# Vibration Response of Railway Bridge Piers to Nearby Pile Driving

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# Vibration Response of Railway Bridge Piers to Nearby Pile Driving

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SYNOPSIS: In the development of a new highway system approaching Tel Aviv, Israel, long piles had to be driven very close to an existing railway bridge. In view of the danger of structural damage to the bridge, as a result of ground vibrations excited by the pile-driving work, field measurements were ordered.

Vibrations recorded at the bridge pier heads in the course of the driving operation were analyzed and compared with those excited by trains. Results showed that the former are generally lower than the latter, and neither one exceeds the limits prescribed by the codes.

## INTRODUCTION

The new highway system approaching Tel Aviv from the north, as part of the Ayalon project, will include several overpasses and intersections. The new construction will require piles to be driven very close to the Tel Aviv-Haifa railway bridge currently in use. In view of the danger of structural damage to the bridge as a result of ground vibrations excited by the pile-driving work, a program was carried out which included field measurements of the bridge piers while test piles were driven into the ground in various locations.

## THE SITE

The bridge, built in the fifties with its one-track railway and several spans, is a steel bridge supported on reinforced concrete piers, based on driven reinforced concrete piles. It is situated very close to the merging point of the Ayalon with the Yarkon River (Figs. 1 and 2).

Ground conditions, according to borehole investigations, were as follows: soft and medium strength clays on the south bank, as deep as 60m and similar clays on the north bank, interrupted by an intermediate 6m thick layer of compact sand between 16m and 22m deep. The water table is 2.5m below ground level.

These ground conditions required the use of prestressed concrete piles for the foundations of the new overpasses which will be built.

## TEST PROGRAM

The piles, 0.50m in diameter and 45m long, were composed of two equal shorter piles welded to one another after the first part has been driven into the ground. A special test pile differed from the others in that special metal devices were contained, to extend out of its faces, after the pile had been driven. The hammer in use was a Delmag-D44 with a rate of about one stroke per second.

Most of the vibration measurements (using 1Hz, nat. freq. seismometers) were carried out simultaneously at two different pier heads, and in each one - in both the longitudinal and transverse directions of the bridge. In some cases (Tests 1 and 2), the measurements were taken at the foot of the pier, on the ground, and at the same time at its head.

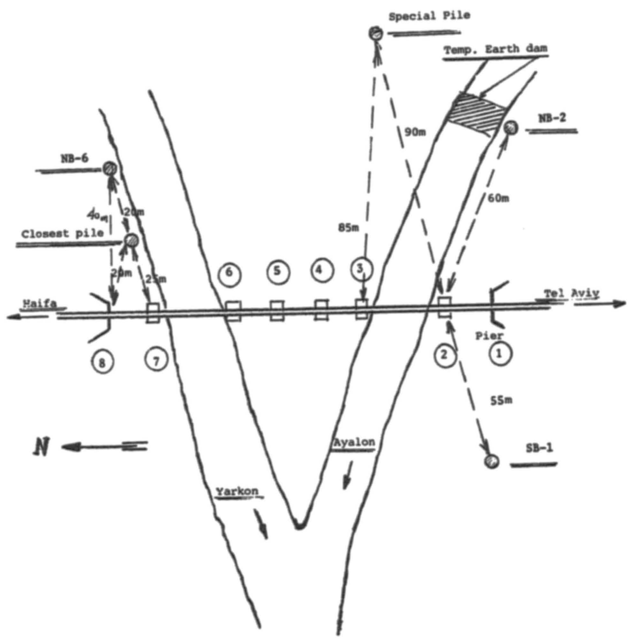


Fig. 1 - Layout of the railway bridge and location of the test piles.

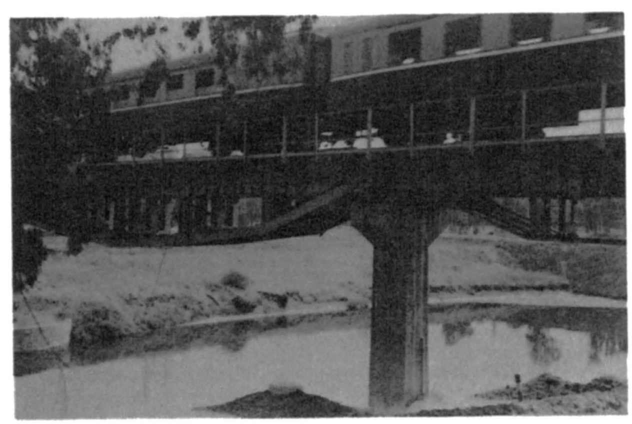

Fig. 2 - Eastward view of pier 2.

The piers selected were always the closest ones to the driven pile, and the vibrations were measured at different pile depths. Vibrations were also measured during the passage of trains for comparison purposes.

## RESULTS

The detailed measurements appear in Table 1 with typical vibration records shown in Fig. 3 to 7. Velocity spectrum (third octave) were found for all tests, but only few of them are shown in Fig. 8 to 11. Results are also given in Table 2, including the computed maximum displacements of the vibrations.

Table 1 - Details of the measurements at various piers (see Fig. 1)

| Test | Pile | Depth | Source of Vibration | Channel 1 | Channel 2 | Channel 3 | Channel 4 |
|---|---|---|---|---|---|---|---|
| 1 | SB-1 | 18m | Hammer | 2gt(*) | 2gl | 2ht | 2hl |
| 2 | | 18m | Train | " | " | " | " |
| 3 | NB-2 | 26m | Hammer | – | – | 2ht | 2hl |
| 4 | | 32m | " | – | – | " | " |
| 5 | | 37m | " | – | – | " | " |
| 6 | Special | 10m | Hammer | – | 3hl | 2ht | 2hl |
| 7 | Pile | 16m | " | 3ht | " | " | " |
| 8 | | 22m | " | " | " | " | " |
| 9 | | 28m | " | " | " | " | " |
| 10 | Closest | 16m | Rap.Train | 8ht | 8hl | 7ht | 7hl |
| 11 | to | 16m | Hammer | " | " | " | " |
| 12 | pier 8 | 18.5 | " | " | " | " | " |
| 13 | | 23m | " | " | " | " | " |
| 14 | | 30m | " | " | " | " | " |
| 15 | | 35m | " | " | " | " | " |
| 16 | | 38m | " | " | " | " | " |
| 17 | NB-6 | 15m | Hammer | 8ht | 8hl | 7ht | 7hl |
| 18 | | 17m | " | " | " | " | " |

(*)The figure denotes = pier number;
gt,gl = ground measurements at the pier foot in the transverse, and longitudinal directions of the bridge;
ht,hl = pier head measurement in the transverse and longitudinal directions of the bridge.

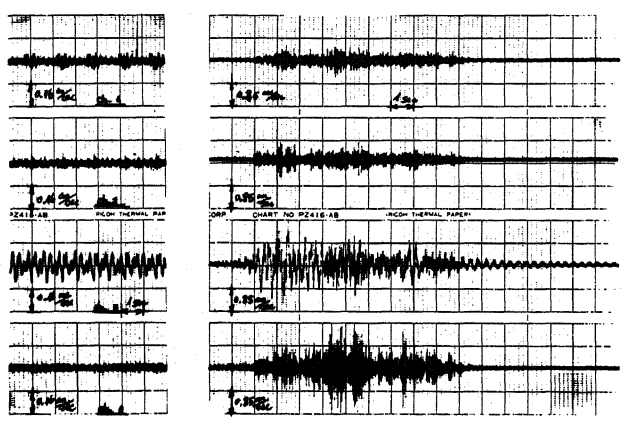

Fig. 3 - Vibration Measurements at pier 2: Test 1 (left); Test 2 (right).

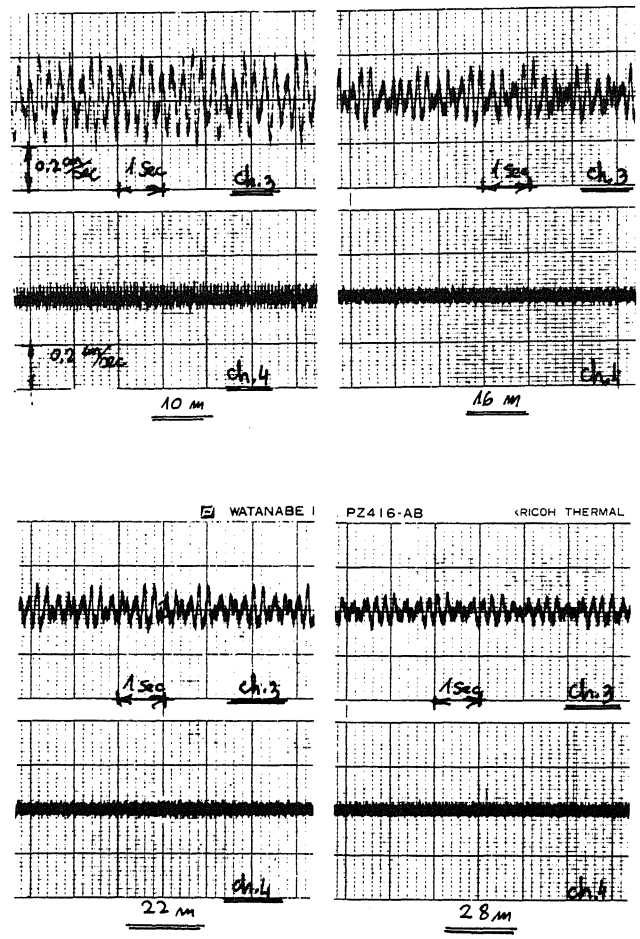


Fig. 4 - Vibration measurements at pier head 2, during driving of the special pile at various depths (Tests 6 to 9).

## DISCUSSION

It has been observed from all the measurements that the stiffness of the bridge is much greater in the longitudinal direction than in its transverse direction. The natural frequency and critical damping ratio of the bridge in that direction are 3 Hz and 1.5%, according to its free vibration after the passage of the train (Ch. 3, Fig. 3). As a result, the most significant quantities to be considered in the dynamic analysis of the bridge are therefore the transverse velocities or displacements.

It is interesting to note also, according to Tests 1 and 2, that the vibration at the pier head was amplified by a factor of two, as compared with the measurements at the foot of the same pier. This behaviour occurred during both hammer operation and train passage as well (Fig. 3). Vibrations at the pier heads 2 and 3 decreased with increasing depth of the driven pile, as can be seen in Fig. 4, mainly in the transverse direction. In the present case, the ground had a uniform composition in

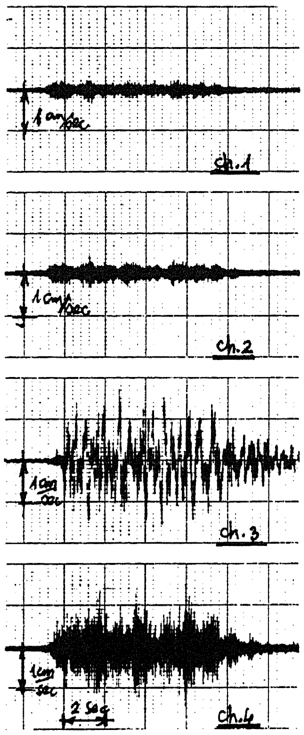


Fig. 5 - Measurements during passage of a rapid train (Test 10).

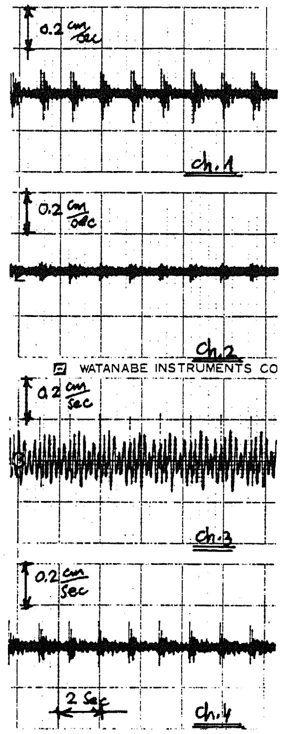


Fig. 6 - Measurements during driving of the closest pile (30m)(Test 14).

the south bank of soft and medium strength clays, as mentioned earlier. On the north bank, where there was a compact sand layer between 16m and 22mm, the same phenomenon was observed again, as the vibration at the pier head No. 7 was drastically reduced when pile NB-6 was driven from 15m to 17m deep (Fig. 7). This reduction in the amplitude is due to the high strength of the compact sand layer.

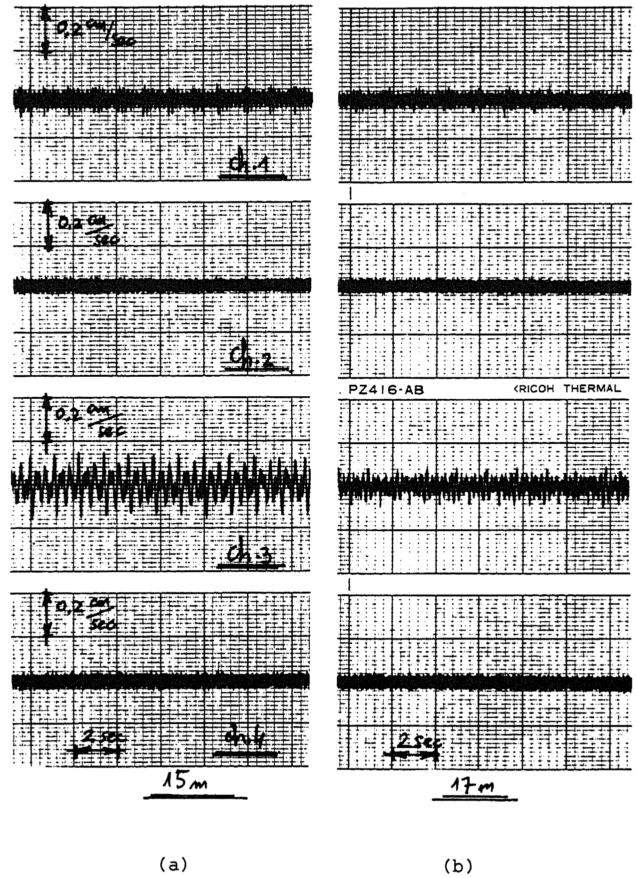


Fig. 7 - Vibration measurements during driving of pile NB-6: (a) Test 17 (15m); (b) Test 18 (17m).

Table 2 - Frequencies, peak velocities and displacements as obtained at the bridge piers

| Test | Channel | Pier | Frequency Hz | Peak Velocity ±mm/sec | Maximum Displacement ±mm | Source of Vibration |
|---|---|---|---|---|---|---|
| 1 | 3 | 2ht | 4.4 | 0.66 | 0.024 | Hammer, pile SB-1 (18m) |
| 2 | 3 | " | 2.5 | 2.50 | 0.159 | Train |
| 2 | 4 | 2hl | 50.0 | 1.50 | 0.005 | Train |
| 3 | 3 | 2ht | 6.4 | 0.36 | 0.009 | Hammer, NB-2 (26m) |
| 4 | 3 | " | 6.1 | 0.33 | 0.009 | " (32m) |
| 5 | 3 | " | 4.5 | 0.39 | 0.014 | " (37m) |
| 6 | 3 | 2ht | 4.3 | 1.20 | 0.044 | Hammer, Special pile (10m) |
| 7 | 3 | " | 4.8 | 0.75 | 0.025 | " (16m) |
| 8 | 3 | " | 4.8 | 0.42 | 0.014 | " (22m) |
| 9 | 3 | " | 4.8 | 0.33 | 0.011 | " (28m) |
| 10 | 1 | 8ht | 80.0 | 1.20 | 0.002 | Rapid Train |
| 10 | 2 | 8hl | 160.0 | 1.80 | 0.002 | " |
| 10 | 3 | 7ht | 4.0 | 5.40 | 0.215 | " |
| 10 | (3) | " | (1.6) | (4.20) | (0.418)(*) | " |
| 10 | 4 | 7hl | 56.0 | 8.40 | 0.024 | " |
| 14 | 1 | 8ht | 14.4 | 0.48 | 0.005 | Hammer, Pile closest to pier 8 (30m) |
| 14 | 3 | 7ht | 4.2 | 0.75 | 0.028 | " |

(*)This maximum displacement is obtained according to a lower velocity (4.20mm/sec) than the maximum measured value (5.40mm/sec), and occurs at a frequency of 1.6 Hz.

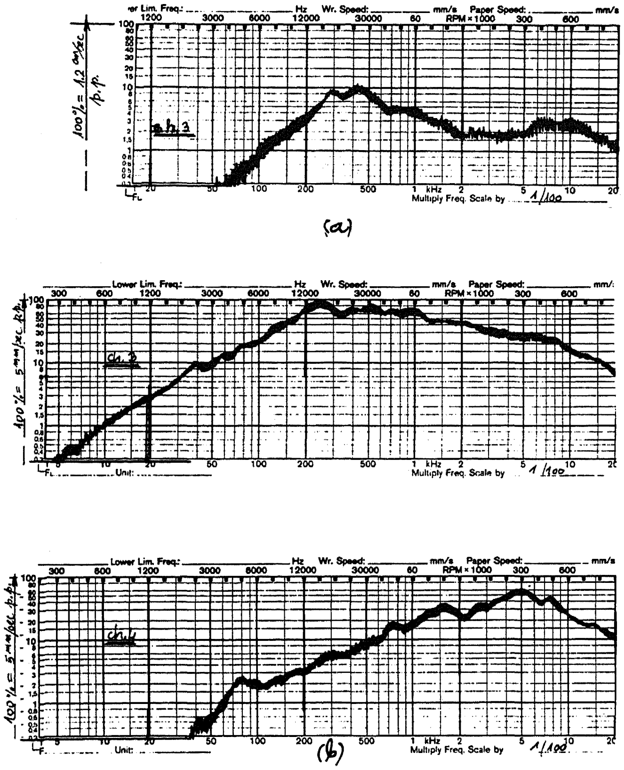


Fig. 8 - Velocity spectrum (third octave):
a) Test 1, channel 3: driving of pile SB-1,
b) Test 2, channels 3,4: passage of train.

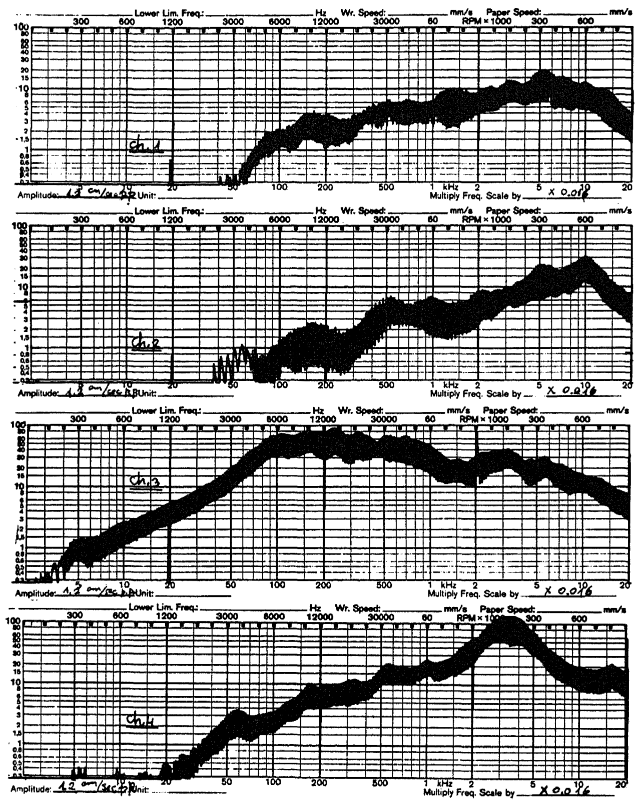


Fig. 9 - Velocity spectrum (third octave): Test 10, channels 1 to 4: passage of a rapid train.

Pier 8 was actually a retaining wall, and not a column type pier, like all other intermediate ones, and as a result its vibrations were always very low.

Spectrum analysis (third octave) of the measured velocities yielded the frequencies of the vibrations and permitted the calculation of the pier head displacement amplitudes (Table 2). In most cases the spectrum had only one peak velocity. However, that of Test 10, Channel 3 (Fig. 9), was rather a type of "plateau", and could be interpreted to show a peak at 4 Hz. In this particular case, a lower velocity, at a lower frequency of 1.6 Hz, yielded displacement of $\pm$0.418mm instead of $\pm$0.215mm, as computed from the peak velocity.

Analysis has shown that the peak transverse amplitude at pier head obtained during the passage of a rapid train were at least 10 times higher than during pile driving operation. Considering an amplification factor of 2 of the vibration at the pier head (see Fig. 3,a), the ground vibration at the pier foot should be half of it, or $\pm$0.209mm. This result is in good agreement with amplitude limits of foundation block, which do not cause any damage to the structure, as mentioned in the British Code of Practice, i.e. $\pm$0.200mm, up to 20 Hz. Other codes (e.g. DIN 4150), specify a limit velocity of 20mm/sec up to 10 Hz, as one which generally does not produce any damage to a structure. The maximum velocity, as measured at pier head 7, was $\pm$8.40m/sec or 16.80mm/sec peak to peak, which does not yet endanger the bridge.

## CONCLUSIONS

1. Vibrations recorded at pier heads during pile driving operation, were much higher in the lateral direction of the bridge than in its longitudinal direction, and pier head vibration was almost twice as large as that measured at the pier base. The natural frequency of the bridge in the transverse direction was 3 Hz, and the critical damping ratio of the system, about 1.5%.

2. Train passage caused lateral vibration at pier heads which was at least ten times bigger than the one caused by pile driving, namely $\pm$0.418mm, or $\pm$0.836mm peak to peak.

3. Neither velocity nor displacement obtained exceeded the limit values prescribed by the codes, thus indicating that no structural change is anticipated to the bridge during pile driving or even rapid train passage.

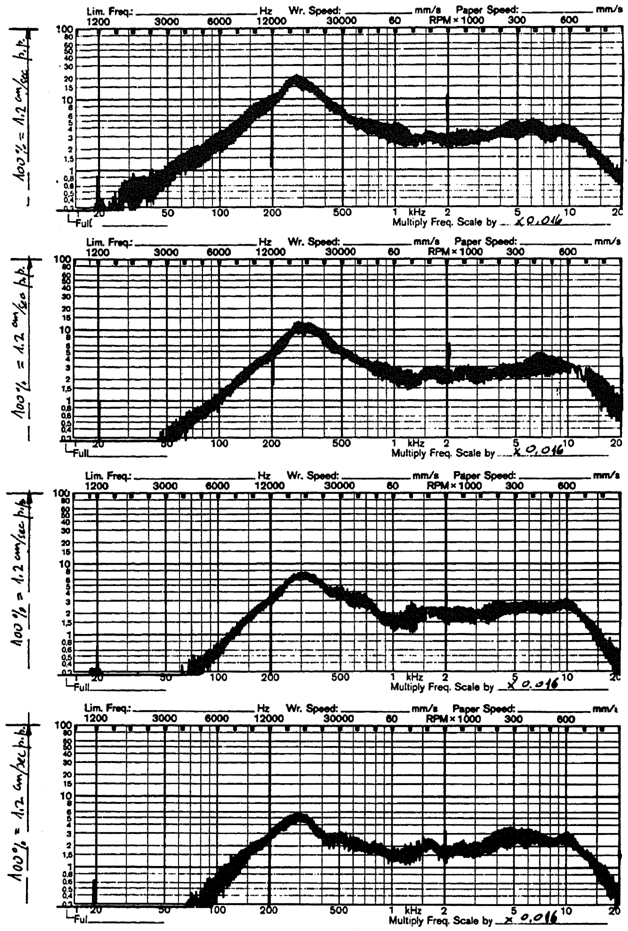


Fig. 10 - Velocity spectrum (third octave), Tests 6 to 9, channel 3: driving of special pile, at various depths.

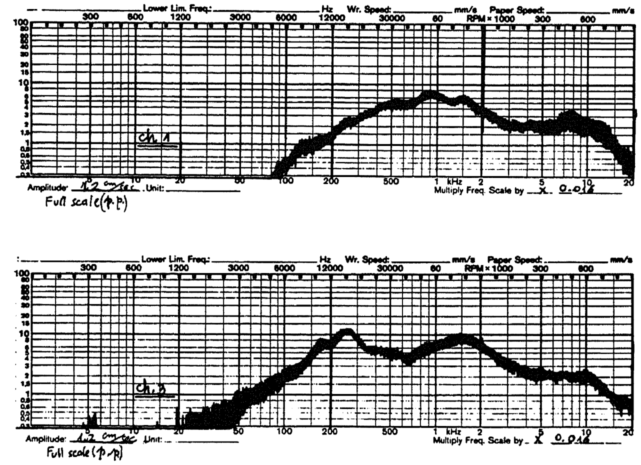


Fig. 11 - Velocity spectrum (third octave), Test 14, channels 1 and 3: driving of closest pile at 30m deep.

4. Thanks to the field measurements, this stage of the Ayalon Project may be regarded safe, as far as causing damage to the existing bridge is concerned.

REFERENCES

British Code of Practice - CP 2012, Part 1, 1974.

Deutsche Norm - DIN 4150, Part 3, May 1986.