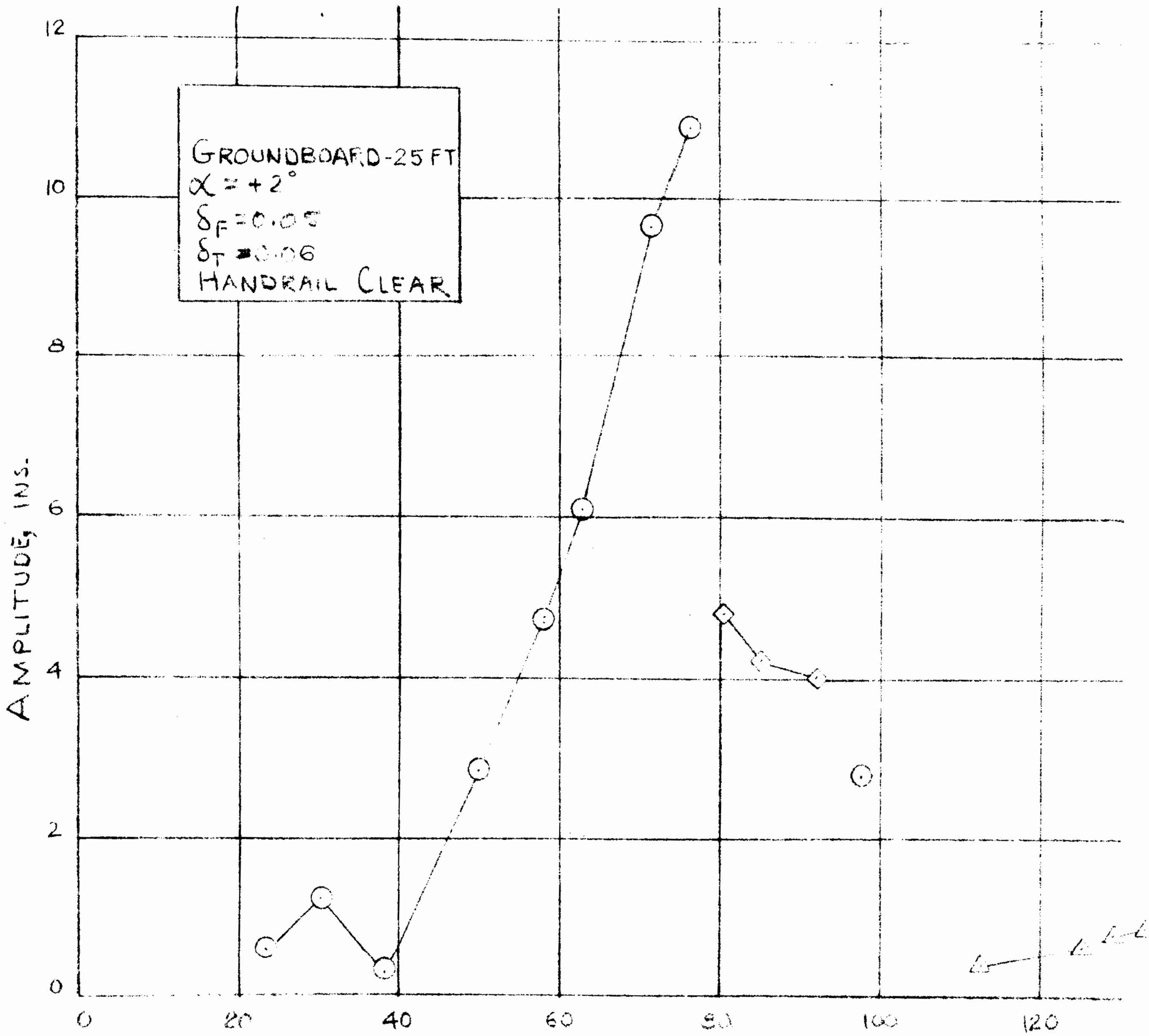


FIGURE 5 q - VIBRATION AMPLITUDE

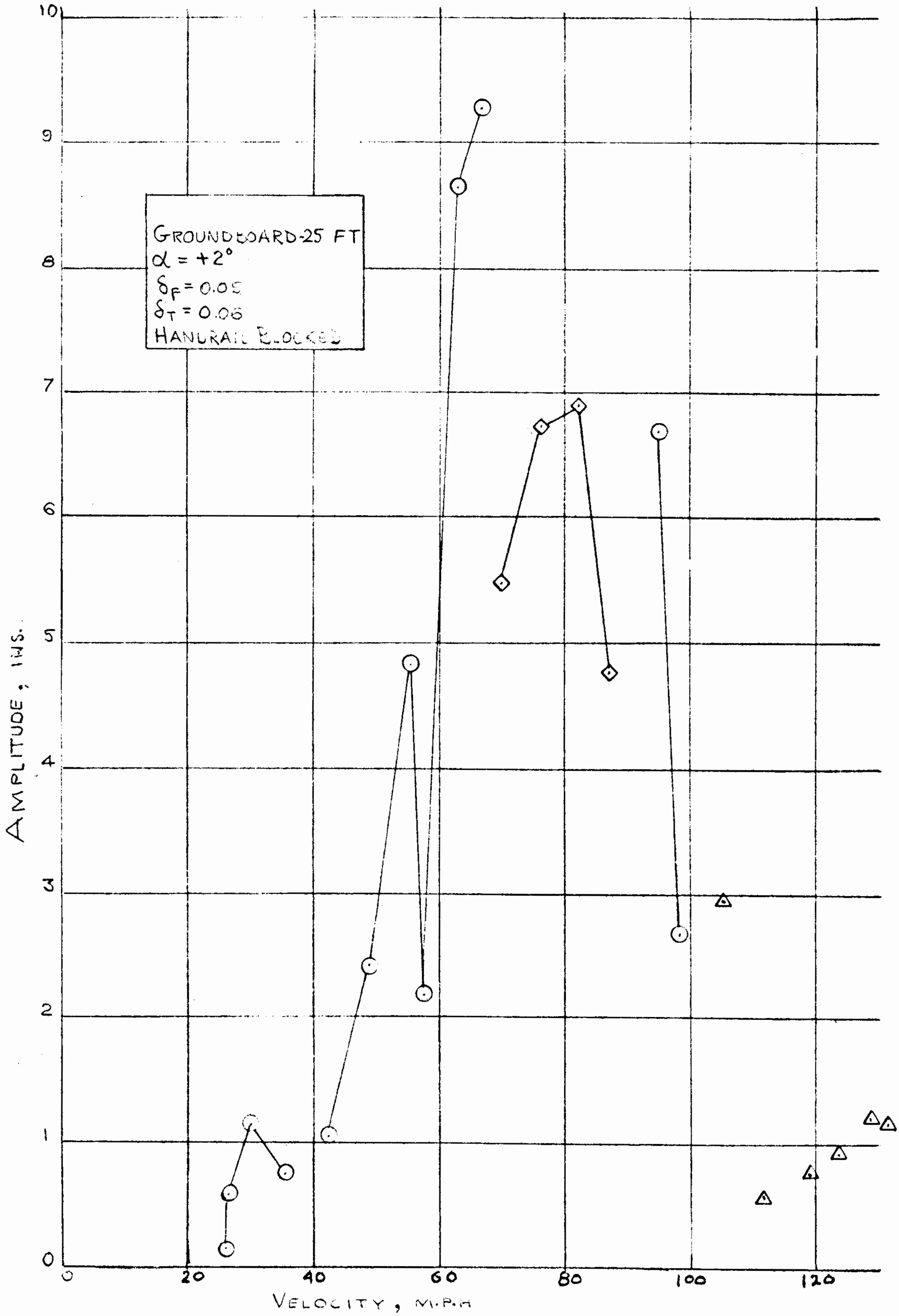


FIGURE 5 r - VIBRATION AMPLITUDE

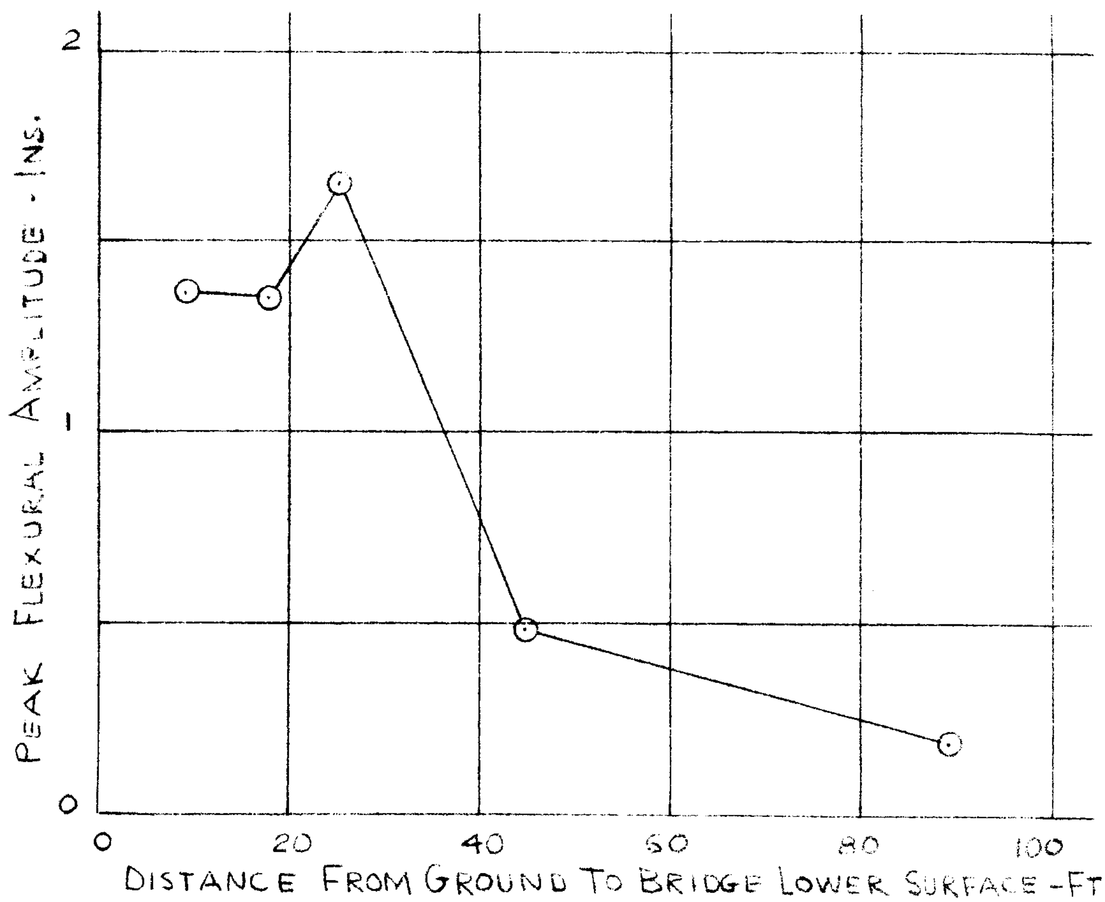


FIGURE 6 - EFFECT OF HEIGHT ON AMPLITUDE IN THE VELOCITY RANGE 20 - 40 M.P.H. - HANDRAILS CLEAR