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THE KINGDOM OF THAILAND

REPORT ON
THE RAMA VI BRIDGE
REHABILITATION PROJECT

DECEMBER, 1982

JAPAN INTERNATIONAL COOPERATION AGENCY
(JICA)

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PREFACE

In response to the request of the Government of the Kingdom of Thailand, the Government of Japan decided to conduct a study on the Rama VI Bridge Rehabilitation Project in the Kingdom of Thailand and entrusted it to the Japan International Cooperation Agency (JICA). The JICA sent to Thailand a survey team headed by Mr. Toshiyo Nobusawa several times in the period from January to October, 1982, under the guidance of the Supervisory Committee chaired by Mr. Hiroshi Tsuboi of the Ministry of Transport.

The team exchanged views with the officials concerned of the Government of Thailand and the State Railway of Thailand the Project, and conducted a field survey. After the team returned to Japan, further studies were made and the present report has been prepared.

I hope that this report will serve for the development of the Project and contribute to the promotion of friendly relations between our two countries.

I wish to express my deep appreciation to the officials concerned of the Government of Thailand and the State Railway of Thailand for their close cooperation extended to the team.

December, 1982

Keisuke Arita

Keisuke Arita
President

Japan International Cooperation Agency

CONTENTS

CONTENTS

APPENDIXES

1. Site Survey Data
   - 1-1 Soil survey data
   - 1-2 Vibration survey report
   - 1-3 Subaqueous survey report
   - 1-4 Land surveying data

2. Data on Deterioration and Future Forecast Analysis
   - 2-1 Dynamic analysis report
   - 2-2 Static analysis report on deterioration
   - 2-3 Study report on long rail conversion on the bridge

3. Required Data for Rehabilitation Measures
   - 3-1 Repair for Piers C & D
     - (1) Design calculation sheet
     - (2) Bill of quantities
     - (3) Construction plan
     - (4) Particular specification
     - (5) Construction price list
     - (6) Design drawing
   - 3-2 Resetting of shoes
     - (1) Design calculation sheet
     - (2) Bill of quantities
     - (3) Construction plan
     - (4) Particular specification
     - (5) Construction price list
     - (6) Design drawing

SUMMARY AND CONCLUSION

SUMMARY AND CONCLUSION

Mentioned hereunder are the summary and conclusion on the result of survey on the rehabilitation project for RAMA VI Bridge in Thailand.

1. Survey Purpose and Outline

1-1 Background behind implementation of survey

RAMA VI Bridge is situated at the north of the city of Bangkok and is the largest throughout the SRT's network with a maximum span of 120 m and total length of 445 m as the bridge of combined use for both railway (single track) and highway (two vehicle lanes).

In the railway network, it holds a position on the southern line of the main route connecting the Metropolitan Area of Bangkok and the southern part of Thailand. As the vehicle traffic means, the bridge is used as a part of the ring road in Bangkok, playing a very vital role in the metropolitan traffic.

The bridge was constructed originally in 1927 and fell into collapse by bombing during the World War II (1939 ∿ 1945). At the same time, bridge piers and abutments were damaged, and the bridge could no longer be used.

The rehabilitation work of this bridge continued for a period of 1948 ∿ 1950. Only the truss of superstructure was reconstructed with new bridge structure while piers and abutments were rehabilitated with enclosure of reinforced concrete.

With a time elapse of about 30 years since its completion of rehabilitation, today Public Works Department of Thailand contemplates new construction of the road bridge with six (6) lanes as a part of the ring road in Bangkok, in place of RAMA VI Bridge with the two (2)-lane road, on the upstream side of the existing bridge. After completion of the new bridge constuction, the SRT will remove the present two (2)-lane road out of the bridge and reconstruct into a single track railway line, so that RAMA VI Bridge after reconstruction may be used to serve as the double-track railway bridge in its entirely.

The planning and designing services for this double track project were requested by the Chief Civil Engineer of SRT to Mr. Torigoe as the JICA-assigned expert for a period of 1980 to 1981 and detailed design drawings were completed. In the process of designing, Mr. Torigoe presumed some problems including structural deterioration in the substructure and pointed out necessity to go into detailed survey.

Later, this matter was taken over by Mr. Ohtaka as the successor to Mr. Torigoe. Hence, for a period from June to July 1981 the survey was conducted under the guidance of Mr. Ohtaka to assess the deteriorated conditions of RAMA VI Bridge.

As the result, the conditions required continuation of the deterioration survey in further details and the latest survey started in February 1982 in accordance with the SRT's request.

1-2 Summary of deterioration and immediate measures before latest survey

The following are the summaries dealing with the result of surveys conducted to check deterioration of RAMA VI Bridge under the guidance of Mr. Ohtaka for a period of June to July, 1981 and immediate measures taken to improve the deteriorated conditions by due reference to the survey result.

RAMA VI Bridge presented such deteriorations that Abutments A and F on both sides were moved forward by about 50 to 60 mm and movable shoes of Piers B and D were tilted toward Bangkok by about 50 to 30 mm respectively as may be so determined from relative displacement between upper and lower shoes. The link (69 mm at movability) of the suspended truss was moved already up to its maximum possible limit toward Bangkok and the central span appeared to have been wide spreading. Damage took place over the end face of concrete pavement at the expansion joint on the road side. In addition to those, vibrations were felt against human bodies at the crown of Pier C as if its amplitude would be 10 to 15 mm.

As the result, the following immediate measures were taken actually or reviewed in accordance with the advices given by Mr. Ohtaka.

(1) Restriction of train speed (20 km per hour for all trains)

(2) Regulations of road traffic including traffic ban to large-size vehicles or their speed limit (repair and improvement of road pavement taking advantage of the road being closed temporarily to traffic and traffic ban upon large vehicles after completion of improvement works)

(3) Immediate repair of road expansion joint and track maintenance

(4) Study on traffic suspension and other necessary measures against any case of emergency

1-3 Summary of survey

Prior to start of the site survey, various studies were conducted in Japan by due reference to the Report on Deterioration of RAMA VI Bridge as presented by Mr. Ohtaka and necessary site survey items for clarification of the causes for deterioration of RAMA VI Bridge were summarized for formulation.

According to the aforestated Report, it is noted that vibrations arising from Pier C are extraordinarily large in amplitude with appearance of deterioration at shoes and links of the bridge superstructure.

The following items were cited as the causes for deterioration of RAMA VI Bridge as presumed in Japan:

(1) Due to structural defect of Pier C

(2) Due to localized scouring around Pier C

(3) Due to unequal earth pressure

(4) Due to compound causes

As approaches to clarify the causes for deterioration and to provide necessary measures for repair and maintenance, the following survey items were proposed:

(1) Survey on geology and soil conditions

(2) Survey on river-bed scouring

(3) Vibration measurement

(4) Subaqueous survey by divers

(5) Land surveying

(6) Excavation survey in circumference of Pier B

(7) Survey related to design and construction

After surveys conducted at the site for one month period of March in 1982, the clarified facts were summarized as follows:

From the result of survey on geology and soil conditions it appeared that no particular problems would be involved in the stability of foundation because of deep reach of Piers B thru E down to the foundation assured of sufficient bearing power. However, abutments A and F on both sides appeared to have been rooted to a relatively insufficient depth, as compared with Piers B thru E, and presented some problems in regard to bearing power in the horizontal direction.

The result of survey on river-bed scouring revealed that there appeared no such localized scouring as should, in particular, give rise to any problems.

The result of vibration measurement turned out to be a small figure of 0.64 mm for maximum horizontal displacement at the crown of Pier C. No particular problems were anticipated from this measured result as the horizontal displacement for the pier of this type. It should be noted, however, that this matter should not be discussed only to seek a general conclusion because the site conditions including roads and others as of March in 1982, in which vibration was measured, were changed from those corresponding conditions in June 1981 when this vibration was initially taken up as a problem by Mr. Ohtaka. Therefore, the result of the study will be stated later in this Report mainly by reference to the analysis result obtained from vibration simulation in Japan.

From the result of subaqueous survey by divers it was revealed that Piers C and D had hollows and cracks near the boundary between the upper and middle stages and Pier C was eccentric by about 1 m at its upper and middle stages on the upstream side. Although those drawbacks were not immediately influential to the growing deterioration, they appeared with necessity to take any suitable means of repair and remedy.

The result of land surveying presented dimensional difference or discrepancy at some parts by comparative check with the original design drawing. However, whether or not such difference or discrepancy was deviated from the design drawing back at the early stage of construction could not be traced clearly for lack of any recorded data in details from the beginning of construction.

Abutments A and F on both sides are moved forward under the unequal earth pressure from the back of abutment.

Assuming that each pier would have been erected correctly as per the design drawing available now, it would appear that Piers B thru D should be tilted by about 19 ∿ 55 mm toward Bangkok and Pier E, by about 47 mm toward Haad Yai respectively as estimated on a calculated basis.

Displacement of shoes and links was varied within a diversified range of 15 to 76 mm depending upon where they were. This result was nearly same as obtained from the result of survey in June 1981. Although this extent of displacement may not immediately affect the growing paces of deterioration, it would impede the normal functioning inherent to shoes and links and it would be desirable that the present condition should be corrected not later than the scheduled timing for conversion into double track.

As the result of excavation survey in circumference of Pier B, the workmanship near the boundary between upper and middle stage of Pier B was found satisfactorily good.

On the basis of results obtained from such various site surveys as aforestated, analysis on the cause for deterioration, along with analysis on the forecast of such deterioration, was conducted to further details in Japan.

2. Causes for Deterioration of RAMA VI Bridge

In an attempt to clarify the causes for bridge structure deterioration, such deterioration was assumed to be attributable to three different causes such as the cause on a long-term basis, on a short-time basis and from the initial stage of construction, after due reference to the results of site surveys and their subsequent analysis results obtained later in Japan.

2-1 Deterioration due to causes over long term

(1) Abutment A on the Bangkok side began to show the moving phenomenon to the front, immediately after completion of the rehabilitation work in 1950, probably because of unequal earth pressure resulting from banking on the rear side of the abutment. At the reach of its horizontal displacement up to 18 cm the repair work was done in 1965 by resetting of shoes. However, the movement was still noticed. Therefore, the backfill on Bangkok side was replaced by composite girders at a distance about 500 m from the abutment. With further development of deterioration another horizontal displacement of about 7 cm was added to such an extent that the girder end of superstructure has entered into contact with the abutment parapet.

Thus, deterioration may be considered to be attributable to the causes over a long term settlement due to consolidation of the embankment estimated at about 2 m, sliding failure of soil during construction and after completion of the embankment, and unequal earth pressure arising from banking of soil on the rear side.

(2) Abutment F on the Haad Yai side is also moved to the front in a horizontal displacement of about 7 cm, though not so conspicuously deformed as Abutment A was, because the foundation of its upper soft soil layer still provides favorable conditions as compared with that for Abutment A.

(3) Since partial removal of the rear side embankment for Abutment A, the loading condition has been in unbalance over a long period when viewed from the total structure of RAMA VI Bridge.

That is to say, the way of conception that the whole superstructure would serve as if it were a strut against unequal earth pressure from the rear side of Abutment F, in the judgement of complexed interaction between concrete slab, stringers and shoes, would be closer to the reality as compared with the other way of conception, that the shoe for Abutment F was acting against unequal earth pressure individually because of its movability.

Likewise, the phenomenon of about 10 mm move of Abutment F after improvement of road pavement can also be easily explained from the above conception.

Meanwhile, the movable shoe is showing some degree of ability to catch up with displacement due to transitional load like train load during its passage.

With regard to the link, it is not necessarily in a condition to give justifiable reason for this observed phenomenon. As stated later, it may be attributable to the cause in a short term and the cause arising from the initial phase of construction.

(4) As made clear from the result of diver's exploitation, the upper part of Pier C is eccentric on its upstream side by about 1 m toward Bangkok. It is further clarified that horizontal displacement of about 1 cm will arise at the crown of Pier C from eccentric load resulting from such eccentricity. Although such displacement is still limited to a small degree, it exists as long duration load and is considered as a factor to deterioration of the bridge.

(5) The result of river-bed scouring survey reveals that there exists hardly any difference as compared with the condition at the initial stage of construction. Nothing particular is contained in the cause for deterioration to derive from localized scouring around Pier C, contrary to the original assumption in Japan.

(6) Except otherwise under such conditions as stated earlier, there is not observed any possibility, in particular, for causing deterioration due to the cause accumulated over a long period.

2-2 Deterioration due to causes within short term

(1) As the cause for deterioration within a short term, there is to be considered the extremely deteriorated road condition for the period starting from asphalt removal from pavement in February 1981 till traffic closure in October 1981.

To further explain the reason for assumption, it may be well conceivable that by exposure of the uneven surface of concrete slab after removal of surface asphalt pavement, the end face of concrete slab might have been smashed by direct impact from passing vehicles of large size, though protected with steel plate covering, thus acting against the link toward expansion by force of smashing impact.

(2) No accurate recorded data is available to determine the displacement at the link. However, such displacement may be estimated at 10 to 15 mm from peel-off of paint recently coated.

(3) Road vehicle traffic is featured by vehicles, mostly with load on the up lane toward Bangkok and by empty vehicles on the opposite lane toward Haad Yai. However, the vehicle loading condition still remains unknown, although the data dealing with trip survey per unit hour is available. As a general tendency it appears that vehicles are overloaded to a considerable extent.

All those things considered, it seems that vehicles proceeding to Bangkok are conspicuous of their load, such as braking load, which acts in the horizontal direction.

(4) From the result of field vibration measurement in March, 1982 no vibration displacement to be considered as a matter of grave concern was detected. This may be mainly because of the improved road pavement and the restrictive traffic to large vehicles. That is to say, from the result of vibration analysis on the assumed basis of the road condition around August 1981, it is conceivable that there might have been a possibility of causing the vibrating phenomenon sensible to the human body under the specific condition.

In conclusion, therefore, it can be inferred that the vibration sensible to the human body at Pier C, which led to the latest survey, should be a transient phenomenon arising from damage over the end surface of road pavement.

It is furthermore estimated that the horizontal displacement degree at the crown of Pier C during the large vehicle passage under the deteriorated condition was limited only to about 1 to 2 mm, even in the worst case by combination with both wheel base and running speed of the large vehicle of most disadvantage to such displacement.

2-3 Deterioration due to causes originating from initial stage of construction

(1) Difficulty experienced in the site survey was that detailed construction records were not available.

Therefore, although there should naturally exist some unknown factors to determine the exact degree of displacement for clarification of the causes for deterioration it can be observed that the largest discrepancy between the design drawings still available and the actually existing structure is the eccentricity at the upper part of Pier C as referred to in the foregoing item 2-1 (4).

(2) No accurate past record of construction for the link portion is available in any way. Therefore, although any decisive comment must be refrained as far as this portion goes, it can be assumed from the following points that those links might have not been set correctly at their regular positions in all instances.

① The existing bridge structure does not always accord exactly with the design drawings still available.

② The notch of angle members used at the link does not form normal curve. Besides, there exists a difference of 4 mm in notch of angle members between railway side and highway side. This seems to be the old mark of adjustment which might have been made at the initial time of bridge girder erection.

③ Although coated paint is peeled off to a length of 10 to 15 mm, any coating before peel-off still remains unclarified.

(3) From the result of measurement made to see the tilted degree of piers, it is estimated that Piers B ~ D may be tilted about 50 mm to Bangkok. Although this point may be justifiable to some extent, as observed in the foregoing item 2-1 (3), by attribution to the unequal earth pressure influence from the Haad Yai side, it may not be proved with definite justification because of lack of detailed construction records at the initial stage.

It should further be noted that the extent of such structural tilt can in no way be made definite in all cases in relation to the accuracy of measurement.

(4) The result of survey by divers revealed that the subaqueous portions of both Pier C and Pier D had hollows which were assumed to be existing from the initial stage of construction. However, no

particular phenomenon worthy of serious consideration was observed after vibration tests.

That is to say, deterioration is not attributable to any cause due to the structural defect of Pier C which was assumed as one of the deterioration causes as the initial stage of work in Japan.

3. Safety of RAMA VI Bridge

In view of the causes for deterioration, the problems related to the safety of the bridge must be reviewed separately by a short-term vision and by a long-term vision.

3-1 Safety from a short-term vision

(1) To meet immediate needs for safety of RAMA VI Bridge the interim report pointed out that earliest repair of stringers and brackets should be needed, in particular, near the link of the suspend span. As of this date, the repair work has been completed and the problem in this regard has been dissolved accordingly.

(2) Vibrations arising at Pier C from damage on the paved surface of road and also from traffic of large-size vehicles have been dissolved to this date by restriction on the large-vehicle traffic and repair of the road surface.

(3) With regard to the safety for the bridge substructure, it appears that the future structure after doubling of the track as well as the existing structure would not pose any safety problem, as estimated from the large safety factor which is obtained after calculation of stability of the caisson foundation, even though no improvement may be made any more.

(4) The conversion plan into the long-rail system on the bridge for the purpose of diminishing the vibration would be of less significance as may be noted from the results of vibration test and analysis.

For further reference, this Report includes details on necessary conditions for conversion into the long-rail system on the bridge.

(5) In the judgement from various survey results, it is not feared that RAMA VI Bridge would be jeopardized by deterioration at a rapid tempo.

3-2 Safety from a long-term vision

(1) After subaqueous check by divers it was found that the underwater portion of Piers C and D had hollows.
From the results of vibration test and analysis it appeared that there were no problems to be considered as the cause for deterioration, probably due to structural defects of Piers C and D as originally anticipated.

However, it is considered desirable in a long run that repair work should be done for the underwater portions of Piers C and D.

(2) The vibration phenomenon as a trigger to the latest deterioration survey was attributable mainly to the deteriorated paving condition of the road. No particular problem is, however, contained in the improved condition of road pavement as of March 1982. Full maintenance care of roads will be required in the future as well as at present.

(3) From the overall view of the whole RAMA VI Bridge, it is conceivable that the total bridge structure is put under the unbalanced

load condition by embankment at the rear of Abutment F on the Haad Yai side. It is therefore desired that same as was terated for Abutment A on the Bangkok side embankment soil at the rear should be removed.

The removal section should be of about 40 m extension in the longitudinal direction by the desired timing of such removal not later than scheduled conversion into the souble track system.

The timing for construction may be left to the potional judgement by due reference to the result of follow-up studies after deterioration.

(4) Shoes and links show displacement of 15 to 76 mm to hteir normal positions. Since such shoes are placed at their undesirable positions with their declined normal function, resetting of shoes and links is recommended.

From the present status, however, there seems to be no danger of going into further deterioration at raped paces.

The suitable timing for resetting of shoes may be determined from the result of follow-up studies as later informed, probably not later than the scheduled completion of the double track system but after repair of Piers C and D.

(5) There are observed no other points which shoule require serious consideration, besides all those aforestated.

4. Immediate Measures to be Taken

It seems that the vibration problem under the latest survey may be a transient phenomenon which took place because of special circumstances on the road side in the historical process over 30 years after reconstruction of the existing RAMA VI Bridge.

Although from the present condition of consolidation over the upper soft soil foundation, the paces of progress into future deterioration would be very slow, it is recommended that follow-up studies as itemized hereunder should be conducted for the period starting from the present stage until the scheduled completion of the double-track system so as to make the result of studies available as the referential data.

(1) To measure the progressed degree of inclination by use of the inclinometer installed at each crown of Piers C and D.

(2) To measure horizontal displacement of shoes and link.

(3) To measure the gap between parapet of Abutment F and girder edge.

(4) To repeat measurements of (1) thru (3) above at a specified day and hour, about once every other month.

4-1 Train speed restriction

No particular phenomenon containing serious problems was pbserved from the vibration tests conducted at the speed of 20 to 80 km/h.

From actual conditions of deterioration of the underwater portion of Piers C and D, deterioration on Abutments A and F and movement of shoes, it is considered desirable that speed restriction should be lifted after rehabilitation of the bridge. In case of easement of speed restriction before rehabilitation, it is advisable that careful judgement should be made by due reference to the result of follow-up studies of future development of deterioration and the maintenance conditions and that such restriction should be eased on a step-by-step basis.

4-2 Traffic restriction on large vehicles

With regard to traffic restriction on large vehicles on the road, it is considered desirable that careful judgemement should be used, same as on the railway side, by due reference to the follow-up study result on deterioration.

5. Repair and Improvement

The basic conception for repair and improvement is as stated hereunder.

5-1 Repair for Piers C and D

According to the result of visual check by divers on the underwater portion of Piers C and D it was revealed that there existed many hollows and cracks near the boundary between the upper and middle parts of piers.

Therefore, particular emphasis is placed upon reinforcement near the boundary between the upper and middle parts in the planning for repair of piers.

(1) Piers C and D will be reinforced with reinforced concrete covering of about 50 cm thickness around the pier structure.

(2) Piers will not be influenced by any horizontal force of extraordinary magnitude like earthquake. Therefore, it is estimated that the sectional area of the existing pier will be fully resistible against horizontal force at a transitional loading (such as braking load).

(3) Reinforcing bar will therefore be restrained to possible minimum volume.

(4) The repair work is planned for maximum possible saving of manpower because of its operational difficulty under the water.

For placing of reinforced concrete it is planned that the under-water portion will be enclosed with pre-packed concrete while the structure above the water will be enclosed with ordinary concrete. Reinforcing bars will be prefabricated on the ground and be set up in place. The mold of concrete will be built with a panel of about 15 m^2.

5-2 Resetting of shoes

The result of measurement reveals that movable shoes and link have been moved largely, some beyond the possible limit of displacement.

As some of those shoes far out of their normal positions may not serve well with their functional role, it has been planned that those shoes should be reset aright roward normal position.

(1) Those movable shoes at Abutments A and F, Pier B and the link should be reset into their normal positions.

(2) The shoe only at Pier E is moved less than all the rest, having potential to further displacement. Resetting is not necessary.

(3) For resetting of shoes at Abutments A and F, the work will be done in such a way that the upper half of each movable shoe will be moved to set at the center of the lower half by taking apart set bolts of the upper half of shoe from the lower chord of truss and jacking up lateral girder of truss.

(4) For resetting of shoes at Pier B and link, the girder will be jacked up by the lower chord after cutting the welded joint between

the lower half of the fixed shoe at Pier C and the base plate. Then, the movable shoes at Pier B and link will be reset into their correct positions by simultaneous transfer of both anchor truss and suspend truss in a longitudinal direction toward the Haad Yai side.

CHAPTER I

CHAPTER I SITE SURVEY

1. General Summary

This survey was conducted extensively on and from March 1, 1982, following after the preliminary survey previously conducted at the site for about a week starting from January 31, 1982 with the object to summarize required contents of survey into items, which were drafted under the decision to clarify the causes for deterioration of RAMA VI Bridge as well as to seek the way of repair and maintenance, after various pre-investigations in Japan by due reference to the 'Report on Deterioration of RAMA VI Bridge' prepared by Mr. Ohtaka who had been assigned as the expert to the State Railway of Thailand by JICA.

According to the aforestated Report, it is reported that vibrations arising from Pier C are extraordinarily large in amplitude and, besides, deterioration appears at shoes and link of the bridge superstructure. Cited as follows were the causes for deterioration of the bridge as previously assumed in Japan.

(1) Due to defect on the structure of Pier C

(2) Due to localized scouring around Pier C

(3) Due to unequal earth pressure

(4) Due to compound causes

In order to clarify the causes for deterioration the following items of site survey were proposed as necessary approach to repair and maintenance.

(1) Survey on geology and soil conditions

(2) Survey on river-bed scouring

(3) Vibration measurement

(4) Subaqueous survey by divers

(5) Land surveying

(6) Excavation survey in circumference of Pier B

(7) Survey on design and construction

Explanations are given as stated hereunder in the sequential order of those survey activities.

2. Survey on Geology and Soil Conditions

Prior to clarification of the cause for deterioration of RAMA VI Bridge it is basically important to know correctly the foundation conditions bearing both piers and abutments of the bridge. With this view in mind, geological and soil tests of various types were conducted under the criteria of either ASTM or JIS.

(1) Boring and standard penetration tests for each pier and abutment

(2) Laboratory soil tests

1) Unit weight test
2) Unconfined compression test
3) Natural water content test
4) Liquid limit test
5) Plastic limit test
6) Specific gravity test
7) Consolidation test

Full particulars of these outcomes obtained from tests as cited above are annexed as a report to Appendix 1-1.

Here, the soil profile is shown in Fig. 1-3 as a part of representative data. From Fig. 1-3 it is made clear that Piers B through E stretch down to such a depth that the bearing strength of sufficiency can be fully assured as mentioned in CHAPTER II of this Report.

At each of Abutments A and F, it appears that piers extend their bases to a sufficient depth assuring the bearing strength in the vertical direction but to an insufficient depth in respect of the bearing strength working in the horizontal direction. Therefore, this is evidently a cause of deterioration for each abutment because of unequal earth pressure in rear side soil of Abutments A and F.

3. Survey on River-bed Scouring

On the assumption that the localized scour in the river-bed may be one of the causes for deterioration of RAMA VI Bridge, survey on the river-bed scouring around Piers C and D were conducted by plumbing from the boat or by divers. However, the result revealed that nothing particular was observed as shown in Fig. 1-4 and 1-5 respectively.

From this proven result it was made clear that such localized scouring should not be considered as a cause of deterioration.

Detailed data, together with the result of soil survey, are included in Appendix 1-1.

4. Vibration Measurement

The latest particular attention to deterioration of RAMA VI Bridge was drawn initially by vibrations perceived to the human body at Pier C. To probe into the problem, vibration measurement was made mainly at Pier C and other piers, abutments and superstructures.

Main features for vibration measurements are as follows:

(1) Train speed:
Four (4) different speeds of 20, 40, 60 and 80 km per hour

(2) Train classification and number:
Eight (8) passenger trains, three (3) freight trains

(3) Road traffic:
When the bridge is closed to traffic, and when the bridge is open for traffic (80 km/h only)

(4) Measured portions:
Superstructure and substructure of RAMA VI Bridge

(5) Measuring point:
Twenty-one (21) points

Typical results of vibration measurement are shown in Fig. 1-6 and Table 1-1. Details are as shown in Appendix 1-2.

As noted from Table 1-1, the maximum among all the measured values was no more than 0.64 mm in amplitude at the horizontal displacement, as measured at the crown of Pier D. The result of observation after these measurements is stated in details in CHAPTER II.

5. Subaqueous Survey by Divers

All the subaqueous survey on Piers C and D and the river-bed condition was conducted by use of the underwater TV camera and the monitoring TV as well as by employment of divers.

Results are partly shown in Figs. 1-7 and 1-8.

From Fig. 1-7 it is noted that an eccentricity of about 1 m exists on the upstream side of Pier C and Fig. 1-8 shows existence of hollows in some portions of Pier D.

Those results of observation suggest necessity of repair works on the underwater structure of Piers C and D. However, as mentioned in CHAPTER II, it appears that those existing hollows are of no grave concern in particular with regard to the vibration problem.

The report in details on this matter is shown in Appendix 1-3.

6. Land Surveying

Total extension of land surveying reached a length of about 850 m by inclusion of RAMA VI Bridge and its connected access sections of embankment. The result is as shown in Appendix 1-4.

Notwithstanding the necessity to refer back to the original detailed data available at the initial stage of construction in order to check and examine the result of surveying, the Bridge under this project lacks its relevant data. Even the design drawings still in survival would be only those prepared at the early designing stage, which may apparently remain partially unrevised or uncorrected though some points should have required revision or correction from the site conditions during construction.

Therefore, the measured data herein are no other else than the result of measurement made in March 1982, for which there is no definite evidence to determine if those data reflect different conditions from those at the time of construction.

(1) Level surveying

The result of level surveying is as shown in Fig. 1-9 and reveals that Abutments A and F on both sides are sagged down in a shape of convex.

Abutments A and F are rooted into a relatively shallow depth as compared with Piers B thru E. Nevertheless, it may be generally difficult, judging from the existing foundation conditions beneath Abutments A and F, to infer that the difference in elevation between Piers C and D and Abutments A and F denotes the settlement of Abutments A and F. That is to say, it is inferable that the present difference between Abutments A and F and Piers C and D should constitute a total sum of the original difference at the time of construction plus the difference arisen from consolidation and settlement of embankment at a later date after completion.

Whilst the land subsidence over a wide city area due to pumping-up of ground water has become a serious problem now in Bangkok, it may not be a matter of so grave concern to RAMA VI Bridge concerned. Even though problems may arise easily in Sections A ∿ B and E ∿ F, when viewed from such differences as may be observed in the depth of foundation and in the load acting upon the foundation, those sections form the simple spanning structure fully endurable against any possible uneven settlement.

(2) Span surveying

Fig. 1-10 shows points of span surveying and Table 1-2 includes its result.

From what are shown in Table 1-2 it is inferable that any design changes might have been incorporated into the design drawings available even today prior to start of construction since there exists considerable degree of difference between the originally designed value and the actually measured result. No detailed data to explain this remain still unknown.

(3) Inclination surveying

On assumption that each pier would have been erected correctly as per design drawings furnished for construction at its early stage, because all the relevant data at the original time of construction remain unavailable as of this date, the degree of tilts lately measured for each Pier B to E, that is to say, horizontal displacement at each crown of Piers B ∿ E could be calculated as follows:

Bangkok←	← 19 mm	← 41 mm	← 55 mm	→ 47 mm	
	B	C	D	E	→ Haad Yai

Now, the movement of Piers B thru D may be well understandable when considered in connection with the movements of upper shoes and links of those Piers. The only one thing beyond a normal level of understanding is that Pier E alone is moved in the opposite direction. The reason behind this irregularity remains unknown as to whether it is caused by any effect from bombing during the World War II or by any workmanship from the beginning of construction.

(4) Details on shows and links

The result of detailed surveying on shoes and links is as shown in Fig. 1-11, showing no large difference from the result measured in June 1981. It is featured by an existing gap between the parapet and the girder end of Abutment F. It is narrowed by 10 mm from the measured date in June 1981 to the date in March 1982. This phenomenon may be explained justifiably by such inference that with dissolution of strutted condition of superatructure because of recovery of both construction and expansion joints to the required conditions after repair and improvement of the road section on the bridge, Abutment F might have been moved foward by unequal earth pressure arising from back soil of that Abutment.

7. Excavation Survey in Circumference of Pier B

The surrounding area allowable for visual check around Pier B was excavated to look into the boundary between the upper and middle parts of the pier, with the object to draw an analogy of the structural condition at corresponding elevations of Piers C and D.

The result of analogy reveals good marks of the work performance. It should be noted, however, that the survey result by such analogy should be used only for referential purpose and superseded, as a matter of fact, by the data obtained from the subaqueous survey by divers.

8. Design and Construction Survey

The survey was conducted together with collection of various data on necessary design, construction and cost estimate for rehabilitation of RAMA VI Bridge.

This Report does not specifically indicate each individual item of such data. However, the rehabilitation plan was worked out by due reference to those data.

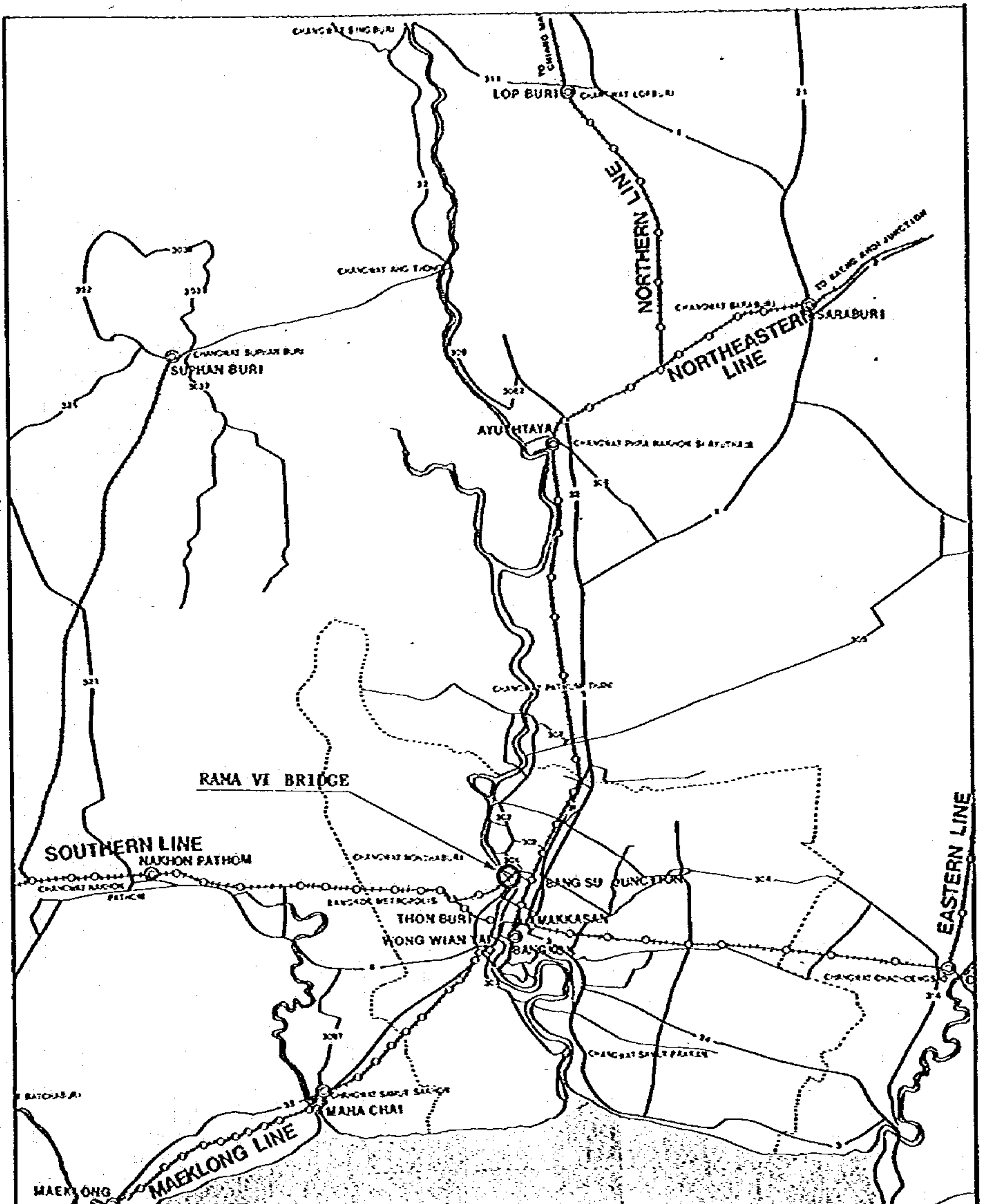
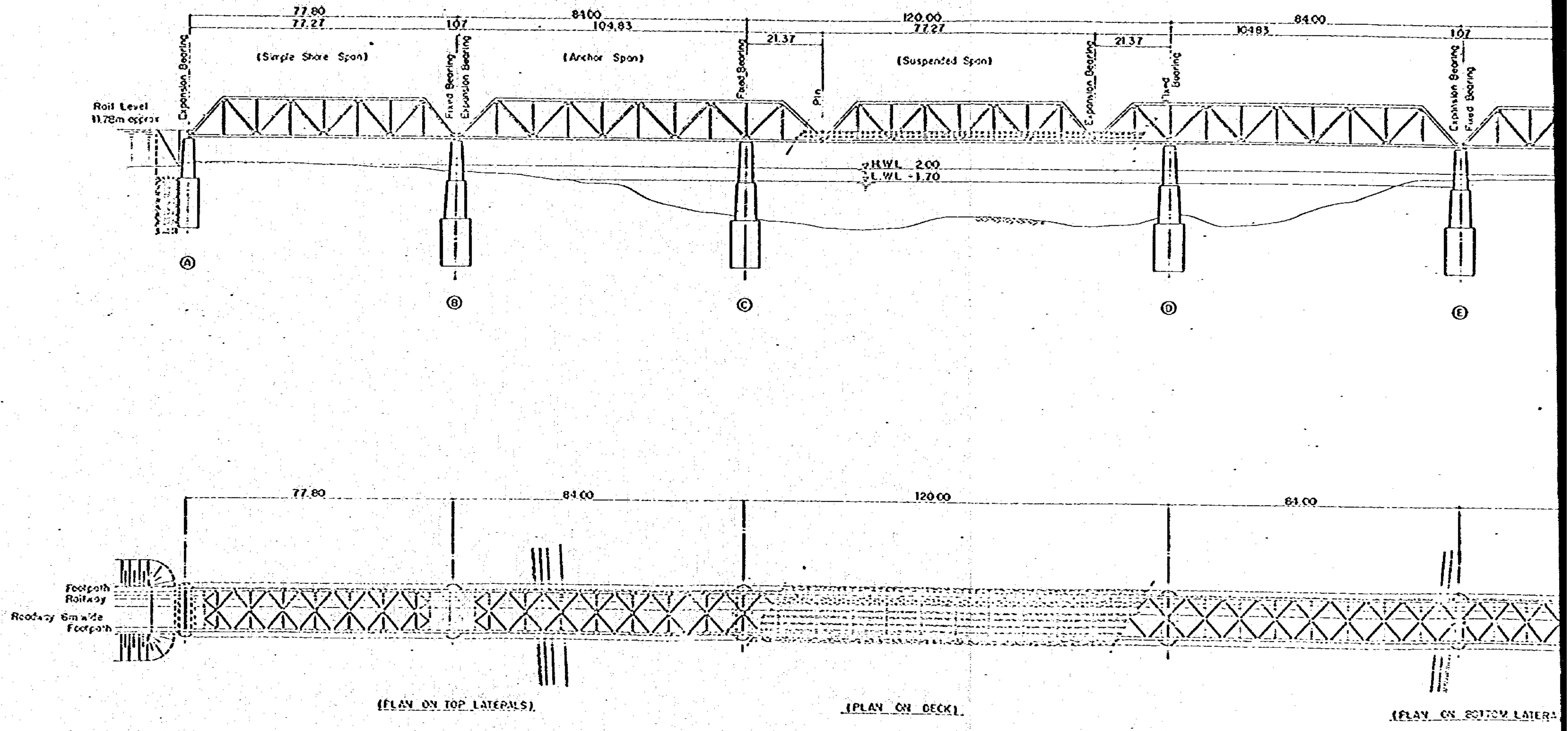


Fig. 1-1 LOCATION OF RAMA VI BRIDGE

Fig. 1-2 GENERAL VIEW OF RAMA VI BRIDGE

S = 1 : 500



VIEW OF RAMA VI BRIDGE

S = 1 : 500

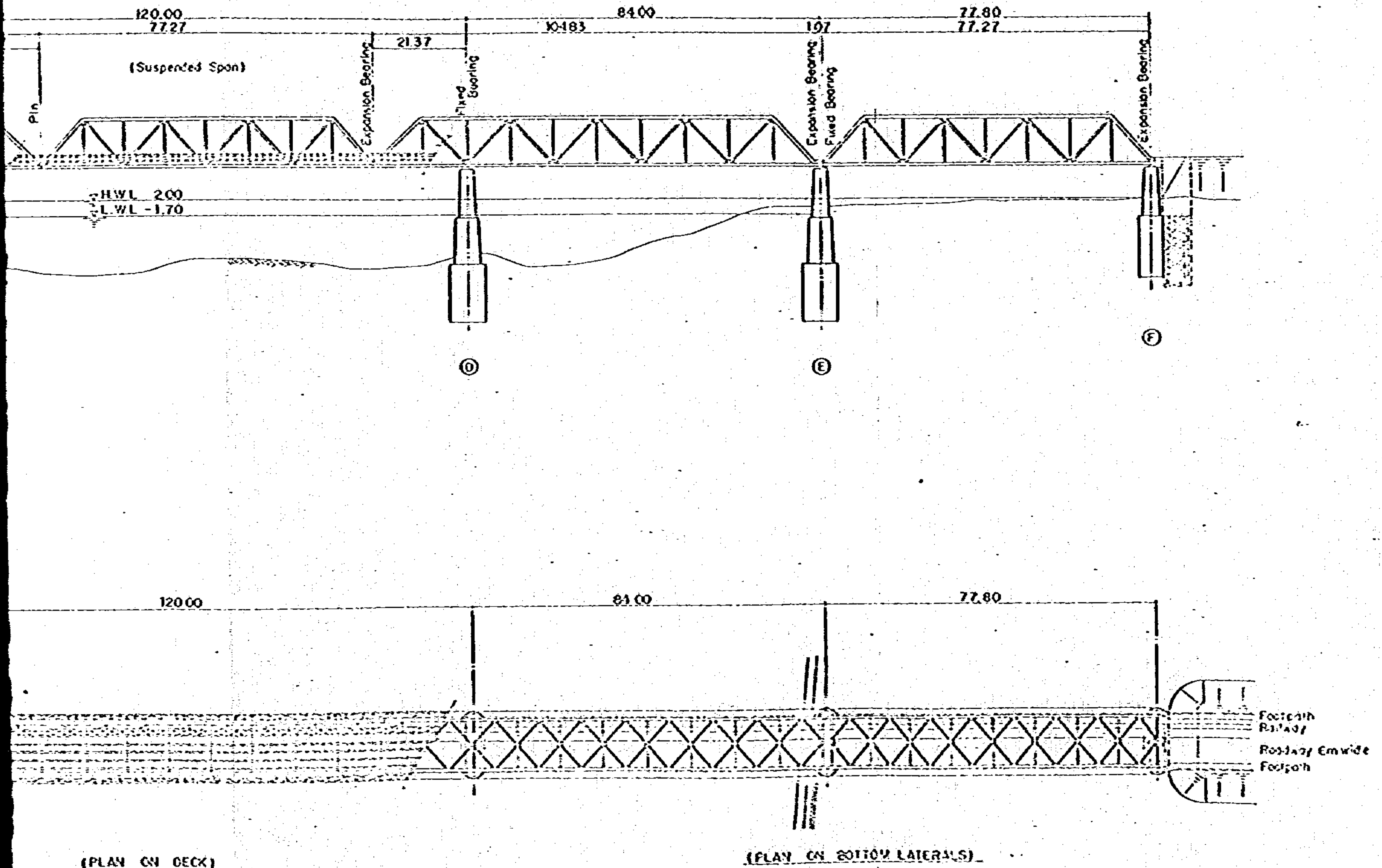


Fig. 1-3 SOIL PROFILE OF RAMA VI BRIDGE

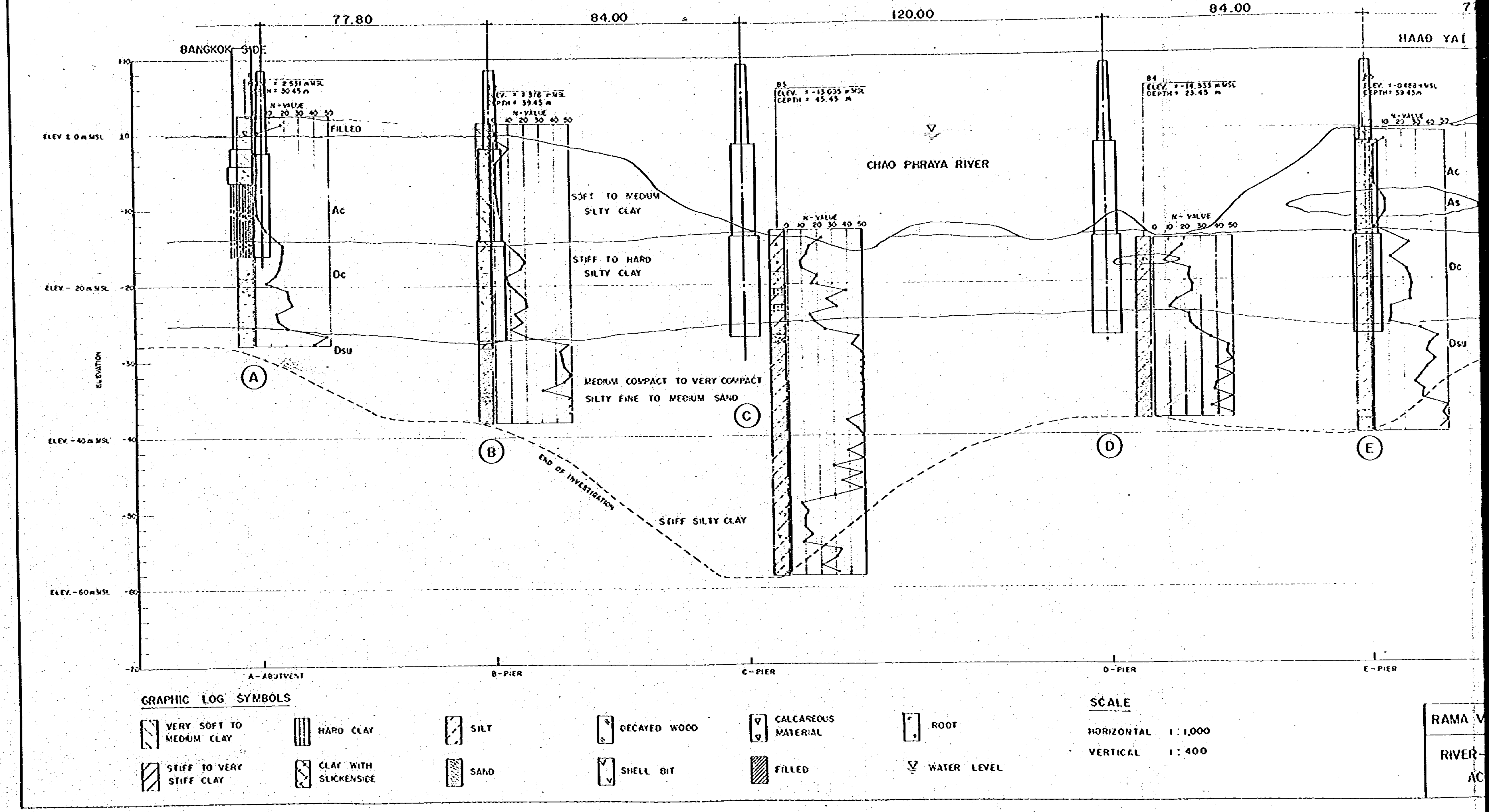


Fig. 1-3 SOIL PROFILE OF RAMA VI BRIDGE

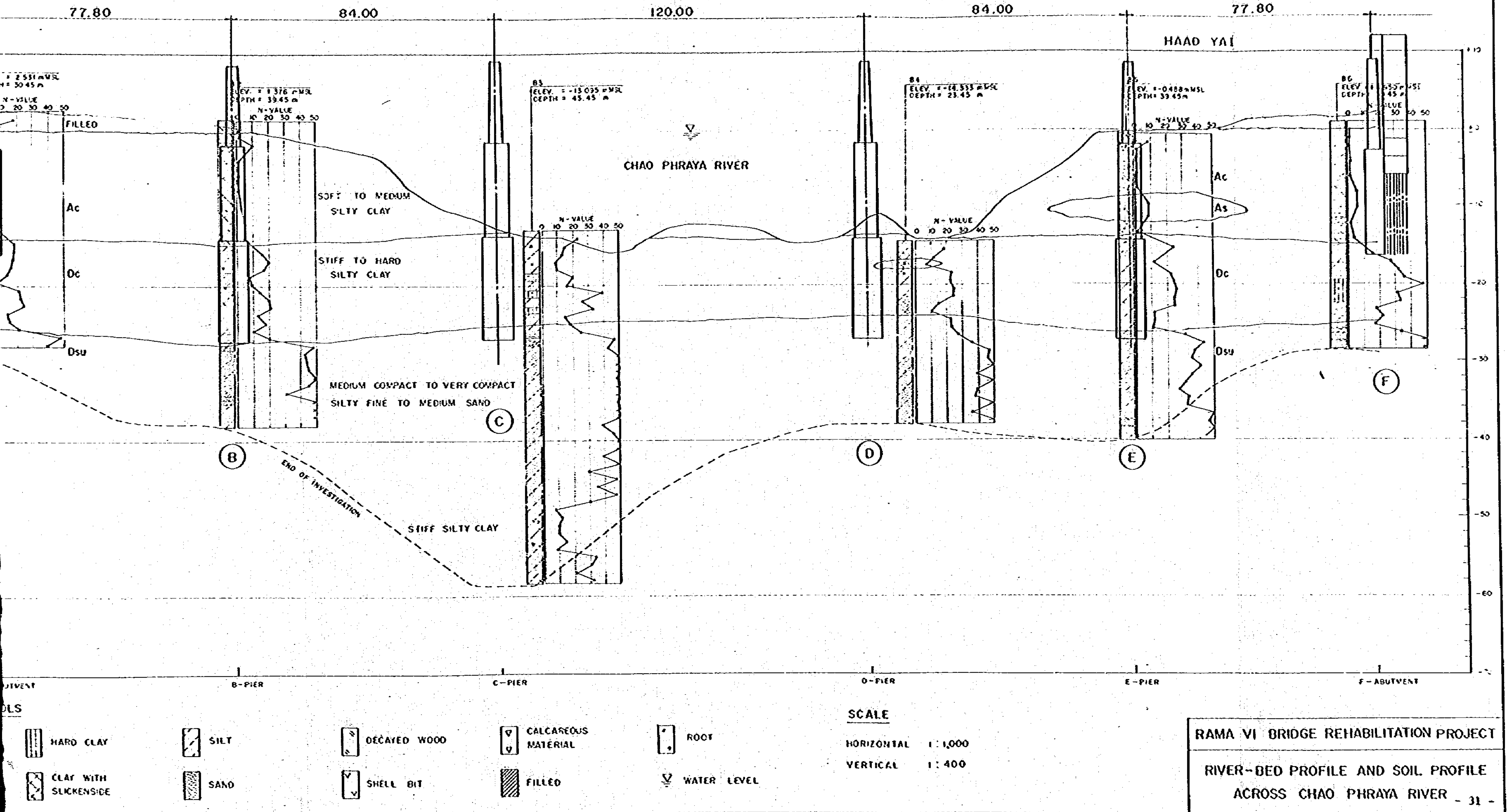
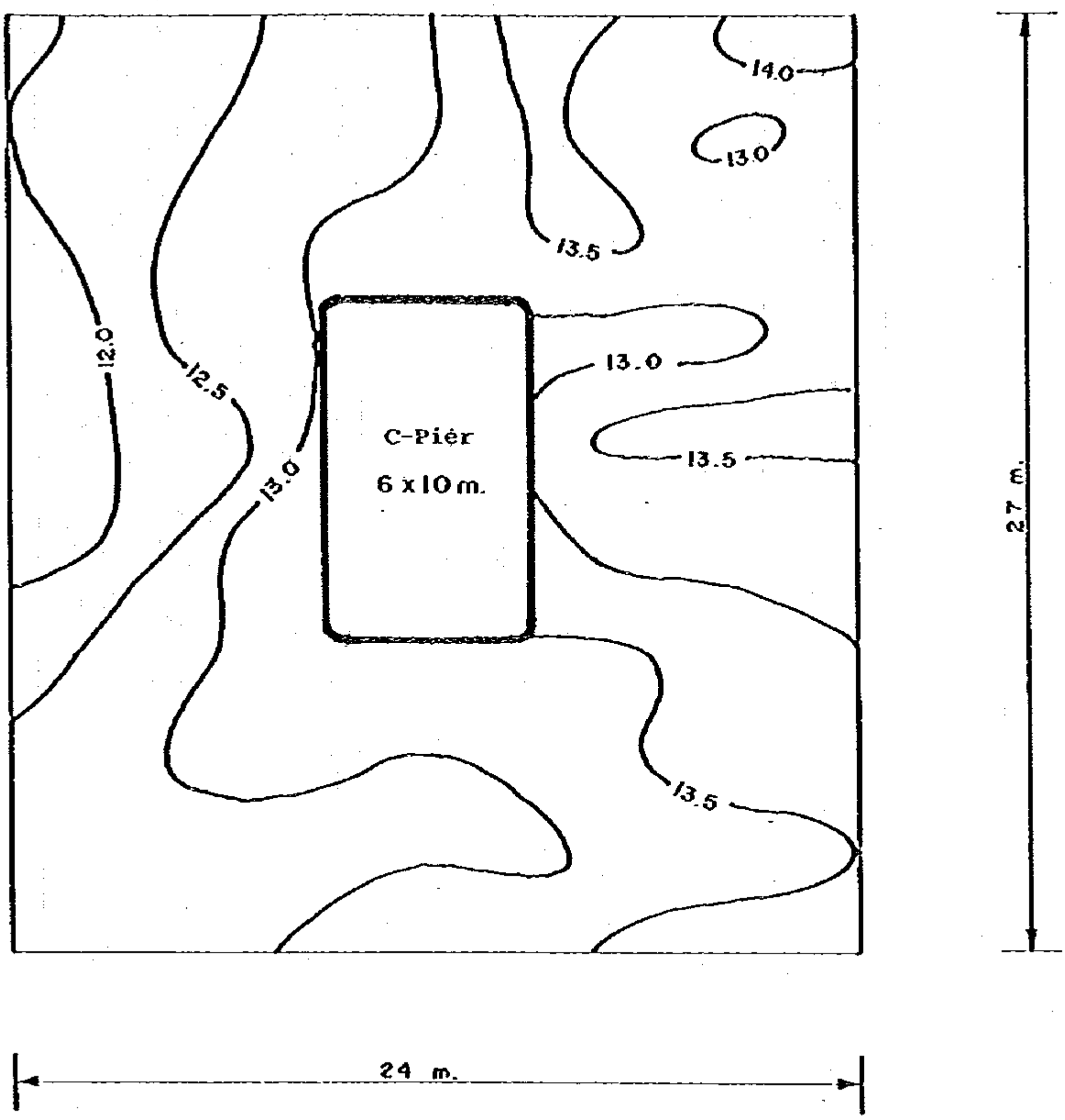


Table 1-1 Results of Vibration Test of Rama VI Bridge

Speed	Train No	Actual Speed	Pier C (top) displacement Y (mm)	Pier C (top) period T (sec)	Pier C (first step) displacement Y (mm)	Pier C (first step) period T (sec)	Pier C (second step) displacement Y (mm)	Pier C (second step) period T (sec)	Pier D (top) displacement Y (mm)	Pier D (top) period T (sec)	Pier D (first step) displacement Y (mm)	Pier D (first step) period T (sec)	Suspended span (centre) displacement Z (mm)	Suspended span (centre) period T (sec)	Remarks type	Remarks classification
20 km/h closed to traffic	20	18	0.09	0.47	0.05	0.47	0.04	0.47	0.14	0.47	0.06	0.47	1.72	0.47	Passenger	Up train
	43	25	0.12	0.48	0.06	0.48	0.04	0.48	0.12	0.49	0.05	0.49	1.20	0.49	Passenger	Down train
	758	15	0.34	0.79	0.18	0.79	0.09	0.79	0.18	0.48	0.09	0.48	4.95	0.79	Freight	Up train
40 km/h closed to traffic	20*	26	0.09	0.45	0.05	0.45	0.04	0.45	0.13	0.46	0.05	0.46	1.61	0.52	Passenger	Up train
	43*	21	0.10	0.47	0.05	0.47	0.02	0.47	0.12	0.47	0.06	0.47	1.15	0.47	Passenger	Down train
	730	42	0.15	0.33	0.13	0.33	0.08	0.33	0.22	0.82	0.13	0.82	3.50	0.82	Freight	Up train
60 km/h closed to traffic	20*	56	0.23	0.50	0.11	0.50	0.04	0.50	0.29	0.50	0.13	0.50	2.33	0.77	Passenger	Up train
	43*	58	0.18	0.56	0.08	0.56	0.05	0.56	0.18	0.58	0.07	0.58	2.16	0.55	Passenger	Down train
	729	30	0.57	0.48	0.29	0.48	0.10	0.48	0.64	0.47	0.33	0.47	5.00	0.47	Freight	Down train
	752	45	0.28	0.76	0.15	0.76	0.07	0.76	0.23	0.76	0.07	0.76	4.08	0.76	Freight	Up train
80 km/h closed to traffic	20*	77	0.25	0.47	0.15	0.47	0.05	0.47	0.30	0.47	0.18	0.47	2.69	0.45	Passenger	Up train
	43*	69	0.35	0.49	0.17	0.49	0.07	0.49	0.46	0.49	0.24	0.49	3.00	0.48	Passenger	Down train
	752	39	0.28	0.49	0.15	0.49	0.07	0.49	0.35	0.49	0.19	0.49	3.10	0.50	Freight	Up train
80 km/h open for traffic	20*	81	0.29	0.44	0.17	0.44	0.05	0.44	0.31	0.50	0.18	0.50	2.76	0.47	Passenger	Up train
	43*	75	0.36	0.49	0.18	0.49	0.08	0.49	0.32	0.49	0.18	0.49	2.45	0.49	Passenger	Down train
	741	24	0.39	0.47	0.20	0.47	0.08	0.47	0.46	0.49	0.25	0.49	6.00	0.49	Freight	Down train

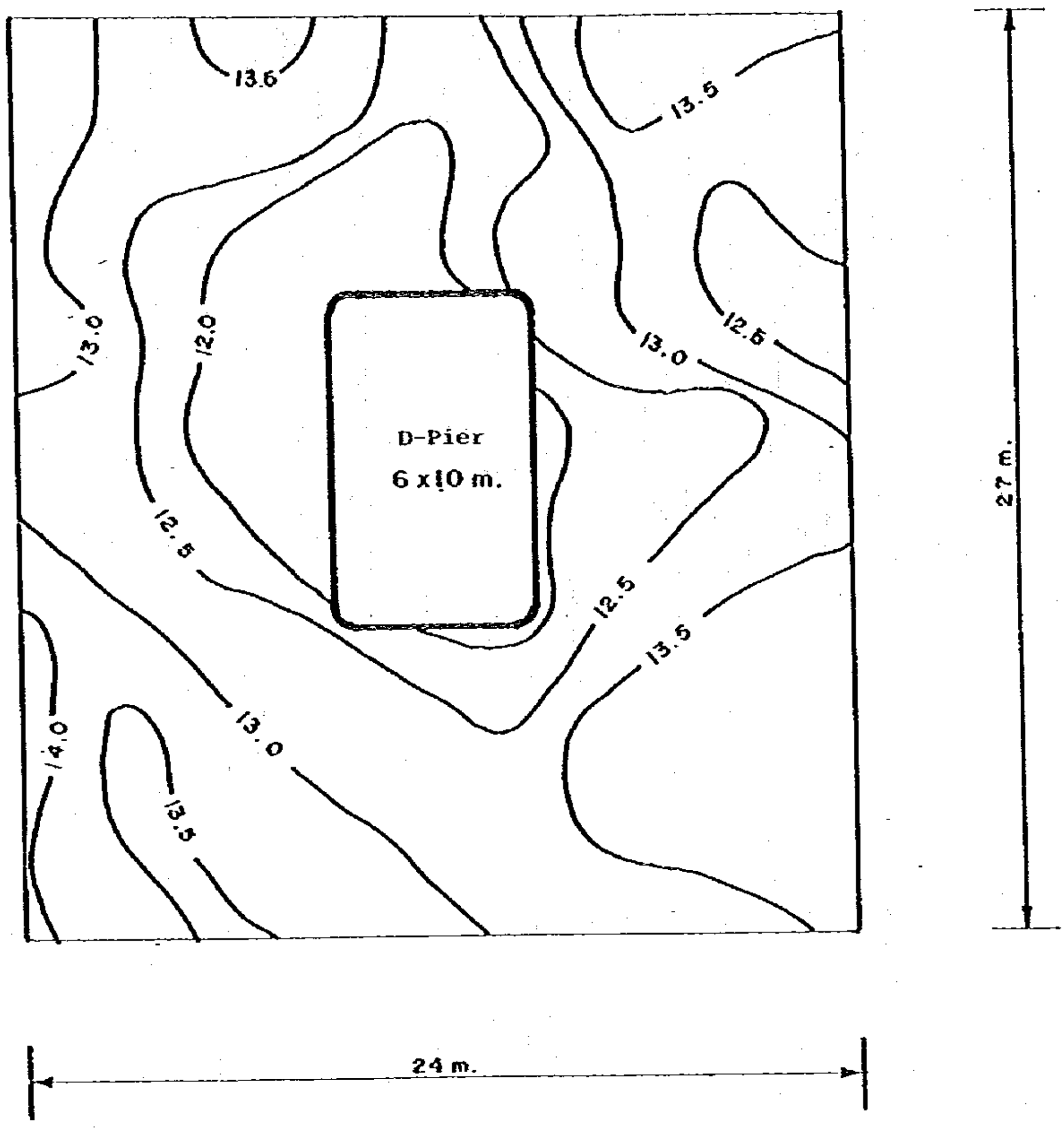
* Brake Down



Note

The Elevation of Contour lines are in m -- MSL

Fig. 1-4 River - Bed Contours at C-Pier



<u>Note</u>

The Elevation of Contour lines are in m – MSL

Fig. 1-5 River - Bed Contours at D-Pier

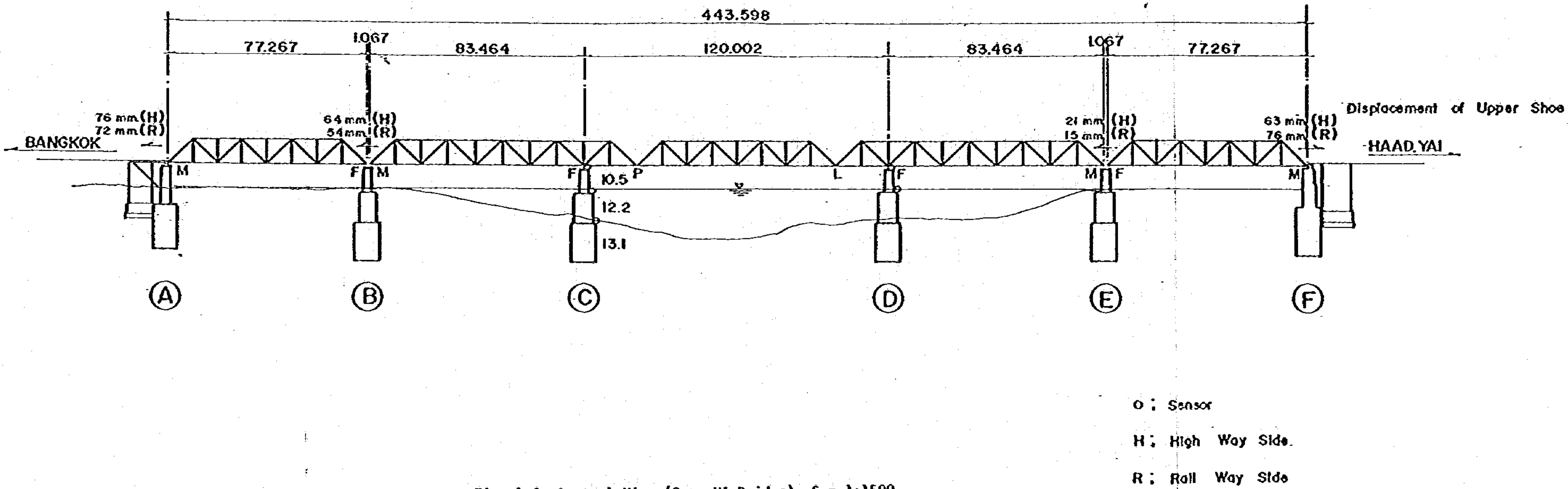


Fig. 1-6 General View (Rama VI Bridge) S = 1:1500

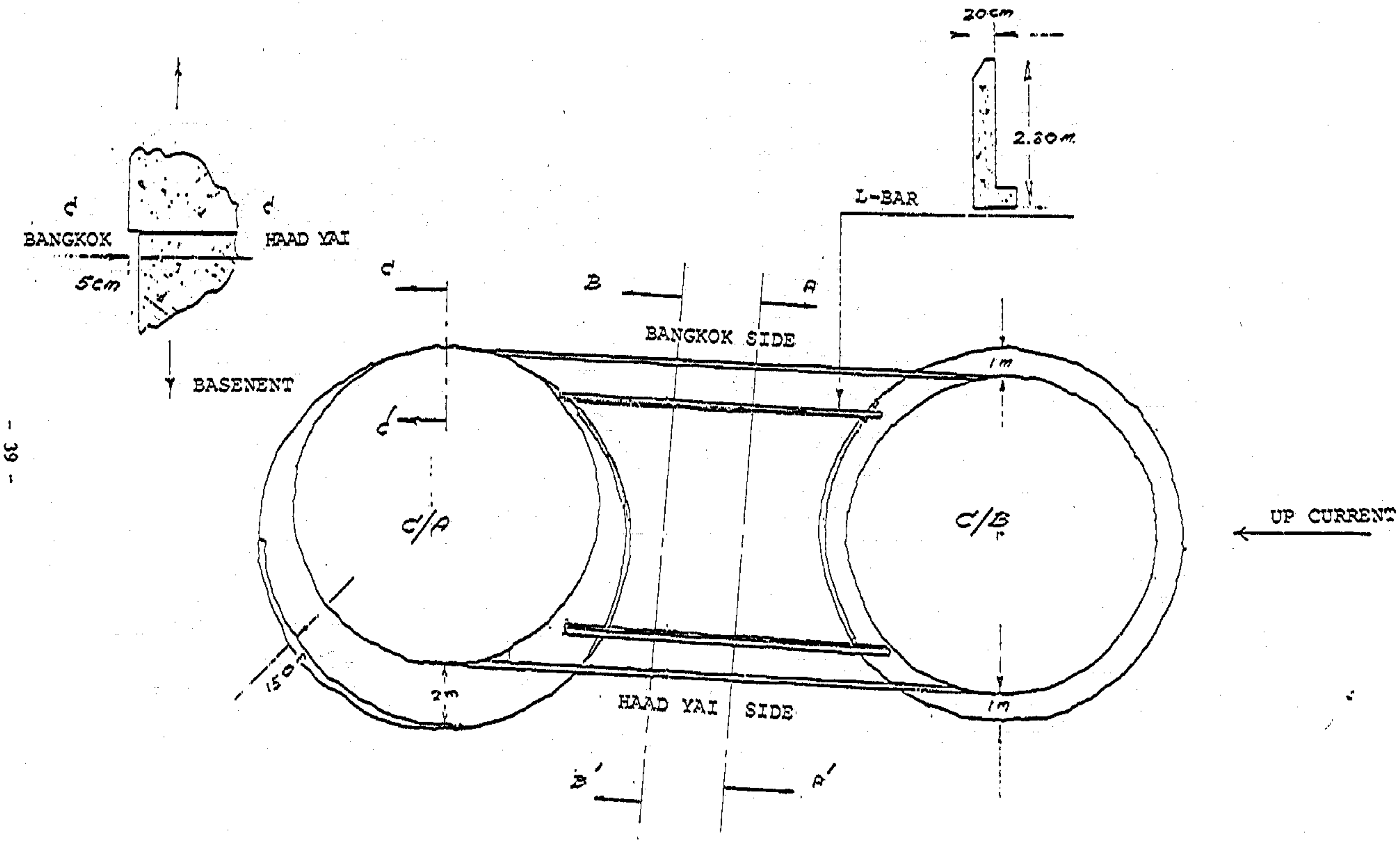


Fig. 1-7 C Pier

C-PIER , (TOP VIEW) ENCLOSURE 5
This drawing shows the off-center placement of the top portion of column A (C/A). The upper portion of column A is approximately 1 M off-center as compared with column B(C/B).

COLUMN A, D-PIER, ISOMETIC VIEW FROM THONBURI SIDE OF RIVER: GENERAL DETERIORATION AT POINT OF JUNCTION OF TOP AND MIDDLE PORTION OF COLUMN. ESTIMATED SIZES OF DAMAGE SHOWN IN DRAWING a1, a2, B-B' and a3. (Portion shown as appears to be crumbling the full width of the junction and up to 1 M in heigth.

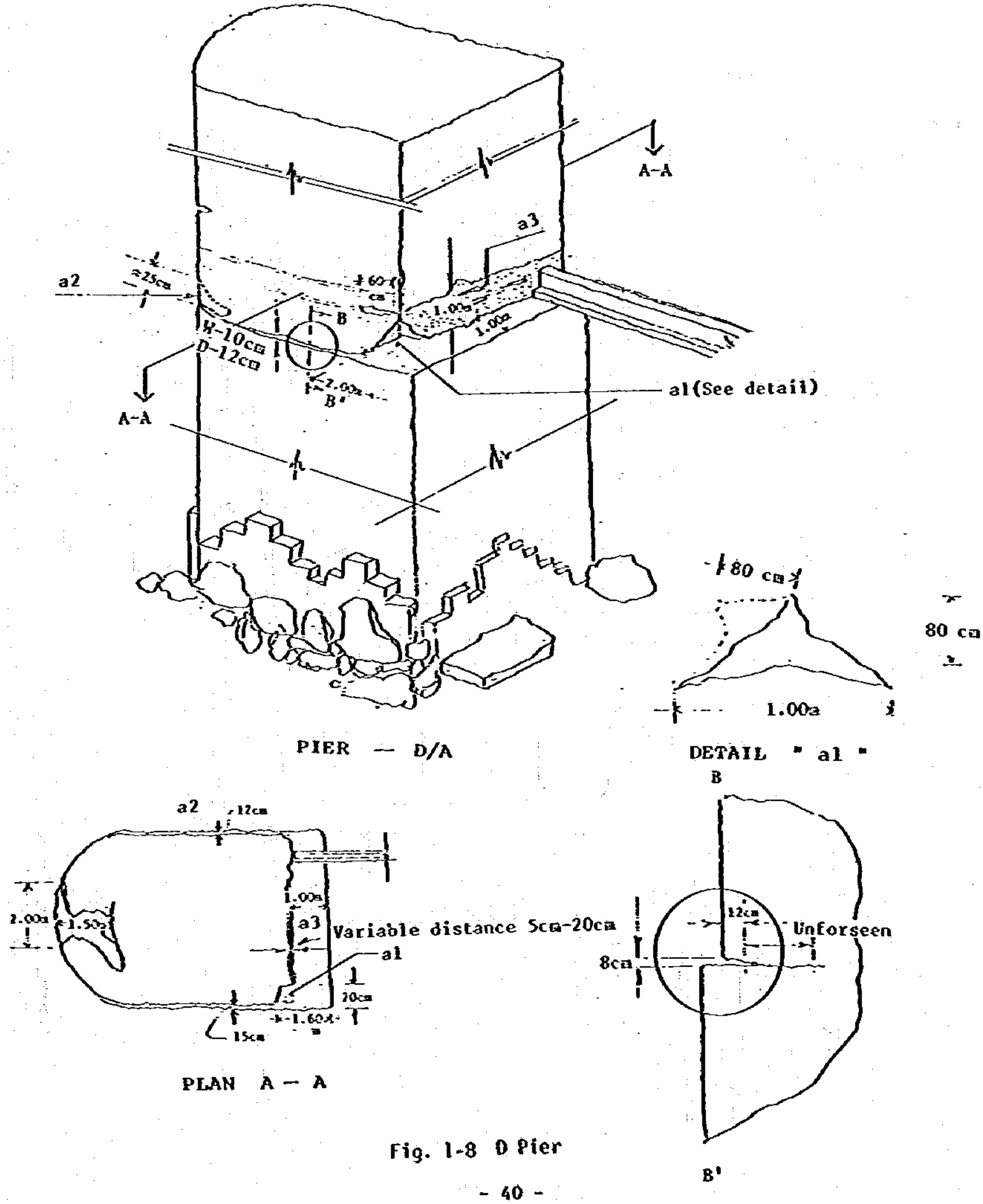


Fig. 1-8 D Pier

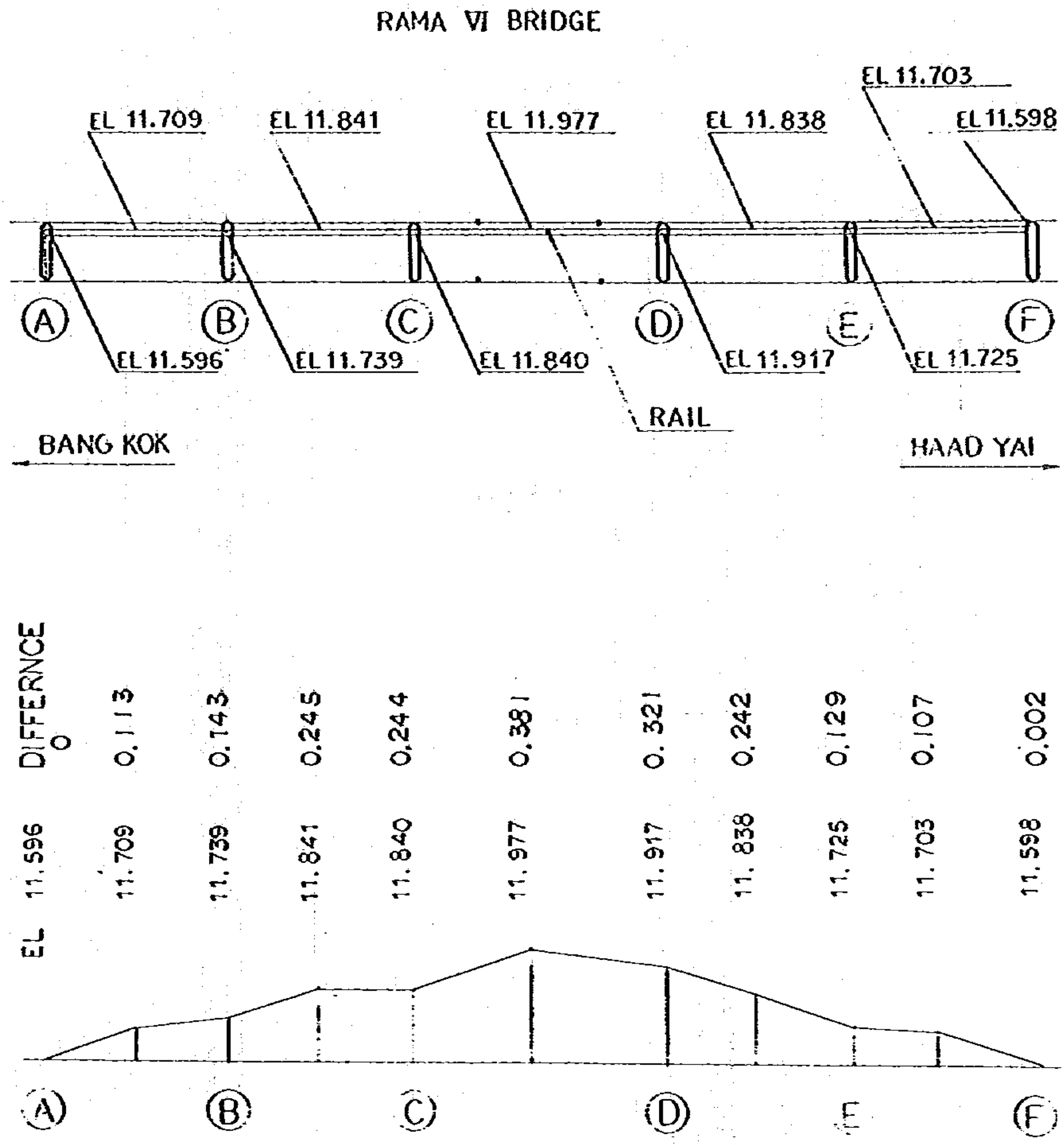
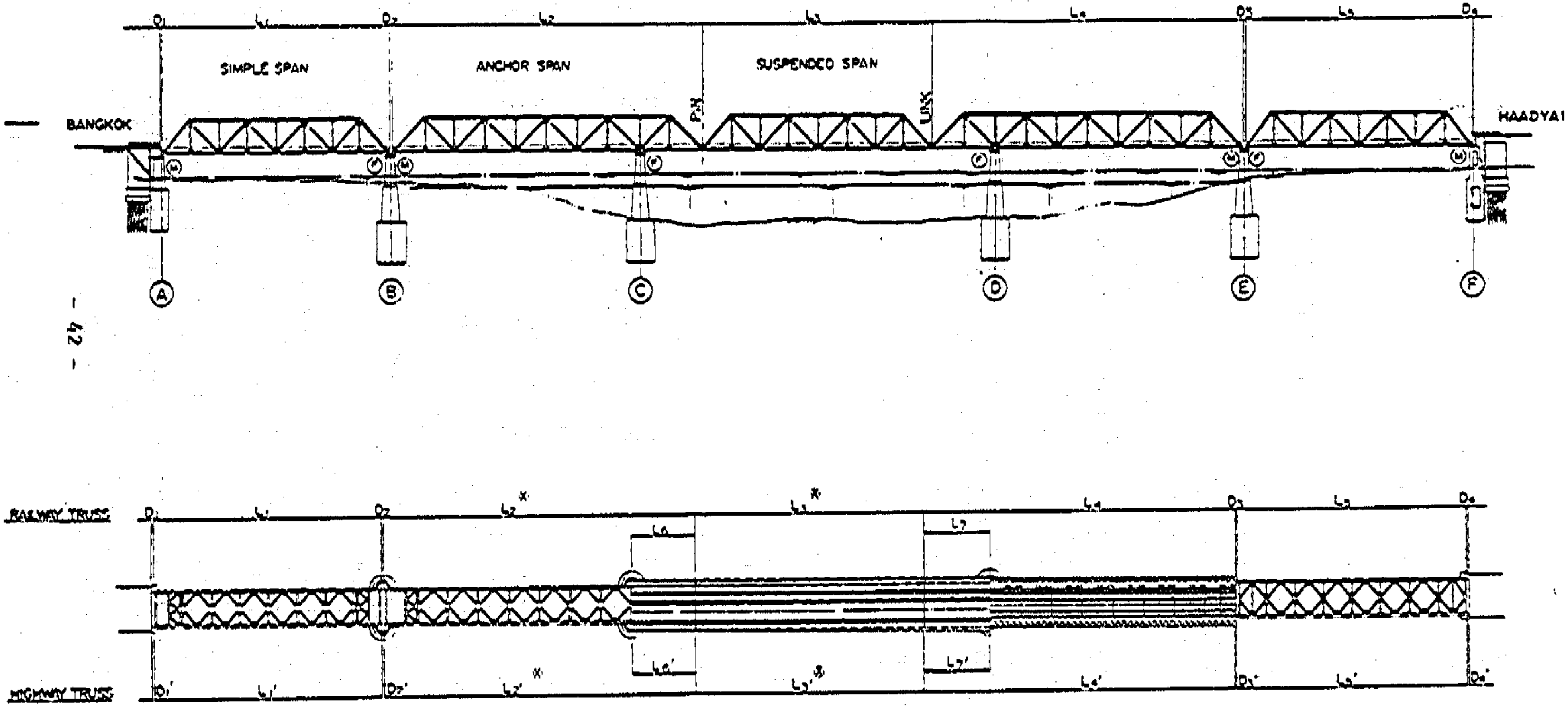


Fig. 1-9 Elevation of Top of Rail for Longl. Section

Fig. 1-10 DISTANCE SURVEY FOR LENGTH OF TRUSS GIRDER

WORK PLAN



ACTUAL LENGTH				ORIGINAL LENGTH
(R)		(H)		
L1	78.051	L1′	78.056	78.029
L2	104.672	L2′	104.701	105.199
L3	77.873	L3′	77.838	77.267
L4	105.204	L4′	105.180	105.224
L5	78.024	L5′	78.062	78.029
L6	20.692	L6′	20.708	21.354
L7	21.351	L7′	21.337	21.379
D1	0.026	D1′	0.032	
D2	0.230	D2′	0.217	0.307
D3	0.319	D3′	0.314	0.307
D4	0.147	D4′	0.143	

ACTUAL TOTAL LENGTH (R) $L_1 \sim L_5 + D_2 + D_3 = 444^{m}.373$ (A)

——— 〃 ———(H) $L_1' \sim L_5' + D_2' + D_3' = 444.368$

(R) −(H) = 0.005

ORIGINAL TOTAL LENGTH 444.362 (B)

(A) − (B) = 0.011 ~ 0.006
(R) (H)

R : Railway side

H : Highway side

Table 1-2 Distance Survey for Length of Truss Girder

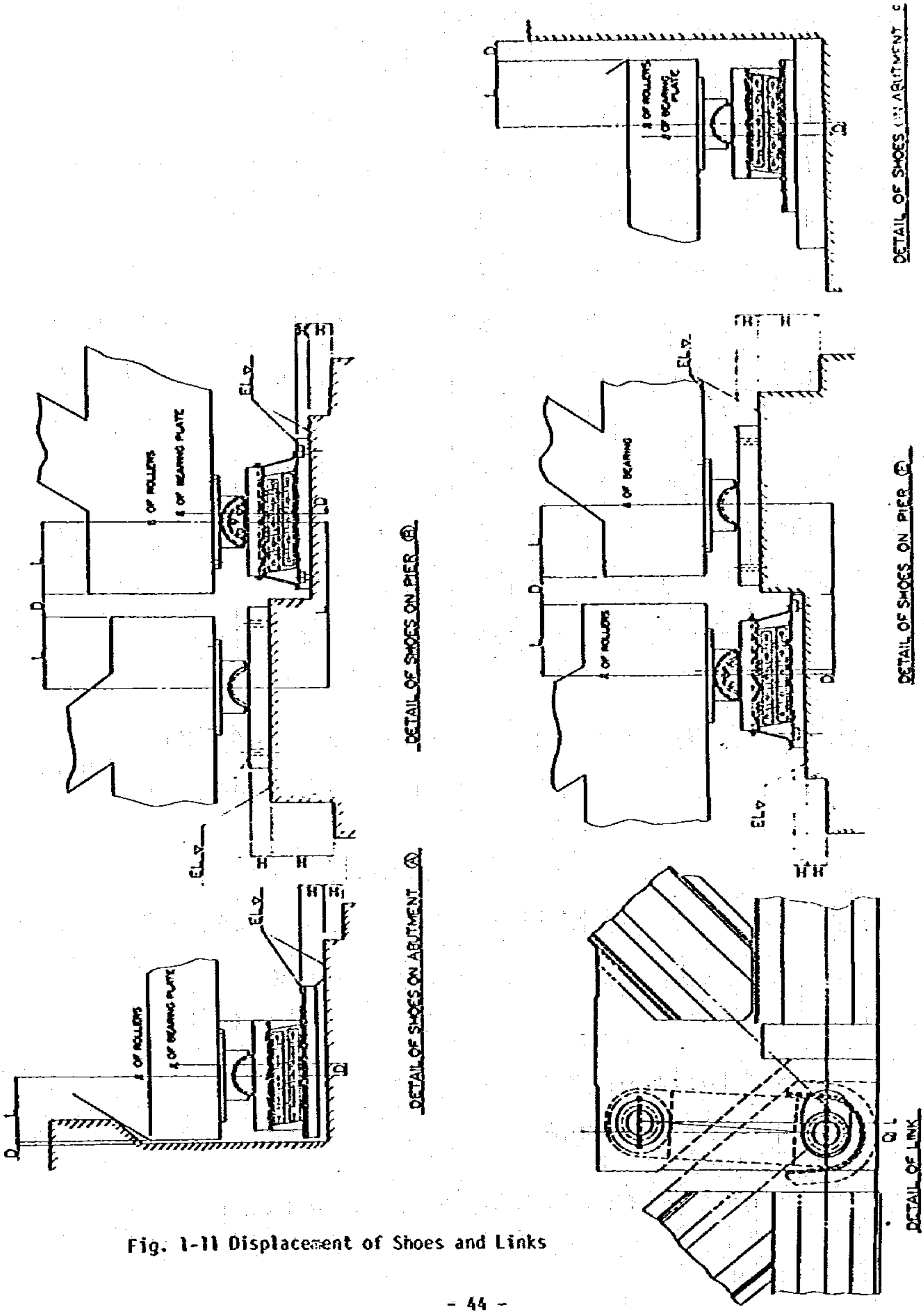


Fig. 1-11 Displacement of Shoes and Links

CHAPTER II

CHAPTER II ANALYSIS OF CAUSE FOR DETERIORATION AND FORECAST ANALYSES

1. Summary

In order to clarify the cause for structural deterioration, it is the basic practice to seize accurately and correctly the real circumstance of deterioration appearing on the structure at present, after full comprehension of the historical background on the existing structure tracing back to the original start of construction. Despite such established rule, however, since the historical process of bridge structure construction remained unknown, the main approach to clarification of the cause for deterioration of RAMA VI Bridge had inevitably to be concentrated solely to analysis of the present deterioration status.

To further explain the above, for instance, because of many discrepancies observed in the dimensional elements between the original design drawings and the actual existing structures, such design drawings can no longer help application of various analysis methods for analogy of the process of deterioration subsequent to the original design. For this reason, actions were taken to clarify the cause for deterioration through vibration survey and its analysis result.

Therefore, any further action for static analysis to be based upon the historical background of the existing bridge structure did not go beyond the scope of study only to forecast the possibility of deterioration.

2. Vibration Analysis

Vibration analysis was conducted for the purpose of obtaining the basic design data for rehabilitation of the bridge after comprehension of vibration characteristics, assurance of safety for the existing bridge structure and study on the possible variation of vibration characteristics resulting from rehabilitation of the bridge.

Analysis was divided largely into the following three items.

(1) Phenomenal analysis

This is to clarify vibration characteristics of the bridge by analysis of frequency characteristics and response variations to result from the change of train speed, by due reference to the recorded data on survey results.

(2) Model analysis

This is to simulate the train running by formation the vibration model, on the basis of the result obtained from phenomenal analysis.

(3) Study for bridge rehabilitation

This is to calculate the basic data for rehabilitation design of the bridge by forecasting of probable change of vibration characteristics, by use of the vibration model, after doubling of the track and renewal of shoes.

2-1 Phenomenal analysis

Phenomenal analysis is purposed to examine safety of the existing bridge structure after full grasp of vibration characteristics of the bridge by reproduction of the wave form, reading-out of the maximum value and analysis of the frequency from the recorded data on vibration test results. This analysis study covered the following four (4) items.

(1) Check to see any crack or gap on piers

(2) Variation of frequency characteristics in connection with train speed

(3) Estimation of normal vibration frequency of structure

(4) Variation of response values in connection with train speed

2-1-1 Crack check on piers

Originally, one of the causes for deterioration as assumed in Japan was attributed to the structural defect of Pier C. On this assumed basis, study was made for clarification, from the following three different viewpoints, as to if there is any crack or gap at upper, middle and lower parts of the pier.

(1) Comparison by wave forms

If there were any gap or crack at upper, middle and lower parts of the pier, the test results would reveal any difference or discrepancy in the normal vibration, thus obtaining response waves of different characteristics at each measured point of the pier. In the reality of test results, however, no difference or discrepancy was observed as shown in Fig. 2-1.

(2) Vibration mode

If there were any gap or crack at upper, middle and lower parts of the pier, the mode of vibration would not indicate any form of rigidity rocking (which means the linear shape of mode). The real result shows, however, the linear line as shown in Fig. 2-2.

(3) Comparison by Fourier Spectrum

If there were any gap or crack at upper, middle and lower parts of the pier, the peak in prominence should be observed at a different frequency. In the reality, however, such peak is seen at any phase of same frequency as shown in Fig. 2-3.

After all, therefore, it is finally concluded that as referred to in the foregoing items (1) thru (3) the study result does not indicate any such characteristics as may be identifiable for any gap or crack which would exist at upper middle and lower parts of the pier. Accordingly, any deterioration attributable to the structural defect of the pier is not conceivable in any event.

The train for this study was freight train No. 729 (operated at a speed of 30 km per hour on March 10) with the largest amplitude of displacement.

2-1-2 Variation of frequency characteristics in connection with train speed

Frequency characteristics of train load can be determined from both train speed and axial constitution.

Whilst the concrete method of analysis is shown in Appendix 2-1, this Section deals with only the outcome of analysis as shown in Fig. 2-4 and Fig. 2-5 respectively.

At measuring points 2P-1 (H_L) and 3P-1 (H_L) the peak was observed commonly at or near a frequency of 2 Hz irrespective of train speed. This tendency is also recognized as a common phenomenon to both passenger and freight trains which are of different design structure in wheels and axles. Those results reveal evidently that normal vibration frequency arises from the structure at and around a frequency of 2 Hz.

2-1-3 Estimation for normal vibration frequency of the structure

Behaviors of the structure at train running may be influenced largely by the predominant frequency from train load as aforestated. For the superstructure, it can be estimated that because of its structural difference normal vibration frequency should naturally differ between the central span and the side span.

In line with the above assumption, the whole bridge structure was divided into three (3) blocks and examined on vibration during passage of train, reverberation and constant microvibration. The result of this comparative check is, as shown in Figs. 2-6 thru 2-8, conspicuous of 1.4 Hz for the central span, in addition to predominance at and near 2 Hz.

After all with other results of study, normal vibration frequency may be summarized as follows:

(1) Normal vibration frequency for the central span (C ~ D) is at and near 1.4 Hz.

(2) Normal vibration frequency for both side spans (A ~ B and E ~ F) is at and near 2.2 Hz.

(3) Normal vibration frequency is around 1.9 Hz between B and C and between D and E.

(4) No crack or gap exists at Piers C and D, which are largely influenced by vibration characteristics of the bridge superstructure without self-predominance of normal vibration frequency.

From the eigenvalue obtained in the model analysis, as stated later, it is conceivable that the frequencies of 1.4 Hz and 1.9 Hz referred to in (1) and (3) should be deemed as primary and secondary normal vibration frequency respectively for the 3-span continuous girder between B and E.

2-1-4 Variation of response value in connection with train speed

Normally, the predominant frequency of vibration for train load may be varied depending the speed. However, in the case of the freight train, it is not suited for following up the variation of response to train speed because of lack of unity in the train make-up and the wheel base, though response emerges conspicuously. For this reason, twelve (12) passenger trains with a speed range of 20 km to 80 km per hour were selected as the objects for measurement of longitudinal displacement at each crown of Piers C and D during passage of trains. Results are as shown in Fig. 2-9 and Table 2-1, which indicate, in response to train speed, the predominant frequency of vibration for each train load.

Obviously, there is a growing tendency toward larger response with closer approach to the normal vibration frequency of the superstructure, though it does not go up to the extent of resonance. This result reveals that for model analysis train load is replaceable by distributed load of sine wave.

2-2 Model analysis

2-2-1 Vibration model

The vibration model as shown in Fig. 2-10 was formulated by due reference to the result of phenomenal analysis. The required conditions for setting of the vibration model are detailed in Appendix 2-1.

2-2-2 Eigenvalue analysis

As the result of eigenvalue analysis by use of the vibration model as shown in Fig. 2-10, the following values were attained:

f_1 = 1.425 Hz:	Behavior of 1st. mode (1.4 Hz at actual measurement) as the 3-span continuous girder
f_2 = 1.910 Hz:	Behavior between B ~ C and D ~ E piers, corresponding to 2nd. mode behavior of the 3-span continuous girder (1.9 Hz at actual measurement)
f_3 = 2.240 Hz, f_4 = 2.245 Hz:	Behavior between Abutment A and Pier B and between Pier E and Abutment F (2.1 Hz at actual measurement)

The normal vibration mode is shown in Fig. 2-11.

2-2-3 Train running simulation

The result of eigenvalue analysis by use of the vibration model was of close proximity to the normal vibration frequency as actually measured.

Now then, train load was regarded as the sine wave external force of uniform distribution and acted to the section of Piers B ~ E, so that magnitude of such external force could be determined so as to justify the horizontal displacement at each crown of Piers C and D as actually measured.

The result of train running simulation is shown in Fig. 2-12. Although the maximum response value could hardly be assessed strictly because the train speed was limited up to 80 km per hour, it was certain that response characteristics of actual measurement could almost be justified by the simulation test result.

2-3 Study for bridge rehabilitation

By means of the model analysis, the vibration model which can explain the result of actual measurement could be formed up and the magnitude of train load for replacement with the sine wave external of uniform distribution could be estimated.

In this section, study was made to see, by use of this vibration model, the possible change of vibration characteristics in connection with the changing situations by conversion into doubling of the track or improvement of shoes and to calculate the horizontal force to act upon the crown of each pier in the event of rehabilitation designing for the bridge substructure.

2-3-1 Changes in vibration characteristics resulting from doubling of track

The constants variable of the bridge after conversion into doubling of the track may be both rigidity and weight of the bridge superstructure. However, on assumption that weight would be far less variable, the increased rigidity of the superstructure was put into the sole focus. Movable shoes, link and pin were assumed without any change or modification, and further study were made in the following 2-3-2.

Since the degree of increase in rigidity can hardly be made definite, calculation was made for each case of increase up to 1.2, 1.5 and 2.0 times as compared with the present level of rigidity. Fig. 2-13 shows the changing trend of normal vibration frequency and the change in the first mode normal frequency may be denoted by the following formula:

$f/f_0 \doteqdot \sqrt{I/I_0}$ Where,

I_0, f_0: Rigidity of superstructure and 1st. mode normal frequency at the present status

I, f: Rigidity and 1st. mode normal frequency after conversion into double track

The changing trend of longitudinal displacement at the pier crown during passage of the running train is shown in Fig. 2-14. However, the value for train load was estimated up to double as much as the value used for model analysis in anticipation of the future conversion into doubling of the track. If maximum response alone is put into the focus, the calculated result was 0.83 times of present one for 1.2 I_0, 0.67 times for 1.5 I_0 and 0.49 times for 2.0 I_0.

2-3-2 Changes in vibration characteristics resulting from renewal of shoes

Study was made to foresee the probable change of vibration characteristics after renewal of movable shoes, pin and link in parallel with doubling of the track.

The model conditions were to use the value of 1.2 I_0 for rigidity as shown in the foregoing 2-3-1 in anticipation of the increased rigidity resulting from doubling of the track and to treat the conditions for renewal of shoes, pin and link as the change of connection at each joint in Fig. 2-10 as shown in Table 2-2.

Table 2-2 Changed Conditions due to Renewal of Shoes

Item	Model joint	Changed condition
Movable shoe	①	Change in horizontal spring
	⑪ - ⑦	"
	㊴ - ㊵	"
	㊾	"
Pin	⑱ - ㉓	Change in rotary spring
	㉗ - ㉘	"
Link	㉗ - ㉘	Change in horizontal spring

The probable change of normal vibration frequency was as shown in Table 2-3. No remarkable change was observed in respect of the change in normal vibration frequency in the vertical direction of the bridge superstructure. However, as shown in Fig. 2-15, normal vibration frequency of horizontal behaviors appeared at the low vibration zone.

Table 2-3 Change of Normal Vibration Frequency Resulting from Renewal of Shoes

Not renewed	Renewed	Vibration configuration
-	0.825 (Hz)	Predominance of horizontal behaviors
-	0.930	"
-	1.025	"
-	1.035	"
1.557(Hz)	1.541	Superstructure 3-span continuous beam, first mode
2.063	1.940	Superstructure 3-span continuous beam, second mode
2.360	2.361	Superstructure side span, first mode
2.370	2.367	Superstructure side span, second mode

Variation of response value measured during passage of train load are shown in Fig. 2-16. Although response at the first mode frequency of superstructure was most prevailing as expected, it was declined to a certain extent as the result of renewal with shoes.

2-3-3 Calculated horizontal force for substructural design

Horizontal force to arise at each pier crown from passage of the running train may be varied more or less depending upon the correlation between predominant frequency of vibration from train load and normal vibration frequency of the superstructure. It may be variable as follows:

(1) Non-resonance: $H = H_{TS} + H_{TD}$

(2) Resonance: $H = H_{TA}$

Where, H: Total horizontal force to act upon pier

H_{TS}: Horizontal force to arise from static action by train load

H_{TD}: Horizontal force to arise from vibration due to train running

H_{TA}: Horizontal force to arise from cycling action by total train load

Therefore, the value for H_{TD} is zero at the time of train stoppage. H_{TS} denotes horizontal force in the event that total train load may take its static action.

Now, by use of the value of K = 57.05 t/cm for the spring constant as equivalent conversion of the pier into the horizontal spring to act upon the pier crown, the value for H as horizontal force to act upon the pier crown during passage of the running train was calculated from horizontal displacement at the pier crown as obtained from response calculation.

Firstly, horizontal force in the state of non-resonance on the existing track (single track) is calculated as follows:

From the result mentioned in the preceding item 2-3-3,

$$H_{TD} = 57.05 \times \underbrace{\frac{1}{2} \times 0.048}_{\text{To convert max. full amplitude to half amplitude}} = 1.369 \text{ tons}$$

$$H_{TS} = 57.05 \times \underbrace{\frac{1}{2} \times 0.015}_{\text{Static displacement at loading of 0.5 t per m}} \times \underbrace{\frac{2.0 - 0.5}{0.5}}_{\text{Static load except cyclic force of 0.5 t per m}} = 1.284 \text{ tons}$$

$$\therefore \; H = H_{TD} + H_{TS} = 2.653 \text{ tons}$$

Secondly, horizontal force is sought as follows in anticipation of falling into the state of resonance (most dangerous condition) after completion of the double track (however, rigidity would remain unchanged at the present level):

Maximum displacement may be estimated at 3.16 mm (full amplitude) as against load of 1.0 ton per m (as cyclic force in the state of non-resonance with the double track) as per Fig. 2-14. Therefore,

$$\text{Horizontal force } H = 57.05 \times \frac{1}{2} \times 0.316 \times \underbrace{\frac{4.0}{1.0}}_{\substack{\text{4.0 tons per m if acted} \\ \text{upon by total train load} \\ \text{as cyclic force}}} = 36.056 \text{ tons}$$

Namely, from the result obtained above any horizontal force to act upon the pier crown, as may arise out of the running train, may be estimated at about 36 tons even at the ceiling maximum.

2-4 Vibration analysis and future forecast

The outcome of future prediction from the result of vibration analysis, as stated earlier, may be considered as one of important matters.

With this view in mind, study has been made to clarify responses at each crown of Piers C and D on assumption that cyclic external force in the horizontal direction would act upon the link position, which was taken up particularly as a matter of grave concern around August 1981. Study was based on such assumed conditions that the road condition should be traced back to around August, 1981 or the expansion joint at the link would be deteriorated, and the link would be affected horizontally by repeated impact load during passage of heavy vehicles on the road side. This repeated impact load is of a cyclic nature subject to variations depending upon the wheel base and running speed of the vehicle and unit external force of horizontal cyclic would act upon the link toward Bangkok in consideration of load difference between the truck with load proceeding to Bangkok and the truck without load proceeding to Haad Yai.

Horizontal response displacement at each crown of Piers C and D is shown in Fig. 2-17. Pier C is showing larger response than Pier D as noted from Fig. 2-17; the response displacement at the time of resonance being about double as much as from Pier D. This is because, as noted from the vibration mode shown in Fig. 2-15 Pier C tends to show larger behavior than Pier D in the primary vibration mode dominating all responses. Furthermore, Pier D does not show any such clear-cut peak as can be seen at Pier C, since the secondary vibration mode for Pier D is also predominant.

The values of wheel base and load of the large-size vehicle which are used for this study are taken from the AASHTO or the road bridge construction specifications enforced in Japan. As for the large-size vehicle with wheel base of 9 m, a peak value as the resonance phenomenon can be assumed at a running speed close to 26 km per hour.

For instance, in the event that on the road conditions as of August, 1981 large vehicles with wheel base of 9 m passed through the section at a speed of 26 km per hour by continuous series of 4 to 5 units, there would have been horizontal displacement of about 2 mm (within a range of 5 to 10 times as large as this figure when bodily felt by individuals). Paradoxically speaking, if the road maintenance service level would decline to the same level as in August 1981, there would be possibility of causing the similar degree of vibrations to the feeling of human bodies.

3. Static Analysis of Deterioration

Static analysis of deterioration should absolutely require such recorded data as past construction record and secular change of deterioration. In fact, however, since those recorded data are not available, various studies were made from possibilities of deterioration and the results as stated hereunder were obtained.

3-1 Review on consolidation and settlement of embankment

Embankment on both Bangkok side (partially removed at present) and Haad Yai side, forming respectively an approach to RAMA VI Bridge, is laid on top of the Bangkok soft clayey soil layer which may be considered as the normal consolidation layer. The clayey soil layer of that kind may be affected by a pretty large degree of consolidation and settlement.

Since such settlement was considered as the cause for moving of Abutments A and F toward the front, an attempt was made to seek the extent of influence from such settlement.

Shown in Fig. 2-18 is the relation between the critical embankment height (Hc) and the cohesion (Cu) of clayey soil foundation. The results of soil test conducted near abutments of RAMA VI Bridge were $Cu \fallingdotseq 1.75$ t per m^2 and $Hc = 3.6 \sim 4.9$ m.

Incidentally, since the critical height of embankment with full assurance of safety from slide failure during construction is $Hc = 3.6 \sim 4.9$ m, it is easily imagined that at the time of original construction people took much trouble for banking of soil on a step-by-step basis to build up the present embankment, especially when the preceding figures of critical height are compared with the actual height of embankment to the value $h = 10 \sim 12.5$ m.

Fig. 2-19 shows the relation between settlement and embankment height. Fig. 2-20 shows the relation of construction embankment height (h = 12.20 m) versus embankment height (h' = 10.45 m) after 80% and 100% consolidation. Although the construction embankment height can hardly be made definite with its accurate height for lack of the original construction records, the settlement at the most probable embankment height can be estimated at S = 1.75 m at 80% consolidation and S = 2.2 m at 100% consolidation.

Fig. 2-21 shows the settlement curve of embankment on the assumption that embankment would have been completed on a step-by-step basis in a construction period of about one (1) year. As noted from Fig. 2-21, it shows about 90 percent completion of consolidation after elapse of 55 years counting from 1927 as the year of work commencement for construction of RAMA VI Bridge up to this date (in 1982). Although consolidation may be 100 percent completed mathematically, the calculated values do not reflect any correction in particular because of no supporting data for such completion being available in hands. The normal practice of consolidation calculation can allow for this minor degree of calculated error as a marginal tolerance.

From the foregoing point of view, it can be said in many instances that about 10 percent of total settlement, that is S = 216 cm as shown in Fig. 2-21, may be represented by horizontal displacement, though more or less varied depending on foundation type of each abutment. It can therefore be inferred that any horizontal displacement produced at each abutment should reach 10 ~ 20 cm.

3-2 Review on slide failure of embankment

Although there remains none of construction records for embankment, review has been made to examine the possibility of causing slide failure of embankment, on such assumed condition that construction of embankment on the similar scale would be performed for a period of about one (1) year as stated in the foregoing 3-1. The results are shown in Figs. 2-22 thru 2-24.

There is a case where the value of Fs (the safety factor) falls down below 1 at those steps of embankment as shown in Figs. 2-23 and 2-24. It appears that this declined value may implicate outbreak of slide failure on the embankment, thereby providing difficult jobs done for such embankment at sacrifice of human effort.

Fig. 2-25 shows the status of embankment with time elapse of 10 years after its construction.

3-3 Study on bridge displacement by influence of unequal earth pressure

Abutment A on the Bangkok side began to show the moving phenomenon to the front, immediately after completion of the rehabilitation work in 1950, probably because of unequal earth pressure resulting from banking on the rear side of the abutment. At the reach of its horizontal displacement up to 18 cm the repair work was done in 1967 by partial removal and replacement of the rear side soil embankment with composite girders. With further development of deterioration another horizontal displacement of about 7 cm was added to such an extent that the girder end of superstructure has entered into contact with the abutment parapet.

Thus, deterioration may be considered to be attributable to the causes over a long term due to consolidation and settlement of the embankment estimated at about 2 m sliding failure of soil during construction and after completion of the embankment, and unequal earth pressure arising from banking of soil on the rear side.

Abutment F on the Haad Yai side is also moved to the front in a horizontal displacement of about 7 cm, though not so conspicuously deformed as Abutment A was because the foundation of its upper soft soil layer still provides the favorable conditions as compared with that for Abutment A.

Since partial removal of the rear side embankment for Abutment A in 1967, the loading condition has been in unbalance over a long period when viewed from the total structure of RAMA VI Bridge.

That is to say, the bridge structure was in a stabilized status as a whole, prior to 1967, with existence of the rear embankment of both Abutments A and F, though the ground condition may differ more or less. Nevertheless in and after 1967 the balancing condition has been undergoing changes.

Then, on assumption that the whole bridge structure might be forced out toward Bangkok by the rear soil embankment of Abutment F, study was made to check displacement as influenced against the bridge by unequal earth pressure to act upon Abutment F, on the supposition that the whole structure of bridge would work as a strut as shown below.

(1) To neglect axial deformation of girder

(2) To assume no gap between joints of girders

(3) To assume the joint between girder and pier (abutment) as pin

(4) To neglect bending deformation of pier

Firstly the RAMA VI Bridge shown in Fig. 2-26 will be converted into the calculation model as shown in Fig. 2-27. Then, by substitution of the horizontal displacement factor of Piers A ~ E in horizontal spring constants K_A thru K_E at each crown of piers from the relationship between load and displacement of each crown of piers, the model can be simplified as shown in Fig. 2-28.

The result is as shown in Fig. 2-29. That is to say, horizontal displacement at Abutment F may be calculated at δ_1 = 5 cm at the top of pier and δ_2 = 16 cm at the bottom of caisson.

Meanwhile, since the shoe on Abutment F is movable, the displacement on assumption that Abutment F along should bear unequal earth pressure may be calculated at δ_1 = 122 cm at the top of Abutment F and δ_2 = 3 cm at the bottom of caisson. Therefore, the shoe at Abutment F may be conspicuous of its large move.

However, as a matter of fact, the movement of the upper shoe at Abutment F was measured at 7 cm. Therefore, it was rather closer to the reality to assume that the whole bridge structure was in the state of strut. The conditions as the strut may be attributable to concrete slab on the highway side, complex interaction between longitudinal girders and shoes and difference in the main truss section on both railway and highway sides.

As an evidence to justify those conditions, the movement of upper shoe at Abutment F was observed under the following conditions. Namely, the result of comparison in horizontal displacement of upper shoe at Abutment F between the worst time of road conditions in August 1981 and the improved time of road pavement in March 1982 reveals that there is observed displacement which may be assumed to be a movement of Abutment F by about 10 mm further to the front.

Meanwhile, the shoe at Abutment F has a certain degree of function as the movable shoe for train load, thus showing its movement to catch up with train running at its passage.

This makes us feel that it would be rather closer to the reality to consider that the whole bridge structure would have been in a state of strut to long duration load like unequal earth pressure as influenced by road concrete slab, though it may indicate response, to some extent, to any transitional load like train load.

Incidentally, the condition at the link is such that the said phenomenon can not be explained by the above idea in all instances. The reason for that may be assumed as follows:

The present position of pin at the link has been mostly moved toward Bangkok, some being moved by 10 to 15 mm as estimated from the paint peel-off at the link.

Suppose if the whole bridge structure would have been moved toward Bangkok by influence of unequal earth pressure from the Haad Yai side, pin at the link should have been moved toward Haad Yai in the link holes. This is indeed the point of contradiction with the assumed basis that the deterioration would come from the unequal earth pressure of Haad Yai side.

Although any conclusive assumption must be refrained at the present time for lack of detailed records from the start of construction, such contradictory point may be answered by the following explanations:

(1) It is wondered that the pin might have been unbiased irregularly to Bangkok. The reasons for this assumption may be as follows.

① The existing design drawings does not accord always with the dimensional factors of the bridge structure.

② The angle notch at the link does not form a regular curve, with a difference of 4 mm in the notch of angle between both railway side and highway side. It is conceived that this might be the mark of adjustment made at the time of girder erection.

③ Despile paint peel-off to the extent of 10 to 15 mm, nothing still remains clarified with regard to the painted condition before.

(2) As stated in 3-5, Pier C tends to be tilted towards Bangkok because of eccentricity of Pier C at the initial time of erection.

(3) It can be estimated that from the worst time of road condition in 1981 after removal of asphalt pavement the rugged concrete slab surface was exposed and both end surface of concrete slab might have been smashed by impact from large vehicles during passage, though protected by steel plate covering, thus acting toward gradual expansion of the link.

In addition to the condition stated above, the joints of concrete slab on the highway side might have been plugged up with concrete fragments, and it is assumed that the whole bridge structure work as a strut and might have been moved toward Bangkok by unequal earth pressure at rear soil of Abutment F, inclination of Pier C due to eccentricity and impact from vehicle load on the highway side.

3-4 Foundation stability calculation at present and future conversion into double track

Study was made by stability calculation of Pier C at the present status and for future conversion into doubling of track by due reference to soil test and scour survey results. This study was purposed to check if the caisson base was supported by the stabilized soil foundation. Because there was similarity between Piers C and D with regard to configuration and geological condition, calculation was conducted solely on Pier C alone. The result is as shown in Table 2-4.

Table 2-4 Stability Calculation of Caisson (Safety Factor)

	Safety factor (Fs1) at present	Safety factor (Fs2) after doubling of track	Required safety factor (Fa)
Vertical bearing strength	7.98	7.10	2.0
Horizontal bearing strength	60.50	60.50	2.0
Overturn moment	8.64	8.64	2.0

The result reveals, as noted from the above Table, that they all appear to be pretty high safety factors and can fully ensure safety as the fundamental structure even after conversion into doubling of the track.

3-5 Review on effect from eccentricity of Pier C

As the result of subaqueous check by divers, it was made clear that as shown in Fig. 1-5 Pier C on its upstream side is eccentric toward Bangkok by about 1.0 m. Since superstructural and pier load of the bridge act as eccentric load constantly, an attempt was made to seek its effect upon the caisson base.

As noted from Fig. 2-31, if the upper part of Pier C is eccentric by 1.0 m on the upstream, the value of eccentric moment to act upon the top face of caisson base would be $M = 906$ t·m and displacement to arise therefrom at the crown of pier would be $y = 0.9$ cm.

3-6 Displacement of Pier C due to temperature variations

Since the bridge superstructure is now of tight complex to the link, it can be easily estimated that any expansion or shrinkage of the superstructure by temperature variations would give its direct influence to piers.

For this reason, study was made by use of a simple pattern of model so as to check how much displacement would be brought to the bridge substructure by expansion or shrinkage resulting from temperature variations on the superstructure between Piers C and D distanced 120 m apart from each other.

The premises for calculation were based on such assumption that, as shown in Fig. 2-32, the superstructure would form up a bar member consisting of upper and lower main girders and both of Piers C and D would be supported solidly by the soil foundation.

The calculated result is as shown in Table 2-5. (P. 88)

3-7 Static analysis and forecast of structural deterioration

The future forecast based upon the result of static analysis of deterioration as stated in the preceding sections may be summarized as follows.

(1) Settlement due to consolidation as shown in Fig. 2-21 show the fact that there exist some amount of unfinished consolidation in the Dc layer after almost completion of consolidation in the Ac layer as shown in Fig. 1-1.
However, it may possibly be conseivable that consolidation and settlement have been almost completed in general as a matter of fact and, therefore, there may hardly be any possibility of causing such deterioration as may be attributable to consolidation and settlement.

(2) The safety factor to prevent slide failure of embankment as shown in Fig. 2-25 is Fs = 1.11.
Although no circular slide failure would occur from the calculated result, it is difficult to make definite judgement that has it would in no case take place. Rather, it is suggested that there would be ample potentiality to cause slide failure at such a low safety factor. In other words, it is likely that there is to be anticipated danger of causing collapse of embankment or slope of embankment if the safety factor falls down below 1 because the water level inside embankment rises due to rainfalls.
Slope surface collapse as observed on a part of embankment toward Haad Yai may be justified for the reason mentioned above.

(3) Displacement of Abutment F due to unequal earth pressure toward Bangkok as shown in Fig. 2-29 is nearly same as horizontal displacement on assumption that the whole superstructure would

work as a strut. After improvement of road pavement and released from strut condition, Abutment F was moved by about 10 mm in the horizontal direction.

This may be considered as an influential factor to make Abutment F move toward its front because of unequal earth pressure, though such displacement may go on at a very slow pace.

For this reason, careful monitoring will be required to watch the future progress of deterioration on Abutment F.

(4) Although influence upon eccentric load by Pier C as shown in Fig. 2-31 may be relatively small, utmost care should still be required because it is duration load.

(5) In respect of temperature variation as shown in Fig. 2-32, horizontal displacement of about 1 to 2 cm may take place at the crown of Pier C due to temperature variations near the Bridge. However, this may not be so serious problem in particular. It seems that paint peel-off at the link to 10 ∿ 15 mm may be attributable mainly to such temperature variations.

4. Study for Conversion into Long Rail on the Bridge

4-1 Conversion into long rail over the Bridge and vibration of piers

In the judgement of the result of preceding vibration analysis, it is expected that if the rail is converted into long rail vibrations on the piers will be reduced to some extent because each girder end will form up tight joint to the other end with rail, though slightly, by friction between the rail and the fastening device, thus causing vibrations nearly in solid unity for horizontal exciting force which may arise at passage of the motor-vehicle over the broken link.

However, it is conceivable that vibrations on the piers would remain almost unchanged against vertical exciting force from running of the train in so far as the train runs at its normal speed.

All those above things taken into consideration, along with the result of measurement that vibrations of the piers arising from running of the train still remain at a low level, the long rail conversion of the existing railway would be of less significance with the objective to reduce vibrations even at the time for doubling of the track.

4-2 Track structure for long rail conversion

Truck structure was taken up for study in anticipation of the conversion into long rail as may be normally planned for the purpose of reducing maintenance workload at rail joints and improving the riding quality.

4-2-1 Potential conditions for long rail conversion

In view of the relationships with arrangement of fix and movable shoes of girder, girder length, position of expansion joint and temperature variation, either one of the following alternative cases can accept conversion into long rail:

(1) Axial force to yield on the rail shall not exceed the permissible limit.

(2) Opening size at breakdown of long rail shall not exceed the permissible limit.

In the case of conversion into long rail of the existing track on the RAMA VI Bridge, the required conditions for track structure are specified as follows after studies.

(1) Rail shall be welded with brand new of 50 N Rail now being used by the JNR for long rail or its equivalent or above.

(2) Ties shall be newly made of wood and laid in a number of two (2) per m at average.

(3) Rail fastening shall be so devised as to enable rail to slide over ties at expansion contraction of girder due to temperature variation. Rail shall be fastened up with longitudinal resistance strength of 0.25 ton per 1 m in rail length.

(4) Ties shall be bolted up tightly to the bridge and shall be assured of safety against any lateral force of 1 ton and over per 1 m in track length.

(5) Designed temperature variation from the time of long rail conversion shall be limited to 35°C.

4-2-2 Axial force to yield on long-rail

In the case that the track before or after the bridge is laid on the soft soil foundation and not suited for conversion into long rail or axial force of long rail on the bridge must be reduced, either the expansion joint or the buffer rail will be inserted before or after the long rail section.

In the meantime, if the section before or after the bridge is suited for conversion into long rail, long rail may often be laid over a long span including the track on the bridge.

The following are the results of calculation made on the alternative cases as aforestated.

(1) Use of expansion joint or buffer rail ·

Axial force of rail in the section where either expansion joint or buffer rail has been inserted will theoretically be zero value.

Fig. 2-33 shows distribution of rail axial force.

(2) Laying of long rail over a long span including bridge

If there is no insertion of expansion joint or buffer rail to the bridge girder end, axial force of rail will not be zero at the girder end theoretically, often very great in excess of the allowable limit.

Fig. 2-34 shows distribution of rail axial force.

(3) Allowable limit for axial force of long rail

The allowable limit for axial force of long rail on the bridge is determined subject to the type of rail and the type and arrangement of rail fastening device. That is to say, the limit is set below axial force of rail at buckling as may be determined from both longitudinal and lateral resistance force of the track.

Rail axial force at buckling under the conditions of (1) thru (5) in the foregoing item 4-2-1 is estimated at 82 tons. This can theoretically assure safety unless axial force to yield on the long rail exceeds that limit.

4-2-3 Opening size at breakdown of long rail

At shrinkage of long rail by temperature fall, rail will produce large tensile strength, thus yielding a large gap between two rails by breakdown of rail.

Since the track would be jeopardized by increased opening, check must be made so as to assure that the opening would not exceed a limit at any time of long rail breakdown.

The calculated size of opening is shown in Figs. 2-33 and 2-34 at long rail breakdown on the bridge.

4-2-4 Allowable limit for opening at rail breakdown

Although there is no definite basis of justification theoretically to explain to what extent safety can be assured if gap is created between two rails at rail breakdown, the JNR's normal practice is to consider about 50 mm as the most appropriate limit of allowance.

4-2-5 Summary

Axial force of rail does not exceed the allowable limit in case of track condition of item 4-2-1, regardless of whether both ends of rail are equipped with expansion joint or not.

However, in anticipation of the opening at breakdown of long rail the track structure should require insertion of either expansion joint or buffer rail.

If buffer rail is used, it is recommended that the joint gap for the buffer rail should follow as shown by Fig. 2-35.

The JNR makes it its established practice to use rail above the level of 50 N taking the safety into consideration and, in line with this standard, wooden ties each of 20 cm × 20 cm size are laid at a space of 50 cm between the two. The fastening device is of combined use of Type 50 III (with longitudinal resistance force of 0.25 ton) as shown in Fig. 2-36 and Type 50 IV (with longitudinal resistance force of 0 ton) as shown in Fig. 2-37 so as to attain the required strength of longitudinal resistance.

From the study result as aforestated and the operating experience in the JNR's system, required truck structure and maintenance for conversion into long rail on the RAMA VI Bridge may be summarized as follows:

Rail	:	Equivalent to 50 N rail (for the J.N.R.) or above
Fastening device	:	Fastening device applicable to long rail on the bridge (by alternate use of Type 50 III and Type 50 IV)
Tie	:	Wooden tie of 20 cm × 20 cm section area at space of 50 cm between two ties
Tie-fixing bolt	:	Diameter of 25 mm
Rail expansion joint	:	To install buffer rail at both ends of bridge
Track maintenance	:	Careful maintenance to be required for rail fastening device, tie-fixing bolt and gaps on buffer rail

4-3 Problems related to long rail conversion on RAMA VI Bridge

The track conditions to accept conversion into long rail on the RAMA VI BRIDGE should be as previously stated in 4-2. This matter was taken up for further study with regard to the present status of the Bridge and the future Bridge after completion of the double track system. After study, the problems related to conversion into long rail are specified as follows separately for the superstructure and the substructure.

4-3-1 Problems on superstructure

Since the original design for the bridge superstructure is based on the British Standards Specifications (equivalent to DL-18), considerably larger train load than the present load of DL-14 is incorporated into the design. Furthermore, the original design for shoe does not reflect horizontal load (for braking force) but does reflect only vertical load.

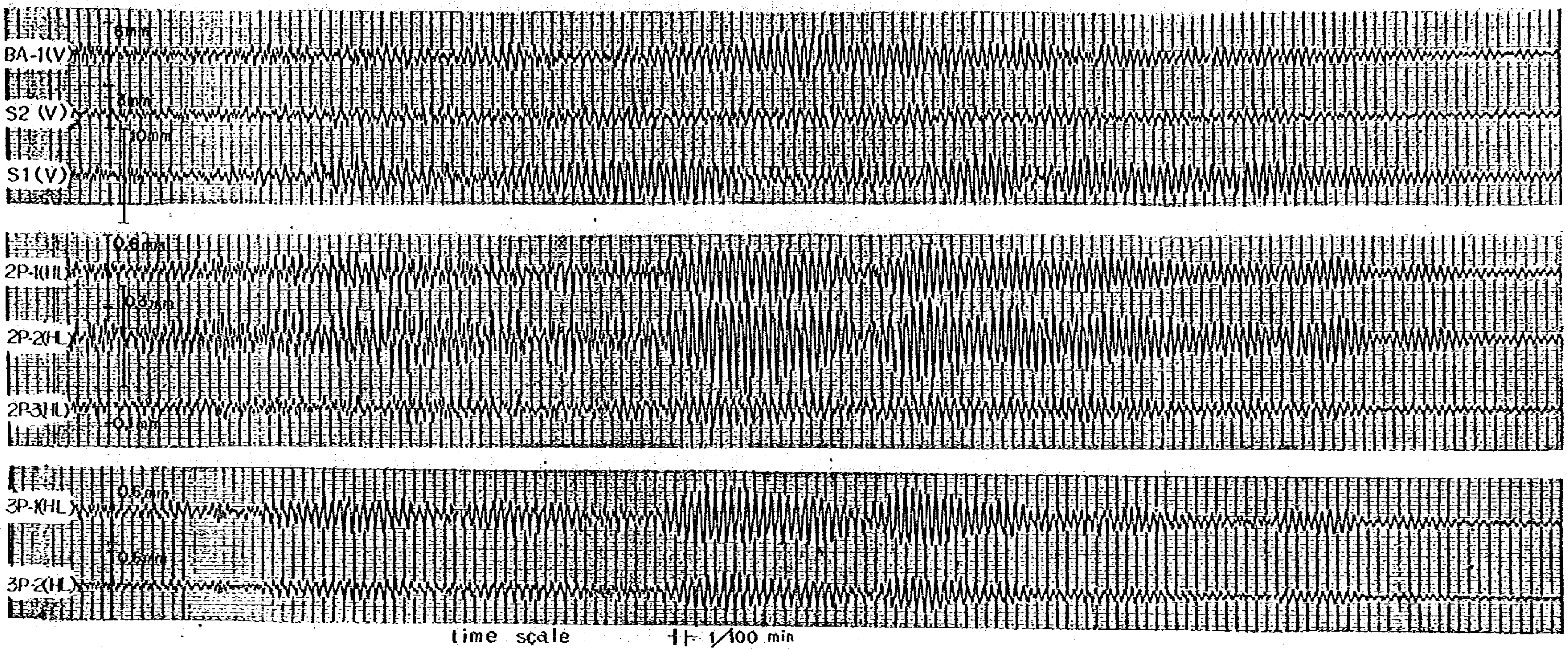
The latest study was made with due consideration to both braking force and longitudinal force of long rail in addition to vertical load. Then, the result of study assures that there is no problem involved in shoe itself, concrete bearing stress of shoe seats and connection between main structure of girder and upper shoe because they are all within the allowable limit of stress. However, only the brake-truss must be reinforced since its stress exceeds the allowable limit if the S.R.T. specification applies to it.

As for conversion into long rail at doubling of the track, shoe itself, concrete bearing stress of shoe seats and connection between main structure of girder and upper shoe can be restrained within the allowable stress. However, the brake truss and a part of main truss structure should require reinforcement.

If the track has been converted into the long rail system after consideration to the aforestated problems, utmost care must be paid to the maintenance conditions.

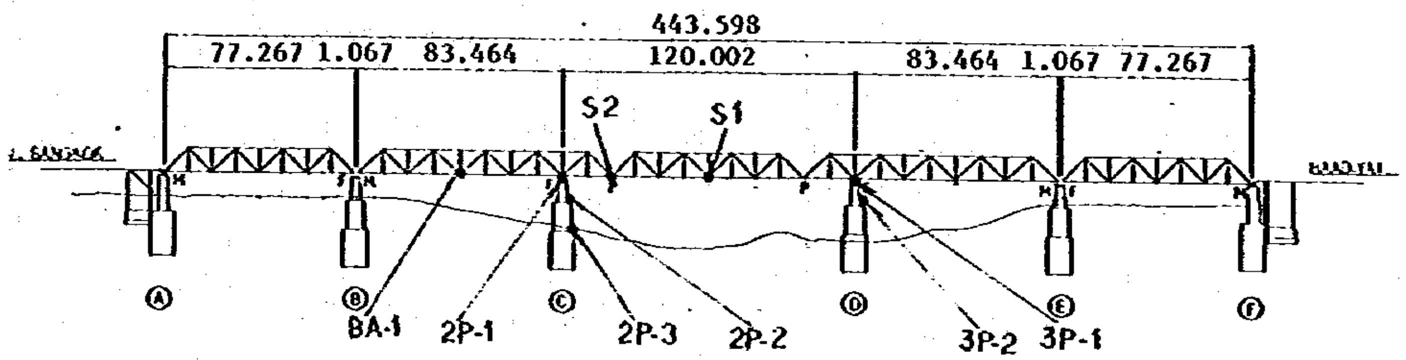
4-3-2 Problems on substructure

It appears that the substructure after conversion into long rail would have no particular problems with regard to foundation stability and stress of the pier structure.

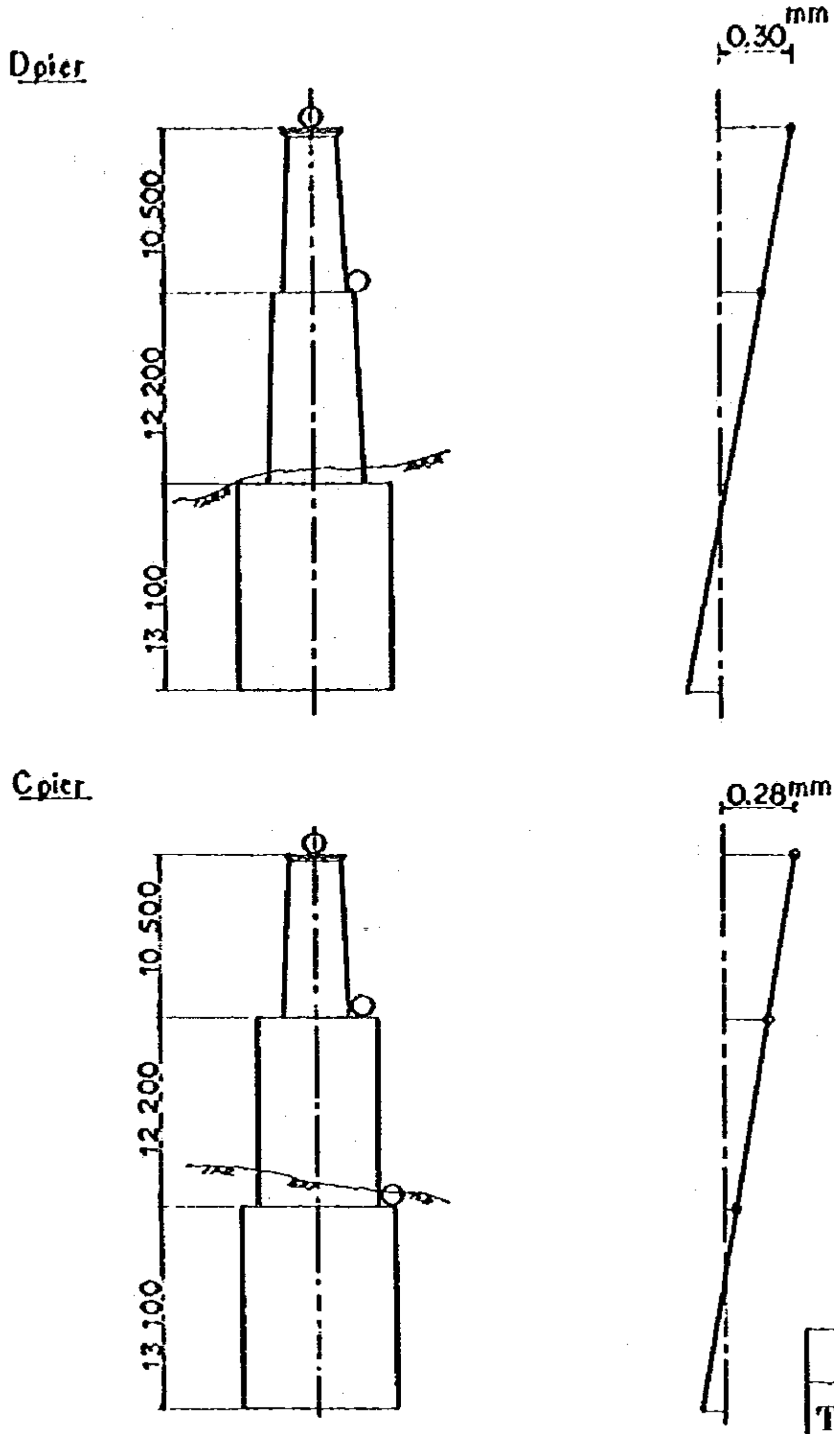


Date	March 10th
Train No.	729 (Down)
Train speed	30 km/h

Fig. 2-1 Examples Responded Waves

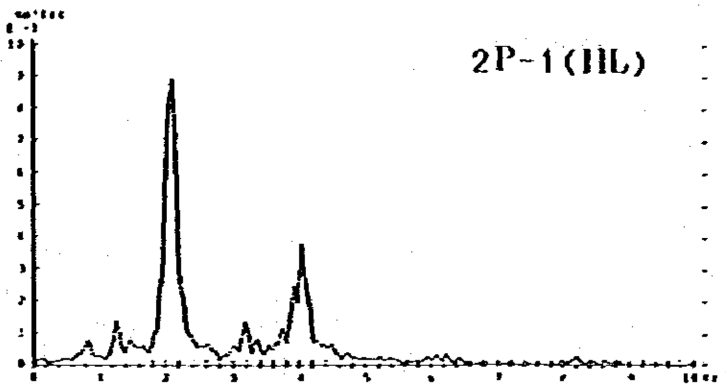


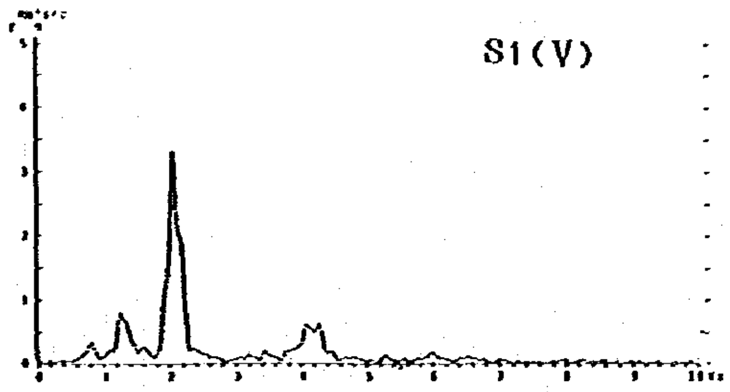
Location of points measured S=1/1500

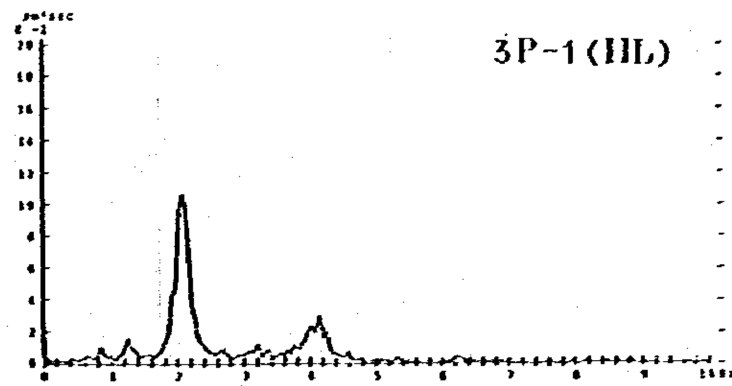


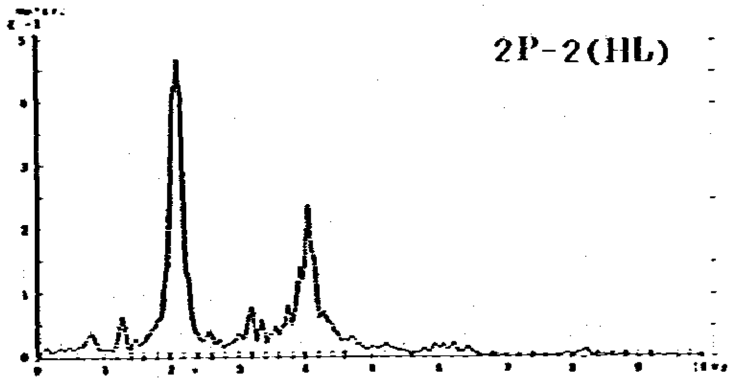
Date	Mar. 10
Train No	729(Down)
Speed	30 km/h

Fig. 2-2 Vibration Mode at Piers









Date	Mar. 10
Train No.	729 (Down)
Train speed	30 km/h

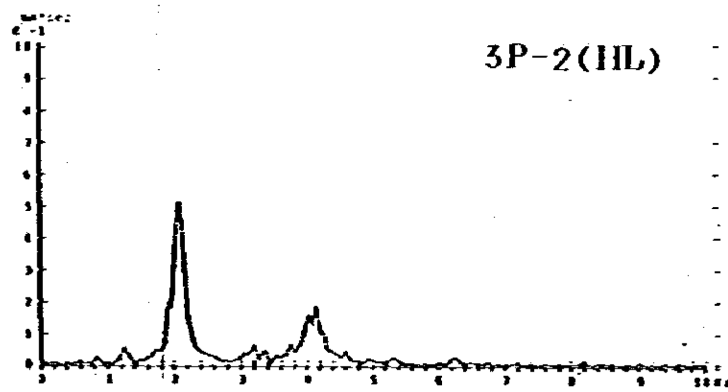
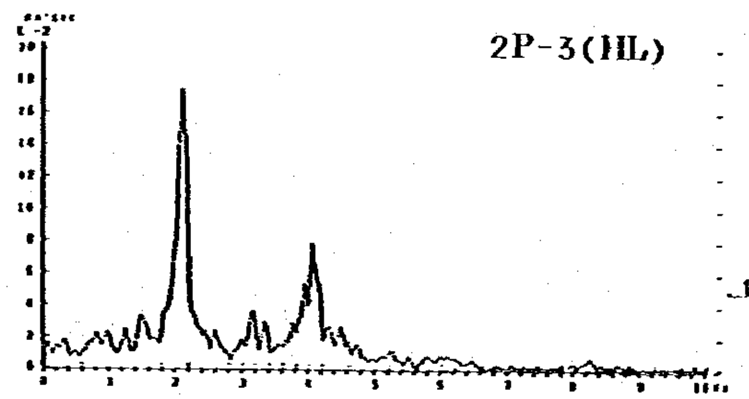
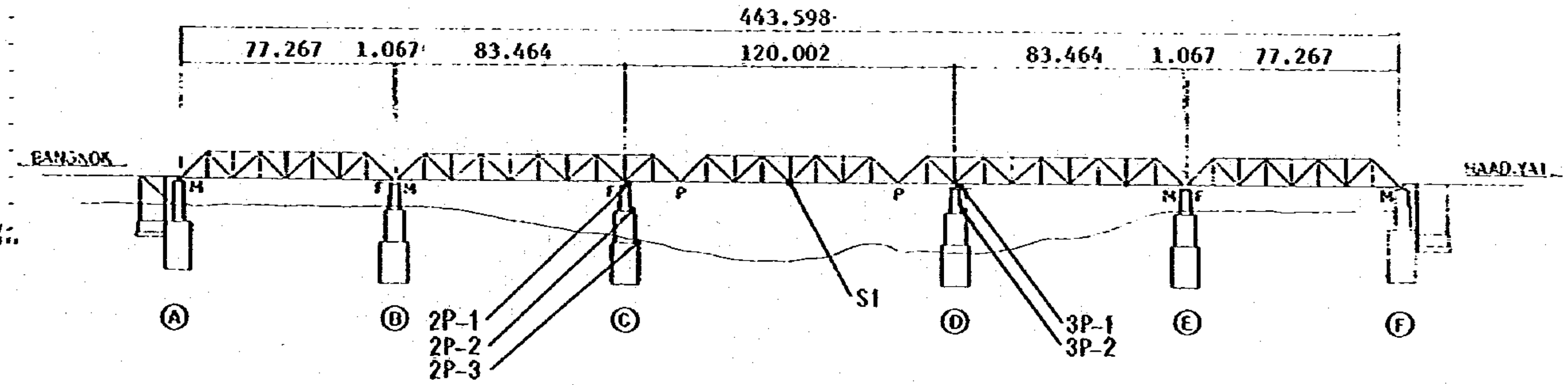


Fig. Fourier Spectra while Train Passing





Location of points measured

Fig. 2-3 Spectrom While Train is Passing

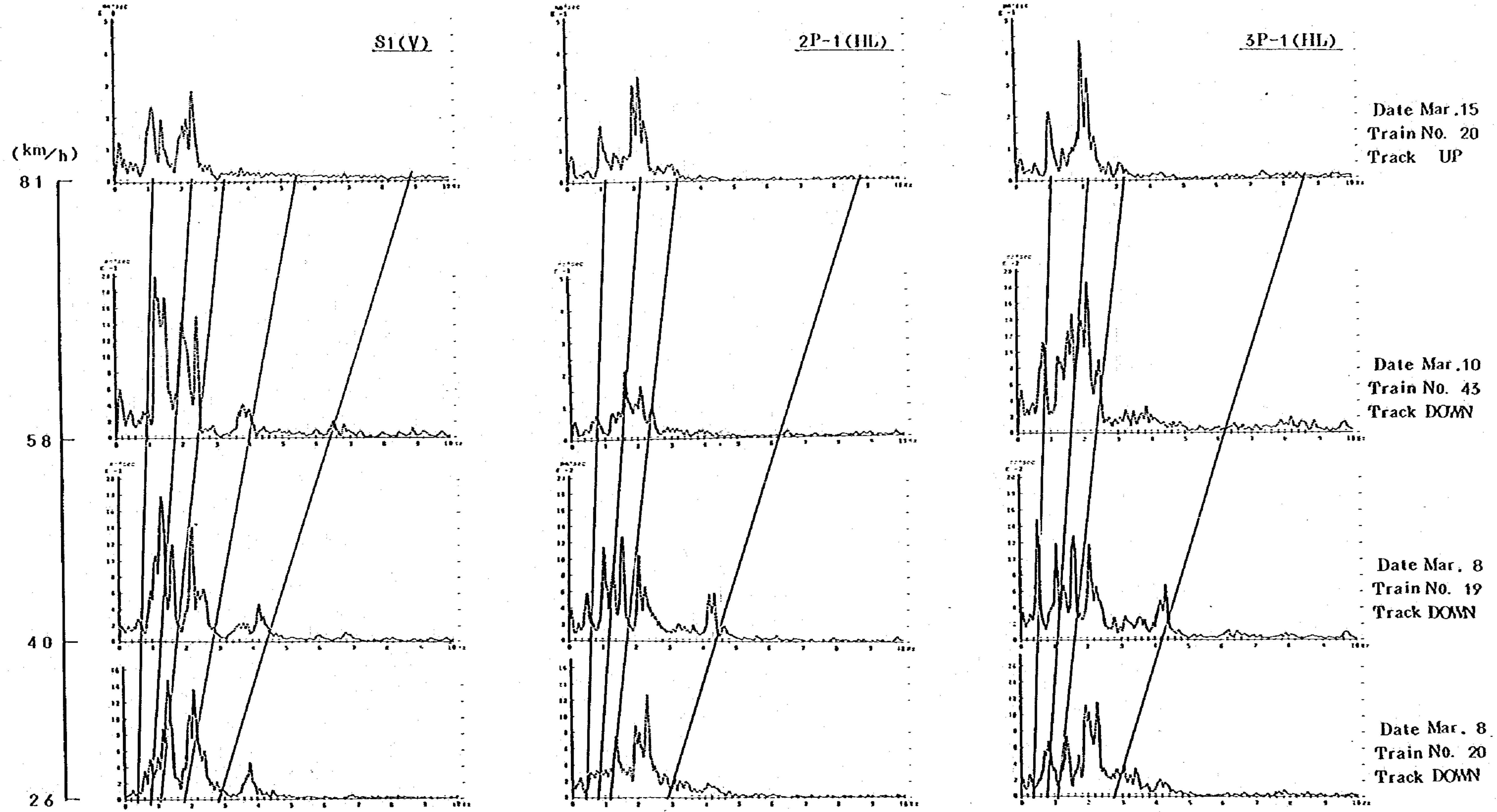


Fig. 2-4 Examination of Train Speed Effect (Passenger Train)

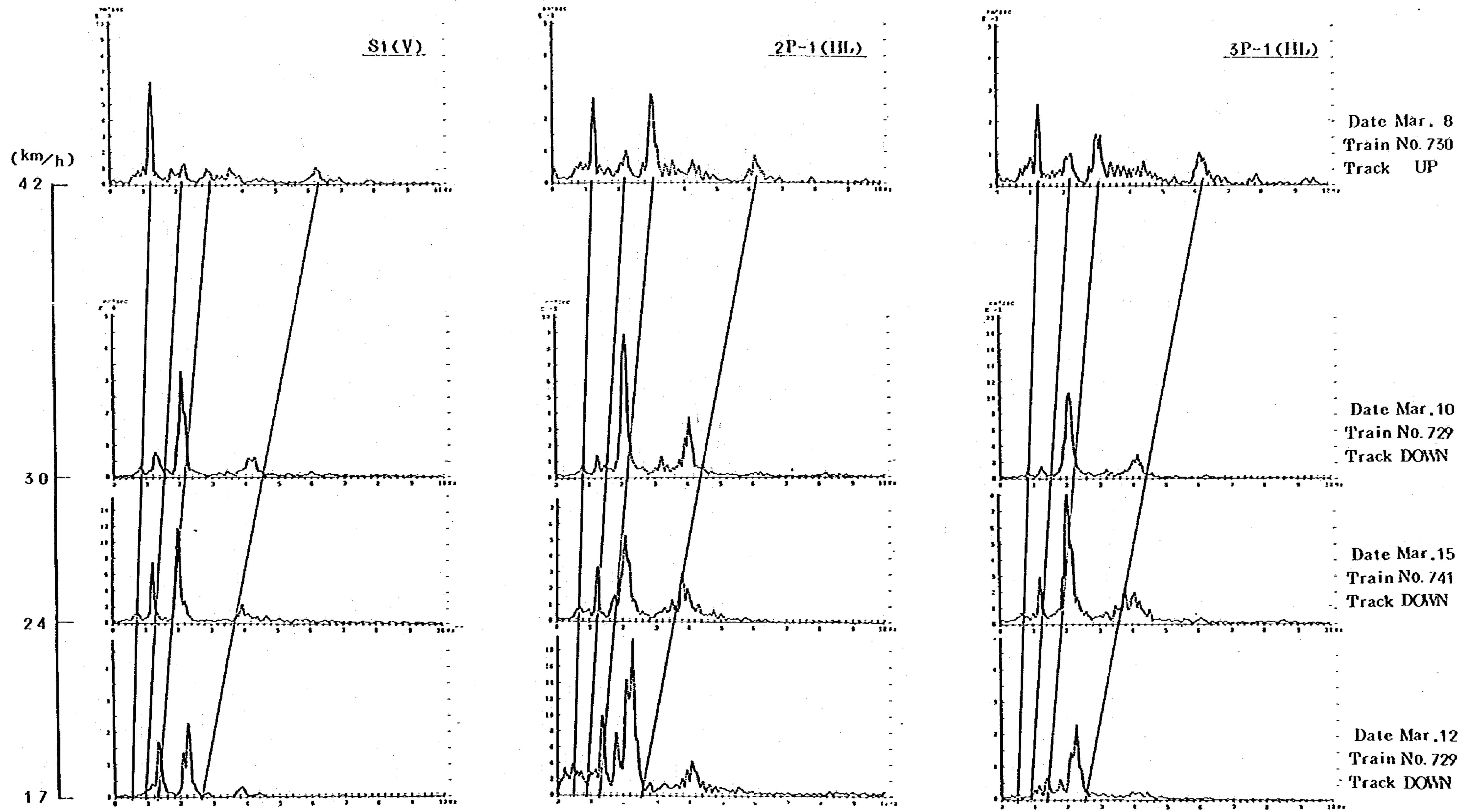


Fig. 2-5 Examination of Train Speed Effect (Freight Train)

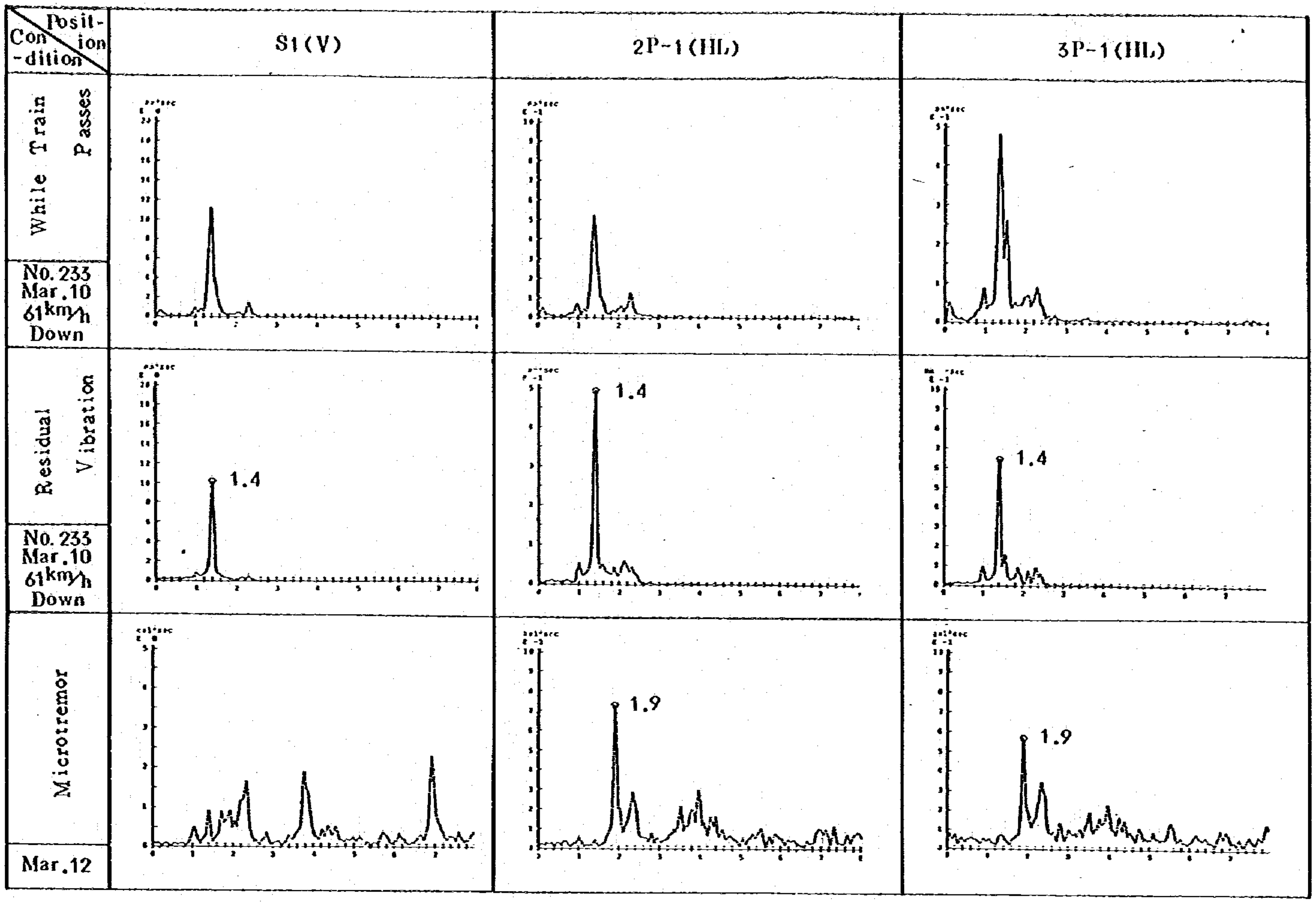
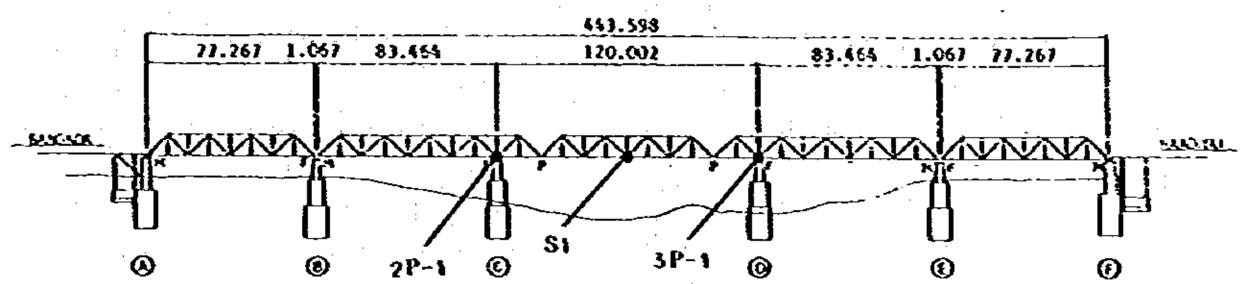


Fig. 2-6 Estimation for Natural Frequency in the Structure (1)



Location of points measured

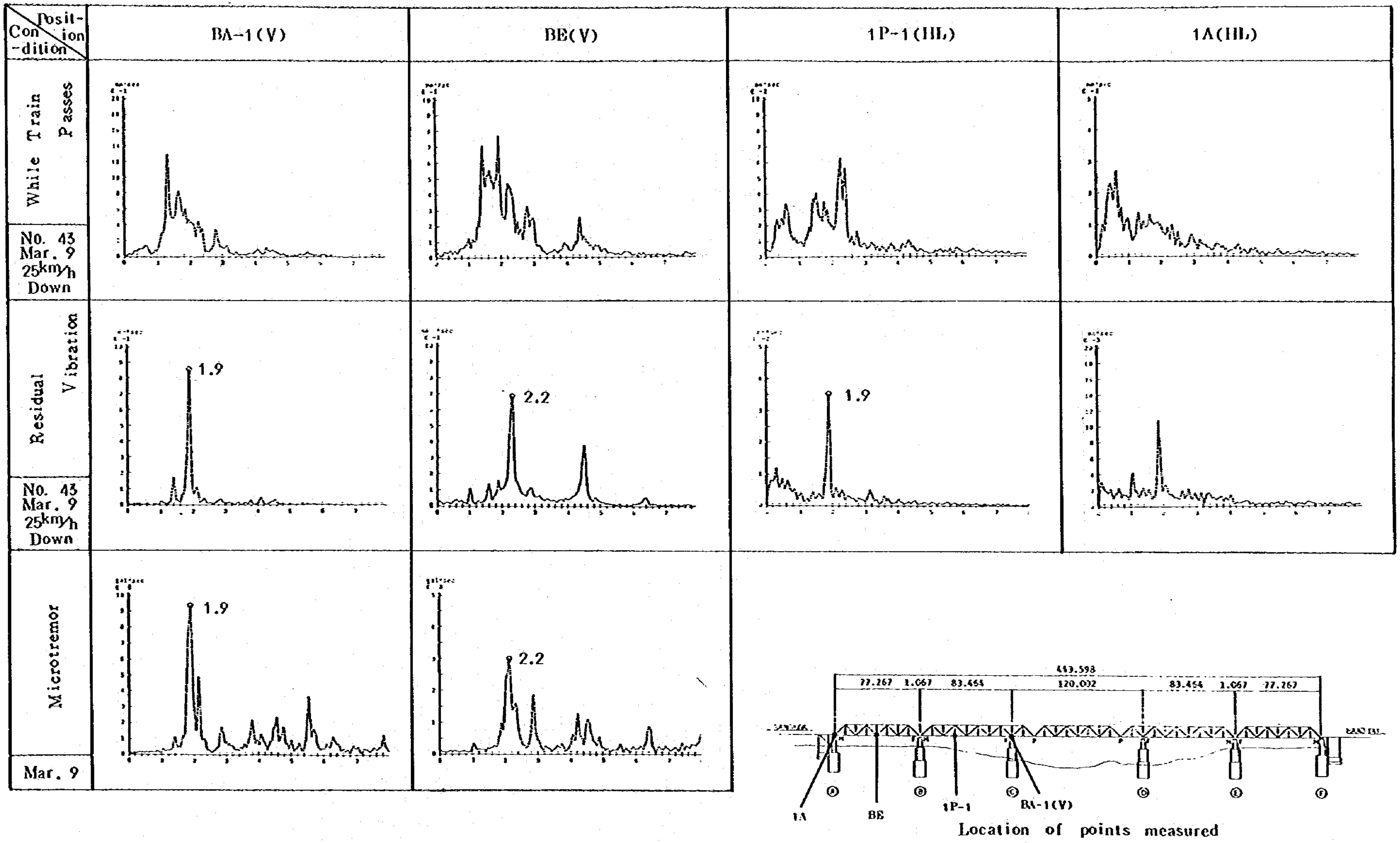
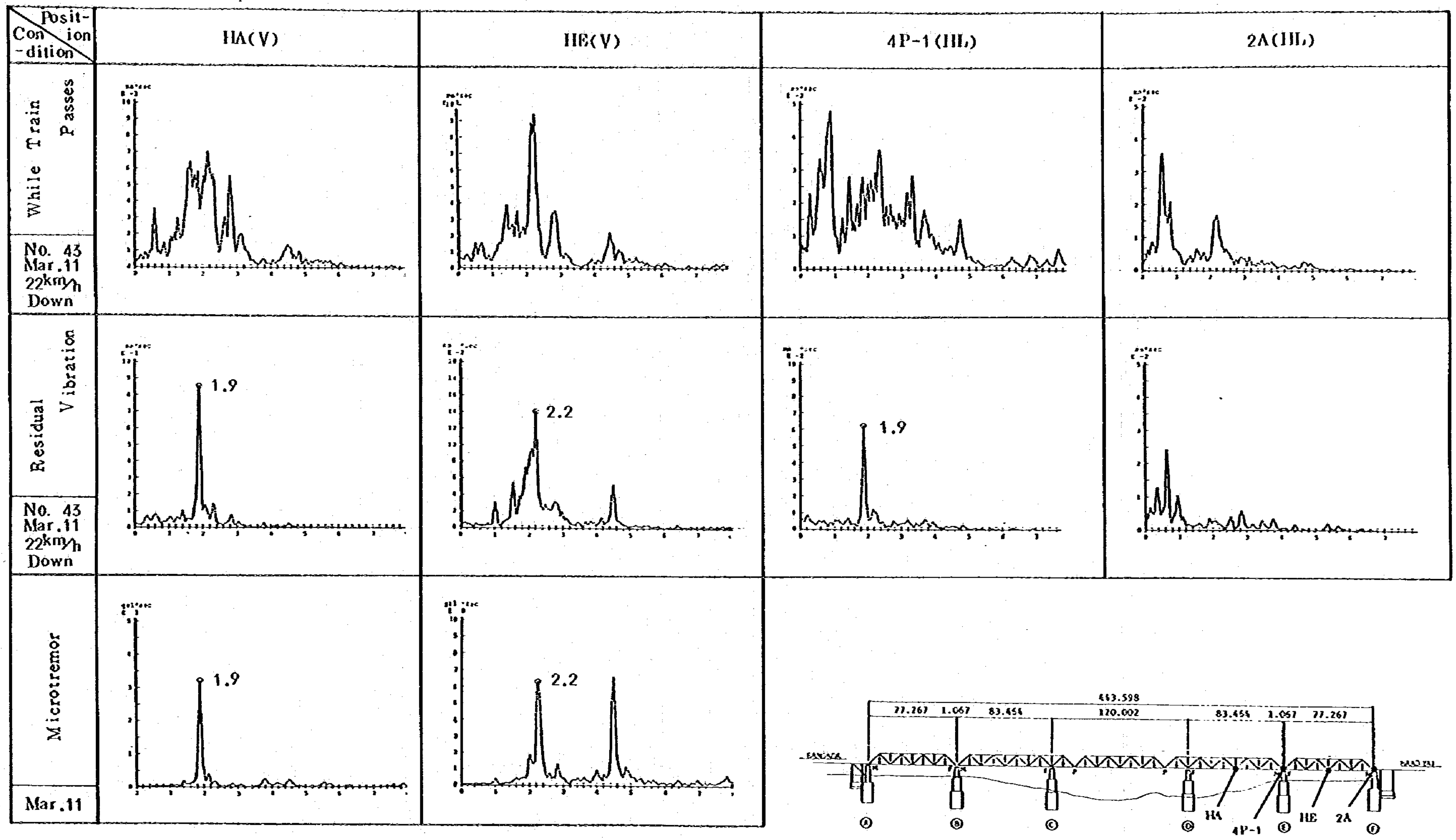


Fig. 2-7 Estimation for Natural Frequency in the Structure (2)



Location of points measured

Fig. 2-8 Estimation for Natural Frequency in the Structure (3)

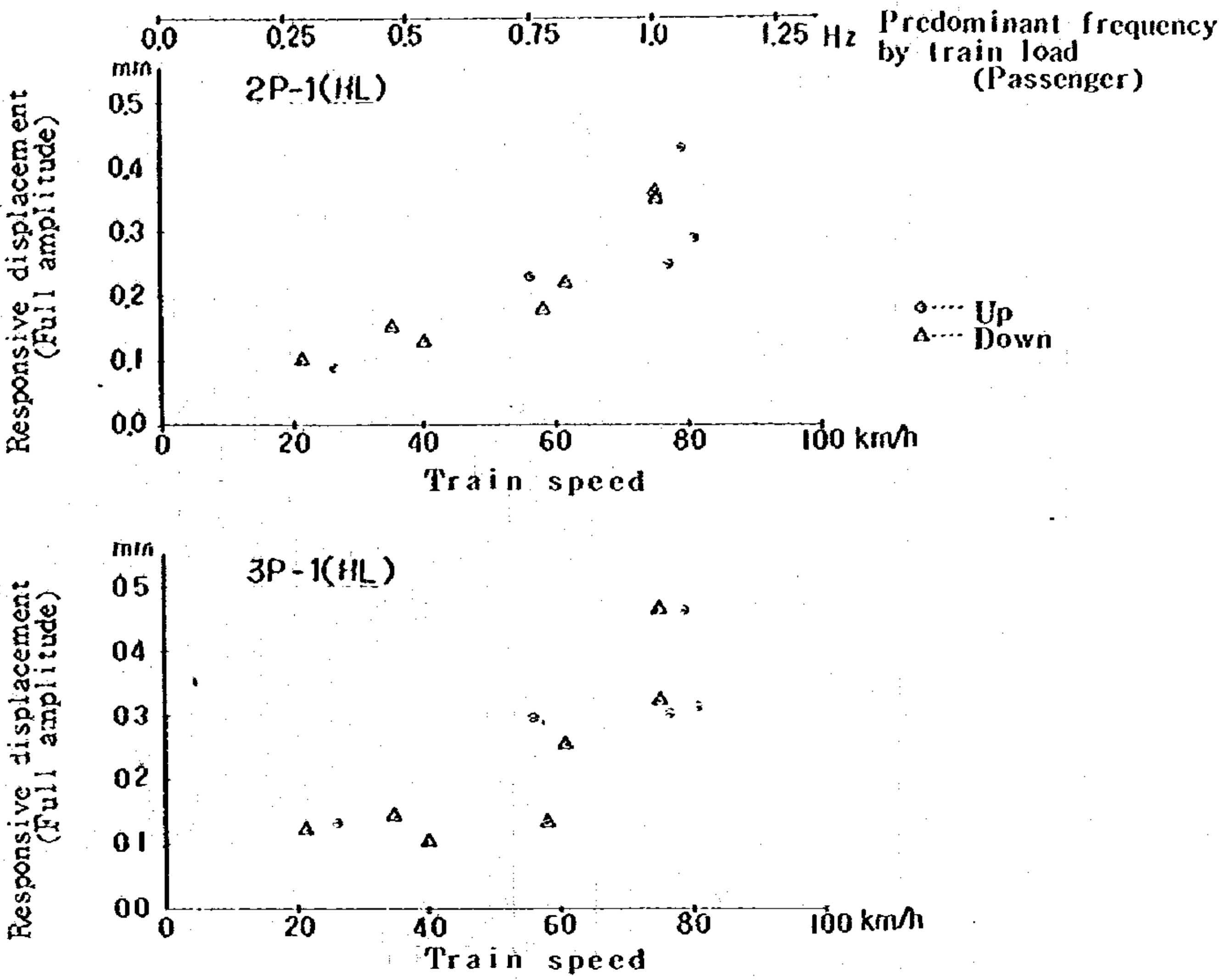
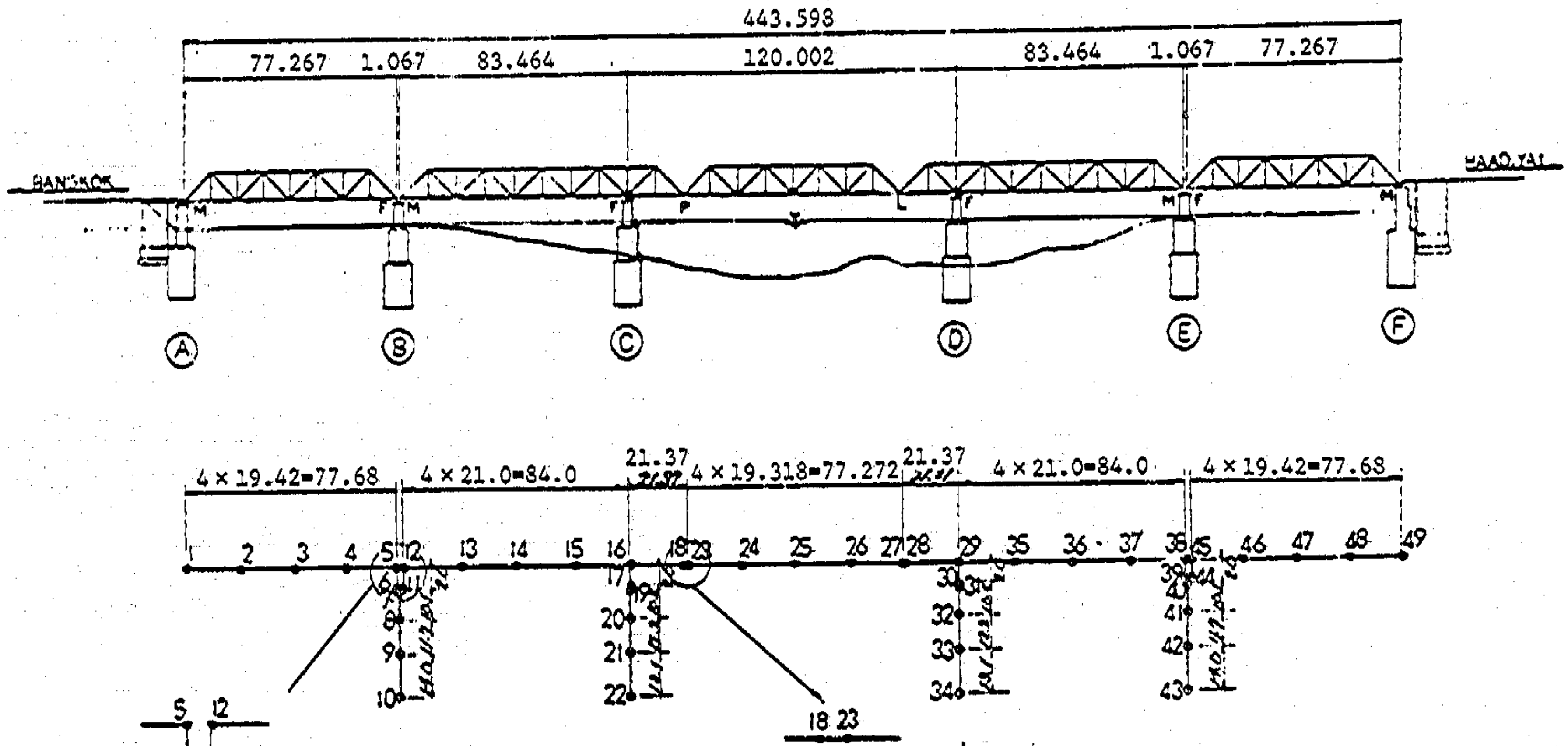
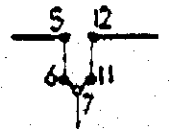


Fig. 2-9 Variation of Response due to Change in Train Speed

Table 2-1 Variation of Response due to Change in Train Speed

Date	Train No.	Speed	2P-1(HL)	3P-1(HL)	Track
3/8	20	26	0.09 mm	0.13 mm	Up
〃	43	21	0.10	0.12	Down
〃	19	40	0.13	0.10	〃
〃	11	35	0.15	0.14	〃
3/10	233	61	0.22	0.25	〃
〃	20	56	0.23	0.29	Up
〃	43	58	0.18	0.13	Down
3/12	44	79	0.48	0.46	Up
〃	20	77	0.25	0.30	〃
〃	43	75	0.35	0.46	Down
3/15	20	81	0.29	0.31	Up
〃	43	75	0.36	0.32	Down





Displacements or joints between 6 and 7, 7 and 11 are continuous in both directions of horizontal and vertical. And any rotating moment is transferred. Same as in the joints between 17~19, 30~31, 39~40 and 40~44.

Displacement of joints between 18 and 23 is continuous in both directions of horizontal and vertical, and rotating moment is also continuous through rotating cushion describable by measurement. Same as in joints between 27 and 28.

Fig. 2-10 Vibration Model

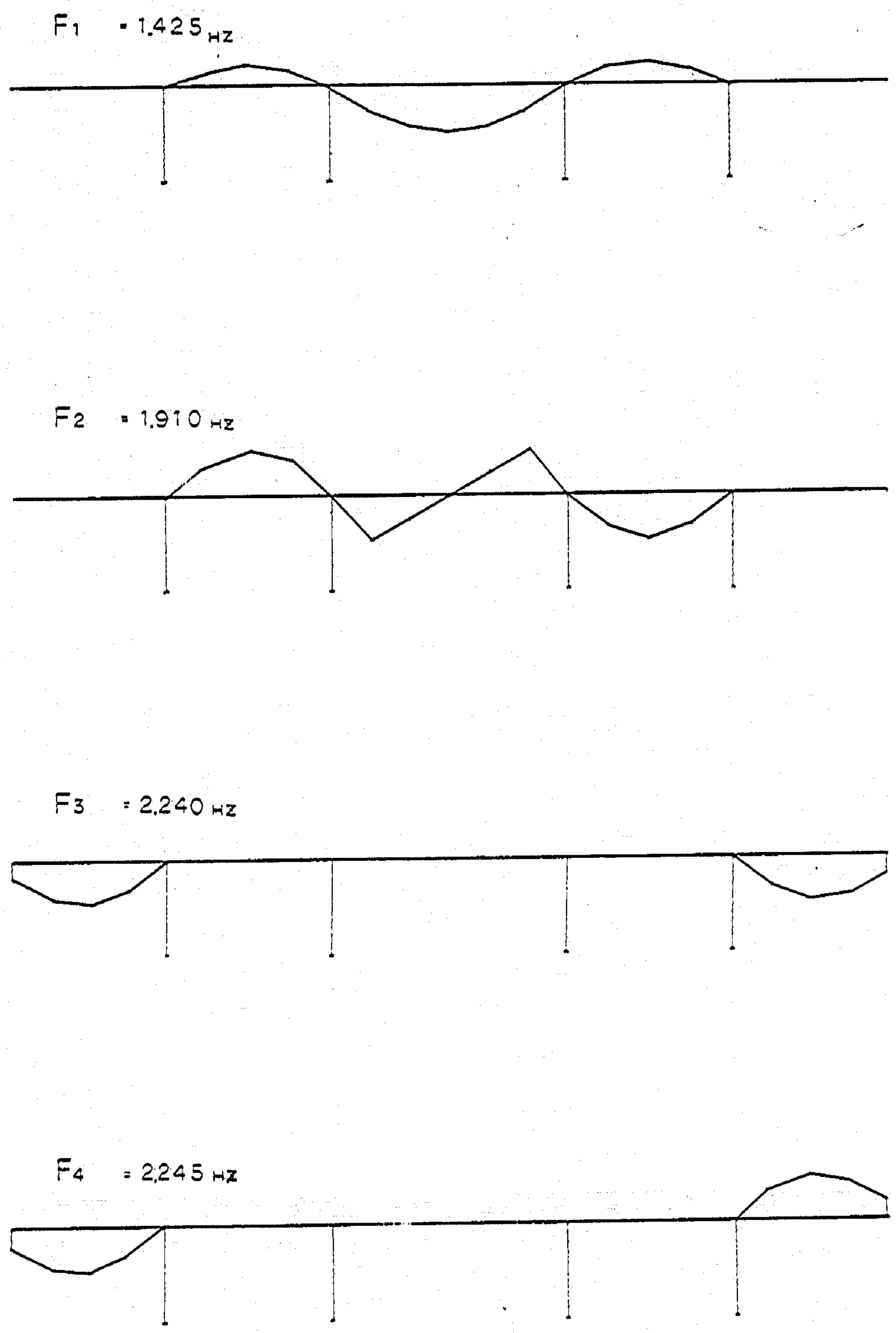


Fig. 2-11 Vibration Mode

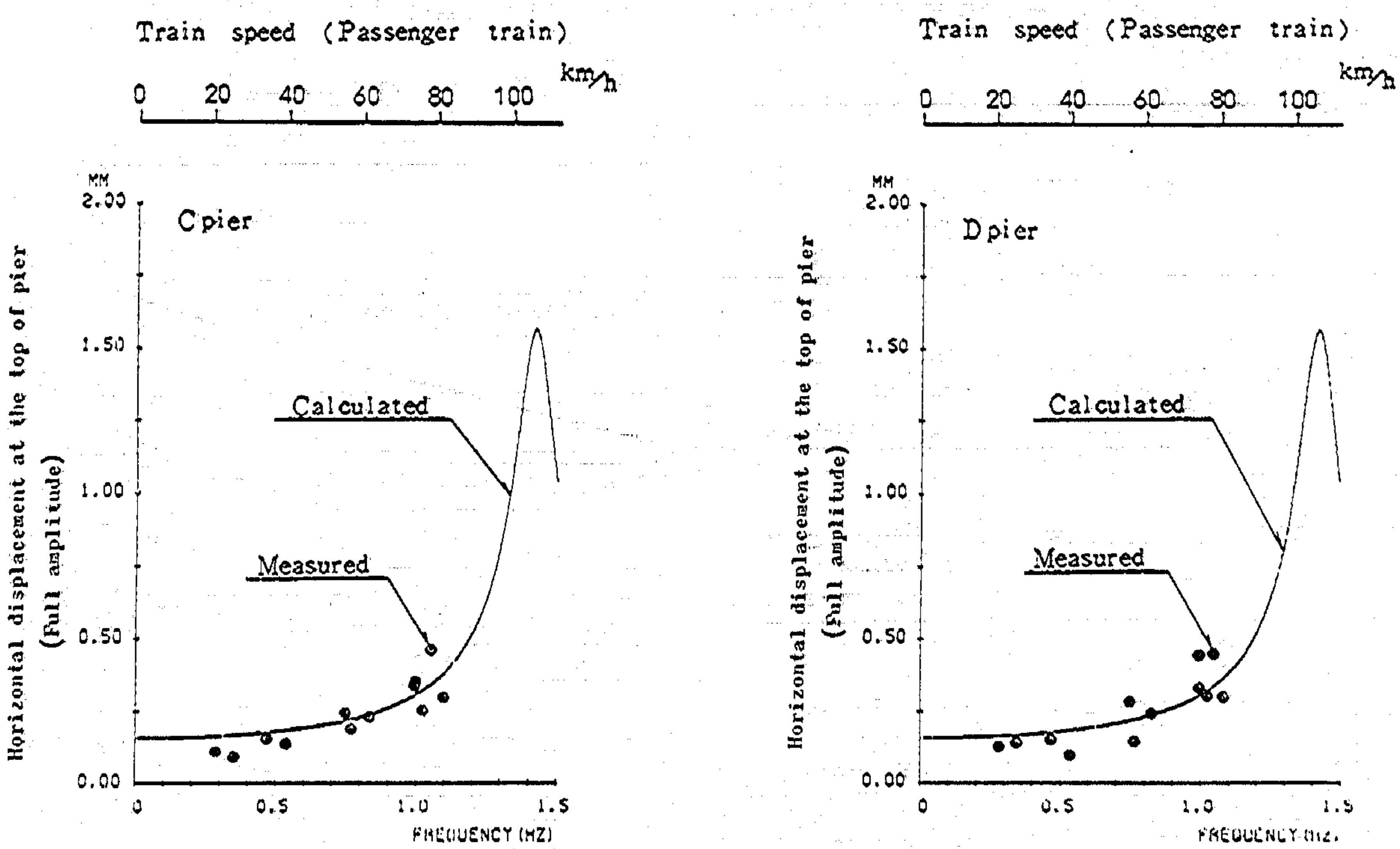
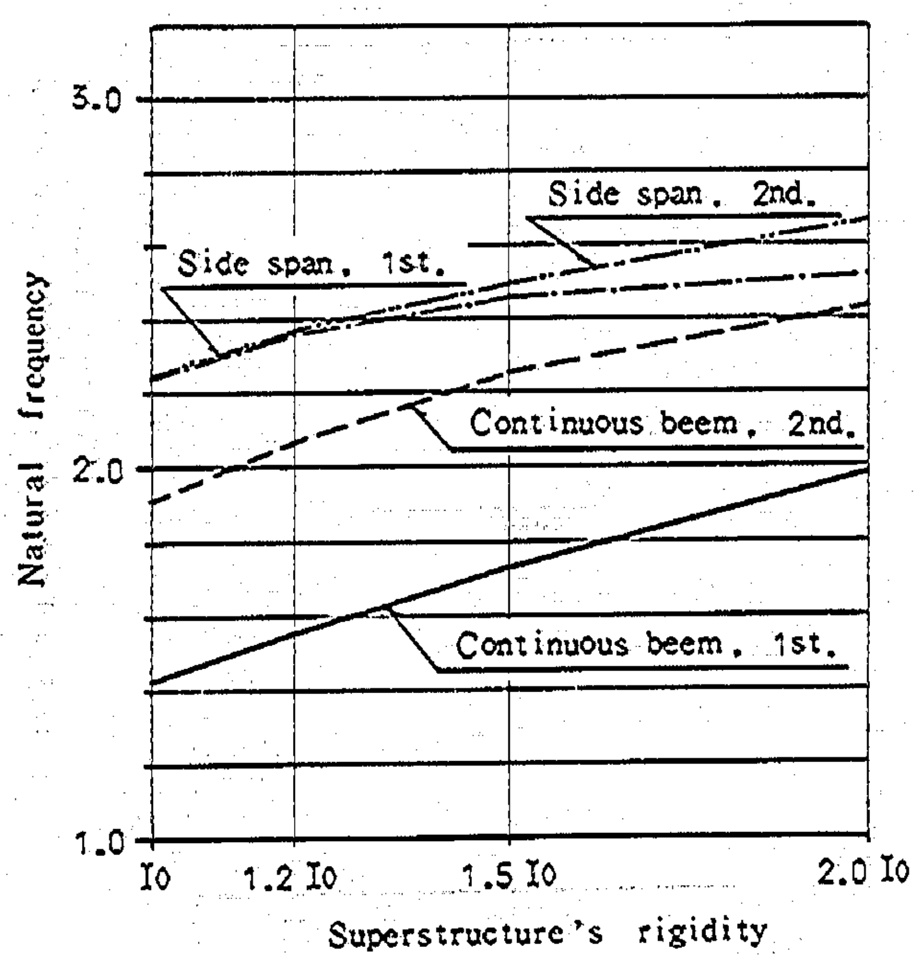


Fig. 2-12 Variation of Responses due to Change in Train Speed



Configuration out line of vibration	Io	1.2 Io	1.5 Io	2.0 Io
Continuous beem, first mode	1.425 (Hz)	1.557	1.732	1.982
Continuous beem, second mode	1.910	2.063	2.253	2.437
Side span, first mode	2.240	2.360	2.464	2.524
Side span, second mode	2.245	2.370	2.497	2.677

Io: Present status

Fig. 2-13 Variation of Natural Frequency due to Doubling of Track

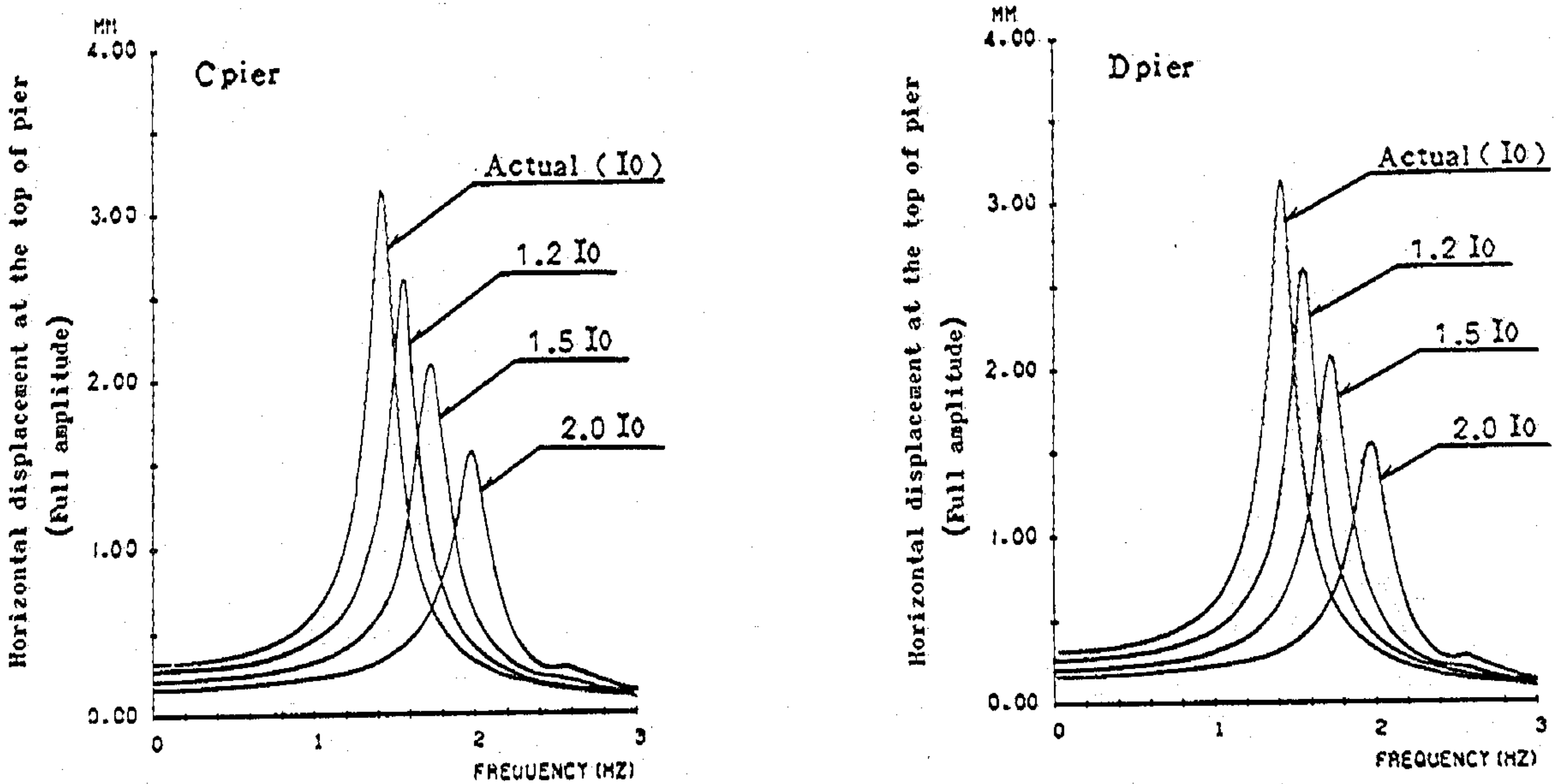


Fig. 2-14 Variation of Responses due to Doubling of Truck

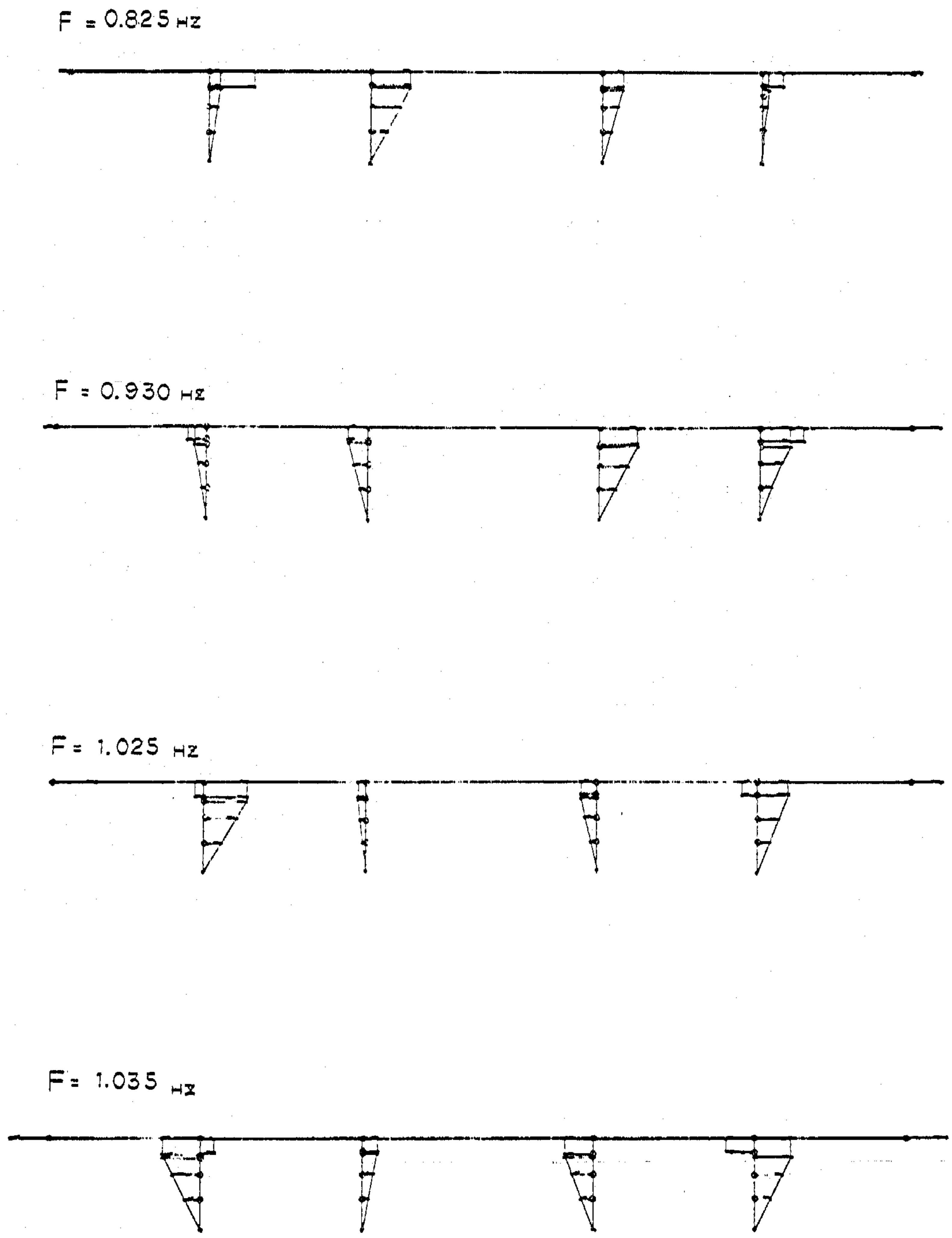


Fig. 2-15 Vibration Mode (Horizontal Behavior)

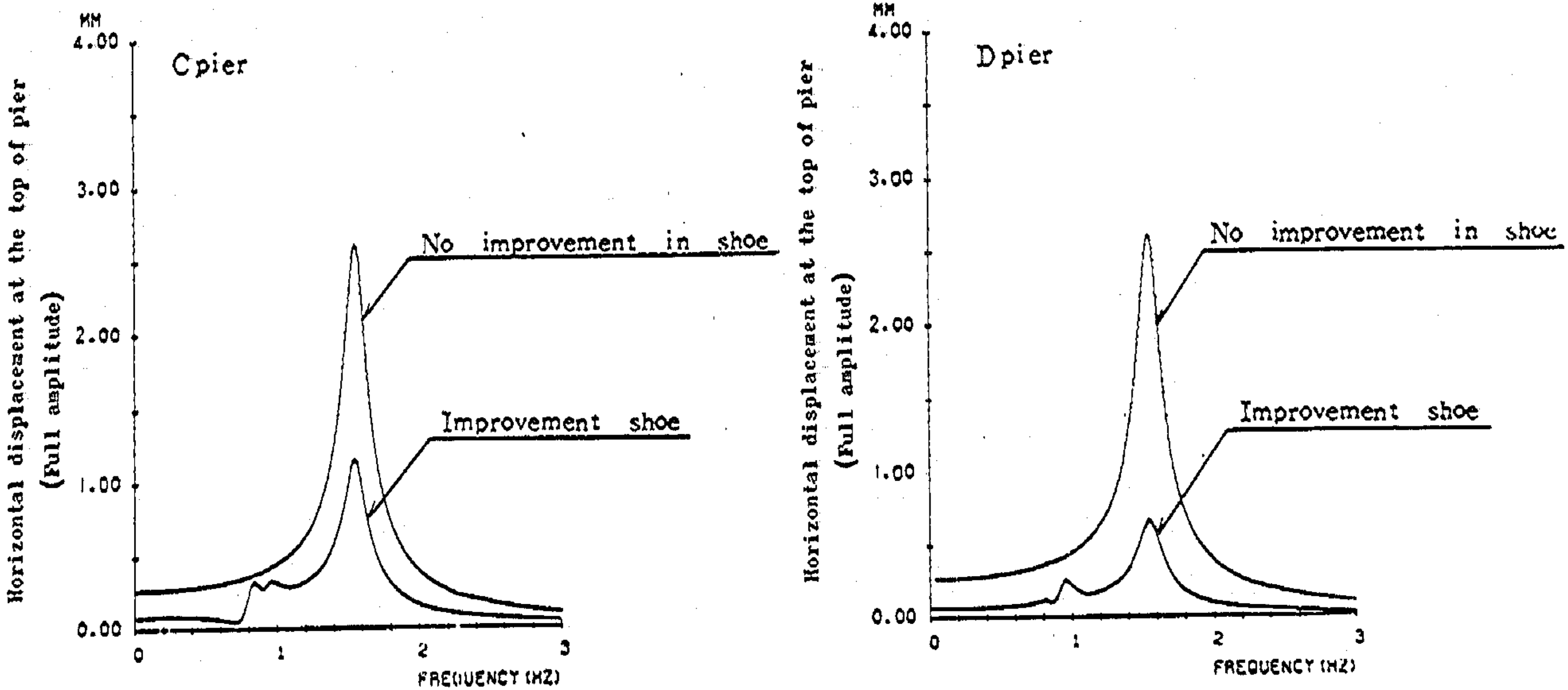
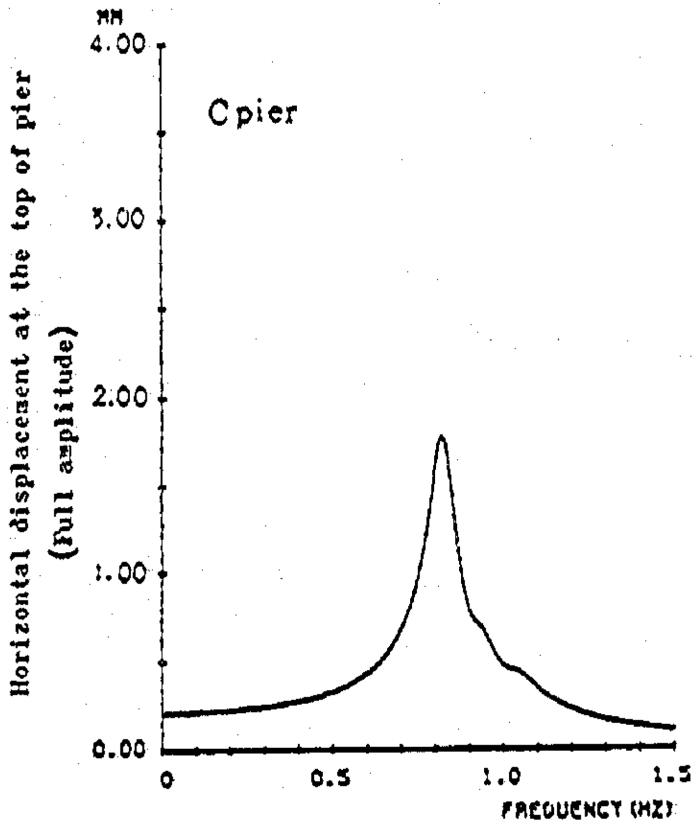


Fig. 2-16 Variation of Responded Values due to Changes in Shoe Conditions



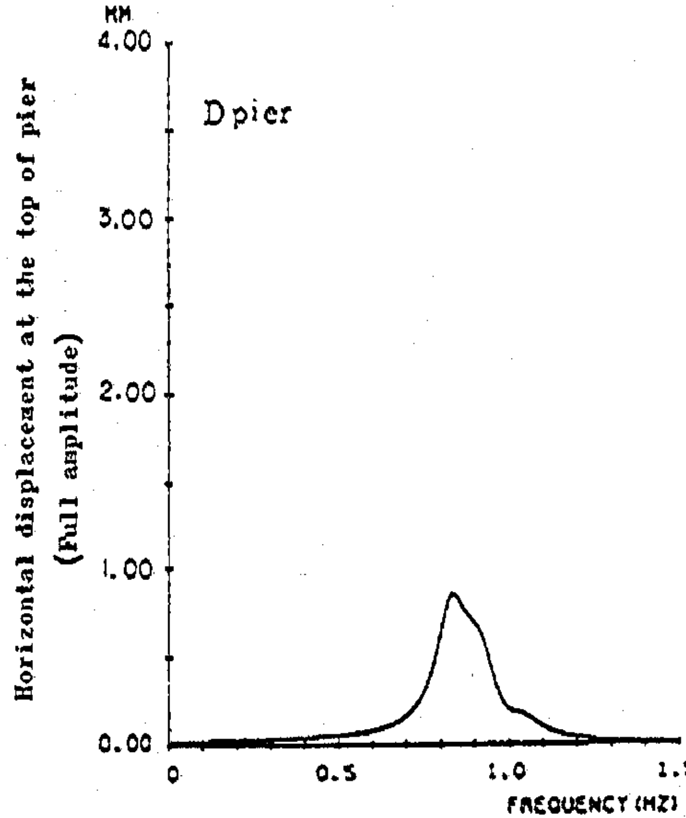


Fig. 2-17 Responses Acted by Horizontal Load on Link Position
(Movable shoe, Pin and Link conditions are excellent.)

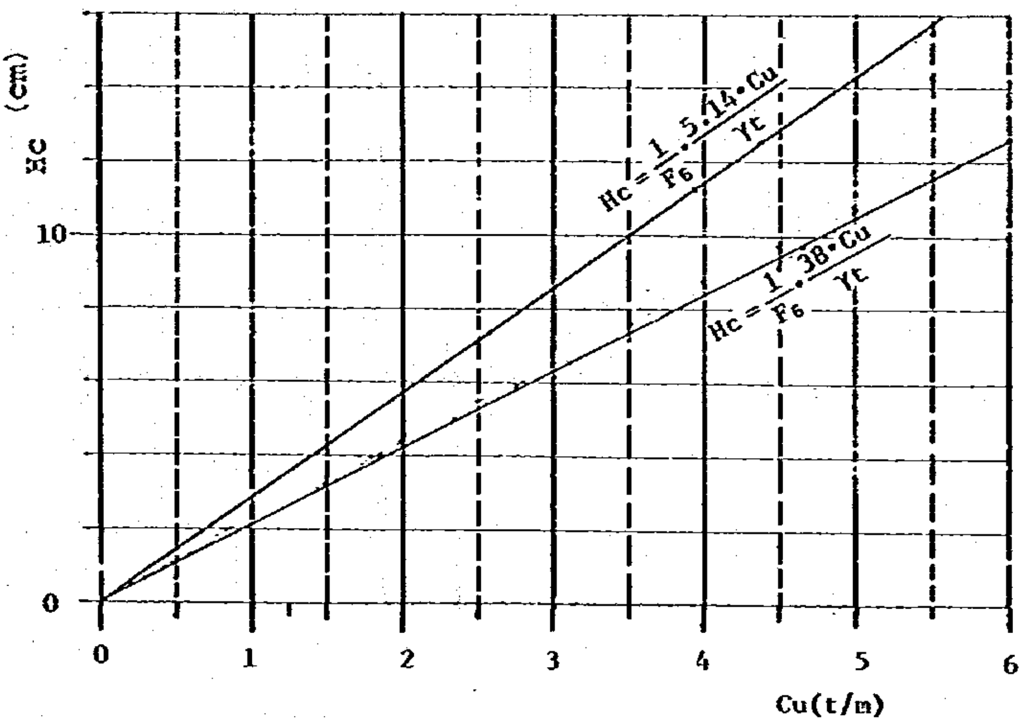
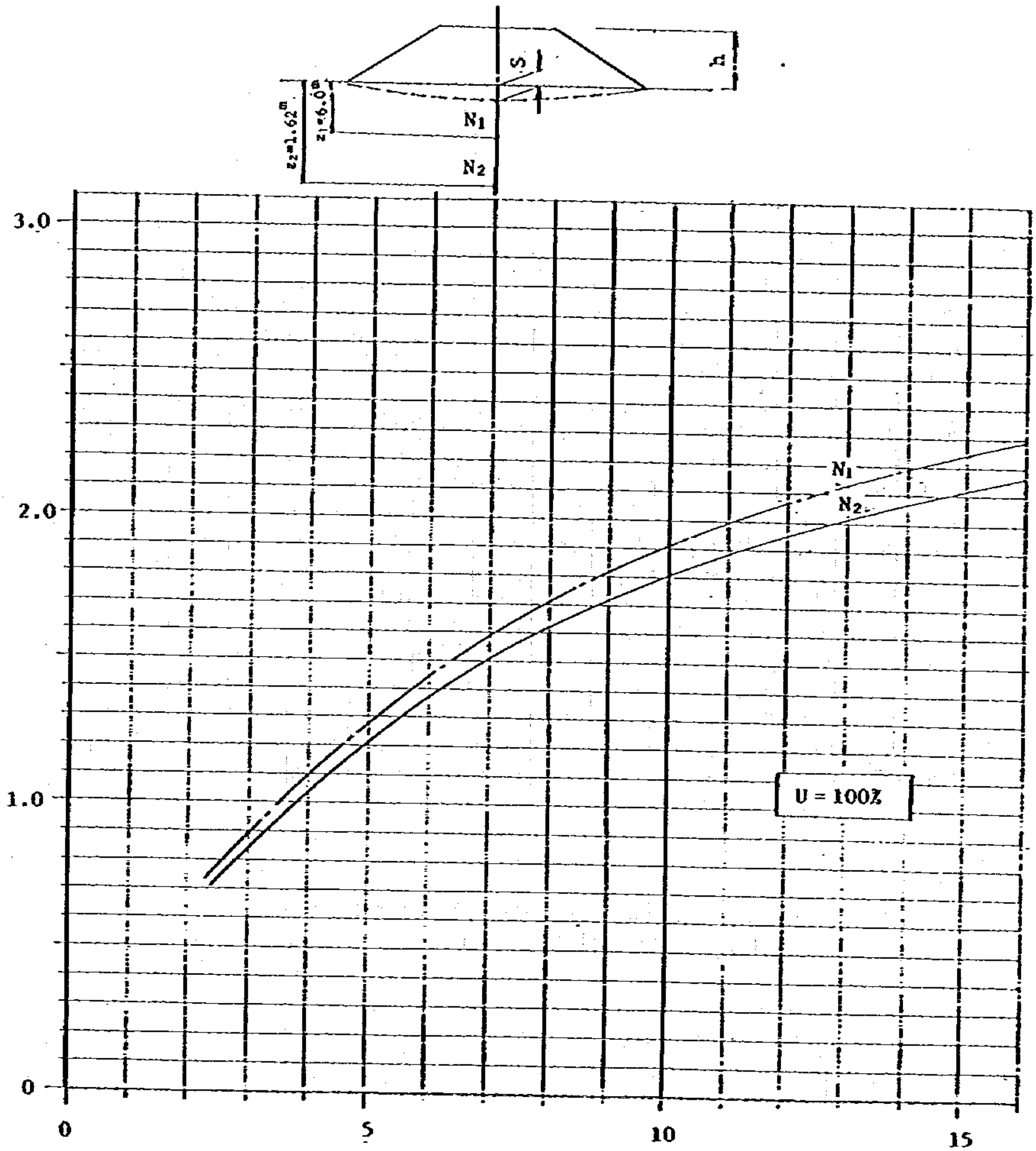


Fig. 2-18 Relation between Critical Embankment Height and Soil Cohesion



Settlment S (m)

Embankment height h (m)

Fig. 2-19 Total Settlment after 100% Consolidation

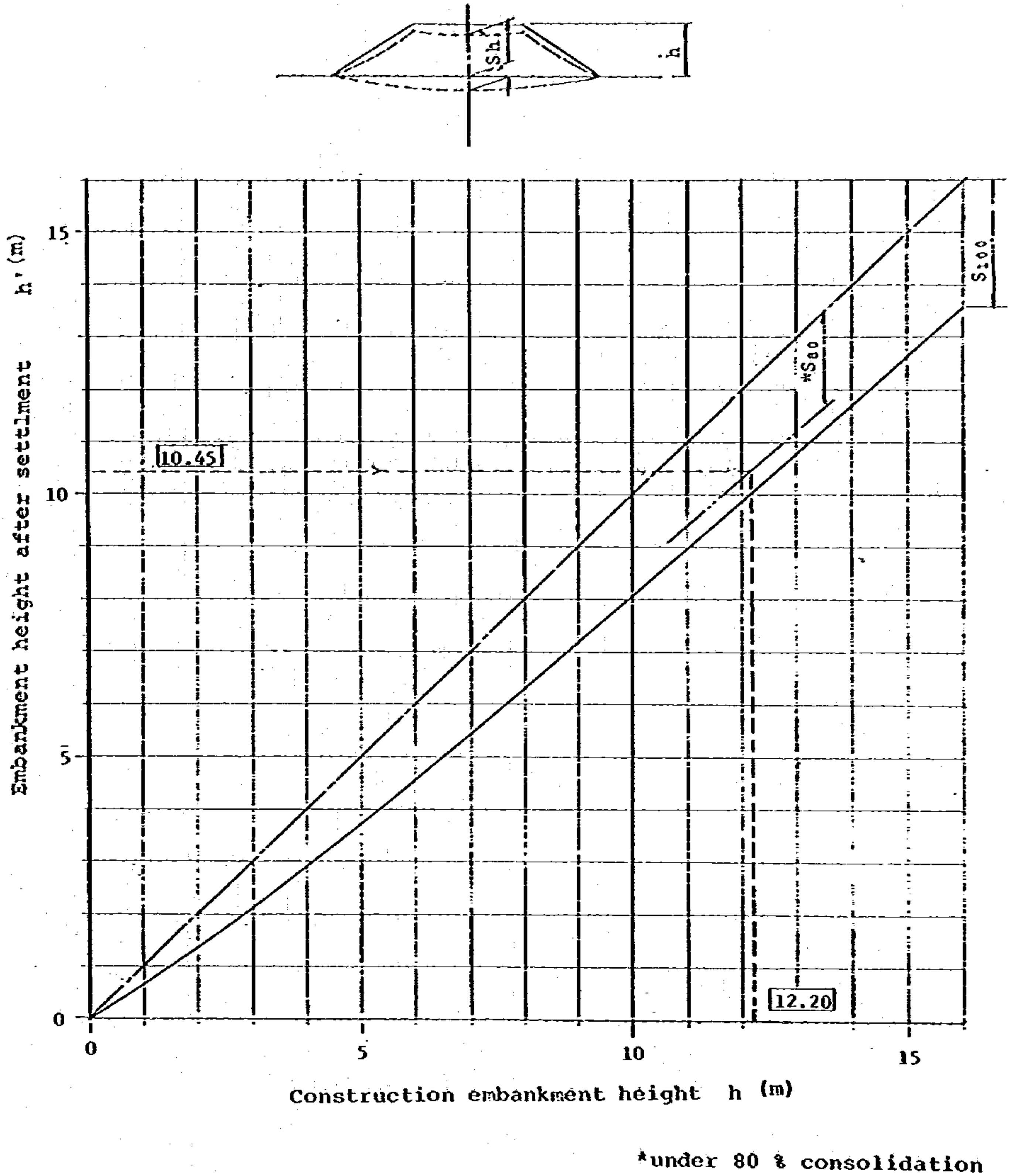


Fig. 2-20 Relation between Construction Embankment Height and Embankment Height after 100% Consolidation

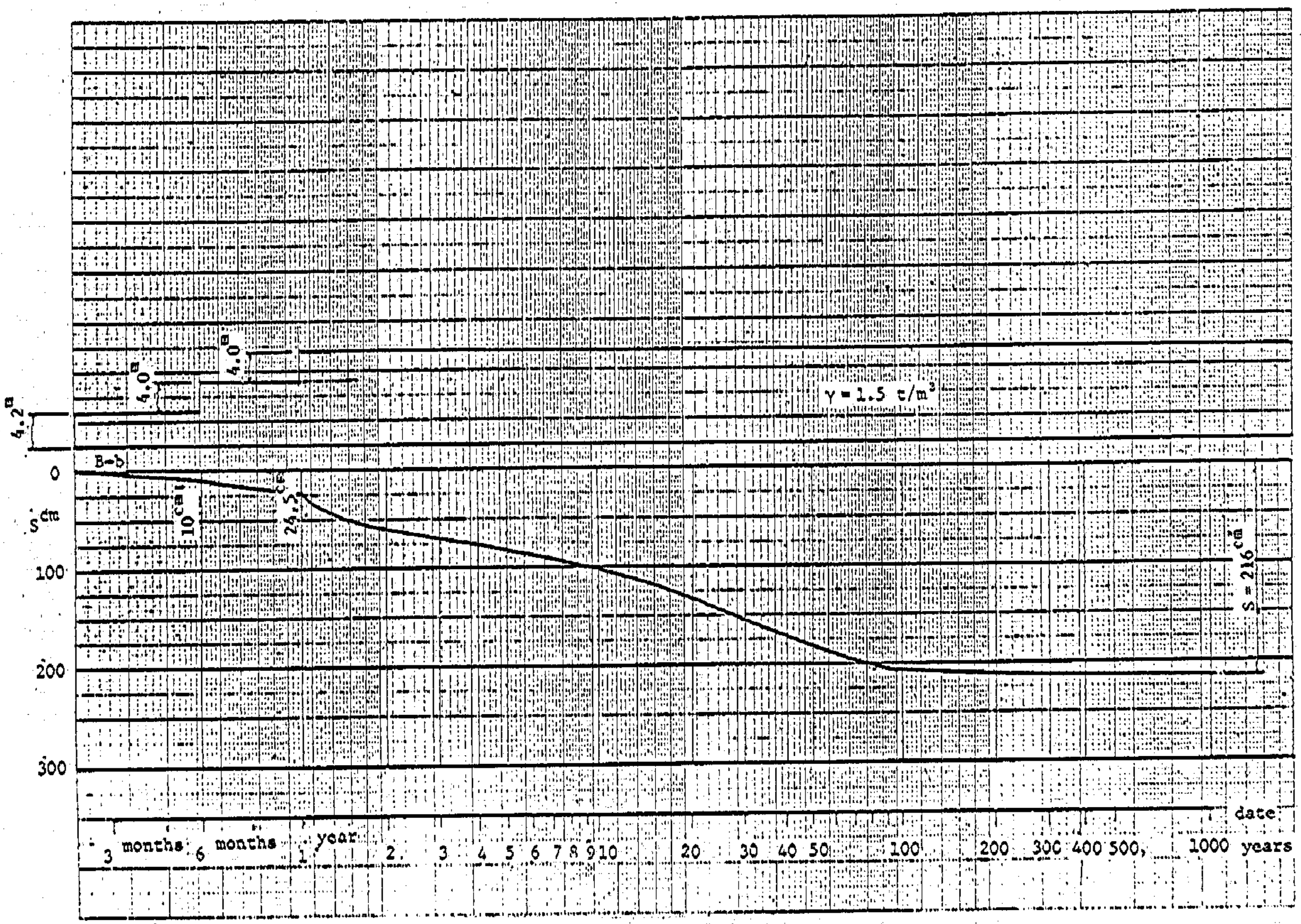


Fig. 2-21 Settlment Curve

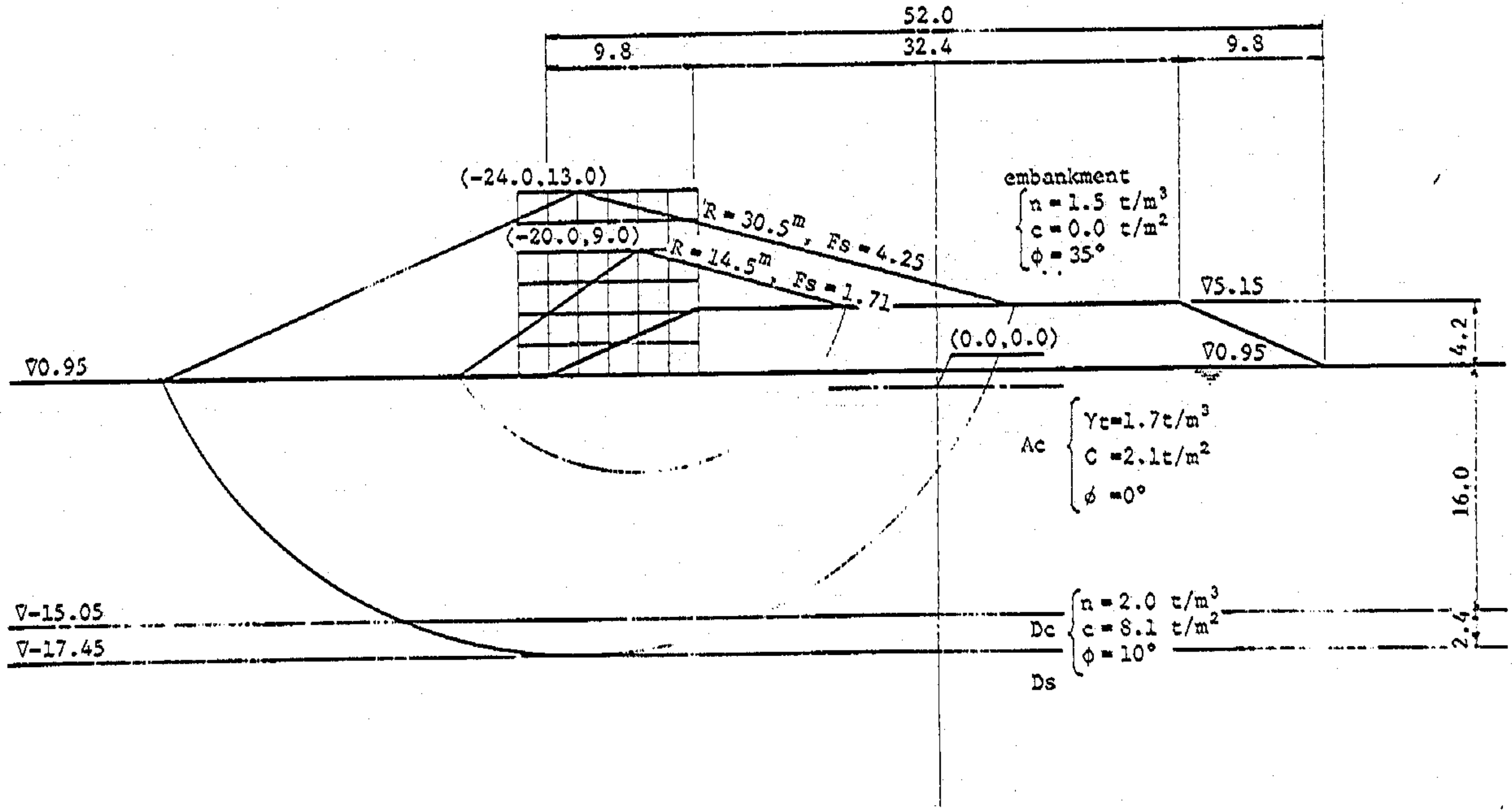


Fig. 2-22 1st Stage: 1st Embankment

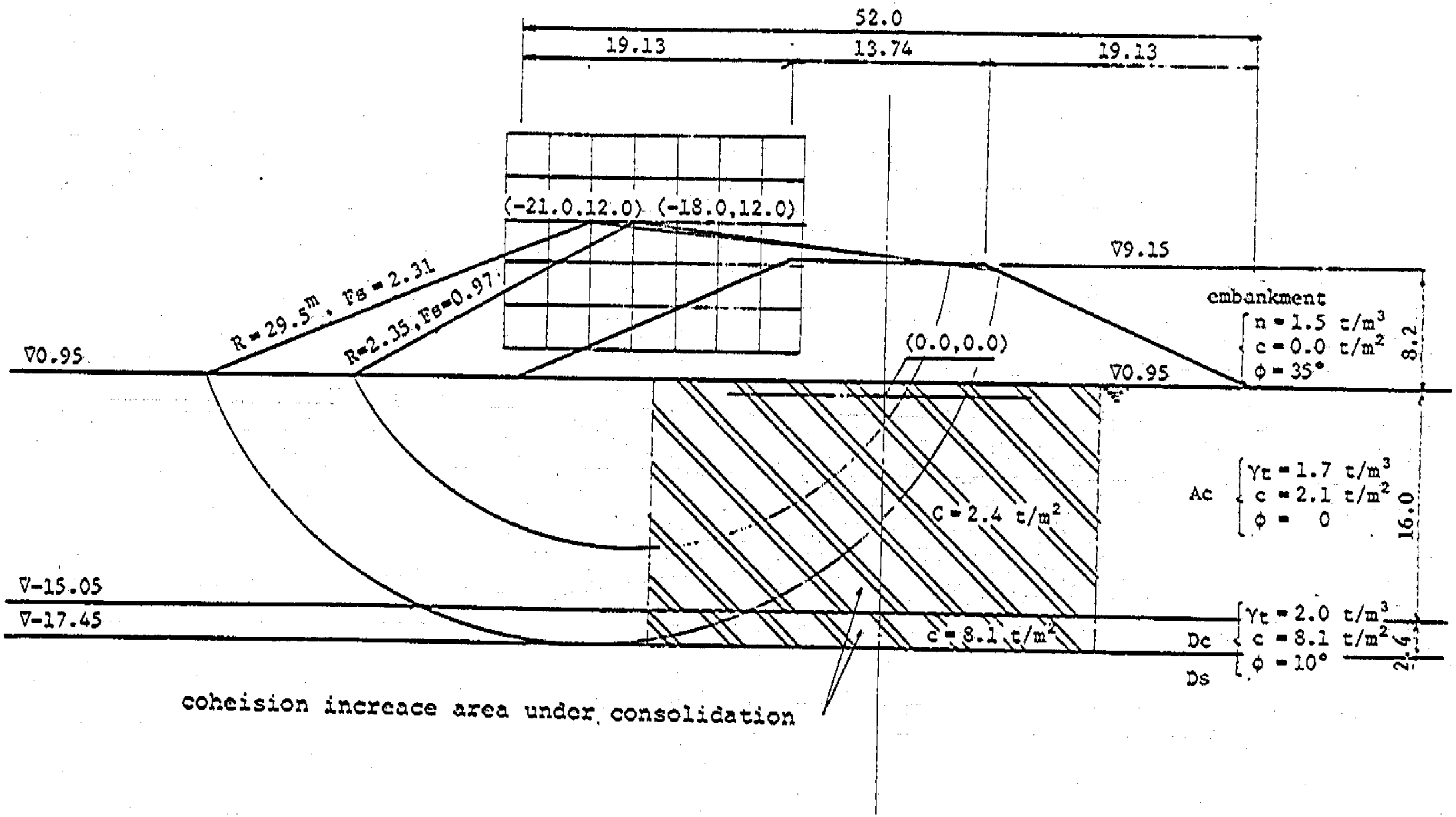


Fig. 2-23 2nd Stage: 2nd Embankment (t = 6 months)

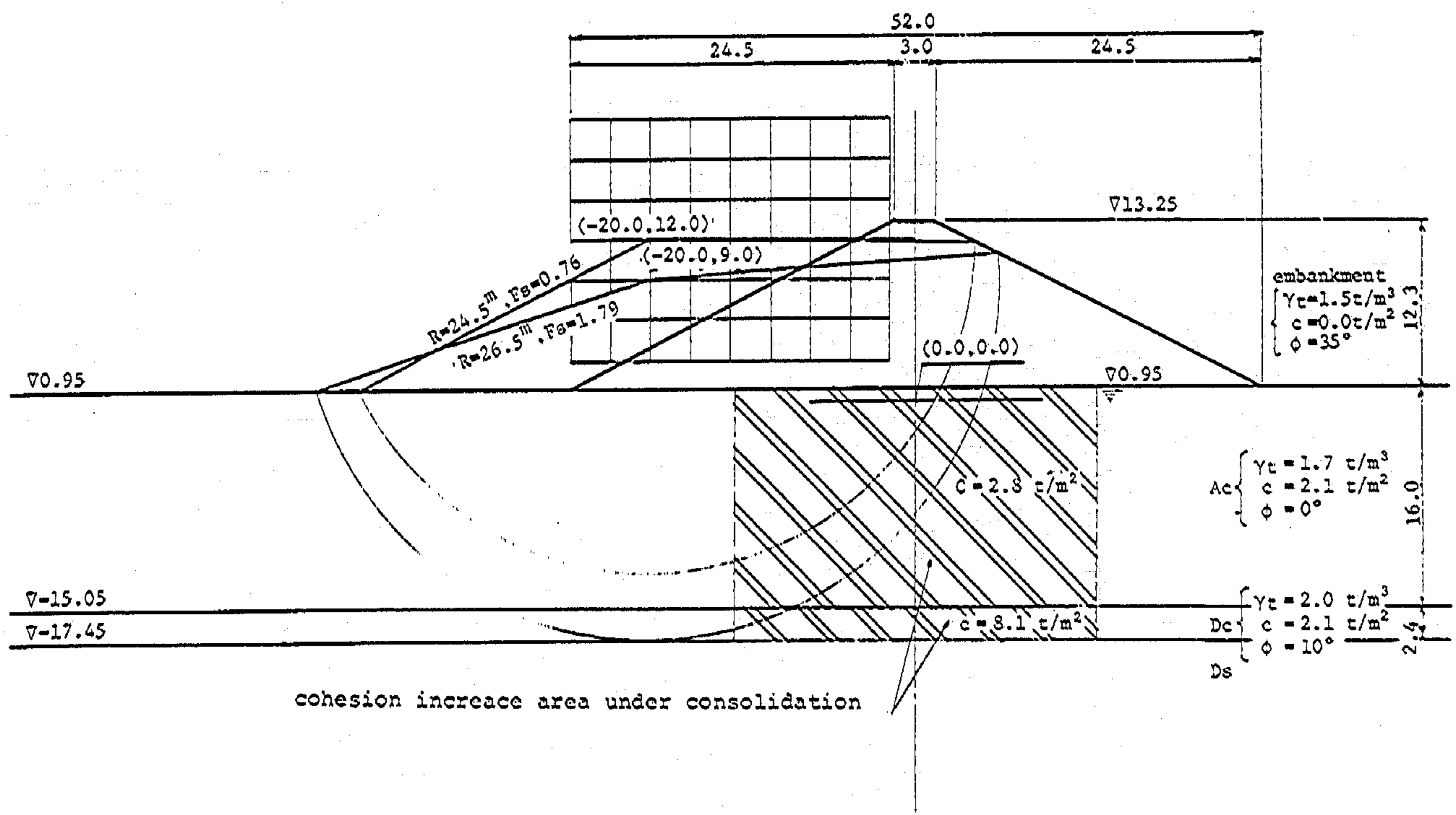


Fig. 2-24 3rd Stage: Last Embankment (t=12 months)

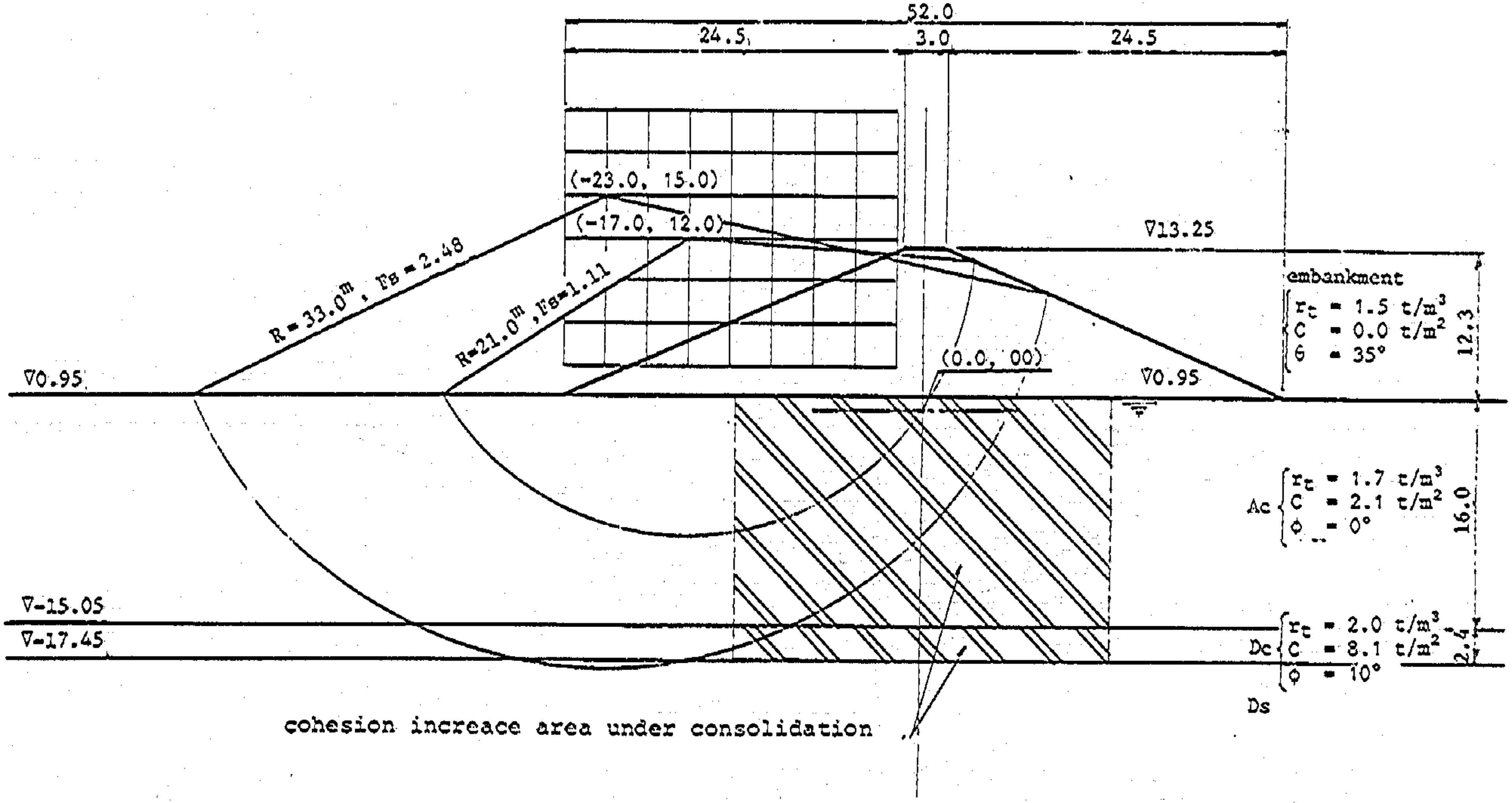


Fig. 2-25 4th Stage: (t = 10 years)

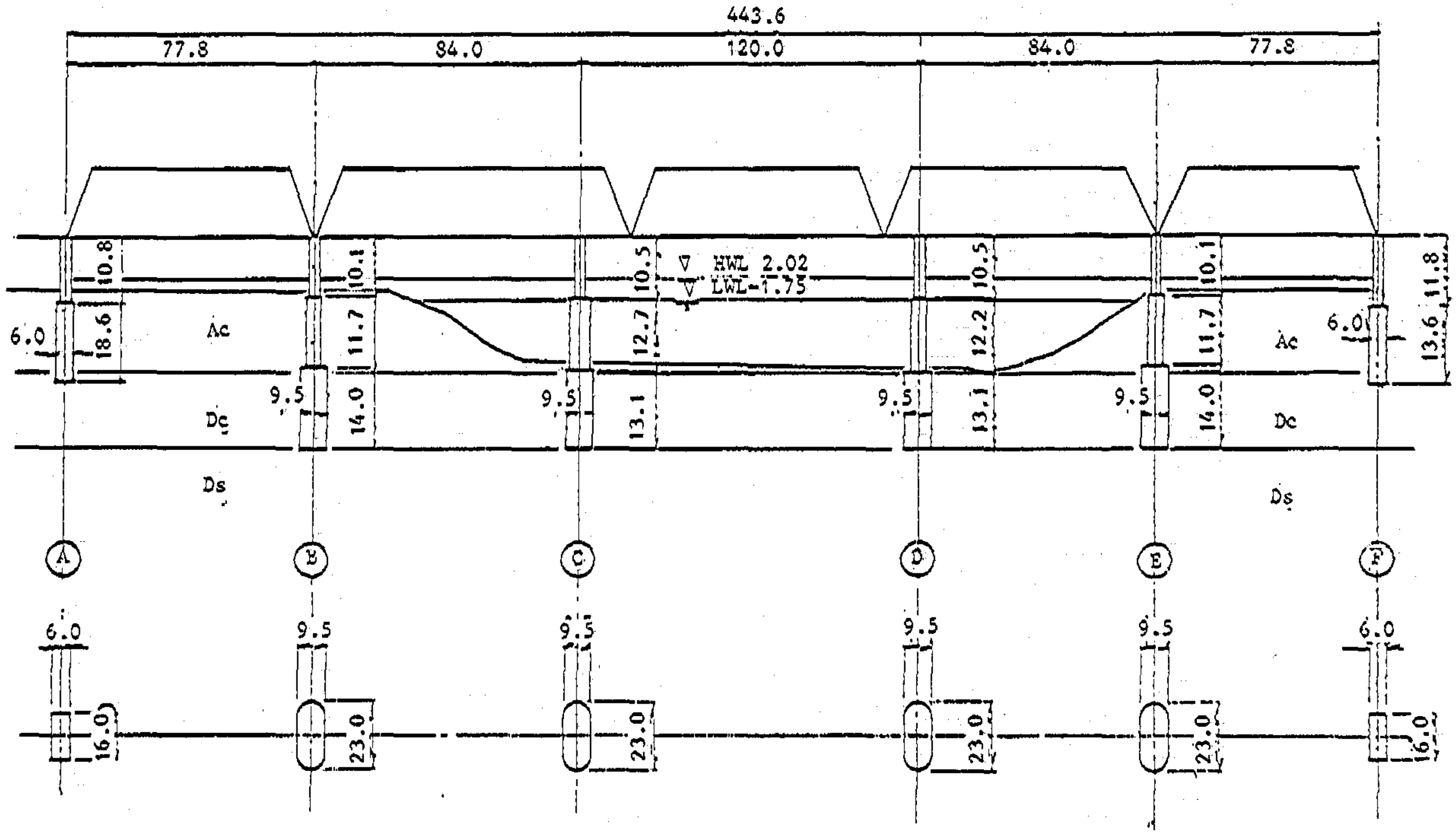


Fig. 2-26 General Arrangement of Rama VI Bridge

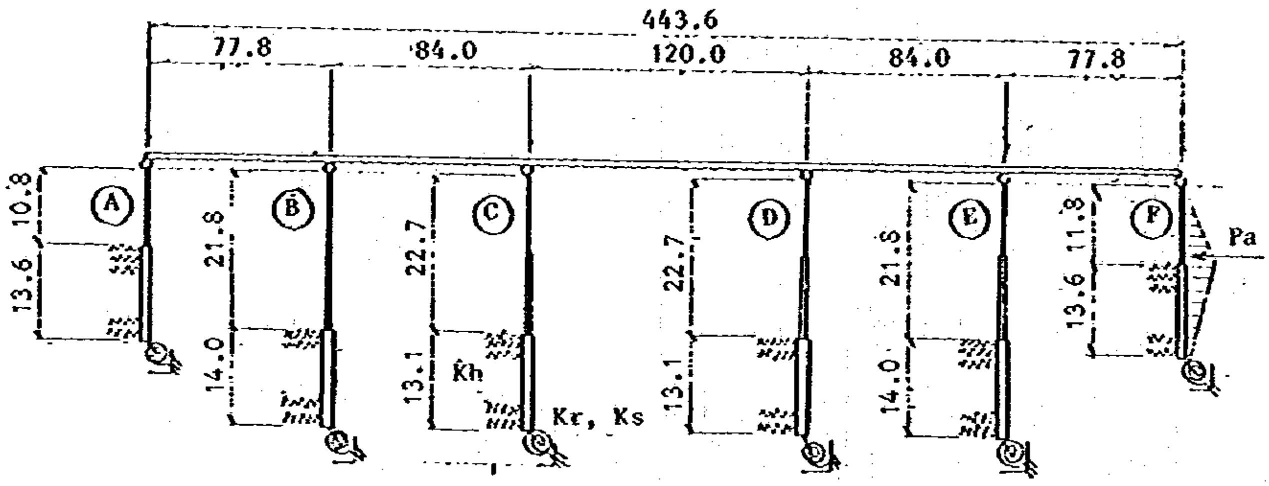


Fig. 2-27 Simulation Analysis Model of Rama VI Bridge

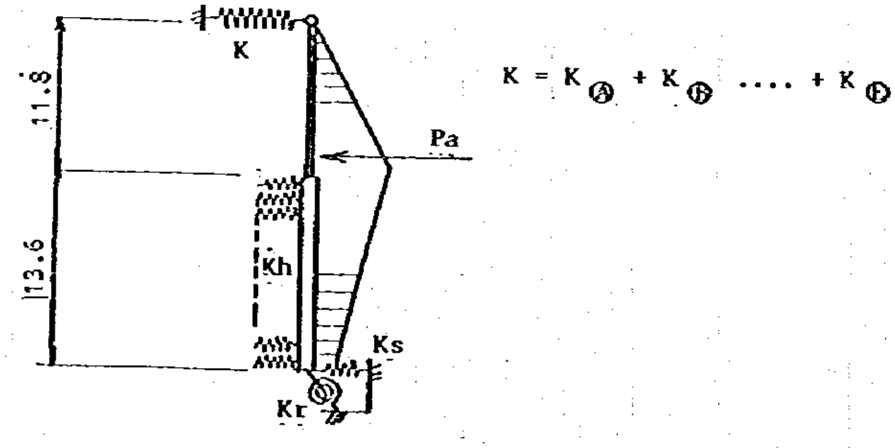
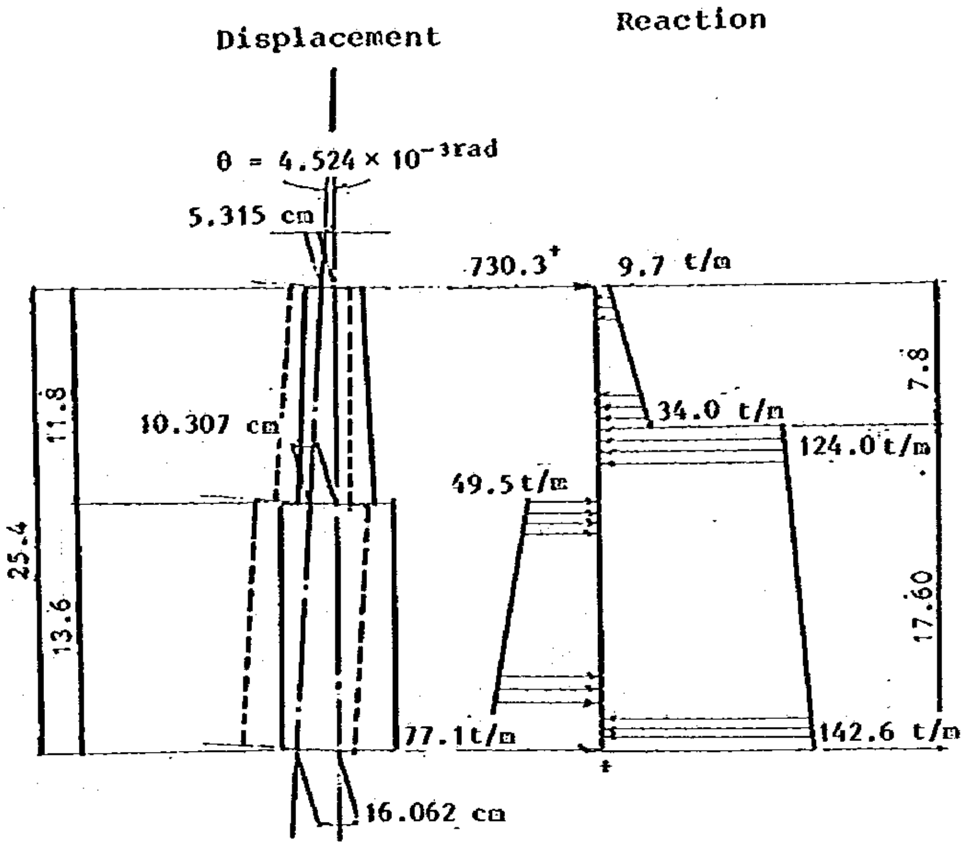


Fig. 2-28 Simulation Analysis Model of F - Abutment



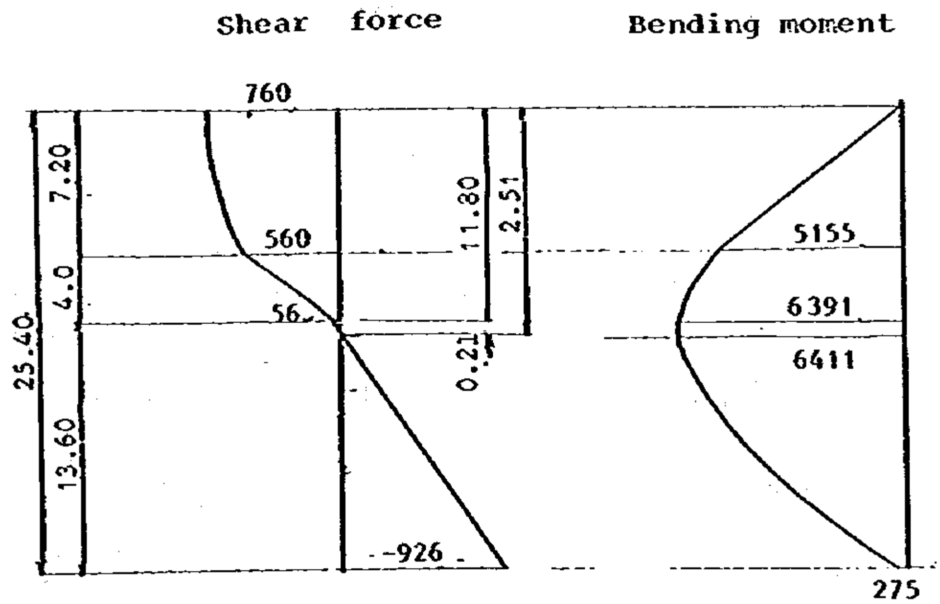


Fig. 2-29 Result of Simulation Analysis (No. 1)

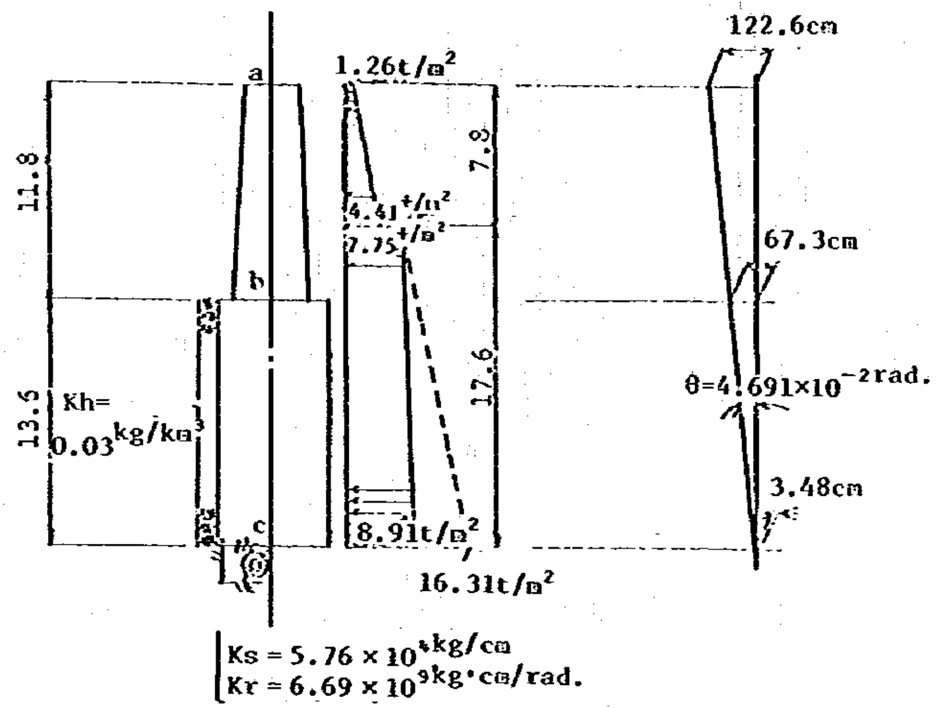


Fig. 2-30 Result of Simulation Analysis (No. 2)

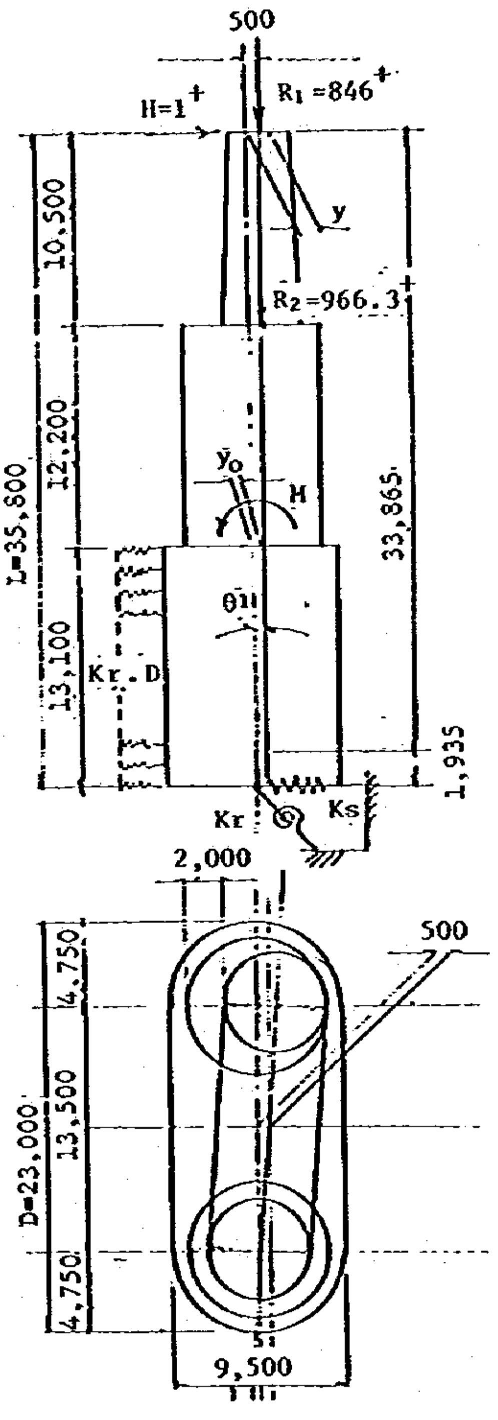


Fig. 2-31 C-pier Excentricity Force by Unusual Pier Position

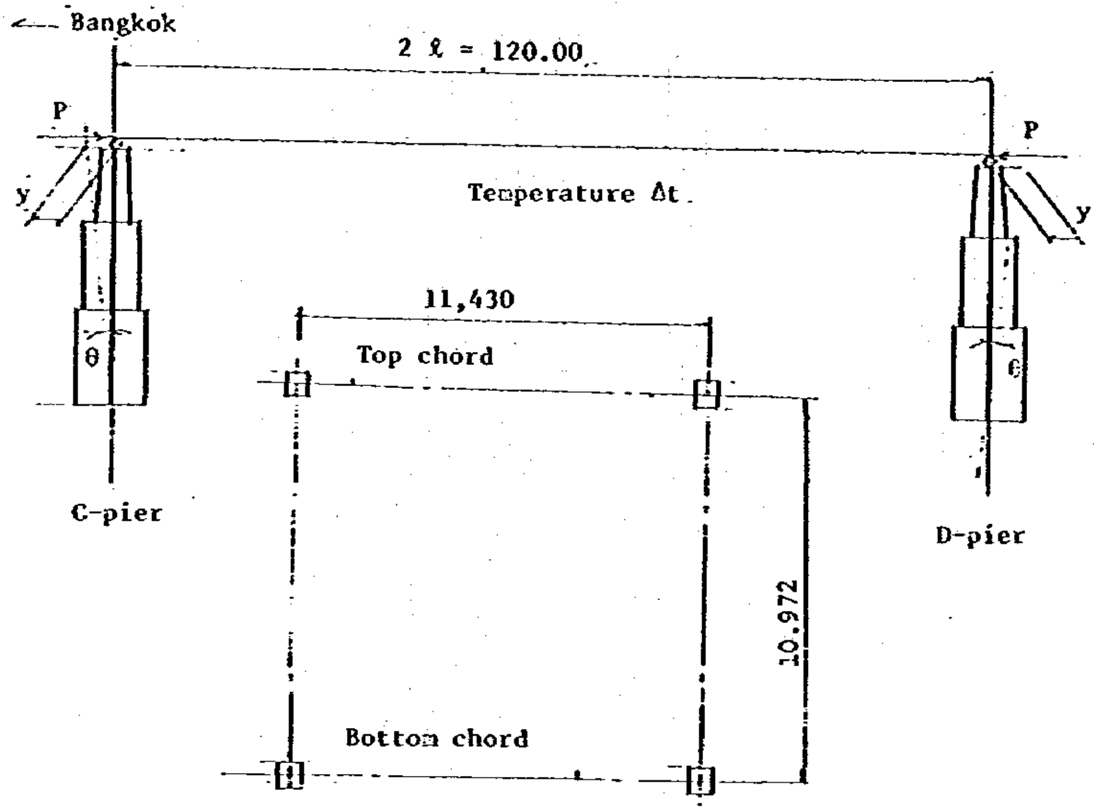
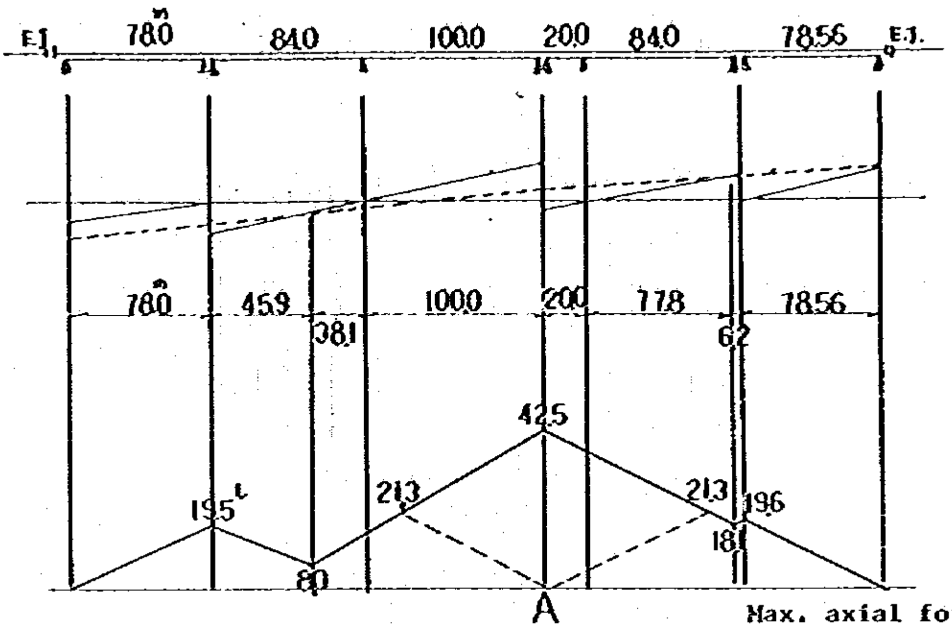


Fig. 2-32 C-Pier Analysis Model by Temperature Variation

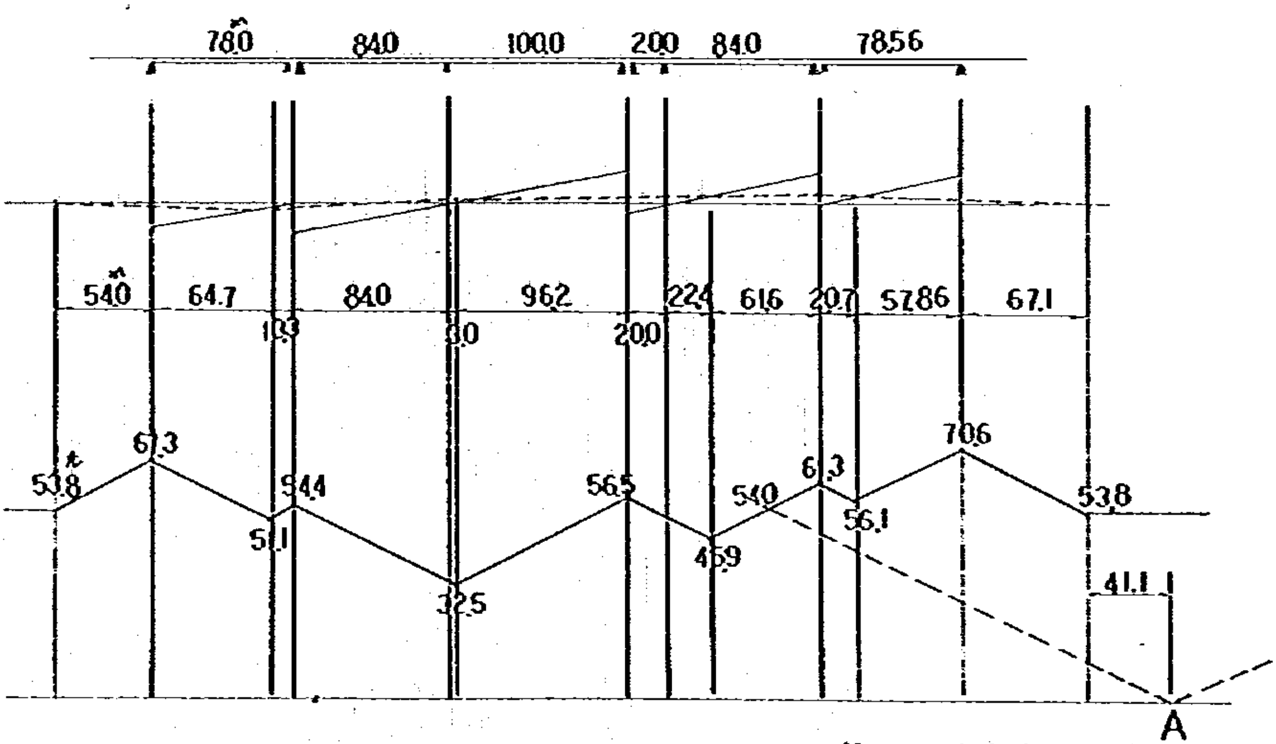
Table 2-5 Displacement of Pier C due to Temperature Variations

Temperature variation (Δ t°C)	10	15	35	60
Horizontal displacement at the top end of Pier C (cm)	0.68	1.01	2.37	4.06



Max. axial force Pt=42.5t
Opening at rail breakdown (Point A) SD=2.68cm

Fig. 2-33 Axial Force Distribution of Long Rail on the Bridge
(with insertion of expansion joint to bridge end)



Max. axial force Pt=70.6t
Opening at rail breakdown SD=9.57cm

Fig. 2-34 Axial Force Distribution of Long Rail on the Bridge
(without insertion of expansion joint to bridge end)

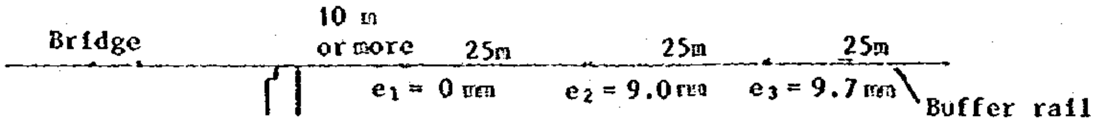


Fig. 2-35 Buffer Rail Joint Gap

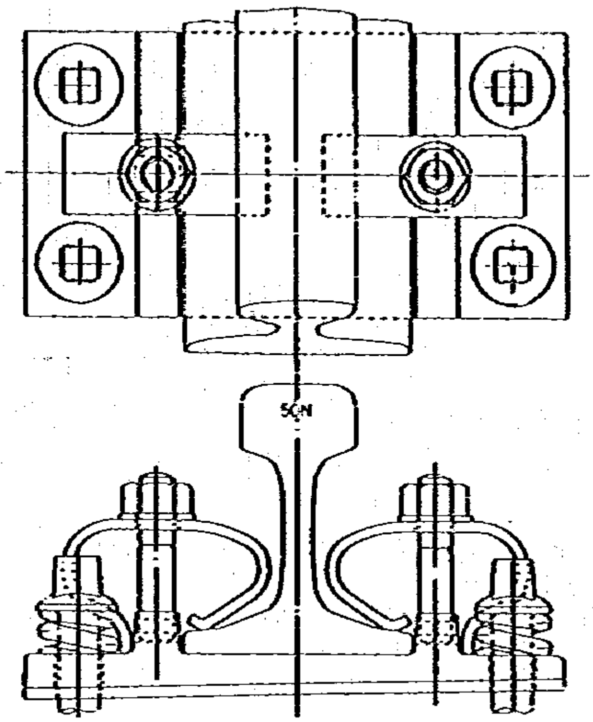


Fig. 2-36 Fastening Device (JNR TYPE 50 III)

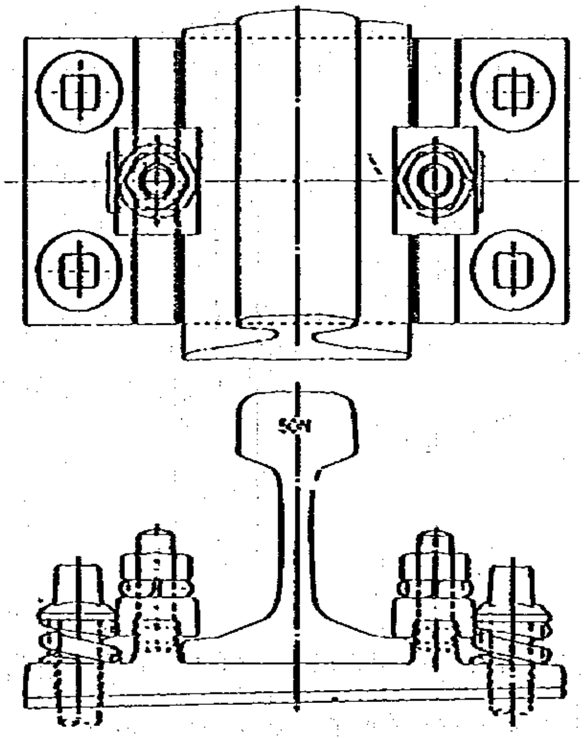


Fig. 2-37 Fastening Device (JNR TYPE 50 IV)