तुरन्त/समय अनुबंधित
विशेष वाहक द्वारा

राष्ट्रीय राजधानी राज्य क्षेत्र शासनः दिल्ली
पर्यटन विभाग
पुराना सचिवालय, कमरा नं0 176–183, दिल्ली–110054.

संख्या फा. 8/509/पर्यटन/वि.स./2019/4653-57 दिनांकः 24/03/2022

सेवा में,

उप सचिव,
प्रश्न शाखा
विधानसभा सचिवालय
पुराना सचिवालय, दिल्ली–110054

विषयः विधानसभा अतारांकित प्रश्न सं0 134 दिनांक 28.03.2022 के लिए

महोदय,

आपके ई–मेल दिनांक 17.03.2022 के संदर्भ में उपरोक्त प्रश्न के उत्तर की 100 प्रतियां अग्रिम आवश्यक कार्यवाही हेतु संलग्न हैं।

यह सक्षम प्राधिकारी, माननीय उप–मुख्यमंत्री/पर्यटन मंत्री के पूर्व अनुमोदन से जारी किया जा रहा है।

भवदीय,
24/3/2022
(खेमचंद बडगुजर)
सहायक निदेशक, पर्यटन

संख्या फा. 8/509/पर्यटन/वि.स./2018/4653-57 दिनांकः 24/03/2022

प्रतिलिपि प्रेषितः
1. निजी सचिव, पर्यटन मंत्री, दिल्ली ।
2. निजी सचिव, मुख्य सचिव, दिल्ली ।
3. निजी सचिव, सचिव, पर्यटन, दिल्ली ।
4. निदेशक, सूचना एवं प्रसारण विभाग, पुराना सचिवालय, दिल्ली–110054
   150 प्रतियों सहित ।

24/3/2022
(खेमचंद बडगुजर)
सहायक निदेशक, पर्यटन

राष्ट्रीय राजधानी क्षेत्र
पर्यटन विभाग, दिल्ली सरकार
सी–ब्लॉक, दूसरा तल, विकास भवन–2, अपर बेला रोड, दिल्ली–110054

अतारांकित प्रश्न संख्या : 134

दिनांक : 28.03.2022

प्रश्नकर्ता का नाम : श्री जितेंद्र महाजन

क्या माननीय उपमुख्यमंत्री / पर्यटन मंत्री यह बताने की कृपा करेंगे कि :

क्रम सं0	प्रश्न	उत्तर
क)	क्या यह सत्य है कि डीटीटीडीसी द्वारा रोड नं. 68 पर नत्थू कालोनी चौक पर 126 करोड़ की लागत से बनवाए गए फ्लाई ओवर के पीडब्ल्यूडी को सौंपने के 1 वर्ष के भीतर टूटने/स्लैब गिरने की जांच हेतु एक कंसलटेंट नियुक्त किया गया था;	उक्त कार्य **DTTDC** द्वारा 2011 में पूरा किया गया था तथा बाद में इसे **PWD** को स्थानांतरित कर दिया गया था। कुछ समय बाद **PWD** द्वारा कुछ **Panels** में **Cracks** नोटिस किये गए थे।
ख)	यदि हां, तो जांच रिपोर्ट की प्रतिलिपि सहित उस कंसलटेंट का नाम, पता एवं उसको किए गए भुगतान की जानकारी उपलब्ध कराएं;	**M/s. CCPL Construma Consultancy Pvt. Ltd.** मुम्बई को **PWD** द्वारा सलाहकार नियुक्त किया था, अतः उनको **PWD** द्वारा 7,78,000/–रुपए का भुगतान किया गया है (प्रतिलिपि संलग्न है)
ग)	डीटीटीडीसी के अनुसार गैम्मन इंजीनियर्स एंड कांट्रेक्टर्स प्राइवेट लि. को फ्लाइओवर बनाने की पूरी पेमेंट करने में कोर्ट के आदेश को सरकार द्वारा माननीय कोर्ट में चुनौती न दिए जाने के क्या कारण थे;	फ्लाईओवर के अंतिम भुगतान को रोकने हेतु माननीय हाई कोर्ट में चुनौती दी गयी है।
घ)	उक्त फ्लाइओवर निर्माण के लिए जिम्मेदार डीटीटीडीसी अधिकारियों के खिलाफ क्या कार्रवाई की गयी, पूर्ण जानकारी दें;	भारत सरकार के उपक्रम **CRRI** से **NDT Test, Compressive Strength, Chemical Testing Core of Concrete etc.** की रिपोर्ट आना बाकी है।
ड)	क्या यह सत्य है कि चीफ इंजीनियर के 15 जनवरी 2020 के पत्रांक के अनुपालन में डीटीटीडीसी द्वारा किसी प्रतिष्ठित कंसलटेंट द्वारा पुनः सेकिंड ओपिनियन लेने की बात की गयी थी;	जी हां।
च)	यदि हां, तो उक्त दूसरे कंसलटेंट का नाम, पता एवं उसको किए गए भुगतान की जानकारी एवं उसके द्वारा प्रस्तुत रिपोर्ट की प्रति उपलब्ध कराएं; और	**M/s. B &S Engineering Consultants Pvt. Ltd. Nodia**, जांच प्रक्रिया जारी है अभी तक 9.44 लाख रुपए का भुगतान किया है।
छ)	फ्लाइओवर रिपेयर करने की तिथि, कार्य समाप्ति की समय–सीमा तथा लागत की पूर्ण जानकारी दें?	जांच प्रक्रिया पूरी होने के बाद कोई आगे की उचित कार्यवाही की जाएगी।

स्वाति शर्मा

सचिव (पर्यटन)

SWATI SHARMA
I.A.S.
Secretary Tourism
Department of Tourism
Govt. of NCT of Delhi
C-Wing, 2nd Floor,
Vikas Bhawan-II, Delhi-110054

दिल्ली सरकार Government of NCT of Delhi
लोक निर्माण विभाग Public Works Department
ऊपरगामी सेतु परियोजना मंडल एफ–51
Flyover Project Division F-51
रिंग रोड फ्लाईओवर के नीचे, विपरीत आई0 जी0 स्टेडियम गेट न0 9, नई दिल्ली–110002
Under Ring Road Bypass Flyover, Opp. of IG Stadium Gate No.9,
New Delhi-110002
Tel. No. 011-21400870, Mobile no. : +919560804745
E-mail : eepwddelhif51@gmail.com

सं0 54(10) / फ्लाईओवर परियो0 मं0 एफ–51 / दि0 स0 / 2022 / 142 दि0 21/3/22

सेवा में,

The Chief Porject Manager
DTTDC Ltd.
New Delhi

विषयः– दिल्ली विधानसभा के प्रश्न सं0 134 के संबंध में।

सन्दर्भः– Your E-mail dated 21-03-2022.

उपरोक्त प्रश्न के संदर्भ में रोड नं0 68 (नत्थू कॉलोनी चौक के फ्लाईओवर) के कार्य के लिए M/s CCPL, (Construma Consultancy Pvt. Ltd. Mumbai) को प्रथम चलन बिल राशि रू 7,78,800/- इस कार्यालय द्वारा भुगतान किया जा चूका है।

यह आपको सूचनार्थ एवं आवश्यक कार्यवाई हेतू प्रेषित है।

राजेश

कार्यपालक अभियंता
फ्लाईओवर परियो0मंड़ल एफ–51

REPAIR & REHABILITATION OF ROB NANDNAGRI FLYOVER (at Road No. 68), DELHI

CONDITION STATUS REPORT AND REMEDIAL MEASURES ON NANDNAGRI FLYOVER (AT ROAD No.68) - DELHI

FEBRUARY, 2019

Submitted to:

Public Works Department
Flyover Project Division F-23,
Ramesh Park, Near Shakarpur Police Station,
Delhi - 110 092.

CONSTRUMA CONSULTANCY PVT. LTD.

CONSTRUMA CONSULTANCY Pvt. Ltd.

Regd. Off. : 2nd Floor, Pinky Plaza, 5th Road, Khar (W), Mumbai-400 052.
Tel. : 022-26487415, 26050208. Fax: 26463181. E-mail: bridges@construma.in

Branch Office : D-59, Ground Floor, Sector - 52, Noida - 201 301.
Tel. : 0120-4570152, 4570153. Fax : 4570154. e-mail: ccpl_delhi@rediffmail.com

TABLE OF CONTENT

S.No.	Description of NDT Reports	Page No.
1.	**INTRODUCTION**	1
1.1	Structure Summary	2
1.2	Structural Arrangement	2
1.3	Evaluation of the Structure	3
1.4	Objectives of the overall investigation	4
1.5	Planning of Investigation & Methodology	4
2.	**VISUAL INSPECTION & HAMMER RAP SURVEY**	5
2.1	Visual Survey	6
2.2	Hammer Rap Survey	33
3.	**TEST PLAN**	39
3.1	Test Plan	40
3.2	Samples and Site Data Collection	41
3.3	Number of Test to be Conducted	41
4.	**NDT TEST RESULTS & INTERPRETATION**	42
4.1	Ultrasonic Pulse Velocity Test	43
4.2	Rebound Hammer Test	53
4.3	Cover Depth Test	62
4.4	Chemical Test	63
4.5	Carbonation Test	64
4.6	Concrete Core Extraction	65
4.7	Crack Pattern Analysis	70
4.8	Thickness Measurement	72
5.	**CONCLUSION**	75
6.	**REMEDIAL MEASURES**	83
7.	**ANNEXURE**	86
A.	Marked Test Locations	87
B.	Testing Photographs	102
C.	Journal Reference	108
D.	Structural Drawings	118

1. INTRODUCTION

1.1 STRUCTURE SUMMARY:-

The Existing structure is unique combination of Railway Over bridge (ROB) & Road Under Bridge (RUB) at Nand Nagri ,North East Delhi having 21 pier, 21 Span & 2 Abutment with 623 panels having different length with 6 steel girders in each span.

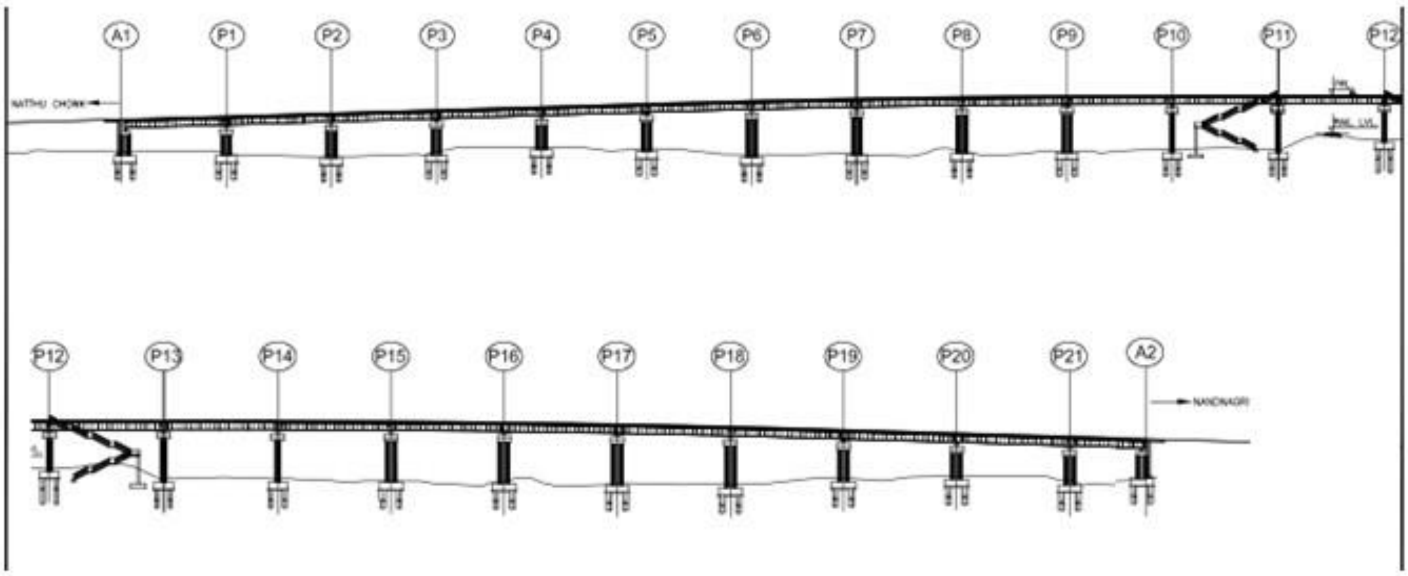


Fig.1. Typical elevation sketch of NandNagri Flyover

The **Public Works Department, Delhi** has interested the work of Repair and Rehabilitation of NandNagri Flyover, North East Delhi to **Construma Consultancy Pvt. Ltd**. As a part of it, the condition assessment of **Nand Nagri Flyover, North East Delhi** by using Non-destructive testing, Hammer Rap Survey & Visual Inspection was carried out on site. This report pertains to the stated safety appraisal, health assessments. The following are our previous report in accordance with the above mentioned work:

1. Inception report on NandNagri Flyover – Delhi, October 2018.
2. Methodology for Recasting of 4 panels on NandNagri Flyover – Delhi, October 2018.
3. BOQ, Measurement Sheet and Rate Analysis for recasting of 4 panels on NandNagri Flyover – Delhi, November 2018.

1.2 STRUCTURAL ARRANGEMENT

Details from Drawing No. DD001 (GENERAL ARRANGEMENT DRAWING-PLAN & PROFILE (ROB, RUB & SERVICE ROADS) (Refer Annexure D)

- This bridge is a unique combination of Road over Bridge (ROB) and Road under Bridge (RUB) having 21 Piers and 2 Abutments of 595 panels of different length and width with 6 steel girders in each span.

- The entire span of the elevated portion is 651m. The span over the railway portion is a 4 span module having 120m in length.
- The bridge span is divided into 1 no. two span continuous unit, 4 nos. three span continuous units and 2 nos. four span continuous units with expansion joints at ends.
- The approach towards Road no.66 has RE wall of length 138m and RC wall of length 65m respectively.
- The approach towards Dilshad Garden has RE wall of length 121m and RC wall of length 30m respectively.

Details from Drawing No. DD110B (GENERAL ARRANGEMENT DRAWING OF CAST IN SITU AND PRECAST SLAB FOR 42.25-30.0M SPAN) (Refer Annexure D)

- There are five full panels and two half panels in the transverse direction.
- The panel width is 2850 mm and the half panel width is 1425mm in transverse direction and the length of the panel varies with respect to the overall span.
- The width of carriage way is 16.2m with a median of 1.2m dividing the road into two equal spans of 7.5m.
- Single precast deck slab has 4 lifting hooks at distance of 1000mm from the transverse joint.
- The precast panels are connected to one another by the in-situ stitch concrete.
- The steel girder and the precast slab panels are connected using the shear studs over which the in-situ stitch concrete is poured.
- The half panels are placed on the end steel girders where the metallic crash barriers are placed over the panels.
- The precast panels are placed on bearing strips made of high density polystyrene of 40mm thick (varying).
- The steel hollow box diaphragms are placed on top of the bearing and splicing's done at predominant location using HSFG bolts.

1.3 EVALUATION OF THE STRUCTURE

In order to assess the condition of the RCC superstructure, a thorough evaluation was performed. The evaluation determined the structural condition, the need for repair and rehabilitation or maintenance and provided an indication as to the safety and strength of the structure.

Primary Evaluation:

- A visual inspection of the exterior exposed elements to determine, whether there are any visible/ obvious signs of distress, deflection or deterioration in the structure.
- Hammer Rap Survey to determine the distress zones in RCC Panels and marked with paint for NDT testing.

Secondary Evaluation:

- At selective locations extracting concrete to examine the condition of the underlying reinforcing steel.
- In-Situ and laboratory testing to determine concrete homogeneity, compressive strength & condition of steel and RCC structure etc.

1.4 OBJECTIVES OF THE OVERALL INVESTIGATION:

The overall objective of the investigation carried out for the structure is to obtain an up to date account of the health condition of the structure so that appropriate repair measures can be taken up to make up for the damages sustained. Keeping this in view the basic objectives of the investigation formulated are as given below.

1. To assess the existing condition of the structural elements.

2. To determine the extent of damages in the structure, so as to undertake suitable remedial measures for rehabilitation of the structure.

1.5 PLANNING OF INVESTIGATION AND METHODOLOGY:

Walk over survey:

First and foremost activity in a condition survey and structural investigation, especially in distressed superstructure, is a Walk over survey & Hammer Rap survey so as to gather readily available information about the structure in question. Further, careful visual observation of the nature of the crack & spalling can furnish valuable information regarding the distresses. A systematic visual observation has been recorded in this investigation and the findings are presented in later part of this report.

1. **Visual survey** of the structure/RCC/MS structural members and documenting the damage if any with the help of photographs.

2. **Hammer Rap Survey** and marking of all distress zones with paint.

2. VISUAL INSPECTION
&
HAMMER RAP SURVEY

(PRIMARY EVALUATION)

2.1 VISUAL SURVEY:-

Photo 1		
	**Location**	P11-P12 PANEL-13
	Nature of Distress	Spalling
	Extent & Severity	Severe

Photo 2		
	**Location**	P5-P6 PANEL-3
	Nature of Distress	Minor & Major Cracks
	Extent & Severity	Severe

Photo 3		
	**Location**	P5-P6 PANEL-4
	Nature of Distress	Minor & Major Cracks
	Extent & Severity	Severe
Photo 4		
	**Location**	P5-P6 PANEL-3
	Nature of Distress	Minor & Major Cracks
	Extent & Severity	Severe

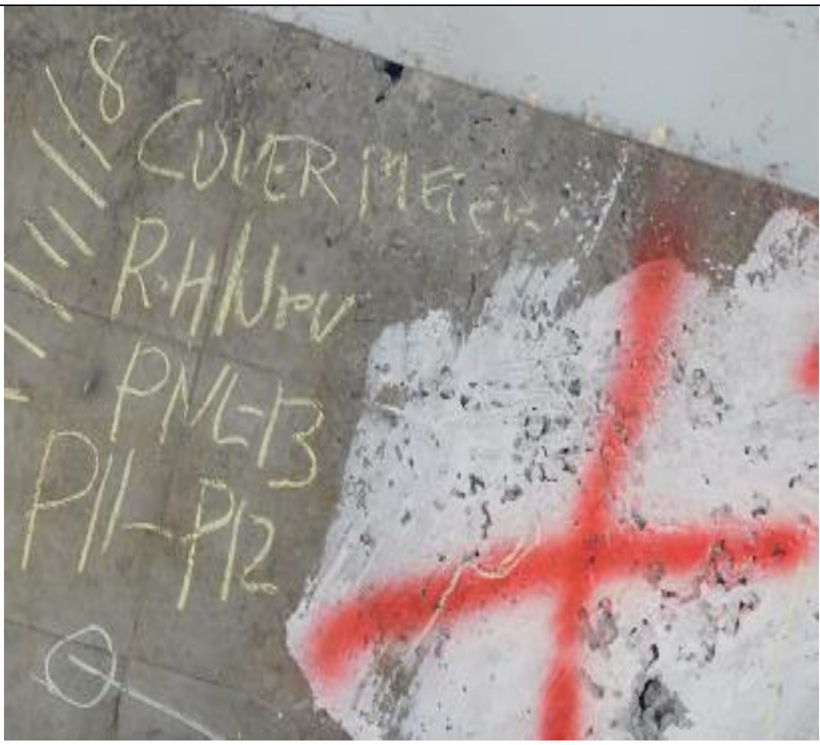
Photo 5	**Location**	P11-P12 PANEL-13
	**Nature of Distress**	Spalling & Major Cracks
	Extent & Severity	Severe
Photo 6	**Location**	P16-P17 PANEL-25
	**Nature of Distress**	Minor Cracks & Spalling
	Extent & Severity	Moderate

Photo 7		
	**Location**	P9-P10 PANEL-19
	Nature of Distress	Minor Cracks, Honeycombing spalling
	Extent & Severity	Moderate
Photo 8		
	**Location**	P17-P18 PANEL-6
	Nature of Distress	Minor & Major Cracks
	Extent & Severity	Severe

<table>
<tr><td>Photo 9</td><td rowspan="2">Location

Nature of Distress

Extent & Severity</td><td rowspan="2">P9-P10
PANEL-19

Minor Cracks & Honeycombing

Moderate</td></tr>
<tr><td>
</td></tr>
<tr><td>Photo 10</td><td rowspan="2">Location

Nature of Distress

Extent & Severity</td><td rowspan="2">P6-P7
PANEL-13

Major Cracks & Honeycombing

Severe</td></tr>
<tr><td></td></tr>
</table>

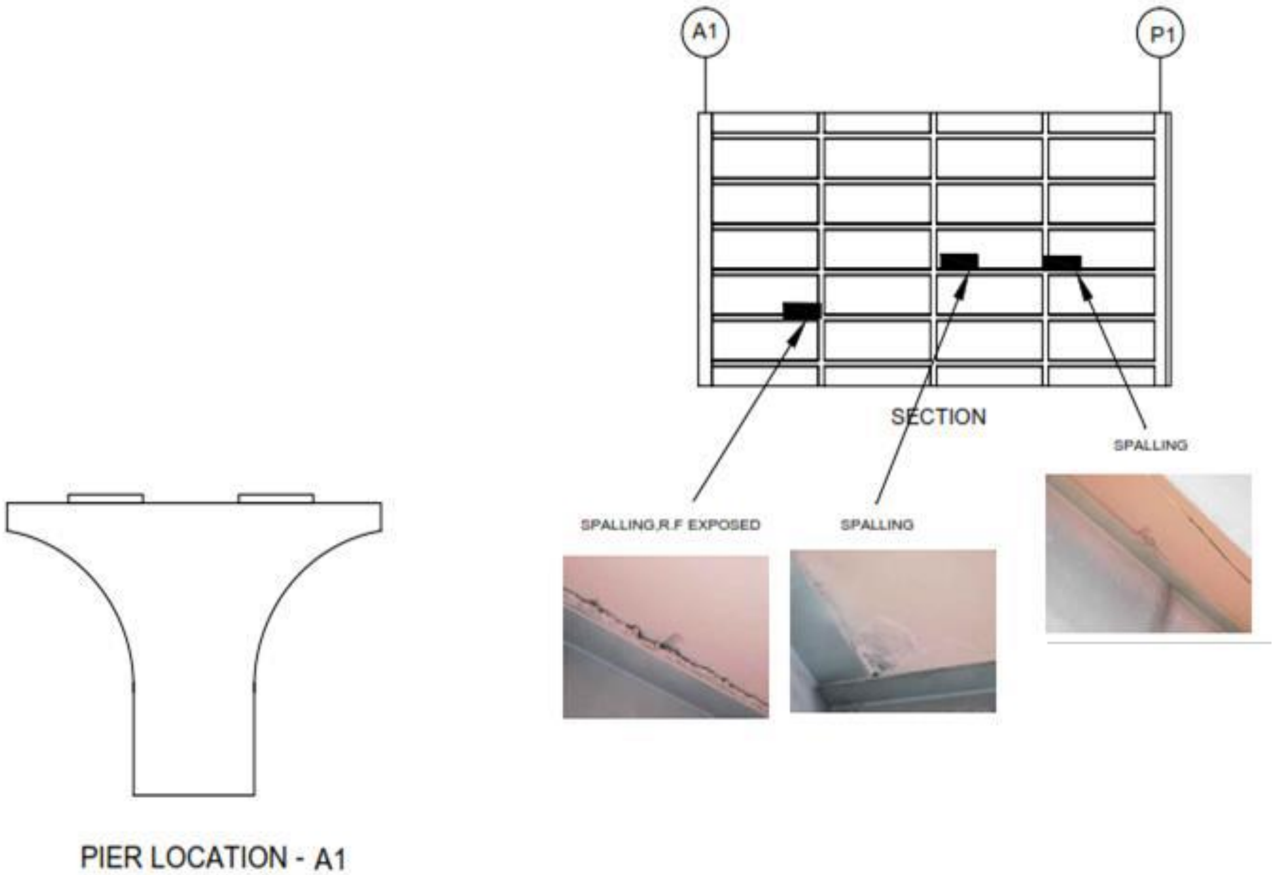


Fig. 2.Spalled concrete on the soffit of the panel maybe between A1-P1

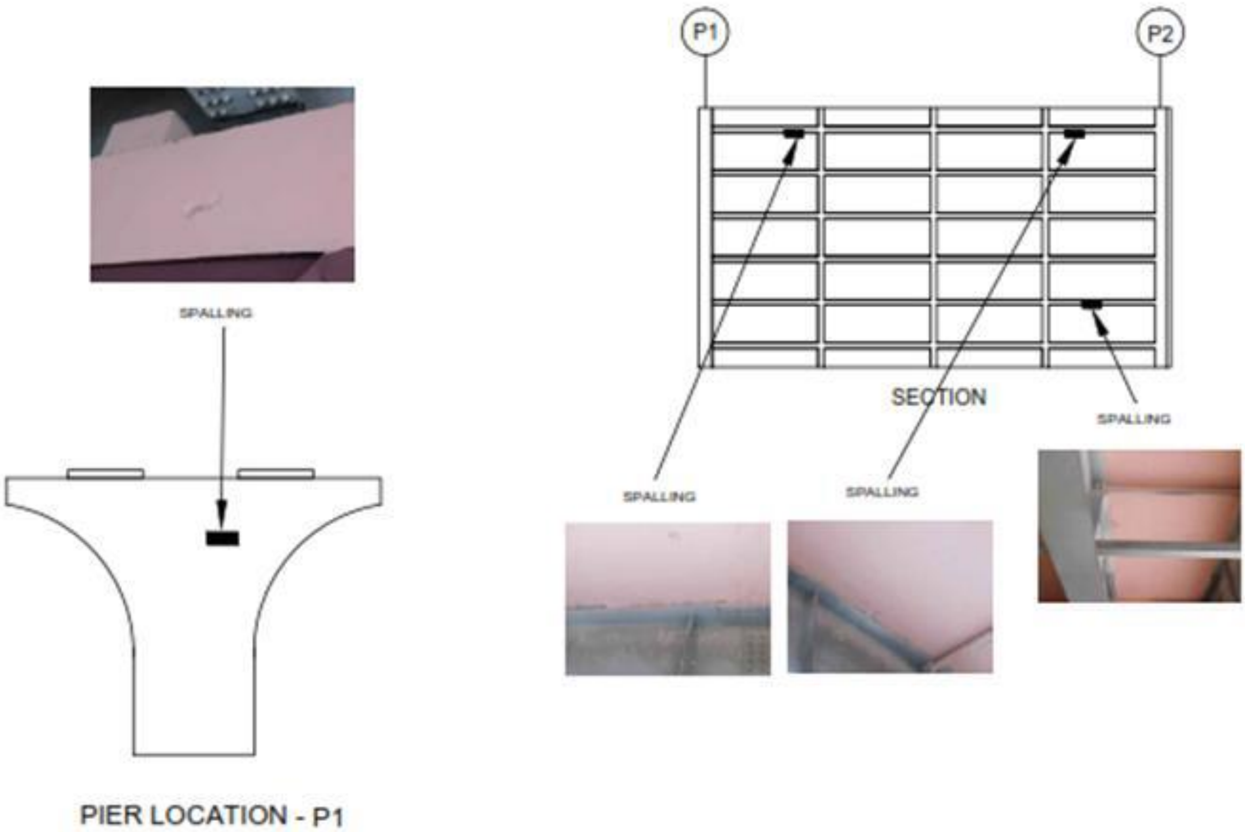


Fig. 3.Spalling of concrete near the junction of girder and panels between P1-P2

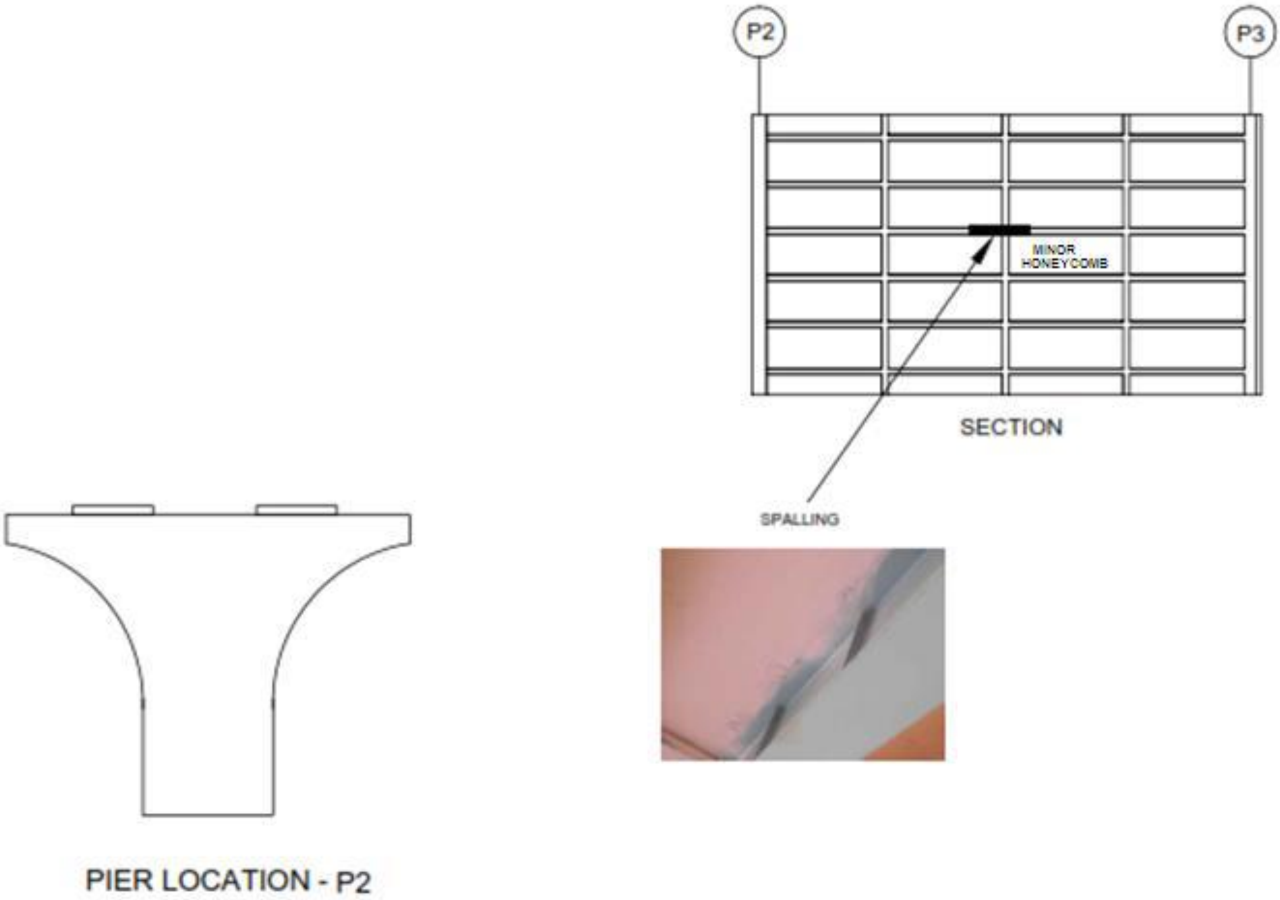


Fig. 4.Minor honeycomb formed on the soffit of the panels between P2-P3

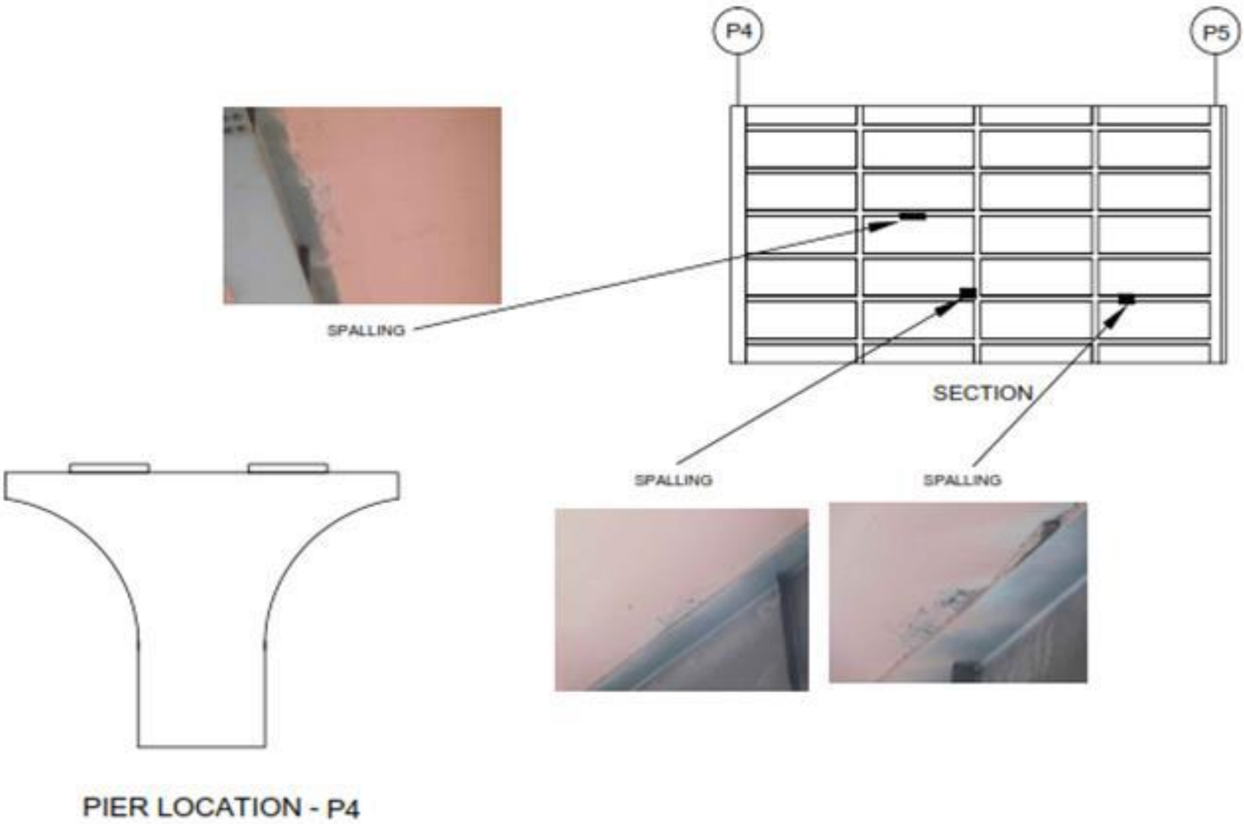


Fig. 5. Spalling of concrete near the junction of girder and panels between P4-P5

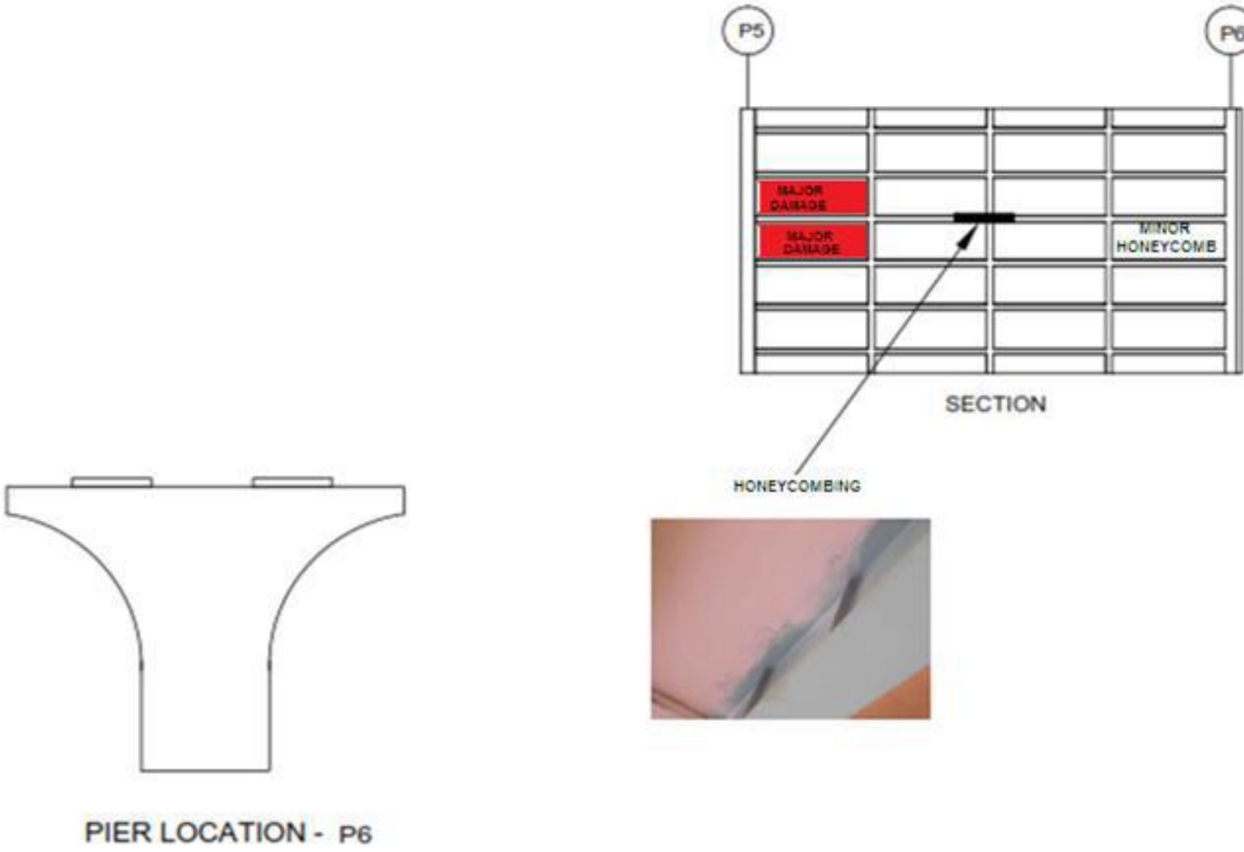


Fig. 6.Major damaged panels

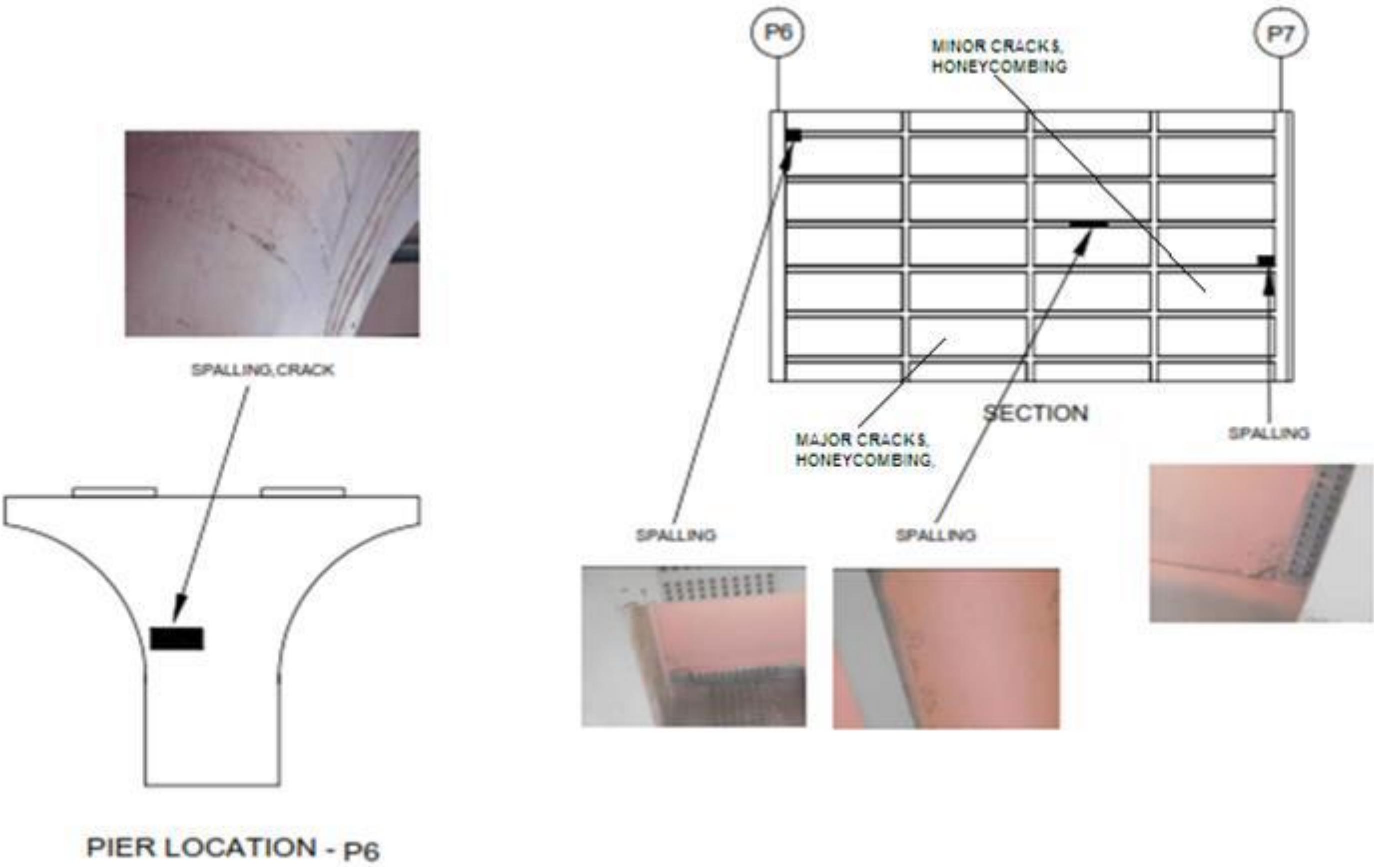


Fig. 7. ***Spalling of concrete near the junction of girder and panels between P6-P7***

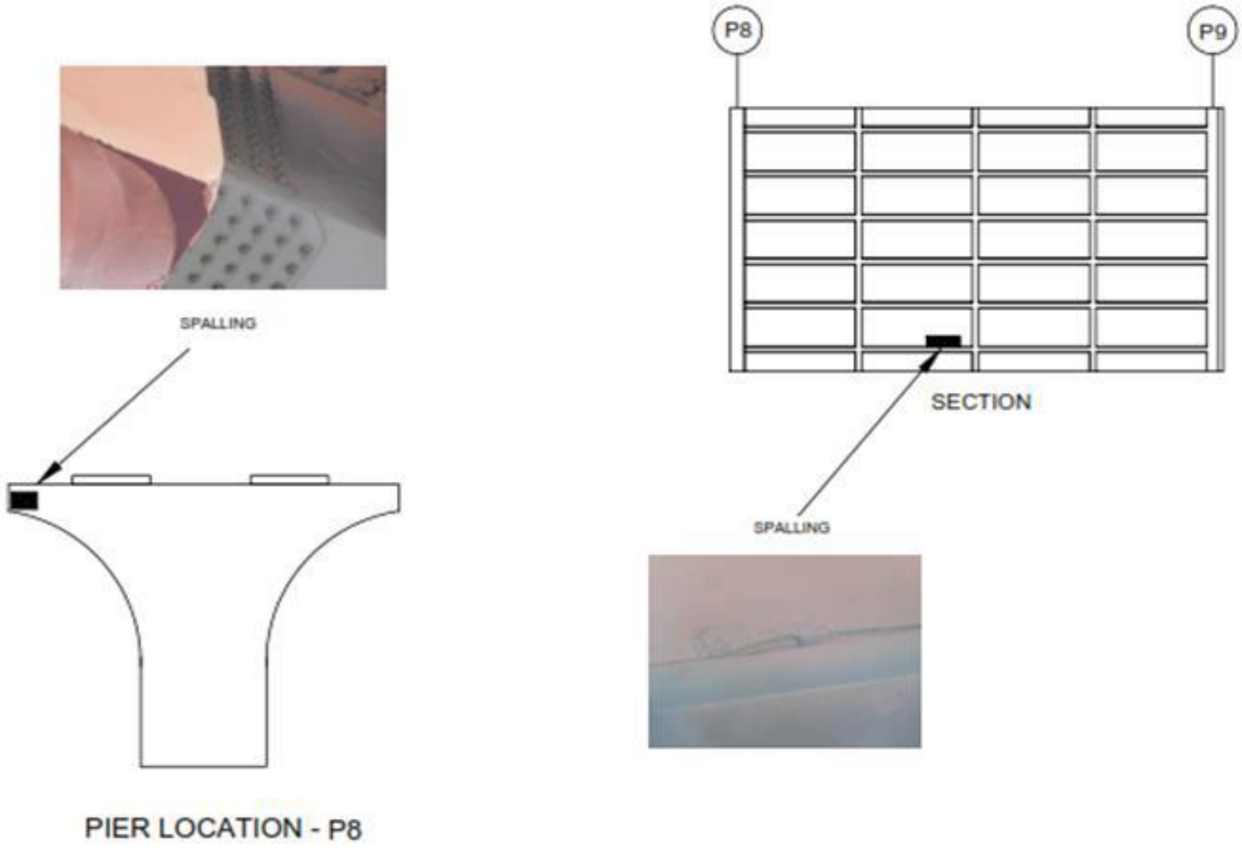


Fig. 8.Spalling piercap and junction of panel with steel girder in P8-P9

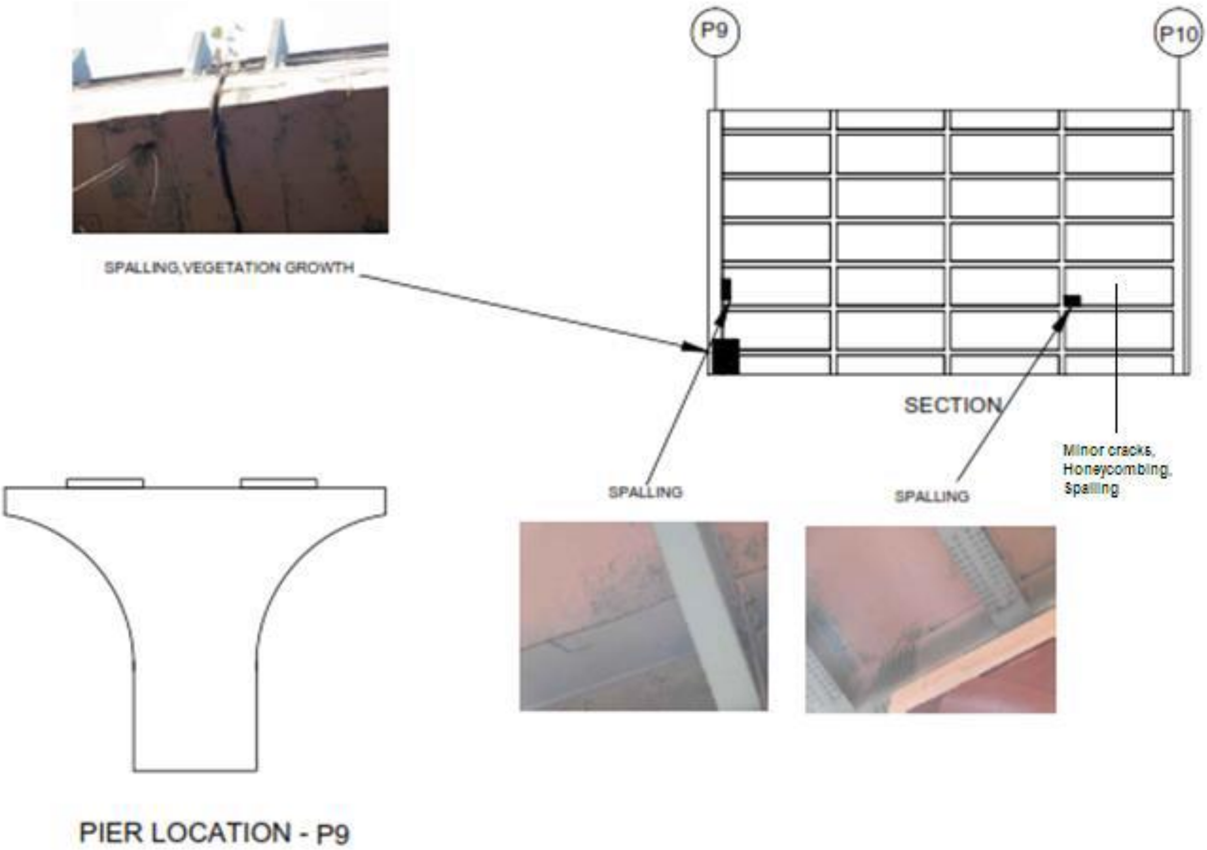


Fig. 9.Spalling and Vegetation growth seen in between P9-P10

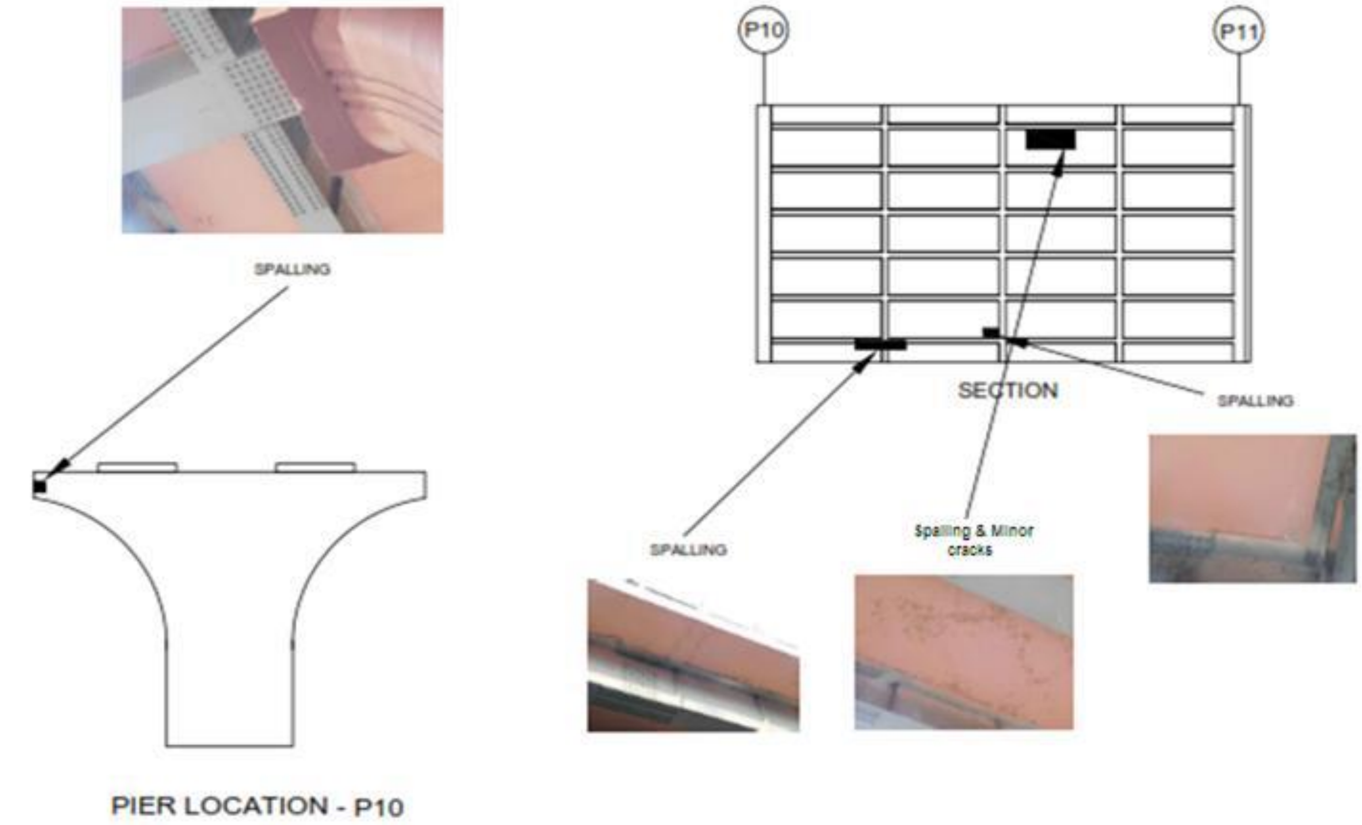


Fig. 10.minor cracks seen in between P10-P11

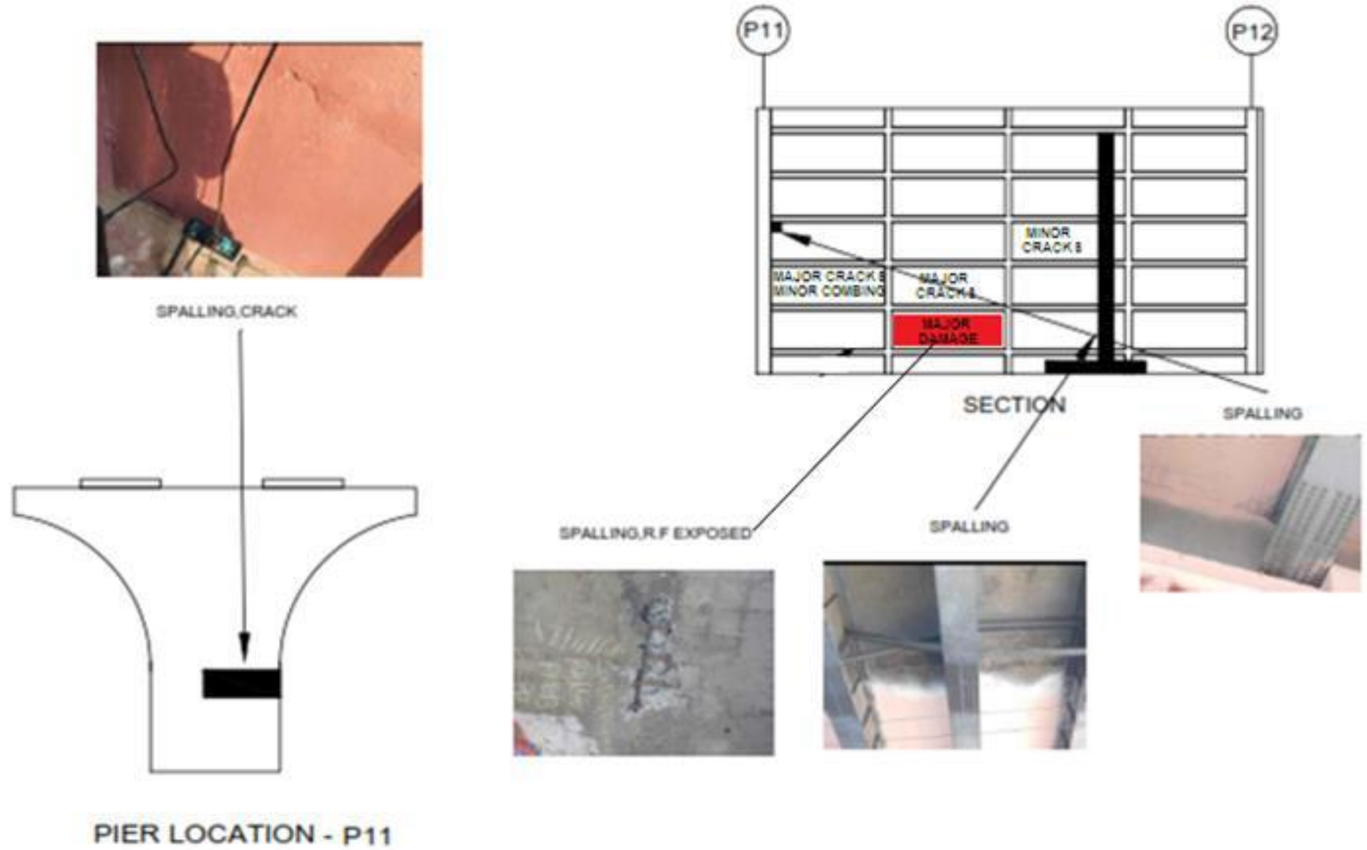


Fig. 11.Major damaged panel in between P11-P12

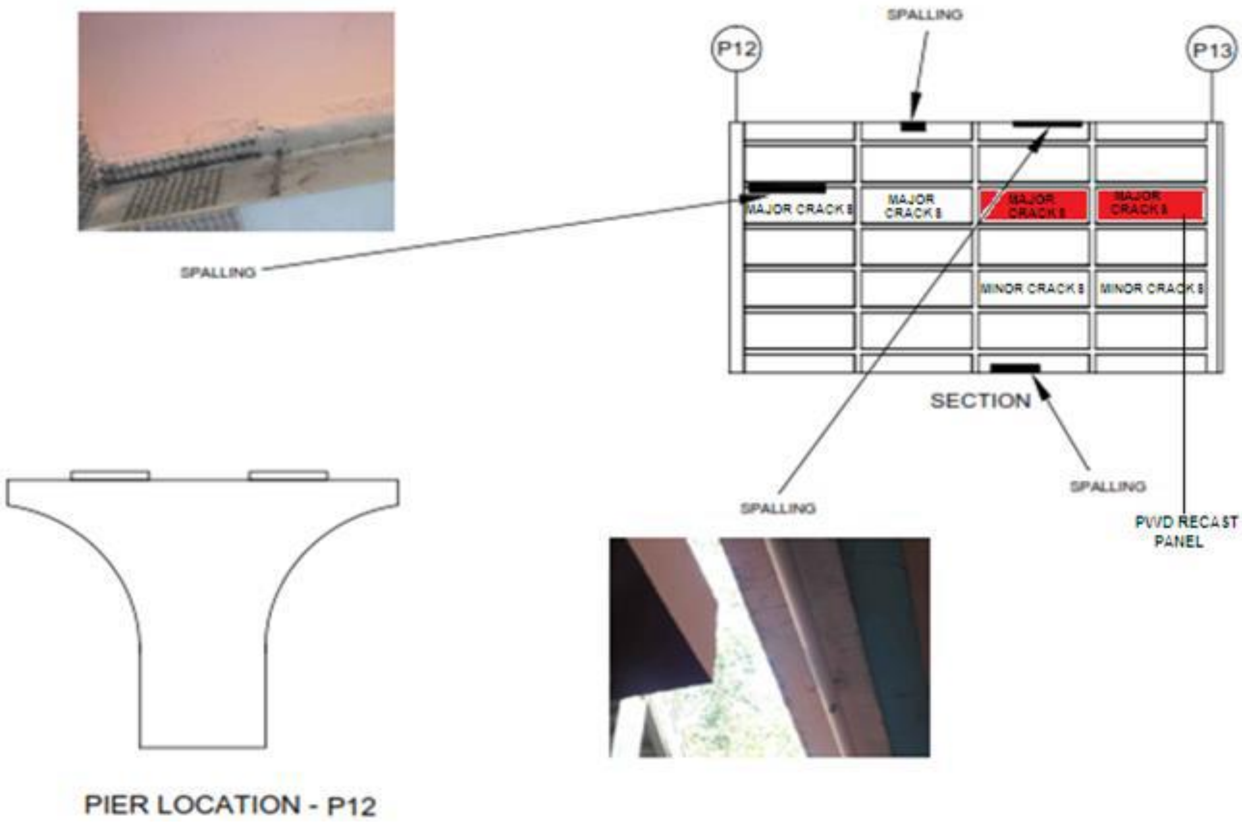


Fig. 12. Major damaged panel with minor cracks in between P12-P13

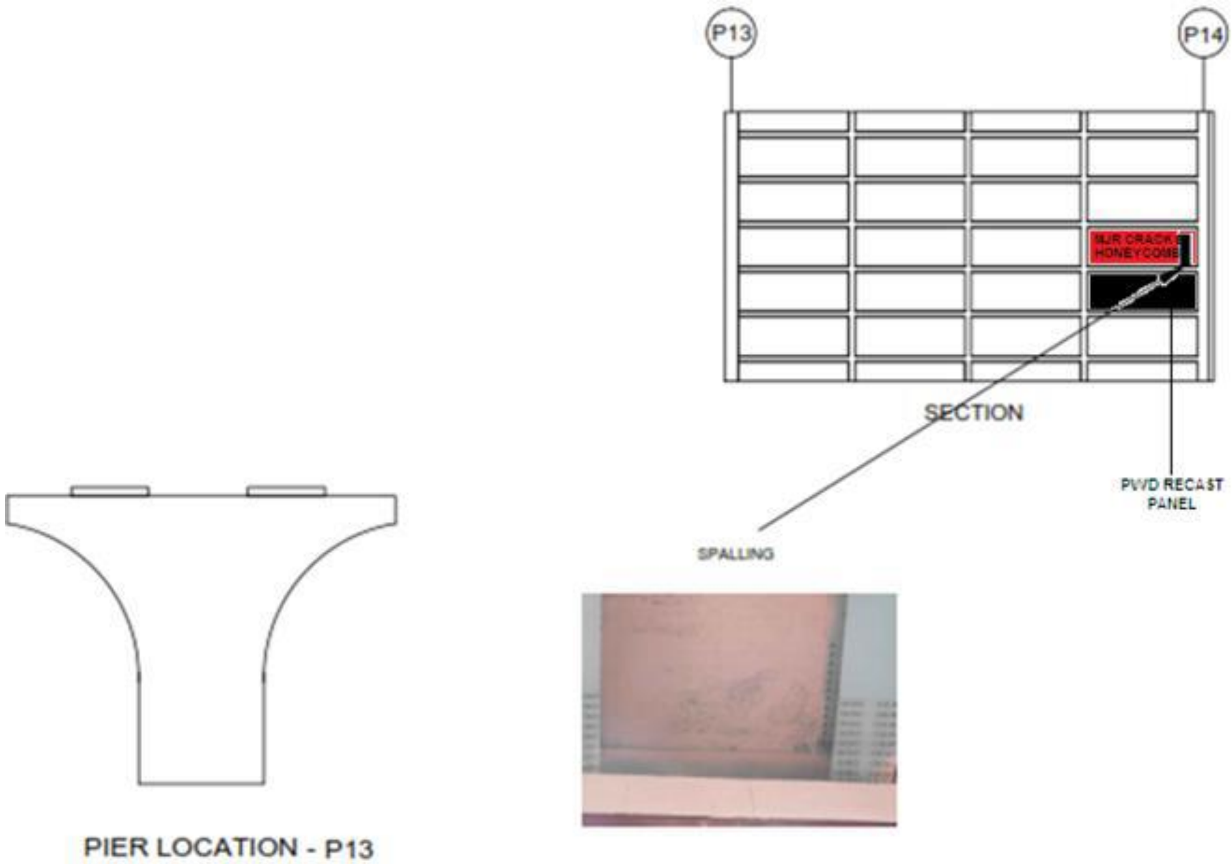


Fig. 13.Major distress panel in between P13-P14

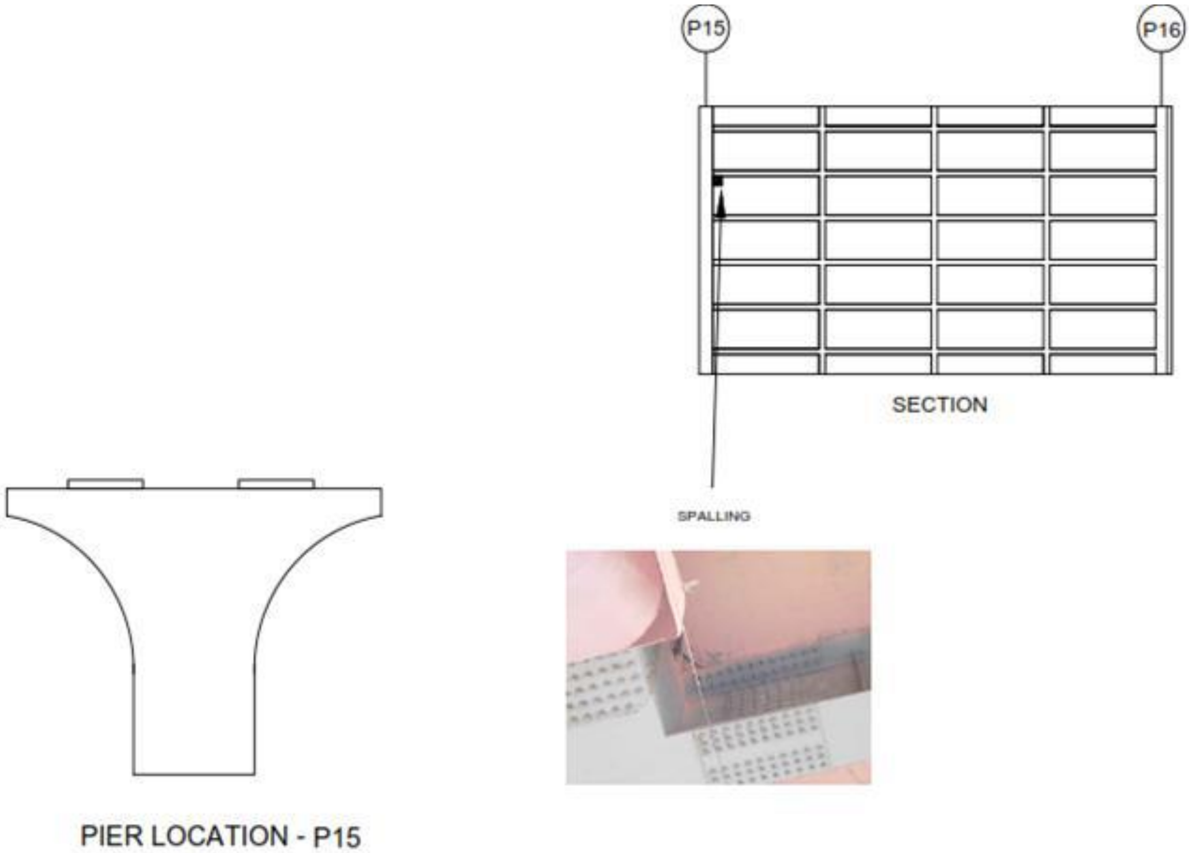


Fig. 14. No major damage in between P15-P16

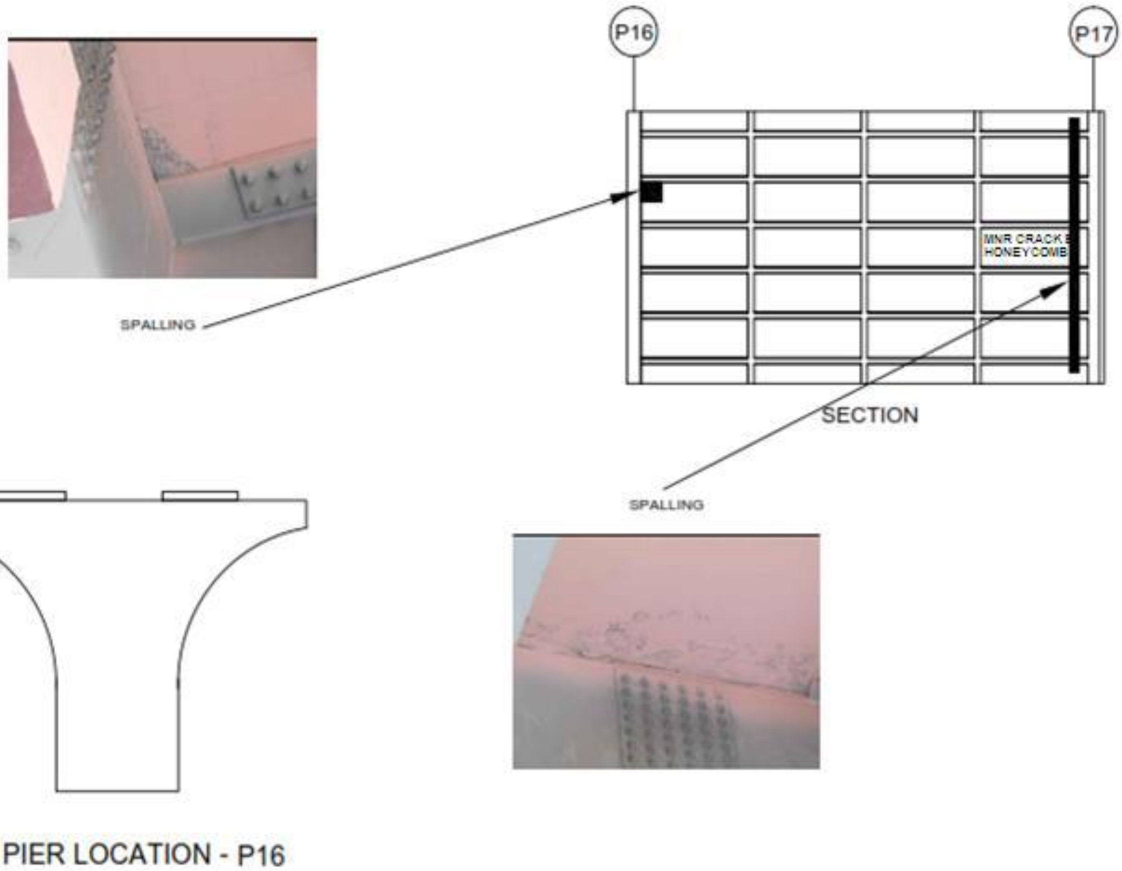


Fig. 15.Minor cracks with spalling noted in between P16-P17

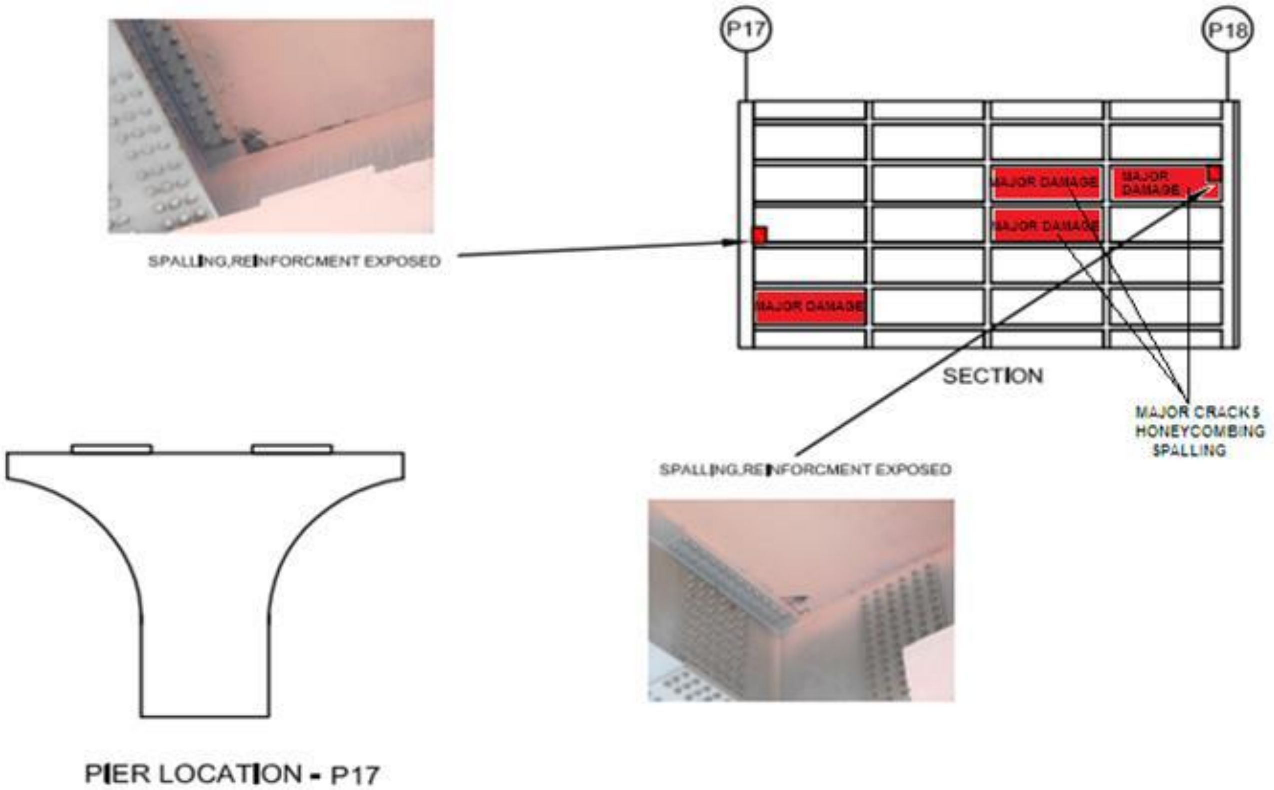


Fig. 16.Major distress seen in 4 panels in between P17-P18

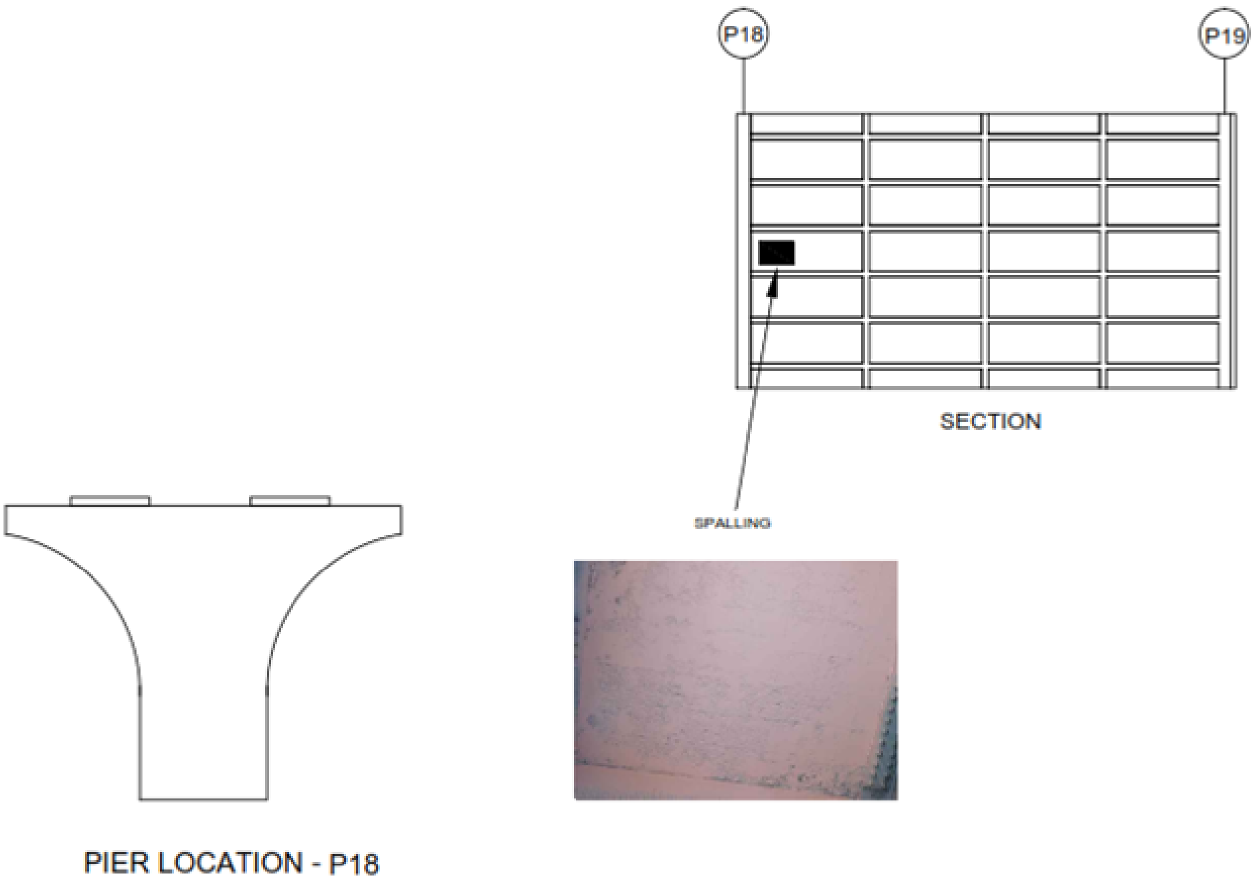


Fig. 17.Spalling noticed in one panel in between P18-P19

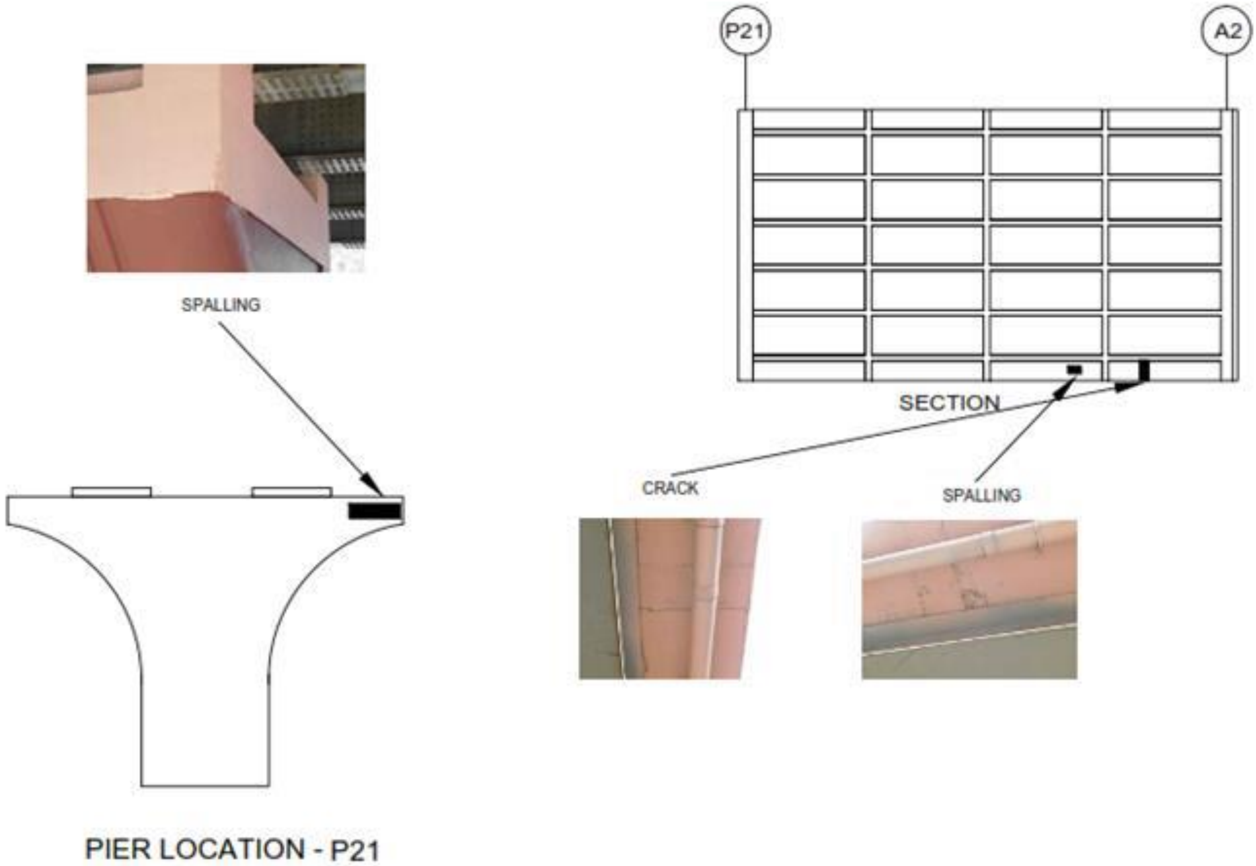


Fig.18. Minor cracks noticed in between P21-A2

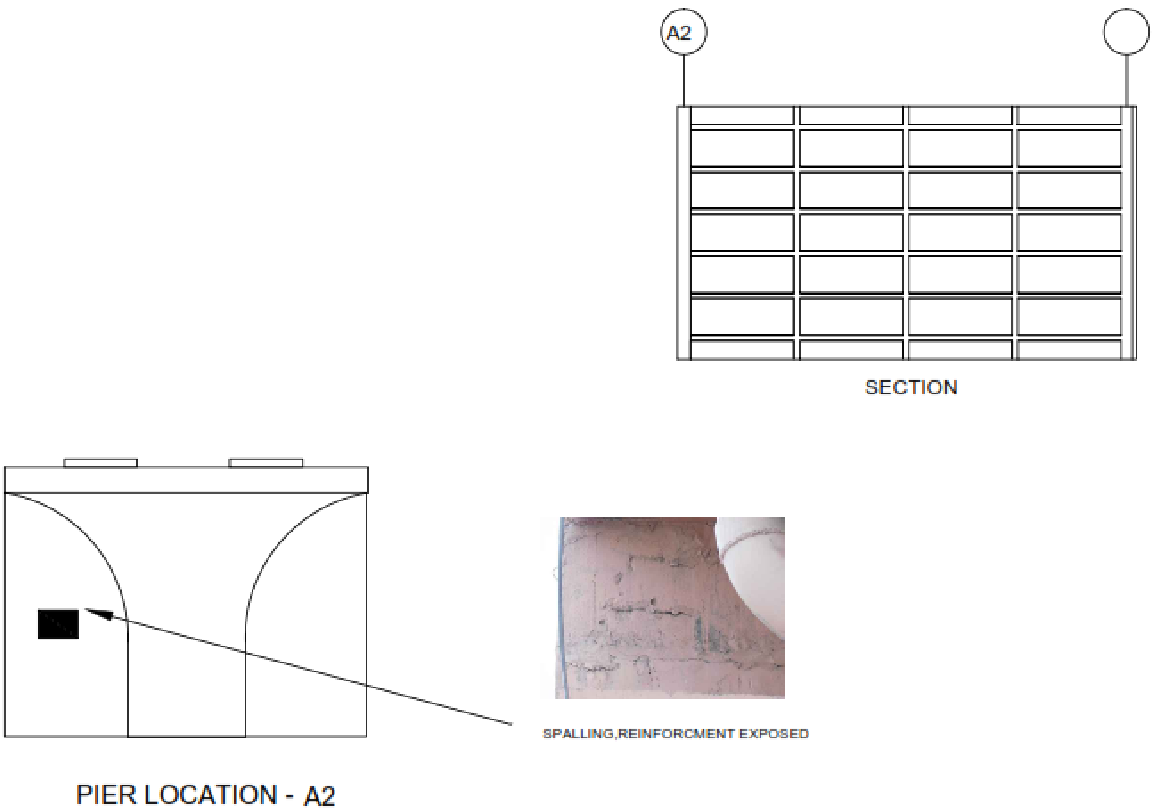


Fig. 19.Spalled concrete on the abutment A2

DAMAGES IN BEARING STRIPS

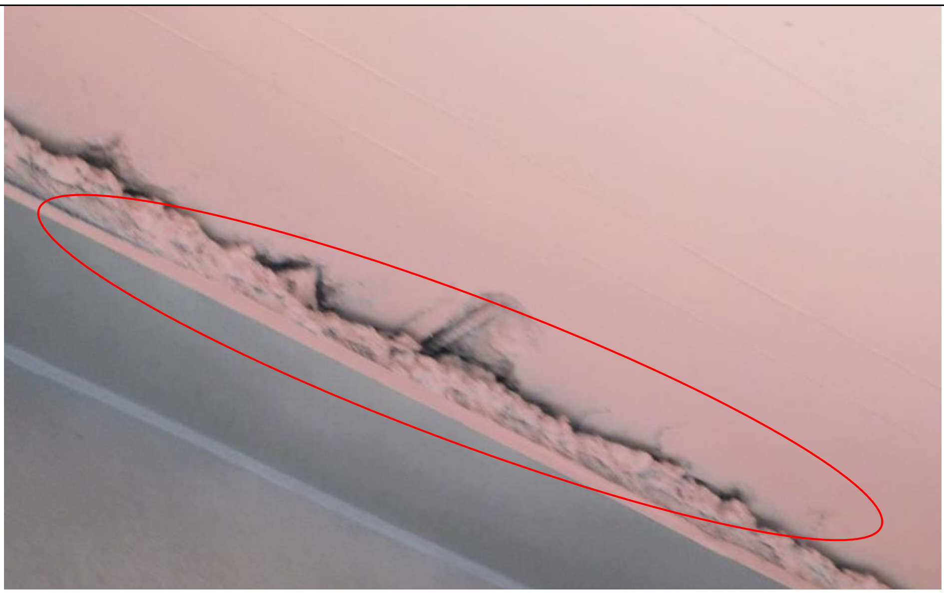
 *Photo 11*	**P8-P9** **Panel 28**
 *Photo 12*	**A1 P1**
 *Photo 13*	**A1 P1** **Panel 5**

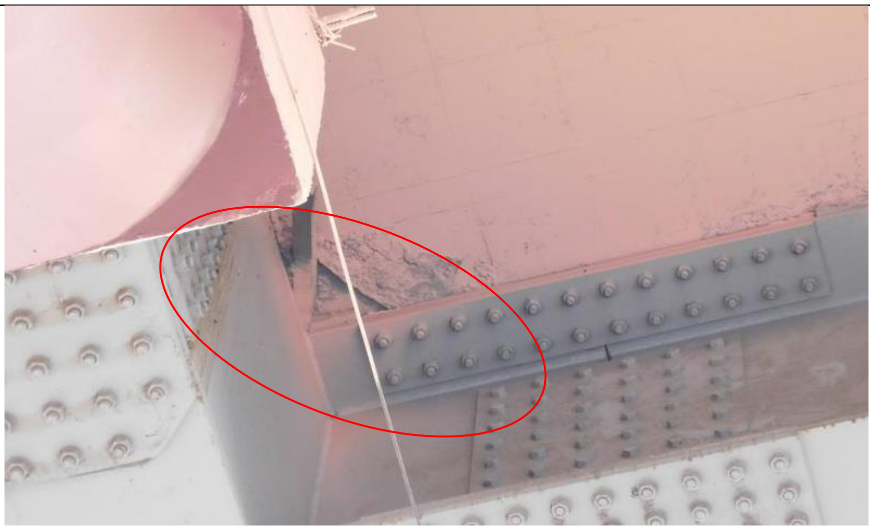
 *Photo 14*	**P10 P11** **Panel 13**
 *Photo 15*	**P7 P8** **Panel 28**
 *Photo 16*	**P4-P5** **Panel 25**

 *Photo 17*	**P8-P9** **Panel 13**

STEEL STAIRCASE

Photo 18. Typical elevation of Staircase

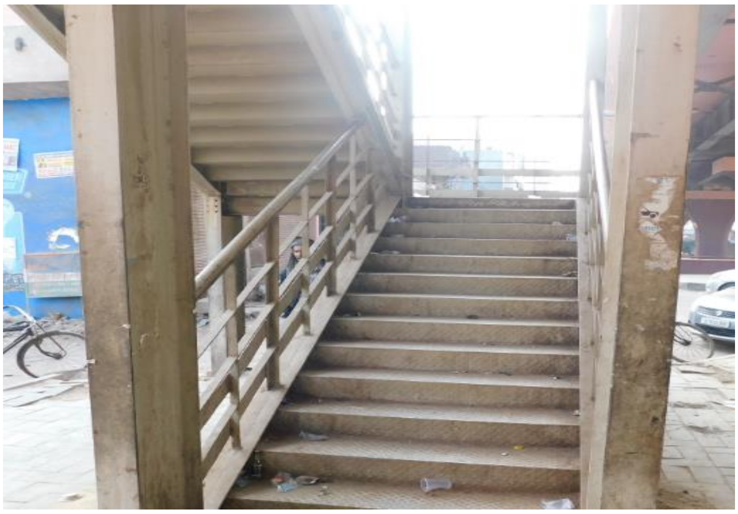


Photo 19.Staircase

Photo 20. Typical elevation of Staircase

WALK THROUGH/VISUAL SURVEY OBSERVATION

There are 4 no's of steel staircases at Nand Nagri flyover between P11- P12 (left-right side) and P12-P13 (left-right side)

Corrosion and paint failure noticed on Steel Staircases

STEEL GIRDERS

Photo 21. Outer steel girder

Photo21.a) Inner Steel girders with cross bracings

Photo 22. Steel girders showing no sign of corrosion

Photo 22.a) Steel bolt connection

Photo 23. Cross and rectangular bracings

Photo 23.a) End Diaphragms with no distress

Photo 24. Steel girders connection with bearings

Photo 24.a) Girders resting in abutments

WALK THROUGH/ VISUAL SURVEY OBSERVATION

No paint peeling off, **No** paint bubbled and **No** corrosion noticed on web, flange and bracings with diaphragms in steel superstructure HSFG bolts are in tight and showing no sign of corrosion are observed.

STEEL GIRDERS’ DIMENSION

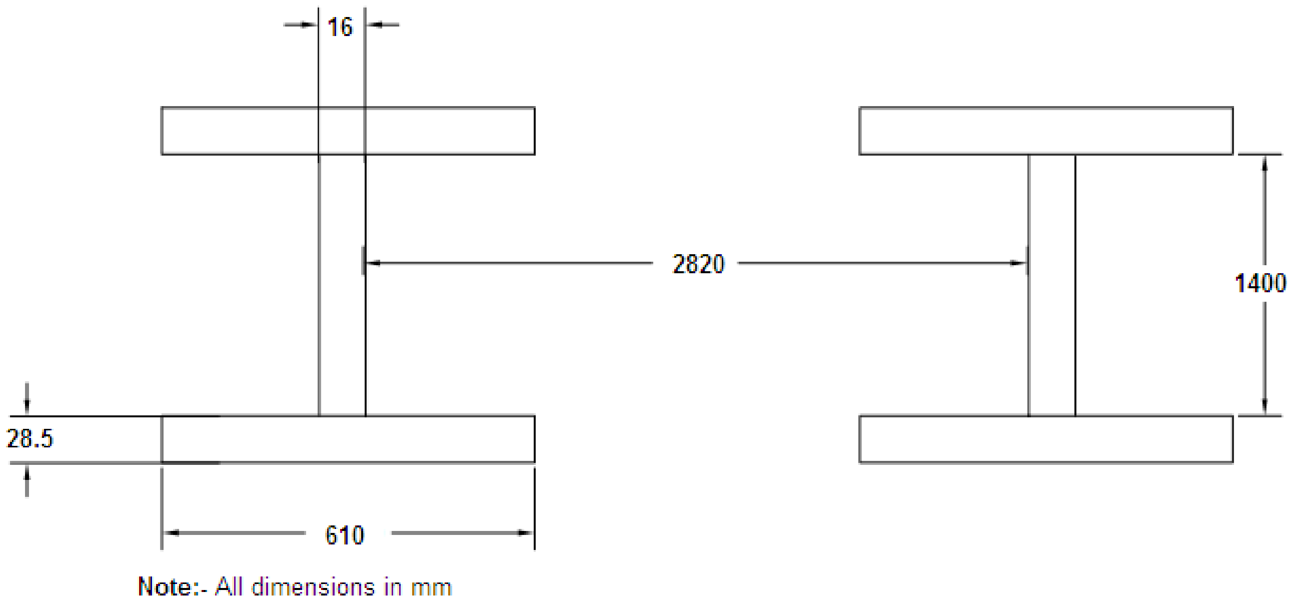


Fig. 20.The dimension of steel girders

Fig. 21.Panel numbers with steel girder.

WALK THROUGH/VISUAL SURVEY OBSERVATION

There are **6 Nos.** of steel girders and its sectional dimensions are mentioned in the above Fig. 20.

ROAD SIDE GRILL

Photo 25.Roadside Grills in good condition.

Photo 26. Crash Barrier are in good condition without any damages

R.E. WALL

Photo 27.Approach R.E wall

Photo 28. Approach side RE wall

Photo 29. Side wall in RUB

Photo 30.Side wall in RUB

ABUTMENT

Photo 31. Abutments noted with no distress

Photo 31.a) Abutments noted with no distress

Photo 32. Outer side of Abutments

COPPER U STRIP & JOINT FILLER TYPE EXPANSION JOINT

Photo 33. No damages in expansion joints

Photo 34. Transverse crack due to tension near Expansion joints

WALK THROUGH/VISUAL SURVEY OBSERVATION

There are no visual distress/damage found in Abutment, Retaining wall, Road side grill & Copper U strip & joint filler type expansion joint.

PIER/ PIERCAP

Photo 35. Piers showing no distress or damages

Photo 36. Pier in the RUB portion

Photo 37. Pier caps with any cracks or damges

Photo 38. Pier protection cover

WALK THROUGH/VISUAL SURVEY OBSERVATION

There are no observations of distress/damage found in Pier/Pier Cap/Pier Foundation.

Size of pier cap (L×W×H) = 5.5m × 2.08m × 0.762m

BEARING/ BEARING PEDESTAL AND SEISMIC RESTRAINERS

Photo 39.Bearings without any damges

Photo 40. Bearings are in good condition

Photo 41. Seismic Restrainers without any damage

Photo 42. Seismic Restrainers showing no distress or damage

WALK THROUGH/VISUAL SURVEY OBSERVATION

On observation there are no distress/damages found in Bearings & its Pedestals.

Size of Side Bearing Pedestal (L×W×H) = 900mm × 900mm × 250mm
Size of Centre Bearing Pedestal (L×W×H) = 1300mm × 1300mm × 150mm
Total Number of Bearings (Left× Centre× Right) = 23-7-23= 53 Nos.

KERB STONE

Photo 43.Kerb Stones in the median

Photo 44.Kerbstones along the outer side of the road

ROAD SIDE FOOTPATH

Photo 45. Footpath in the Raily portion (ROB)

Photo 46. Footpath are in good condition

Photo 47. Outlet pipes are not connected to the downtakes

Photo 47.a) Drainage pipe missing.

WALK THROUGH/VISUAL SURVEY OBSERVATION

There is RCC work in place of side kerb stones found at Nand Nagri flyover. There is footpath existing in railway portion (ROB) on either side of the road. The outlet pipes from the road are free below without further connection with the drainage pipe system.

DEPRESSION/POT HOLES

Photo 48. Depression / cracking in the bituminous layer

Photo 49.Distressed pattern on wearing coat

WALK THROUGH/VISUAL SURVEY OBSERVATION

There is Depression/ pot holes observed on damage panels like; P17-P18 (Panel-6) on road at Nand Nagri Flyover.

2.2 HAMMER RAP SURVEY:-

Quality Assurance in Concrete using Non Destructive Testing			
Client: PWD, DELHI		**Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**	
Hammer Rap Survey On RCC at Nand Nagri Flyover, New Delhi			
SL. No.	**Location**	**Remarks**	**Sound Observations** (Dull Hollow Sound or Solid Sound)
1	P11-P12 (Panel 18)	Cracks	Dull Hollow Sound
2	P11-P12 (Panel 19)	Cracks	Dull Hollow Sound
3	P11-P12 (Panel 12)	Major Cracks	Dull Hollow Sound
4	**P11-P12 (Panel 13)**	**Honeycombs, Major Cracks, Spalling, Panel Damaged**	**Dull Hollow Sound**
5	P11-P12 (Panel 5)	Major Cracks	Dull Hollow Sound
6	P12-P13 (Panel 3)	Major Crack	Dull Hollow Sound
7	P12-P13 (Panel 10)	Major Crack	Dull Hollow Sound
8	**P12-P13 (Panel 17)**	**Major Crack, Panel Damaged**	**Dull Hollow Sound**
9	**P12-P13 (Panel 24)**	**Major Crack, Panel Damaged**	**Dull Hollow Sound**
10	P12-P13 (Panel 26)	Cracks	Dull Hollow Sound
11	P12-P13 (Panel 27)	No Distress	Solid Sound
12	P13-P14 (Panel 10)	No Distress	Solid Sound
13	P13-P14 (Panel 11)	No Distress	Solid Sound
14	P13-P14 (Panel 17)	No Distress	Solid Sound
15	P13-P14 (Panel 18)	No Distress	Solid Sound
16	P13-P14 (Panel 19)	No Distress	Solid Sound
17	P13-P14 (Panel 24)	No Distress	Solid Sound
18	**P13-P14 (Panel 25)**	**Major Cracks**	**Dull Hollow Sound**
19	P13-P14 (Panel 26)	Damage Panel, Cracks, Honey Combing	Dull Hollow Sound
20	P16-P17 (Panel 17)	No Distress	Solid Sound
21	P16-P17 (Panel 18)	No Distress	Solid Sound
22	P16-P17 (Panel 25)	Honey Combing, Cracks	Dull Hollow Sound
23	**P17-P18 (Panel 6)**	**Damage Panel, Major Cracks**	**Dull Hollow Sound**
24	P17-P18 (Panel 10)	No Distress	Solid Sound
25	P17-P18 (Panel 16)	Major Cracks, Panel Damage	Dull Hollow Sound
26	**P17-P18 (Panel 17)**	**Major Cracks, Honeycombs**	**Dull Hollow Sound**
27	**P17-P18 (Panel 18)**	**Panel Damage, Major Crack**	**Dull Hollow Sound**
28	**P17-P18 (Panel 24)**	**Panel Damage, Major Crack**	**Dull Hollow Sound**
29	P17-P18 (Panel 25)	Honeycombs, Cracks	Dull Hollow Sound
30	P-19-P-20 (Panel 3)	Minor cracks	Dull Hollow Sound
31	P3-P4 (Panel 9)	No Distress	Solid Sound
32	P3-P4 (Panel 23)	No Distress	Solid Sound

33	A1-P1 (Panel 3)	No Distress	Solid Sound
34	A1-P1 (Panel 19)	No Distress	Solid Sound
35	P1-P2 (Panel 4)	No Distress	Solid Sound
36	P1-P2 (Panel 14)	No Distress	Solid Sound
37	P2-P3 (Panel 18)	Minor Honeycombs	Dull Hollow Sound
38	P2-P3 (Panel 16)	No Distress	Solid Sound
39	P4-P5 (Panel 26)	No Distress	Solid Sound
40	P4-P5 (Panel 4)	No Distress	Solid Sound
41	P5-P6 (Panel 25)	Minor Honeycombs	Dull Hollow Sound
42	P5-P6 (Panel 19)	No Distress	Solid Sound
43	**P5-P6 (Panel 4)**	**Major Cracks, Honeycombs**	**Dull Hollow Sound**
44	**P5-P6 (Panel 3)**	**Major Cracks, Honeycombs**	**Dull Hollow Sound**
45	P6-P7 (Panel 13)	Major Cracks, Honeycombs	Dull Hollow Sound
46	P6-P7 (Panel 27)	No Distress	Solid Sound
47	P6-P7 (Panel 26)	Minor Cracks, Honeycombs	Dull Hollow Sound
48	P7-P8 (Panel 20)	No Distress	Solid Sound
49	P7-P8 (Panel 17)	No Distress	Solid Sound
50	P8-P9 (Panel 3)	No Distress	Solid Sound
51	P8-P9 (Panel 20)	No Distress	Solid Sound
52	P9-P10 (Panel 18)	No Distress	Solid Sound
53	P9-P10 (Panel 19)	Minor Cracks, Honeycomb	Dull Hollow Sound
54	P10-P11 (Panel 19)	Minor Honeycombs	Dull Hollow Sound
55	P10-P11 (Panel 6)	No Distress	Solid Sound
56	P18-P19 (Panel 11)	No Distress	Solid Sound
57	P19-P20 (Panel 18)	No Distress	Solid Sound
58	P20-P21 (Panel 3)	Minor Honeycombs	Dull Hollow Sound
59	P21-A2 (Panel 17)	No Distress	Solid Sound
60	P21-A2 (Panel 6)	No Distress	Solid Sound

Interpretation(Sound Observed):- Dull Hollow Sound **/** Solid Sound

Dull Hollow Sound: - This sound is heard where Cracks, Honeycombing and Loose concreting etc. suspect to be in the concrete structure.

Solid Sound: - This sound is heard where the concrete structure is in good condition.

ALL THE PANELS OF BRIDGE ARE COVERED BY HAMMER RAP & VISUAL SURVEY

PHOTOGRAPHS OF HAMMER RAP SURVEY

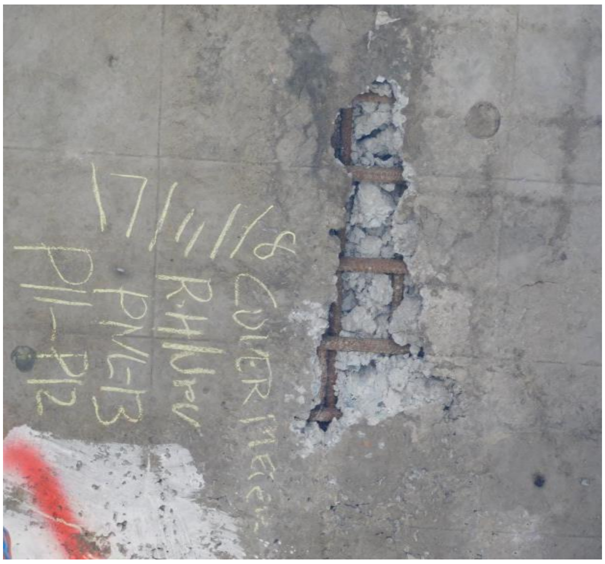
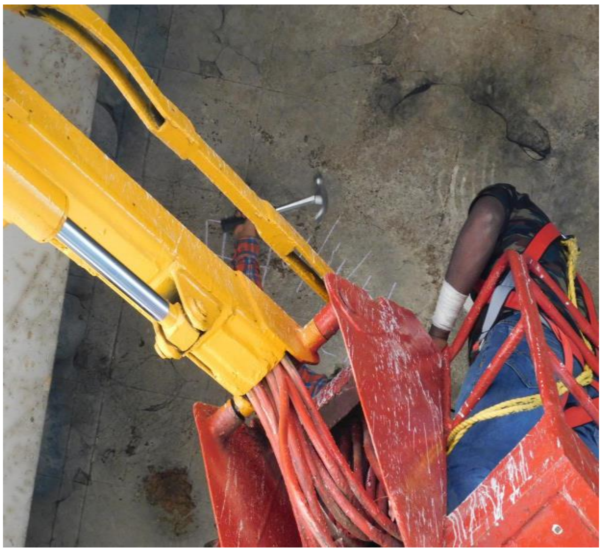
	

Photo 50	**Photo 51**
	
Photo 52	**Photo 53**
	
Photo 54	**Photo 55**

	
Photo 56	**Photo 57**
	
Photo 58	**Photo 59**
	
Photo 60	**Photo 61**

	
Photo 62	**Photo 63**
	
Photo 64	**Photo 65**

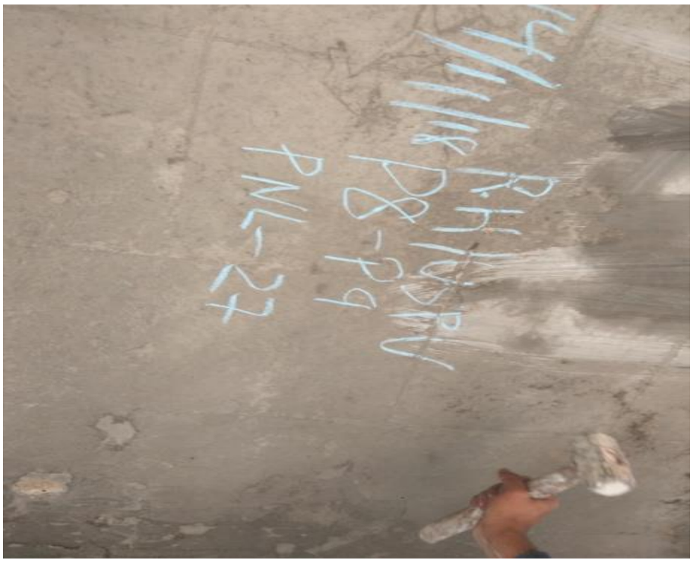



Photo 66	**Photo 67**

Photo 68

Photo 69

3. TEST PLAN

3.1. TEST PLAN:-

On behalf of primary evaluation further In-situ and laboratory testing was selected to know the nature of the Cracks, Spalling, Compressive Strength, and Cover etc. As per surface condition based on Visual Survey and Hammer Rap Survey, test locations were decided to cover the secondary evaluation of overall structure.

These are the tests executed to provide an idea about the degree of damage encountered in the concrete structure:-

1) **Ultrasonic Pulse Velocity Test** as per IS: 13311 (Part-1)-1992 for ascertaining the quality of concrete, soundness and density of concrete.

2) **Rebound Hammer Test:** For determining the estimated compressive strength of concrete and uniformity of concrete in terms of surface hardness as per IS 13311 (Part-2)-1992.

3) **Cover Meter Test:** Conducting cover meter test at selected locations on RCC members of the structures covered under the study to see the adequacy of concrete cover to rebars and effect of carbonation.

4) **Chemical Tests on Concrete** in the laboratory to determine the following parameters to understand the chemical deterioration /degradation of concrete and its effect on reinforcement corrosion.
    a) Chloride content as per IS: 14959 (Part 2) – 2001, B.S. 5328, ACI 201.2R-92.
    b) Sulphate content as per IS: 4032.
    c) PH value as per relevant B.S. 5328, ACI 201.2R-92 and ACI-318-99.

5) **Carbonation Test** as per BS EN: 14630 Measurement of carbonation depth by phenolphthalein spray test at selected locations on RCC members of the structures covered under the study to see the depth of carbonation.

6) **Core Extraction**: IS: 516, concrete extraction for exact in-situ compressive strength evaluation of concrete, grade and f_{ck} value of concrete.

7) **Crack pattern Analysis**: Ultrasonic Pulse Velocity was used to examine the crack propagation with its depth in the existing cracked surface.

8) **Thickness Testing** to check reduction in thickness of steel members caused due to corrosion.

3.2 SAMPLE AND SITE DATA COLLECTION:-

These five instruments i.e. Ultrasonic Pulse velocity meter, Rebound Hammer, bar locator, concrete core cutting machine & Thickness Gauge were used to collect the data from the structure.

As per the visual inspection and Hammer Rap Survey on the entire bridge, the numbers of tests which are required on super structure and sub structure have been decided to find out the condition of the bridge.

3.3 NUMBER OF TESTS TO BE CONDUCTED:

S.No.	Name of the Bridge	Ultrasonic Pulse Velocity Test	Rebound Hammer Test	Cover Meter	Crack Pattern
1.	Nand Nagri Flyover North East Delhi	138 Nos.	121 Nos.	30 Nos.	8 Nos.
		Thickness Test	**Core Test**	**Carbonation Test**	**Chemical Test**
		6 Nos.	10 Nos.	10 Nos.	10 Nos.

4. NDT TEST RESULTS & INTERPRETATION

(SECONDARY EVALUATION)

4.1 ULTRASONIC PULSE VELOCITY:-

Purpose:-

Although there is no fundamental relationship between pulse velocity and strength, an estimation of strength can be obtained by correlation. The method has perhaps a greater potential for comparing known sound concrete with affected concrete.

Ultrasonic pulse velocity is a means of assessing variations in the apparent strength of concrete.

The quality gradation of concrete can be appraised at best qualitatively as **'Excellent', 'Good', 'Medium' or 'Doubtful'.** The meanings of the term `excellent', `good', `medium' and `doubtful' are based on ultra-sonic pulse velocity measured at site and are as per the nomenclature of IS 13311(part-1): 1992. To strike balance between the reliability, speed and damage to structure, core test have to be used to establish a correlation between rebound number index and the estimated in-situ strength with the USPV test results in the investigation.

Objective of testing:-

Ultrasonic pulse velocity test is used to establish the following:

- ✓ Homogeneity of concrete
- ✓ Presence of cracks voids, honeycombing and other imperfections
- ✓ Changes in the structure of concrete which may occur with time.
- ✓ Quality of one element of concrete in relation to another i.e. comparative quality analysis and gradation of concrete.
- ✓ The values of dynamic elastic modulus of the concrete.

References:-

- ✓ BS 6089:1981 and BS 1881:Part203
- ✓ IS 13311:Part1:1992
- ✓ ASTM: C597-83.

Influencing factors:-

The velocity of a pulse of ultrasonic energy in concrete is influenced by the stiffness and mechanical strength of the concrete

- ✓ Moisture content: The moisture content of the concrete have a small effect in the velocity and can increase the pulse velocity by 2%.
- ✓ Surface condition: The testing surface should be smooth any roughness cannot provide reliable readings because of gap between transducers and testing surface.
- ✓ Temperature: Ideal Temperature is between 5°C and 30°C; Temperature between 30°C to 60°C can reduce the pulse velocity up to 5%; below freezing temperature results in an increase the pulse

velocity up to 7.5%.

- ✓ Stress: When concrete is subjected to a stress which is abnormally high for a quality of concrete, the pulse velocity may be reduced due to development of micro-cracks.
- ✓ Reinforcing bars: The velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete because pulse velocity in steel is 1.2-1.9 times the velocity in plain concrete. Wherever possible, measurements should be made in such a way that steel does not lie in the path of the pulse.

Testing method:-

According to IS 13311(Part1):1992 clause 5.2 transducers with a frequency of 50 to 60 kHz are useful for most all round applications, and as per IS 13311(Part1):1992 clause 6.2 the path length should be long enough not to be significantly influenced by the heterogeneous nature of concrete. This test requires a flat surface generally only appropriate for unspalled surfaces.

In view of inherent variability in the test results, sufficient number of readings should be taken by dividing the entire structure in suitable grid of markings 30cm x 30 cm or even smaller. Each junction point of the grid becomes a point of observation.

Methods of Measuring Pulse velocities:

There are three ways of measuring pulse velocity through concrete.

a) ***Direct method or Cross Probing method-*** Most preferred method- whenever access to opposite sides of the structural component is available.

b) ***Surface probing or indirect method -*** whenever access to only one surface of the structural component is available. Surface probing method gives lower velocities than with cross probing method.

c) ***Semi - Direct method -*** whenever access to different but not opposite sides of the structural component is available.

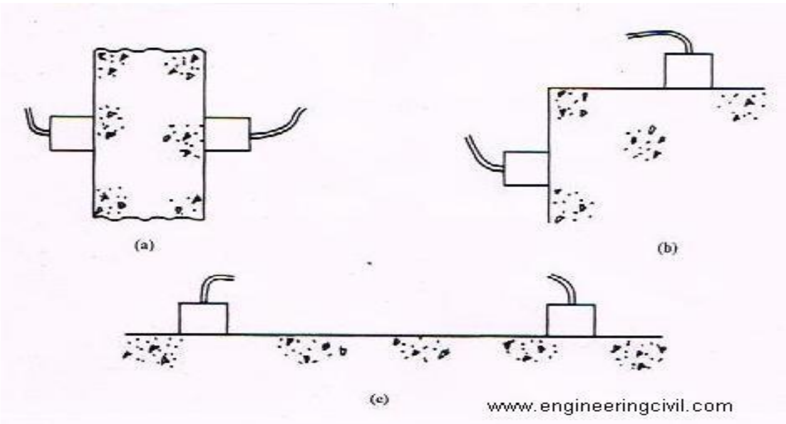


Fig. 22.The three different types of measuring the pulse velocity through concrete

Velocity Criterion for Concrete Quality Grading [Ref: IS13311 (part-1)]

Sr. No.	**USPV by Cross Probing (km/sec)**	**Concrete Quality Grading**
1	**Above 4.5**	**Excellent**
2	**3.5 - 4.5**	**Good**
3	**3.0 - 3.5**	**Medium**
4	**Below 3.0**	**Doubtful**

<u>USPV relevant testing code:-</u>

IS 13311 (Part 1) : 1992
(Reaffirmed 1999)

भारतीय मानक

कंकरीट का अविनाशी परीक्षण — परीक्षण पद्धतियां

भाग 1 पराश्रव्यी स्पन्द वेग

Indian Standard

NON-DESTRUCTIVE TESTING OF CONCRETE — METHODS OF TEST

PART 1 ULTRASONIC PULSE VELOCITY

First Reprint SEPTEMBER 1996

UDC 666·972·620·179·16

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

January 1992

Price Group 4

TEST CERTIFICATE: - ULTRASONIC PULSE VELOCITY TEST

Quality Assurance in Concrete using Non Destructive Testing

Ultrasonic Pulse Velocity (IS: 13311 Part 1)

Client: PWD, DELHI **Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**

Non Destructive Testing at NandNagri Flyover

DECK SLAB

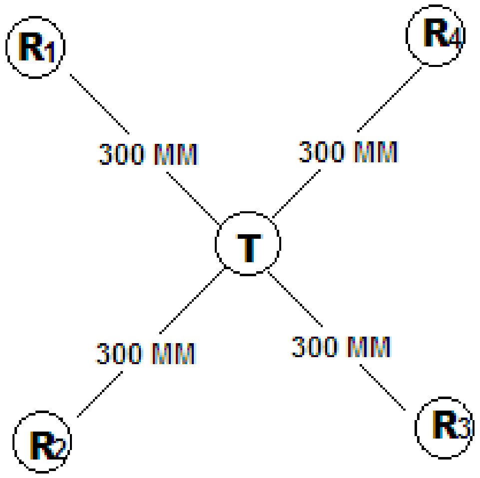
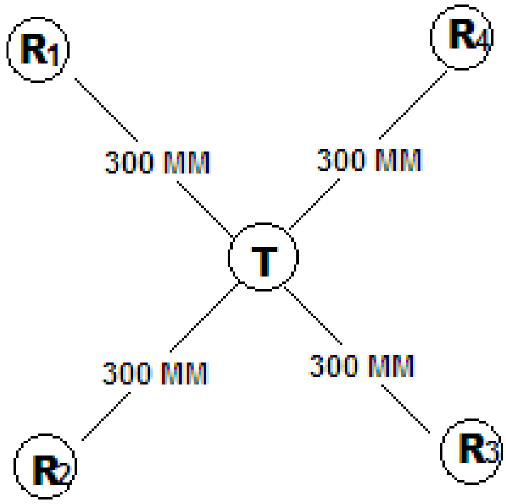
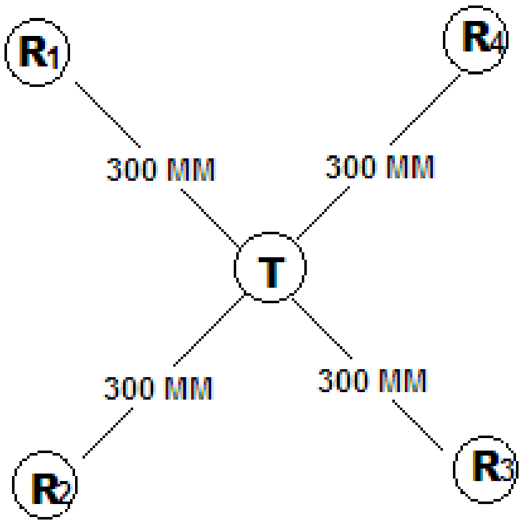
SL. No.	Sample Identification/ Location	Type of surface	Distance (mm)	Travel Time (micro sec.)	Avg. Velocity (km/sec)	Direct Proportionate Velocity (IS 13311 part 1 cl. 5.4.1)	Concrete Quality
1	P-9,P-10, PNL-18	Indirect	300	138.12	2.172	3.172	Medium
2	P-9,P-10, PNL-18	Indirect	300	83.71	3.584	4.584	Excellent
3	P-9,P-10, PNL-15	Indirect	300	81.61	3.676	4.676	Excellent
4	P-9,P-10, PNL-6	Indirect	300	97.66	3.072	4.072	Good
5	P-8,P-9, PNL-27	Indirect	300	80.6	3.722	4.722	Excellent
6	P-8,P-9, PNL-18	Indirect	300	90.2	3.326	4.326	Good
7	P-8,P-9, PNL-3	Indirect	300	90.61	3.311	4.311	Good
8	P-7,P-8, PNL-25	Indirect	300	92.11	3.257	4.257	Good
9	P-7,P-8, PNL-17	Indirect	300	116.6	2.573	3.573	Good
10	P-7,P-8, PNL-20	Indirect	300	99.21	3.024	4.024	Good
11	P-7,P-6, PNL-16	Indirect	300	127.55	2.352	3.352	Medium
12	P-7,P-6, PNL-13	Indirect	300	144.93	2.07	3.07	Medium
13	P-7,P-8, PNL-15	Indirect	300	109.41	2.742	3.742	Good
14	P-5,P-6, PNL-25	Indirect	300	82.6	3.632	4.632	Excellent
15	P-5,P-6, PNL-19	Indirect	300	84.51	3.55	4.55	Excellent
16	P-5,P-6, PNL-11	Indirect	300	74.81	4.01	5.01	Excellent
17	P-5,P-6, PNL-4	Indirect	300	146.33	2.05	3.05	Medium
18	P-5,P-6, PNL-5	Indirect	300	108.7	2.76	3.76	Good
19	**P-5,P-6, PNL-2**	**Indirect**	**300**	**234.38**	**1.28**	**2.28**	**Doubtful**
20	P-11,P-12, PNL-18	Indirect	300	124.9	2.402	3.402	Medium
21	P-11,P-12, PNL-17	Indirect	300	82.21	3.649	4.649	Excellent
22	P-11,P-12, PNL-16	Indirect	300	122.6	2.447	3.447	Medium
23	P-11,P-12, PNL-12	Indirect	300	130.61	2.297	3.297	Medium
24	P-11,P-12, PNL-5	Indirect	300	134.11	2.237	3.237	Medium
25	P-11,P-12, PNL-4	Indirect	300	109.41	2.742	3.742	Good
26	P-11,P-12, PNL-6	Indirect	300	96.59	3.106	4.106	Good
27	P-10,P-11, PNL-26	Indirect	300	105.12	2.854	3.854	Good
28	P-10,P-11, PNL-19	Indirect	300	131.58	2.28	3.28	Medium
29	P-10,P-11, PNL-20	Indirect	300	107.41	2.793	3.793	Good
30	P-11,P-12, PNL-41	Indirect	300	104.46	2.872	3.872	Good
31	P-11,P-12, PNL-34	Indirect	300	137.17	2.187	3.187	Medium
32	P-12,P-13, PNL-13	Indirect	300	84.6	3.546	4.546	Excellent
33	P-12,P-13, PNL-12	Indirect	300	100.6	2.982	3.982	Good
34	P-12,P-13, PNL-5	Indirect	300	93.2	3.219	4.219	Good
35	P-12,P-13, PNL-6	Indirect	300	135.38	2.216	3.216	Medium

36	P-12,P-13, PNL-2	Indirect	300	135.87	2.208	3.208	Medium
37	P-12,P-13, PNL-9	Indirect	300	131.93	2.274	3.274	Medium
38	P-12,P-13, PNL-5	Indirect	300	135.38	2.216	3.216	Medium
39	P-12,P-13, PNL-3	Indirect	300	144.79	2.072	3.072	Medium
40	P-12,P-13, PNL-4	Indirect	300	111.4	2.693	3.693	Good
41	P-12,P-13, PNL-11	Indirect	300	121.61	2.467	3.467	Medium
42	P-12,P-13, PNL-20	Indirect	300	125.21	2.396	3.396	Medium
43	P-12,P-13, PNL-27	Indirect	300	97.69	3.071	4.071	Good
44	P-12,P-13, PNL-19	Indirect	300	121.41	2.471	3.471	Medium
45	P-12,P-13, PNL-26	Indirect	300	128.15	2.341	3.341	Medium
46	P-12,P-13, PNL-25	Indirect	300	95.91	3.128	4.128	Good
47	P-19,P-20, PNL-11	Indirect	300	101.39	2.959	3.959	Good
48	P-19,P-20, PNL-10	Indirect	300	108.38	2.768	3.768	Good
49	**P-19,P-20, PNL-3**	**Indirect**	**300**	**180.61**	**1.661**	**2.661**	**Doubtful**
50	P-19,P-20, PNL-17	Indirect	300	115.38	2.6	3.6	Good
51	P-13,P-14, PNL-10	Indirect	300	104.38	2.874	3.874	Good
52	P-13,P-14, PNL-11	Indirect	300	102.7	2.921	3.921	Good
53	P-13,P-14, PNL-17	Indirect	300	121.9	2.461	3.461	Medium
54	P-13,P-14, PNL-18	Indirect	300	111.4	2.693	3.693	Good
55	P-13,P-14, PNL-19	Indirect	300	111.61	2.688	3.688	Good
56	P-13,P-14, PNL-26	Indirect	300	89.9	3.337	4.337	Good
57	P-13,P-14, PNL-24	Indirect	300	144.86	2.071	3.071	Medium
58	P-14,P-15, PNL-9	Indirect	300	143.75	2.087	3.087	Medium
59	P-14,P-15, PNL-12	Indirect	300	141.11	2.126	3.126	Medium
60	P-14,P-15, PNL-18	Indirect	300	94.91	3.161	4.161	Good
61	P-15,P-16, PNL-16	Indirect	300	144.23	2.08	3.08	Medium
62	P-15,P-16, PNL-26	Indirect	300	142.18	2.11	3.11	Medium
63	P-15,P-16, PNL-25	Indirect	300	127.88	2.346	3.346	Medium
64	P-16,P-17, PNL-17	Indirect	300	109.61	2.737	3.737	Good
65	P-16,P-17, PNL-18	Indirect	300	140.38	2.137	3.137	Medium
66	**P-16,P-17, PNL-25**	**Indirect**	**300**	**177.94**	**1.686**	**2.686**	**Doubtful**
67	P-17,P-18, PNL-10	Indirect	300	129.93	2.309	3.309	Medium
68	P-17,P-18, PNL-17	Indirect	300	141.38	2.122	3.122	Medium
69	P-17,P-18, PNL-16	Indirect	300	143.2	2.095	3.095	Medium
70	**P-17,P-18, PNL-25**	**Indirect**	**300**	**157.4**	**1.906**	**2.906**	**Doubtful**
71	**P-17,P-18, PNL-18**	**Indirect**	**300**	**303.64**	**0.988**	**1.988**	**Doubtful**
72	P-18,P-19, PNL-3	Indirect	300	89.61	3.348	4.348	Good
73	P-18,P-19, PNL-11	Indirect	300	90.69	3.308	4.308	Good
74	P-18,P-19, PNL-4	Indirect	300	183.15	1.638	2.638	Doubtful
75	P-18,P-19, PNL-2	Indirect	300	123.71	2.425	3.425	Medium
76	P-20,P-21, PNL-13	Indirect	300	136.12	2.204	3.204	Medium
77	P-20,P-21, PNL-27	Indirect	300	132.92	2.257	3.257	Medium
78	P-20,P-21, PNL-6	Indirect	300	121.11	2.477	3.477	Medium
79	P-20,P-21, PNL-12	Indirect	300	120.92	2.481	3.481	Medium
80	P-20,P-21, PNL-23	Indirect	300	120.1	2.498	3.498	Medium
81	P-21,P-22, PNL-18	Indirect	300	128.26	2.339	3.339	Medium
82	P-21,A-2, PNL-6	Indirect	300	104.71	2.865	3.865	Good
83	P-22,A-2, PNL-6	Indirect	300	139.41	2.152	3.152	Medium
84	P-22,A-2, PNL-3	Indirect	300	133.21	2.252	3.252	Medium

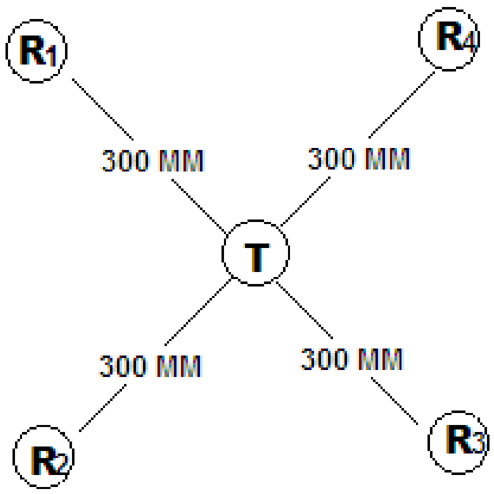
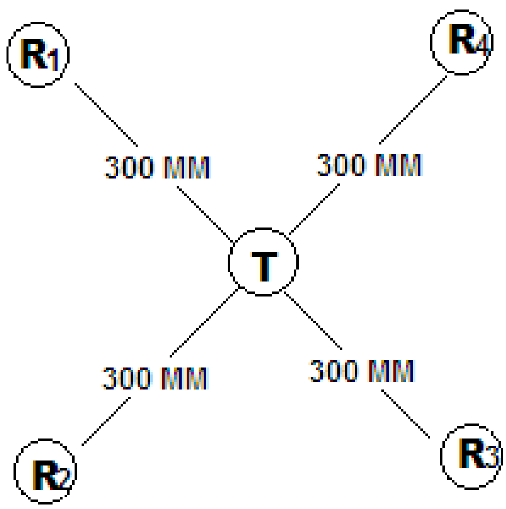
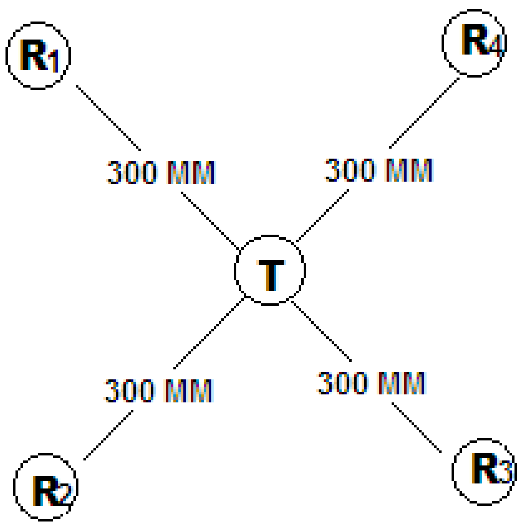
85	P-22,A-2, PNL-23	Indirect	300	113.94	2.633	3.633	Good
86	P-4,P-5, PNL-10	Indirect	300	108.58	2.763	3.763	Good
87	P-4,P-5, PNL-5	Indirect	300	134.89	2.224	3.224	Medium
88	P-4,P-5, PNL-27	Indirect	300	134.89	2.224	3.224	Medium
89	P-3,P-4, PNL-26	Indirect	300	109.89	2.73	3.73	Good
90	P-3,P-4, PNL-23	Indirect	300	128.26	2.339	3.339	Medium
91	P-3,P-4, PNL-9	Indirect	300	122.1	2.457	3.457	Medium
92	P-2,P-3, PNL-23	Indirect	300	118.72	2.527	3.527	Good
93	P-2,P-3, PNL-9	Indirect	300	99.11	3.027	4.027	Good
94	P-2,P-3, PNL-3	Indirect	300	113.12	2.652	3.652	Good
95	P-1,P-2, PNL-23	Indirect	300	127.5	2.353	3.353	Medium
96	P-1,P-2, PNL-16	Indirect	300	136.86	2.192	3.192	Medium
97	P-1,P-2, PNL-11	Indirect	300	124.84	2.403	3.403	Medium
98	P-1,A-1, PNL-25	Indirect	300	130.89	2.292	3.292	Medium
99	P-1,A-1, PNL-27	Indirect	300	112.07	2.677	3.677	Good
100	P-1,A-1, PNL-11	Indirect	300	93.14	3.221	4.221	Good
101	P-1,A-1, PNL-12	Indirect	300	97.06	3.091	4.091	Good
102	P-1,A-1, PNL-2	Indirect	300	118.06	2.541	3.541	Good
PIER							
103	P11	Indirect	300	99.83	3.005	4.005	Good
104	P12	Indirect	300	99.44	3.017	4.017	Good
105	P13	Indirect	300	136.36	2.2	3.2	Medium
106	P14	Indirect	300	98.91	3.033	4.033	Good
107	P17	Indirect	300	99.65	3.01	4.01	Good
108	P18	Indirect	300	147.06	2.04	3.04	Medium
109	P20	Indirect	300	93.18	3.22	4.22	Good
110	P8	Indirect	300	116.28	2.58	3.58	Good
111	P5	Indirect	300	145.07	2.068	3.068	Medium
112	P3	Indirect	300	113.21	2.65	3.65	Good
ABUTMENT							
113	A1	Indirect	300	106.91	2.806	3.806	Good
114	A2	Indirect	300	118.72	2.527	3.527	Good

As per USPV(IS:13311 part 1)		
Sr. No.	**USPV by Cross Probing (km/sec)**	**Concrete Quality Grading**
1	Above 4.5	Excellent
2	3.5 - 4.5	Good
3	3.0 - 3.5	Medium
4	Below 3.0	Doubtful

USPV TESTING AT ONE LOCATION FOR VELOCITY VARIATION

Velocity Variation Graph (**T**-Transducer/**R**-Receiver/**Distance b/w T & R-** 300 mm)	**Direct Velocity (m/s)**	**Remarks**
	**1. 2279** **2. 2978** **3. 2024** **4. 3114**	**P11-P12** **Panel-24** (There is low variation in velocity, only minor surface cracks are present in panel)
	**1. 2046** **2. No Reading** **3. 3879** **4. 2570**	**P11-P12** **Panel-13** (There is High variation in velocity, Minor & Major surface cracks/ Honeycombing are present in panel)
	**1. 2740** **2. 2277** **3. 2989** **4. 3108**	**P12-P13** **Panel-24** (There is medium variation in velocity, Minor & Major surface cracks/Honeycombing are present in panel)

USPV TESTING AT ONE LOCATION FOR VELOCITY VARIATION

Velocity Variation Graph (**T**-Transducer/**R**-Receiver/**Distance b/w T & R-** 300 mm)	**Direct Velocity**	**Remarks**
	**1.2586** **2.3204** **3.2874** **4.3717**	**P13-P14** **Panel-25** (There is medium variation in velocity, Minor & Major surface cracks are present in panel)
	**1.2482** **2.No Reading** **3.2197** **4.No Reading**	**P14-P15** **Panel-3** (There is High variation in velocity, Minor & Major surface cracks are present in panel) No reading because of major cracks & Honeycombing
	**1.2806** **2.2403** **3.3206** **4.2762**	**P11-P12** **Panel-24** (There is low variation in velocity, only minor surface cracks are present in panel)

A. Analysis of uniformity and imperviousness of concrete on the basis of USPV test results:-

USPV makes possible an examination of material homogeneity. Analyzing the ultrasonic velocity wave propagation variations, it is possible to verify the compact of the structure or detect heterogeneous regions. The ultrasonic test methodology in concrete is based on the fact that the propagation time expresses the density of the material. Histogram of USPV test results is analyzed in same pattern as rebound hammer is done but basic difference is that USPV results are interrelated in terms of density and rebound hammer results are interrelated in terms of surface hardness. It has non-uniform concrete quality in terms of density. There are indications of air-pockets and voids as significant from USPV test results as per **IS: 13311 part 1.** Statistical data says that **over all concrete has varied grade quality of density pattern.**

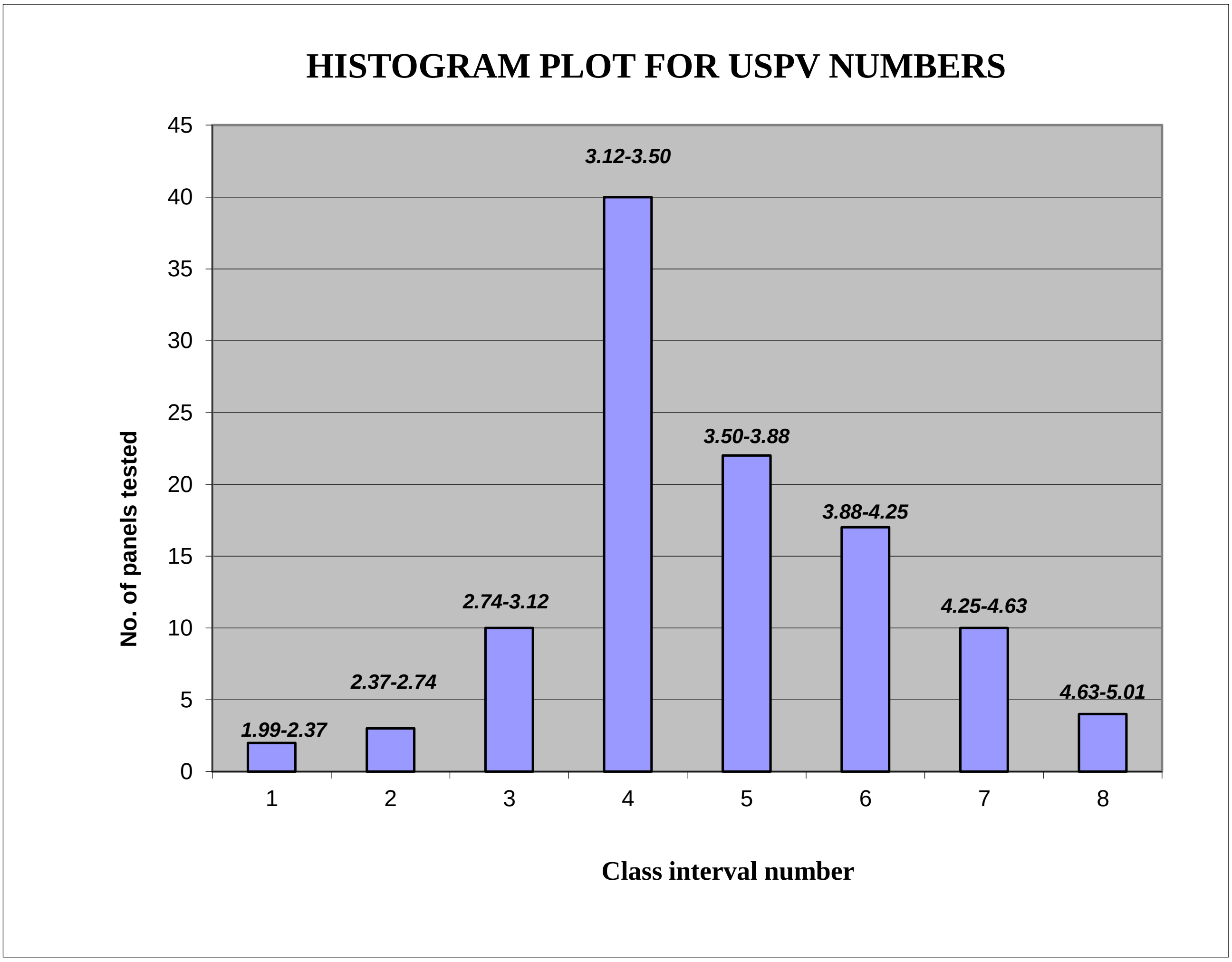


Fig. 23.Shows the Histogram plot for USPV numbers

Total No. of Test Location on panels	Min.	Max.	Mean	Mode	Median	Standard Deviation S/P
102	1.99	5.01	3.61	3.74	3.49	0.54

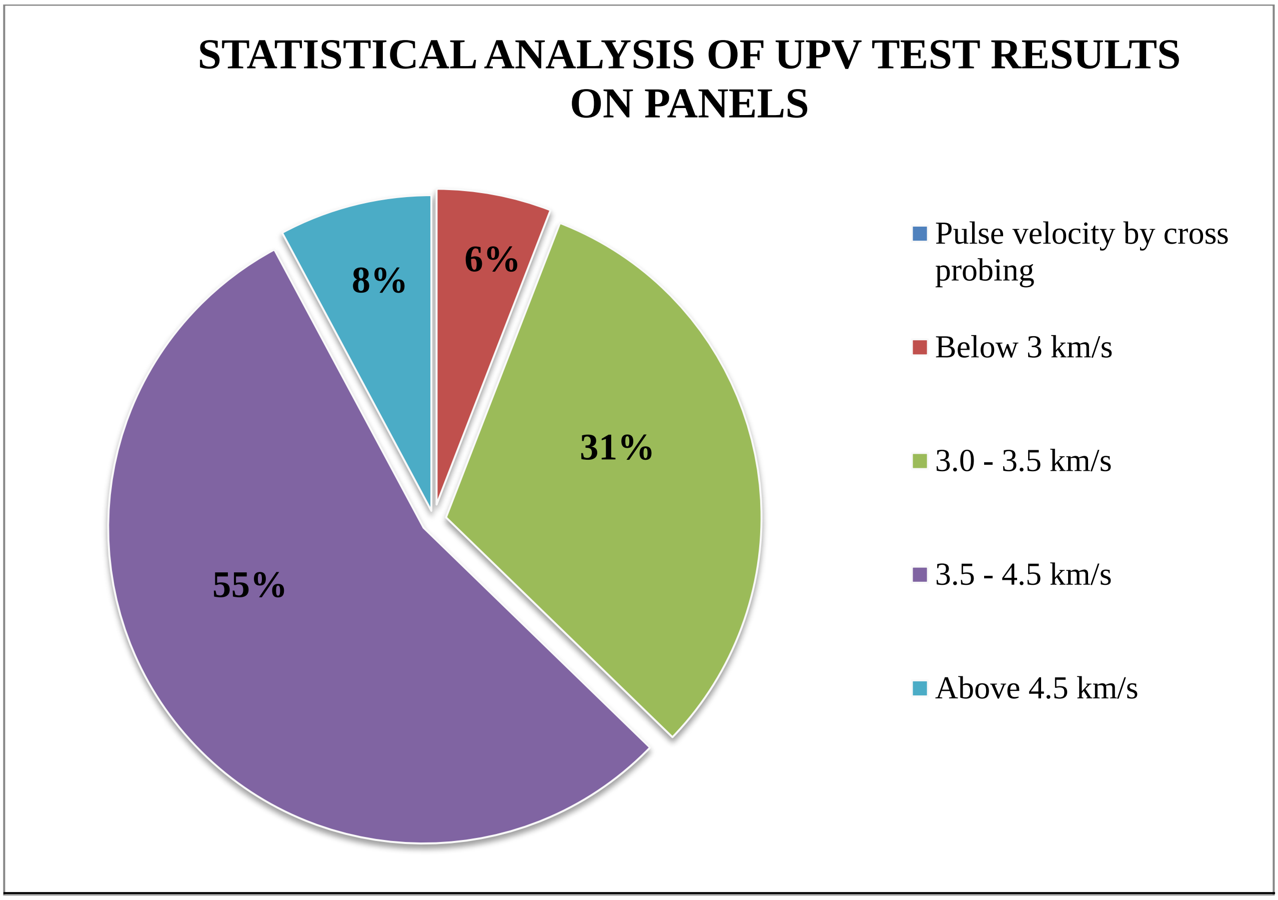


Fig. 24.Statistical Analysis of UPV test results on panel

CONCRETE QUALITY GRADING FOR PANLES		
Excellent	**-**	**6%**
Good	**-**	**55%**
Medium	**-**	**31%**
Doubtful	**-**	**8%**
Note- This is prepared on the basis of UPV test results as per IS: 13311 part 1		

4.2 REBOUND HAMMER TEST:-

Purpose:-

This test gives a measure of the surface hardness of the concrete surface. Although there is no direct relationship between this measurement of surface hardness and strength, an empirical relationship exists. Rebound hammer is the best known methods of comparing the concrete in different parts of a structure and indirectly assessing concrete strength. The rebound hammer should be considered as a means of assessing variations of strength within a structure rather than an accurate means of assessing the strength.

Objective of testing:-

Rebound hammer test is performed to determine the following:

- ✓ Surface hardness
- ✓ Uniformity of concrete over the structure
- ✓ Grade of concrete
- ✓ Estimated strength which is derived from establishing a relationship between in-situ core strength and rebound number.

References:-

- ✓ BS 6089:1981 and BS 1881:Part 202,
- ✓ IS13311(Part2):1992
- ✓ ASTM C 805-02

Influencing factors:-

Rebound hammer test results are considerably influenced by these factors:

- ✓ Size, shape and rigidity of the specimen
- ✓ Age of test specimen
- ✓ Smoothness of surface and internal moisture condition of the concrete
- ✓ Carbonation of concrete surface

Testing Method:-

According to ASTM C 805-02 clause 7.1 the concrete members to be tested shall be at least 100mm thick and fixed within a structure. Toweled surfaces generally exhibit high rebound numbers than screed or formed finishes. Do not compare the test results if the form material against which the concrete is placed is not similar.

Heavily textured, soft or surfaces with loose mortar shall be ground flat with abrasive stone. Smooth formed or toweled surfaces do not have to be ground prior to testing.

Also this test is not conducted directly over the reinforcing bars having cover less than 20mm. The surface under test should be clean and smooth because rough surfaces cannot be tested as they do not give reliable results. Dirt or other loose material on the surface can be removed using a grinding stone prior to test.

Test results analysis of the Rebound Number values is based on test results conducted over concrete surfaces. Obtained test results explain about pattern of concrete quality of whole structure sections in terms of surface hardness. So there is no indication of blistering of concrete surface as per IS 13311 (Part- 2)-1992. Estimated strength of concrete calculated from rebound hammer number is based on correlation graph between core strength v/s corresponding rebound hammer values. Rebound hammer has been carried out in all three directions horizontal, vertical down and vertical up. By using manufacturer graph, all vertical up/vertical down rebound hammer readings has been converted into the equivalent horizontal readings. Histogram plot of the Rebound Number values is based on test results conducted over concrete surfaces. Histogram plot explains about pattern of concrete quality of whole structure sections in terms of surface hardness. Rebound number helps to obtained Estimation of Strength of concrete from correlation between Rebound Hammer V/S Core Compressive strength. Estimated strength of concrete (obtained from correlation between Rebound Hammer V/S Core Compressive strength in **table & fig**) is explained in the **table**

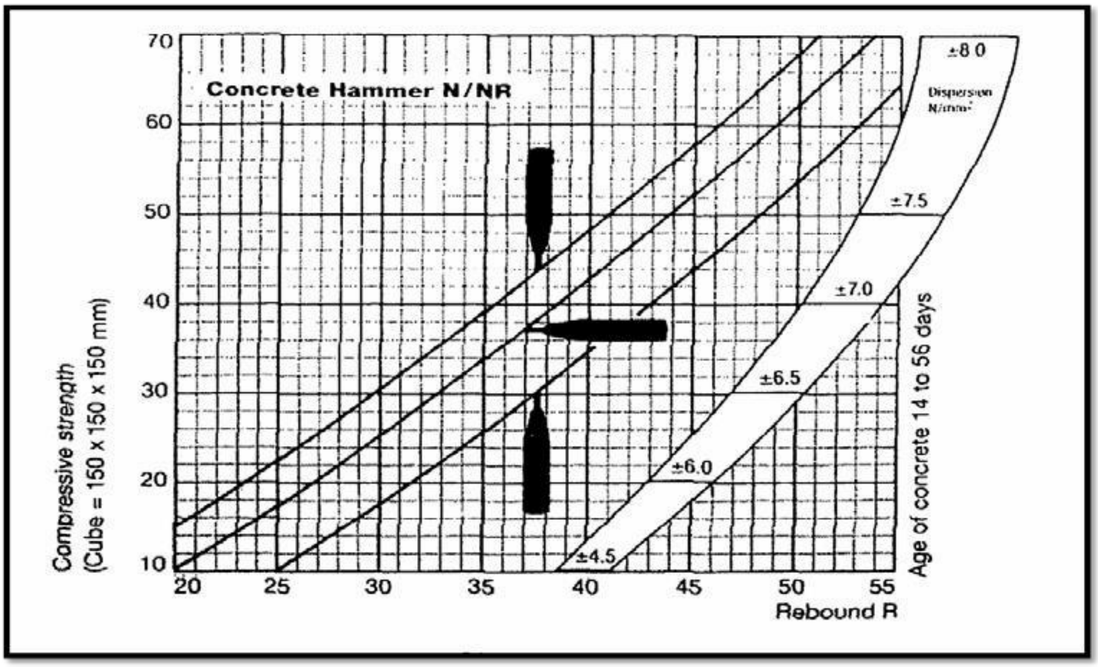


Fig. 25.The graph between the Rebound value and Core compressive strength of concrete

Rebound hammer relevant testing code:-

IS 13311 (Part 2) : 1992

भारतीय मानक

कंकरीट का अविनाशी परीक्षण—परीक्षण पद्धतियां

भाग 2 प्रतिक्षेप हथौड़ा

Indian Standard

NON-DESTRUCTIVE TESTING OF CONCRETE — METHODS OF TEST

PART 2 REBOUND HAMMER

(First Reprint JUNE 1995)

UDC 666·972 : 620·179·1

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

April 1992

Price Group 3

TEST CERTIFICATE: - REBOUND HAMMER TEST

Quality Assurance in Concrete using Non Destructive Testing

Client: PWD, DELHI **Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**

Non Destructive Testing at Nand Nagri Flyover

S. No./Location		Rebound Hammer Test									
SL. No.	Sample Identification/ Location	Hammer Alignment	Rebound No. Points						Avg. Rebound No.	Quality Of Concrete	Estimated Strength (MPa)
DECK SLAB											
1	P-9,P-10, PNL-18	Vertical Up	42	44	44	46	44	46	44	Good Layer	25
2	P-9,P-10, PNL-18	Vertical Up	54	52	52	56	54	58	54	Very Good Layer	34
3	P-9,P-10, PNL-15	Vertical Up	52	50	52	54	52	56	53	Very Good Layer	33
4	P-9,P-10, PNL-6	Vertical Up	50	52	52	54	56	52	53	Very Good Layer	33
5	P-8,P-9, PNL-27	Vertical Up	54	58	56	54	54	58	56	Very Good Layer	35
6	P-8,P-9, PNL-18	Vertical Up	44	40	40	38	40	42	41	Good Layer	22
7	P-8,P-9, PNL-3	Vertical Up	52	52	56	54	56	56	54	Very Good Layer	34
8	P-7,P-8, PNL-25	Vertical Up	58	58	52	54	52	52	54	Very Good Layer	34
9	P-7,P-8, PNL-17	Vertical Up	54	56	52	58	54	56	55	Very Good Layer	35
10	P-7,P-8, PNL-20	Vertical Up	56	58	58	56	52	60	57	Very Good Layer	36
11	P-7,P-6, PNL-	Vertical Up	42	44	44	46	42	42	43	Good Layer	25
12	P-7,P-6, PNL-13	Vertical Up	38	40	36	36	38	36	37	Fair Layer	19
13	P-7,P-8, PNL-15	Vertical Up	58	56	56	60	58	54	57	Very Good Layer	37
14	P-5,P-6, PNL-25	Vertical Up	46	46	44	42	44	46	45	Good Layer	26
15	P-5,P-6, PNL-19	Vertical Up	56	58	54	56	58	58	57	Very Good Layer	36
16	P-5,P-6, PNL-11	Vertical Up	58	58	56	58	60	56	58	Very Good Layer	37
17	P-5,P-6, PNL-4	Vertical Up	46	40	42	44	46	46	44	Good Layer	25
18	P-5,P-6, PNL-5	Vertical Up	54	58	58	52	54	50	54	Very Good Layer	34
19	P-5,P-6, PNL-2	Vertical Up	32	34	32	34	36	38	34	Fair Layer	17
20	P-11,P-12, PNL-18	Vertical Up	54	56	54	58	58	56	56	Very Good Layer	36
21	P-11,P-12, PNL-17	Vertical Up	48	50	48	46	50	52	49	Good Layer	29
22	P-11,P-12, PNL-24	Vertical Up	48	52	52	54	52	50	51	Very Good Layer	32
23	P-11,P-12, PNL-16	Vertical Up	54	58	58	56	54	58	56	Very Good Layer	36

24	P-11,P-12, PNL-13	Vertical Up	40	38	38	44	40	40	40	Fair Layer	21
25	P-11,P-12, PNL-14	Vertical Up	42	38	40	42	44	44	42	Good Layer	23
26	P-11,P-12, PNL-12	Vertical Up	42	40	40	42	44	44	42	Good Layer	23
27	P-11,P-12, PNL-5	Vertical Up	44	48	46	44	50	48	47	Good Layer	27
28	P-11,P-12, PNL-4	Vertical Up	52	56	56	54	58	56	55	Very Good Layer	35
29	P-11,P-12, PNL-6	Vertical Up	50	52	54	48	52	52	51	Very Good Layer	32
30	P-10,P-11, PNL-26	Vertical Up	50	52	52	54	54	52	52	Very Good Layer	32
31	P-10,P-11, PNL-19	Vertical Up	52	52	56	54	54	54	54	Very Good Layer	34
32	P-10,P-11, PNL-20	Vertical Up	48	48	52	52	50	50	50	Very Good Layer	30
33	P-10,P-11, PNL-9	Vertical Up	54	54	56	56	52	52	54	Very Good Layer	34
34	P-11,P-12, PNL-41	Vertical Up	54	54	56	56	56	52	55	Very Good Layer	34
35	P-11,P-12, PNL-34	Vertical Up	56	56	54	54	54	54	55	Very Good Layer	34
36	P-12,P-13, PNL-13	Vertical Up	56	58	58	56	56	58	57	Very Good Layer	37
37	P-12,P-13, PNL-12	Vertical Up	60	60	58	58	58	56	58	Very Good Layer	38
38	P-12,P-13, PNL-5	Vertical Up	58	56	56	54	54	56	56	Very Good Layer	35
39	P-12,P-13, PNL-6	Vertical Up	54	52	54	56	54	54	54	Very Good Layer	34
40	P-12,P-13, PNL-2	Vertical Up	54	56	54	58	56	58	56	Very Good Layer	36
41	P-12,P-13, PNL-9	Vertical Up	58	58	60	58	56	58	58	Very Good Layer	37
42	P-12,P-13, PNL-5	Vertical Up	58	56	58	54	52	58	56	Very Good Layer	36
43	P-12,P-13, PNL-3	Vertical Up	48	46	52	46	50	46	48	Good Layer	29
44	P-12,P-13, PNL-4	Vertical Up	50	50	52	54	56	52	52	Very Good Layer	32
45	P-12,P-13, PNL-11	Vertical Up	56	54	56	58	54	56	56	Very Good Layer	35
46	P-12,P-13, PNL-20	Vertical Up	58	60	58	54	56	58	57	Very Good Layer	37
47	P-12,P-13, PNL-27	Vertical Up	52	54	48	48	50	54	51	Very Good Layer	31
48	P-12,P-13, PNL-19	Vertical Up	50	52	54	50	52	52	52	Very Good Layer	32
49	P-12,P-13, PNL-26	Vertical Up	50	52	48	48	50	52	50	Very Good Layer	30
50	P-12,P-13, PNL-25	Vertical Up	58	56	54	54	56	56	56	Very Good Layer	35
51	P-12,P-13, PNL-24	Vertical Up	50	48	48	46	50	50	49	Good Layer	29
52	P-19,P-20, PNL-11	Vertical Up	58	60	58	54	54	52	56	Very Good Layer	36
53	P-19,P-20, PNL-10	Vertical Up	56	58	58	54	56	54	56	Very Good Layer	36
54	P-19,P-20, PNL-3	Vertical Up	50	48	46	48	50	52	49	Very Good Layer	29
55	P-19,P-20, PNL-17	Vertical Up	58	58	54	56	54	60	57	Very Good Layer	36

56	P-13,P-14, PNL-10	Vertical Up	52	52	54	56	54	52	53	Very Good Layer	33
57	P-13,P-14, PNL-11	Vertical Up	48	50	50	52	48	54	50	Very Good Layer	31
58	P-13,P-14, PNL-17	Vertical Up	50	46	48	46	52	50	49	Good Layer	29
59	P-13,P-14, PNL-18	Vertical Up	52	54	54	50	56	56	54	Very Good Layer	34
60	P-13,P-14, PNL-19	Vertical Up	54	56	52	52	50	52	53	Very Good Layer	33
61	P-13,P-14, PNL-26	Vertical Up	54	54	50	52	56	56	54	Very Good Layer	34
62	P-13,P-14, PNL-25	Vertical Up	40	42	42	40	44	38	41	Fair Layer	22
63	P-13,P-14, PNL-24	Vertical Up	54	52	54	50	54	54	53	Very Good Layer	33
64	P-14,P-15, PNL-9	Vertical Up	52	50	50	54	52	52	52	Very Good Layer	32
65	P-14,P-15, PNL-3	Vertical Up	40	40	38	42	42	40	40	Very Good Layer	22
66	P-14,P-15, PNL-12	Vertical Up	52	54	52	50	54	52	52	Very Good Layer	32
67	P-14,P-15, PNL-18	Vertical Up	54	54	56	58	56	54	55	Very Good Layer	35
68	P-15,P-16, PNL-16	Vertical Up	58	54	56	56	54	52	55	Very Good Layer	35
69	P-15,P-16, PNL-26	Vertical Up	50	50	48	48	52	50	50	Very Good Layer	30
70	P-15,P-16, PNL-25	Vertical Up	58	56	58	60	54	58	57	Very Good Layer	37
71	P-16,P-17, PNL-17	Vertical Up	58	60	60	58	56	58	58	Very Good Layer	38
72	P-16,P-17, PNL-18	Vertical Up	58	60	54	58	56	56	57	Very Good Layer	37
73	P-16,P-17, PNL-25	Vertical Up	42	44	40	40	42	40	41	Good Layer	23
74	P-17,P-18, PNL-10	Vertical Up	56	56	52	54	58	54	55	Very Good Layer	35
75	P-17,P-18, PNL-17	Vertical Up	40	42	40	42	44	40	41	Good Layer	23
76	P-17,P-18, PNL-25	Vertical Up	28	30	32	30	34	32	31	Fair	14
77	P-17,P-18, PNL-26	Vertical Up	26	30	34	36	32	30	31	Fair	14
78	P-17,P-18, PNL-18	Vertical Up	24	30	28	30	30	28	28	Fair	11
79	P-18,P-19, PNL-3	Vertical Up	38	40	44	44	42	44	42	Good Layer	23
80	P-18,P-19, PNL-11	Vertical Up	48	50	46	46	50	46	48	Good Layer	28
81	P-18,P-19, PNL-4	Vertical Up	42	38	38	40	36	38	39	Good Layer	20
82	P-18,P-19, PNL-2	Vertical Up	48	48	44	44	46	48	46	Good Layer	27
83	P-20,P-21, PNL-13	Vertical Up	48	52	50	52	48	48	50	Good Layer	30
84	P-20,P-21, PNL-27	Vertical Up	46	44	44	46	48	44	45	Good Layer	26
85	P-20,P-21, PNL-6	Vertical Up	48	46	50	48	48	46	48	Good Layer	28
86	P-20,P-21, PNL-12	Vertical Up	48	46	48	50	50	46	48	Good Layer	29

87	P-20,P-21, PNL-23	Vertical Up	50	50	48	46	48	46	48	Good Layer	29
88	P-21,P-22, PNL-18	Vertical Up	42	46	46	44	48	44	45	Good Layer	26
89	P-21,A-2, PNL-6	Vertical Up	44	48	46	48	50	54	48	Good Layer	29
90	P-22,A-2, PNL-6	Vertical Up	48	46	48	50	46	48	48	Good Layer	28
91	P-22,A-2, PNL-3	Vertical Up	48	46	48	50	46	48	48	Good Layer	28
92	P-22,A-2, PNL-23	Vertical Up	50	52	48	48	50	50	50	Very Good Layer	30
93	P-4,P-5, PNL-10	Vertical Up	42	44	44	46	48	46	45	Good Layer	26
94	P-4,P-5, PNL-5	Vertical Up	48	50	50	46	46	48	48	Good Layer	29
95	P-4,P-5, PNL-27	Vertical Up	46	48	50	48	50	52	49	Good Layer	29
96	P-3,P-4, PNL-26	Vertical Up	52	50	50	48	50	48	50	Very Good Layer	30
97	P-3,P-4, PNL-23	Vertical Up	42	44	46	46	44	48	45	Good Layer	26
98	P-3,P-4, PNL-9	Vertical Up	44	46	48	44	50	48	47	Good Layer	27
99	P-2,P-3, PNL-23	Vertical Up	48	50	50	48	46	48	48	Good Layer	29
100	P-2,P-3, PNL-9	Vertical Up	48	52	54	52	50	48	51	Very Good Layer	31
101	P-2,P-3, PNL-3	Vertical Up	50	52	52	48	48	46	49	Very Good Layer	30
102	P-1,P-2, PNL-23	Vertical Up	50	44	44	46	48	46	46	Good Layer	27
103	P-1,P-2, PNL-16	Vertical Up	46	46	48	46	44	48	46	Good Layer	27
104	P-1,P-2, PNL-11	Vertical Up	48	48	50	46	44	46	47	Good Layer	28
105	P-1,A-1, PNL-25	Vertical Up	42	38	40	38	42	40	40	Good Layer	22
106	P-1,A-1, PNL-27	Vertical Up	44	46	46	48	46	50	47	Very Good Layer	27
107	P-1,A-1, PNL-11	Vertical Up	52	50	50	48	48	50	50	Very Good Layer	30
108	P-1,A-1, PNL-12	Vertical Up	50	48	52	48	48	50	49	Very Good Layer	30
109	P-1,A-1, PNL-2	Vertical Up	46	46	48	52	50	50	49	Good Layer	29
PIER											
110	P11	Horizontal	46	48	46	46	48	50	47	Very Good Layer	39
111	P12	Horizontal	50	48	44	56	48	52	50	Very Good Layer	43
112	P13	Horizontal	48	48	50	44	48	46	47	Very Good Layer	39
113	P14	Horizontal	48	48	46	48	48	48	48	Very Good Layer	41
114	P17	Horizontal	52	48	46	44	48	46	47	Very Good Layer	39
115	P18	Horizontal	42	44	48	48	48	46	46	Very Good Layer	38
116	P20	Horizontal	46	48	50	42	48	48	47	Very Good Layer	39

117	P8	Horizontal	42	44	48	48	48	46	46	Very Good Layer	38
118	P5	Horizontal	50	48	48	46	44	48	47	Very Good Layer	39
119	P3	Horizontal	46	48	50	50	48	46	48	Very Good Layer	41
ABUTMENT											
120	A1	Horizontal	48	46	50	50	48	46	48	Very Good Layer	41
121	A2	Horizontal	44	46	48	44	48	50	47	Very Good	39
Rebound Hammer (IS: 13311 part 2):- Surface Hardness indices value should be more than 30 Rebound Numbers to get correlation with estimated strength, uniformity of concrete.											

Interpretation of RCC Surface condition, uniformity of concrete and f_{ck} value of concrete obtained from Rebound hammer:-

Test results analysis of the Rebound Number values are based on test results conducted over concrete surfaces. Obtained test results explain about pattern of concrete quality as a whole structure in terms of surface hardness.

Statistical data shows that percentage indicating the **quality of concrete is order of minimum M11 to maximum of M38, where only at very few locations found the grade was below M20. Concrete surfaces are not suffered from surface hardness problem. This indication of low grade of concrete is due to the presence of cracks, honeycombing, spalling at particular locations as observed.**

HISTOGRAM PLOT FOR AVG. REBOUND NOS.

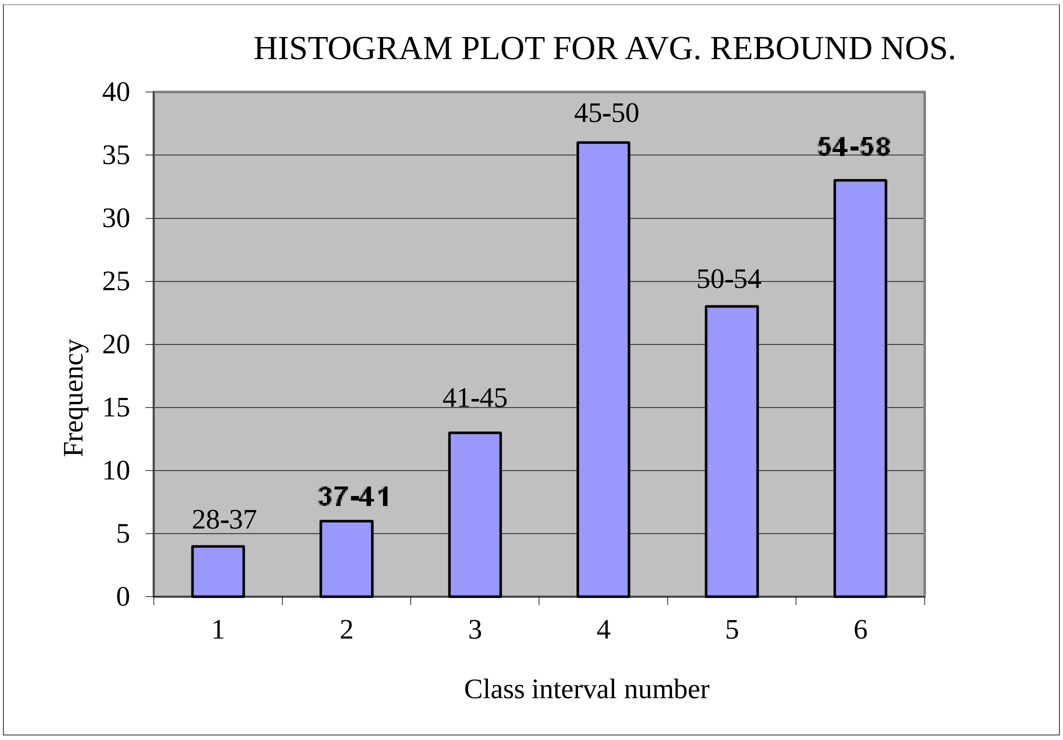


Fig. 26.Bar graph showing the relationship between class interval number and frequency

Total No. of Test Location on the panels	Min.	Max.	Mean	Mode	Median	Standard Deviation S/P
109	28	58	49.78	56	50	6.23

***Note**: Total no of test sample = Sum of frequency

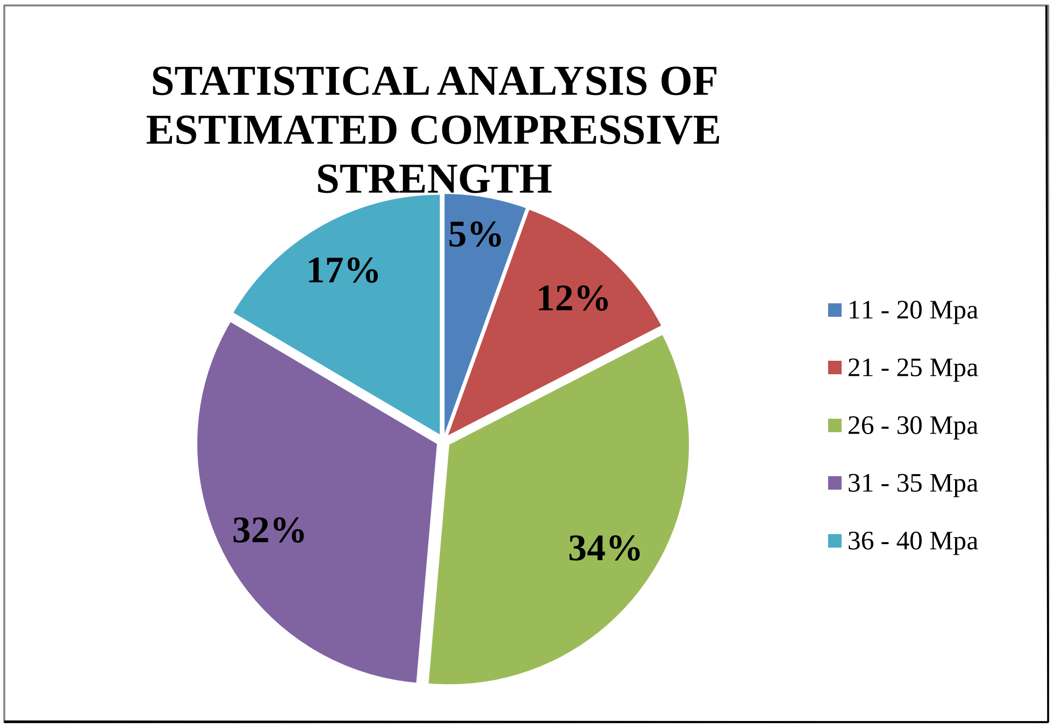


Fig. 27.Piechart depicting the estimated compressive strength

Total No. of Test Location on the panels	Min.	Max.	Mean	Mode	Median	Standard Deviation S/P
109	11	38	30.15	29	30	5.49

4.3 COVER DEPTH TEST:

Conducting cover meter test at selected locations on RCC members of the structures covered under the study to see the adequacy of concrete cover to rebar's and effect of carbonation by using Profometer to cross check whether the cover as per **IRC:112** is sufficient or not.

COVER DEPTH MESUREMENT			
Client: PWD, DELHI		**Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**	
Non Destructive Testing at NandNagri Flyover			
Sr. No.	**Sample Identification/Location**	**Cover** (mm)	**Remark**
1	P-9,P-10 (Panel-18)	40-45	Sufficient cover depth
2	P-8,P-9 (Panel-20)	42-48	Sufficient cover depth
3	P-7,P-6 (Panel-27)	40-45	Sufficient cover depth
4	P-5,P-6 (Panel-4)	40-45	Sufficient cover depth
5	P-10,P-11 (Panel-26)	40-45	Sufficient cover depth
6	P-12,P-13 (Panel-13)	40-45	Sufficient cover depth
7	P-12,P-13 (Panel-26)	42-48	Sufficient cover depth
8	P-12,P-13 (Panel-24)	40-45	Sufficient cover depth
9	P-13,P-14 (Panel-10)	40-45	Sufficient cover depth
10	P-13,P-14 (Panel-19)	40-45	Sufficient cover depth
11	P-14,P-15 (Panel-3)	40-45	Sufficient cover depth
12	P-15,P-16 (Panel-16)	40-45	Sufficient cover depth
13	P-17,P-18 (Panel-10)	40-45	Sufficient cover depth
14	P-17,P-18 (Panel-25)	40-45	Sufficient cover depth
15	P-18,P-19 (Panel-3)	40-45	Sufficient cover depth
16	P-19,P-20 (Panel-13)	40-45	Sufficient cover depth
17	P-21,A-2 (Panel-18)	40-45	Sufficient cover depth
18	P-4,P-5 (Panel-27)	44-50	Sufficient cover depth
19	P-3,P-4 (Panel-23)	40-45	Sufficient cover depth
20	P-1,P-2 (Panel-16)	40-45	Sufficient cover depth
21	P-1,A-1 (Panel-25)	40-45	Sufficient cover depth
22	P-1,A-1 (Panel-12)	40-45	Sufficient cover depth
23	P-2,P-3 (Panel-18)	45-50	Sufficient cover depth
24	P-2,P-3 (Panel-4)	42-48	Sufficient cover depth
25	P8	85-90	Sufficient cover depth
26	P11	85-90	Sufficient cover depth
27	P20	100-110	Sufficient cover depth
28	P18	80-90	Sufficient cover depth
29	A1	115-125	Sufficient cover depth
30	A2	85-95	Sufficient cover depth
The clear cover provided in panels as per the received drawings is 40mm.			

4.4 CHEMICAL TEST:

Generally cast-in chlorides are chemically bound within the cement matrix and don’t migrate through the concrete, while chlorides in-grassed are substantially free to move and diffuse through the pore solution into cement matrix and leads to corrosion in RCC. It is important to note that whether free chloride ions are leading to chloride-induced corrosion of the reinforcement or not.

Chloride (water soluble) % mass of concrete (IS: 14959 (Part 2) – 2001, B.S. 5328)

CHEMICAL TEST RESULTS							
Client: PWD, DELHI				Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.			
Quality Assurance in Concrete of NandNagri Flyover, North East Delhi							
Sr. No./Location		CHEMICAL ANALYSIS OF CONCRETE					
Sr. No.	Sample Identification	pH Value	Impression	Sulphate (%)	Impression	Chloride (%)	Impression
1	P3-P4 (Panel 9)	12	Alkaline	0.85	Low	0.034	Low
2	P10-P11 (Panel 17)	11.5	Alkaline	0.75	Low	0.042	Low
3	P11-P12 (Panel 17)	11	Alkaline	0.67	Low	0.032	Low
4	P12-P13 (Panel 19)	10.5	Alkaline	0.81	Low	0.034	Low
5	P13-P14 (Panel 9)	12.5	Alkaline	0.77	Low	0.032	Low
6	P13-P14 (Panel 19)	11.5	Alkaline	0.69	Low	0.034	Low
7	P17-P18 (Panel 19)	11	Alkaline	0.76	Low	0.023	Low
8	P21-A2 (Panel-27)	11.5	Alkaline	0.69	Low	0.040	Low
9	P13 (Pier)	12	Alkaline	0.77	Low	0.026	Low
10	A1 (Abutment)	12	Alkaline	0.73	Low	0.028	Low

Chloride % - (IS: 14959 (Part 2) – 2001, B.S. 5328) and (IS 456:2000)
Sulphate % - (IS: 4032) (IS 456:2000) – less than 4%
pH Value - (BS 4248) – Not less than 8

Test Results Interpretation:-

- PH value of concrete having Maximum of 12.5 & Minimum of 10.5.
- Chloride percent in concrete is the Low Risk level (Maximum 0.042 & Minimum 0.023)
- Sulphate percent in concrete is the Low Risk level (Maximum 0.85 & Minimum 0.67)

4.5 CARBONATION TEST:

Concrete cover layer acts as a good protective layer for the reinforcement. When whole Protective layer/cover depth is carbonated as per carbonation (B.S 4248) deterioration of structure will fall in deterioration period with linear rate. So full carbonated cover depth removal is mandatory to protect the steel bar from further corrosion and we have to increase the thickness of cover depth to protect the steel from futuristic corrosion.

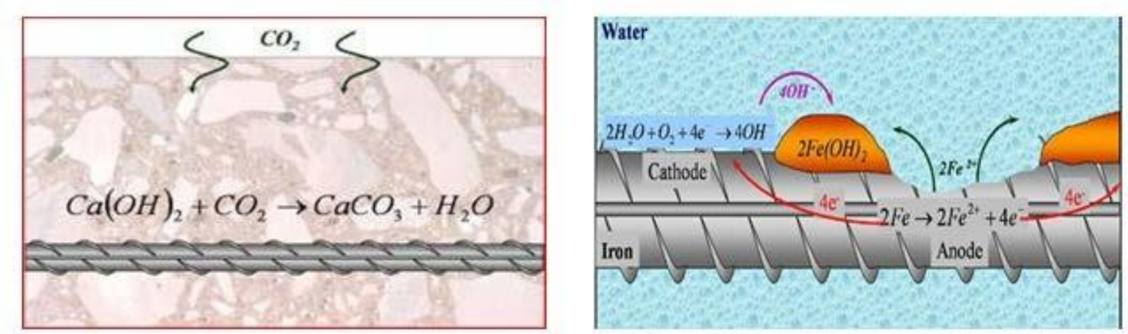


Fig .28.The image shows the entry of CO_2 and its reaction with the underlying steel

Sr. No.	Sample Identification / Location	**Carbonation Depth** (mm)	Remark
1	P3-P4 (Panel 9)	12	Less than Cover
2	P10-P11 (Panel 17)	11	Less than Cover
3	P11-P12 (Panel 17)	10	Less than Cover
4	P12-P13 (Panel 19)	16	Less than Cover
5	P13-P14 (Panel 09)	8	Less than Cover
6	P13-P14 (Panel 19)	10	Less than Cover
7	P17-P18 (Panel 19)	9	Less than Cover
8	P21-A2 (Panel-27)	10	Less than Cover
9	P 13 (Pier)	11	Less than Cover
10	A1 (Abutment)	10	Less than Cover

Quality Assurance in Concrete using Non Destructive Testing

NAND NAGRI FLYOVER

Client: PWD, DELHI **Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**

CARBONATION DEPTH

Carbonation (BS 4248): - If depth of carbonation is less than cover depth, but the concrete surface has started its carbonation; Degradation rate will be accelerated.

4.6 CONCRETE CORE EXTRACTION:

Purpose:-

This test is known as a confirmatory test to get the idea about the compressive strength of the existing concrete. Core compressive strength is the best known methods of getting the in-situ concrete compressive strength in different parts of a structure at present time and indirectly assessing the f_{ck}value of concrete.

Objective:-

- ✓ Compressive strength (Grade) of concrete
- ✓ f_{ck} value of concrete
- ✓ Estimated strength which is derived from establishing a relationship between in-situ core strength and rebound number.

References:-

- ✓ IRC 112:2011
- ✓ IS 516:1959
- ✓ IS 1199:1959
- ✓ ASTMC-42
- ✓ IS 456:2002

Influencing factors:-

- ✓ Age of test specimen
- ✓ Internal Voids/honeycombing
- ✓ Carbonation of concrete surface

Testing Procedure:-

Diameter of core size: - The general rule adopted for fixing the core size, besides the L/D ratio, is the nominal size of stone aggregate and the diameter should be not less than 3 times the maximum size of stone aggregate. **Reference ASTM C-42 article clause number 6.1 and part 4 of IS: 1199-1959.**

L/D ratio: Its value should be minimum 0.95 and maximum 2 (without capping but after trimming). Higher ratio would cause a reduction in strength. L/d of extracted core after capping should be1 < L/d <2.

Capping size should be 0.5% of core diameter. It depends upon diameter of core.

Reference code: - IS 516: 1959 Article clause number 4.3 and ASTM C-42 article clause number 6.1 and 6.5

Drilling operations: The strength of cores is generally less than that of standard cylinders, partly as a consequence of disturbance due to vibrations during drilling operations. It disturbs the micro-structure of concrete core (body centered cubic) so it affects the bonding between aggregate to aggregate (directly strength of concrete). Whatever best precautions are taken during drilling; there is always a risk of slight damage. Machine should be installed on separate platform to avoid vibration. Here testing house has used hand operated (eccentricity problem) core extraction machines (it is evident in submitted report). **Reference code: - ASTM C-42 Article clause number: - 4.1.1**

Compressive Strength of Concrete Core:-

Sample has been taken from best locations as well as from most deteriorated places to get mixed idea about the existing strength of concrete. Strength level of the concrete and the disturbance of the specimen caused by drilling operation are the most important factors influencing the strength of cores. Obtained concrete cores don't have had any sort of cracks. Concrete core was not extracted from damaged/cracked part of RCC. Minimum grade of concrete for RCC work is M20 as per the code IS456:2000. Equivalent cubic strength of concrete (IS 516:1991) obtained from core test results are explained in the table 1 (concrete core test results)

	Reference for Test results of concrete core tests
1	Site location where core was extracted.
2	Diameter of extracted core Sample
3	Length of prepared core sample after trimming & capping.
4	Ratio of prepared core length and diameter Ratio (Core Length / Core Diameter).
5	Top Surface Area of Core Sample (πr^2)
6	Failure Load obtained from CTM machine in kN.
7	Cylindrical Compressive Strength is (Failure Load / Loading Surface Area)
8	Correction factor obtained from Code (IS 516: 1959/1991) = 0.1* (L/D ratio) + (0.8)
9	Equivalent cube strength = (Cylindrical Compressive Strength) * (Correction Factor) * (1.25)
10	In-situ Strength of Concrete = Equivalent cube strength / 0.85

COMPRESSIVE STRENGTH OF CONCRETE

Quality Assurance in Concrete using Non Destructive Testing

Compressive Strength of Concrete: Concrete Core (IS516: 1959/1991)

Client: PWD, DELHI **Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**

Non Destructive Testing at NandNagri Flyover

Sr. No	**Identification mark/Serial No**	**Dia of core** (d in mm)	**Core Length** (l in mm)	**l/d ratio**	**Loading surface area** (m^2)	**Failure load** (kN)	**Cylindrical compressive strength** (MPa)	**Correction factor**	**Equivalent cube strength** (MPa)
1	P3-P4 (Panel 9)	75	89	1.19	0.00442	151.80	34.35	0.9187	39
2	P10-P11 (Panel 17)	75	92	1.23	0.00442	127.70	28.89	0.9227	33
3	P11-P12 (Panel 17)	75	83	1.11	0.00442	136.8	30.95	0.9107	35
4	P12-P13 (Panel 19)	75	88	1.17	0.00442	120.9	27.36	0.9173	31
5	P13-P14 (Panel 09)	75	76	1.01	0.00442	152.2	34.44	0.9013	39
6	P13-P14 (Panel 19)	75	110	1.47	0.00442	159.7	36.13	0.9467	43
7	P17-P18 (Panel 19)	75	85	1.13	0.00442	145.8	32.99	0.9133	38
8	P21-A2 (Panel-27)	75	94	1.25	0.00442	149.20	33.76	0.9253	39
9	P 13 (Pier)	75	93	1.24	0.00442	138.6	31.36	0.9240	36
10	A1 (Abutment)	75	88	1.17	0.00442	158.30	35.82	0.9173	41

Codal Procedure:

- Cylindrical compressive strength (MPa) = Failure load (kN)/ Loading surface area ($m^2 = \pi.d^2/4$).
- Correction factor = IS Code 516:1959 Page no 12 fig 1.
- Equivalent cube strength (MPa) = Cylindrical compressive strength * 1.25.
- Concrete construction should be considered structurally adequate if average of three cores from questionable region/sections is equal to or exceed the 85% of specified strength as per ACI 318.

Interpretation of concrete core test results: - The Equivalent cube compressive strength obtained for the cores ranges from **31 MPa to 43 MPa for panels**, **41 MPa for abutments and 36MPa for Piers.**

Average Grade of Concrete: M37 in RCC Panels.

Rebound hammer and corresponding Core strength for correlation to estimate strength of concrete

Sr. No	Sample Location	Equivalent cubic strength obtained from core test (MPa)	Correspondent rebound number at same location core has been taken
1	P3-P4 (Panel 9)	39	46
2	P10-P11 (Panel 17)	33	43
3	P11-P12 (Panel 17)	35	44
4	P12-P13 (Panel 19)	31	42
5	P13-P14 (Panel 09)	39	46
6	P13-P14 (Panel 19)	43	53
7	P17-P18 (Panel 19)	38	45
8	P21-A2 (Panel-27)	39	46
9	P 13 (Pier)	36	43
10	A1 (Abutment)	41	52

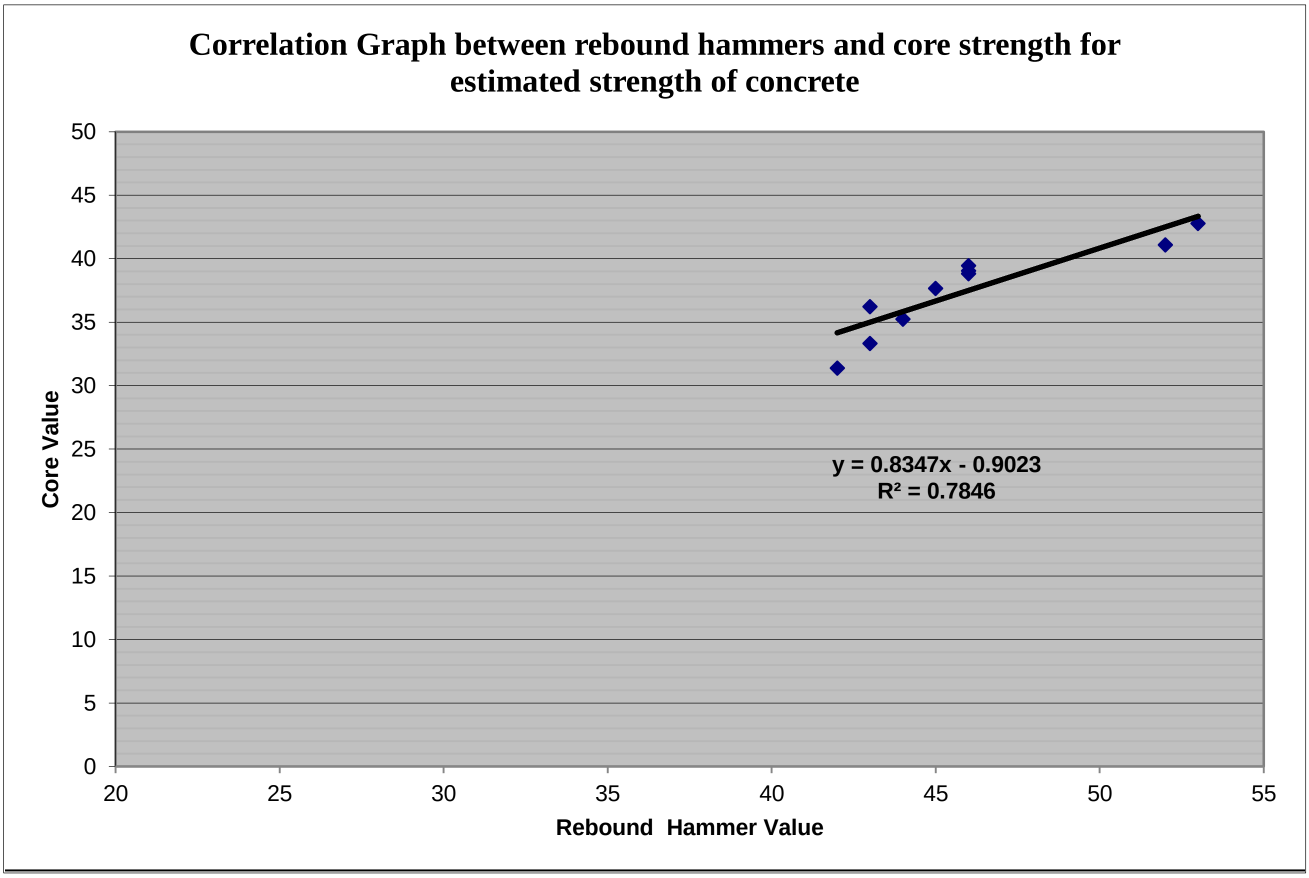


Fig 29.Graph correlating the rebound values and core strength for estimated strength of concrete

Rebound Hammer Tests were separately performed where Core Tests were executed

IS : 516 - 1959
(Reaffirmed 2004)
Edition 1.2
(1991-07)

Indian Standard
METHODS OF TESTS FOR STRENGTH OF CONCRETE

(Incorporating Amendment Nos. 1 & 2)

UDC 666.97 : 620.17

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

Price Group 6

4.7. CRACK PATTERN ANALYSIS:

Ultrasonic Sonic Pulse Velocity was used to investigate the phenomenon of crack propagation in existing cracked concrete surface. Ultrasonic was used to measure the crack depth. In this study, the path travelled time method was used to check the depth of crack. The crack depth was measured at the center of crack line along with its corner points (left & right) as shown in picture.

There are two type of crack.

The initiation and propagation of crack in concrete represent the change of self-load state. A thorough observation of the crack and readings of ultrasonic pulse velocity in concrete element helps find out the direction of crack development. There are two possibilities to be developed the crack either perpendicular or inclined on the concrete surface.

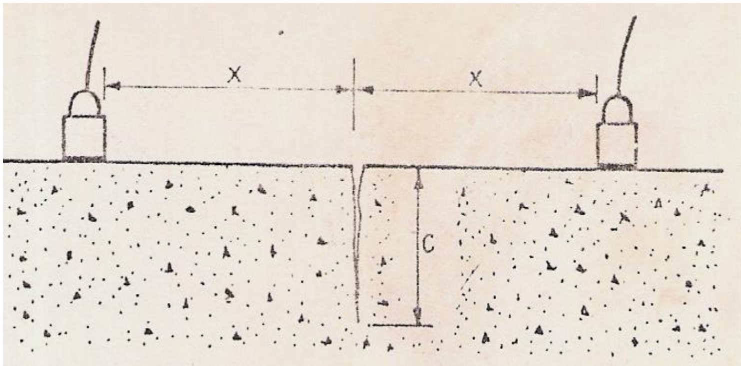


Fig .30.Transducers kept at equal intervals from the crack on either side of it

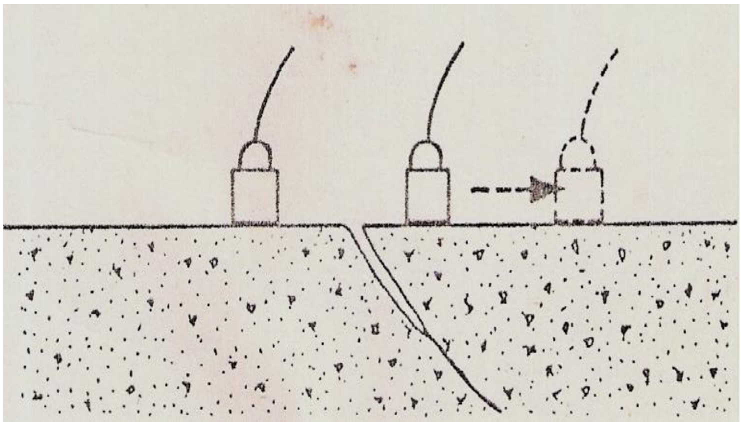


Fig 31.The change in the position of transducer

Principal of Test:

The principle of crack detection using the test system described is to time of flight techniques. A signal emitted by the transducer will be detected after a certain travel time and with a certain amplitude or energy, respectively (refer Fig.30).

We placed transducers near to the crack and on opposite side of it. We moved one of them away from the crack line (shown in fig 31). If transit time decreases this indicates that the crack slopes towards the direction in which the transducer was moved. We decrement in time was minor so we found that crack is perpendicular to its concrete surface.

For measuring perpendicular crack depth:

If a surface crack with a tip depth *C* is present between emitter and sensor,

$$\text{Crack Depth} = C = X_1\sqrt{\frac{4T_1^2 - T_2^2}{T_2^2 - T_1^2}}$$

Photo 62. UPV meter showing T1 and T2 readings

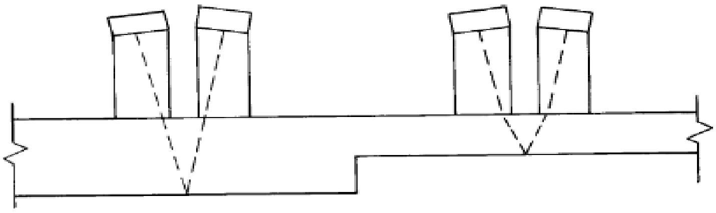
Where First value of X is chosen T1 and second value is T2 and transit times corresponding to these be T1 and T2 respectively.

TEST CERTIFICATE FOR CRACK WIDTH MEASUREMENT

Quality Assurance in Concrete using Non Destructive Testing							
Ultrasonic Pulse Velocity (IS: 13311 Part 1)							
NAND NAGRI FLYOVER							
Client: PWD, DELHI				**Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**			
Quality Assurance in Concrete of Nand Nagri Flyover, North East Delhi							
SL. No.	**Sample Identification/Location**	**X** (mm)	**T1** (µs)	**T2** (µs)	**Crack Depth** (mm)	**Crack Width** (mm)	
1	P6-P7 (Panel-13)	100	3.04	4.09	164	0.1mm	
2	P11-P12 (Panel-12)	Major Crack			No Reading	1.2 mm	
3	P12-P13 (Panel-26)	100	117.9	199.70	78	0.3mm	
4	P12-P13 (Panel-24)	100	149.7	179.70	241	1.1mm	
5	P16-P17 (Panel-25)	100	99.7	198.00	14	0.1mm	
6	P17-P18 (Panel-17)	100	85.6	132.20	108	0.5mm	
7	P20-P21 (Panel-13)	100	113.4	212.00	45	0.3mm	
8	P13-P14 (Panel-25)	100	71.4	134.00	44	0.1mm	
Test Results Interpretation (IS: 13311 Part 1):- Major & Minor cracks observed in panels of deck slab.							

4.8 THICKNESS MEASUREMENT:

Thickness measurement used to check steel thickness reduction due to corrosion as per ASTM E797/E 797M by compare actual thickness with thickness reading.



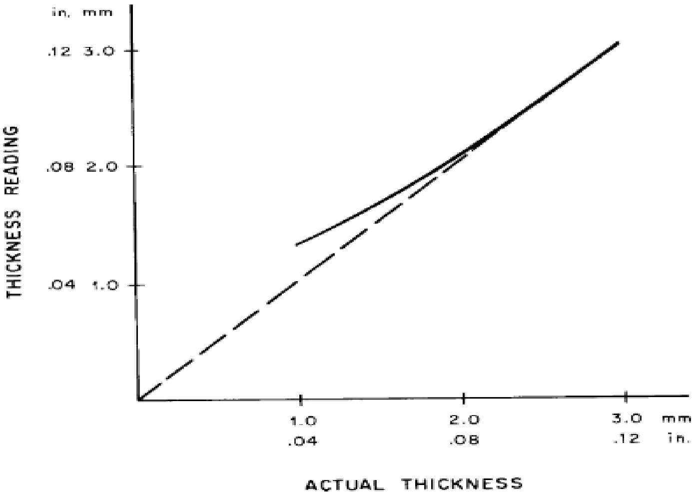


Fig .32Graph showing the relation between actual thickness and thickness reading

TEST CERTIFICATE THICKNESS MEASUREMENT

Quality Assurance in Steel Members using Non Destructive Testing				
Thickness Measurement (ASTM E797/E 797M)				
Nand Nagri Flyover				
Client: PWD, DELHI		**Consultant: - CONSTRUMA CONSULTANCY PVT. LTD.**		
SL. No.	**Sample Identification/Location**	**Actual Thickness** (mm)	**Thickness Reading** (mm)	**Remarks**
1	P21-A2	16	16	Vertical Plate
2	P21-A2	28.5	28.5	Horizontal Plate
3	P1-A1	16	16.1	Vertical Plate
4	P1-A1	28.5	28.6	Horizontal Plate
5	P10-P11	16	16.2	Vertical Plate
6	P12-P13	28.5	28.7	Horizontal Plate
Test Results Interpretation: - Steel girders are in good condition no corrosion/thickness reduction observed.				

Designation: E 797 – 95

An American National Standard

Standard Practice for Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method[1]

This standard is issued under the fixed designation E 797; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This practice[2] provides guidelines for measuring the thickness of materials using the contact pulse-echo method at temperatures not to exceed 200°F (93°C).

1.2 This practice is applicable to any material in which ultrasonic waves will propagate at a constant velocity throughout the part, and from which back reflections can be obtained and resolved.

1.3 The values stated in either inch-pound or SI units are to be regarded as the standard. The values given in parentheses are for information only.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 317 Practice for Evaluating Performance Characteristics of Ultrasonic Pulse-Echo Testing Systems Without the Use of Electronic Measurement Instruments[3]

E 494 Practice for Measuring Ultrasonic Velocity in Materials[3]

E 1316 Terminology for Nondestructive Examinations[3]

2.2 *ASNT Document:*

Nondestructive Testing Handbook, 2nd Edition, Vol 7[4]

3. Terminology

3.1 *Definitions*—For definitions of terms used in this practice, refer to Terminology E 1316.

4. Summary of Practice

4.1 Thickness (*T*), when measured by the pulse-echo ultrasonic method, is a product of the velocity of sound in the material and one half the transit time (round trip) through the material.

$$T = \frac{Vt}{2}$$

where:

T = thickness,
V = velocity, and
t = transit time.

4.2 The pulse-echo ultrasonic instrument measures the transit time of the ultrasonic pulse through the part.

4.3 The velocity in the material under test is a function of the physical properties of the material. It is usually assumed to be a constant for a given class of materials. Its approximate value can be obtained from Table X3.1 in Practice E 494 or from the *Nondestructive Testing Handbook*, or it can be determined empirically.

4.4 One or more reference blocks are required having known velocity, or of the same material to be tested, and having thicknesses accurately measured and in the range of thicknesses to be measured. It is generally desirable that the thicknesses be "round numbers" rather than miscellaneous odd values. One block should have a thickness value near the maximum of the range of interest and another block near the minimum thickness.

4.5 The display element (CRT (cathode ray tube), meter, or digital display) of the instrument must be adjusted to present convenient values of thickness dependent on the range being used. The control for this function may have different names on different instruments, including *range, sweep, material calibrate*, or *velocity*.

4.6 The timing circuits in different instruments use various conversion schemes. A common method is the so-called time/analog conversion in which the time measured by the instrument is converted into a proportional dc voltage which is then applied to the readout device. Another technique uses a very high-frequency oscillator that is modulated or gated by the appropriate echo indications, the output being used either directly to suitable digital readouts or converted to a voltage for other presentation. A relationship of transit time versus thickness is shown graphically in Fig. 1.

5. Significance and Use

5.1 The techniques described provide indirect measurement

[1] This practice is under the jurisdiction of ASTM Committee E-7 on Nondestructive Testing and is the direct responsibility of Subcommittee E07.06 on Ultrasonic Testing Procedure.

Current edition approved Dec. 10, 1995. Published February 1996. Originally published as E 797 – 81. Last previous edition E 797 – 94.

[2] For ASME Boiler and Pressure Vessel Code applications, see related Practice SE-797 in Section II of that Code.

[3] *Annual Book of ASTM Standards*, Vol 03.03.

[4] Available from the American Society for Nondestructive Testing, 1711 Arlingate Plaza, Columbus, OH 43228.

SUMMARY REPORT BASED ON NON DESTRUCTIVE TESTING OF NAND NAGRI FLYOVER, DELHI

Name of Test	Limit value/Parameters as per given Standards/codes	Results
Rebound Hammer IS 13311 (Part-2)-1992	Average rebound numbers indicating Quality of concrete • Above 40 Nos. - Very good layer • Between 30 and 40 Nos. - Good layer • Between 20 to 30 Nos. - Fair • Less than 20 Nos. - Poor concrete • Less than 10 Nos. – Delaminated.	Good & Very Good Layer of Concrete
USPV **IS 13311 (Part- 1)-1992**	USPV by Cross Probing (km/sec) • Above 4.5 – Excellent • 3.5 - 4.5 – Good • 3.0 - 3.5 – Medium • Below 3.0 – Doubtful	• **Medium to Good** at Pier • **Good** at Abutment • **Good & Excellent** at most of the Panels. • **Doubtful** at selective panels.
Concrete Core Test **(IS:516-1959,** **IRC112:2011)**	Minimum grade of concrete for RCC work is M20 as per the code IRC112:2011.	Grade of concrete is **M37**
Chemical Test	• Chloride % - (IS: 14959 (Part 2) – 2001, B.S. 5328). • Sulphate % - (IS: 4032). • pH Value - (BS 4248).	Chloride- Low risk level Sulphate- Low risk level pH Value is in permissible limit
Cover Depth Measurement	Minimum required cover depth for Pier:50 mm, Deck slab:40 mm IRC:112	Sufficient Cover at piers & deck slab
Carbonation **BS 4248** **Using Phenolphthalein Indicator**	If depth of carbonation is greater than cover, then incubation period is over; degradation rate will be accelerated.	Carbonation Less than cover at Piers & Deck Slab
Thickness Test **ASTM E797/E 797M**	If thickness reading of steel members is less than actual thickness, then steel thickness reduction due to corrosion	No Thickness Reduction Observed

5. CONCLUSION

5. CONCLUSION:-

1. The structure is an integrated ROB and RUB however the current scope of work is limited to ROB only.
2. Based upon the preliminary observation on photos taken, it is clear that **distress is only restricted to precast panels** and all other portions of flyover substructure and superstructure including piers, piercaps, pedestals, bearings, I girders, stitch concrete, crash barrier etc. **are in good condition**.
3. Detailed panel by panel assessment based upon methodology outlined in our previous report will be followed in order to reach decision about panels**- retain as is/ partially repair/ remove concrete and recast** with micro concrete.
4. In this report, the panels for conducting NDT's are selected based on condition status established on basis of Visual Inspection + Hammer Rap Survey.
5. **Piers:** No major distress on the pier such as structural cracking or major spalling or corrosion of reinforcement is observed. *(Refer Photo No.35 to 38)*
6. **Piercaps:** In very few piercaps, some minor spalling of concrete and honeycombs (minor) is noticed. They are not a cause for alarm and are easily repaired by local patch work. *(Refer Photo No.35 to 38)*
7. **Pedestals:** Pedestals are in excellent condition with no distress. *(Refer Photo No.35 to 38)*
8. **Bearings**: Bearings are in excellent condition showing no sign of any corrosion or any undue loss of alignment/ settlement. *(Refer Photo No.39 to 42)*
9. **Seismic Restrainers:** Longitudinal and horizontal seismic restrainers appeared to be ok but the conditions of the elastomeric pads are suspect and may require to be replaced. This could not be easily ascertained due to low gap; however provision for same must be made. *(Refer Photo No.35,37,41and 42)*
10. **Steel Girders**: The steel girders appeared to be in excellent condition with no undue changes in profile and alignment due to live load over time or as an initial construction defect. They are functioning as designed and installed. All stiffeners, bracings and diaphragms are in excellent condition. Nowhere is any loss of cross section noticed in steel girder, stiffeners, bracings and diaphragms i.e. their current day load carrying capacity of steel grillage is as per original design with no reduction due to any thickness loss on account of corrosion. *(Refer Photo No.21 to 24)*
11. **Paint:** However, epoxy paint applied is noticed to be failing at various locations. Paint is bubbled or peeled away or its dry film thickness is greatly reduced in localized location throughout the entire superstructure.

    It is recommended to remove existing paint and reapply as per original specification at time of construction as paint life is approaching nearly 10 years and cannot be expect to perform as it is already showing initial signs of failing at locations.

12. **Deck Drainage** – The drainage openings at deck level are chocked up and these have to be air blown, water flushed with high jet and opened up. At location where bitumen has also moved into the opening it will have to be removed. Drainage pipes and down takes appeared to be in good condition but these will need to be verified in monsoon for any leakage at joint locations.

    However at many locations the horizontal connecting pipe is missing. The water spouts at the end are not connected with the main drainage pipe which leads to accumulation of water/dampness on the end steel girders where corrosion can be accelerated in future. *(Refer Photo No.47 and 47.a)*
13. **Deck Condition:** Deck has been inspected from above and below. At 4 locations, bitumen surface is highly cracked up, depressed and subsided indicating distress in panel below. *(Refer Photo No.48 and 49 and also refer photos in our earlier report Inception Report on NandNagri Flyover – Delhi, October 2018).*
14. **Expansion Joints:** There are 4 span, 3 span and 2 span modules with expansion joints at the ends. Many are completely filled with dust and dirt and as such all will have to cleaned and removed to prevent any future distress due to locking up of expansion with clogged materials leading to restraint stresses and cracking around expansion joint. *(Refer Photo No.33 and 34).*
15. **Steel Staircases:** They are in steel connected to additional girders and paint has failed at various locations. Rust needs to be removed, some members may require additional welding of plates due to high corrosion. Entire staircase members will need to be full anticorrosive treatment. Girders of superstructure to support footpath are in good condition except for paint failure. They need to be repainted. *(Refer Photo No.18 to 20)*
16. **Deck furniture:** The deck furniture such as the medians, kerb stones, metallic crash barrier are largely in good condition without any distress or damage. *(Refer Photo No.25 and 26, 43 to 46)*
17. **R.E. Wall:** The R.E. wall and the R.C. wall along with approaches on either sides are in good condition without any sign of distress or damage.
18. **Bearing Strip: Major defect is seen in polystyrene pads provided as bearings for precast panels' *(Refer Photo No.11 to 17)*. It is clear that it is also used to make up camber and cross fall difference between top flange of steel girder and bottom flat surface of precast panel.**

    This is a fundamental defect since polystyrene has bulged along the length or come out and at places and there is clear separation/gap seen between precast panels and top flange of steel girder with loss of contact or imminent loss of contact.

    Three specialist literatures referred to are:

    1) ***Repair of Construction- Related Deterioration in Precast Deck-Panel Bridges,*** Atiq H. Alvi, Ivan Gualtero, Rajan Sen and Gray Mullins. Transportation Research Record: Journal of the Transportation Research Board, No. 2292. DOI: 10.3141/2292-13. (Refer Annexure C)
    2) ***Replacement Prioritization of Precast Deck Panel Bridges (Final Report)***, Rajan Sen, Gray Mullins, Ivan Gualtero and A. Ayoub. Florida Department of Transportation, March 2005

3) ***Monthly Deck Panel Inspection Reports,*** Florida Department of Transportation, District 7. Tallahassee, 2003-2011.

In the journal 1), page 108, under the ‘Grout Packing’ heading, *as a result of the effects of creep and shrinkage, initial separation and longitudinal cracks are inherent in precast deck-panel construction. The most important conclusion drawn from forensic study of journal 2) was that the lack of positive panel bearing was clearly the main factor responsible for the occurrence of major deck deterioration, such as cracking, delamination, spalling, failing repairs, and, in worst case, localized punch- through deck failures.*

The fiberboard bearing material is replaced with Non-shrink Portland Cement grout or Epoxy grout to provide positive bearing (refer fig 6 of page 109 in journal 1)). *Grout packing is more cost-effective than other, more involved repair methods and causes little to no disruption to traffic.*

This method is developed to address the initial construction error. To be effective, however grout packing must be applied to a bridge before spalls and failing repairs causes it to deteriorate. In 2000, the Florida DOT performed grout packing repairs on six bridges on I-75 in the Tampa area. Eleven years later, those bridges still performed satisfactory refer journal 3).

This systemic defect occurs at various locations leading to distress in the concrete panels. The distress already noticed is bound to increase over time and new panels will show distress since there is a loss of contact in bearing and repeated loading and unloading under live load will give rise to tension and shear in the panels due to loss/reduction in support in direction of traffic.

This defect is to be dealt with by either grouting with non-shrink grout at where polystyrene pad has failed / come out / absent by systematically replacing entire length of polystyrene pad with non-shrink grout.

19. The transverse distribution of load assumed in the design and also longitudinal distribution of wheel loads will not be transferred across poor quality of concrete of distressed precast panels in the load path. This has direct implication on flyover safety and **heavy vehicles must be restricted from use of flyover until the superstructure panels are repaired/ reinstated** – partially or completely.
20. **During execution of repairs, complete wearing coat will have to be removed**, so that concrete can be observed panel by panel from top and decision taken on the category of intervention to be implemented following by use of water proofing membrane/ deck slab proofing prior to reinstatement of bituminous wearing coat.

Based on NDT results:

21. **Precast Panels:** Equivalent cube compressive strength obtained at Panels is in the ranges of 31 MPa to 43 MPa. Average compressive strength is 37 MPa i.e. M37. The core samples extraction are not carried out in the damaged 10 panels since distress noticed in the Visual Inspection and Hammer Rap Survey.
22. Test results analysis of the Rebound Number values is based on test conducted over concrete surfaces. Obtained test results explain about pattern of concrete quality of whole structure sections in terms of surface hardness. As per statistical data the value of Rebound number varies from 28 to 58. Concrete surfaces are started suffering from surface hardness problem. However indication of cracks, Honeycombing & spalling is observed on concrete surface.
23. Histogram of USPV test results is analyzed and found varying concrete quality in terms of density. As per test conducted on different locations the quality of concrete is found medium, doubtful, good & excellent at Deck slab due to presence of air-pockets, Cracks, Spalling, Honeycombing and voids.
24. Based on Velocity variation graph & Crack pattern test, Major and Minor cracks found at different location on deck slab.
25. Carbonation depth is less than cover at Deck slab but carbonation is started in the surface of the panel upto the depth of 16mm (maximum). Hence possibility of corrosion to reinforcement is low but the rate will be accelerated in the future.
26. The pH value of panels having 12.5 as maximum and 10.5 as minimum at one location showing the carbonation on the surface concrete will hardens and reduce the alkaline nature of the concrete.
27. The Sulphate and Chloride percentages present in the panels are in safe limit as per the codal specifications.

Based on the Visual Inspection and the NDT test results, the following panels are condemned and recommend to be recast completely.

Panels for Recasting

S.No.	Pier	Panel No.	Visual Inspection	Hammer Rap Survey	Ultrasonic Pulse Velocity Test	Rebound Hammer Test	Crack Width / Crack Depth (mm)
1.	P5 – P6	3	Major and Minor cracks	Dull Hollow Sound	-	-	-
2.	P5 – P6	4	Major and Minor cracks	Dull Hollow Sound	Good	Good Layer	-
3.	P11 – P12	13	Spalling, Reinforcement exposure and Major Cracks	Dull Hollow Sound	Marching UPV (**High** variation in velocity and major and minor cracks with honeycombs)	Good Layer	-
4.	P12 – P13	17	Major Cracks	Dull Hollow Sound	-	-	-

5.	P12 – P13	24 (PWD recast)	Major Cracks	Dull Hollow Sound	Marching UPV (**Medium** variation in velocity and major and minor cracks with honeycombs)	Good Layer	1.1 / 241
6.	P13 – P14	25	Major Cracks and Honeycombs	Dull Hollow Sound	Marching UPV (**Medium** variation in velocity and major and minor cracks with honeycombs)	Good Layer	0.1 / 44
7.	P17 – P18	6	Minor and Major cracks	Dull Hollow Sound	-	-	-
8.	P17 – P18	17	Major cracks, Honeycombs and spalling	Dull Hollow Sound	Medium	Good Layer	0.5 / 108
9.	P17 – P18	18	Major cracks, Honeycombs and spalling	Dull Hollow Sound	Doubtful	Fair	-
10.	P17 – P18	24	Major cracks, Honeycombs and spalling	Dull Hollow Sound	-	-	-

Following table shows the panels to be repaired:

Panels to be Repaired				
Sr.No	**Pier No.**	**Panel No.**	**Panel Condition**	**Based On**
1.	A1-P1	5	Spalling and Reinforcement Exposed	Visual Inspection
		18	Spalling of concrete	Visual Inspection
		25	Spalling of concrete	Visual Inspection
2.	P1-P2	2	Spalling of concrete	Visual Inspection
		23	Spalling of concrete	Visual Inspection
3.	P2-P3	11	Spalling of concrete	Visual Inspection
		18	Minor Cracks and Honeycombs	Hammer Rap Survey
4.	P3-P4		No issues	
5.	P4-P5	11	Spalling of concrete	Visual Inspection
		12	Spalling of concrete	Visual Inspection
		27	Spalling of concrete	Visual Inspection
6.	P5-P6	10	Honeycombs	Visual Inspection
		2	Doubtful	USPV Results
			Estimated Strength – 19 MPa	Rebound Hammer Test
		25	Minor Cracks and Honeycombs	Hammer Rap Survey

7.	P6-P7	1	Major cracks and Honeycombs	Visual Inspection
		13	Spalling of concrete	Visual Inspection
			Estimated Strength – 17 MPa	Rebound Hammer Test
		18	Spalling of concrete	Visual Inspection
		25	Spalling of concrete	Visual Inspection
		26	Minor Cracks and Honeycombs	Hammer Rap Survey
8.	P7-P8		No issues	
9.	P8-P9	13	Spalling of concrete	Visual Inspection
10.	P9-P10	7	Vegetation Growth and Spalling of concrete	Visual Inspection
		26	Minor cracks, Honeycombs and Spalling of concrete	Visual Inspection
		19	Minor Cracks and Honeycombs	Hammer Rap Survey
11.	P10-P11	7	Spalling of concrete	Visual Inspection
		13	Spalling of concrete	Visual Inspection
		16	Minor Cracks and Spalling of concrete	Visual Inspection
		19	Minor Cracks and Honeycombs	Hammer Rap Survey
12.	P11-P12	4	Spalling of concrete	Visual Inspection
		18	Minor Cracks	Visual Inspection
		5	Major Cracks	Hammer Rap Survey
		12	Major Cracks	Hammer Rap Survey
			Major cracks	Crack Width Measurement
		19	Minor Cracks	Hammer Rap Survey
13.	P12-P13	3	Major Cracks and Spalling of concrete	Visual Inspection
		8	Spalling of concrete	Visual Inspection
		10	Major and Minor cracks	Visual Inspection
		15	Spalling of concrete	Visual Inspection
		19	Major and Minor cracks	Visual Inspection
		21	Spalling of concrete	Visual Inspection
		26	Major and Minor cracks	Visual Inspection
		24	Recast Panel	Crack Width Measurement
14.	P13-P14	26	Minor Cracks and Honeycombs	Hammer Rap Survey
15.	P14-P15	3	Major and Minor cracks on surface	Marching USPV
16.	P15-P16	3	Spalling of concrete	Visual Inspection

17.	P16-P17	3	Spalling of concrete	Visual Inspection
		25	Minor Cracks and Honeycombs	Visual Inspection
			Doubtful	USPV Results
18.	P17-P18	4	Reinforcement Exposed and Spalling of concrete	Visual Inspection
		16	Major Cracks	Hammer Rap Survey
		17	Recast Panel	Crack Width Measurement
		18 (recast panel)	Doubtful	USPV Results
			Estimated Strength – 11 MPa	Rebound Hammer Test
		25	Minor Cracks and Honeycombs	Hammer Rap Survey
			Doubtful	USPV Results
			Estimated Strength – 14 MPa	Rebound Hammer Test
		26	Estimated Strength – 14 MPa	Rebound Hammer Test
19.	P18-P19	4	Spalling of concrete	Visual Inspection
20.	P19-P20	3	Minor Cracks	Hammer Rap Survey
			Doubtful	USPV Results
21.	P20-P21	3	Minor Cracks and Honeycombs	Hammer Rap Survey
22.	P21-A2	21	Spalling of concrete	Visual Inspection
		28	Minor Cracks	Visual Inspection

28. **Piers and Abutments:** The USPV results show Good results at many locations and Medium at few locations whereas the rebound number on the same locations resulted maximum of 50.
29. The cover of 10 to 11 mm was carbonated and the chemical result, the pH value of 12 is noticed. The sulphate and chloride content in percentage is under the permissible limit.
30. On core test, the equivalent cube strength of 36MPa and 41MPa is noticed on the pier and abutment.
31. Based on the conducted NDTs, Piers and Abutments are in good conditions.
32. **Steel Girder:** No sign of corrosion or reduction in the thickness is noticed in the Thickness measurement test.

6. REMEDIAL MESAURES

6. REMEDIAL MEASURES:-

On the basis of detailed Visual Inspection, Hammer Rap Survey and Non-Destructive Tests carried out at various locations on R.C.C. piers, piercaps, abutments, soffit of deck slab precast panel, Steel girders, deck furniture, Steel staircases, bearings, pedestals, approaches etc. we recommend the following remedial measures to the structure.

S.No	Structure	Remedial Measures
1.	Piers	No remedial measures required
2.	Piercaps	Minor patch works repairs on certain locations
3.	Abutments	On abutments A2, patch work repairs required on the exposed steel area.
4.	Pedestals	No remedial measures required
5.	Bearings	No remedial measures required
6.	Seismic Restrainers	Conditions of the elastomeric pads are suspect and require to be replaced
7.	Steel Girders with bracing and diaphragms	No remedial measures required
8.	Bearing Strip (polystyrene pad)	Replace all polystyrene pads by Non-Shrink Grout. Construction records to be verified for the polystyrene bearing and their mechanical and material properties at the time of initial construction. There may be quality control and manufacturing issues.
9.	Panels	• **The Methodology for full panel recast** of 4 panels, (now 10 panels identified) was submitted in our previous report (refer Methodology for recasting of 4 panels on NandNagri Flyover – Delhi on OCTOBER 2018) **Methodology for panels requiring localized repairs**: • Anti-corrosive treatment on exposed reinforced surface along with epoxy mortar and/or polymer modified mortar on spalled and honeycombed portion. • The cracks on the panels should be sealed by injecting low viscosity epoxy resin by nozzle injection. • Anti-carbonation paint should be applied on the exposed concrete surface.
10.	Deck Furniture	• Proper Drainage Pipes should be provided where missing.

		• No remedial measures are required on kerb, median and crash barrier.
11.	Steel Staircase	• Rust to be removed by grinding down. • Localized plate portion shall be welded to make up loss of thickness. • Full anti-corrosive treatment and epoxy repainting shall be done.
12.	Expansion Joint	Accumulated dust and dirt to be removed for unrestrained movement of expansion joint
13.	Bituminous Wearing Coat	Wearing coat is cracked up and failed or subsided on localized locations, patch work repairs of bituminous surface to be carried out. Since waterproofing membrane initially applied at the time of construction is damaged, waterproofing membrane/ low viscosity high molecular weight thermoset polymer must be applied locally before reinstating the bitumen. Bitumen surface to be applied compatible with the existing. Seal coat on top of bituminous area to be reapplied over local area.
14.	Approaches and RE wall	No remedial measures required
15.	Paint	Remove existing paint and reapply as per original specification at the time of construction.

7. ANNEXURES

A. MARKED TEST LOCATIONS

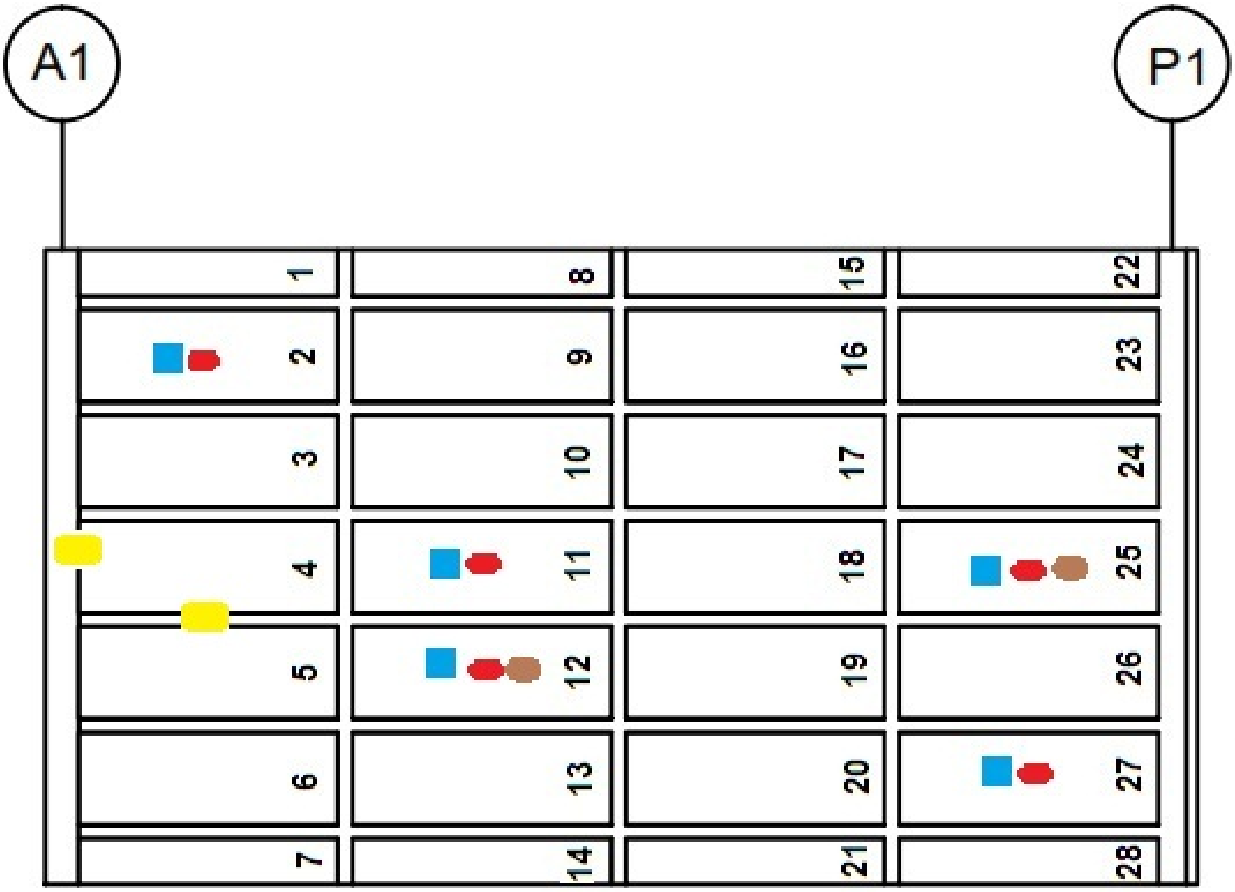


Fig. 33.Tests conducted at various locations between Abutment A1 and Pier P1

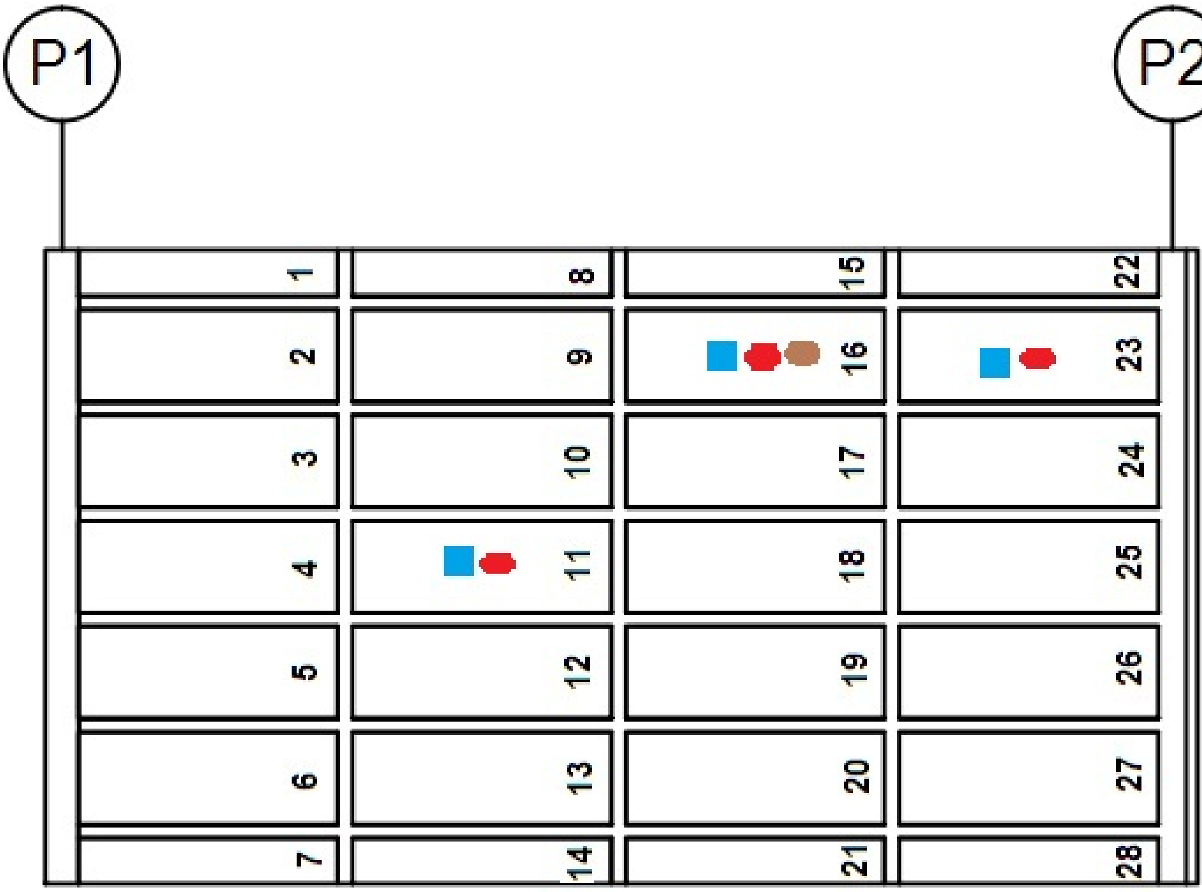


Fig. 34.Test conducted at various locations between Pier P1 and Pier P2

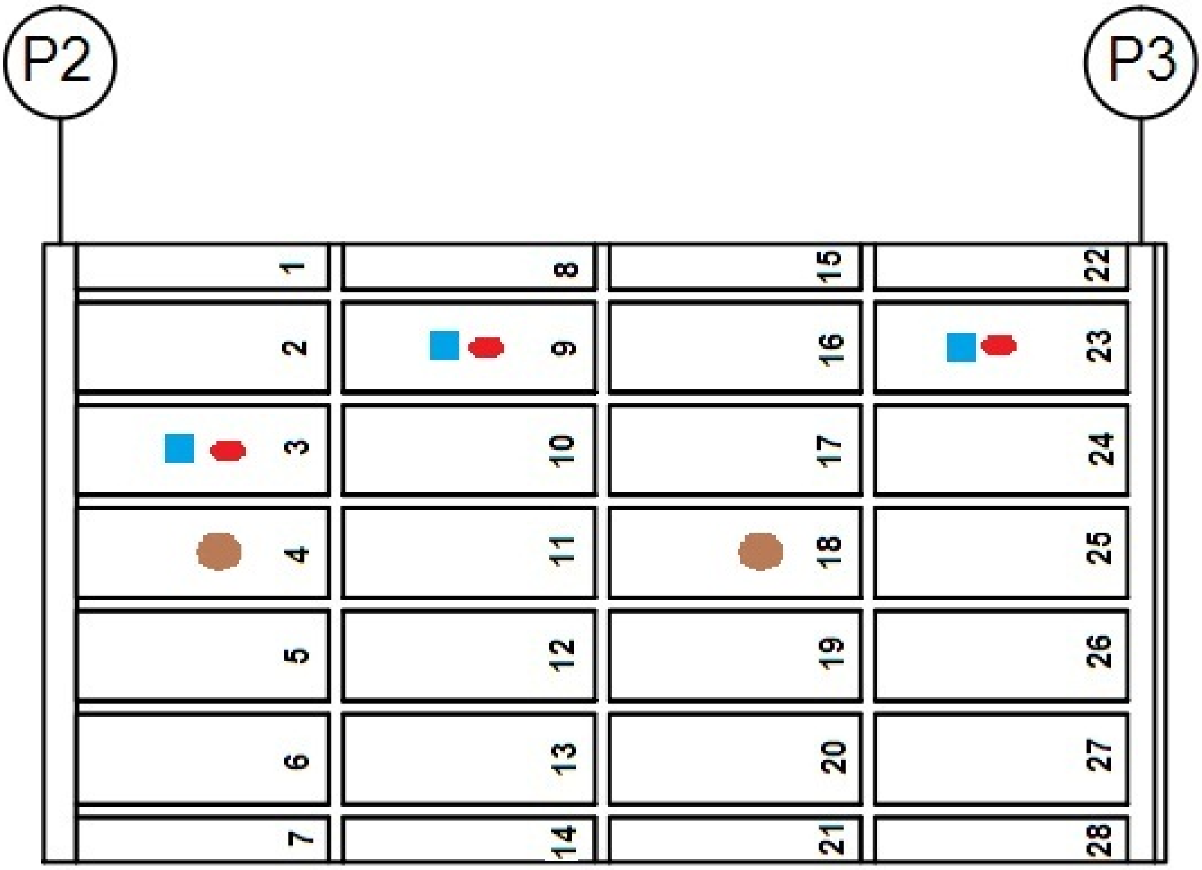


Fig. 35.Test conducted at various locations between Pier P2 and Pier P3

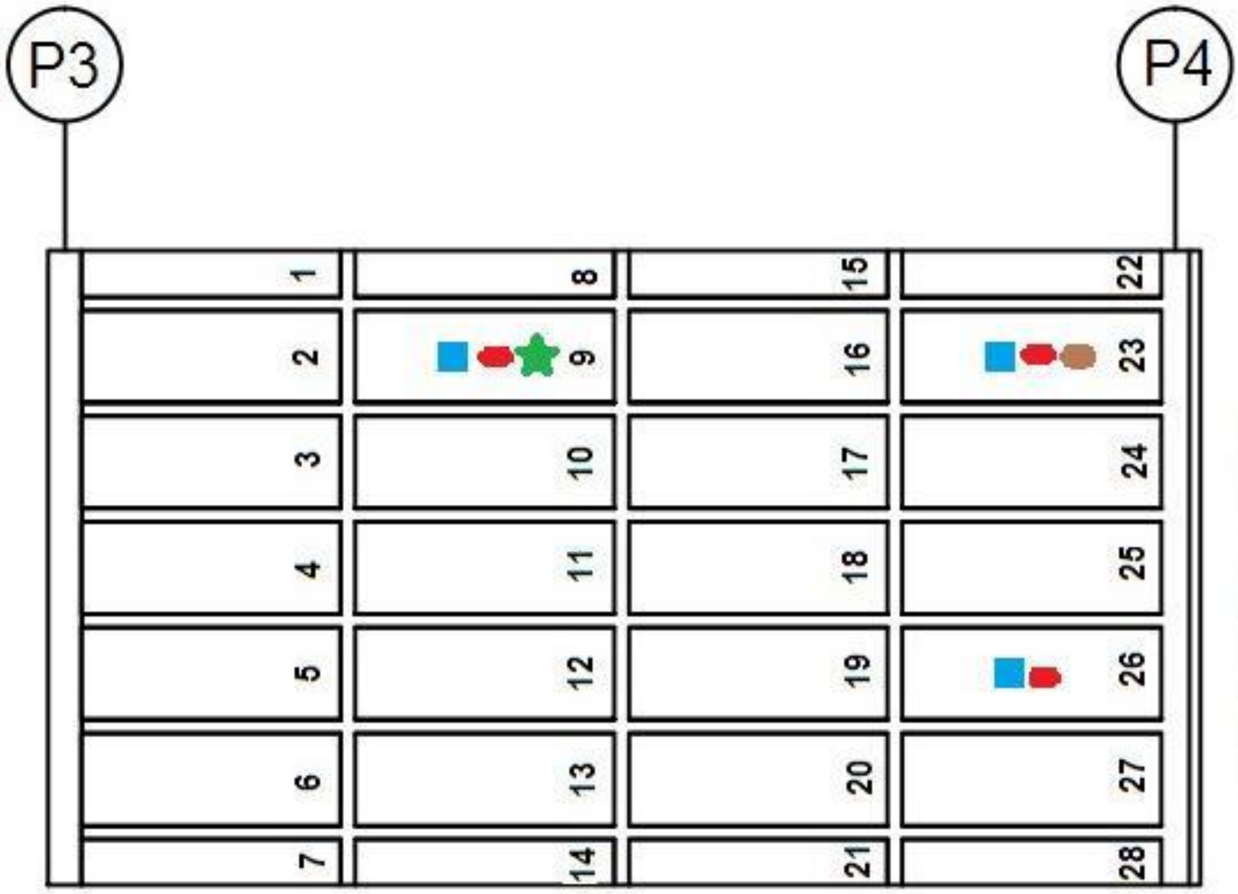


Fig. 36.Test conducted at various locations between Pier P3 and Pier P4

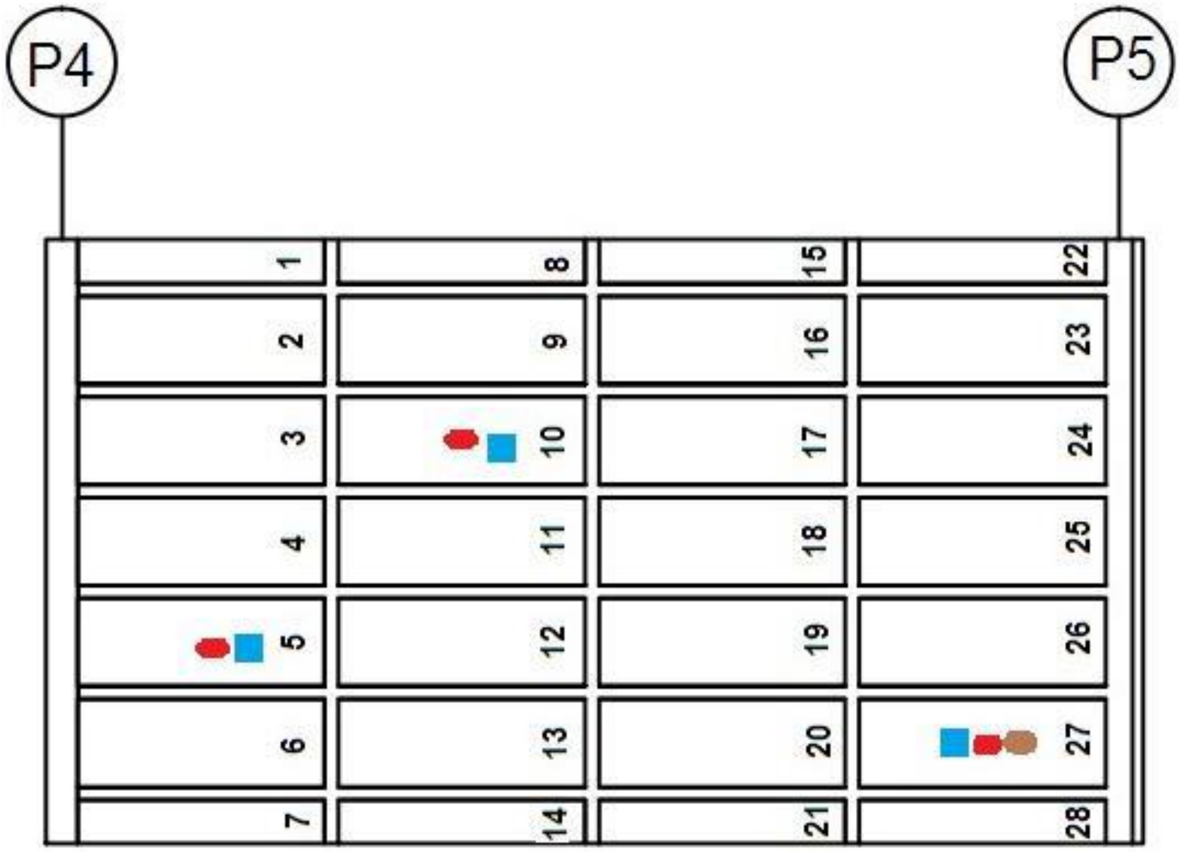


Fig. 37.Test conducted at various locations between Pier P4 and Pier P5

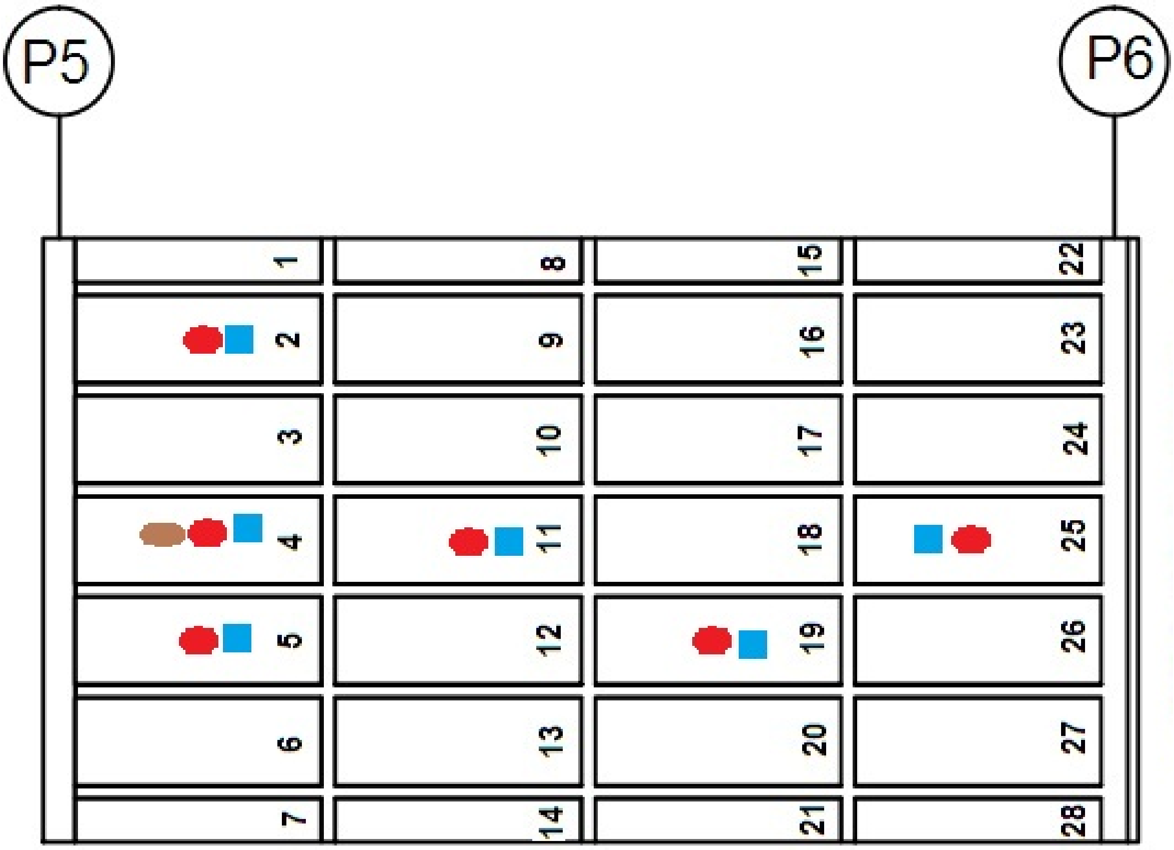


Fig. 38.Test conducted at various locations between Pier P5 and Pier P6

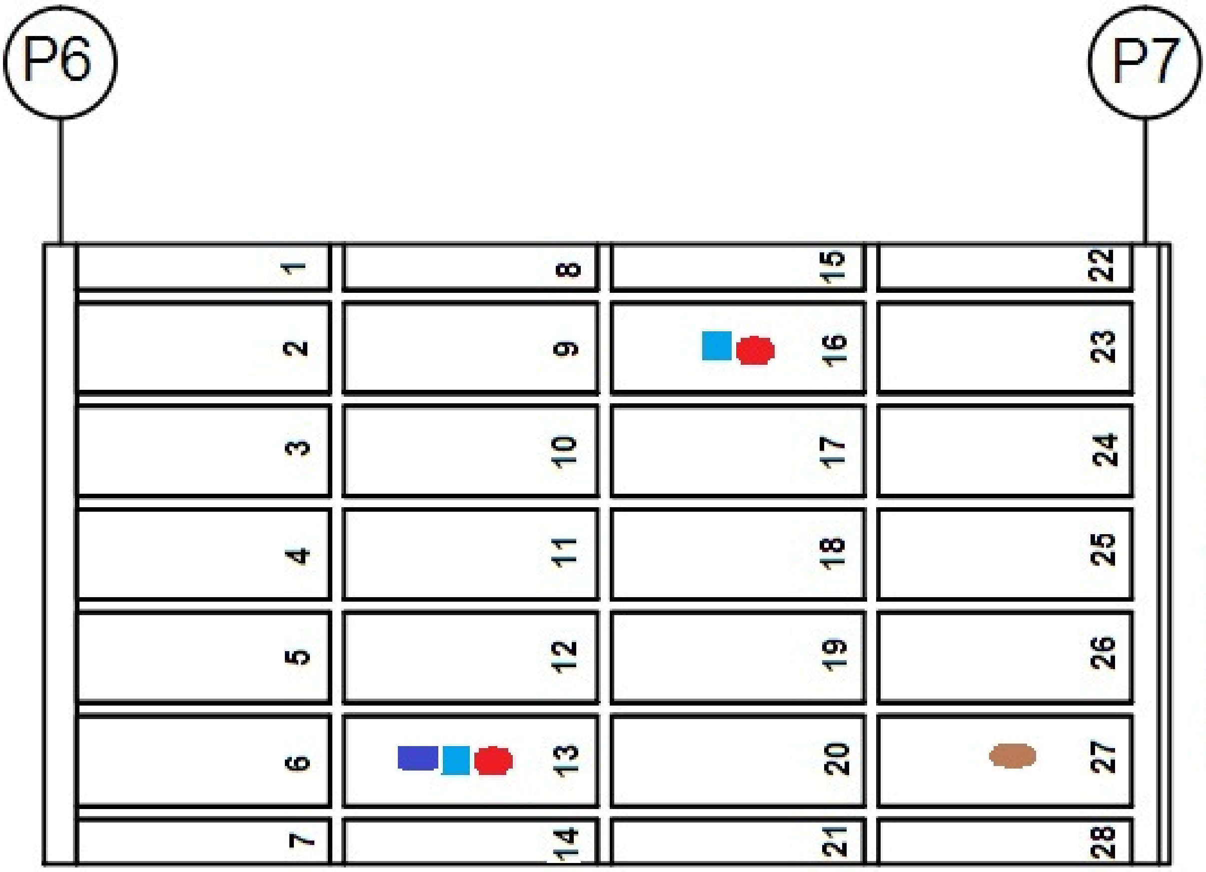


Fig. 39.Test conducted at various locations between Pier P6 and Pier P7

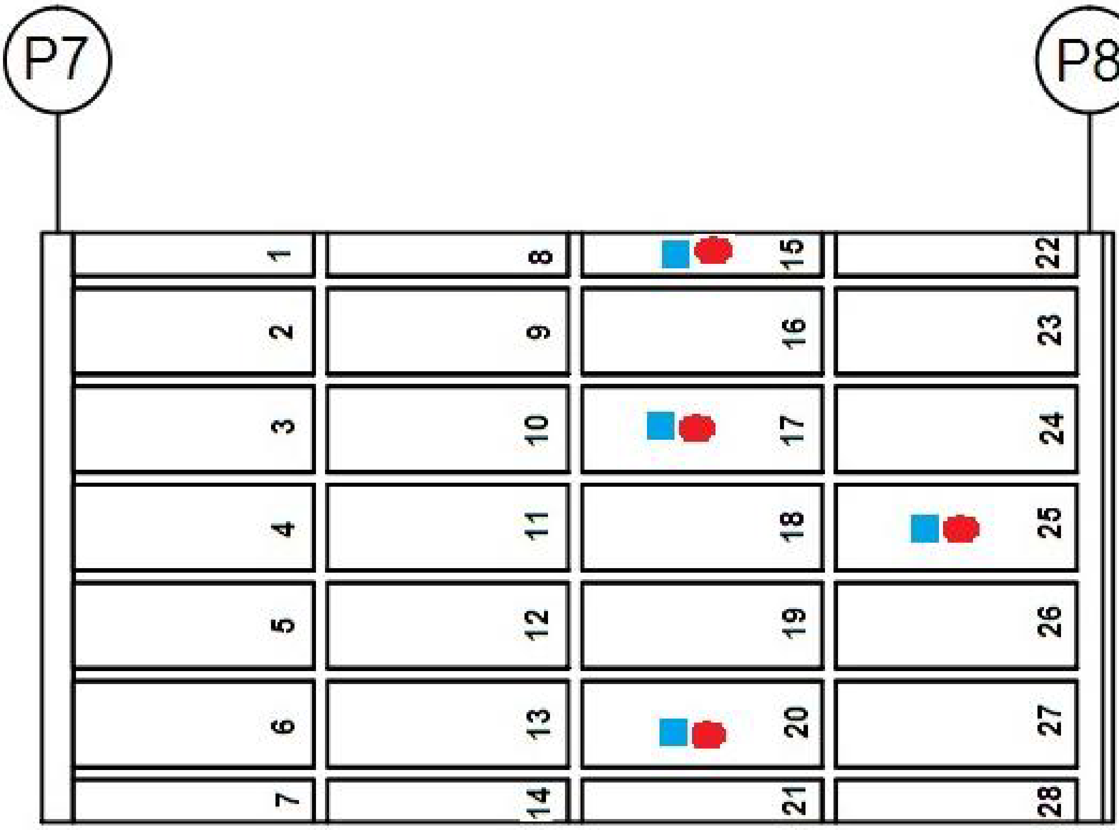


Fig .40.Test conducted at various locations between Pier P7 and Pier P8

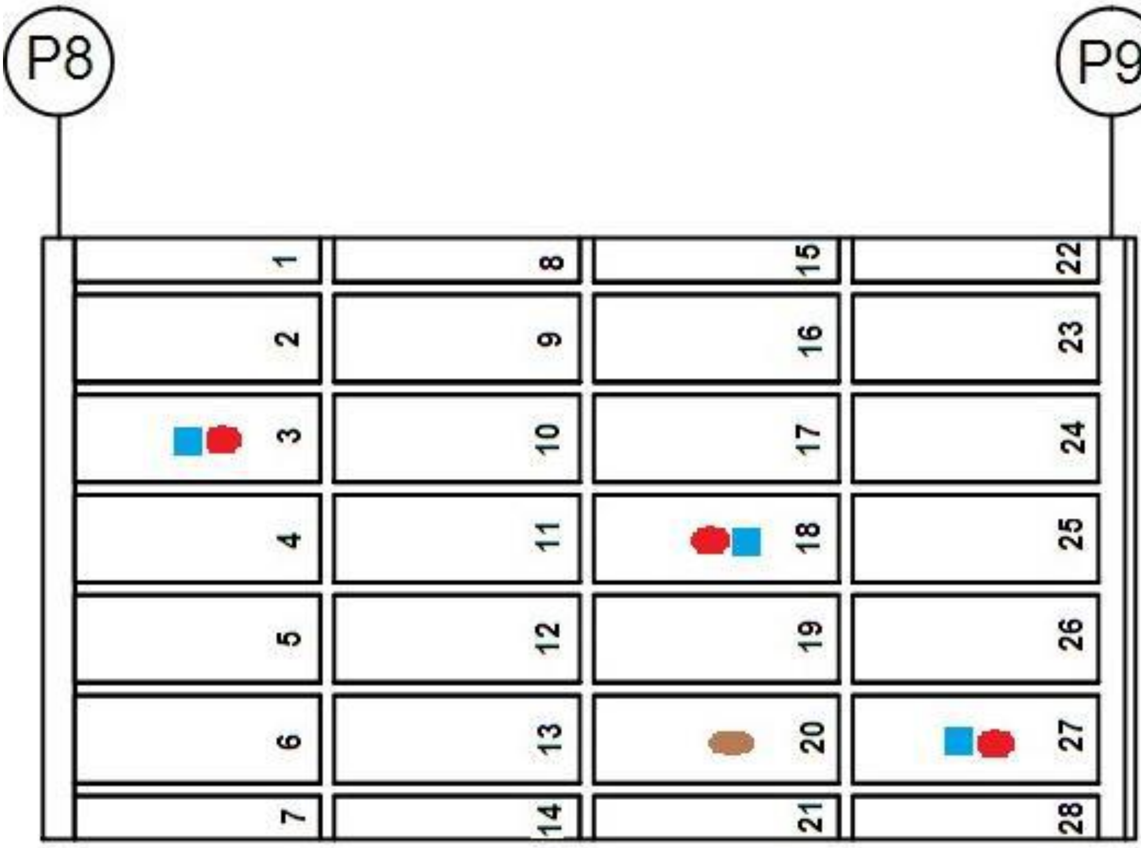


Fig .41.Test conducted at various locations between Pier P8 and Pier P9

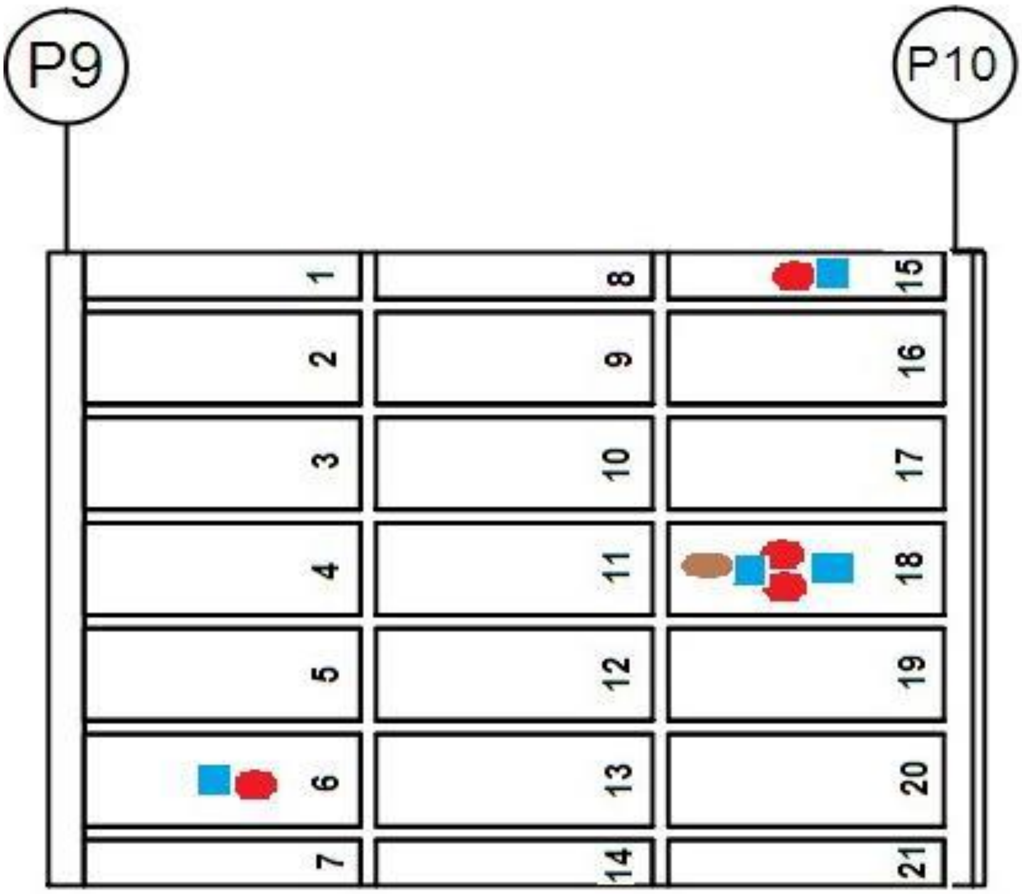


Fig. 42.Test conducted at various locations between Pier P9 and Pier P10

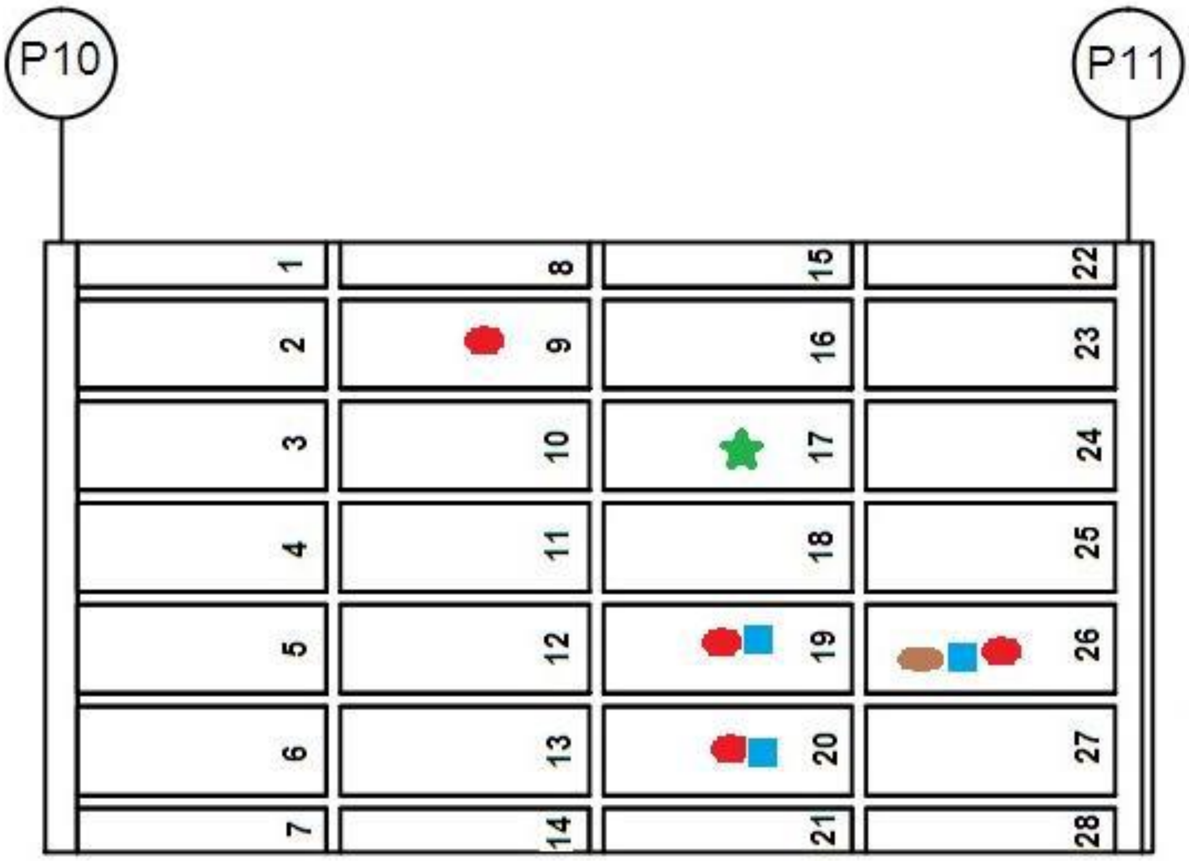


Fig .43.Test conducted at various locations between Pier P10 and Pier P11

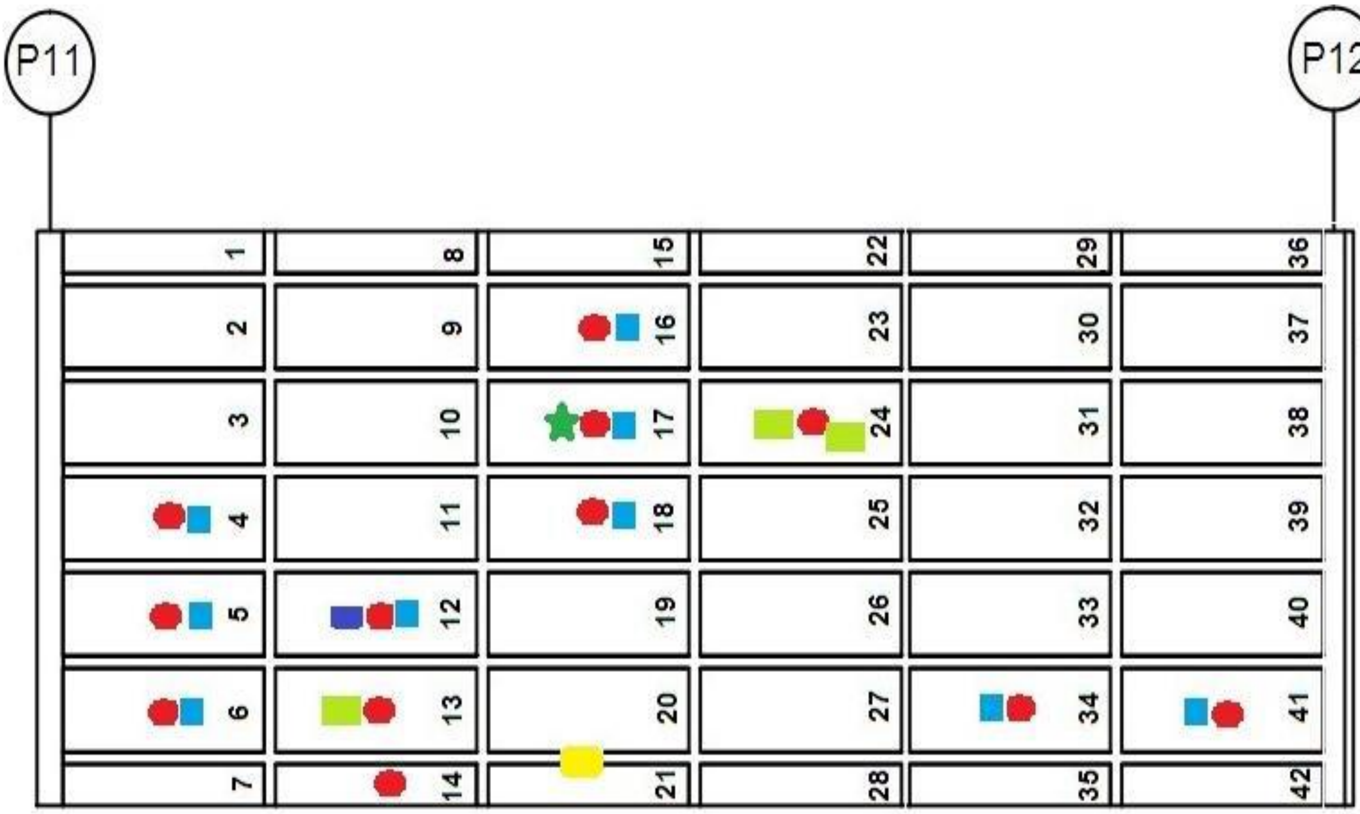


Fig.44.Test conducted at various locations between Pier P11 and Pier P12

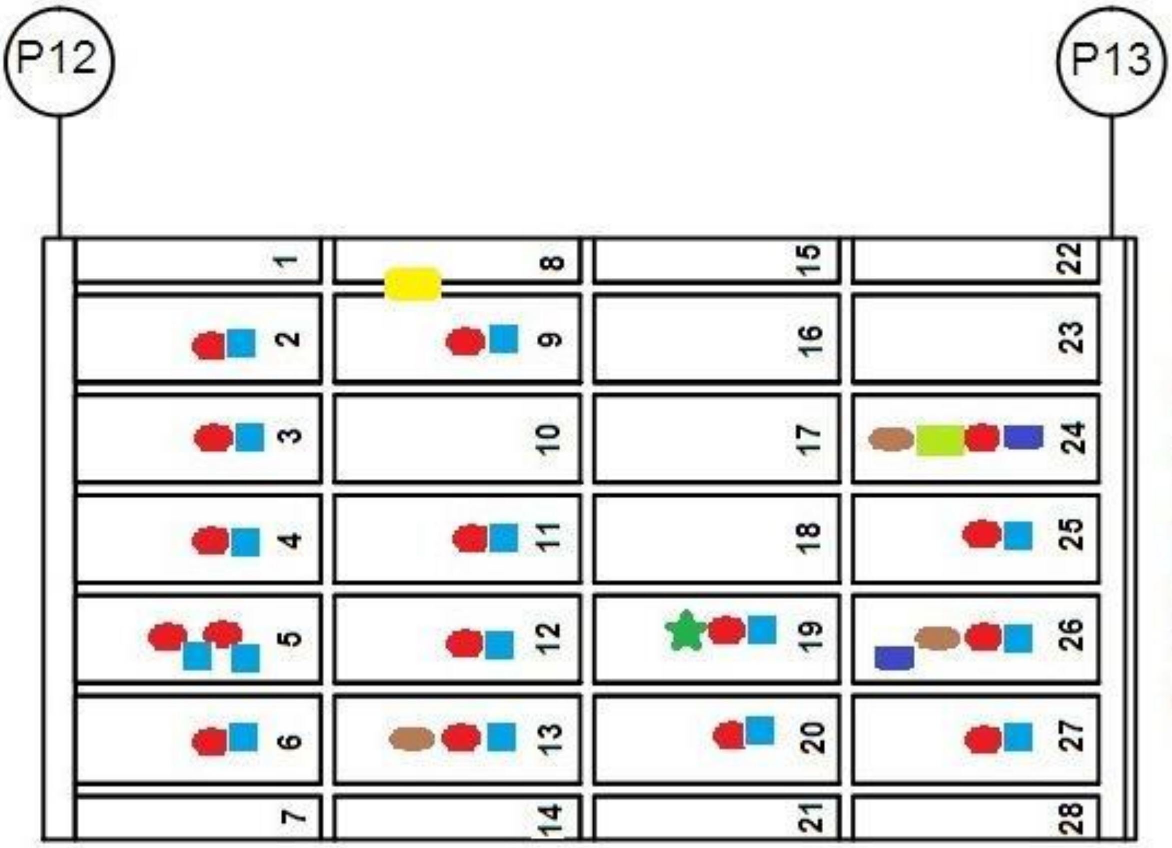


Fig .45.Test conducted at various locations between Pier P12 and Pier P13

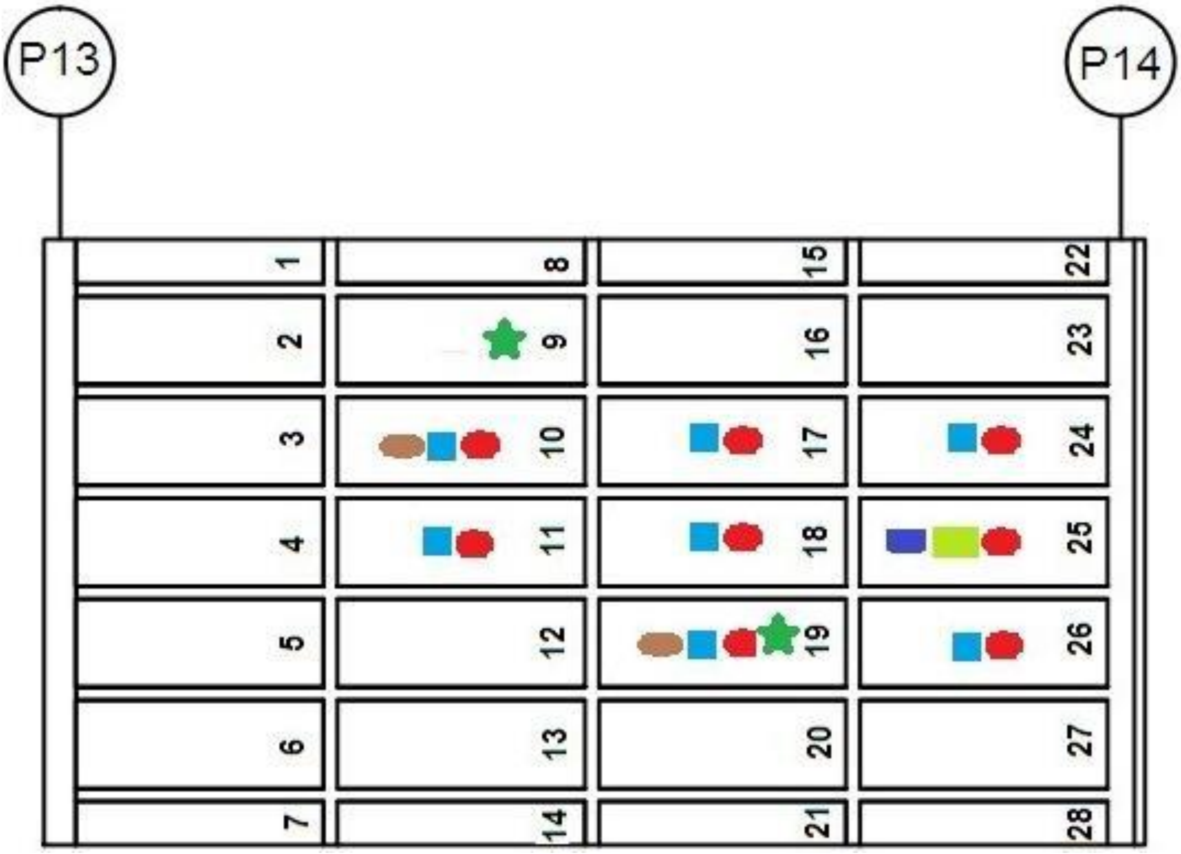


Fig. 46.Test conducted at various locations between Pier P13 and Pier P14

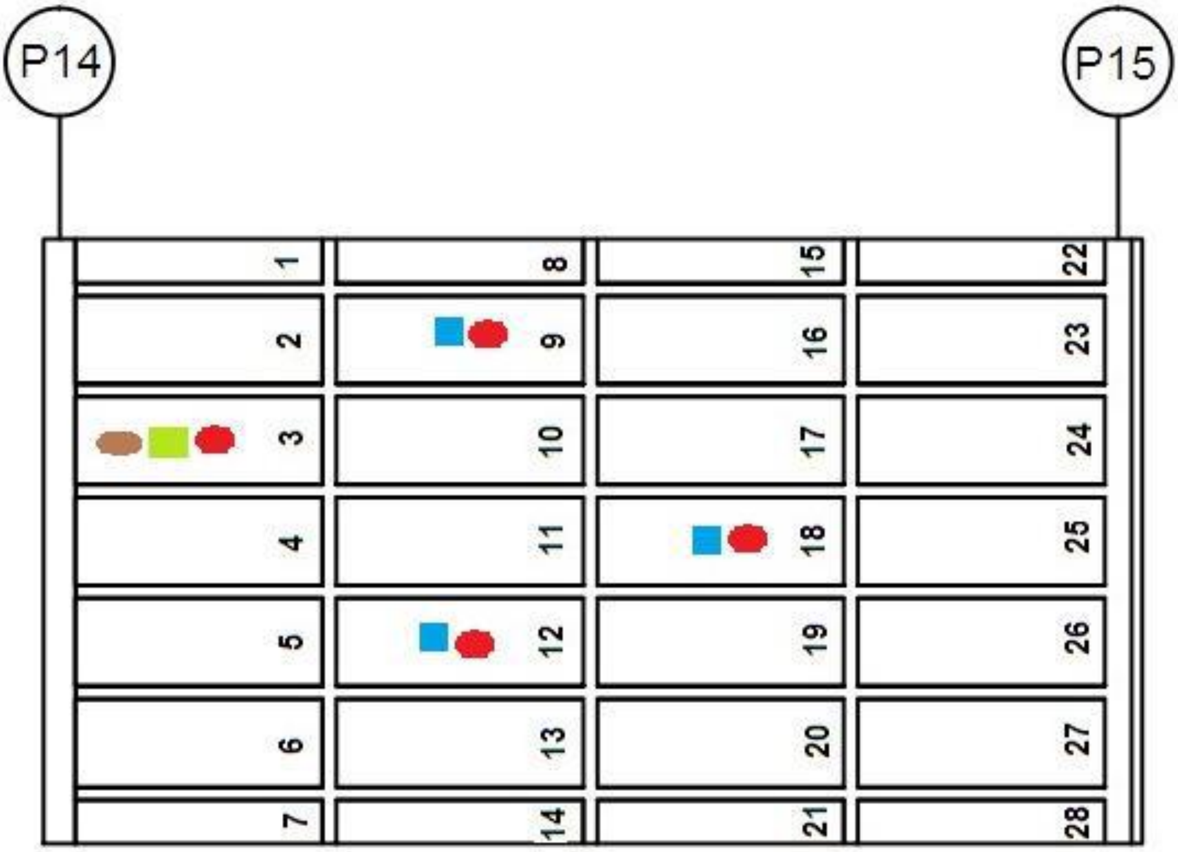


Fig. 47.Test conducted at various locations between Pier P14 and Pier P15

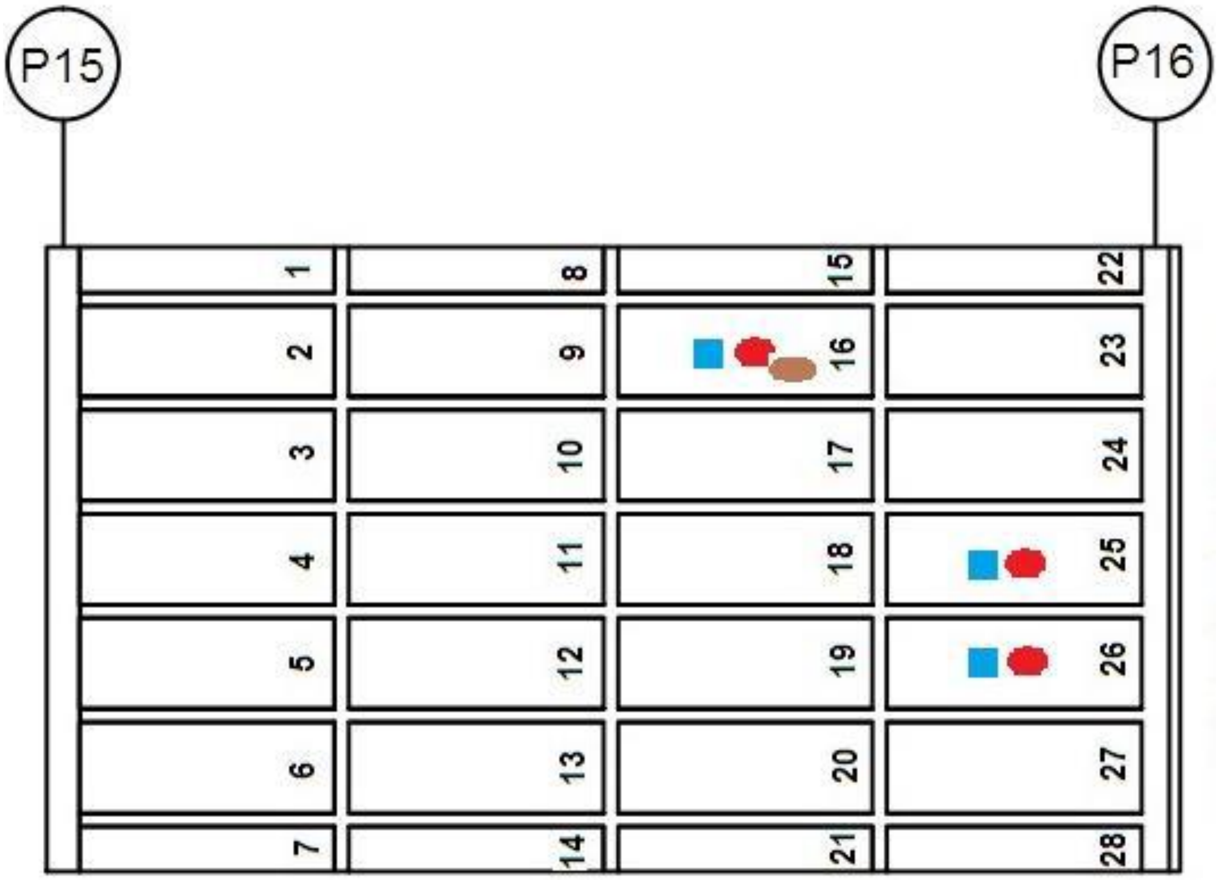


Fig. 48.Test conducted at various locations between Pier P15 and Pier P16

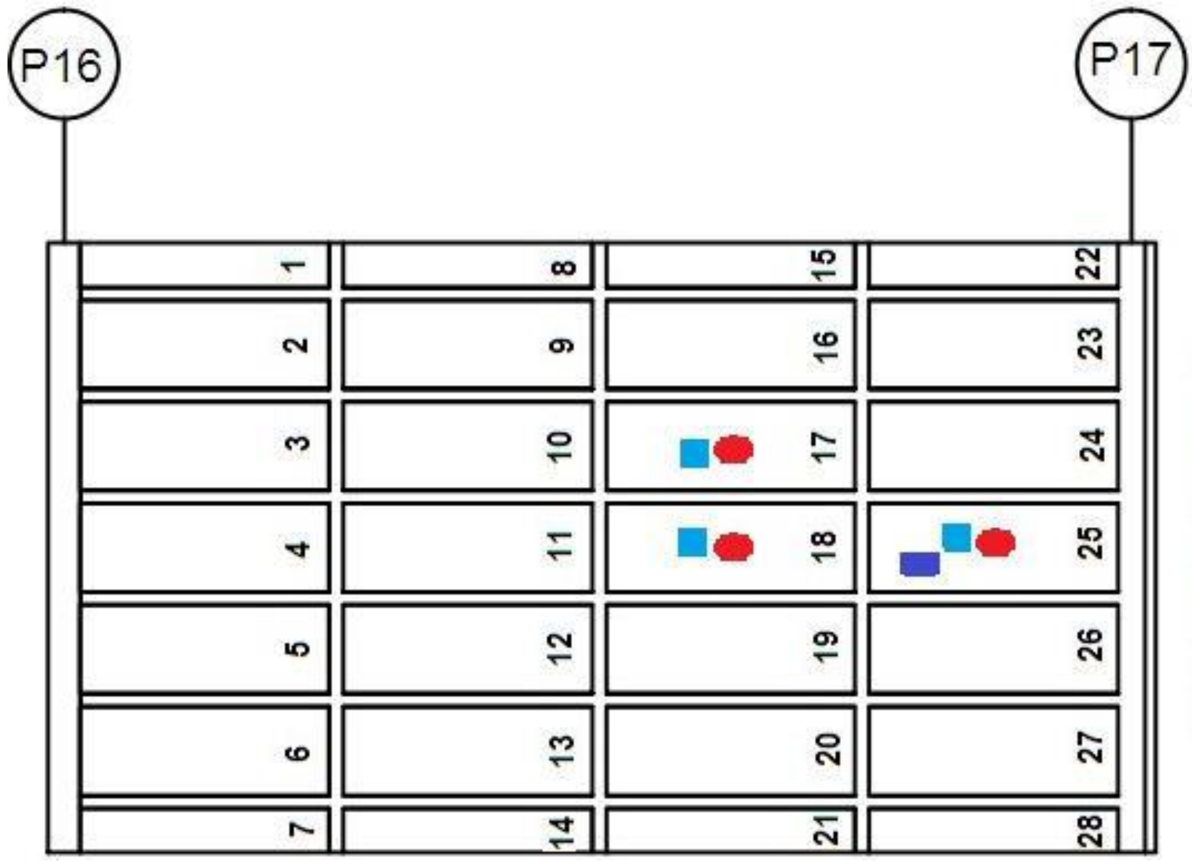


Fig. 49.Test conducted at various locations between Pier P16 and Pier P17

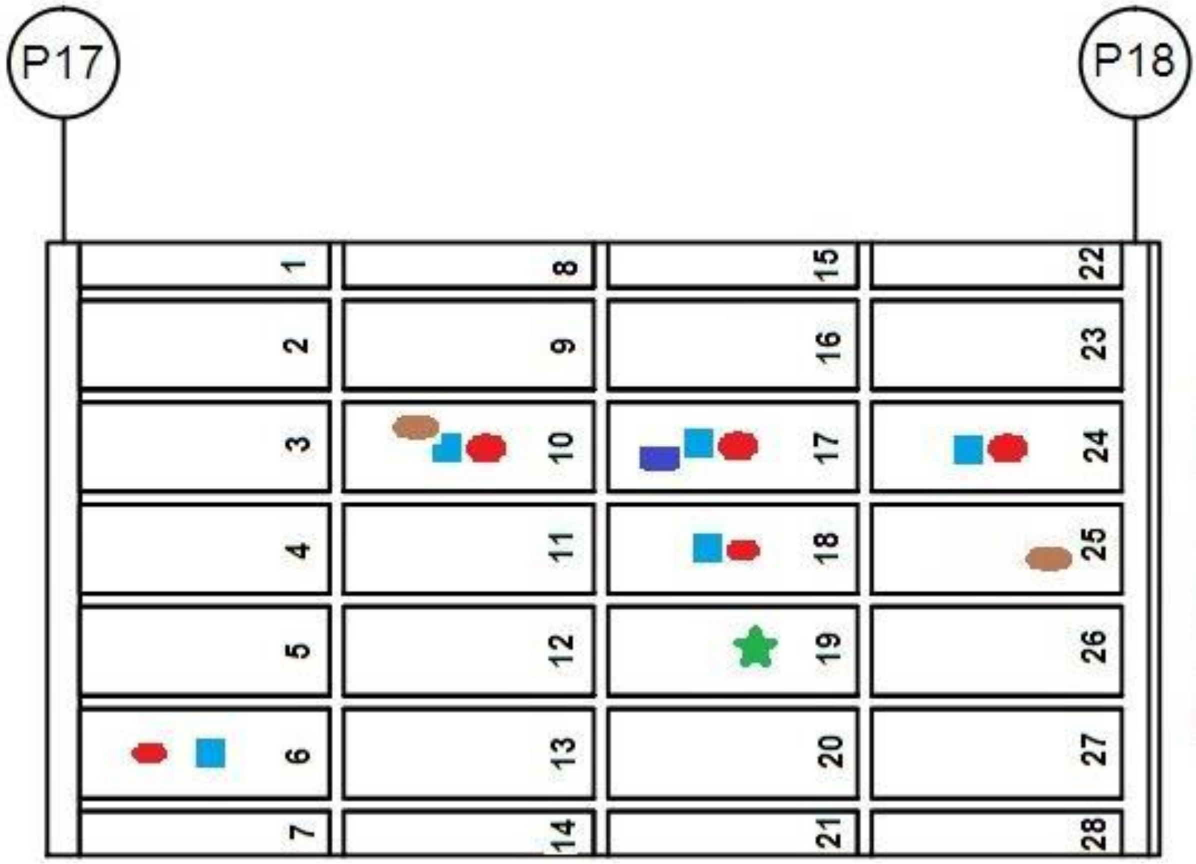


Fig. 50.Test conducted at various locations between Pier P17 and Pier P18

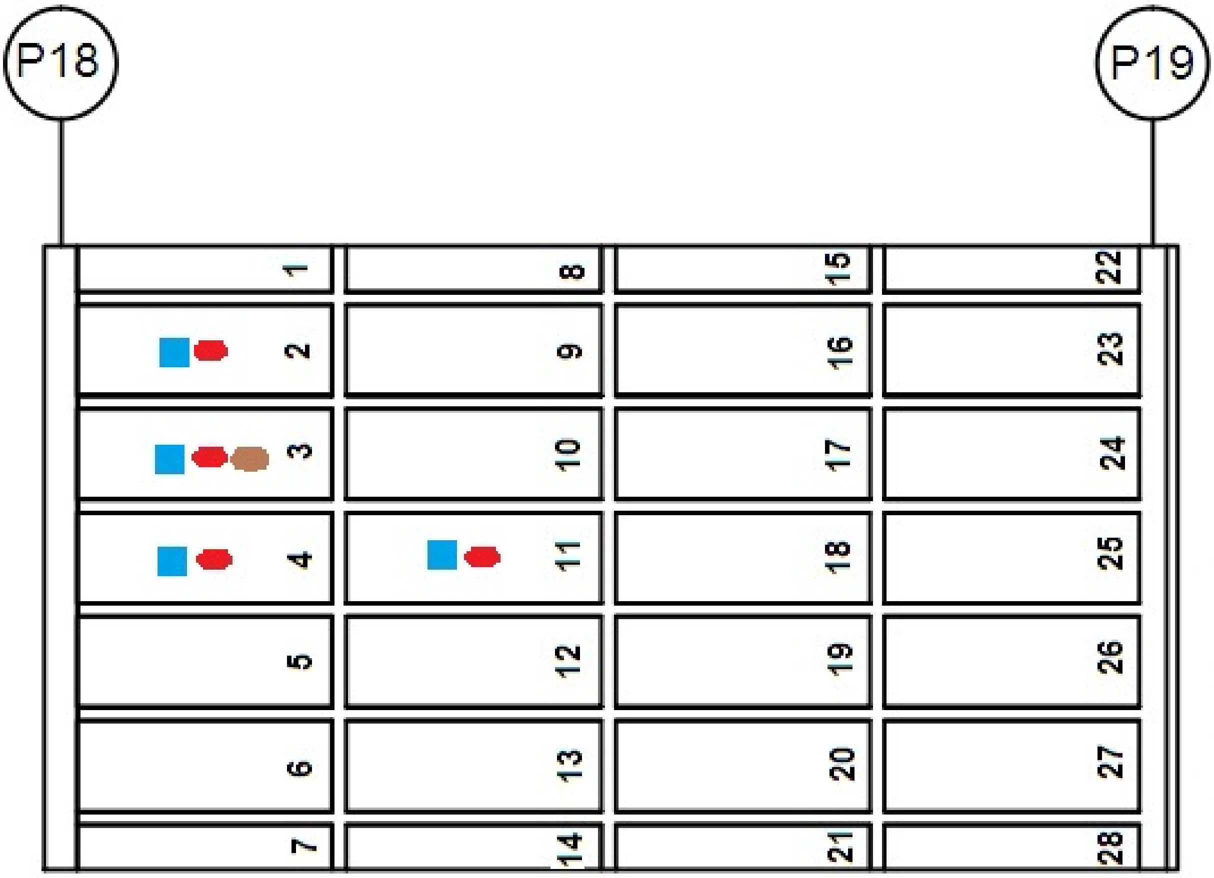


Fig. 51.Test conducted at various locations between Pier P18 and Pier P19

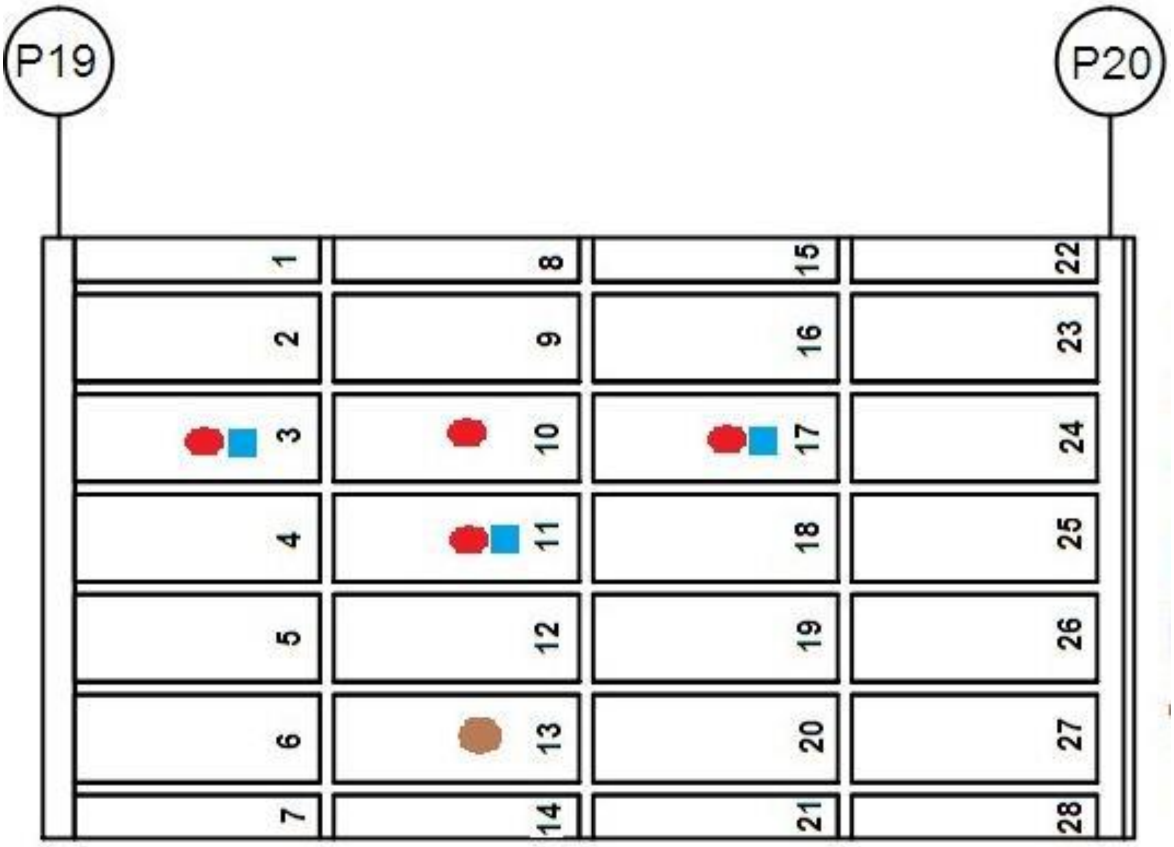


Fig. 52.Test conducted at various locations between Pier P19 and Pier P20

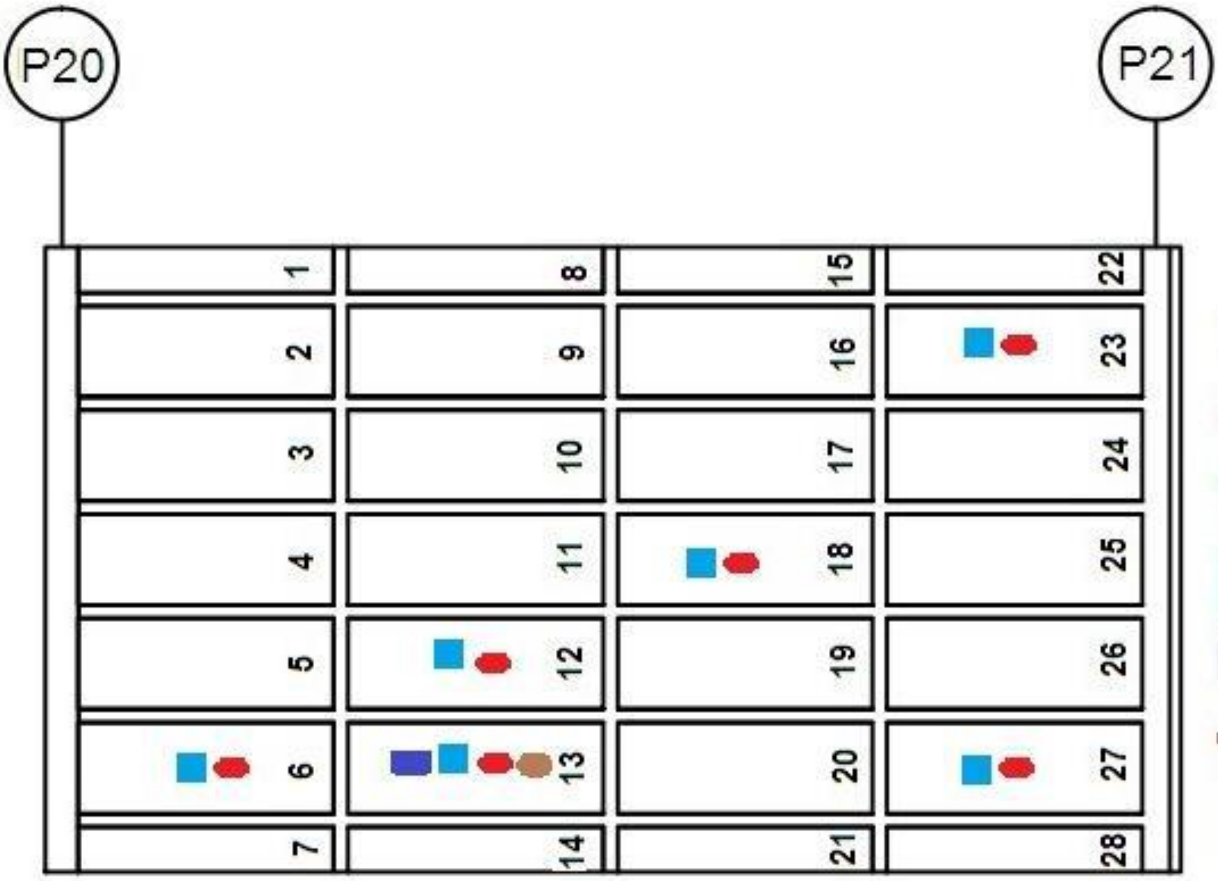


Fig. 53.Test conducted at various locations between Pier P20 and Pier P21

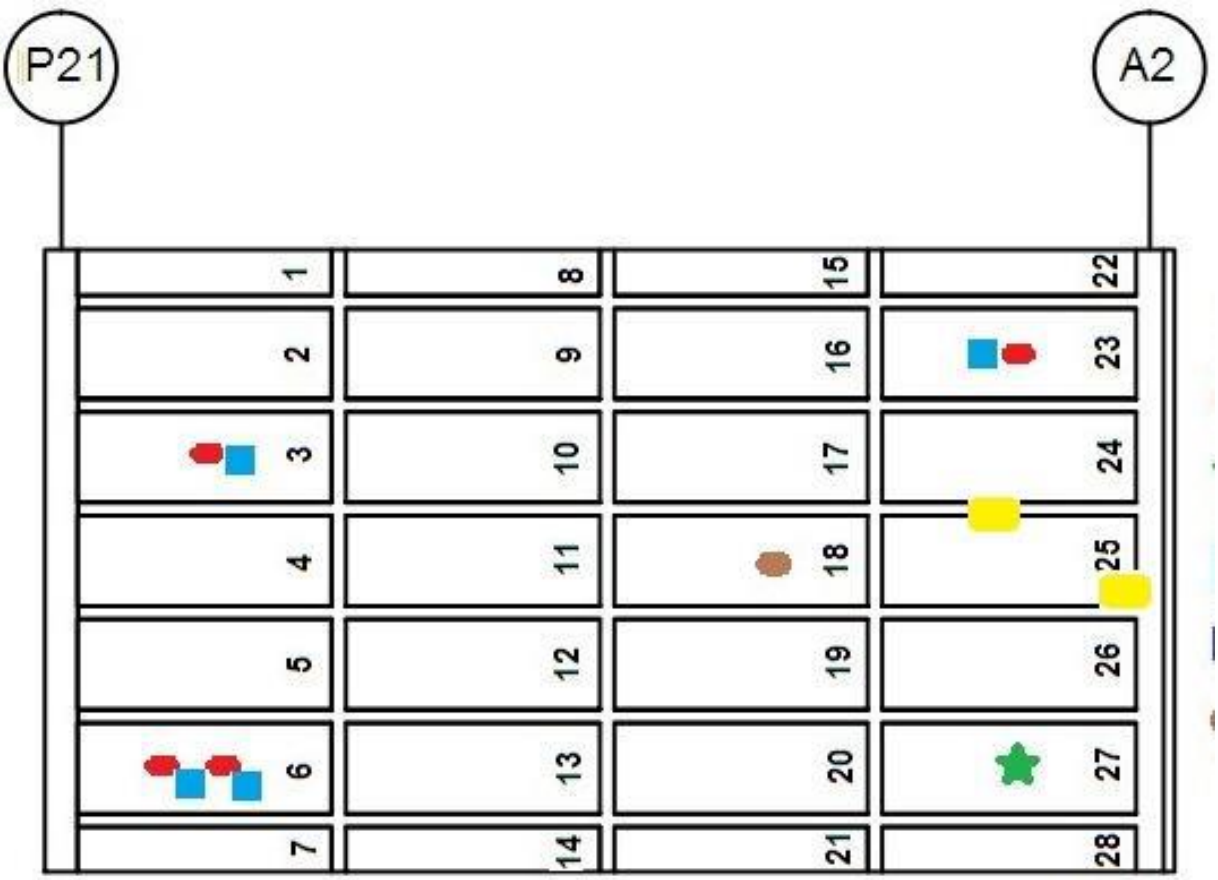


Fig. 54.Test conducted at various locations between Pier P21 and Pier P22

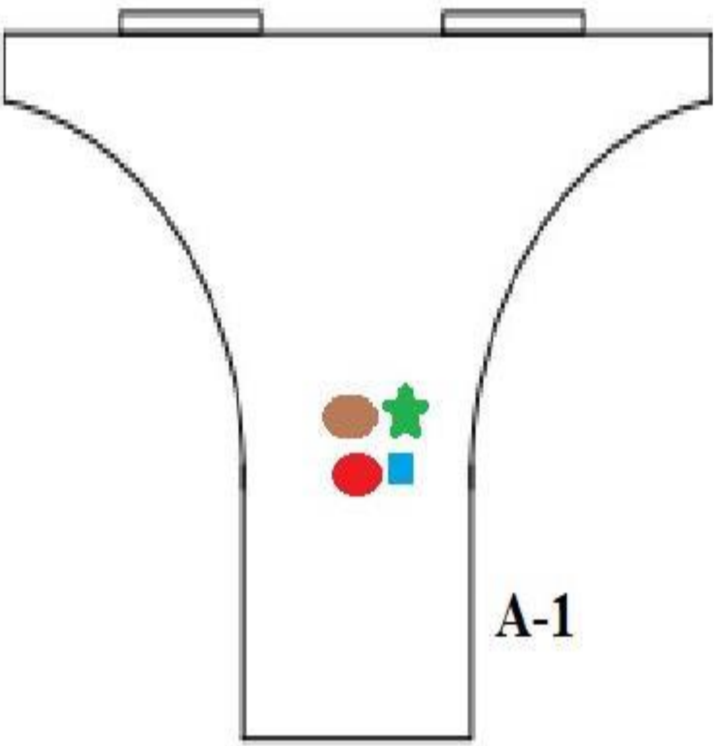


Fig. 55.Test conducted at various locations on the Abutment A1

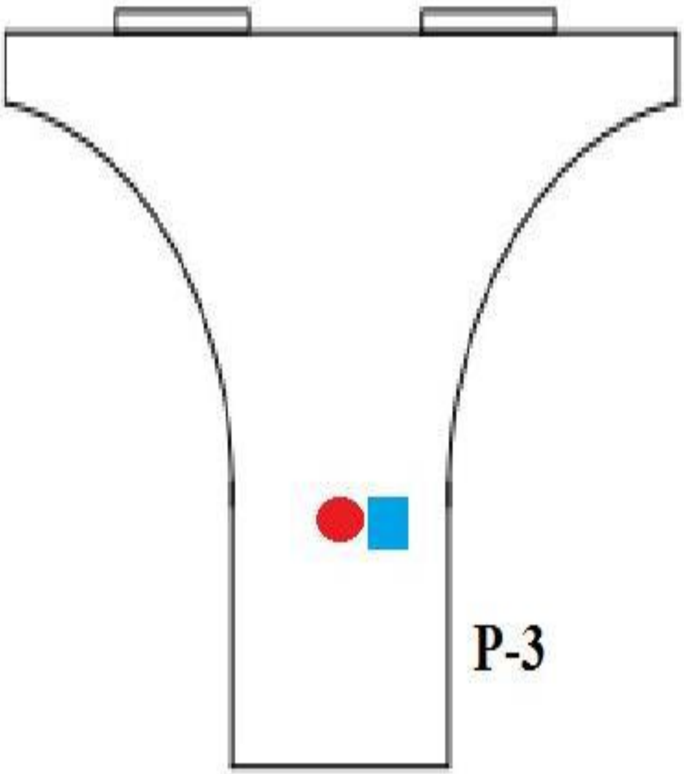


Fig. 55.Test conducted at various locations on the Abutment A1

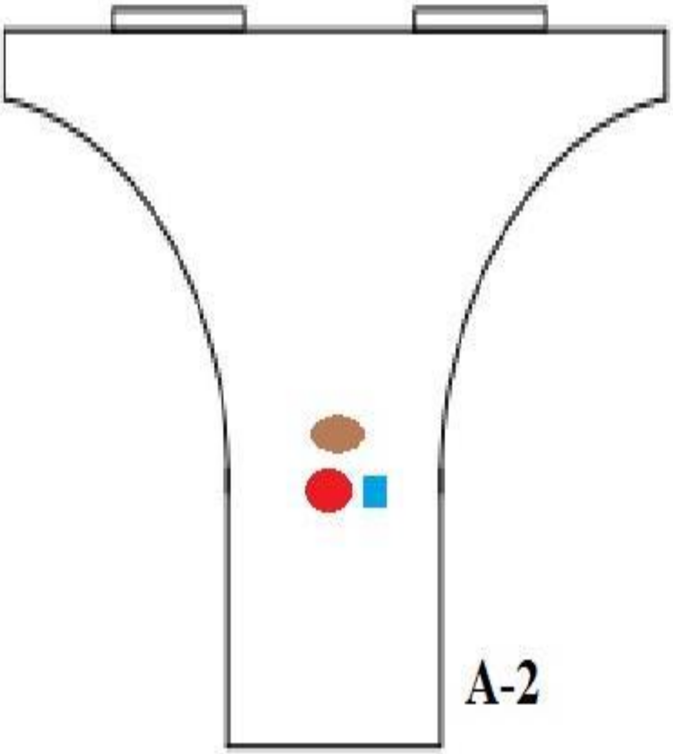


Fig. 56.Test conducted at various locations on the Abutment A2

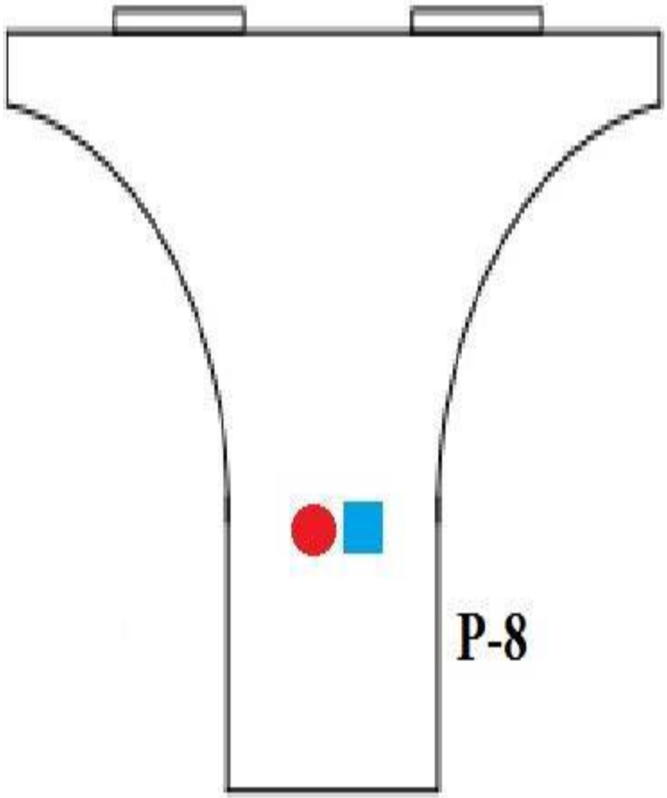


Fig. 59.Test conducted at various locations on the Pier P8

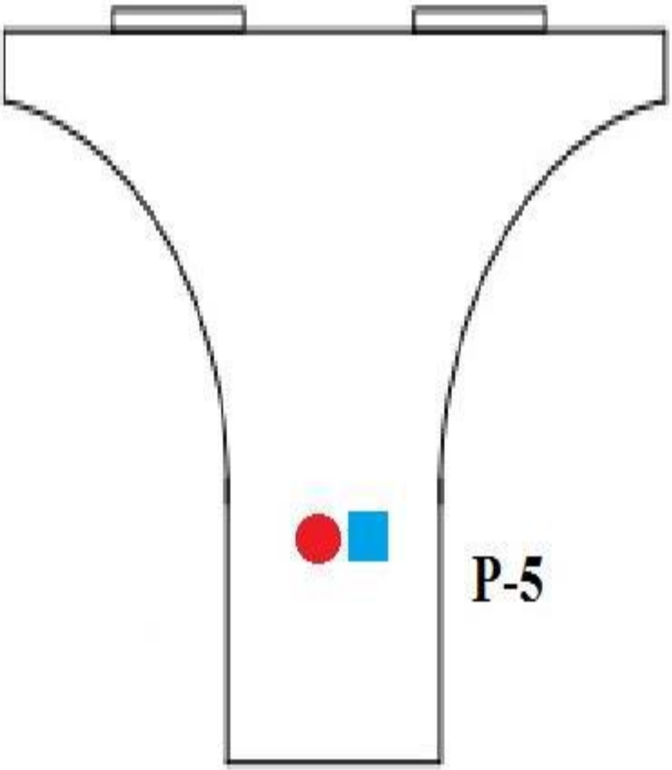


Fig. 57.Test conducted at various locations on the Pier P5

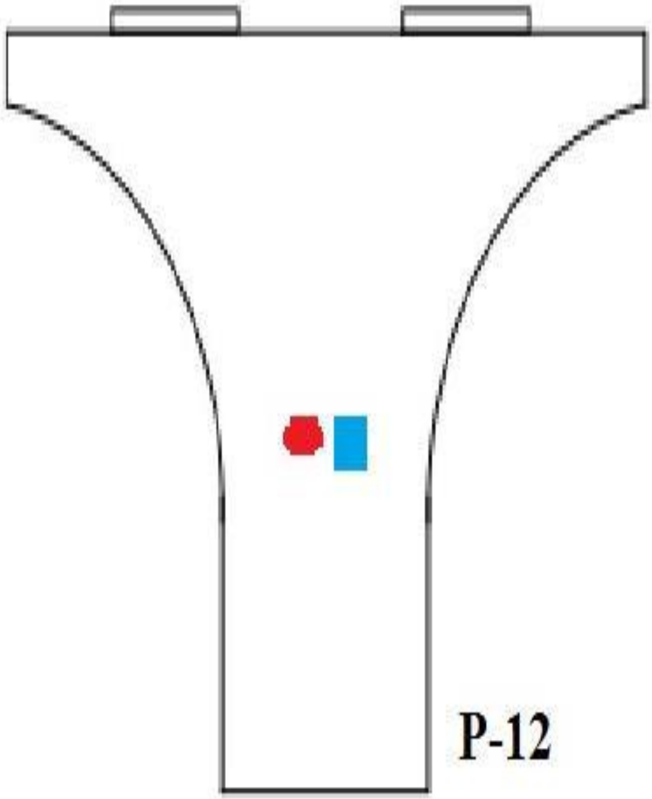


Fig. 61.Test conducted at various locations on the Pier P12

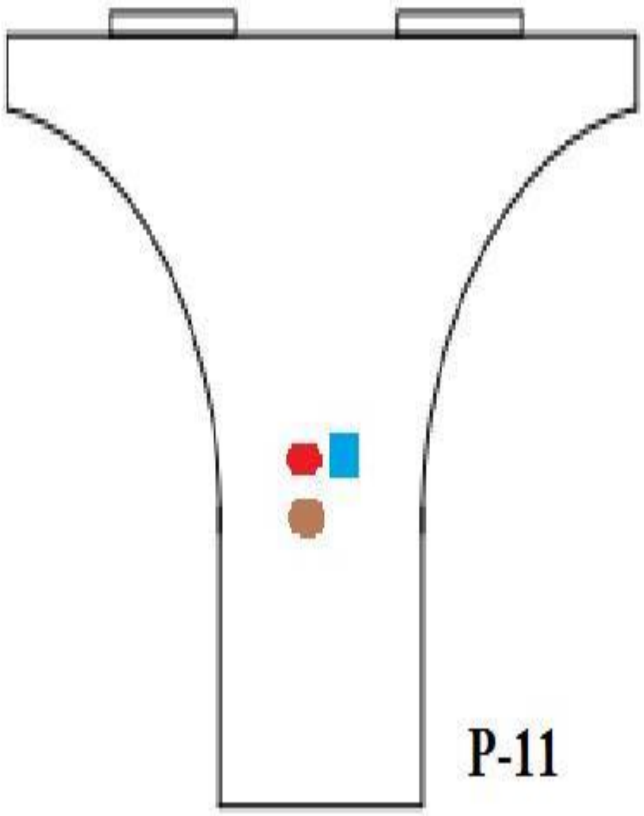


Fig. 60.Test conducted at various locations on the Pier P11

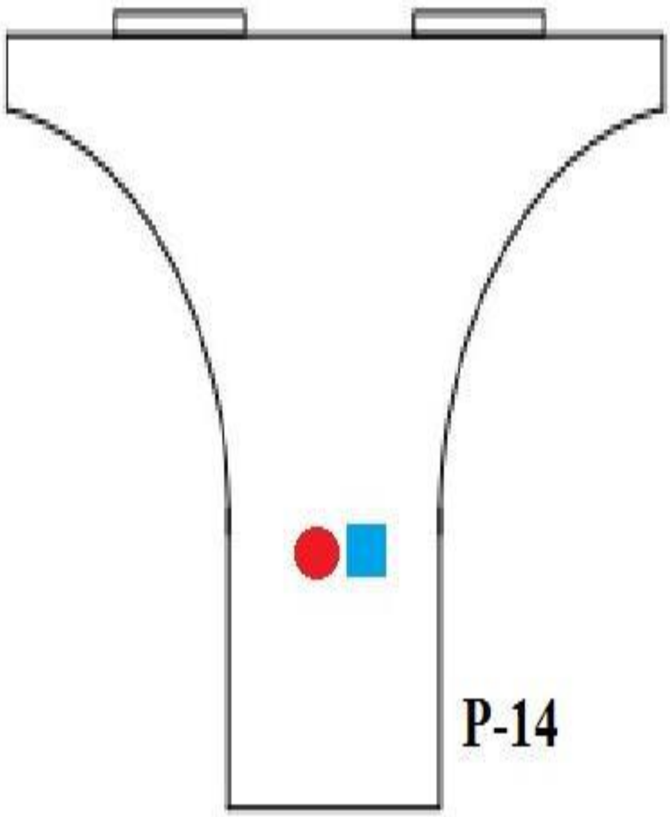


Fig. 63.Test conducted at various locations on the Pier P14

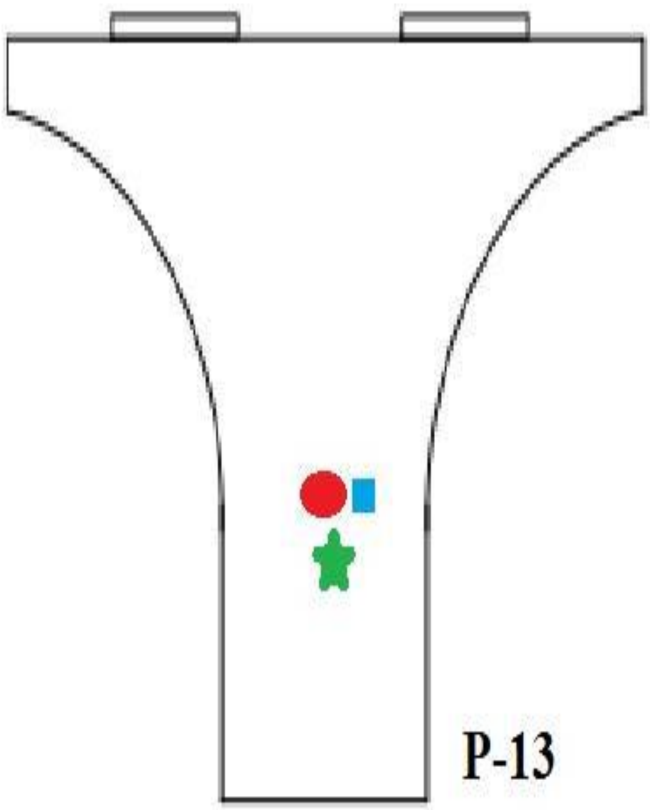


Fig. 62.Test conducted at various locations on the Pier P13

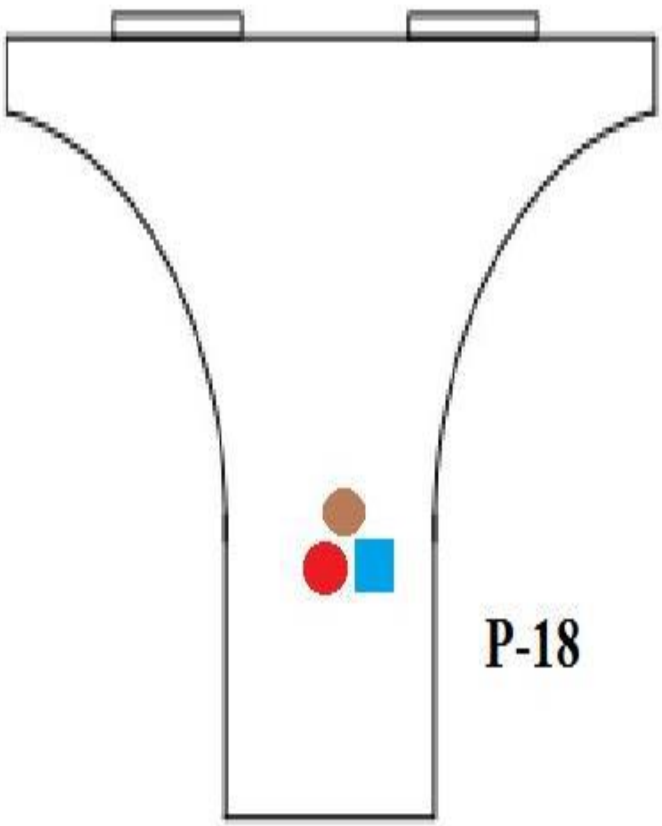


Fig. 65.Test conducted at various locations on the Pier P18

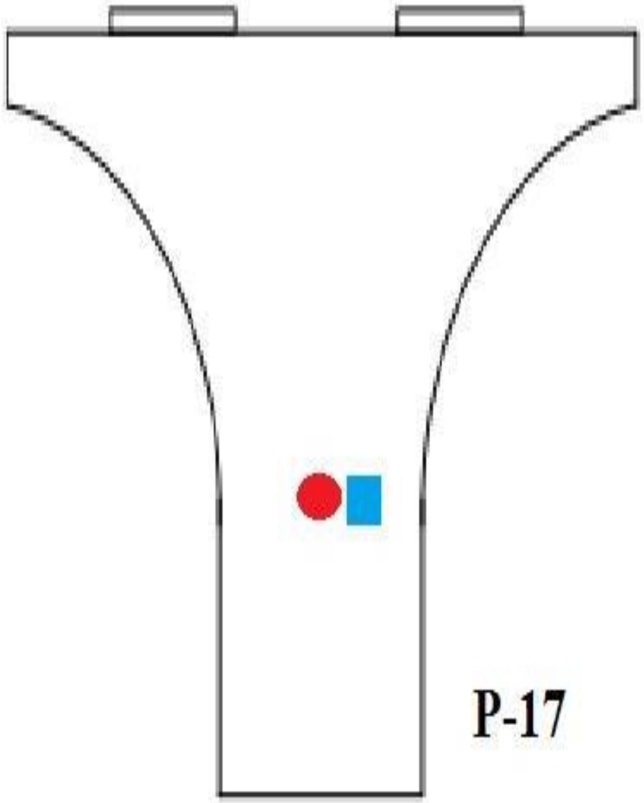


Fig. 64.Test conducted at various locations on the Pier P17

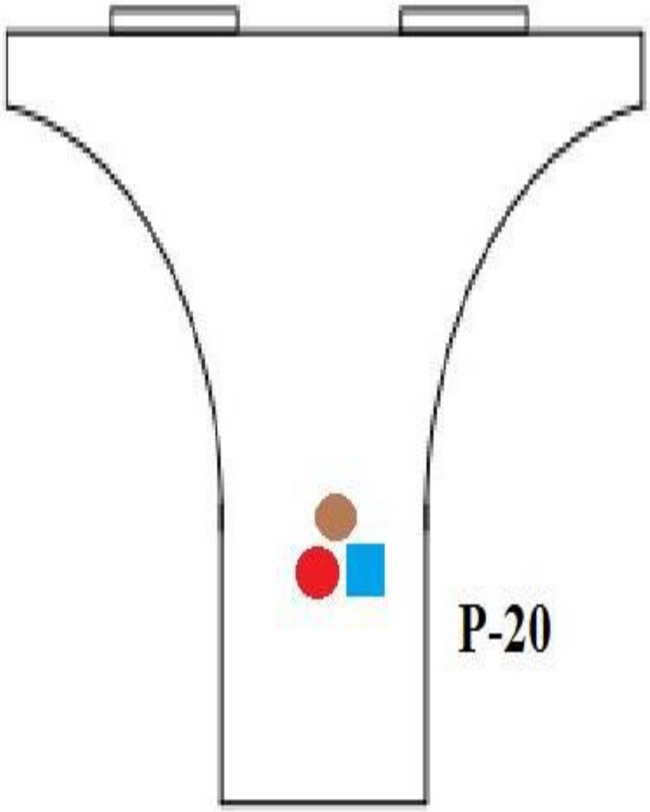


Fig. 66.Test conducted at various locations on the Pier P20

B.TESTING PHOTOGRAPHS

REBOUND HAMMER TEST

Photo 70. Rebound Hammer Test

Photo 71. Rebound hammer Test on Panel

USPV TEST

Photo 72.USPV Test on Pier P11

Photo 73.USPV Test on Panel

COVER METER

Photo 74.Profometer is being used for finding the cover meter

CORE EXTRACTION

Photo 75.Core sample is being extracted for compression test

CORE SAMPLE TEST

Photo 76.The core sample is placed and tested using Compression Testing Machine

CRACK PATTERN

Photo 77.Crack pattern analysis using the USPV

THICKNESS TESTING

Photo 78.Thickness Testing on steel girders and diaphragms

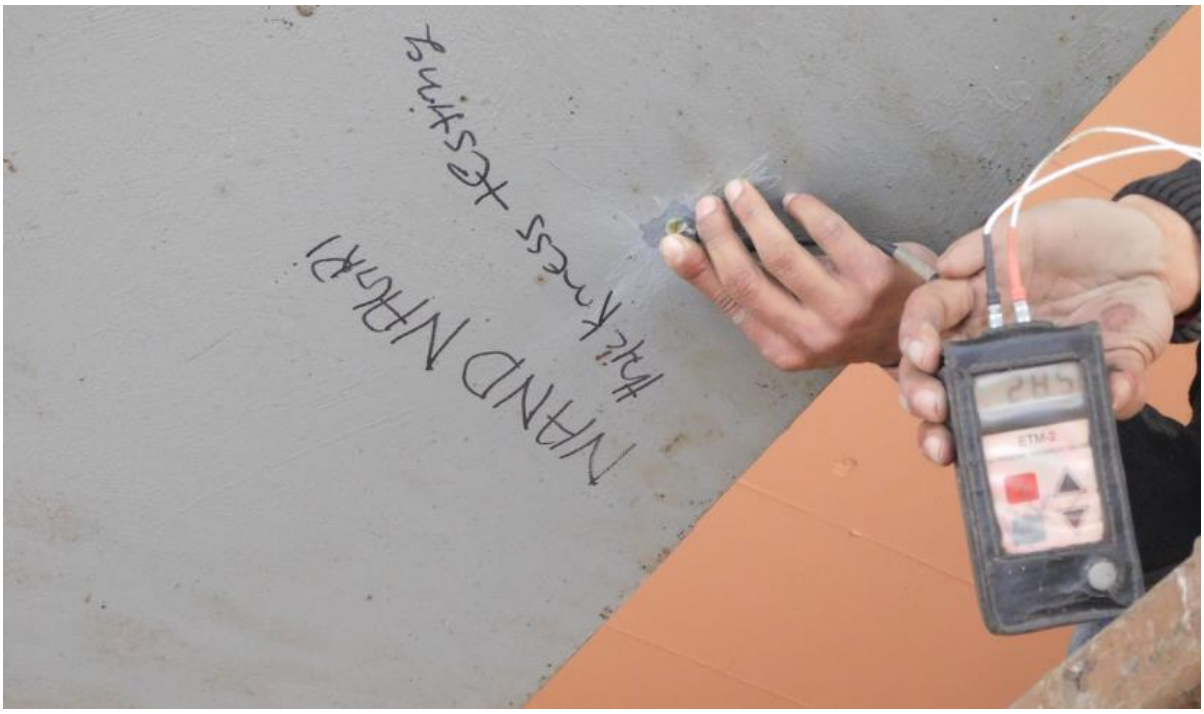


Photo 79.Thickness Testing on steel with Ultrasonic thickness gauge

C. JOURNAL REFERENCE

Repair of Construction-Related Deterioration in Precast Deck-Panel Bridges

Atiq H. Alvi, Ivan Gualtero, Rajan Sen, and Gray Mullins

Precast, partial-depth deck panels have been used throughout the United States as stay-in-place forms and to provide a portion of deck strength. In Florida, fiberboard material was routinely placed along the edges of the panels to seal the overlay of concrete, rather than embed the panels in grout. This approach did not allow the concrete to flow fully underneath the panel ends and did not provide a reliable, rigid bearing. The seriousness of this seemingly minor change in practice was only fully recognized nearly two decades later, when seven punching shear failures occurred on major highways. This paper reviews eight repair methods employed by the Florida Department of Transportation to maintain 200 deck-panel structures until they could be replaced. The paper highlights the difficulties that were faced in devising repairs when the underlying cause of the damage was not understood fully. Full-depth bay replacement with cast-in-place concrete was the most effective approach but required extended lane closures. Full-depth precast panels could be installed during nighttime lane closures but cost more. The most important lesson learned was that flexible materials, such as asphalt, were best avoided to repair the bridge decks.

Bridge deck deterioration most commonly results from corrosion (*1*). Although corrosion damage is expensive to repair, its cause is well understood, and proven methods, such as cathodic protection, are available to mitigate it (*2*). By contrast, construction-induced problems in which identical bridges in identical environments are exposed to similar loading (e.g., northbound and southbound Interstate bridges over the same crossing) may not necessarily deteriorate in the same manner. Such unpredictable deterioration poses special problems to highway agencies responsible for bridge maintenance and service.

Precast deck-panel bridges have an excellent track record, except in Florida, where they have a long history of premature deterioration. The state was an early adopter of this type of bridge, and once had an inventory of nearly 200 of them, although the number has dwindled as they gradually have been replaced. Research has indicated that poor performance was the result of an unintentional construction error (*3–6*). Fiberboard bearings used to support precast panels were positioned at their ends with no overhang. That arrangement did not leave space for the concrete to flow underneath the panel and provide rigid support when the cast-in-place (CIP) concrete was poured (Figure 1). This error changed the load path for shear with disastrous consequences, which led to seven, localized punching shear failures from 2000 to 2007. Several of the bridges had been in service for more than two decades (Table 1). These failures highlight the enormous difficulties that are faced in repairing and maintaining bridges when the underlying cause of deterioration is unclear or cannot be predicted without destructive bedding evaluation at the panel ends.

This paper assesses eight repair methods used by the Florida Department of Transportation (DOT) to maintain such bridges in service. Background information on precast deck-panel bridges, including their expected structural response, is presented first. Information on the type of cracking that developed under service was retrieved from inspection reports. Particular reference was made to a bridge over an Interstate highway that experienced localized failure in 2000 after 20 years of service. This information provided a platform for a critical review of the repair methods. Additional information may be found in a comprehensive report (*7*) that was updated recently (*8*).

BACKGROUND

Precast deck-panel highway bridges were first constructed in Illinois in the early 1950s. Unlike today's full-depth precast decks, a precast deck panel served as a stay-in-place form for a CIP slab placed on top and in between the panels. As a result, field forming was needed only for the exterior girder overhangs, which resulted in considerable savings in construction time and costs. Bridges of this type were constructed successfully in several other states, most notably in Texas, where more than 1,650 of them exist in the state and county systems (*9*). Most of the bridges have performed well: 833 of them were rated Condition 8, and 20 were rated Condition 9 in the National Bridge Inventory, in which Condition 0 = failure and Condition 9 = excellent (*7*). These ratings are in contrast to Florida's dismal experience with this construction technique.

Deck-panel bridges were first constructed in Florida in the 1970s and by the early 2000s there were approximately 200 such bridges in the state. Of these, 127 are located in Districts 1 and 7. These districts consist of 17 counties, which range from the central to the southern regions of the state. Originally, full-depth CIP decks were planned but, during construction, a change was proposed to use the deck-panel option. In general, the precast panels were 8 × 10 ft in plan and 3.5 to 4 in. thick. They provided

A. H. Alvi, T. Y. Lin International Group, 12802 Tampa Oaks Boulevard, Suite 245, Tampa, FL 33637. I. Gualtero, Ayers Associates, 8875 Hidden River Parkway, Suite 200, Tampa, FL 33637-1035. R. Sen and G. Mullins, Department of Civil and Environmental Engineering, University of South Florida, 4202 East Fowler Avenue, Tampa, FL 33620-5350. Corresponding author: R. Sen, sen@usf.edu.

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DOI: 10.3141/2292-13

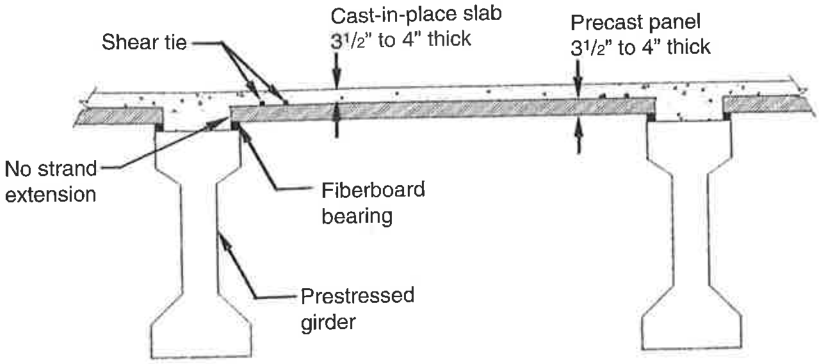


FIGURE 1 Cross section of precast, prestressed panel deck.

support for a 3.5- to 4-in. thick CIP overlay and were intended to act as a composite deck system under live and superimposed dead load (*4, 5*).

REQUIREMENTS FOR COMPOSITE ACTION

Composite action implies that the CIP slab and the prestressed deck panel act in concert to resist the applied loading. This requires development of horizontal shear stresses at the interface of the CIP slab and the precast panel for flexure (*10*). Because the CIP slab is cast monolithically over the prestressed panel, it spans continuously over the prestressed beams. Thus composite action is required to resist both positive and negative moments.

If the surface of the precast panel is roughened, codes allow 80 psi of horizontal shear transfer (*11*). Because the interface is large at the middle of the panel, composite action is automatic; at its ends, composite action necessitates steel from the precast panel to extend into the CIP slab (*7*). Bearing length and height below the panel also must be sufficient so that vertical shear can be transferred to the prestressed girder. In the drawing shown in Figure 1, these conditions are not met; the panel is supported only by the fiberboard material at its end. Moreover, the steel strands from the panel terminated at the end of the panel and did not extend into the CIP concrete.

EXPECTED CRACKING

Under live loading, tension develops perpendicular to the traffic direction. Therefore, cracking can be expected only in the longitudinal traffic direction (transverse to the panel). In the mid-span region, visible longitudinal cracks can occur on the underside of the precast panel if the loads exceeded the cracking moment at the section. Similarly, in the negative moment region, longitudinal cracks

TABLE 1 Localized Deck Failures

Bridge No.	District (Bridge Location)	Year Built (Failure Date)	Condition Rating Before Failure	Rainfall in Week Before Failure, days (in.)	ADT (% truck)	Failure Size (in.)	Location in Panel	Comment
170146	1 (Sarasota, I-75 NB over Bee Ridge Road)	1981 (2/12/2000)	6 (satisfactory)	0	34,000 (30)	18 × 24	Edge or corner?	Failure at asphalt patch with full-depth spall repair
170086	1 (Sarasota, I-75 NB over Clark Road)	1980 (11/27/2000)	7 (good)	2 (0.68)	34,000 (30)	36 × 60	Corner support	Localized full-depth CIP Repair
170085	1 (Sarasota, I-75 SB over Clark Road)	1980 (12/20/2000)	7 (good)	4 (0.2)	34,000 (30)	18 × 18	Corner	Asphalt patch adjacent to M1 repair
100332	7 (Tampa Crosstown Viaduct WB Span 38)	1980 (10/2/2002)	5 (fair)	2 (0.55)	23,000 (8)	48 × 30	Near corner	Asphalt patch
100332	7 (Tampa Crosstown Viaduct WB Span 70)	1980 (9/5/2003)	5 (fair)	3 (1.1)	23,000 (8)	24 × 36	Edge	Failed M1 repair with flexible patch material
100332	7 (Tampa Crosstown Viaduct WB Span 39)	1980 (3/5/2007)	5 (fair)	3 (0.21)	23,000 (8)	18 × 8	Edge	Failed localized patch repair
100436	7 (I-75 over East Broadway Avenue CR 574, and CSX Railroad)	1983 (9/12/2007)	5 (fair)	3 (0.54)	45,000 (30)	24 × 24	Edge	Failed localized patch repair

NOTE: ADT = average daily traffic; NB = northbound; SB = southbound; WB = westbound; CR = county road; M1 = repair involving removal of patched concrete section.

can develop in the CIP slab that spans the prestressed beams. This type of cracking has been reported by highway authorities (e.g., in Iowa and Michigan) (*7*).

OBSERVED CRACKING

Observed cracking in the Florida bridges was at variance with the expected crack pattern. Inspection records, which extended over 20 years, were compiled for all 127 deck-panel bridges in Districts 1 and 7. Figure 2 summarizes defects cataloged in the last five inspection reports for a bridge constructed in 1980 that experienced a localized failure 20 years later (Figure 3). The most recent inspection had been conducted just 6 months before failure (*7*).

Figure 2 shows the following:

- Longitudinal cracks developed along beam lines at the top in 1985 and remained dormant for 11 years. The long interval suggests that fatigue was a factor.
- Transverse cracks were first reported in 1998 at the top of the slab and not at the bottom. Transverse cracks crossed longitudinal cracks, which led to spalling.
- Spalling repairs were noted in the 1998 and 2000 reports.

Longitudinal cracks along beam lines indicate a loss of continuity so that the slab acts as simply as support between the girders. This behavior was confirmed in field tests conducted on the Peace River Bridge near Punta Gorda, Florida (*3, 4*). Simple supports imply higher positive moments and zero negative moments that can be resisted readily by the deck slab. This observation is reflected in the inspection reports (Figure 2) in which no deterioration was recorded for 11 years.

Transverse cracking was sporadic and occurred in the top slab. In part, this cracking was reflective (i.e., cracks occurred at the transverse joints of the panel projected on the CIP slab). This type of cracking leads to spalling, especially when transverse and longitudinal cracks intersect.

FDOT Bridge Inspection Report (Deck)

05/08/00

Unit: 0 Decks

ELEMENT/ENV:98/4 Conc Deck on PC Pane 1309 sq.m. ELEM CATEGORY: Deck/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2 % or less of the deck area.	1309	0 Do Nothing

ELEMENT INSPECTION NOTES:

Minor longitudinal and transverse cracks are present on the deck top. Moderate abrasive wear is present throughout. There is a .1m x .1m x 10mm spall with no exposed reinforcing steel at the south end of an asphalt patch at the center of the west lane, 3 m from the Abutment 5 joint. Minor cracks and spalls are present in and on the edges of random patch areas. Minor longitudinal and transverse cracks are present on random deck panels and in random repair areas.

05/04/98

G1.01 DECK (TOP)

The deck top exhibits Class 1 to Class 2 longitudinal and transverse cracks throughout. The longitudinal cracks appear to run over or adjacent to the beams. Repairs made to the deck top in Span 1 exhibit Class 1 to Class 5 cracks and Class 1 spalls along the edges of the repairs. The deck exhibits moderate abrasive wear throughout. There is a deck repair 8m x 1.2 m at Abutment 5.

06/19/96

G1.01 DECK (TOP)/SURFACING

The deck top contains longitudinal class 1 cracks that run along the beams and occasional class 1 cracks at the panel joint. These cracks are due primarily to the deck panel type construction. These cracks have shown no significant change since May 1985.

08/24/94

G1.01 DECK (TOP)/SURFACING

The deck top contains class 1 cracks that run longitudinal along the beams and occasional class 1 cracks at the panel joint. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the May 1985 report and appear to show no change.

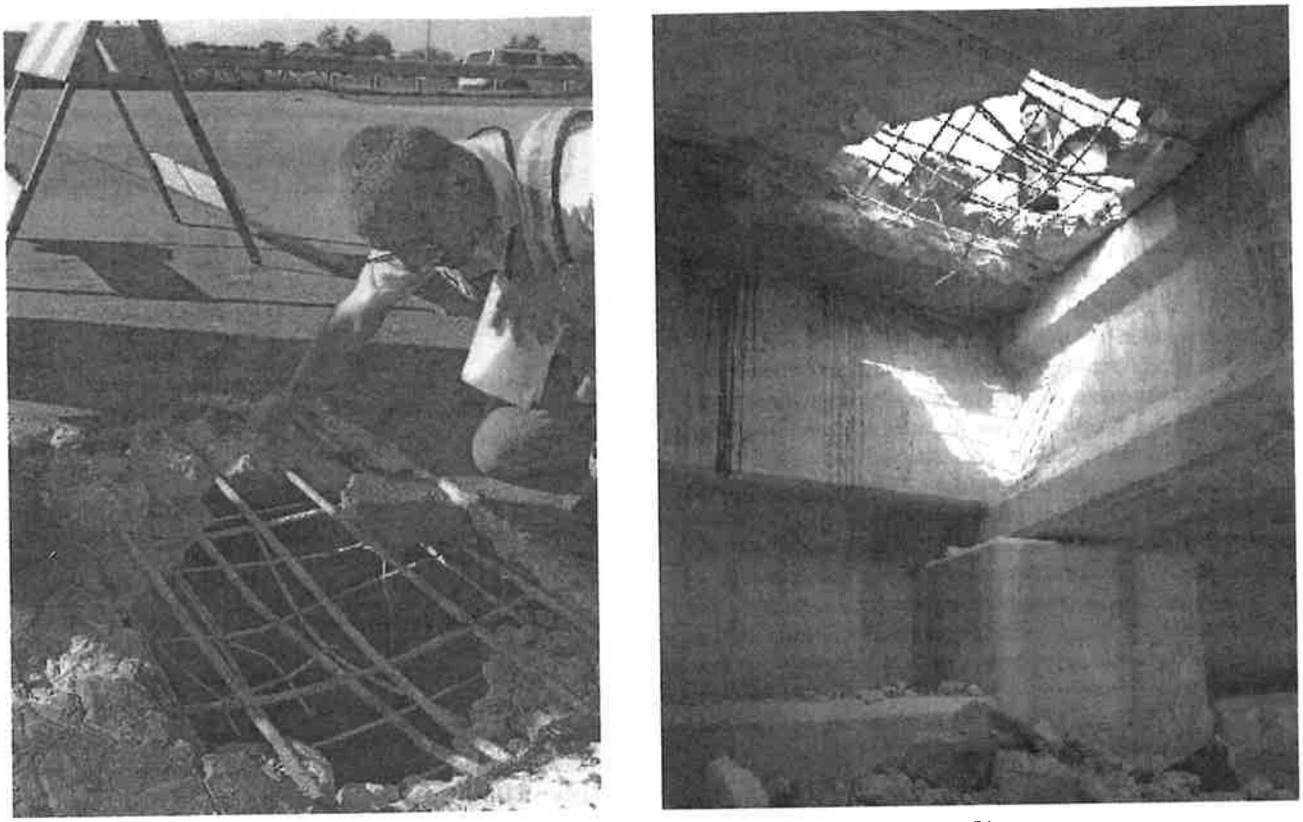
01/04/93

Deck Component

1.01 Deck (Top)

There are Class 1 and 2 cracks that run longitudinally along the beams, with an occasional Class 1 transverse crack at the panel joints. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the report dated 5/85 and appear to show no change.

FIGURE 2 Excerpt from inspection report, Bridge No. 170086, deck assessment.



(a) (b)

FIGURE 3 Failed panel on Bridge No. 170086: (*a*) View 1 and (*b*) View 2. (Source: *Sarasota Herald*.)

SHEAR CRACKING

Shear in beams and slabs is associated with diagonal tension. If a diagonal tension crack extended through the CIP slab to the surface, it would appear as a crack that was parallel to the original longitudinal crack. In the case of shear failure, a second parallel crack would emanate, in addition to the longitudinal cracking first observed (Figure 4).

Shear capacity of the compromised section was exceeded when lane markings coincided with the longitudinal panel joints in the traffic lane that carried the heaviest load (i.e., right lane). With this configuration, wheel loads from trucks needed to be transferred to the prestressed girder through the fiberboard. Code-based calculations indicated that the punching shear capacity in this case was lower than the maximum wheel load, which resulted in localized failure (*7*). In these calculations, the capacity of the CIP concrete was disregarded because of the spalling.

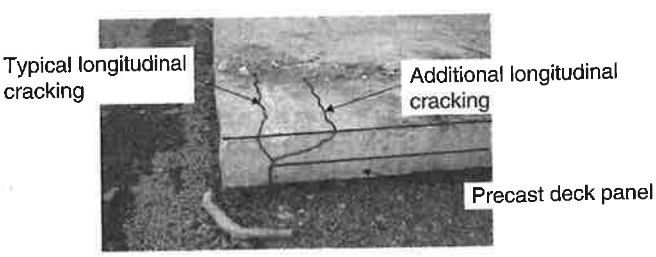


FIGURE 4 Additional longitudinal cracking from shear.

BRIDGE DECK REPAIR METHODS

The Florida DOT allocated $78 million in 2001 to replace the existing precast deck-panel systems on I-75 in Districts 1 and 7 with full-depth CIP concrete decks. Presently, the decks of 51 bridges have been replaced. Most of the funding was consumed in District 1. Because it was difficult to acquire additional funding on account of the sluggish economy, the remaining deck-panel bridges will have to be repaired and rehabilitated, rather than replaced as originally planned.

Repair Types

Eight repair methods have been used by the Florida DOT. Because the underlying effect of the construction error was initially unknown, repairs were undertaken on the basis of individual judgment. The following repairs, employed historically, were reviewed:

- Crack repair,
- Maintenance spall patching,
- Localized spall repair,
- Grout packing,
- M1 repair,
- Full-span M1 repair with grout packing,
- M2 repair, and
- Full-depth bay replacement.

After a better understanding was gained from a prioritization study (*7*), the District 1 and 7 Structures Maintenance Office adopted a policy that only full-depth bay replacement addressed the construction error and thus was the only repair considered permanent.

Crack Repair

Most longitudinal and transverse cracking occurred in the top slab. The cause was determined from finite element modeling (*7*) to be creep-induced by prestressing forces in the precast panel and differential shrinkage between the CIP concrete and the deck precast panel. Once longitudinal cracking begins, sporadic transverse cracks can develop in the deck.

Methods to repair cracks are well known and described in the literature (*12*). Cracks can be repaired by epoxy injections or sealants. Crack injection is a structural repair intended to restore the structural strength of the deck. Sealants penetrate and cover the cracks to avoid the entry of water and other impurities into the deck (*13*). If a crack is active, (i.e., it opens and closes under loading), epoxy crack injection should not be used, because it does not have the flexibility of a sealant. Initially, sealants were used in the early stages of deck-panel deterioration. After the fundamental construction error was identified, however, cracks were no longer sealed, nor were sealants used on any of the seven failed decks (Table 1).

Maintenance Spall Patching

After a second, parallel crack occurs (Figure 4), the concrete trapped between the two cracks, which already is internally cracked, starts to crumble; a spall develops and is repaired by patching. The Florida DOT classifies deck patching on the basis of the depth of the repair (*14*). The most common and simplest repair method is maintenance spall patching, which is used to repair spalls in the CIP portion of a deck. When a deficiency such as a spall appeared on the bridge deck, it was common practice for Florida DOT maintenance crews to patch the spall with flexible, "cold patch," asphaltic concrete.

To patch with asphaltic concrete is not labor-intensive and can be done in a matter of minutes with minimal disruption to the traveling public. It is also an inexpensive procedure. The maintenance crew closes a lane temporarily, if necessary; cleans debris out of the spall with hand tools; and patches the spall with a ready-mix bag. The asphalt patch is intended to minimize immediate danger to the motoring public as well as to avoid an increase in the size and intensity of the spall (Figure 5).

This repair method was never meant to be a permanent solution. The maintenance crew was to return within a week and perform a permanent repair in accordance with the policy of the District Structures Maintenance Office. Sometimes, however, other priorities meant that these temporary patches remained in place for extended periods of time. This practice proved to be detrimental, especially in cases in which asphalt was used to patch spalls inside or adjacent to a deficient repair. Rather than distributing the load evenly, the flexible asphalt, which had negligible compressive strength, would pound the precast panel below the surrounding CIP section, which resulted in an increase in the area and depth of the spalls and in the cracks of the precast panel (*15*).

In the worst cases, the pounding punched a hole through the lower deck panel. Six of the seven punching failures (Table 1) occurred after rainfall. Water filled the crevices between the panel and patch material in which subsequent wheel loading caused pumping action between the asphalt and the precast panel until failure (*15*). Four

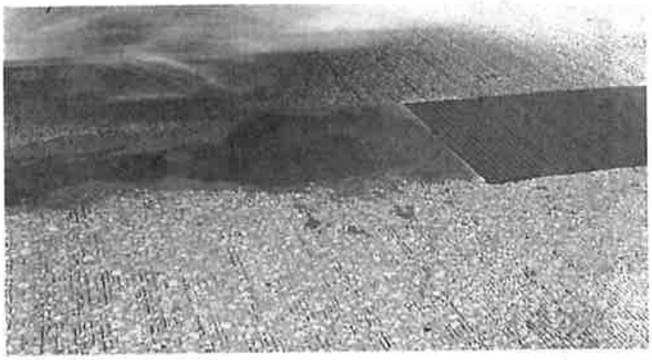


FIGURE 5 Spalls patched with asphalt and walking spalls.

stand-alone asphalt patches and two asphalt patches had been used to address deficiencies within existing CIP repairs.

Localized Spall Repair

Unlike the maintenance spall patch, localized spall repair technically is considered permanent. This repair method is the immediate follow-up to the previously described repair. It is performed with a concrete repair material. Like patching, localized spall repair is not labor-intensive and can be done at a relatively low cost. If high-strength, fast-setting material is used, this repair can be performed during nighttime lane closure to reduce the impact on traffic.

The Florida DOT has had limited success with the longevity of localized spall repairs. Because of the lack of composite action, new spalls can appear after some time in the areas adjacent to the repaired spall. After a spall is created, the residual shear capacity of that region is almost zero, even after it is patched. Therefore, the shear that was to be supported by that region has to be redistributed to sections adjacent to the spall. This creates additional stresses in that region and accelerates its deterioration, which generates new spalls, referred to as "walking spalls." In general, these too were treated with flexible repair material (Figure 5). Better understanding of the underlying causes of spalls, however, has led to a change in the use of localized spall repairs. Once considered permanent, now they are used only as a temporary measure. One of the seven failures reported in Table 1 occurred at an area in which localized spall repair had been done.

Grout Packing

Most deck-panel bridges in Florida were built with fiberboard bearing material to support the precast deck panels on the girders. With this method of construction, no positive (rigid) bearing is provided at the ends of the precast panel. As a result of the effects of creep and shrinkage, initial separation and longitudinal cracks are inherent in precast deck-panel construction. However, the few bridges in Florida that used positive bearing have performed much better and consequently have had longer service lives. The most important conclusion drawn from the forensic study (*7*) was that the lack of positive panel bearing was clearly the main factor responsible for the occurrence of major deck deterioration, such as cracking, delamination, spalling, failing repairs, and, in the worst case, localized punch-through deck failures.

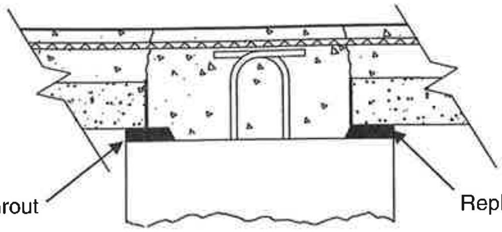


FIGURE 6 Bearing detail after grout packing repair.

Grout packing is a form of repair in which the fiberboard bearing material is replaced with nonshrink Portland cement grout or epoxy grout to provide positive bearing (Figure 6). Grout packing is more cost-effective than other, more involved repair methods and causes little to no disruption to traffic because the work can be performed with a bucket truck or scissor lift from beneath the bridge.

None of the failures reported in Table 1 had grout-packing repairs performed on them. This method was developed to address the initial construction error. To be effective, however, however, grout packing must be applied to a bridge before spalls and failing repairs cause it to deteriorate. In 2000, the Florida DOT performed grout-packing repairs on six bridges on I-75 in the Tampa area. Eleven years later, those bridges still performed satisfactorily (*16*).

M1 Repair

After several patches and repatches, an M1 repair generally is performed in the affected area. The M1 (and M2) were the Florida DOT's recommended methods of repair in the 1980s (*4*). In an M1 repair, all the patched, spalled, and unsound concrete section is removed and replaced by repair material (Figure 7). Unlike localized repairs, the depth of M1 goes to the top of the precast panel. Although M1 repairs hold up better than localized repairs, the edges eventually separate, and walking spalls continue in front of the repaired area because the bridge deck system does not act compositely.

Although this repair method was implemented to address the construction error, it did not work as anticipated. Two of the seven failures described in Table 1 were associated with M1 repairs. On Bridge No. 170085, a walking spall was patched with asphalt adjacent to an M1 repair, and on Bridge No. 100332, Span 70, asphalt was used to patch a deficiency within an existing M1 repair.

Full-Span M1 Repair with Grout Packing

This somewhat modified M1 repair also was used to repair longitudinal spalling along the edge of a beam. The difference is that the CIP concrete portion on top of the precast beams is fully removed, and additional steel is added to the area on top of the beams. The fiberboard bearing material is replaced with nonshrink Portland cement grout or epoxy to provide positive bearing. This repair is extended longitudinally throughout the length of the span.

This procedure is labor-intensive, costly, and disruptive to traffic. However, with the exception of full-depth bay replacement, full-span M1 repair is the next most effective repair method, because it fills the spalled area under the wheel lines with sound, incompressible material and provides positive bearing for the deck panels. Nevertheless, even these repairs can lead to deficiencies, such as longitudinal cracks within or adjacent to them. None of the failures reported in Table 1 was associated with M1 or grout packing as a method of repair.

M2 Repair

The M2 repair method, which was developed specifically for precast deck-panel deficiencies, was not encountered in any of the inspections. M2 repair (Figure 8) is used to fix cracks and spalls along transverse joints of the precast panel. The unsound material is removed approximately 6 in. on each side of the transverse joint,

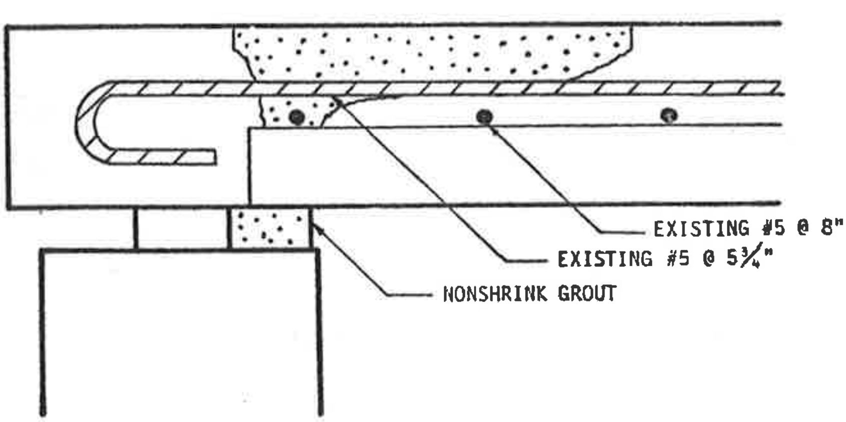


FIGURE 7 M1 with grout packing repair detail (*4*).

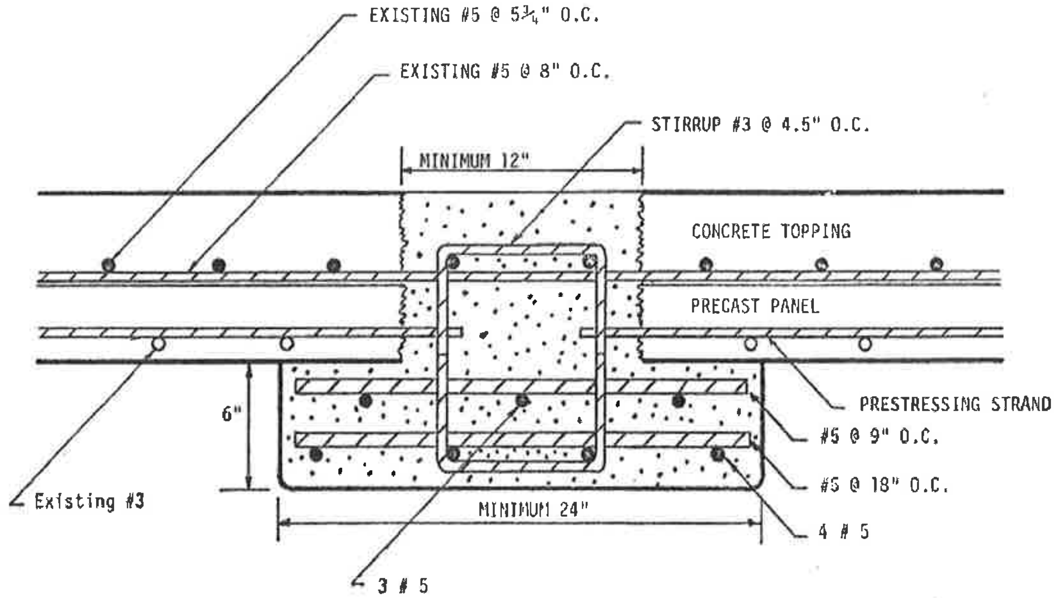


FIGURE 8 M2 repair (*4*).

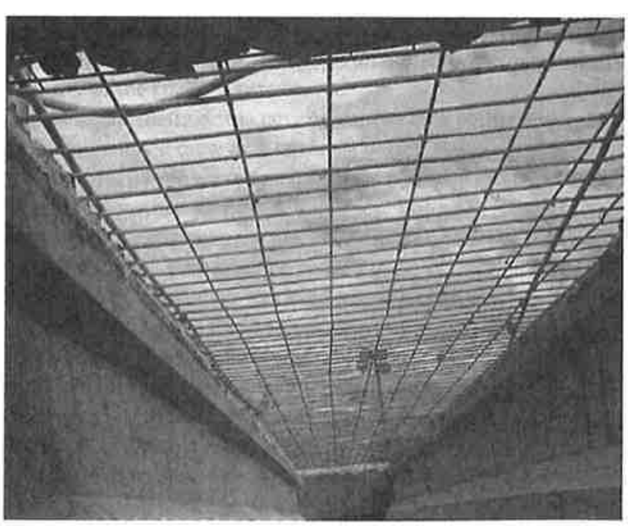
and an inverted T-beam is formed with the bottom of the precast panel that sits on the flange of the inverted T-beam. The flange of the T-beam is required to be at least 24 in. wide. The inverted T-beam is provided a positive bearing on the girders (*4*). M2 repairs are more costly than other repair methods and have an adverse impact on traffic flow.

Full-Depth Bay Replacement

Full-depth bay replacement is the most effective repair method for deficient precast deck-panel bridges (Figure 9). As stated previously, it is the directive of the District Structures Maintenance Office to use this method for all permanent repairs. At a minimum, the repair is done in a bay (the transverse distance between two beams) and throughout the length of the span.

When only one bay is replaced, the CIP concrete and precast panel are demolished, which leaves only the reinforcing steel grid originally within the CIP section for continuity. A new bottom steel mat is designed (Figure 9) and placed as an alternate to the precast panel. A standard compression test is performed to verify that the concrete has gained the required strength before the bridge or repaired area is opened to traffic. None of the failures reported in Table 1 occurred on decks that had been repaired by full-depth bay replacement.

Table 2 provides an overview of the eight repair methods discussed, and highlights their advantages, disadvantages, and effectiveness.



(a)

(b)

FIGURE 9 Full-depth bay replacement repair: (*a*) existing and (*b*) new steel.

TABLE 2 Excerpts from Inspection Report, Bridge No. 170086, Description of Repair Types, Characteristics, and Effectiveness

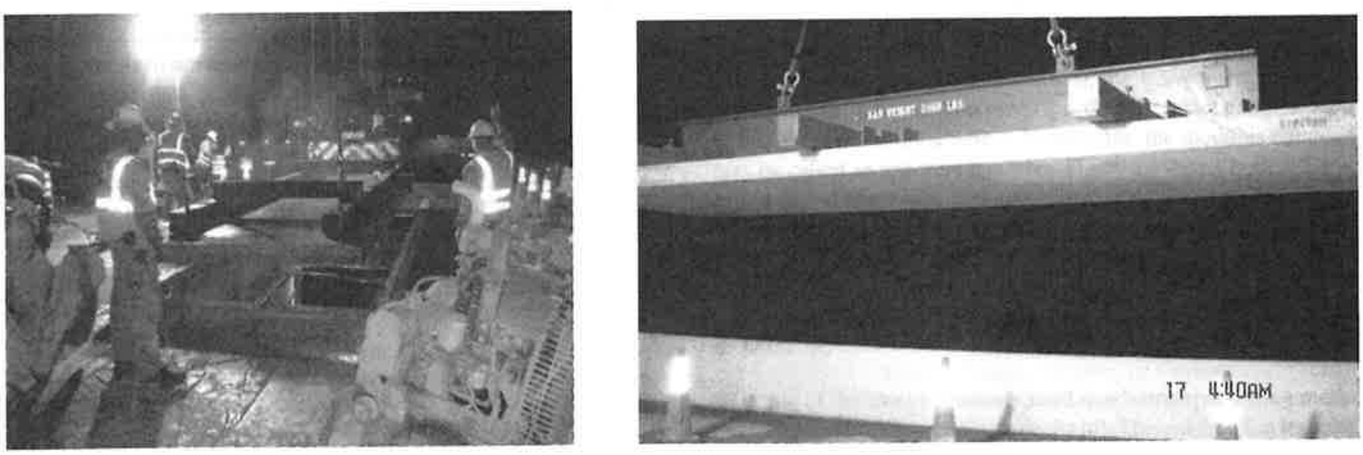
Repair Type	Favorable Characteristics	Unfavorable Characteristics	Effectiveness of Repair
Crack repair	Helps keep out debris and impurities that may accelerate deterioration.	Does not impede the deterioration process or help structurally.	Not effective
Maintenance spall patching (asphalt)	Easy to place without much disruption to traffic. Very inexpensive repair.	Only for temporary use. If left longer than a week, could be detriment rather than a benefit to the bridge.	Not effective
Localized spall repair	Provides a repair with compressive strength in comparison to maintenance patching with asphalt.	The nature of the deck panel system not acting compositely, the localized repairs start to separate at the edges. New spalls described as "walking spalls."	Not effective
Grout packing	Good to slow down deterioration process. Provides positive bearing and extends bridge life. No traffic impact.	Does not mitigate deficiencies that were present before grout packing.	Good to slow down deterioration
M1 repair	Repair replaces deteriorated CIP component by extending to top of precast panel.	Can separate from panel, start to separate at the edges, and new walking spalls start to appear. Process is moderately labor intensive and impacts traffic.	Better than spall repair but not very effective
Full-span M1 repair with grout packing	Repair fixes transverse cracking and spalling along precast panels. It has worked well in other parts of the state.	Process is labor-intensive and affects traffic.	Effective
M2 repair	Lasts longer than any other type aside from full-depth bay replacement.	No bridges with this repair were encountered in study.	na
Full-depth bay replacement	Addresses the root cause of problem: elimination of vertical and longitudinal separation between precast deck panel and CIP surfaces.	Costly, labor-intensive, and causes significant impact to traveling public.	Very effective

NOTE: na = not applicable.

Full-depth bay replacements are problematic if average daily traffic on an Interstate is high, which it was on the I-75 Bridge No. 100436, across East Broadway Avenue and the CSX Railroad. The last recorded failure is shown in Table 1. Traffic analysis and lane closure calculations indicated that I-75 in this area could tolerate lane closures at night only. Therefore, the bridge was temporarily repaired and shored in September 2007, and a repair project was programmed to start construction in late 2009.

The District Structures Maintenance Office hired a firm to develop a pilot project to replace the deficient bays on Bridge No. 100435 and its twin bridge on I-75, Bridge No. 100436, without daytime lane closure (*17*). An innovative method of full-depth bay replacement was devised (*18*) on the basis on the successful findings of a similar system used in other states (*19*). The deteriorated deck was removed and replaced in 30-ft long sections with full-depth, precast concrete panels at night only (Figure 10). Near-surface-mounted bars of reinforced carbon polymer fiber were installed to transfer shear into the existing deck.

Construction took place during the fall of 2009 and was the first time that this method was applied in Florida. The method was unique, in that it transferred forces between the full-depth, precast panels longitudinally, rather than transversely, with the near-surface-mounted bars of reinforced carbon polymer fiber (*19*). The project was successfully completed in spring 2010.



(a) (b)

FIGURE 10 Full-depth panel: (*a*) removal of existing deteriorated deck and (*b*) new installation.

SUMMARY AND CONCLUSIONS

Deterioration of bridge decks caused by the use of compressible bearing material under precast deck panels is difficult to predict. With respect to the exact position of the fiberboard material, little to no positive (rigid) bearing may be present. Its use changes the load path for shear, which causes delamination and loss of composite action between the precast panel and the CIP overlay. This problem tended to manifest itself when lane markings coincided with the longitudinal panel joints (girder line) in the most heavily loaded traffic lane (i.e., right lane) (*7*), although not always. Unfortunately, initial attempts to repair the decks tackled the symptoms, rather than the underlying cause, and were unsatisfactory, which the seven, localized shear failures revealed (Table 1).

A review of eight repair methods employed over two decades indicated that most were unsuitable. Grout packing was a good method, because it provided a rigid shear support for the panels. However, it needed to be done early. Both M1 and M2 repairs with grout packing were acceptable repair procedures. Eventually, however, their effectiveness weakened because of the separation between the precast panel and the CIP section as the result of long-term creep and shrinkage effects (*7*).

Full-depth bay replacement with CIP concrete was the most effective repair method, because it addressed the initial construction error and thereby provided positive bearing and eliminated the vertical and longitudinal interface between the precast deck panel and CIP concrete surfaces. However, the method was difficult to apply to highways with high average daily traffic because of the extended lane closures required to allow concrete to cure. In such conditions, the use of full-depth, precast, modular deck replacement is recommended. A summary is provided in Table 2 of the relative advantages, disadvantages, and effectiveness of the repair methods.

The most important finding of the study was that asphalt patching can actually exacerbate rather than mitigate the deterioration problem. Six of the seven punching shear failures were associated with asphalt patching intended as a temporary repair. Given this history, the use of asphalt (or similar material) should be prohibited, even to patch spalls temporarily in precast deck-panel bridges.

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The Structures Maintenance Committee peer-reviewed this paper.

D. STRUCTURAL DRAWINGS