



**IMPROVEMENT OF QUALITY MANAGEMENT FOR
HIGHWAY AND BRIDGE CONSTRUCTION AND
MAINTENANCE, PHASE II**

**BRIDGE ENGINEERING
INSPECTION MANUAL**

2014

Department of Public Works and Highways
Japan International Cooperation Agency

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DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS

JAPAN INTERNATIONAL COOPERATION AGENCY



FOREWORD

Pursuant to Department Order No. 47, Series of 2004, the Bridge Management System (BMS) was prescribed and adopted as the official procedures or system in conducting inspection, inventory and condition rating of bridges. The BMS aims to support the needs analysis, multi-year programming and annual budgeting for the preservation of our national bridges.

Recognizing the importance and contribution of safe and well-maintained bridges to the steady growth and development of the national economy of the country, this Bridge Engineering Inspection Manuals was developed under the JICA-Assisted Technical Cooperation Project (TCP) on the *“Improvement of Quality Management for Highway and Bridge Construction and Maintenance, Phase II”* (JICA-TCP II). The comprehensive and step-by-step procedure in conducting bridge inspection using Non-Destructive Testing (NDT) equipment are discussed in this manual. The use of this state-of-the-art equipment had been demonstrated by the JICA experts to the concerned engineers from the DPWH pilot regions as part of the numerous TCP activities. This manual, therefore, is essentially a compendium of the pertinent activities under the JICA-TCP II regarding bridge inspection.

It is hoped that this manual shall serve as an effective reference material to engineers and other technical personnel involved in the inspection/assessment of bridges.


ROGELIO L. SINGSON
Secretary

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This Bridge Engineering Inspection Manual for the BMS Type 3 Inspection is one of several manuals developed by the Japan International Cooperation Agency (JICA) together with the Department of Public Works and Highways (DPWH) during the implementation of the Technical Cooperation Project for the Improvement of Quality Management for Highways and Bridge Construction and Maintenance, Phase II (2011-2014). Special Thanks to the following individuals who contributed to the development of this very important manual:

Mr. Hideo Nagao	JICA Team Leader/ Long Term Bridge Expert
Mr. Takeyuki Takada	JICA Short Term Bridge Expert
Ms. Judy F. Sese, PhD	Project Manager, Bureau of Research & Standards
Ms. Carolina S. Canuel	Deputy Project Manager, DPD, Planning Service
Mr. Aristarco M. Doroy	TWG Member, Bureau of Construction
Mr. Adriano M. Doroy	TWG Member, Bureau of Design
Ms. Edna F. Meñez	TWG Member, Bureau of Maintenance
Mr. Felipe S. Ramos	TWG Member, Bureau of Research & Standards
Ms. Elsa T. Naboye	Regional Project Manager, CAR
Ms. Ramie B. Doroy	Regional Project Manager, Region VII
Ms. Rowena P. Jamito	Regional Project Manager, Region XI
Mr. Emmanuel A. Adriano	CWG Member, DPD, Planning Service
Mr. Rufino D. Valiente	CWG Member, Bureau of Design
Mr. Bobby Z. Fodulla	CWG Member, Bureau of Research & Standards
Ms. Nelia I. Antonio	CWG Member, Construction Division in Region CAR
Ms. Feliciano R. Espina	CWG Member, Planning & Design Sec., Negros Or. 1 st DEO
Ms. Alvin C. Cabueñas	CWG Member, Maintenance Division in Region XI
Ms. Mary Ann T. Bautista	TCP Coordinator, DPD, Planning Service
Mr. Rodrigo A. Yago	JICA Team, Asst. Engineer 1

ABBREVIATIONS

ASTM	:	American Society for Testing and Materials
AASHTO	:	American Association of State Highway and Transport Officials
BMS	:	Bridge Management System
BRM	:	DPWH-JICA Bridge Repair Manual
BS	:	British Standard
CWG	:	Counterpart Working Group
DEO	:	District Engineering Office
DO	:	Department Order
DPWH	:	Department of Public Works and Highways
EIF	:	Engineering Inspection Form
HTB	:	High Tension Bolt
JICA	:	Japan International Cooperation Agency
JIS	:	Japanese Industrial Standards
KPa	:	Kilo-Pascal
MPa	:	Mega-Pascal
MLRB	:	DPWH-JICA Manual for Load Rating of Bridges
NDT	:	Non-Destructive Test
ODA	:	Official Development Assistance
PC	:	Prestressed Concrete
PCDG	:	Prestressed Concrete Deck Girder
pH	:	Potential Hydrogen
PPP	:	Public-Private Partnership
RC	:	Reinforced Concrete
RCDG	:	Reinforced Concrete Deck Girder
Rebar	:	Reinforcing Steel Bar
RO	:	Regional Office
SO	:	Special Order
SONAR	:	SOund Navigation And Ranging
TCP-I	:	Improvement of Quality Management for Highway and Bridge Construction and Maintenance, Phase-I
TCP-II	:	Improvement of Quality Management for Highway and Bridge Construction and Maintenance, Phase-II
TWG	:	Technical Working Group
WB	:	World Bank

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BRIDGE ENGINEERING INSPECTION MANUAL

(BMS TYPE 3 BRIDGE ENGINEERING INSPECTION)

CHAPTER 1 ENGINEERING INSPECTION

1.1 Background

The Philippines has a huge stock of bridges along local and national road networks. There are about 8,000 bridges under the responsibility of DPWH, 90% of which are concrete bridges and 10% are steel bridges including temporary bridges. Majority of these bridges were constructed through Official Development Assistance (ODA), before and after the World War II.

DPWH established the Bridge Management System (BMS) in 2003 supported by World Bank (WB) and Asian Development Bank (ADB) under the Road Information Management and Support System and implemented through Department Order No.47 in 2004. BMS was revised in 2007 which focused on the procedures in the conduct of bridge condition inspection and condition rating. Other revisions in BMS consisted of reduction of types of bridge inspections from seven to five types. The five types of bridge inspection are: Type 1 – Routine Inspection, Type 2 – Condition Inspection, Type 3 – Engineering Inspection, Type 4 – Emergency inspection and Type 5 – Inventory Inspection.

Subsequently, JICA Technical Cooperation Project entitled “Improvement of Quality Management for Highway and Bridge Construction and Maintenance, Phase I (TCP-I)” had been implemented in DPWH from 2007 to 2010 for capacity development of DPWH engineers. During TCP I, bridge maintenance manuals/guidelines such as Bridge Inspector’s Handbook, 1st Mandaue-Mactan Bridge Inspection Manual, Marcelo Fernan Bridge Inspection Manual and Bridge Repair Manual were developed to enhance DPWH maintenance techniques. Non-Destructive Test (NDT) equipment were donated to DPWH for use in Engineering Inspection and the corresponding on-the-job trainings on the usage of these NDT equipment were provided during TCP-I. Furthermore, DPWH created Inspectorate Teams and Advisory Team for the conduct of Type 3 - Engineering Inspection through S.O.No.307 series 2010.

However, Engineering Inspection in BMS only describes the outline of procedures of bridge inspection, and the present Engineering Inspection does not include procedures in conducting non-destructive testing and assessment of inspection results. Thus, the improvement of Engineering Inspection Manual is necessary and should be completed as soon as possible.

JICA Technical Cooperation Project “Improvement of Quality Management for Highway and Bridge Construction and Maintenance, Phase II” has developed this Bridge Engineering Inspection Manual. JICA Project Team and Counterparts of TCP-II cooperated in developing the manual.

1.2 Purpose

Bridge Engineering Inspection is intended to obtain detailed data regarding deterioration and performance of the structure when visual inspection is insufficient for estimating the deteriorating mechanism and assessing/judging the state of deterioration and performance degradation, or when the performance due to damage or deterioration are not evident.

Bridge Engineering Inspection is required to be undertaken as a follow-up to bridge condition inspection when the overall condition of the bridge with Poor or Bad rating is recorded based on the result of the condition rating of any primary component/attribute and/or secondary component/attribute that affects structural performance such as bearings/restraints.

The purposes of Bridge Engineering Inspection are as follows:

- 1) To confirm the results of bridge condition inspection;
- 2) To undertake additional investigations necessary for evaluation of defects including physical testing and/or structural analysis, and documentation of the observed defects with sufficient information for future design of required major maintenance works;
- 3) To determine the cause/mechanisms of damage;
- 4) To assess the current bridge structural condition, behavior and load capacity;
- 5) To assess the rate of deterioration and residual life expectancy; and
- 6) To evaluate the need for repair and rehabilitation.

1.3 Process

The general process for a bridge engineering inspection is shown in Figure 1-1.

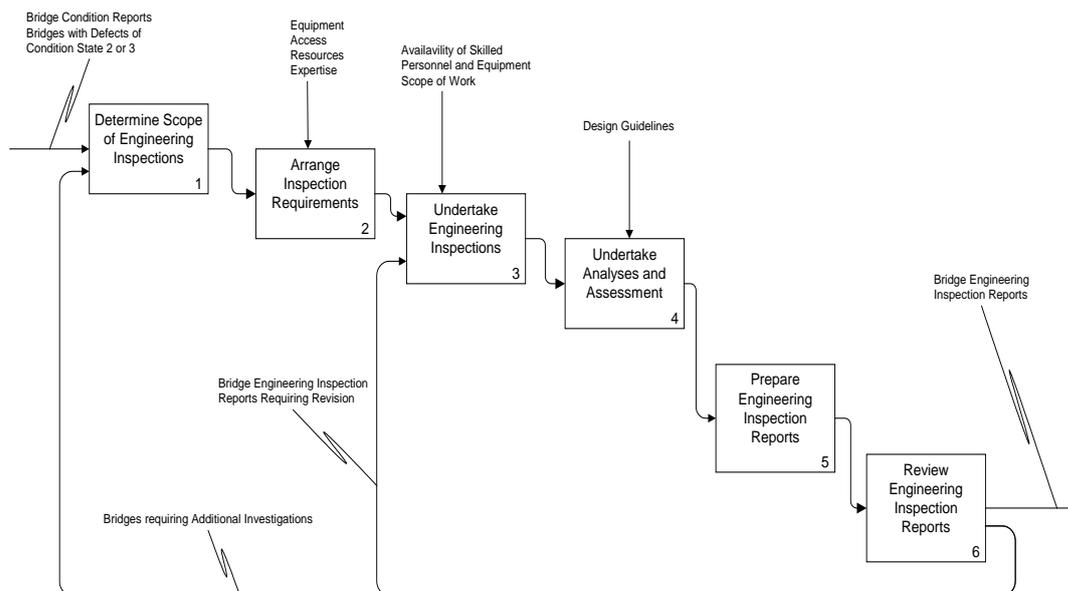


Figure 1-1 Process of Bridge Engineering Inspection

The step-by-step procedure for conducting the Engineering Inspection is illustrated in the following flowchart.

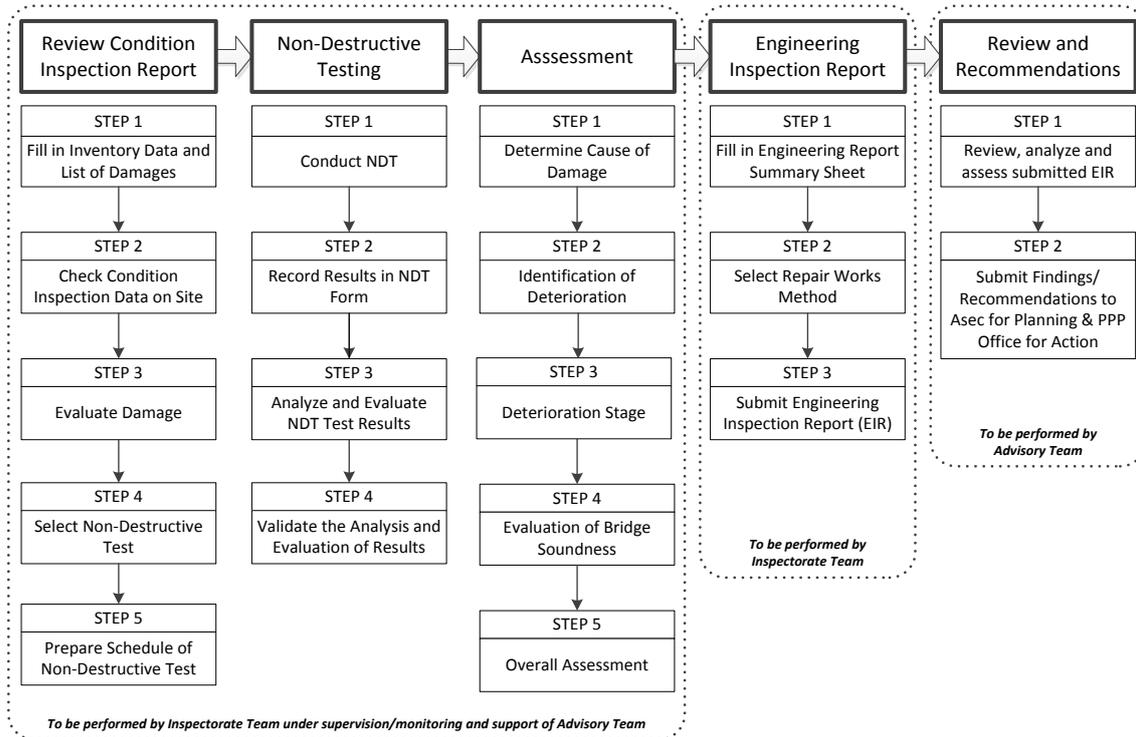


Figure 1-2a Flowchart of Bridge Engineering Inspection

This is a generic inspection model as the scope and extent of the Engineering Inspection will vary depending on the nature, extent, magnitude and severity of the observed defects. This type of inspection shall be undertaken by the Inspectorate Team and Advisory Team who would develop the appropriate scope necessary to address the identified defects and any particular bridge requirement.

1.4 Scope

The Engineering Inspection is a detailed investigation and evaluation of damage and/or material properties of specified elements. The damage may be due to environmental impact (climate, saline soil, etc.), wear (insufficient maintenance), design and construction errors and overloading or similar conditions.

The scope of the Engineering Inspection will include:

- 1) Review of any previous inspection reports;
- 2) Determination and provision of equipment and resources required for the inspection including preparation of a safety plan;
- 3) Inspection of all relevant bridge attributes including measurements, testing and analysis as necessary to supplement the visual inspection; and
- 4) Identification of the probable causes and projected rate of deterioration and the effects of continued deterioration on the performance, durability and residual life of the structure.

If Special Non-Destructive Tests are conducted, scope will include:

- 5) Review of calculation of Load Rating (refer to DPWH-JICA Manual for Load Rating of Bridges, 3rd Edition, 2014);
- 6) Determination of material properties and structural behavior;
- 7) Identification of components limiting the performance of the structure due to the bridge current condition and capacity or are likely to deteriorate to such a level within the next five years;
- 8) Identification of factors which will influence the dynamic load allowance to be used in Load rating.

The Engineering Inspection will also be undertaken in accordance with the requirements of the BMS Operation and User manual.

1.5 Procedure

1.5.1 Selection of Inspection Team Members

In order to evaluate and assess bridge soundness, inspectorate and advisory teams are created to conduct the Bridge Engineering Inspection as follows:

A. INSPECTORATE TEAM

Team Leader: BMS Coordinators (Regional Office Concerned)

Member : Accredited Bridge inspectors (District Engineering Office Concerned)
: Bridge Designer (Regional Office Concerned)

B. ADVISORY TEAM

Team Leader : BMS Manager/ Bridge Inspector (Lead Office; Planning Service: PS)

Member : Bridge Design Engineer (Bureau of Design: BOD)
: Materials Engineer (Bureau of Research and Standards: BRS)
: Maintenance Engineer (Bureau of Maintenance: BOM)
: Representative (Office of the Assistant Secretary for PMO-Operations)

1.5.2 Preparations Before Inspection

- 1) Inspectorate Team collects the Condition Inspection Report and all available documents (plans, drawing and reports) of the bridge to be inspected.
- 2) Inspectorate Team requests District Office to clear bridge and surrounding areas that may hamper the Engineering Inspection activity.
- 3) Inspectorate Team checks equipment (including all Non-Destructive Apparatus) and/or vehicles required for Engineering Inspection.
- 4) Inspectorate Team checks that all necessary materials and safety gadgets are prepared.
- 5) Inspectorate Team prepares Engineering Inspection Form (EI-01) and fills out Defect Condition Inspection Record.
- 6) Inspectorate Team makes sure that safety measures are in-place prior to inspection.

1.5.3 Review of Damage Rating of Condition Inspection Results

In the review of damage rating of condition inspection results, Inspectorate Team shall visually observe the bridge defects on site and undertake the following:

- 1) Fill in Engineering Inspection Form EI-02a (attribute, type of material, type of damage, damage rating) for each damage rated 2 or 3 only.
- 2) Verify the damage using BMS condition rating criteria.
- 3) Evaluate the damage condition such as:

A: Structural: Mark if damage is caused by structural problem. For the description of structural cracks, refer to Chapter 3.

(Concrete Primary Attribute : Cracking, Rebar Exposure/Corrosion)

(Steel Primary Attribute : Corrosion, Cracking, Deformation/Buckling, Loose Connection/Abnormal Vibration)

B: Chloride/Carbonation: Mark if bridge location is close to the coastal area and/or hot spring area. Coastal areas as defined in this manual are those areas located within one (1) km distance from shore line.

(Concrete Primary Attribute : Cracking, Rebar Exposure/Corrosion, Spalling, Honeycomb, Scaling, Delamination)

(Steel Primary Attribute : Corrosion)

C: Progress: Mark if damage has worsened.

(Concrete Primary Attribute : Cracking)

(Steel Primary Attribute : Corrosion, Cracking, Deformation/ Buckling, Loose Connection)

D: Accessibility: Mark if location of damage is not accessible to the inspector

(Concrete Primary Attribute : Spalling, Void, Scaling, Delamination)

E: Load Carrying Capacity, Abnormal Vibration: Mark if observed

F: Underwater: Mark if damage is suspected. (Scouring/Settlement)

- 4) Select the types of Non-Destructive Tests to be conducted

If the cause of defects is clearly visible or if the degree of progress of deterioration is obvious, it may not be necessary to conduct NDT (For example: Water Leakage on Deck Slab, Structural Cracks with more than 1.0mm width).

1.5.4 Selection of Non-Destructive Test Methods

Table 1-1 Matrix of NDT Methods

NDT Methods	Damage Condition					
	A	B	C	D	E	F
I. Standard NDT Methods						
A. For Concrete Attributes						
N01 Rebound Hammer Test	**	**	**		*	
N02 Carbonation Depth Measurement Test	**	**	**			
N03 Ultrasonic Pulse Velocity Test	**	*	**			
N04 Half-Cell Electrical Potential Test	*	**	*			
N05-01 Rebar Detection Test Magnetic Type (For Superstructure)	**	**	**		*	
N05-02 Rebar Detection Test Radar Type (For Substructure)	**	**	**		*	
N06 Infrared Thermal Image Test		*		**		
N07 Compressive Strength Test	*	*	*		*	
B. For Steel Attributes						
N08 Paint Thickness Test		*				
N09 Metal Thickness Test	**	*				
II. Special NDT Methods						
S01 Ultrasonic Flaw Detection Test (For Cracks of Steel Attributes)	**		**			
S02 Static Load Carrying Test					*	
S03 Dynamic Load Carrying Test					*	
S04 Ambient Vibration Test (For Superstructure)	*					
S05 Impact Vibration Test (For Substructure)	*				*	
S06 SONAR for Scouring Test						*
S07 Impact Echo Test (For PC structure)	*				*	
S08 X Ray Test (For PC structure)	*				*	
S09 In-Situ Stress Measurement Test (For PC Structure)	*				*	

** : Required,

* : Depending on the recommendation of Advisory Team

1.5.5 Special Investigation

If it is necessary to verify the damage condition under the ground, Inspectorate Team shall recommend the conduct of subsurface exploration as special investigation.

1.5.6 Non-Destructive Testing

- 1) Inspectorate Team shall conduct selected Non-Destructive Tests.
- 2) After finishing the tests, Inspectorate Team shall fill in the results to data sheet.
- 3) Inspectorate Team shall analyze the result and evaluate overall bridge condition
- 4) Detailed process of Non-Destructive Tests is described in CHAPTER 2.
- 5) Inspectorate Team shall ask technical advice from the Bureau of Design, if necessary.

1.5.7 Assessment

- 1) Inspectorate Team shall analyze the overall assessment.
- 2) Detailed procedures of Assessment are described in CHAPTER 3.

1.5.8 Reporting

Inspectorate Team shall submit the Engineering Inspection Report to Advisory Team promptly.

1.5.9 Flowchart in Accomplishing the Forms

The process in accomplishing the inspection forms are shown in the figure below.

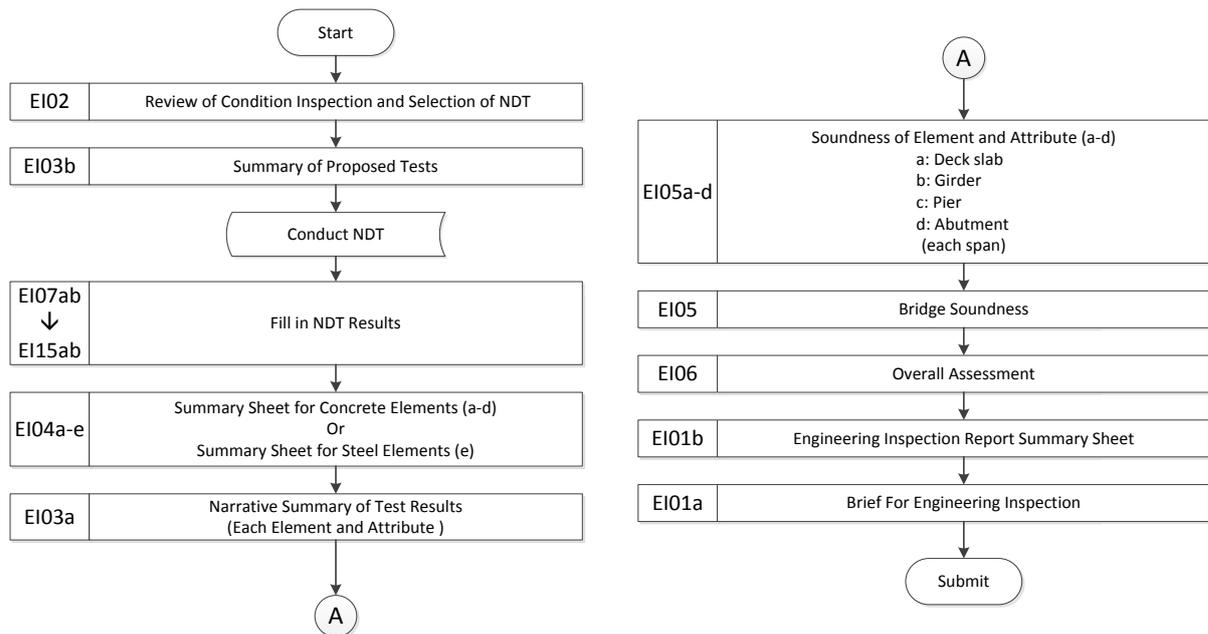


Figure 1-2b Flowchart in Accomplishing Inspection Forms

1.6 Deliverables

The deliverable will be the Engineering Inspection Report. This will be an official report discussing all aspects of the Engineering Inspection. The report shall include recommendations such as:

- 1) Immediate remedial action;
- 2) Access restrictions, including load and speed limits, vehicle paths or number of lanes and detour if applicable;
- 3) Further investigation, testing and structural analysis;
- 4) Further inspection and monitoring period; and
- 5) Load limit

The report will include the Engineering Inspection Report Summary Sheet, as shown in Figures 1-3a and 1.3b, and Reporting Forms of Engineering Inspection Results as shown in Figures 1-4 to 1-8e including all inspection records. Prescribed inspection forms are shown in APPENDIX A and a sample inspection report is shown in APPENDIX B of this manual.

Data recording will be similar to that required for Condition Inspection with additional references as to the types and locations of all testing and sampling conducted as part of inspection. Component designation and condition rating shall be identified in accordance with Condition Inspection procedures.

The original Engineering Inspection Report shall be retained in the Regional Offices for future reference and use. A copy of the report shall be submitted to the Advisory Team.

1.7 Tasks/Activities of ADVISORY TEAM

- 1) Supervise/monitor the activities of the Inspectorate Team;
- 2) Provide the requirements (i.e. expertise, skilled personnel, equipment, etc.) to the Inspectorate Team if necessary;
- 3) Review, analyze and assess the Engineering Inspection Report submitted by the Inspectorate Team; and,
- 4) Submit findings and/or recommendations to the Assistant Secretary for Planning and PPP for appropriate action necessary for the success of the operations of the BMS.

Figure 1-3a Brief for Engineering Inspection

Form EI-01a

Department of Public Works and Highways

BRIEF FOR ENGINEERING INSPECTION

FISCAL YEAR: <insert budget year>

REGION: VII

ENGINEERING DISTRICT: <insert engineering district>

Road ID	
Section ID	
Road Name	
Bridge ID	
Bridge Name	
Type of Bridge (Bridge Code)	

Conclusions of Current Bridge Condition Report

Overall Condition of Bridge	
Recommended Action to Bridge	
Major Reason for Recommendation	
Estimated Remaining Bridge Life	

Previous Engineering Inspection Report

Prior Engineering Inspection report Available?	
Date of prior Engineering Inspection Report	
Type of Engineering Inspection Report Required	

DATE OF ISSUE: <insert date of printing from BMS>

Report BRE_01

Figure 1-3b Engineering Inspection Report Summary Sheet

Form EI-01b

Department of Public Works and Highways

**ENGINEERING INSPECTION REPORT
SUMMARY SHEET**

BRIDGE INFORMATION

Bridge ID		Region	
Bridge Name		Engineering District	
Road Name		Province	
Road ID		Congressional District	
Section ID		Municipality	
Location		Barangay	

CONCLUSIONS FROM FINDINGS OF ENGINEERING INSPECTION

Recommended Work:		Overall Assessment	
None		Estimated Remaining Bridge Life (years)	
Major Maintenance		Comments on recommended work:	
Upgrading			
Replacement			
Estimated Cost of Recommended work			
Urgency of Recommended work			

DATE OF ISSUE:

<insert date of printing from BMS>

**Figure 1-4b Reporting Form of Engineering Inspection Result
(Summary of Tests Conducted)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)							Form EI-02b		
Bridge Name		Bridge ID			Date				
<p>Reporting Form of Engineering Inspection Result (Summary of Tests Conducted)</p>									
For Concrete Bridge									
Bridge Element	Attribute	Type of Material	Type of Test						
			N01	N02	N03	N04	N05-1	N05-2	N06
Span 1	Deck Slab	Concrete							
	Main Member	Concrete Girder							
Span 2	Deck Slab	Concrete							
	Main Member	Concrete Girder							
Span 3	Deck Slab	Concrete							
	Main Member	Concrete Girder							
Abutment A	Main Structure	Concrete							
	Foundation	Concrete Pile							
Abutment B	Main Structure	Concrete							
	Foundation	Concrete Pile							
Pier 1	Main Structure	Concrete							
	Foundation	Concrete Pile							
Pier 2	Main Structure	Concrete							
	Foundation	Concrete Pile							
For Steel Steel Bridge									
Bridge Element	Attribute	Type of Material	NDT						
			N08	N09					
Span 1	Deck	Steel							
	Main Member	Steel Girder/ Truss							
Span 2	Deck	Steel							
	Main Member	Steel Girder/ Truss							
Span 3	Deck	Steel							
	Main Member	Steel Girder/ Truss							
<p>If the cause of defects is clearly visible or if the degree of progress of deterioration is obvious, it may not be necessary to conduct NDT (For example: Water Leakage on Deck Slab, Structural Cracks with more than 1.0mm width).</p>									
Special NDT (If necessary or if recommended)									
Bridge Element	Attribute	Type of Material	Special NDT						

**Figure 1-5a Reporting Form of Engineering Inspection Result
(Bridge Soundness)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)					Form EI-03a	
Reporting Form of Engineering Inspection Result (Overall Stage of Soundness)						
Bridge Name				Bridge ID		
				Date		
Attributes	Carbonation (Table 3.5-3)	Chloride-Induced Deterioration (Table 3.5-7)	Alkali-Silica Reaction (Table 3.5-9)	Fatigue (Table 3.5-11)	Corrosion (Table 3.5-13)	
Deck Slab						
Girder				/		
Pier				/		
Abutment				/		
Overall Stage of Soundness						

NOTE: For overall stage of soundness, select the worst stage

**Figure 1-5b Reporting Form of Engineering Inspection Result
(Stage of Soundness of Attribute – Deck Slab)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)							Form EI-03b			
Reporting Form of Engineering Inspection Result (Stage of Soundness of Element and Attribute)										
Bridge Name		Bridge ID			Date					
Deck Slab	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction		Fatigue		Corrosion	
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Stage of Soundness										

NOTE: For stage of soundness, select the worst stage

**Figure 1-5c Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute – Girder)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)						Form EI-03c		
<p>Reporting Form of Engineering Inspection Result (Stage of Soundness of Element and Attribute)</p>								
Bridge Name		Bridge ID		Date				
Girder	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction		Corrosion	
	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-13)	
Span No. _ Girder No. _	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Span No. _ Girder No. _	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Span No. _ Girder No. _	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Stage of Soundness								
<i>NOTE: For stage of soundness, select the worst stage</i>								

**Figure 1-5d Reporting Form of Engineering Inspection Result
(Soundness of Element and Attribute – Pier)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)					Form EI-03d				
Reporting Form of Engineering Inspection Result (Stage of Soundness of Element and Attribute)									
Bridge Name				Bridge ID			Date		
Pier	Carbonation			Chloride-Induced Deterioration			Alkali-Silica Reaction		
Pier No. ____	Appearance (Table 3.5-1)			Appearance (Table 3.5-5)			Appearance (Table 3.5-9)		
	Rebar Corrosion (Table 3.5-2)			Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)			Soundness (Table 3.5-7)					
Pier No. ____	Appearance (Table 3.5-1)			Appearance (Table 3.5-5)			Appearance (Table 3.5-9)		
	Rebar Corrosion (Table 3.5-2)			Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)			Soundness (Table 3.5-7)					
Pier No. ____	Appearance (Table 3.5-1)			Appearance (Table 3.5-5)			Appearance (Table 3.5-9)		
	Rebar Corrosion (Table 3.5-2)			Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)			Soundness (Table 3.5-7)					
Stage of Soundness									
<i>NOTE: For stage of soundness, select the worst stage</i>									

**Figure 1-5e Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute – Abutment)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)					Form EI-03e				
<p>Reporting Form of Engineering Inspection Result (Stage of Soundness of Element and Attribute)</p>									
Bridge Name				Bridge ID			Date		
Abutment	Carbonation			Chloride-Induced Deterioration		Alkali-Silica Reaction			
Abutment A-1	Appearance (Table 3.5-1)			Appearance (Table 3.5-5)			Appearance (Table 3.5-9)		
	Rebar Corrosion (Table 3.5-2)			Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)			Soundness (Table 3.5-7)					
Abutment A-2	Appearance (Table 3.5-1)			Appearance (Table 3.5-5)			Appearance (Table 3.5-9)		
	Rebar Corrosion (Table 3.5-2)			Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)			Soundness (Table 3.5-7)					
Stage of Soundness									
<i>NOTE: For stage of soundness, select the worst stage</i>									

**Figure 1-6 Reporting Form of Engineering Inspection Result
(Overall Assessment)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)					Form EI-04				
<p>Reporting Form of Engineering Inspection Result Overall Assessment (Based on Table 3.6-3)</p>									
Bridge Name				Bridge ID			Date		
Overall Condition Rating from Condition Inspection Report	Overall Stage of Soundness			Overall Assessment					

**Figure 1-7 Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)				Form EI-05	
Reporting Form of Engineering Inspection Result (Narrative Summary of Test Results)					
Bridge Name		Bridge ID		Date	
Bridge Element		Attribute			
N01 Rebound Hammer Test <input type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings:					
Conclusions/Recommendations:					
N02 Carbonation Test					
Findings:					
Conclusions/Recommendations:					
N03 Ultrasonic Pulse Velocity Test					
Findings:					
Conclusions/Recommendations:					
N04 Half-Cell Electrical Potential Test					
Findings:					
Conclusions/Recommendations:					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input type="checkbox"/>					
Findings:					
Conclusions/Recommendations:					
N06 Infrared Thermal Image Test					
Findings:					
Conclusions/Recommendations:					
N08 Paint Thickness Test					
Findings:					
Conclusions/Recommendations:					
N09 Metal Thickness Test					
Findings:					
Conclusions/Recommendations:					

**Figure 1-8a Reporting Form of Engineering Inspection Result
(Summary Sheet for Concrete Elements 1/4)**

Bridge Engineering Inspection Form (TYPE-3)				Form EI-06a		
Inspection Record (Summary Sheet for Concrete Elements 1/4)						
Bridge Name				Bridge ID		
Inspector		Date		Weather		
Bridge Element	Span	Attribute	Deck			
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
Maximum					mm	
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
Maximum					mm	
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
Maximum					mm	
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Carbonation Test		Un-Carbonated Depth (D)		Ultrasonic Pulse Velocity Test		
Degree of Damage				Degree of Damage		
I		D ≥ 25mm		I		
II		D < 25mm		C/2 > D: (C=Conc. cover)		
				C > D ≥ C/2 (C ≥ 40mm)		
				C > D ≥ C/2 (C < 40mm)		
				D ≥ C		
Half Cell Test		Potential Difference Levels (mV) <E>		Rebound Hammer Test/ Compressive Strength Test		
Degree of Damage				Degree of Damage		
I		-200 < E		Fc = Design compressive strength		
II		-200 ≤ E ≤ -350		F = Actual compressive strength		
III		-350 < E < -500		(in MPa)		
IV		E ≤ -500		I		
				II		
				III		
				F ≥ Fc		
				0.8Fc < F < Fc		
				F < 0.8Fc		

**Figure 1-8b Reporting Form of Engineering Inspection Result
(Summary Sheet for Concrete Elements 2/4)**

Bridge Engineering Inspection Form (TYPE-3)				Form EI-06b	
Inspection Record					
(Summary Sheet for Concrete Elements 2/4)					
Bridge Name				Bridge ID	
Inspector			Date	Weather	
Bridge Element	Span	Attribute	Main Member		
Span _ - Girder _	Carbonation Test		Degree of Damage		
	Un-Carbonated Depth				mm
	Ultrasonic Pulse Velocity Test		Degree of Damage		
	Depth of Cracking				mm
	Half Cell Electrical Potential Test		Degree of Damage		
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage		
	<input type="checkbox"/> Compressive Strength Test				
	Compressive Strength				MPa
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing	Minimum		mm
			Maximum		mm
	Size: ____ mm ϕ	Cover	Minimum		mm
			Maximum		mm
	Stirrups	Spacing	Minimum		mm
			Maximum		mm
Size: ____ mm ϕ	Cover	Minimum		mm	
		Maximum		mm	
Infrared Thermal Image Test					
Span _ - Girder _	Carbonation Test		Degree of Damage		
	Un-Carbonated Depth				mm
	Ultrasonic Pulse Velocity Test		Degree of Damage		
	Depth of Cracking				mm
	Half Cell Test		Degree of Damage		
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage		
	<input type="checkbox"/> Compressive Strength Test				
	Compressive Strength				MPa
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing	Minimum		mm
			Maximum		mm
	Size: ____ mm ϕ	Cover	Minimum		mm
			Maximum		mm
	Stirrups	Spacing	Minimum		mm
			Maximum		mm
Size: ____ mm ϕ	Cover	Minimum		mm	
		Maximum		mm	
Infrared Thermal Image Test					
Span _ - Girder _	Carbonation Test		Degree of Damage		
	Un-Carbonated Depth				mm
	Ultrasonic Pulse Velocity Test		Degree of Damage		
	Depth of Cracking				mm
	Half Cell Electrical Potential Test		Degree of Damage		
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage		
	<input type="checkbox"/> Compressive Strength Test				
	Compressive Strength				MPa
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing	Minimum		mm
			Maximum		mm
	Size: ____ mm ϕ	Cover	Minimum		mm
			Maximum		mm
	Stirrups	Spacing	Minimum		mm
			Maximum		mm
Size: ____ mm ϕ	Cover	Minimum		mm	
		Maximum		mm	
Infrared Thermal Image Test					

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)	Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$D \geq 25\text{mm}$	I	$C/2 > D$; (C=Conc. cover)
II	$D < 25\text{mm}$	II	$C > D \geq C/2$ (C \geq 40mm)
		III	$C > D \geq C/2$ (C < 40mm)
		IV	$D \geq C$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

**Figure 1-8c Reporting Form of Engineering Inspection Result
(Summary Sheet for Concrete Elements 3/4)**

Bridge Engineering Inspection Form (TYPE-3)				Form EI-06c			
Inspection Record							
(Summary Sheet for Concrete Elements 3/4)							
Bridge Name				Bridge ID			
Inspector		Date		Weather			
Bridge Element		Pier		Attribute			
Pier No. ___		Carbonation Test		Degree of Damage			
		Un-Carbonated Depth					
						mm	
		Ultrasonic Pulse Velocity Test		Degree of Damage			
		Depth of Cracking				mm	
		Half Cell Electrical Potential Test		Degree of Damage			
		<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
		Compressive Strength				MPa	
		Rebar Detection Test (Radar Type)					
		Horizontal Rebar Size: ___ mm Ø		Spacing		Minimum	
				Cover		Minimum	
		Vertical Rebar Size: ___ mm Ø		Spacing		Minimum	
		Cover		Minimum			
						mm	
				mm			
				mm			
Infrared Thermal Image Test				With extensive delamination			
Pier No. ___		Carbonation Test		Degree of Damage			
		Un-Carbonated Depth					
						mm	
		Ultrasonic Pulse Velocity Test		Degree of Damage			
		Depth of Cracking				mm	
		Half Cell Electrical Potential Test		Degree of Damage			
		<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
		Compressive Strength				MPa	
		Rebar Detection Test (Magnetic Type)					
		Horizontal Rebar Size: ___ mm Ø		Spacing		Minimum	
				Cover		Minimum	
		Vertical Rebar Size: ___ mm Ø		Spacing		Minimum	
		Cover		Minimum			
						mm	
				mm			
				mm			
Infrared Thermal Image Test							

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)	Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$D \geq 25\text{mm}$	I	$C/2 > D$; (C=Conc. cover)
II	$D < 25\text{mm}$	II	$C > D \geq C/2$ (C \geq 40mm)
		III	$C > D \geq C/2$ (C < 40mm)
		IV	$D \geq C$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>	Rebound Hammer Test/ Compressive Strength Test Degree of Damage	Fc = Design compressive F = Actual compressive strength (in MPa)
I	$-200 < E$	I	$F \geq Fc$
II	$-200 \leq E \leq -350$	II	$0.8Fc < F < Fc$
III	$-350 < E < -500$	III	$F < 0.8Fc$
IV	$E \leq -500$		

**Figure 1-8d Reporting Form of Engineering Inspection Result
(Summary Sheet for Concrete Elements 4/4)**

Bridge Engineering Inspection Form (TYPE-3)				Form EI-06d		
Inspection Record (Summary Sheet for Concrete Elements 4/4)						
Bridge Name				Bridge ID		
Inspector		Date		Weather		
Bridge Element	Abutment	Attribute	Main Structure			
Abutment A1	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth			mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking			mm		
	Half Cell Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength			MPa		
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Abutment A2	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth			mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking			mm		
	Half Cell Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength			MPa		
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)	Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	D ≥ 25mm	I	C/2 > D; (C=Conc. cover)
II	D < 25mm	II	C > D ≥ C/2 (C ≥ 40mm)
		III	C > D ≥ C/2 (C < 40mm)
		IV	D ≥ C

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>	Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	-200 < E	I	F ≥ F _c
II	-200 ≤ E ≤ - 350	II	0.8F _c < F < F _c
III	-350 < E < - 500	III	F < 0.8F _c
IV	E ≤ - 500		

**Figure 1-8e Reporting Form of Engineering Inspection Result
(Summary Sheet for Steel Elements)**

Bridge Engineering Inspection Form (TYPE-3)			Form EI-06e
<p>Inspection Record (Summary Sheet for Steel Elements)</p>			
Bridge Name			Bridge ID
Inspector	Date		Weather
Bridge Element	Attribute		
Girder <input type="checkbox"/> Upper Flange <input type="checkbox"/> Web <input type="checkbox"/> Lower Flange	Paint Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Metal Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Cross Beam <input type="checkbox"/> Upper Flange <input type="checkbox"/> Web <input type="checkbox"/> Lower Flange	Paint Thickness Test	
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Metal Thickness Test			
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Truss <input type="checkbox"/> Upper Chord <input type="checkbox"/> Lower Chord <input type="checkbox"/> Diagonal <input type="checkbox"/> Vertical <input type="checkbox"/> Cross Beam		Paint Thickness Test	
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Metal Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Deck Slab <input type="checkbox"/> Deck Plate	Paint Thickness Test	
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Metal Thickness Test			
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%

CHAPTER 2 NON-DESTRUCTIVE TESTING

2.1 Introduction

Once a bridge has been inspected for its overall condition, it is necessary to carry out Non-Destructive Tests (NDT) in order to further refine the diagnostic process if the overall condition is 2 or 3 (Poor or Bad) and if it is suspected that the bridge has been weakened in some way. Non-destructive testing in this context means that the structure is not destroyed though parts of it may be taken away for testing or inspection.

NDT are normally conducted to determine:

- The physical quality of the materials; and
- The position and extent of hidden defects, elements and material boundaries.

NDT are carried out in-situ to provide further information from which an improved diagnosis can be made to enable the bridge engineer to make decisions on the necessary remedial work.

2.2 Non-Destructive Tests

2.2.1 Types of Non-Destructive Tests

There are several types of Non-Destructive Tests. This manual describes the tests using the different non-destructive test equipment that were provided by JICA in TCP Phase I.

Table 2.2-1 Types of Non-Destructive Tests

For Concrete Attributes
N01 Rebound Hammer Test
N02 Carbonation Depth Measurement Test
N03 Ultrasonic Pulse Velocity Test
N04 Half-Cell Electrical Potential Test
N05 Rebar Detection Test
N05-01 Magnetic Type (For Superstructure)
N05-02 Radar Type (For Substructure)
N06 Infrared Thermal Image Test
N07 Compressive Strength Test
For Steel Attributes
N08 Paint Thickness Test
N09 Metal Thickness Test
Special NDT
S01 Ultrasonic Flaw Detection Test (For Cracks of Steel)

S02 Static Load Carrying Test
S03 Dynamic Load Carrying Test
S04 Ambient Vibration Test (For Superstructure)
S05 Impact Vibration Test (For Substructure)
S06 SONAR Test (For Scouring)
S07 Impact Echo Test (For PC structure)
S08 X Ray Test (For PC structure)
S09 In-Situ Stress Measurement Test (For PC Structure)

It should be pointed out that Special NDT of bridges may be costly in many cases but in general, their cost is insignificant compared to the construction of a new bridge. There are a great number of situations when the test results verify the negative results of calculations (e.g., fatigue calculations) and the old bridge may be utilized without any or with only a few limitations without its replacement being necessary. Special NDT can also show the parts of the structure required to be repaired or strengthened. Moreover, Special NDT usually provide information otherwise not obtainable e.g., concerning reduction of statistical uncertainties, ultimate load, deflection, vibration serviceability, data for fatigue analysis.

NDT and Special NDT are integral parts of bridge engineering inspection and estimation. Special NDT (SO1 to SO8) are normally outsourced to firms specializing on these expensive tests. The compilation of operation manuals of all apparatus used in NDT Tests (NO1 to NO9) are printed separately and are not included in this Bridge Engineering Inspection Manual.

2.2.1 Reliability of NDT

Concrete strength, rebar status, painting conditions, etc. on existing bridges and other structures can be checked by NDT.

NDT result may not always be perfect. There is a possibility that the accuracy of NDT outcome could differ depending on the degree of compliance of the prescribed preparatory works and the technical operation of the NDT apparatus. NDT operator has to undergo proper training.

The result therefore of NDT has to be sufficiently evaluated by the Engineer from the objective and technical point of view.

2.3 NDT for Concrete Attributes

2.3.1 Rebound Hammer Test

2.3.1.1 Fundamental Principles

The test method is based on the principle that the rebound of an elastic mass (the hammer piston or impact plunger) depends on the hardness of the material it strikes, and the assumptions that (a) the hardness is proportional to the materials strength and (b) the material is homogenous.

Rebound hammer test can only assess the compressive strength of the near surface layer of concrete in the zone of influence of hammer impact. It is useful in finding weak areas in concrete in a structure.

2.3.1.2 Description of Apparatus



a) Manual Type



b) Digital Type

Figure 2.3.1-1 Typical Concrete Test Hammers

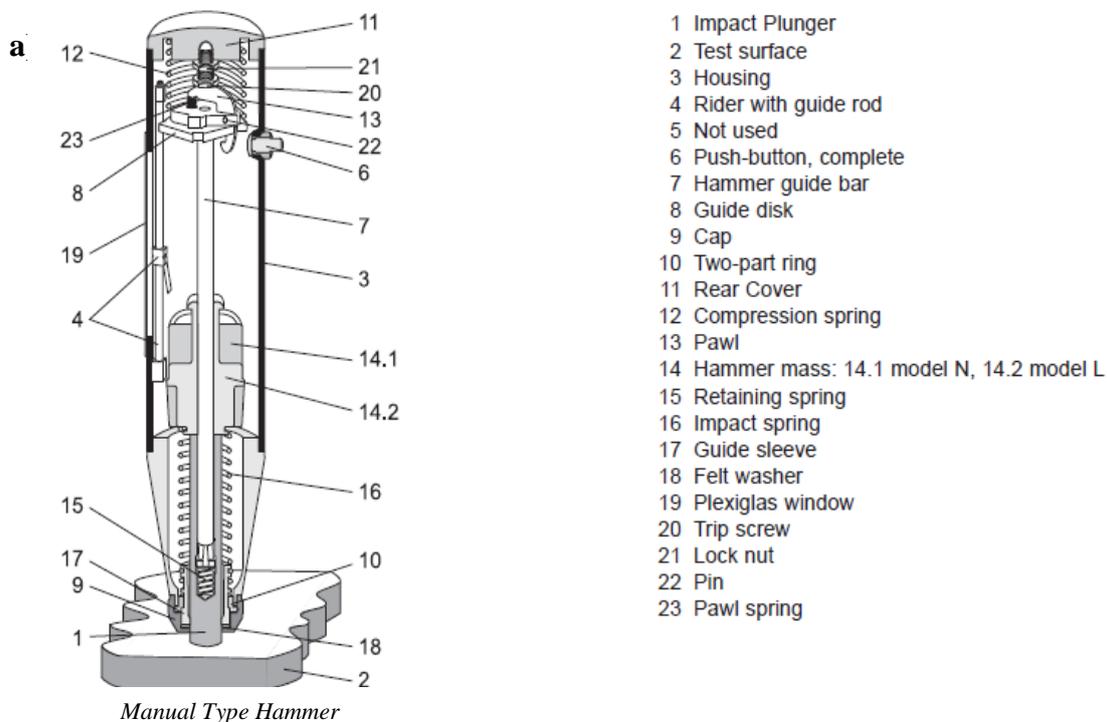


Figure 2.3.1-2 Mechanical Parts of a Concrete Test Hammer

a) Manual Type Concrete Test Hammer

Description

- 1 unit manual concrete test hammer (wt. – 1.7kg)
- 1 unit grinding stone
- 1 unit carrying case: 325 x 125 x 140mm

Specifications

- Impact energy: 2.207 Nm or 2.207 Joules
- Measuring range: 10 to 70 Nm/mm² compressive strength

b) Digital Type Concrete Test Hammer

Description:

- 1 unit electronic concrete test hammer, battery operated, with alpha-numerical display 2x16 characters, keypad with 5 keys and connectors.
- 1 unit battery charger with lead
- 1 unit serial cable for connection of PC
- 1 unit grinding stone

Specifications

- Impact energy: 2.207 Nm or 2.207 Joules
- Measuring range: 10 to 70 Nm/mm² compressive strength
- Dimensions: dia. 54x360 mm long
- Signal conditioning unit for linear potentiometric transducer
- Processor: 16 bit
- Memory ROM: 32 Kbyte
- Memory RAM: 32 Kbyte
- Memory Backup RAM: 16 Kbyte
- A/D resolution: 12 bit
- Accuracy rebound hammer reading: 12 bit
- Accuracy: ± 1 digit
- Standard RS232 Serial output (9600 baud, 8 data bit, 1 stop bit, no parity)

2.3.1.3 Procedures

Explained below are general procedures when using rebound hammer.

a) Gather Reference Information on Structure

Before testing, determine the following information on the structure to be tested:

- Design compressive strength
- Age of concrete (reckoned from date of construction)
- Concrete mix proportions

b) Set-up of Apparatus

After taking out the test hammer from the carrying case, press the plunger head until it is released. Check surface of plunger, and remove any dust or oil with cotton cloth to avoid scratching the surface.

Calibration of the rebound hammer is necessary for its proper operation. A steel anvil is used for verification. Refer to the manufacturer's instruction manual.

c) Preparation of Test Surface

If necessary, grout or plaster that covers the concrete surface should be ground smooth with abrasive stone.

Avoid testing near joints and areas exhibiting honeycomb, scaling and high porosity. In cases where carbonation is present or suspected, the surface layer maybe removed using power grinder to obtain rebound numbers representative of the interior concrete.

Try to avoid tests directly on top of bars. Where possible, use a cover meter to locate bars close to the surface before taking a test or setting out a grid for a number of tests.

Where tests are required on concrete members less than 100mm thick, members should be rigidly supported.

The number of test points shall be determined in the field. Minimum spacing between test points shall be 150 mm.

d) Testing

1) Draw Grid on Concrete Surface to be Tested

See test point example pattern below.

	A	B	C	D
1		25mm		
2	25mm			
3				
4				
5				

Figure 2.3.1-3 Example of Testing Grid

2) Take Measurements

At Cell A1 :

- a) Rest the plunger head at right angles to the concrete surface
- b) Press the hammer until it strikes the surface
(Note: Do not press push-button under any circumstances when taking readings)
- c) The hammer rebounds moving a reference pointer with it
- d) By pushing the button, the pointer is held at its maximum position. Record the reading
- e) Repeat procedures for next measurements until a total of 20 readings (Cells A1 to D5) are taken.



**Photo 2.3.1-1
Testing Position - Manual Type
Concrete Test Hammer**

The hammer can be used to test surfaces at any inclination but only readings at the same inclination (horizontal, up, down, etc.) shall be compared. Test inclination shall be recorded on the test sheet.

3) Test Points

Conduct tests at three (3) test points at each bridge element/attribute (Deck, Girder, Pier and Abutment).

If rebound hammer test is conducted in conjunction with compressive strength test, the test result may be used as reference.

2.3.1.4 Operation and Analysis of Measurements – Manual Type Concrete Test Hammer

a) Operation

The operation of manual type concrete test hammer is explained in Section 2.3.1.3-Procedure

b) Rebound Value and Cube Compressive Strength

The "R" values of rebound have been defined in such a way that they can be converted, by means of diagrams, in terms of resistance to compression for tests carried out on a cube or cylinder. In the diagram below, 5 different curves have been reproduced for cube compressive strength, which take into account the angle of inclination of the hammer. The curves take into account the effect of gravity acting upon the impact shock. Thus the test can be carried out on floors, walls, ceilings and any other inclined surface.

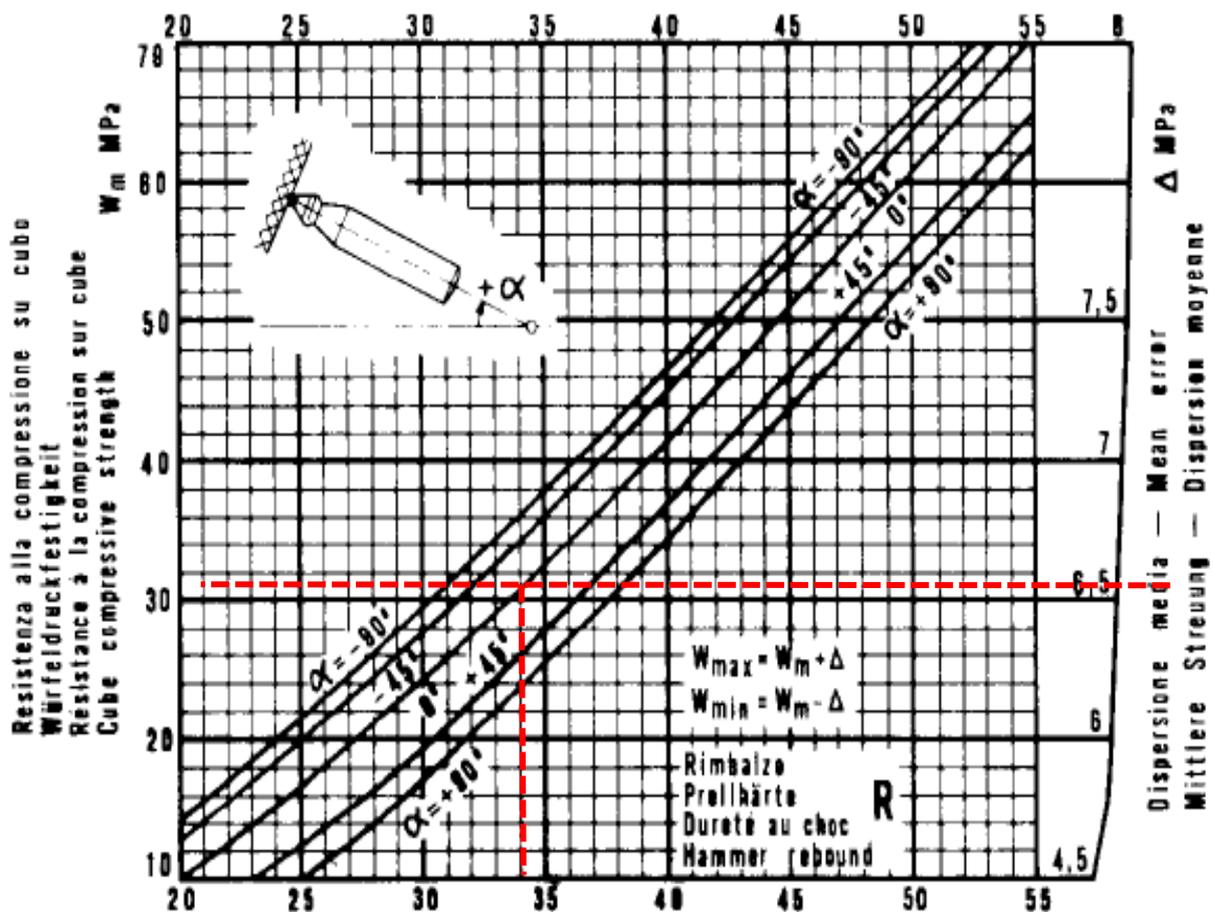


Figure 2.3.1-4 Rebound Value and Cube Compressive Strength Chart

c) Cube Compressive Strength

In the example above, $R = 34$ is the mean or average of 20 readings taken from the test of a vertical wall (angle of inclination $\alpha = 0^\circ$). Projecting a vertical line at $R = 34$ until it

intersects the graph of $\alpha=0^0$ inclination, the resulting mean compressive strength, $f_{c_{mean}}$ (W_m) and Δ (mean error) are obtained .

$$R = 34$$

$$f_{c_{mean}} (W_m) = 31 \text{ MPa}$$

$$\Delta = 6.55 \text{ MPa}$$

$$f_{c_{max}} = 31 + 6.55 = 37.55 \text{ MPa}$$

$$f_{c_{min}} = 31 - 6.55 = 24.45 \text{ MPa}$$

d) Cylinder Compressive Strength

Cylinder strength is 15% lower than that of relative cube strength. Above values are converted to cylinder strength as follows:

$$f_{c_{mean}} (W_m) = 31 \times 0.85 = 26.35 \text{ MPa}$$

$$f_{c_{max}} = 37.55 \times 0.85 = 31.92 \text{ MPa}$$

$$f_{c_{min}} = 24.45 \times 0.85 = 23.60 \text{ MPa}$$

e) Influence of Age of Concrete on Compressive Strength

The strength of concrete decreases as it ages. The time factor applied in calculating the compressive strength of the structure at the time of inspection that can be applied are as follows:

Table 2.3.1-1 Time Factor for Calculation of Compressive Strength

Age (Days)	Time Factor (α)
10	1.55
20	1.15
28	1.00
50	0.87
100	0.78
150	0.74
200	0.72
300	0.70
500	0.67
1,000	0.65
3,000	0.63
10,000	0.57
20,000	0.41

Example: If age of concrete is 20,000 days (54.8 years);

$$F_{c_{mean}} \text{ or } W_m (20,000 \text{ days}) = 26.35 \times 0.41 = 10.80 \text{ MPa}$$

2.3.1.5 Operation and Analysis of Measurements - Digital Type Test Hammer

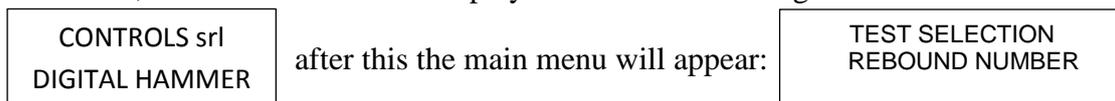
a) Introduction

After extensive testing, research and development aimed at improving the accuracy and reliability of the results, manufacturers like CONTROLS have developed new digital test hammers managed by a microprocessor which can store up to 6000 readings and download these to a printer or PC through a RS232 Serial Port. The integrated Software offers the following options:

- Input via keyboard of number relative to the group of tests to be made (00 to 299) each of which can contain up to 20 tests.
- Selection of input angle (0, +90°, -90°, +45°, -45°).
- Display of present reading (rebound index) and relative compressive strength estimate.
- Selection of measurement unit of compressive strength (kg/cm², MPa, Psi, N/mm²).
- 7 correlation curves rebound index/compressive strength (2 pre-set and 5 to be inserted by operator).
- Coefficients relative to age of concrete result to be obtained (cube or cylinder resistance) and operator experience allows the user to make corrections for errors related to the concrete under examination.
- Automatic processing of test results: calculation of the average value and standard deviation and verification of the significant of the readings.
- Real time display of all results
- Storage of all data and test results organized in groups which are also associated to the data and time of storage.
- Display, printing a selective cancellation and transfer to PC (via RS232 Serial Port) of stored data.
- Time-out for automatic switch-off of instrument after approx. 6 minutes from last operation.
- Easy calibration procedure.

b) Operation

Before proceeding to take measurements the impact rod is placed in its rest position, while releasing the hammer by applying pressure to the impact rod. For the test procedure, please refer to Section 2.3.1.4. To switch on the instruments, press key "MODE I/O". The same key is used to switch off the instrument (keep pressed for at least 3 seconds). When switching off the instrument, release the "MODE I/O" key when the message "SWITCHING OFF" appears on the display. In any case the instrument will switch-off automatically if no operation is made for approx. 6 minutes after switching on the instrument, for a few seconds the display will show the message:



at this point the following selections can be made:

- REBOUND NUMBER is the test menu which allows testing and storage and processing of results.
- CALIBRATION is the calibration menu of the instrument.

- MEMORY/RS232 is the menu which allows access to stored tests so as to display the results, download them via the serial port and delete them.
- SET CURVE is the menu which allows the operator to define a correlation curve based on his needs and experimental experience.
Select the desired function using the “-” and “~” keys. Press **ENTER** to confirm the selection. The desired function will flash on the display for a few seconds before becoming active.

c) Rebound Number

Upon selection of this function the display will show:

Test XXX	Ang. YY
M= NEXT	ent = --->

Test “XXX”: XXX indicates the numerical code which identifies the group of readings in which the test results will be stored. The instrument has a memory which can store 299 test Areas, each of which can contain up to 20 rebound tests and all the test parameters relative to that group of readings. Thus the group refers to a memory cell identified by the assigned number. To input this number use the “-” key to change the digit on which the cursor is positioned, and the “~” to move from one digit to another.

Once the desired test has been selected press **ENTER** to move to the next field. "ANGLE YY which is the impact angle. Use the “-” and “~” keys to select an angle of +90°, +45°, 0°, -45° or -90° (+90° means hammer is vertically upwards, 0° horizontal and -90° downwards, the other values are intermediate).

Press "**ENTER**" if you wish to return to "Test". Press "**MODE**" to move onto the next step. If the selected "test" has already been used or contains data that has not been deleted, the following message appears:

AREA FULL	followed by:	OVERWRITE ? Mode= NO ent=YES
-----------	--------------	------------------------------------

Whereby the operator is advised of the presence of data in the test area and can overwrite and cancel such data (ent=YES) or select another test (mode= NO). The next display shows:

Test XXX	Ang. YY
M= NEXT	ent = --->

By pressing **MODE** it is possible to access the SET parameters menus which are as follows:

- AVERAGE: two selections are possible in the way the average is calculated: M1 calculates the average of all the recorded readings; M2 discards the highest and lowest readings and calculates the average of the remaining readings.
- UNIT: allows the setting of the measurement unit in MPa, kg/cm2, psi and N/mm2. The compressive resistance is estimated by correlation with experimental tests and therefore is only an indicative value. A more accurate estimate can be made by plotting the

characteristic correlation curve of the concrete being investigated with a series of destructive and non-destructive (rebound hammer) test on suitably selected samples.

- **TIME FACTOR:** is the proportional multiple time factor used in the calculation of the resistance to take into account of the age of the concrete. This is initially set as 1. Table 2.3.1-1 suggests some factors that have been obtained by experimentation. They are offered as a guide and are not binding. It is possible, for the operator to establish his own time factors.
- **CORRECTION FACTOR:** is the proportional multiple factor used in the calculation of the resistance to take into account external factors such as humidity which could affect the readings. This is initially set as 1. The use of this factor is left to the discretion of the operator based on local experience. The factor can be set at a value between 0.10 and 1.99.
- **TYPE:** indicates the type of sample: Cube or Cylinder in which to express the readings in.
- **CURVE:** it is possible to select one of the correction curves, 7d or 28d and if inserted UD1, UD2, UD3, UD4 and UD5 which are the five personalized User Defined correlation curves.

To make selection use the “—” key to verify the displayed value and “←” key to move from one figure to another. **ENTER** allows access to the successive selection whilst "**ESC**" exits from the setting menu. Once the parameters have been set, the display will once again show:

<i>Test XXX</i>	<i>Ang. YY</i>
m= SET	ent = TEST

Then press **ENTER** to start a series of readings. The display will show:

Reb. XX.X	n. Y
S=****	Rm=**** ZZ

where:

XX.X is the rebound value of the last test made. When the instrument is in stand by the parameter is shown by "___._".

Y is the number of tests made within a group. This number is updated as the testing is performed.

S is the standard deviation of the readings. This parameters is calculated and displayed from the seventh measurement and above.

Rm is the mean rebound index calculated in basis of the selected mean. This parameter is calculated and displayed from the seventh measurement and above.

Important:

If the impact rod is not in its rest position when the instrument is switched on it will immediately register a reading which will correspond to the last reading made with the hammer.

Next make a reading with the hammer placed on the surface of concrete as described in Section 2.3.1.4.

It should be noted that at least 9 readings should be made in test area so as to obtain reliable results.

The display is updated upon each hammer reading. The reading is updated when the impact rod is in its rest position.

As stated previously, the standard deviation and mean are only calculated from the seventh reading and above. The rebound readings (max. 20) and test parameters are memorized in the "test" and makes the calculations. This is also possible by pressing **ESC** before reaching 20 readings.



**Photo 2.3.1-2 Testing Position -
- Digital Test Hammer**

The screen will show:

```
  **TEST RESULTS**
```

followed by:

```
  S=CCCC Rm=DDDD  
  S=AAAA BBB
```

where:

AAAA is the compressive strength estimated as described above and in accordance with the pre-set parameters.

BBBB is the measuring unit of the compressive strength.

CCCC is the standard deviation of the readings.

DDDD is the mean rebound calculated as basis of the preset method.

The results remain displayed until the "**ENTER**" key is pressed and the following screen appears:

By pressing the mode key, the data is transmitted to the RS232 serial port (see chapter relative to RS232 port).

```
  Test. XXX      Ang. YY  
  m= RS232      ent = END
```

Press **ENTER** to save all the readings made in that "test". The instrument will automatically increase the test number and is thus ready for the next set of readings.

Error Messages

When the results are shown, one of the following error messages may appear:

- **NO DATA !!!** = no reading has been made
- **NOT ENOUGH DATA !!!** = less than 9 readings have been made
- **OFF STANDARD !!!** = where at least 20% of readings vary from the average by 6 or more units. In this case the International Standards indicate that the readings should be discarded and the test repeated. In any case the readings are stored along with the error message but the test data is not explained.

d) Calibration function

This function is used to check, and if necessary adjust the calibration of the test hammer. The display will show:

Rebound n. XX.X

Before proceeding, place the impact rod in its rest position by releasing the hammer by pressing on the rod itself. The display shows the rebound number. When one impact is made the display is updated and maintained until the hammer is released. By making tests on a reference sample (e.g. calibration anvil) it is possible to check the calibration of the hammer.

If the reading differs by a significant amount from the reference value (± 2.0 units) it is possible to adjust the readings so as to bring them to within the limits. This is done with by adjusting the trimmer situated on the rear of the instrument. To access the trimmer the black plastic cover must be removed.

The adjustment is made as follows: make one rebound reading on the calibration anvil and block the release mechanism by pushing the button on the hammer itself. Check the rebound index number shown on the display and adjust it with the trimmer until the reference value appears. Make further tests and adjustments until the hammer is calibrated. Press ESC to exit the calibration function

e) Memory / RS 232 Function

The instrument has a memory for up to 299 Test Areas. Using the memory / RS232 menu it is possible to access all the data present in the memory, display them on the screen and download them to a PC through a RS 232 Serial Port. When this menu is accessed the screen shows:

MEMORY/RS232
Sel. Test: XXX

XXX is the selected test from which it is possible, following the instructions given below, to display all the stored data with relative results and/or download them to a PC through the RS232 Serial Port. The test area can be selected using the "-" and "-" keys. The "**ESC**"

key is used to return to the previous menu. To confirm the desired test area, press “ENTER”, the screen will show:

TEST AREA: XX Download

XXX is the selected test area from which it is possible, following the instructions given below, to display all the stored data with relative results and/or download them to a PC through the RS232 Serial Port.

Using the "-" and "-" keys the following selections are possible:

- download – the PC via the RS232 Serial Port
- delete – cancel all data in selected test area
- view – to visualize all data of selected test area.

Once the desired mode has been selected, confirm with “ENTER”. A further confirmation is requested via the "MODE" key. The operation can be aborted at any time by pressing "ESC".

f) Download Option

To download the data to a PC via the RS232 serial port the test hammer must first be connected using the cable supplied as standard with the machine. Connect the RS232 port to the serial port (COM) of the PC. To receive the data it is recommended that TERMINAL (windows® 3.1 and 3.11) or Hyper terminal (Windows® 95 and 98) is used. Consult the instruction manual of the program used for its correct use. The input parameters must be set as follows: 9600 baud, 8 data bit, 1 stop bit, no parity.

The PC must be ready to receive the data before the command is given to the instrument to transmit the data. The data is in ASCII format, thus once received by the PC, is accessible through any text editor (eg. Word®, Excel®, Notepad®, Wordpad®, etc.). Below are some examples of data transmissions.

```
** REBOUND NUMBER TEST **  
TEST AREA: 1 ANGLE +45 degrees  
DATE 27/06/2000  
TIME 13:05  
REBOUND NUMBER  
47.2  
47.7  
41.9  
50.6  
49.0  
48.4  
43.3  
44.8  
48.2  
51.4  
45.8  
AVERAGE= 47.1 M1 S= 2.8  
TIME FACTOR=1.00
```

CORR. FACTOR=1.00
TYPE= CUBE
CURVE= 28d
sigma = 49.2 Mpa

View option

The view option allows the operator to visualize all the readings of a particular test Area with a format and management similar to the download option. Due to the limited space on the display it is necessary to use the "-" and "~" key to scroll the text. To return to the previous menu press "ESC".

Delete option

The delete function allows the operator to cancel a test Area. Once the area has been selected press "ENTER" to proceed with the cancellation. An extra confirmation is required by pressing "MODE". Then press "ESC" to return to the previous menu.

Error Messages

Selecting an empty test area, the screen will show the flashing message EMPTY for a few seconds. After this another test area may be selected.

g) SET CURVE Function

This menu allows the operator to create up to five user defined (UD) curves of correlation between the rebound index and the compressive strength of the concrete to a maximum of the third degree, setting for each curve the four parameters A, B, C and D relative to the mathematical formula.

Two curves (7d and 28d) are factory set and refer to concrete with an age of 7 or 14 to 56 days. The UD curves have the following form:

$$f_c = A \cdot R^3 + B \cdot R^2 + C \cdot R + D$$

where:

Fc = Compressive resistance (MPa)

A, B, C, D = Coefficient constants to be obtained by experimentation.

R = Rebound index at zero impact angle (i.e. horizontal).

When this menu is entered the display shows

UD X CURVE
m=SET ent=BACK

where X is the number of the selected Curve which can be from 1 to 5 and selected with the "-" and "~" keys. Confirm with "MODE" to proceed to the input of the parameters. The display will show:

A= +000.000000
m=NEXT ent=BACK

Use the “-” key to change the value of the digit on which the cursor flashes and the “ ” to pass from one digit to another. The **MODE** key is used to pass to the next selection, whilst the **ENTER** key allows the operator to return to the previous selection.

It is also possible to change the sign of the value when the cursor flashes on it. At the end of the setting, the instrument exits from the **SET CURVE** menu. Once a curve is defined, its parameters remain in the memory even when the instrument is switched off. All the user needs to do is select the curve in the test menu to use it for the data elaboration.

When a **UD** curve is selected during testing, all the parameters and coefficients that can be set by the user in the test menu as described above (group number, impact angle, type of mean, time factor, correlation factor, measuring unit, sample type...) are used in the same way as for the "7d and 28d" curves.

Therefore, once a curve **UDX** has been defined relative to an impact angle of 0 and value expressed in MPa, it is possible to use this curve for tests at different angles and express results in different units by simply pre-setting the required parameters. Likewise it is possible to apply corrective factors after the test and to select between cube or cylinder resistance.

The **UD** correlation curves defined by the user are obtained by experimentation with rebound hammer and compression tests on concrete samples. MS Excel may be used to elaborate the results and calculate the equation of the relative interpolation curve.

2.3.1.6 Evaluation and Recommendations

- Rebound hammer test is useful in finding weak areas in concrete structures
- Test results depend on angle of inclination of hammer to the horizontal, because of the influence of gravity on the amount of energy imparted to the impact.
- The test only gives indication of surface strength of concrete.

Based on the Manual for Load Rating of Bridges of the DPWH, the concrete design compressive strength used in Philippine bridges are as follows:

<i>Concrete Bending</i>	Year of Construction	Fc (MPa)
	Prior to 1959	17.2
	After 1959	20.7
	1977 to 1981	27.6*
	After 1981	31.0*

**for pre-stressed concrete*

The degree of damage of concrete member based on rebound hammer test results can be determined by the use of the following empirical formula developed in Japan:

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	Fc = Design Compressive Strength F = Actual Compressive Strength from Test (in MPa)
I	$F \geq F_c$
II	$0.8F_c \leq F < F_c$
III	$F < 0.8 F_c$

Example:

Rebound Hammer Test Result (Cube Strength), $f_{c \text{ mean}} (W_m) = 31 \text{ MPa}$

Conversion to Cylinder Strength, $f_{c \text{ mean}} (W_m) = 31 \times 0.85 = 26.35 \text{ MPa}$

F_c prior to 1959 = 17.2MPa, and $0.80F_c = 13.80 \text{ MPa}$

If bridge was constructed prior to 1959, age in days is about 20,000 and

$F_{(20000 \text{ days})} = 26.35 \times 0.41 = 10.80 \text{ MPa}$

Comparing the values, $F_{(20,000 \text{ days})} < 0.80F_c$ and the concrete is considered under Degree of Damage – III.

The test is the first step in the investigation. If the results are unreliable or less than the design compressive strength, detailed investigations including microcore test should be carried out. If the results show abnormal data between three (3) test points, lowest data is indicative of suspected defects in the concrete.

2.3.2 Carbonation Depth Measurement Test

2.3.2.1 Fundamental Principles

Carbonation of concrete occurs when carbon dioxide, in the atmosphere in the presence of moisture, reacts with hydrated cement minerals to produce carbonates, e.g. calcium carbonate. The carbonation process is also called de-passivation. Carbonation penetrates below exposed surface of concrete extremely slow.

The significance of carbonation is that the usual protection of reinforcing steel generally present in concrete due to the alkaline conditions caused by hydrated cement paste is neutralized by carbonation. Thus, if the entire concrete cover over the reinforcing steel is carbonated, corrosion of steel would occur if moisture and oxygen could reach the steel.

2.3.2.2 Description of Apparatus

The 1% Phenolphthalein Solution is made by dissolving 1gm of Phenolphthalein in 90 cc of ethanol. The solution is made up to 100 cc by adding distilled water. The pH value indicates if a solution is acid or alkaline, and therefore corrosion of reinforcing steel bars is determined if possible or not.

- $\text{pH} < 7$: acid
- $\text{pH} = 7$: neutral
- $\text{pH} > 7$ up to 14 : alkaline



Photo 2.3.2-1
Phenolphthalein
Solution

2.3.2.3 Procedures

The freshly extracted core is sprayed with Phenolphthalein solution, and the depth of uncolored layer (the carbonated layer) from the external surface is measured to nearest mm at 4 or 8 positions, and the average is taken. If the concrete still retains its alkaline characteristic, the color will change to red violet. If carbonation has taken place pH will have changed to 7 (i. e. neutral condition) and there will be no color change.

If test sample is cored by coring apparatus:

- Phenolphthalein shall be sprayed on the concrete surface after cleaning.
- The phenolphthalein becomes red-violet when it comes in contact with normally alkaline concrete
- If phenolphthalein becomes colorless, the concrete is carbonated.
- The depth of carbonated concrete shall be measured.

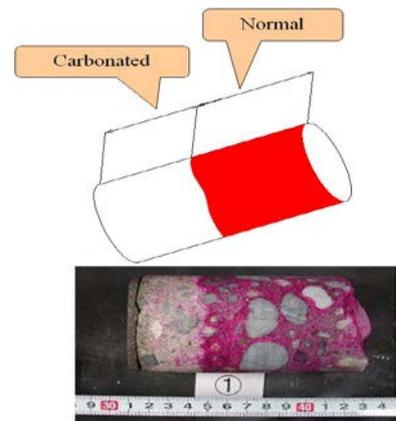


Figure 2.3.2-1 Test for Cored Concrete

Note: When cored sample is used, point load test shall also be carried out on the sample. After coring, the hole shall be sealed by mortar.

If test is to be done in a chipped hole, the dust is first removed from the hole using an air brush and again the depth of uncolored layer is measured at 4 to 8 positions and the average taken. If the concrete still retains its alkaline characteristic, the color will change to red violet. If carbonation has taken place pH will have changed to 7 (i. e. neutral condition) and there will be no color change. See diagram below:

If concrete surface is chipped off by hammer, breaker or other apparatus:

- Spray Phenolphthalein Solution on the chipped surface after cleaning.
- If colorless, the concrete is carbonated.
- The depth of carbonated concrete shall be measured.

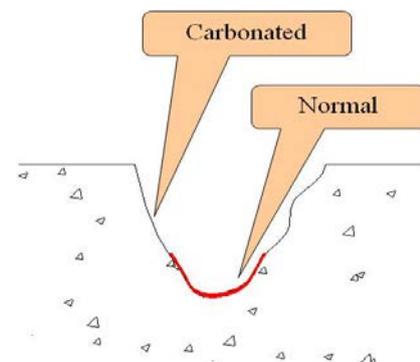


Figure 2.3.2-2 Test for Chipped Concrete

Note: Chipped concrete shall be sealed with mortar after testing

Conduct one test when reinforcing steel bar is suspected of corrosion. Therefore, locate test point near defective site. Test core should not be located where removal of core will affect strength of member (web to end of girder etc.) In such a case, locate test core at another point (curb, etc.).

2.3.2.4 Analysis

a) Compute Carbonation Rate Coefficient and Carbonate Survival Period

It is advisable to determine the carbonation rate coefficient from the carbonation depth measurements, utilizing the fact that the carbonation depth is proportional to the square root of the carbonation period.

$$y = b\sqrt{t}$$

Where:

y: carbonated depth (mm)

b: carbonation rate coefficient (mm/ $\sqrt{\text{year}}$)

t: carbonation period (year)

D: Un-carbonated depth (mm) = C-y

C: Cover

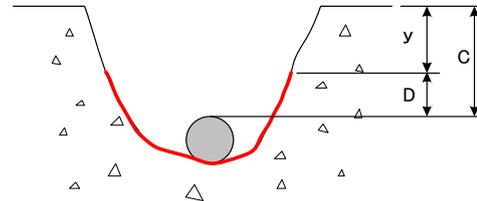


Figure 2.3.2-3 Un-carbonated Depth

2.3.2.5 Evaluation and Recommendations

The result of test is evaluated based on un-carbonated depth as follows:

Table 2.3.2-1 Degree of Damage

Degree of Damage	Un-Carbonated Depth (D)	Status
I	$D \geq 25\text{mm}$	Rebar will be corroded in the future. Monitoring is required.
II	$D < 25\text{mm}$	Rebar may be corroded. Repair is required.

If the Degree of Damage is II, the concrete structure may be affected by chloride deterioration.

2.3.3 Ultrasonic Pulse Velocity Test

2.3.3.1 Fundamental Principles

a) General

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves develops, which include both longitudinal and shear waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. Electronic timing circuits enable the transit time "T" of the pulse to be measured.

b) Applications

The ultrasonic pulse velocity tester is used to measure the velocity of propagation of ultrasonic pulses through concrete for determination of the following:

- Uniformity and homogeneity of concrete;
- Presence of cracks or voids;
- Spalling, delamination, deterioration due to fire, frost or chemical attack;
- Determination of dynamic elastic constants (E and ν); and
- Estimating time for formwork striking

The use of ultrasonic pulse velocity technique to define the extent of internal defects should be restricted to well-skilled personnel.

When an ultrasonic pulse travelling through concrete meets a concrete-air interface, there is negligible transmission of energy across this interface. Thus any air filled void lying immediately between transducers will obstruct the direct ultrasonic beam when the projected length of void is greater than the width of transducers and the wavelength of sound used. Thus the pulse is diffracted around the periphery of the void and transit time will be longer than in similar concrete with no void.

Tests for inspection of delamination, spalling and cracks are described in this section.

2.3.3.2 Description of Apparatus



Specifications

- A. Ultrasonic Pulse Velocity Tester
 - Microprocessor incorporated
 - Transit time measurement : from 0.1 to 1999.9 μ s
 - Pulse Rate: from 1 to 10 per second, selectable
 - Accuracy: 0.1 μ s
 - Transmitter output: 800 v
 - Frequency Range: 24 to 150 kHz
 - Power Supply: Two 1.5 V Alkaline "D" Type Batteries
- B. Tester is supplied complete with:
 - Two 54 KHz Transducers (Transmitter and Receiver) with 2 meter cable
 - Coupling Agent (250 cc bottle)
 - Nylon Carrying Case

Figure 2.3.3-1 Ultrasonic Pulse Velocity Tester

2.3.3.3 Procedures for Investigation of Defects

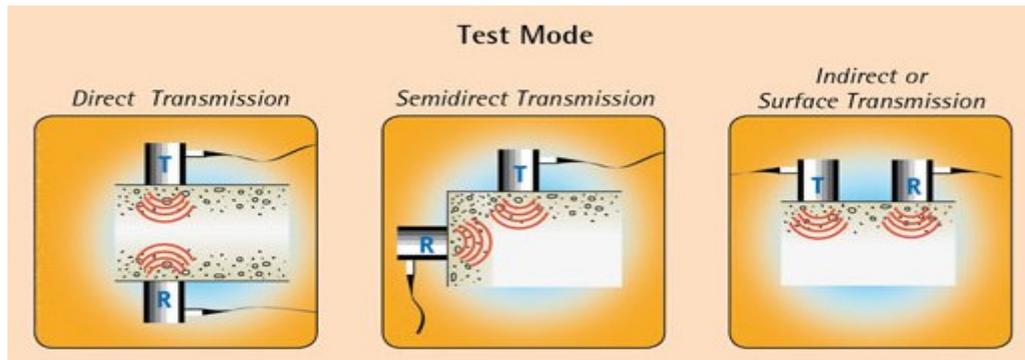
a) Arrangements of Transducers

The receiving transducer detects the arrival of that component of the pulse, which arrives earliest. This is generally the leading edge of the longitudinal vibration. Although the direction in which the maximum energy propagated is at right angles to the face of the transmitting transducer, it is possible to detect pulses, which have travelled through the

concrete in some other direction. It is possible therefore to make measurements of pulse velocity by placing the two transducers on either:

- Opposite faces (direct transmission),
- Adjacent faces (semi-direct transmission); or
- The same face (indirect or surface transmission).

These three (3) arrangements are shown in the figure below.



T – Transducer, R-Receiver

Figure 2.3.3-2 Arrangements of Transducers

a.1) Determination of Pulse Velocity by Direct Transmission

Where possible direct transmission arrangement should be used since the transfer of energy between transducers is at its maximum and the accuracy of velocity determination is therefore governed principally by the accuracy of path length measurement. The couplant to be used should be spread as thinly as possible to avoid any end effects resulting from the different velocities in couplant and concrete.

Longitudinal pulse velocity is given by:

$$v = \frac{L}{T}$$

where:

V is the longitudinal pulse velocity in km/s or m/s

L is the path length, in mm

T is the time taken by the pulse to traverse that length, in μ s

a.2) Determination of Pulse Velocity by Semi-Direct Transmission

The semi-direct transmission arrangement has a sensitivity intermediate between those of the other two arrangements, and, although there may be some reduction in the accuracy of measurement of the path length, it is generally found to be sufficiently accurate to take this as the distance measured from center to center of the transducer faces. This arrangement is

otherwise similar to direct transmission. The formula for direct transmission is also used for determining pulse velocity for semi-direct transmission.

a.3) Determination of Pulse Velocity by Indirect (Surface) Transmission

Indirect transmission is used when only one face of concrete is accessible, when depth of surface crack is to be determined or when the quality of the surface concrete relative to the overall quality is of interest. It is the least sensitive of the arrangements and, for a given path length, produces at the receiving transducer a signal which has an amplitude of only about 2% or 3% of that produced by direct transmission. Furthermore, this arrangement gives pulse velocity measurements which are usually influenced by the concrete near the surface. This region is often of different composition from that of the concrete within the body of a unit and test results may be unrepresentative of that concrete. The indirect velocity is invariably lower than the direct velocity on the same concrete element. This difference may vary from 5% to 20% depending largely on the quality of concrete under test. Where practicable, site measurements should be made to determine this difference. With indirect transmission, there is some uncertainty regarding the exact length of the transmission path because of the significant size of areas of contact between transducers and the concrete. It is preferable to make a series of measurements with the transducers at different distances apart to eliminate this uncertainty.

1. To do this, the transmitting transducer is placed in contact with concrete surface at a fixed point "P" and the receiving transducer shall be placed at fixed increments "x" along a chosen line on the surface. The transmission times recorded should be plotted as points on a graph showing their relation to distance separating the transducers. An example of such plot is shown in Figure 2.3.3-3.

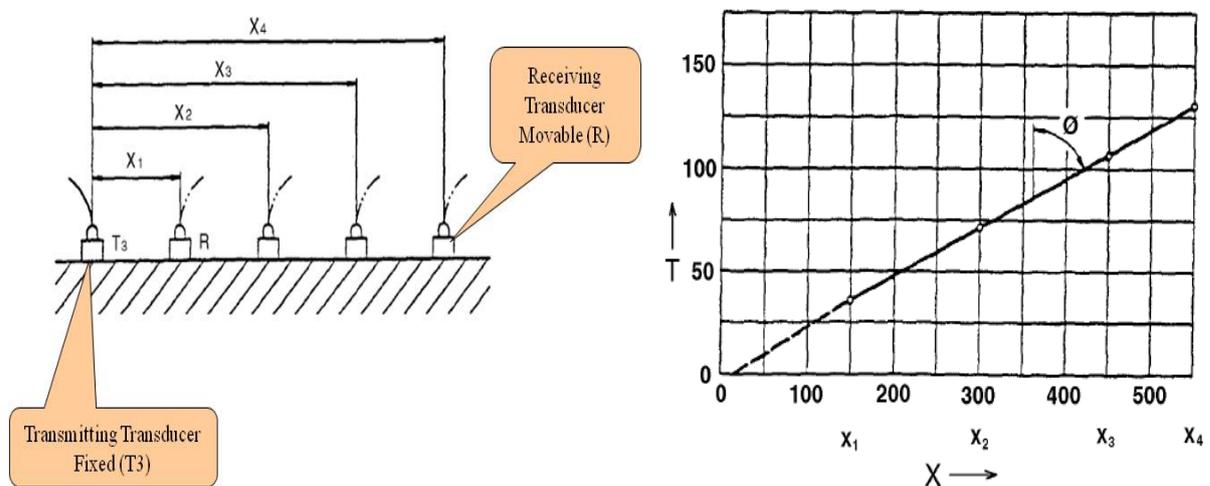


Figure 2.3.3-3 Indirect (Surface) Transmission

2. The slope of the best straight line drawn through the points (tangent) is measured and recorded as the mean pulse velocity along the chosen line on the concrete surface. Where the points measured and recorded in this way indicate a discontinuity, it is likely that a surface crack or surface layer of inferior quality is present and velocity measured in such an instance is unreliable.

b) Investigation of Scaling, Delamination and Other Flaws

The depth of observed delamination and extent of scaling can be determined by this test. This test is suitable considering that pulse velocity in defective concrete layer (V_1) is less than in the sound concrete layer (V_2). Perform readings as shown in Figure 2.3.3.4

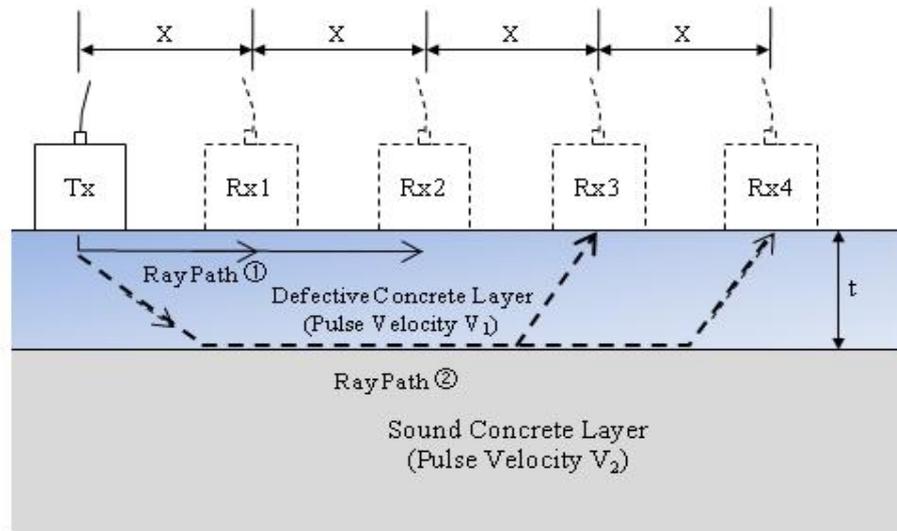


Figure 2.3.3-4 Sampling Pattern for Investigation of Scaling, Delamination, Etc.

1. Set up apparatus. (Refer to Operation Manual)
2. Check rebar locations and mark area/locations for the test.
3. Place the transmitting transducer Tx as shown, then place receiving transducer Rx at distances x , $2x$, $3x$ and $4x$ from Tx.
4. Distance “ x ” should be less than $t/2$ where “ t ” is the thickness of suspected damaged layer.
5. For each location of Rx make sure that couplant is applied. Record measured values of Transit Time (T) and Path Length (L) at each location.

c) Depth of Cracks

Depth of cracks that are ≥ 0.20 mm wide should be inspected. When the depth of a crack has to be measured, perform two pulse transit time readings as indicated in Figure 2.3.3-5.

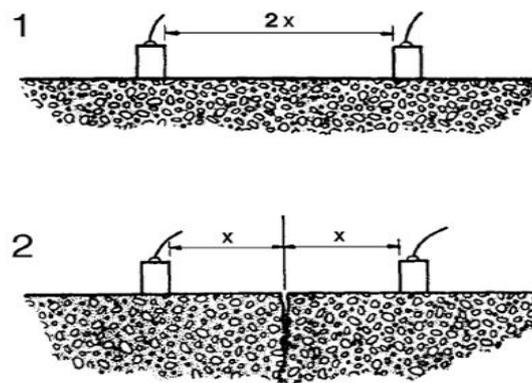
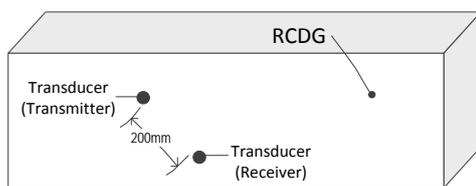


Figure 2.3.3-5 Transducers Positioning for Estimation of Crack Depth

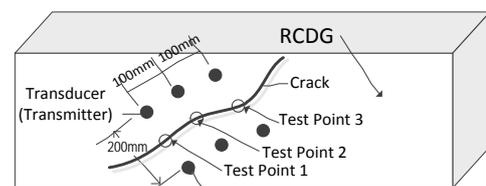
1. Set up apparatus. (Refer to Operation Manual)
2. Readings are taken with the transducers placed in several symmetrical positions with respect to the crack (Figure 2.3.3-5 diagram 2), another reading is with the same distance between the transducers but without any crack between them (Figure 2.3.3-5 diagram 1) along the same attribute (for example girder).
3. Record the transit times for both cases. Make sure that the crack spaces are not filled with water during the testing since transit time will be affected.

d) Test Points

Along the line of crack, determine readings at every 100mm or shorter interval depending on the length of crack. Record maximum crack depth in the engineering inspection form (EIF).



Measure pulse transit time (t_s) in undamaged portion of girder or attribute to be tested



Measure pulse transit times (t_c) at 3 test points along crack of girder or attribute to be tested

2.3.3.4 Analysis

a) Investigation of Scaling, Delamination and Other Flaws

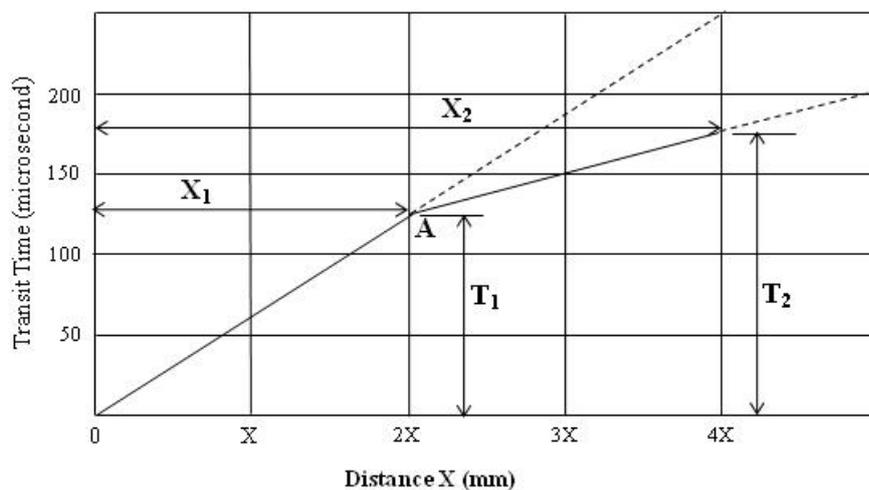


Figure 2.3.3-6 Plot of Transit Time versus Distance

1. Plot results of test graphically as shown in Figure 2.3.3.6.

Ray path ① (Figure 2.3.3.4) in surface layer of defective concrete is fastest/shortest path for test at Rx1 (x) and Rx2 (2x). Ray path ② in surface layer of sound concrete is fastest /shortest path for Rx3 (3x) and Rx4 (4x)

The velocity, $V_d (= X_1/T_1)$ in the defective concrete is less than $V_s (= X_2/T_2)$. Point A is the location of the boundary between defective and sound concrete

2. The thickness of the defective layer, “t” in mm, is estimated from the following relation:

$$t = X_1/2\sqrt{(V_s - V_d)/(V_s + V_d)}$$

where:

V_d = pulse velocity in defective concrete (km/s) = X_1/T_1

V_s = pulse velocity in sound concrete (km/s) = $(X_2-X_1) / (T_2-T_1)$

X_1 = distance at which the change of slope occurs

3. The first set of tests should be on trial and error basis, to confirm the approximate thickness of the defective layer. To investigate the maximum thickness “t” of the defective layer, the value of “x” may be measured in increments of 10 or 20% until maximum is found. This new value of “x” should then be used in further tests on grid basis to define the area affected.

b) Depth of Cracks

1. The following formula is used to calculate the depth “h” of the crack, provided that the crack space is not filled with water:

$$h = x \sqrt{\left(\frac{t_c}{t_s}\right)^2 - 1}$$

where:

x = half distance between the transducers (mm)

h = crack depth (mm)

t_c = transit times across the crack of concrete structure (microsecond)

t_s = transit time along the surface of portion of same concrete structure without defects (microsecond)

2.3.3.5 Evaluation and Recommendations

a) Scaling, Delamination and Other Flaws

1. Confirm depth of suspected defective layer thickness and area affected by core test. Subject core sample to carbonation test to determine if rebars are likely to be corroded.
2. The procedure can also be used to investigate concrete damaged by fire, chemical attack, etc. where a surface or upper layer has less strength than the layer below.

b) Depth of Crack

Table 2.3.3-1 Degree of Damage

Degree of Damage	Crack Depth (D in mm)	Status	Countermeasure
I	$C/2 > D$	Rebar will be corroded in the future.	Monitor
II	$C > D \geq C/2$ ($C \geq 40\text{mm}$)	Rebar may be corroded.	Conduct periodic inspection
III	$C > D \geq C/2$ ($C < 40\text{mm}$)	Rebar may be corroded.	Conduct Carbonation Test
IV	$D \geq C$ C : concrete cover	Rebar must be corroded.	Conduct Half -Cell Test Conduct Carbonation Test Repair Damage

2.3.4 Half-Cell Electrical Potential Test

2.3.4.1 Fundamental Principles

a) General

The corrosion (rusting) of steel rebar is an electro-chemical process, involving anodic (corroding) and cathodic (passive) areas of the metal. By measuring concrete-surface electrical potentials relative to a standard reference electrode on a pre-defined grid, the presence and location of corrosion and its probable future performance may be assessed.

To use this technique, it is necessary that a continuous electrical current is present in the reinforcing bars (this is normally achieved with a metal wire connecting the various reinforcing element, for example horizontal and vertical bars). A multimeter can be used to check that this electric current exists.

b) Applications

This technique is used to assess the durability of reinforced concrete members where reinforcement corrosion is suspected. The method has the advantage of being simple and it allows an almost non-destructive survey to produce equi-potential contour maps of surface of concrete member. Reported uses include the location of areas of high reinforcement corrosion risk in bridge decks and abutments.

The technique may be used to identify areas of reinforced concrete in need of repair or protective treatment and, by regular measurements, monitor the behavior of new and relatively new structures and thus minimize maintenance costs.

2.3.4.2 Description of Apparatus

Specifications



- A. Name: Half-Cell Digital Corrosion Meter
 - Half Cell: Silver/Silver Chloride refillable probe (stable and low corrosive)
 - Measurements are automatically converted and displayed as copper/copper sulphate equivalent potentials
 - Range / Resolution: +1999 mV / +1mV
- B. Tester is supplied with:
 - Electrode complete with 100 ml electrolyte gel
- C. Unit conforms to BS 1881:201, ASTM C876, UNI 9535 standards

Figure 2.3.4-1 Half-Cell Digital Corrosion Meter

2.3.4.3 Procedures

a) Test Surface Preparation

1. Mark out the desired grid on the surface to be surveyed. Grid spacing should be 100mm to 300mm. Use narrow spacing if the test will be carried out around surface crack.
2. Locate and expose a rebar approximately at the center of the area to be surveyed.
3. Clean the exposed rebar to bright metal to ensure good electrical contact and attach the reference cable using the crocodile clip. If the rebar is corroded, drill a small diameter hole into the steel and screw into it a T-bar Gimlet onto which to attach the cable.

b) Equipment Set-up

1. Set-up the equipment as shown in Figures 2.3.4-2 or 2.3.4-3 as appropriate. Apparatus must be calibrated before use. Refer to instruction manual on how to calibrate.

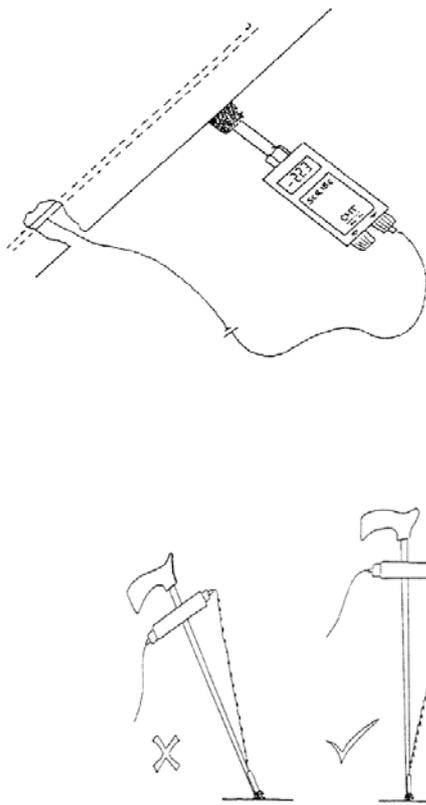


Figure 2.3.4-2 Set-up for Surveys on Vertical Surfaces and Soffits

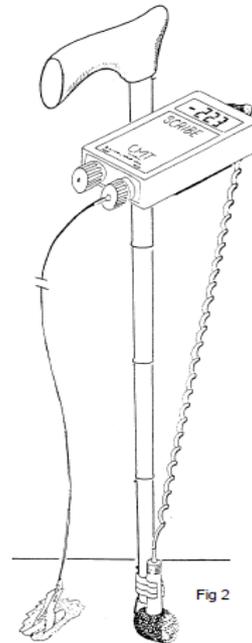


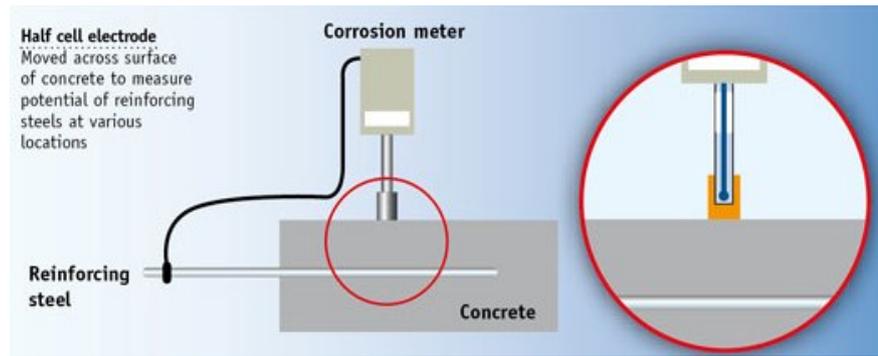
Figure 2.3.4-3 Set-up for Surveys on Horizontal Surfaces such as Bridge Decks/Slabs etc.

(To avoid fatigue when using this setup, lower the probe vertically onto test grid position)

c) Testing

1. Replace the solid cap on the cell electrode with one of the sponge heads and moisten it with clean, slightly soapy water.
2. Dampen the concrete surface over its entire surface or grid points.
3. Place cell electrode probe on first test position. Position should be just above the rebar. Refer to diagram in Figure 2.3.4-4. Only light contact pressure of the sponge head is necessary.
4. The probe is a precision equipment and should be treated accordingly. Inspector should follow manufacturer's recommendation in the instruction manual.
5. Allow reading to settle within 10mV (second least significant digit) and record reading.
6. Proceed to second test position and repeat testing procedure steps 3 and 4.

**Figure 2.3.4-4
Testing Diagram**



d) Test Points

Perform tests near areas where there is corrosion potential of rebar. If the corrosion potential is higher or extensive, implement further testing within its surroundings.

2.3.4.4 Analysis

Measurements are to be presented with an equipotential contour map which will provide a graphical delineation of areas in the member where corrosion activity may be occurring.

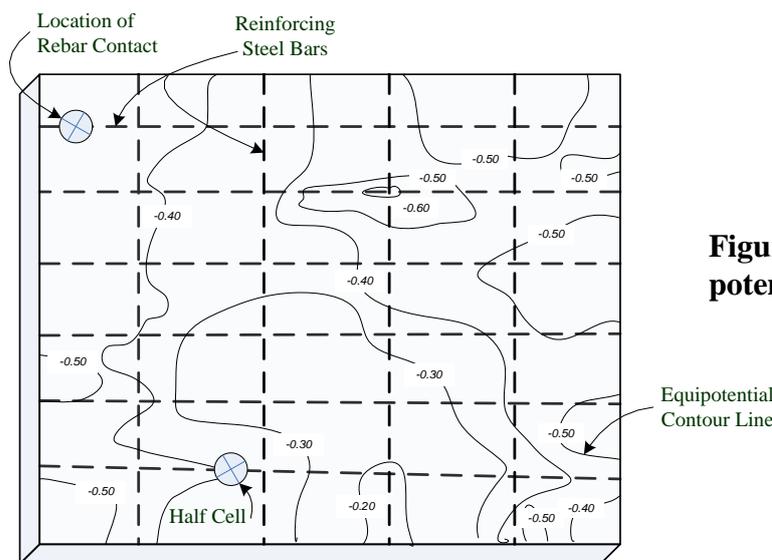


Figure 2.3.4-5 Example of Equipotential Contour Map

The contour map provides a graphical delineation of areas in the member where corrosion activity may be occurring.

The risks of corrosion based on potential difference reading are shown in Table 2.3.4.1. The data are based on the principle of Copper/Copper Sulphate electrode measurement technique as described in ASTM C876-80, in BS1881:201, in ASTM C876-80 and in a number of technical articles.

Table 2.3.4-1 Risk of Corrosion Against the Potential Difference Readings

Potential Difference Levels (mV) <E>	Chance of Rebar Being Corroded
Less than -500 ($E \leq -500$)	Visible evidence of corrosion
-350 to -500 ($-350 < E < -500$)	95% probability of corrosion
-200 to -350 ($-200 \leq E \leq -350$)	50% probability of corrosion
More than -200 ($-200 < E$)	5% probability of corrosion

2.3.4.5 Evaluation and Recommendations

Depending on the analysis of results, the following degrees of damage are established:

Table 2.3.4-2 Degree of Damage

Degree of Damage	Potential Difference Levels (mV) <E>	State
I	$-200 < E$	Rebars will be corroded in the future
II	$-200 \leq E \leq -350$	Rebars may be partly corroded
III	$-350 < E < -500$	Rebars are be corroded
IV	$E \leq -500$	Rebar are seriously corroded

Half cell electrical potential test should be carried out together with rebar detection test investigation and carbonation test at the same time.

A more detailed investigation is required especially around cracks.

It is generally accepted that corrosion potential measurements must be complemented by other methods, because although reliable relationships between potential and corrosion rate can be found in the laboratory for well-established conditions, these can in no way be generalized, since wide variations in the corrosion rate are possibly in very narrow range of potentials.

Values obtained can only provide information for corrosion probability and cannot indicate the rate of corrosion.

2.3.5 Rebar Detection Test

2.3.5.1 Magnetic Type

a) Fundamental Principles

As a magnetic type scanning apparatus is turned on, it generates an electromagnetic field. When a reinforcing bar (rebar) or other metal object lies within this field, the lines of force become distorted. The disturbance caused by the presence of the metal in turn, produces a

local change in field strength as detected by the search head and indicated by the meter. Both the orientation and proximity of the metal to the search head affect the meter reading. It is therefore possible to locate reinforcing bars and determine their orientation. Under ideal conditions, both bar size and cover can be estimated when neither is known.

With apparatus using magnetic induction, a multi-coil search is used with a lower operating frequency than eddy current type. The principle used is similar to that of a transformer. Such instruments are less sensitive to non-magnetic materials than those using eddy current principle.

b) Description of Apparatus



Photo 2.3.5-1
Magnetic Type Rebar Detector

Specifications

- Brand: HILTI PS200 Ferrosan
- Scanner memory capacity: Up to 9 Imagescans plus up to 30 meters of recorded Quickscans
- Monitor memory capacity / type: At least 150 Imagescans or 75 Quickscans plus up to 15 min. of voice recording (32 MB)
- Battery life 8 hours on average
- Protection class IP 54 in accordance with IEC 529
- Operating temperature range -10°C to $+50^{\circ}\text{C}$
- Dimensions / weight of scanner: 260 x 132 x 132 mm / 1.4 kg
- Dimensions / weight of monitor: 264 x 152 x 57 mm / 1.4 kg
- Minimum system requirements: Microsoft Windows 2000, XP, at least 50 MB free hard disk space, CD-ROM drive, for PC software USB V1.1 interface

Advantages

- Immediate high resolution image for a clear picture of the reinforcement.
- Reduce wear on drill/core bits and tool motors when drilling through reinforcement.
- Reduce drilling times – important in repetitive drilling applications.
- Avoid potentially disastrous and costly hits on critical reinforcement.
- Cable less connection between monitor and scanner. No cable to snag or hinder inspector's movement.
- Includes powerful PC software for office analysis and easy creation of reports.

Limitations

- The scanner only detects reinforcing bars that lie perpendicular to the direction of travel. Rebars that lie parallel to the direction of travel will not be detected. Therefore, ensure that the object is scanned in both the horizontal and vertical directions. An incorrect depth may be calculated for bars that lie diagonal to the direction of travel.
- Rebar spacing of 36 mm or less cannot be measured by the scanner.
- Rebar diameter measurement is possible only at minimum depth of 10 mm to a maximum depth of only 60 mm

c) Procedures

c.1) Image Scan

Imagescan is used to create an image of the reinforcement layout so that the depth and diameter of the bars can be determined.

- 1) Fix a PSA 10 or PSA 11 reference grid using adhesive to a wall or girder/beam to be investigated.
- 2) Alternatively, a grid can be marked directly on the wall or girder/beam surface. Using a straight edge (ruler) as a guide, mark a 4 x 4 grid with 150mm spacing between parallel lines.
- 3) The scanner is switched on and set to Imagescan symbol. A representation of the grid appears on the screen with a suggested starting point. This is always upper left and will suffice for most scans. Image data will only be generated for areas of the grid that have been scanned both vertically and horizontally.
- 4) Place the scanner on the grid at the starting point. Ensure the alignment marks on the scanner are aligned correctly with the grid as shown below.

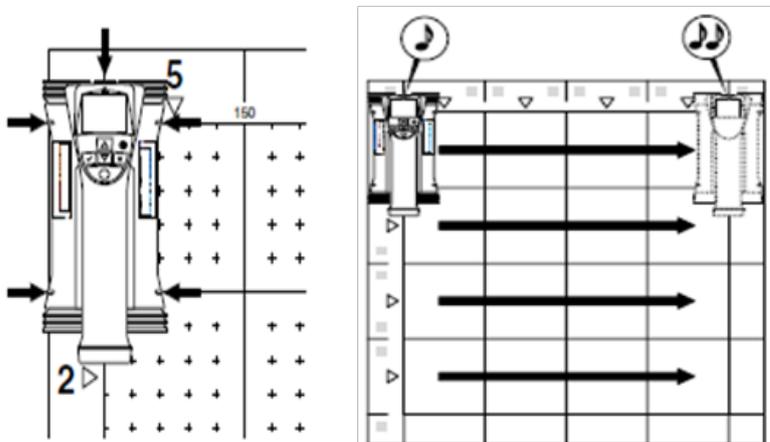


Figure 2.3.5-1 Starting Point of Horizontal Scan



Photo 2.3.5-2 Sequence of Horizontal (Rows) Scan

- 5) When all rows are complete, scan the columns in a similar way.

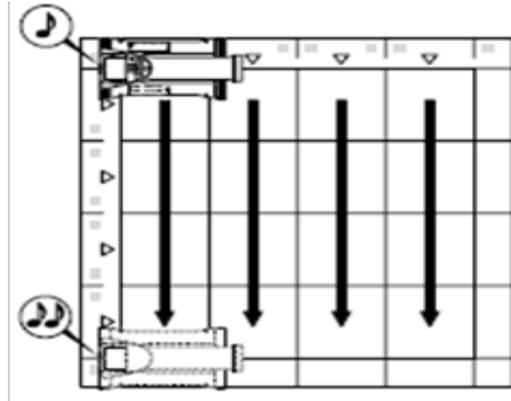


Figure 2.3.5-2 Sequence of Vertical (Columns) Scan

- 6) When the scan is complete, the data may then be transferred to the monitor for viewing and evaluation.
- 7) If a large section of a wall or length of girder/beam has to be investigated, block scan is undertaken. In a block scan a series of imagescans are taken and the data are recorded in the monitor.
- 8) After scanning, scanned data is transferred to the computer for analysis.
- 9) Rebar location and concrete cover are analyzed by the computer using a software.
- 10) The following diagrams show the procedures for imagescan on a beam/girder.

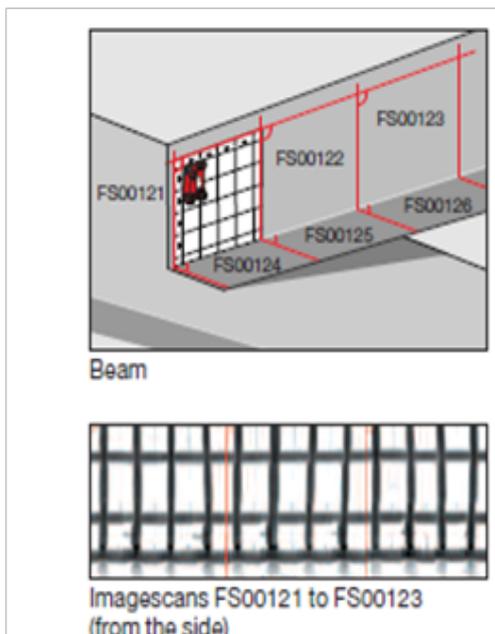


Figure 2.3.5-3 Imagescan Process

c.2) Quickscan

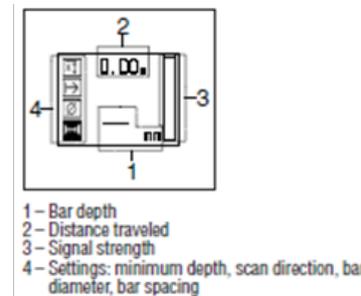
Quickscan is used to quickly detect bar positions and depths that are then subsequently marked on the surface. This procedure is called Quickscan detection. Accurate depth measurement is another Quickscan function in which values for bar diameter and bar spacing must be previously entered.

Alternatively, the data can be recorded and evaluated on the monitor or in the PC application. In this way, the average depth of cover over the reinforcement over large stretches of the surface can be easily determined. This is termed Quickscan recording.

Quickscan settings are detailed in the instruction manual of the apparatus and are not shown in this document.

1) Quickscan Detection

- (a) Switch on the scanner. The Quickscan icon is automatically selected first. The Quickscan screen is then displayed.
- (b) Move the scanner over the surface. Reinforcing bars that lie perpendicular to the direction of travel will be detected. The distance covered by the scanner is recorded. The maximum scanning speed should be 0.5m/s (20 inches/sec.).



**Figure 2.3.5-4
Quickscan Display**

- (c) When approaching a reinforcing bar, the signal strength increases and depth values may appear in the display. When the scanner is at the center of a reinforcing bar the following occurs:
 - the red LED lights,
 - the scanner beeps,
 - the signal strength bar is at maximum,
 - and the approximate depth of the bar is indicated (lowest depth value indicated = center of the bar).
- (d) The bar is positioned along the center line of the scanner and may be marked on the surface using a marker. The accuracy of the depth measurement can be increased by switching to accurate depth measurement measuring mode.



**Figure 2.3.5-5
Display of Results**

2) Quickscan with Accurate Depth Measurement

- (a) The measuring mode "Quickscan with Accurate Depth Measurement" is selected by pressing the "Confirm button" on the main menu of the scanner.
- (b) The diameter must be known and previously entered.
- (c) In addition, the value for the spacing between bars must also be entered if it lies between >36 and <120 mm. If the spacing is known, the value is entered manually.
- (d) The spacing can be calculated automatically using the Quickscan detection function by searching for the center of the bar and pressing the red Record button when the scanner is over the midpoint of the bar. Next, search for the midpoint of the next bar and again press the Record button.
- (e) Bar spacing is then calculated automatically and recorded.
- (f) After setting the bar diameter and bar spacing, the scanning procedure is identical to the procedure described in 1) Quickscan Detection.

3) Quickscan Recording

Always carry out an Imagescan prior to Quickscan recording in order to: establish the direction of the uppermost layer of reinforcement, minimize the risk of measuring on a spliced bar, and immediately see if there are any ferrous materials in the concrete that may affect the accuracy of the result.

- To record the position and depth of all reinforcing bars detected, place the scanner on the surface and use Quickscan detection to find a position where there are no bars present.
- Mark the starting point with a marker and press the *Record* button. The diskette symbol appears on the screen, indicating that the scanner is recording data.
- Move the scanner over the surface. At the end of the scan, take care to ensure that the end point is not directly over a rebar. To stop recording, press *Record* again. Use a marker to mark the end of the stretch that has been scanned.
- Reinforcing bars that lie perpendicular to the direction of travel will be detected and automatically recorded. Ensure that the settings are correctly set before beginning recording is started.

d) Analysis

- The calculations for bar sizes and concrete cover are described in the procedures and / or are available as direct read out from the digital display.
- Rebar location and thickness of concrete cover are analyzed by the computer by using a software.
- Care should be taken to record other details such as direction of rebars and location of area of investigation supported by time-dated photos whenever possible.
- The detection range and accuracy of depth measurement for imagescan and quickscan is typically accurate to within $\pm 10\%$ of actual depth. Refer to table below for detection range and accuracy of the HILTI PS200 Ferroskan when bar diameter is given.

Table 2.3.5-1 Detection Range and Accuracy

Imagescan

Bar Diameter	Depth									
	20	40	60	80	100	120	140	160	180	
6	±2	±3	±3	±4	±5	0	X	X	X	
8	±2	±2	±3	±4	±5	0	0	X	X	
10	±2	±2	±3	±4	±5	0	0	X	X	
12	±2	±2	±3	±4	±5	±10	0	X	X	
14	±2	±2	±3	±4	±5	±10	0	0	X	
16	±2	±2	±3	±4	±5	±10	0	0	X	
20	±2	±2	±3	±4	±5	±10	±11.5	0	X	
25	±2	±2	±3	±4	±5	±10	±11.5	0	X	
28	±2	±2	±3	±4	±5	±10	±11.5	0	X	
30	±2	±2	±3	±4	±5	±10	±11.5	0	X	
36	±2	±2	±3	±4	±5	±10	±11.5	±13	0	

Bar diameter given. Value indicates accuracy of depth measurement in mm.
 0 - bar can be detected at this depth but no depth calculated
 X - Bar cannot be detected at this depth
 Valid for bars conforming to DIN 488

Quickscan Recording

Bar Diameter	Depth				
	20	40	60	80	100
6	±1	±1	±2	±4	±5
8	±1	±1	±2	±4	±5
10	±1	±1	±2	±4	±5
12	±1	±1	±2	±4	±5
14	±1	±1	±2	±4	±5
16	±1	±1	±2	±4	±5
20	±1	±1	±2	±4	±5
25	±1	±1	±2	±4	±5
28	±1	±1	±2	±4	±5
30	±1	±1	±2	±4	±5
36	±1	±1	±2	±4	±5

Bar diameter given. Value indicates accuracy of depth measurement in mm.
 0 - bar can be detected at this depth but no depth calculated
 X - Bar cannot be detected at this depth
 Valid for bars conforming to DIN 488

e) Evaluation and Recommendations

The test is useful for:

- Verification of reinforcement: locate rebar when plans are missing, for acceptance inspections, before structural repair or change of loads on a structure.
- Verification of concrete cover over reinforcement: obtain concrete cover information over large areas for acceptance inspections, before renovation or for quality control.
- Avoidance of accidental hitting of rebars: avoid cutting through critical reinforcement or costly rebar hits.
- Determination of bar depth and diameter.
- Easy determination of average concrete cover over large areas.
- Investigation of concrete members of which records are not available or need to be checked.

2.3.5.2 Radar Type

a) Fundamental Principle

Electromagnetic waves are transmitted from the antenna toward the concrete as shown in diagram below. The electromagnetic waves are reflected by an interface with the reflecting objects (e.g., reinforcing steel bars or cavities) whose electrical property is different from that of concrete. The waves are reflected back into the surface of concrete and received by the receiving antenna placed near the concrete surface. The distance to the reflecting objects can be calculated from the time the reflected waves need to reach the receiving antenna. The horizontal locations of the objects can be detected by moving the main unit on the surface of concrete.

Since this radar is designed to probe objects with high resolution that are near from the surface of concrete, it transmits pulse waves having a width of only about one nanosecond (one-billionth of a second) or less.

The velocity V of electromagnetic waves in concrete is obtained from the following formula:

$$V = \frac{C}{\sqrt{\epsilon_r}} \text{ (m/s)}$$

where:

C : Velocity of electromagnetic waves in vacuum (in air) (3×10^8 m/s)

ϵ_r : Relative dielectric constant of concrete (6 to 11)

The distance D to the reflecting object is obtained from the following formula:

$$D = \frac{1}{2} V T \text{ (m)}$$

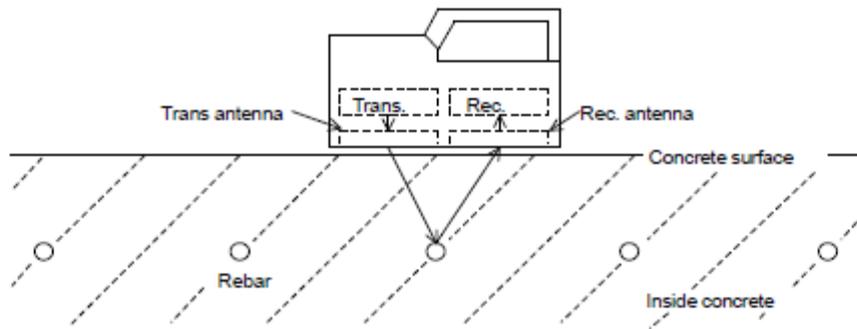


Figure 2.3.5-6 Diagram of Principle

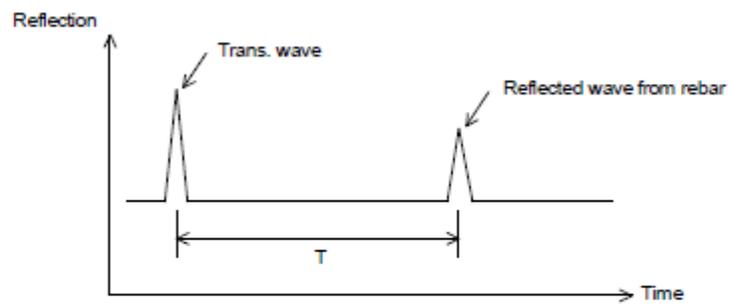


Figure 2.3.5-7 Diagram of Reflected Wave Form

b) Description of Apparatus

**Photo 2.3.5-3
Radar Type Rebar Detector**



Specifications

Brand	JRC NJJ-105 Handy Search
System	Electromagnetic radar
Search target	Reinforcing steel bars, pvc pipes, cavities,etc.

Covering Thickness	5 to 300 mm (for top reinforcement while relative dielectric constant of concrete is 6.2 and rebar diameter is 6 mm or more)
Display	TFT color LCD (640 x 480 bits)
Max. scanning rate	About 40cm/s with overspeed alarm buzzer
Output function	Dedicated printer output function (IrDA)
Search data recording function	Data saved in compact flash memory. Approximately 200 passes (15-m search data saved in binary format) when using 1-
Memory capacity	GB memory
Temperature range	0°C to +50°C
Power supply	Battery pack, AC adapter
Continuous use time	About 1.5 hours (with the battery pack being fully charged)
Structure	Splash/rain resistant
Dimensions	About 149 (W) x 147 (H) x 216 (D) mm

b.1) Applicable measuring conditions

- Scan test depth (covering depth) within 0.5 ~ 30 cm (case where relative permittivity of concrete is 6.2 and the diameter of the rebar is at least 6mm)
- Objects to be probed that are at a depth of less than 75 mm have an interval spacing of at least 75 mm and where the depth of the object to be probed is greater than 75 mm, the interval spacing of the objects is more than the depth
- Quality of concrete: Uniform
- Direction of reinforcing steel: Orthogonal to the Handy Search's traveling direction

b.2) Inapplicable Measuring Conditions

- Scanning of reinforcing steel bars or the like in concrete whose surface contains objects such as metal that reflect radio waves
- Presence of reinforcing steel bars that are parallel to the Handy Search's traveling direction
- Concrete that has a narrow (pitch) arrangement for the interval of rebar in the horizontal direction

(Depth less than 75 mm with spacing between rebar less than 75 mm
 Depth greater than 75 mm with spacing between rebar less than depth)

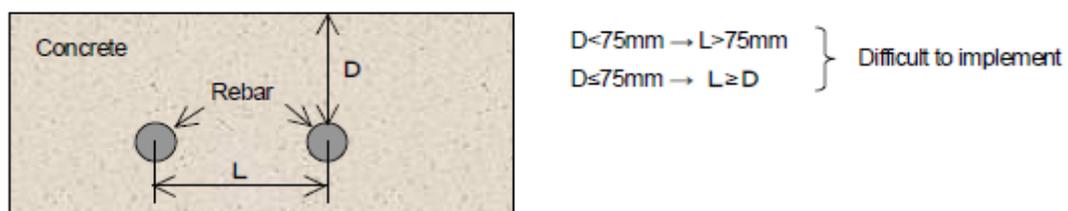


Figure 2.3.5-8 Inapplicable Measuring Conditions

c) Procedures

c.1) Preparation for Scanning

- 1) Using chalk (or similar tool) make markings on the concrete surface to indicate where to start testing (starting line) and where to scan (scan test line).
- 2) Make sure the start line and the scan test line are orthogonal (perpendicular to each other). (As necessary, in order to perform a retest, use an endpoint of the wall as a reference standard for the start line and scan test line).
- 3) A scan test applicable set up example is shown in Figure 2.2.5-9.

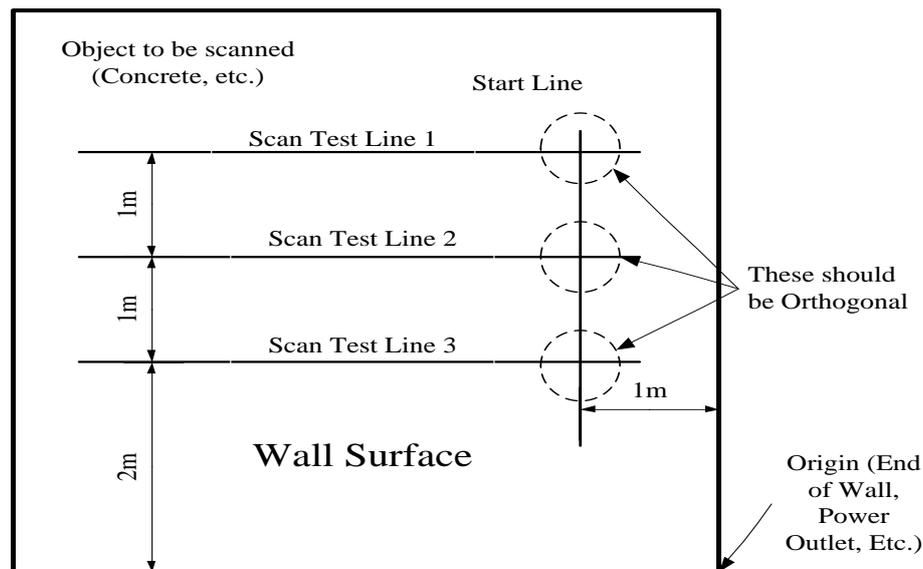


Figure 2.3.5-9 Example of Scan Test Set-up

c.2) Scanning

c.2.1) Power-on Procedure

- 1) The liquid crystal display lights up roughly 5 seconds after the power is turned on by turning on the power switch, and the initialization screen is displayed.
- 2) After the initialization screen completes, the scan test screen is displayed.
- 3) After confirming that the scan test screen is displayed on the liquid crystal display, start the scan test.

c.2.2) Scan Test

- 1) Place the unit over the intersection of start line and the scan test line.
- 2) After about one second after pressing START, a short single buzzer sound is generated, the fixed cursor is displayed at a position of about 10cm of the movement distance scale on the screen, and then scan test preparation is complete.
- 3) Move the unit on top of the scan test line at a speed of less than 40 cm/s by rotating the wheels.
- 4) Press START again to finish. A double buzzer sound is generated and the scan test is stopped.

- 5) However, if the scan distance reaches 15 m, the buzzer sounds twice and the scan test automatically finishes.

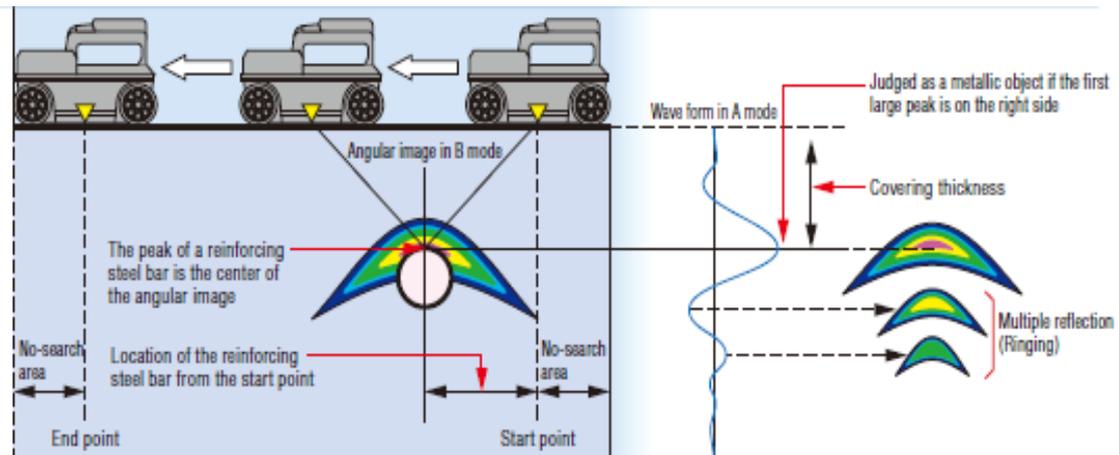


Figure 2.3.5-10 Scanning Diagram

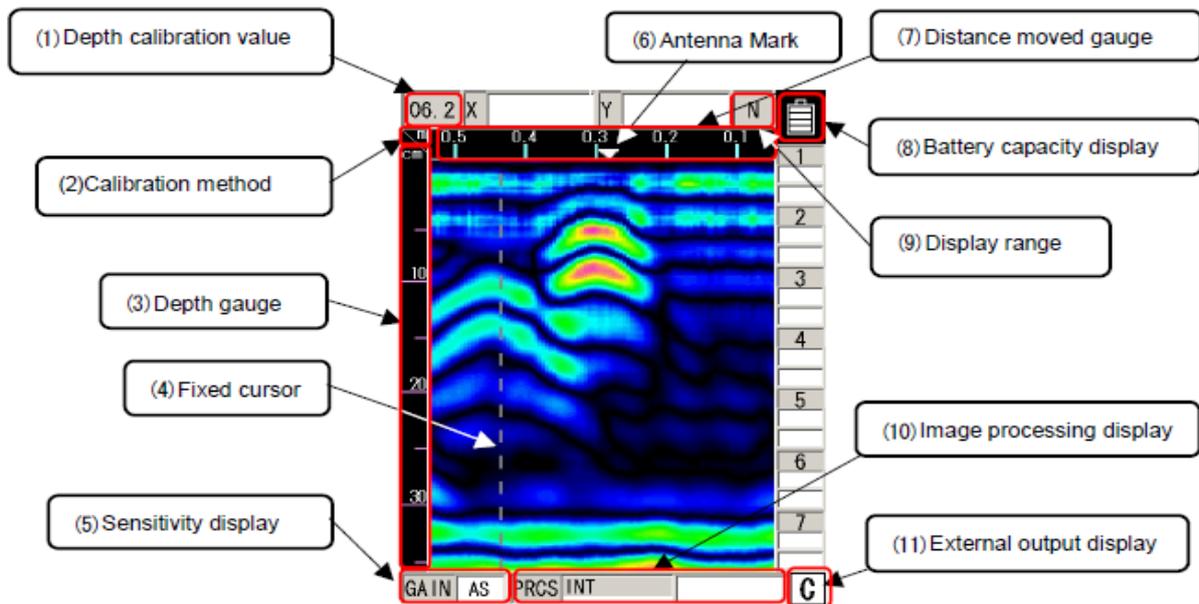


Figure 2.3.5-11 Example of a Test Scan

d) Analysis (Interpretation of Data)

- The reflected waves shown in the scan test example in figure above are reflections of rebar. (↓ position)
- The position of the object to be probed (e.g., reinforcing steel bar) in the traveling direction is determined as the peak of the reflected wave.
- An approximate depth (covering depth) of the object to be probed (e.g., reinforcing steel bar) is determined as the center of the reflected wave.
- Perform depth calibration to reduce the error in the depth (covering depth) of the object being probed (such as rebar). The position of the object to be probed (such as rebar) is shown by the peak on the right side of the A mode waveform.
- Match the cursor to the peak position to determine the depth.

- Refer to Figure 2.3.5.2-12 for example of data analysis

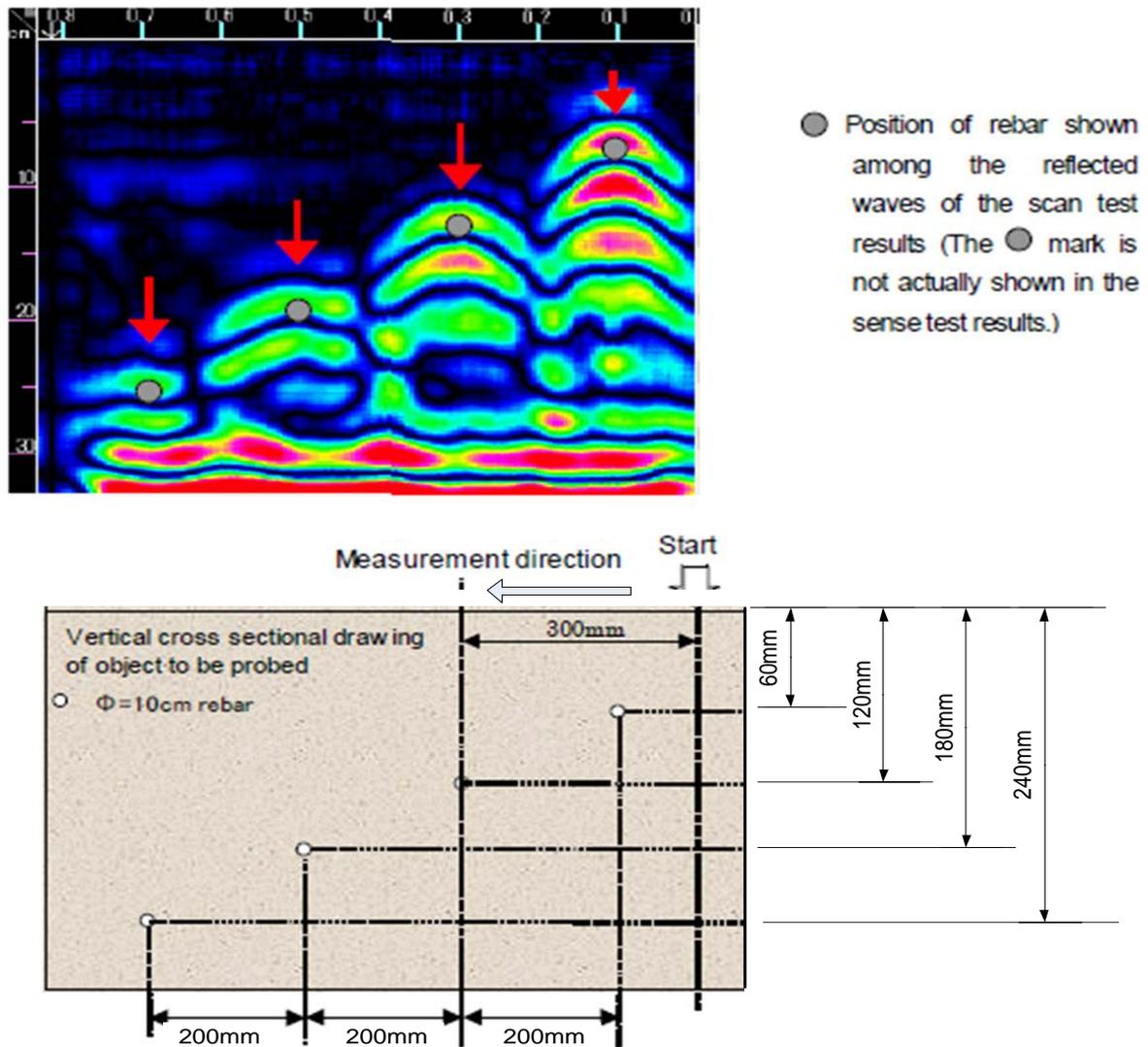


Figure 2.3.5-12 Example Data Analysis

e) Evaluation and Recommendations

The test is useful in the:

- Verification of reinforcement: locate rebar when plans are missing, for acceptance inspections, before structural repair or change of loads on a structure.
- Verification of concrete cover over reinforcement: obtain concrete cover information over large areas for acceptance inspections, before renovation or for quality control.
- Avoidance of hitting rebars: avoid cutting through critical reinforcement or costly rebar hits.
- Determination of bar depth and determination of average concrete cover over large areas.
- Investigation of concrete members of which records are not available or need to be checked.

2.3.6 Infrared Thermal Image Test

2.3.6.1 Fundamental Principles

a) General

Infrared Thermography is the process of using an instrument and a method to detect infrared energies emitted from an object, convert these emitted energies to temperature values, and display an image of the object showing the temperature distribution.

- Infrared was discovered by a British astronomer, Herschel, in 1800. When dispersing sunlight using a prism, Herschel accidentally found that there was an invisible energy just beyond the red color wavelength that increases with temperature.
- Infrared is an electromagnetic wave. Infrared wavelength is $0.7\mu\text{m}$ or longer. For a wavelength of 1mm ($1000\mu\text{m}$) or less the frequency is 300GHz or greater.

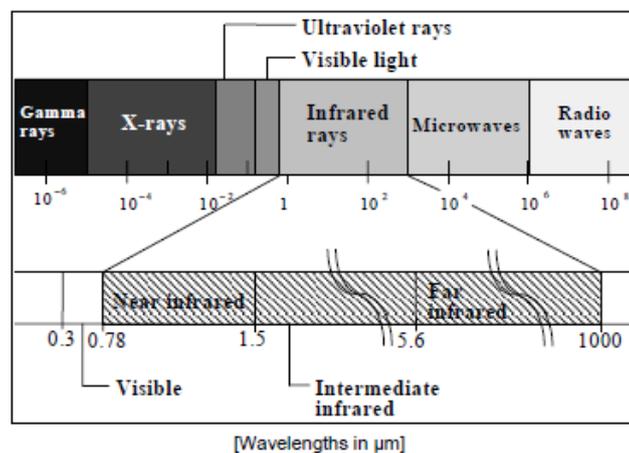


Figure 2.3.6-1 Spectrum of Electromagnetic Radiation

Characteristics of Infrared:

- It is invisible since its wavelength is longer than visible light. It has nothing to do with brightness or darkness of visible light.
- It is emitted naturally from any object of which temperature is absolute zero (Zero Kelvin) or higher. Therefore, it can be applied to any field.
- It has a characteristic of heating an object. Therefore, it is sometimes called “heat wave”.
- Because it is a kind of light (electromagnetic wave) it is transmitted through a vacuum.
- There is a correlation between infrared energy and temperature of an object. Therefore, it can be used to measure the temperature of an object.

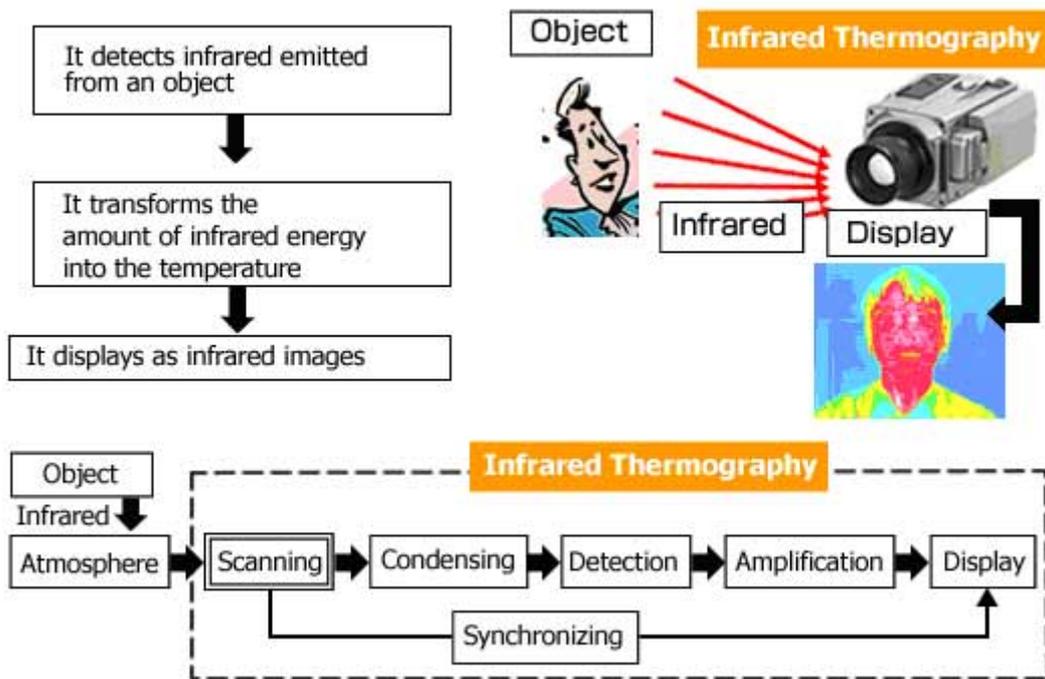


Figure 2.3.6-2 Process of Thermal Imaging

Characteristics of Infrared Thermography Instrumentation:

- Captures a surface temperature distribution and displays it as visible information.
- Temperature is measured from a distance without contacting the measured object.
- Temperature is measured in real time.

b) Applications

Used for detection of defects of concrete structure such as:

- Spalling
- Voids
- Delamination

The detectability of any internal structure such as voids, delaminations or layer thickness depends on the physical properties (heat capacity, heat conductivity, density, emissivity) of the materials of the test object. The interior 'structure' has an effect on temperature distribution on the surface. If the temperature changes on the surface, there is a delay before the effect of this change occurs below where a defect such as void occurs. The longer the time delay before the temperature changes, the greater is the depth of a defect below the surface. Generally anything deeper than 10 cm will only show after a long period of time (>1 hr) after the temperature change has occurred.

Since infrared system measures surface temperatures only, the temperature measured are influenced by three factors: (1) subsurface configuration, (2) surface condition; and (3) environment. As an NDT technique for inspecting concrete, the effect of subsurface configuration is usually the most interesting. All the information revealed by the infrared system relies on the principle that heat cannot be stopped from flowing from warmer to cooler areas; it can only be slowed down by the insulating effect of materials through

which it flowing. Various types of construction materials have different insulating abilities or thermal conductivity values. For example, an air void has a lower thermal conductivity compared with surrounding concrete. Hence, the surface of a section of concrete containing an air void could be expected to have a slightly different temperature from a section of concrete without an air void.

When tests are performed during daylight hours, the defective concrete areas will appear warmer, while tests performed after dark, defective areas will appear cooler.

2.3.6.2 Description of Apparatus



Photo 2.3.6-1 Infrared Camera and Accesories

Specifications

Apparatus: Infrared Camera NEC Thermo Tracer
Detector Type: Uncooled Focal Plane Array (Microbolometer)
Measuring range: -40 to +500°C, in Three (3)ranges
Resolution: 320(H) x 240(V)
Thermal Sensitivity: <0.05°C (at 30°C)
Accuracy: +/-1 °C or +/-1% whichever is greater
Spectral Resolution: 8 to 14µm ; Frame Rate: 60Hz
Spatial Resolution: (I.F.O.V) 1.21mrad
Focus Distance: 10cm to infinity (standard lens)
Focus: Automatic
Display Performance: Visual Image fusion display
Display Unit: 3.5 inch Color LCD Monitor (320x240) with

2.3.6.3 Procedures

Refer to the Operation Manual on how to set-up, operate controls and commands in using the camera. The engineer should be trained in using this type of sophisticated apparatus.

A typical image produced by the camera is shown on Figure 2.3.6.3.

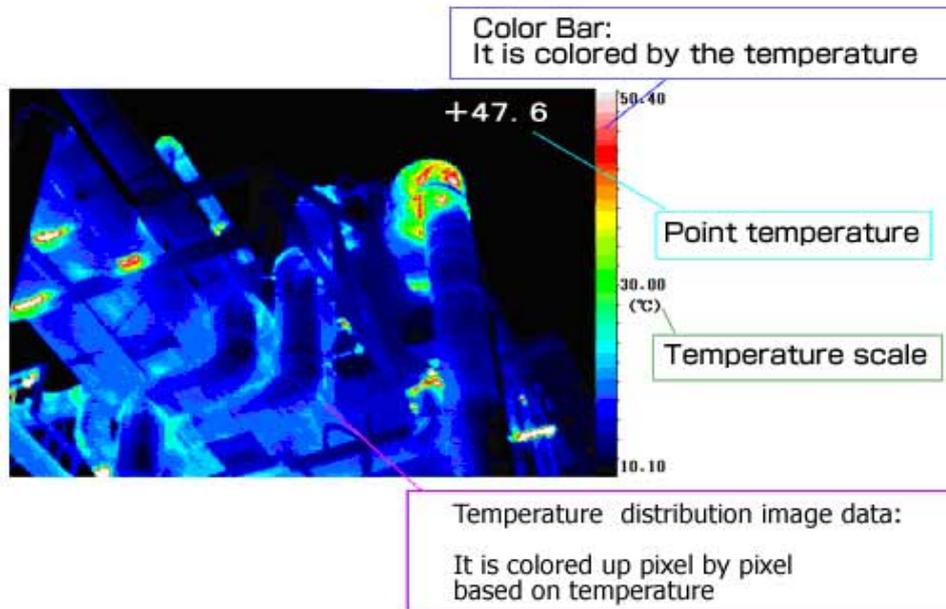


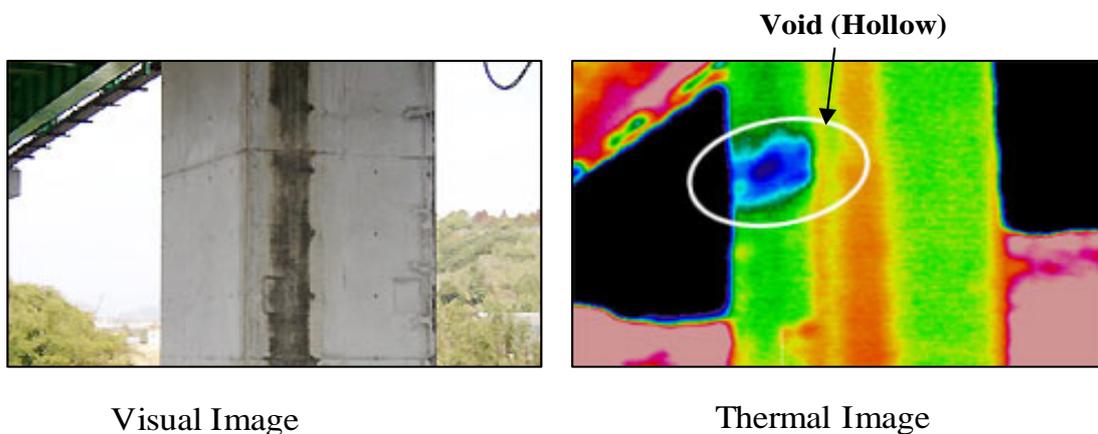
Figure 2.3.6-3 Sample Image from Infrared Camera

Infrared Thermography instruments display temperature distribution image data using a matrix of pixels (each pixel being a miniature infrared energy detector). In the figure above, there are 320 horizontal x 240 vertical pixels (detectors).

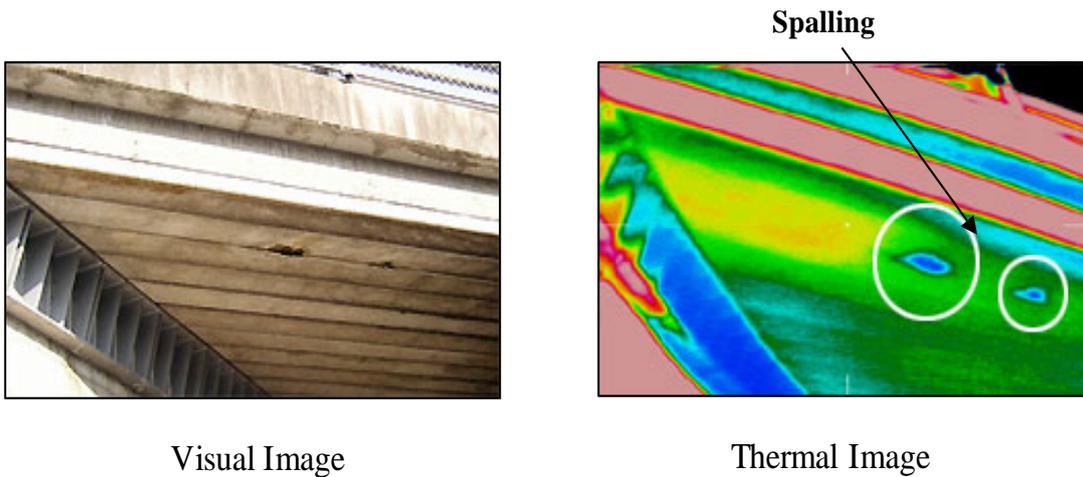
2.3.6.4 Analysis

Images are uploaded to a computer for detailed analysis. With the use of software that interprets infrared images, the location and extent of voids, delamination and spalling of concrete structures are determined.

The following examples show results of infrared thermal imaging of structure defects.



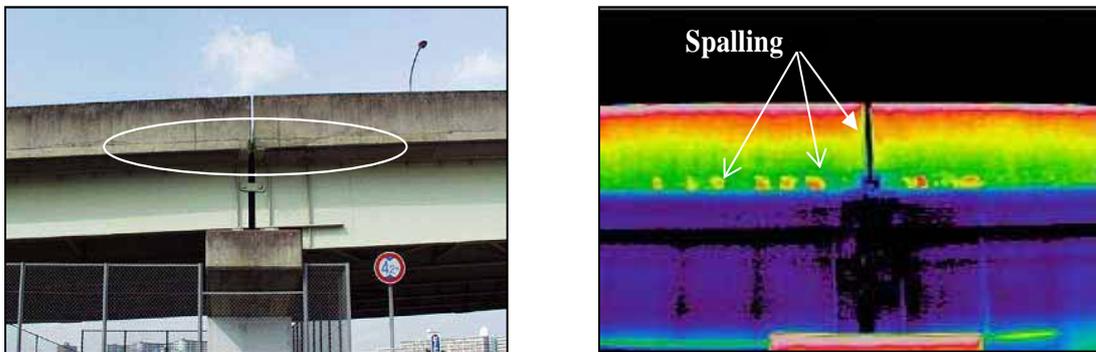
a) Pier Defect



Visual Image

Thermal Image

b) Concrete Deck Slab Defect



Visual Image

Thermal Image

c) Concrete Railing Defect

Figure 2.3.6-4 Examples of Detection of Defects using Infrared Thermal Imaging

2.3.6.5 Evaluation and Recommendations

- Infrared thermographic analysis of large concrete areas does not require destroying concrete structure during testing. Only small calibration corings are used. This results in major savings in time, labor, equipment, traffic control, and scheduling problems.
- Infrared thermographic equipment is safe as it emits no radiation. It only records thermal radiation, which is naturally emitted from the concrete, as well as from all other objects. It is similar in function to an ordinary thermometer, only much more efficient.
- Infrared thermography is an area testing technique, while the other NDT methods are mostly either point or line testing methods. Infrared thermography is capable of forming a two dimensional image of the large test surfaces showing the extent of subsurface anomalies.

- It is recommended that infrared thermography be used to survey large areas for problems. Once specific problem locations are established, radar can be used to spot check the anomaly for its depth and thickness. This combined technique would give the best combination of accuracy, efficiency, economy and safety.
- Temperature and weather are very sensitive. Therefore infrared thermography shall be useful the night time (around 9pm to 7am) or at day time(11am to 5pm).Because accuracy factor of temperature is $\pm 2^{\circ}$ C or $\pm 2\%$ of absolute temp. in $^{\circ}$ C.
- The test locations must be the point by solar radiation, temperature difference of the health point and damage point can be detected by infrared thermography.
- Infrared thermography must be avoided to test during rain, cloudy weather when there are slight changes in temperature after a few hours at the time of highest and lowest temperature (approximately 3 hours).

2.3.7 Compressive Strength Test

2.3.7.1 Fundamental Principles

Microcoreing is the method chosen to obtain samples of existing concrete structures that are used to determine in-situ compressive strength of concrete. In concrete structures with a lot of reinforcement bars, it may be impossible to obtain a large core specimen from which compressive strength may be taken since reinforcing steel may be so prevalent in the concrete. Small cores extracted by microcore apparatus are used as substitutes for large cores to test concrete strength.

Compressive Strength Test in the form of The Point Load Test (PLT) is intended as an index test for the strength classification of rock materials and concrete core samples, but it may also be widely used to predict other material strength parameters with which is correlated. It can provide similar data at a lower cost due to its ease of testing and simplicity of sample preparation.

2.3.7.2 Description of Apparatus

**Photo 2.3.7-1
Microcore Apparatus**



Microcore Apparatus is used for drilling holes (diamond core drilling) of 8–35 mm Ø in reinforced concrete, masonry and natural stone. It drills up to 300 mm depth with little vibration and low noise and can cut through reinforcing steel bars. Its water pump and extraction function are activated/deactivated automatically by the drilling machine.

**Photo 2.3.7-2
Point Load Test Apparatus**



Point Load Test Apparatus is used for compression test on small cylinder specimens and core samples up to $\text{Ø } 60 \times 100 \text{ mm}$. The load is applied by a hand pump and is measured by a precision digital display range 0-56kN, with an accuracy of $\pm 1\%$, resolution 65000 points. The compression platens have $\text{Ø } 65 \text{ mm}$, the upper one has a spherical seat and vertical daylight of 110 mm. It comes with a set of two hardened conical points for point load testing.

2.3.7.3 Procedures

a) Coring

- 1) Assemble the apparatus in accordance with instructions in the Operation Manual
- 2) Drilling
 - To begin drilling, press the control switch before the core bit has been brought into contact with the base material. Begin drilling the hole only when the water flow indicator shows that water is flowing through the core bit
 - Press the core bit gently against the base material. Take care to ensure that the core bit is perpendicular to the base material. The pressure applied to the core bit should be regulated so that the tool continues to run at its highest speed. Application of higher pressure does not increase the rate of drilling progress.
 - Hold the tool straight. Do not tilt it at angle as this may result in reduced drilling performance. Always ensure that the tool continues to run at a high speed.
 - Immediately after beginning drilling, check the water flow rate indicator to ensure that water is flowing.



Photo 2.3.7-3 Microcore Drilling

3) Removing the core from the hole

- Take the core removal tool out of the toolbox. Ensure that the diameter of the core removal tool corresponds to the diameter of the core bit used.
- Push the core removal tool into the hole as far as it will go while rotating it slightly.
- Break the core by applying slight lateral pressure to the core removal tool.
- Use the core removal tool to pull the broken core out of the hole. Turn the core removal tool through 180° and re-insert it in the hole.
- Use a ruler to measure the effective hole depth reached.
- Repeat this procedure, if necessary, several times until the entire core has been removed.



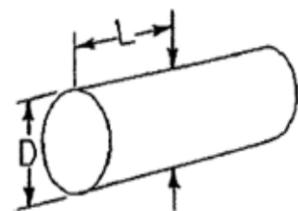
Photo 2.3.7-4 Core Removal Tool and Core Sample

b) Point Load Test (Reference standard ASTM D5731)

1) Diametrical Test

- Mark the desired test orientation on the sample with lines along the surface. These lines are used for centering the sample before the test and checking the proper stress orientation along the compression to failure.
- Close the valve of the hydraulic circuit of the hand pump. Insert the extension rod into the jack lever and zero set the digital readout unit (make reference Instruction Manual for the use of the digital manometer).

$$L > 0.5D$$



**Figure 2.3.7-1
Core Configuration
for Diametrical Test**

- Insert the sample between the conical points along a direction perpendicular to the end faces of the core and act on the jack to close the platens to the core. The core surfaces generally consist of failure planes.
- Before starting compression, check that the conical points will contact the core sample along the diameter D and that the distance L between the contact points and the nearest free end of the core will be as follows:
- Use the graduated scale inserted on the frame of the machine to record the distance D of the points to the nearest $\pm 2\%$.
- Check that the digital manometer records a small load and operate with the pump to increase the load steadily such that the failure occurs within 10 to 60 sec.
- Record the maximum force displayed by the digital manometer (the peak load value is frozen on the display) and measure again the distance D' of the points. If a partial failure occurs, the test result is not considered.
- Open the hydraulic circuit of the pump and push down manually the loading piston, to start again with a new test of core samples. Where possible, the test is repeated with at least 10 core samples.

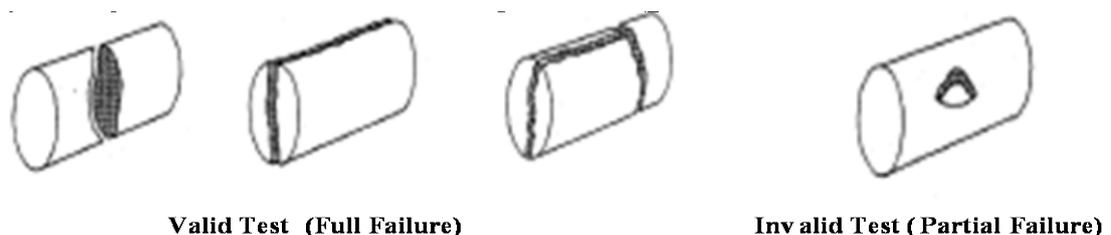


Figure 2.3.7-2 Valid and Invalid Diametrical Test

2) Axial Test

- Measure first the size of the core sample, in order to check that the length/diameter ratio is between $1/3$ and 1 . Mark the desired test orientation on the sample with lines along the surface. These lines are used for centering the sample before the test and checking the proper stress orientation along the compression to failure.

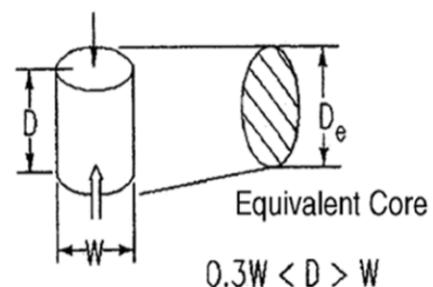


Figure 2.3.7-3 Core Configuration for Axial Test

- Close the valve of the hydraulic circuit of the hand pump. Insert the extension rod into the jack lever and zero set the digital readout unit.

- Insert the sample between the conical points along a direction perpendicular to the end faces of the core and act on the jack to close the platens to the core. The core surfaces generally consist of failure planes.
- Use the graduated scale inserted on the frame of the machine to record the distance D of the points to the nearest $\pm 2\%$.
- Check that the digital manometer records a small load and operate with the pump to increase the load steadily such that the failure occurs within 10 to 60 sec.
- Record the maximum force displayed by the digital manometer (the peak load value is frozen on the display) and measure again the distance D' of the points. If a partial failure occurs, the test result is not considered.

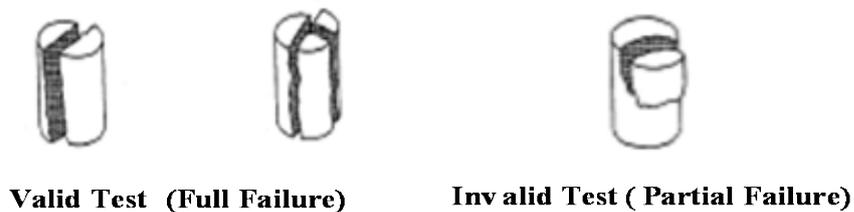


Figure 2.3.7-4 Valid and Invalid Axial Test

3) Test Points

- One test result needed for superstructure and one for substructure.
- One coring piece can be divided to two or three test pieces.
- Be careful so that coring does not affect structure.
- If test results are less than design compressive strength, then take another core and test again.

2.3.7.4 Analysis

1) Calculate Point Load Strength Index I_S (MPa) as follows:

$$I_S = P \times 1000 / D_e^2$$

where:

P : Peak resistance expressed in kN

De: Equivalent diameter of the core expressed in mm

For diametrical test : $D_e = \text{diameter}$

For axial test : $D_e^2 = 4 \times A / \pi$

where : $A = W \times D$

2) Apply size correction factor if core sample diameter is other than 50mm, as follows:

$$I_{S(50)} = F * I_S$$

Where: $F : (De / 50)^{0.45}$

3) Calculate Mean Value

To calculate the average point load index $Is_{(50)}$, at least 10 point load tests are required. From these tests reject the highest and the lowest strength value, and average the eight remaining test results to get the mean value.

4) Estimate Compressive Strength, as follows:

$$\delta_{uc} = C \times Is_{(50)}$$

Where:

δ_{uc} : Uniaxial Compressive Strength

C: factor that depends on site-specific correlation between δ_{uc} and $Is_{(50)}$

$Is_{(50)}$: Corrected Point Load Strength Index

Table 2.3.7-1 Values of “C”

Core Diameter, mm	Value of “C”
20	17.5
30	19
40	21
50	23
54	24
60	24.5

2.3.7.5 Evaluation and Recommendations

- Microcoring has the advantages of cores being easily drilled and cut with minimum damage to structures and requires use of a portable lower-capacity machine.
- Cores also utilize for carbonation test.
- PLT is used as an index test for strength classification of materials such as existing concrete
- PLT requires smaller breaking force so that a small and portable testing machine may be used.
- Cores can be tested in the field without machining since apparatus is portable.

2.4 NDT for Steel Attributes

2.4.1 Paint Thickness Test

Coating thickness measurements are based on the fact that the inductance between an electromagnet and a metallic surface varies with the thickness of a nonmagnetic interface coating. Changes in the inductance are electronically transmitted to a digital measurement of the coating thickness.

2.4.1.1 Fundamental Principles

a) General

When iron is moved toward or away from a steel core coil which carries electric current, self-inductance increases or decreases in accordance with distance. This change in inductance can be read as a deflection of needle of an ammeter, or a digital indicating signal connected as shown in Fig.2.4.1-1 below. This principle is utilized to measure the thickness of non-magnetic surface treated film (D) on iron substrate.

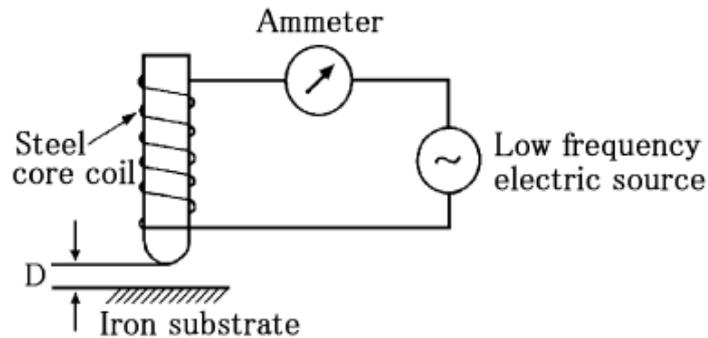


Figure 2.4.1-1 Principle of Coating Thickness Measurement on Surface of Iron Substrate

b) Applications

The technique is applicable for non-destructive measurement of the thickness of a non-magnetic coatings, linings and platings applied to iron substrate including steel and ferritic stainless steel (SUS 430 etc.) such as:

- Paint films on machines, appliances, automobiles, furniture, bridges, ships, steel tubes, cast iron tubes, structures, etc.
- Lining films Resin, tar epoxy, rubber and enamel film etc.
- Non-magnetic plating such as chrome, zinc, copper, tin, electroless nickel,(excluding electrolysis nickel) etc.
- Metalikon, phosphoric acid films, oxide films, spray deposit films, etc.
- Resin films and non-magnetic metal sheets(on iron substrate)

2.4.1.2 Description of Apparatus

Photo 2.4.1-1 Paint/Coating Thickness Meter



Specifications

Name	- UNIBOY-M
Measuring Method	- Electromagnetic induction
Measuring Range	- 0 to 3.00 mm
Resolution	- 1 μ m: within range of 0 to 999 μ m 0.01mm: within range of 1.00 to 3.00 mm
Accuracy	- \pm μ m on uniform surface or \pm 3% of reading
Display	- LCD with display hold function
Probe	- One-point contact pressure type with V-groove, \varnothing 17, Built-in electrode treated with CVD material of super abrasion resistance
Power Source	- Dry batteries R03 (1.5V) x 2 pcs
Dimensions	- 56 x 109 x 34 mm
Weight	- 140 g (including batteries)
Accessories	- Standard thickness plate x 2 pcs, Zero plate for testing

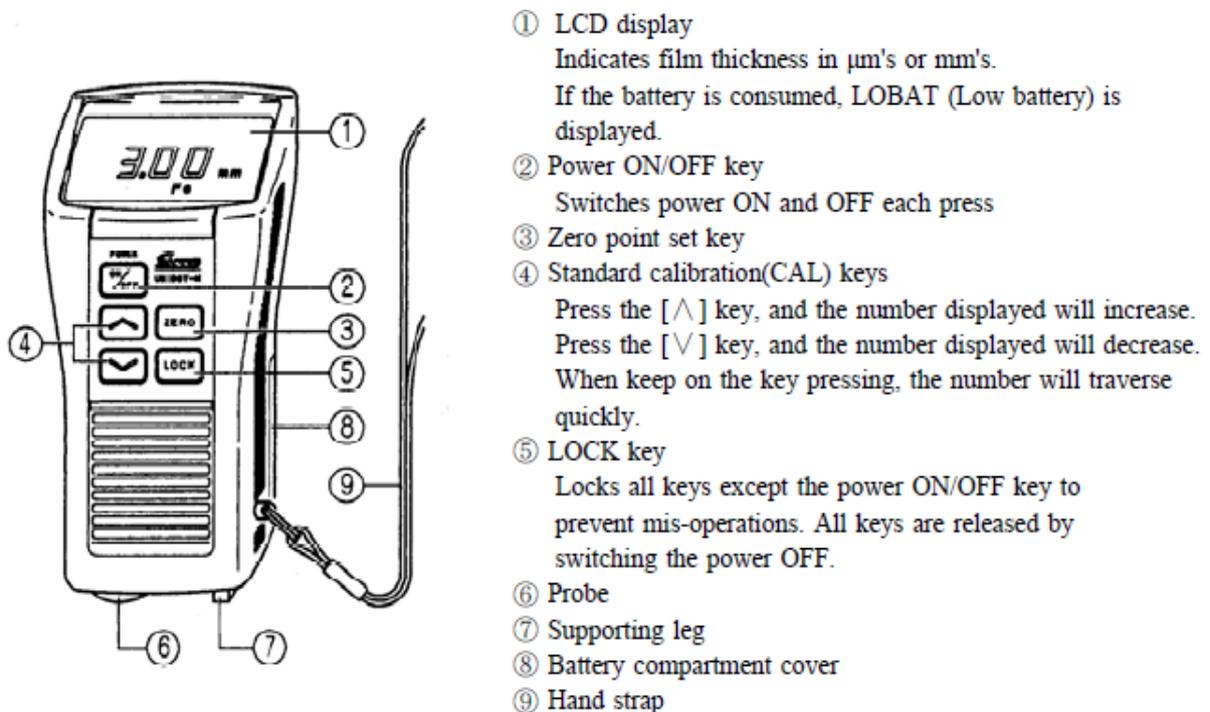


Figure 2.4.1-2 Parts Identification

2.4.1.3 Procedures

a) Operating Instructions

Refer to Operation Manual for adjustments, calibration and proper usage.

b) Investigation

Before testing, the following items should be investigated:

1. Design film thickness
2. Paint material used
3. Painting work records (Date/s when the painting work was done.)
4. The latest test results (thickness)

c) Measurement

1. Press “Power On/Off”. When the buzzer emits beeping sound, the meter is ready.
2. Caution: Press the LOCK key during the measurement to avoid errors and hold the apparatus as shown in diagram below and press it vertically on surface to be measured.



Figure 2.4.1-3 Measurement

3. Press the probe against an object to be measured and the buzzer emits a beeping sound and the measured value is indicated on the LCD.
4. When no beeping sound is emitted, measure again after moving the meter 4 to 5cm or more from the object and waiting for a little while.
5. Record measured film thickness

d) Test Points

1. Measure thickness of paint in good condition as reference.
2. Measure thickness of deteriorated paint areas

2.4.1.4 Analysis

When LCD read out appears as one of 0 to 999, unit of measurement is in μm (1/1000mm).
For example: 0.2mm is 200 μm

When LCD read out appears as one of 1.00 to 3.00, unit of measurement is in mm

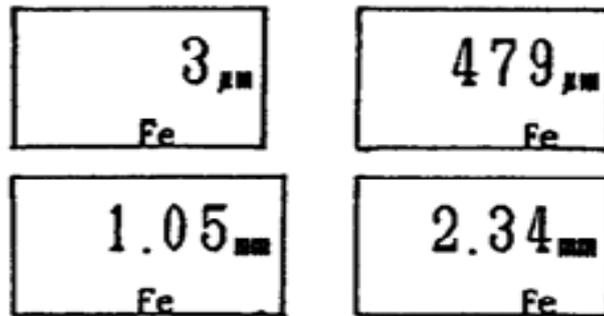


Figure 2.4.1-4 Examples of Read Out

2.4.1.5 Evaluation and Recommendations

- Control of the paint film thickness is important for steel structures. Condition of paint influences the service life of the structure. Therefore, paint film thickness should be inspected at fabrication shop immediately.
- Paint film thickness should be inspected periodically. To avoid paint damage, the surface should be cleaned regularly.
- If the existing film thickness is found to be less than the designed thickness but there is no damage, condition should be monitored. Additional painting will be required in case of new fabrication.
- If damaged, the repair method should be planned based on the degree of damage.
- The coatings must be non-magnetic with a thickness preferably less than 1 mm. The method can be used on paint, plastic, vitreous, enamel, galvanized and hard chrome, to name examples of non-magnetic coatings
- The reliability of the readings depends on the calibration, the test item and the geometry of the item.
- With the exception of thin coatings (less than 0.005 mm), the accuracy is usually within $\pm 3\%$ if the equipment is calibrated for the thickness to be measured. Otherwise, the precision is $\pm 5\%$. The minimum thickness that can be measured without special calibration is 0.003 mm.

2.4.2 Metal Thickness Test

The thickness gauge is used to determine the remaining thickness of corroded metallic items.

2.4.2.1 Fundamental Principles

a) General

This ultrasonic nondestructive testing is used in characterizing material thickness, integrity, or other physical properties by means of high-frequency sound waves. It has become a widely used technique for quality control. In thickness gaging, ultrasonic techniques permit quick and reliable measurement of thickness without requiring access to both sides of a part. Accuracies as high as ± 1 micron or ± 0.0001 inch are achievable in some applications.

Precision ultrasonic thickness gauges usually operate at frequencies between 500 KHz and 100 MHz, using piezoelectric transducers to generate bursts of sound waves when excited by electrical pulses. Typically, lower frequencies will be used to optimize penetration when measuring thick, highly attenuating, or highly scattering materials, while higher frequencies

will be recommended to optimize resolution in thinner, non-attenuating, non-scattering materials.

A pulse-echo ultrasonic thickness gauge determines the thickness of a part or structure by accurately measuring the time required for a short ultrasonic pulse generated by a transducer to travel through the thickness of the material, reflect from the back or inside surface, and be returned to the transducer. In most applications this time interval is only a

few microseconds or less. The measured two-way transit time is divided by two to account for the down-and-back travel path, and then multiplied by the velocity of sound in the test material. The result is expressed in the well-known relationship:

$$d = \frac{V_t}{2} t$$

where :

- d = the thickness of the test piece
- V = the velocity of sound waves in the material
- t = the measured round-trip transit time

b) Applications

Thickness of most engineering materials can be measured ultrasonically, including metals, plastic, ceramics, composites, epoxies and glass. In the field, it is used to determine remaining plate thickness of corroded steel bridge structures. In the fabrication shop, it is used to test uniformity of thickness of steel plates used in various structures

2.4.2.2 Description of Apparatus



Photo 2.4.2-1 Ultrasonic Thickness Gauge

Specifications

Name	:	Ultrasonic Thickness Gauge
Display type	:	4-digit LCD (128X64MM) with back light
Display resolution	:	0.001 inch or 0.01mm
Measuring range	:	0.040 to 15.75 inches (1 to 400mm) in steel
Memory Capacity	:	5000 readings
Sound velocity range	:	3280-32800 ft/s (1000-9999 m/s)
Operating temperature	:	32°F to 122°F (0°C to 50°C)
Frequency	:	5 MHz Output
Connection to Computer	:	USB communication with software

Standard Accessories:

- Transducer 5 MHz
- Integrated Steel Calibration Plate 4.0mm

2.4.2.3 Procedures

a) Operating Instructions

Refer to the Operation Manual for adjustments, calibration and proper usage.

b) Investigation

Before testing, the following items should be investigated:

1. Design steel thickness
2. Repair work records
3. The latest thickness test results



Photo 2.4.2-2 Operation of Ultrasonic Thickness Gauge

c) Measurement

1. Press the probe against an object to be measured and the measured value is indicated on the LCD.
2. Record measured steel plate thickness and its location on the plan.
3. Repeat measurements at other locations as necessary
4. Measure thickness of both corroded and un-corroded steel for comparison. Be sure to remove rust on corroded surface by sanding before measuring

d) Test Guidelines

The transducer should be placed on the surface of the test item. The ultrasonic waves will then be reflected by the opposite surface. The thickness is shown digitally. For uneven or corroded surfaces, it may be necessary to grind the surface at the test positions to make proper contact. Possible rust on the opposite surface does not disturb the reflections. If steel plates are laminated, the measured thickness will only be the depth of the first layer.

Before starting the measurements, a contact liquid is applied to the test locations. Further, the equipment must be calibrated. For common steel alloys, the calibration is performed by means of test blocks. For unknown alloys (or if you are not sure), the calibration is

performed by adjusting the sound velocity setting of the equipment until the equipment shows the same thickness as can be measured by a slide caliper at a free edge.

e) Test Points

1. Measure thickness of good condition metal for reference.
2. Measure thickness of deteriorated metal and analyze deterioration area.

2.4.2.4 Analysis

Record the measured thickness shown in the LCD screen at each test location on the drawings for easy analysis of results. The unit stores up to 5000 readings in its memory. Using USB interface and software, records are also transferred to the computer. Once transferred, records stored in unit can be erased for next use.

2.4.2.5 Evaluation and Recommendations

- Steel structures should be inspected regularly. To avoid damage, surfaces should be applied with appropriate paints.
- If the existing plate thickness is found to be less than the designed thickness due to corrosion, repair method should be planned based on the degree of damage.
- The method is generally applicable and is particularly efficient for measuring possible corrosion of structural elements with access only possible from one side. The gauge uses dual transducers to measure the thickness of corroded, pitted, scaled, granular materials from one side only.
- Thicknesses of between 0.50 mm and 200 mm can usually be determined within $\pm 2\%$.

2.5 Special Non-Destructive Tests

2.5.1 Ultrasonic Flaw Detection Test

2.5.1.1 Purpose

Ultrasonic flaw detection test can detect surface and intentional defects using high frequency elastic pulse wave.

The ultrasonic flaw detection test is that a transmitter transducer introduces acoustic energy of high frequency inside the specimen which propagates in the form of waves. When there is a discontinuity (such as a crack) in the wave path, a part of the energy will be reflected back from the flaw surface. This reflected energy is picked up by a receiver transducer and is an indication of the size of the defect.

Ultrasonic flaw detection test is used to assess the quality of the steel structure in relation to:

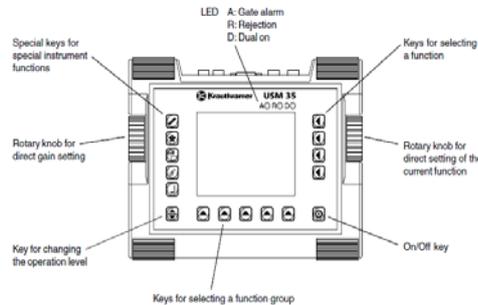
- a) Locating and evaluating defects of welds

- b) Locating and evaluating other flaws such as crack and void
- c) Measuring thicknesses

2.5.1.2 Description of Apparatus



USM 35



Controls



Probes

Figure 2.5.1-1 Ultrasonic Flaw Detector

This ultrasonic flaw detector has a frequency range from 0.5 to 20 MHz and a maximum calibration range of 10 m (steel), Probes connected by cables to the sockets at top right portion of the apparatus are used to send and receive ultrasonic pulses from test material which are then analyzed and displayed on the screen.

2.5.1.3 Test Procedures

Ultrasonic Testing require following procedures:

a) Test Procedures

a.1) Testing of Welds

Before testing, the type of welding (full-penetrated or partial-penetrated groove weld, fillet weld, etc.) should be determined. Ultrasonic test can only assess the full-penetrated weld

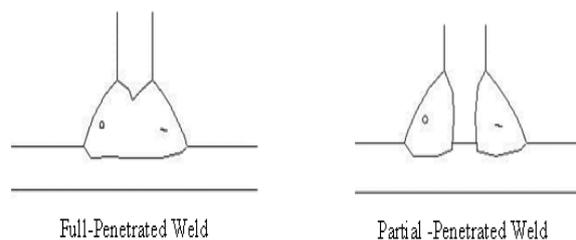


Figure 2.5.1-2 Types of Welding

a.2) Testing of Flaws

Use Straight Beam Probe which has a single transducer to transmit the ultrasonic pulses vertically into the test object and receive the returning reflections. They are suitable for detection and evaluation of flaws which reflect parts of the pulse back to the probe, e.g. shrinkage cavities, gas bubbles, inclusions or flat reflectors which are located vertically to the beam direction (cracks or bonding defects)

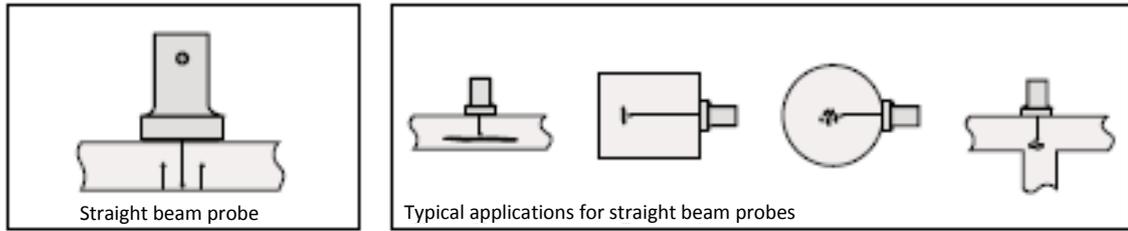


Figure 2.5.1-3 Types of Defects

In testing of the weld, the presence of defect (Flaw or “F”) is indicated by the spike in the graph. This is illustrated below:

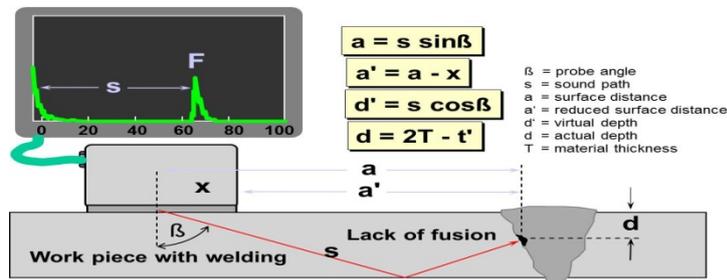


Figure 2.5.1-4 Testing of Flaws

In the detection of flaws, the presence of damage is explained by diagrams below.

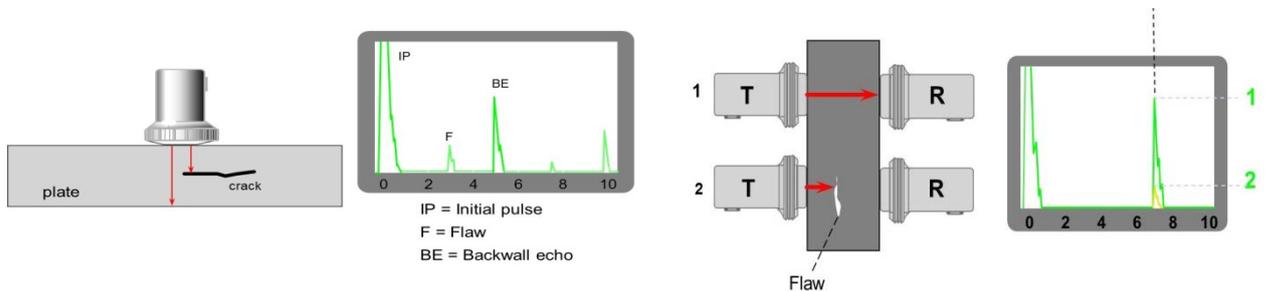


Figure 2.5.1-5 Detection of Flaws

2.5.1.4 Evaluation and Recommendations

- This test method can assess the defect occurring only in the “Full-penetrated” weld. Ultrasonic test should be carried out during steel structure fabrication at the shop for the quality control.
- The inspector should be trained on proper use of the apparatus.
- Advantages
 - Thickness and lengths up to 30 ft can be tested.
 - Position, size and type of defect can be determined.
 - Instant test results.
 - Portable
 - Extremely sensitive if required.

- Capable of being fully automated.
- Access to only one side necessary.
- No consumables required.

- Countermeasures
 - If there is defect on welding, the welding should be repaired in accordance with AWS1.1. Basically, the defect should be removed and re-welded.

After repair, ultrasonic test should be carried out again.

2.5.2 Static Load Carrying Test

2.5.2.1 Purpose

Bridge Load testing consists of determining the safe load carrying capacity of structures, determining if specific legal or overweight vehicles can safely cross the structure and determining if a structure needs to be restricted and the level of posting required.

Where member strength cannot be adequately determined from the results of in-situ material tests, load testing may be necessary.

Static load tests are useful in determining the maximum allowable loads on bridges. They can also be useful in discovering the mechanical properties of materials.

Static load tests are most likely to be needed in the following circumstances:

- Deterioration of structures, due to material degradation or physical damage.
- Structures which are substandard due to quality of design or construction.
- Non-standard design methods which may cause the designer, building authorities or other parties to require proof of the concept used.
- Change in occupancy or structural modification which may increase loading.
- Proof of performance following major repairs.

2.5.2.2 Test Procedures

a) Selection of Span and Method of Loading

In case of multi-span bridges, center span is chosen for load testing. The method of loading should be such as to either simulate the specific class of vehicles or induce in the members, the calculated forces or bending moments at critical sections.

Test loads may be in the form of mobile test vehicles (25 tons) and static loads.



20t Truck



Several Trucks



Measurement

Figure 2.5.2-1 Equipment and Apparatus for Static Load Test

b) Loading and Unloading Sequence

In case of static loads, the test load shall be applied in stages so that timely action such as stopping the test can be taken if any untoward distress is observed at any stage. In most cases, design live load effect would be equal to or less than that due to dead load. The suggested stages of test load placement are 30%, 50%, 70%, 80%, 90% and 100%. Unloading should also be in same stages. The next incremental loading should be added only after the deflections under the previous load have stabilized and all stipulated observations are completed.

c) Preparatory Works

- All visual defects should be measured, mapped and plotted.
- It should be ensured that bearings are functional
- Expansion gaps and joints should be cleared of all debris.
- It will be useful to give the surface of the superstructure a coat of white wash, so that appearance of cracks becomes immediately perceptible.

d) Observations

The following are observed, measured and recorded at regular intervals of one hour over a period of 24 hours.

- Deflections at critical sections (for instance, for simply supported spans, at mid-span and at quarter span. In box girders, it will be useful to record deflections under each of the external ribs)
- Appearance of cracks and their development, length, width, location, and orientation correlated with load.
- Deformation of bearings.
- Ambient temperature and related temperature in the body of the structure.

2.5.2.3 Measurement of Deflections

Deflections can be measured with the following devices:

- (a) Displacement Gauge
- (b) Ruler and Cursor
- (c) Deflectometer
- (d) Precision Level
- (e) Water Level

Methods (a) to (c) are used whenever dry river/stream bed is available under the span. Otherwise methods (d) and (e) can be used by using a reference station at the nearby abutment. When girder bridges are subjected to load tests, it is essential to clear debris in the expansion gaps and to lubricate steel bearings to permit free translation and rotational movements of the spans.

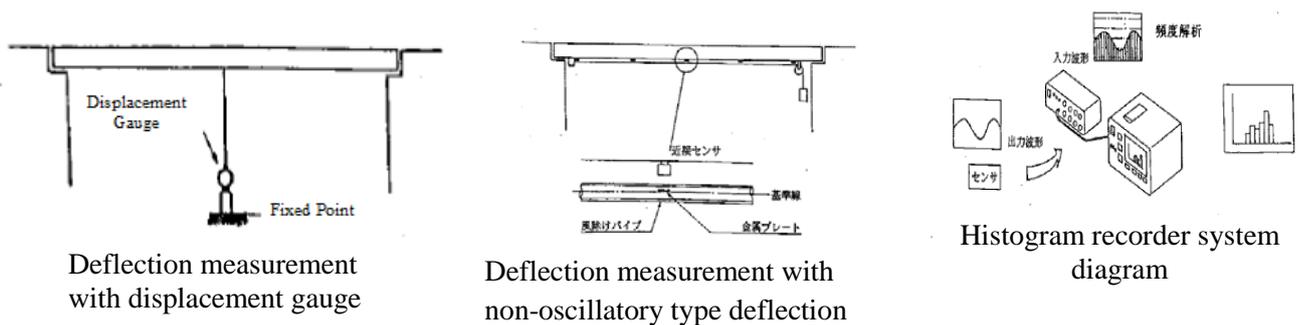


Figure 2.5.2-2 Measurement of Deflections

2.5.2.4 Percentage Recovery of Deflection

The percentage recovery is calculated for values of deflection. The percentage recovery is calculated at 24 hours after removal of the load. The calculation is done after applying temperature and rotation corrections to the deflection data.

2.5.2.5 Evaluation and Recommendations

Static load test can be direct calculation of the load capacity from the test results. It is assumed that the bridge assessment is carried out using the partial safety factor format and the load capacity is the value for which the rating live load should be multiplied to reach the failure limit state.

2.5.3 Dynamic Load Carrying Test

2.5.3.1 Purpose

As a complement to static load carrying test, dynamic tests provide useful information about the actual behavior of the bridge under traffic.

Dynamic load tests are identified in the following circumstances:

- Deterioration of structures, due to material degradation or physical damage.
- Structures which are substandard due to quality of design or construction.
- Non-standard design methods which may cause the designer, building authorities or other parties to require proof of the concept used.
- Change in occupancy or structural modification which may increase loading.
- Proof of performance following major repairs.

2.5.3.2 Test Procedures

Dynamic load testing is performed by exciting vibration of the bridge and by measuring its properties after the excitation vibration has ceased. Several methods are available to generate vibration on the bridge, in particular the passage of a loaded truck. This method is often preferred for the dynamic load testing of bridges because it gives, along with reasonably accurate values of the above mentioned quantities, a good approximation of the effect of the actual traffic on the structure. By varying the speed of the truck on the bridge, the full range of traffic speeds can be investigated. Furthermore, this method is easily implemented.

The measurements are taken and recorded by a dynamic data acquisition system, allowing an immediate interpretation of the results during the test. The trucks used for the dynamic excitation of the bridge are usually 3- axle trucks, with a total weight of 250 kN (total mass of 25 metric tons), traveling on the bridge at several speeds. The effect of a deterioration of the pavement is simulated by the introduction of a wood plank on the path of the truck. This induces a strong impact when the trucks passes at mid-span, that represents the effect of a pothole in the pavement, or the irregularity of the surface.

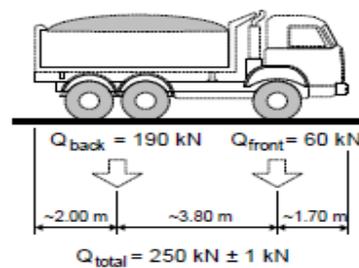


Figure 2.5.3-1 Typical Truck Loading for Dynamic Load Test



Photo 2.5.3-1 Placement of Strain Gauges for Dynamic Load Test

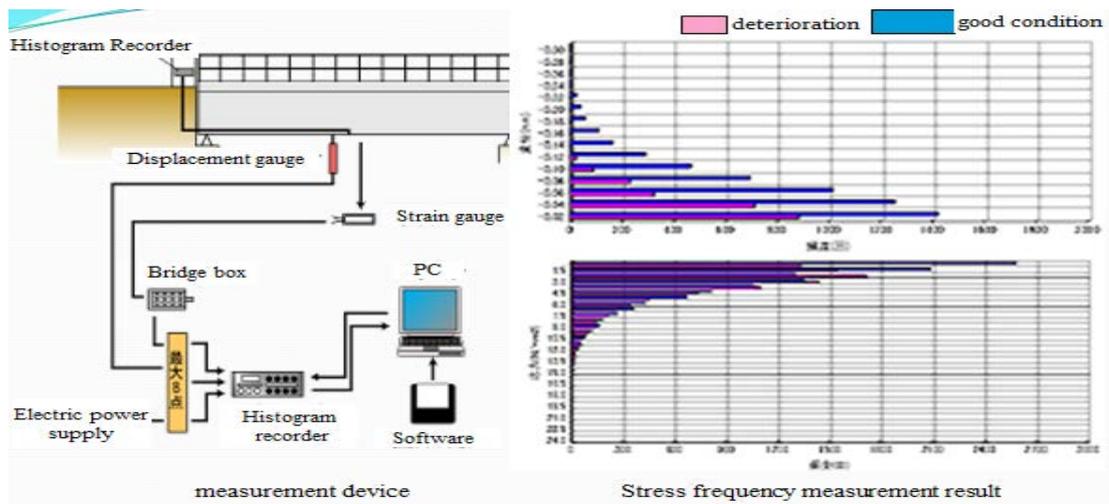


Figure 2.5.3-2 Dynamic Load Test Data Acquisition and Processing System

The dynamic tests can be performed using the following loadings:

- normal traffic, test vehicle or engine
- sudden release of deflection by realizing a load attached to the structure
- sinusoidal exciter, energy input device, braking a vehicle or engine on the bridge
- impact produced by a vehicle running through the standard bar (in case of highway)
- In general, the tests are most often performed under normal traffic or under the test vehicles.

2.5.3.3 Recommendations

Dynamic load testing is usually performed on bridges that have also been subjected to static load testing, comparisons can be made between the behavior of a bridge under static and dynamic loading.

By using a truck to induce vibrations on the bridge, it is possible to simulate the effects of pavement deterioration. Bridges that are especially sensitive to pavement deterioration are identified, and this information can then be used in establishing the maintenance program of the structure. Truck speed is shown as having a great importance on the dynamic response of the bridge, especially in the case of a deteriorated riding surface. There is correlation between the span and the natural frequency of a bridge and in a similar manner, there is some correlation between the bridge stiffness observed in a static load test and the dynamic properties of the bridge.

2.5.4 Ambient Vibration Test

2.5.4.1 Purpose

Ambient Vibration Test is used to determine the dynamic characteristics of structures through ambient vibration measurements. The ambient vibration behavior of a structure is recorded, evaluated, i.e. without artificial excitation, by means of highly sensitive velocity or acceleration sensors. For large and flexible structures, such as suspension bridges,

cable-stayed bridges and other large structures, it becomes too difficult and costly to provide controlled excitation (forced testing) at levels which are significantly higher than the excitation provided by ambient sources. The method only requires the measurement of the response to ambient excitation which might be caused by wind, traffic, waves or micro-seismic activity

2.5.4.2 Test Procedures

Ambient vibration tests are conducted on the bridge using a channel data acquisition system with accelerometers. Measurements are taken continuously for three days, normally at the bridge center span.

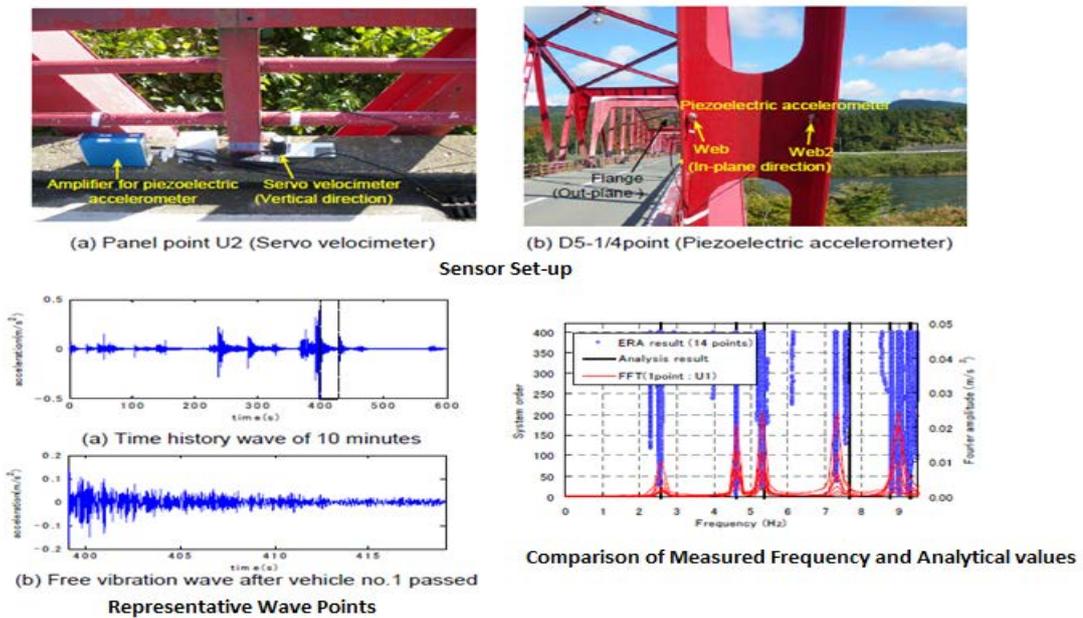


Figure 2.5.4-1 Ambient Vibration Test Measurements

Damping coefficients are the third factor used for describing the dynamic response of a structure. The frequency dependent damping ratios are important criteria for structural assessment due to the fact that these ratios increase significantly when the structural resistance decreases – in other words, high damping ratios are an indicator for reduced safety.

2.5.4.3 Evaluation and Recommendation

Ambient vibration test has an advantage of being simple, fast and cheap, which can be implemented simply by measuring the responses data. The ambient vibration test has been widely used in bridge structural detection.

The main advantage of this method is that normal operation, such as traffic, is not influenced or interrupted during testing. Traffic is a welcome source of excitation which usually provides good wide-band excitation.

The dynamic behavior of a structure depends on:

- a) Mode shapes corresponding to natural frequencies
- b) Vibration limits
- c) Damping coefficients

The real vibration shapes of a structure consist of the mode shapes corresponding to the natural frequencies. Therefore mode shapes are - beside the natural frequencies - the second important quantities that describe the dynamic behavior of a structure. Acceleration records are transformed to displacement values for each natural frequency at each measurement station. The measured mode shapes are then compared to the computed ones

The consideration of vibration limits is important for the maintenance of structural integrity. Structural vibration limits for particular damage risks can be classified according to the level of vibration intensity.

2.5.5 Impact Vibration Test

2.5.5.1 Purpose

Impact vibration test is conducted to determine effects of severe disasters on the soundness of a structure. When an earthquake or scouring occurs, the mass of the structure does not change, however, stiffness and spring constant values drops. Impact Vibration Test can analyze the bridge soundness by natural frequency accurately.

2.5.5.2 Applications

Test will be applied quickly and accurately for following situations:

1. Normal and emergency
 - Determination of structural soundness by deterioration
 - Determination of structural soundness by collision
 - Understanding of the influence by close-set construction
2. Disaster
 - Determination of damage by earthquake
 - Determination of damage by flooding
 - Selection of the degree traffic regulation individualized damage

2.5.5.3 Test Procedures

- Testing is done by recording the response wave caused by hitting the bridge pier with heavy steel ball (bob)
- After analyzing the wave response, the natural frequency level of the bridge is determined.
- Evaluation of bridge structural soundness is quantitatively analyzed without any damage to the structure.
- With drop in level of stiffness and spring constant values, the level of structure damage can be estimated.

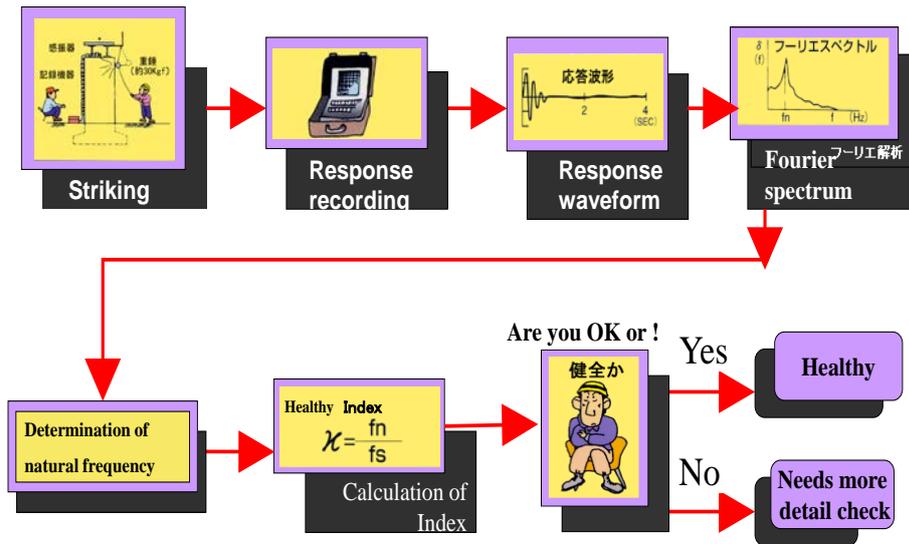


Figure 2.5.5-1 Flowchart of Impact Vibration Test



Photo 2.5.5-1 Impact Vibration of Bridge Piers



Photo 2.5.5-2 Recording of Wave Response

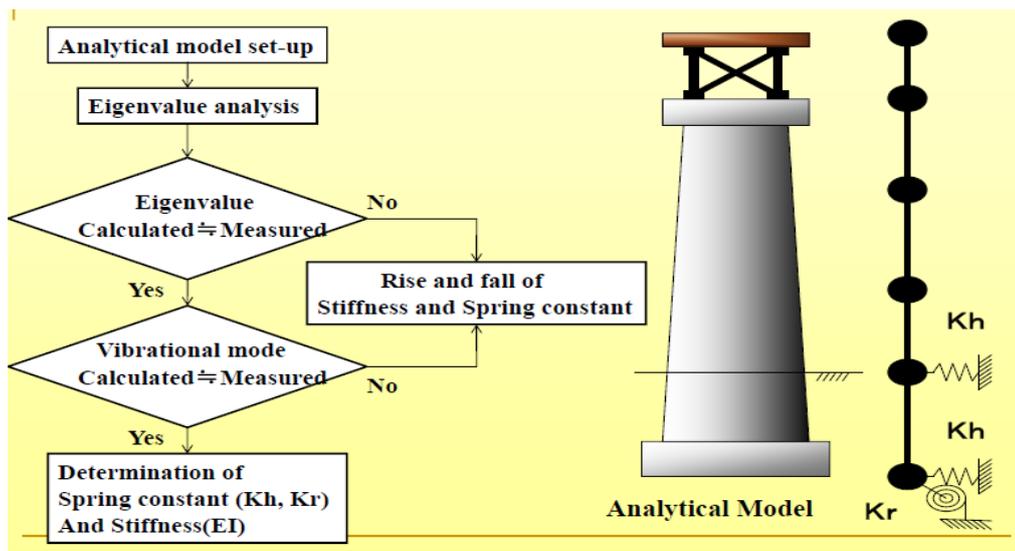


Figure 2.5.5-2 Procedure of Estimation of Stiffness and Spring Constant

2.5.5.4 Soundness

The soundness of the bridge structure can be determined by comparing the measured stiffness value with design stiffness value as well as the measured spring constant values with the design value. For example, in a railway bridge, the following damage criteria have been determined:

Table 2.5.5-1 Soundness of Structure (Railway Bridge)

a = (Measured Stiffness Value) / (Design Stiffness Value)		
Range of a	Class	Observation/Countermeasure
$1.00 \leq a$	S	Structure is sound
$0.75 \leq a < 1.00$	C	There are minor deformation or deflection of structure. Requires routine monitoring
$0.50 \leq a < 0.75$	B	There are possible deformation or deflection of structure. Requires routine monitoring. Some actions may be taken requires appropriate actions
$a < 0.50$	A2	There are possible deformation or deflection of structure. There is threat on safety of travelling public and operations. Actions to be taken

Table 2.5.5-2 Soundness of Footing (Railway Bridge)

a = (Measured Stiffness Value)/(Design Stiffness Value)		
a	Class	Observation/Countermeasure
$a < 0.50$	A2	There are possible deformation or deflection of structure. There is threat on safety of travelling public and operations. Actions to be taken
$0.50 \leq a < 0.75$	B	There are possible deformation or deflection of structure. Requires routine monitoring. Some actions may be taken
$0.75 \leq a < 1.00$	C	There are minor deformation or deflection of structure. Requires routine monitoring

When it is affected by disasters such as scouring or earthquake, the mass will not be changed, but stiffness will drop to a lower value and the spring constant of footing will also drop.

The natural frequency should be fixed by the mass and spring constant, therefore the natural frequency is dropped to a lower value. With dropping level of the stiffness and ground strength, the level of structure damage will be estimated.

2.5.5.5 Evaluation and Recommendation

Impact Vibration Test is used to:

- 1) analyze the bridge soundness by natural frequency;
- 2) take inspection easily;
- 3) measure natural frequency accurately; and
- 4) determine soundness even if there may be defects are hidden in the structure

2.5.6 SONAR Test (For Scouring)

2.5.6.1 Purpose

Earthquakes, flood scouring, and ground movement are the main causes of damage of bridge foundations. Scouring due to floods is the most common cause of bridge failures or severe damages which require repair or strengthening of bridge foundations. Consequently, it is absolutely necessary to investigate the scour condition as early and accurately as possible and to estimate the soundness of bridges in terms of keeping bridge foundations in good condition.

The use of SONAR (SOund Navigation And Ranging) to determine depth and extent of scour around bridge piers has been found to be an accurate and cost-effective technique. The equipment is a lightweight instrument and has capability to deal promptly with investigation since the work is conducted from the bridge deck or the inspection scaffold on the bridge inspection vehicle. Furthermore, the configuration of streambed is easily determined with display of an investigation results on a monitor screen with color image.

2.5.6.2 Outline of Testing

a) Scour Investigation around Piers

- a.1) Scour investigation around the pier on four (4) sides for each pier
- a.2) Periodic measurement: marking of sonar installation position is done on the bridge.
(Time-dependent change is recognized directly)

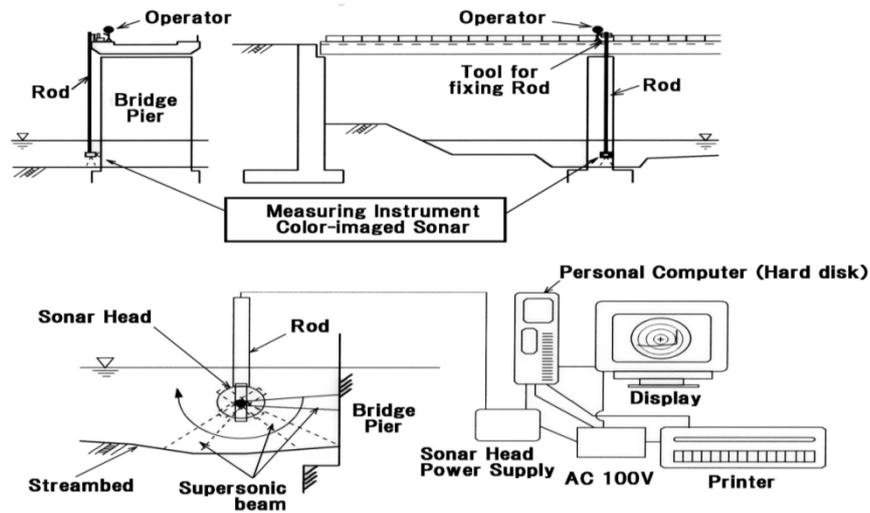


Figure 2.5.6-1 Outline of SONAR Testing

b) Procedures

First, a SONAR head attached to the edge of a sounding rod is brought down from the bridge deck or the inspection scaffold on the bridge inspection vehicle until about 1m under water, and then we can measure through going into a 360-degree roll of the sonar head.

c) System Configuration

Sonar: sending and receiving ultrasonic wave

Computer: creates the picture on the basis of signal sent from sonar

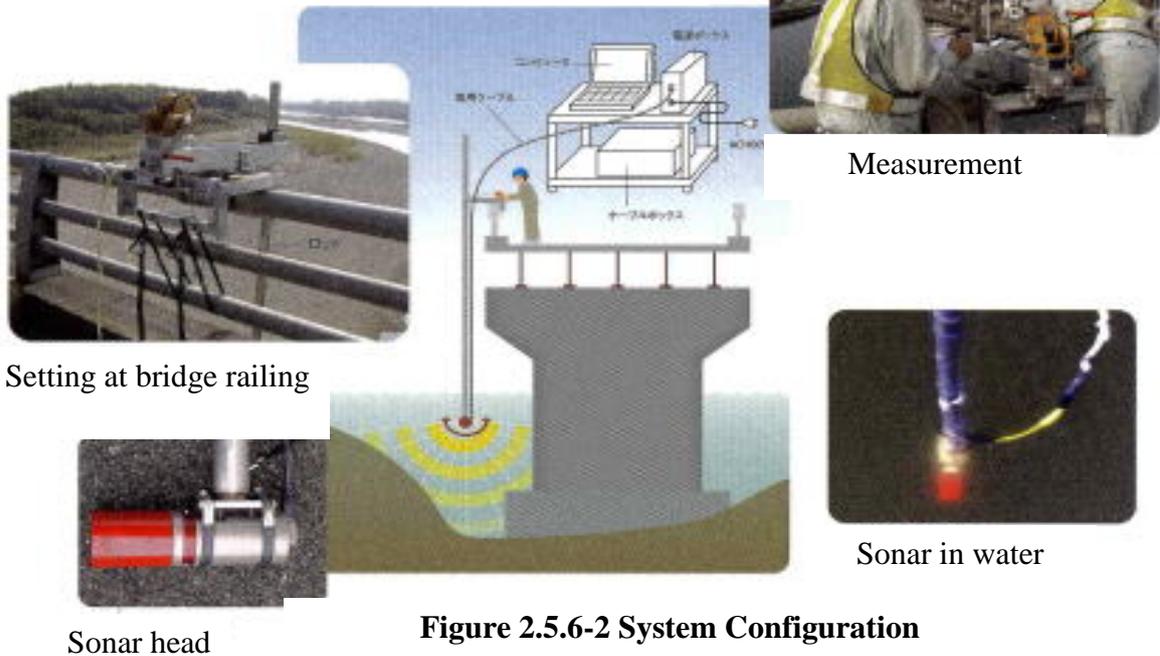


Figure 2.5.6-2 System Configuration

d) Output

After SONAR testing, the scouring is identified near the pier.

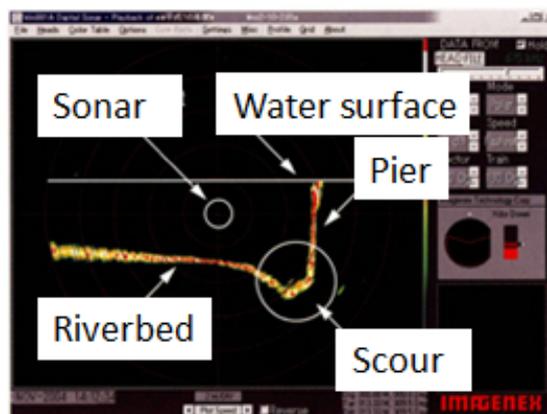


Figure 2.5.6-3 Screen Image of SONAR Test

2.5.6.3 Evaluation and Recommendations

Since sonar can measure one topographic profile only at a time, investigation should be conducted to determine streambed conditions around piers by rotating the sonar around horizontally every 30 degrees. If there are dead angles due to pier's location, measurement can be done while the sonar head is moving.

2.5.7 Impact Echo Test

2.5.7.1 Purpose

a) General

Impact Echo Test can be used to detect voids in post tension tendon ducts of pre-stressed slab and beams. Impact-Echo Testing is a nondestructive testing technique used to evaluate the quality of the concrete by detecting internal flaws, including delamination, honeycombs, voids, etc.

Generally, the tendons are protected by the surrounding grout, which is injected into the tendon ducts. Voids in the grout can occur along the tendon trajectory due to blockages, improper grouting procedures, grouting material problems, and construction oversight. Inadequate grouting may allow water to penetrate into the ducts, causing corrosion of the tendons leading to failure of the structures.

Impact Echo Test is a method for non-destructive evaluation of concrete based on the use of impact-generated stress (sound) waves that propagate through the structure and are reflected by internal flaws and external surfaces. A small steel ball is used to produce the impact.

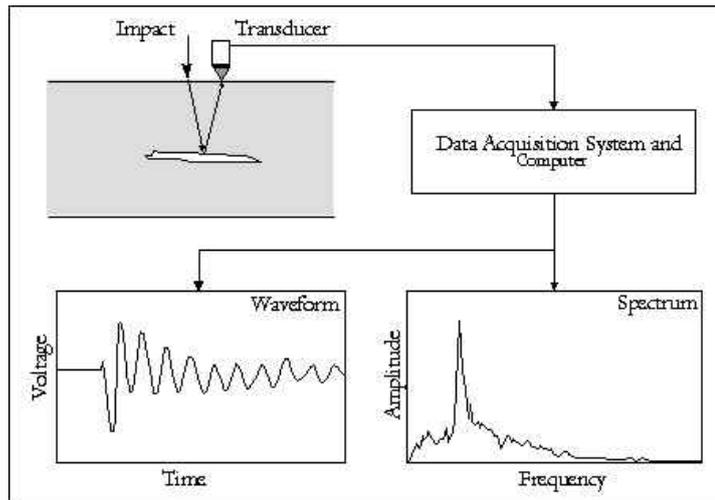


Figure 2.5.7-1 Flowchart of Investigation by Impact Echo Test

b) Applications

The successful outcome of this application depends on several factors, such as the geometry, shape of the test member, position and size of the tendon in the test member. For example, it is difficult to detect voids in tendon duct if the tendon duct is located within the flange of a typical I-beam. Similarly, if a small tendon duct is located in relatively deep portion of a slab or beam, detection is also difficult.

The applications of the impact echo technique include: determining both the thickness and flaws in plate-like structure members, such as slabs, walls; detecting flaws in beams, columns and hollow cylindrical structural members; assessing the quality of bond in overlays; and detecting void in post tension tendon duct etc.

2.5.7.2 Outline of Testing

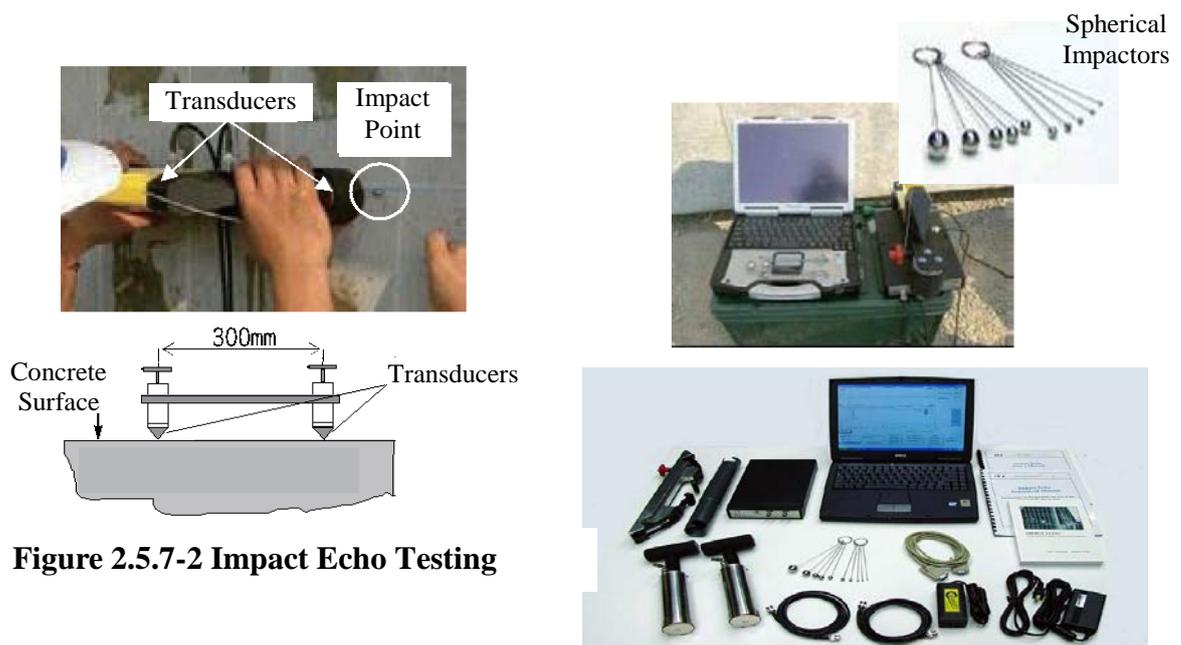


Figure 2.5.7-2 Impact Echo Testing

2.5.7.3 Evaluation and Recommendations

Advantages and limitations of this method are as follows:

- The equipment is relatively portable.
- Access to only one side of the structure is required.
- Provides the ability to collect and digitize large amounts of data.
- Field data acquisition is obtained and processed within a few seconds.
- Requires data interpretation and analysis, which typically requires an experienced operator.
- Relatively expensive equipment.
- Exact size of subsurface anomalies may be difficult to determine.

2.5.8 X-Ray Test

2.5.8.1 Purpose

Radiography technique is one of the non-destructive methods of testing concrete for obtaining information about concrete quality, defects within the pre-stressed concrete bridge.

X-Ray test of pre-stressed concrete girder bridges using high energy x-ray radiography is found to be useful in distinguishing grouted and un-grouted portions of cable sheaths; to find out the uniformity of the cement grout; to determine the condition of the cable sheath; and to determine whether the pre-stressing strands are ruptured or intact.

2.5.8.2 Description of Apparatus

X-Ray Test Method is used in civil engineering works notably to verify the integrity of pre-stressed wires in a pre-stressed concrete structure by using radioisotope sealed sources, X ray machines or linear accelerators. Figure 2.5.8-1 shows a typical set up of radiographic testing.

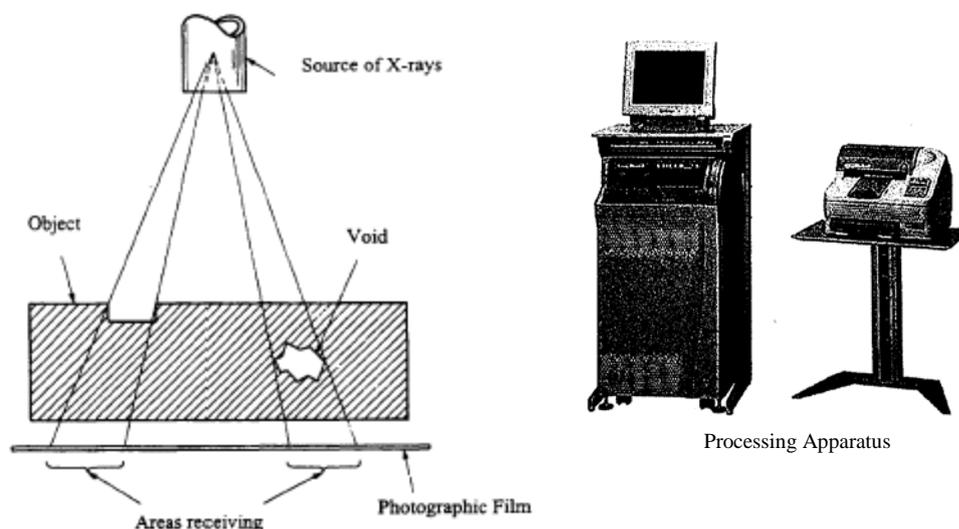


Figure 2.5.8-1 X-Ray Test Set-Up

2.5.8.3 Test Procedures

During the radiography X-rays penetrate through the material under inspection. While traversing through the material, these radiations experience modification by the internal structure of the material through absorption and scattering processes. If the internal structure is homogeneous, the absorption and scattering processes would be uniform throughout the material and radiations that escape from the material would be of uniform intensities.

These radiations are then recorded by a suitable recording medium, typically radiographic film. When the film is processed, a uniform dark image will appear on the film that indicates the homogeneity of the material tested. The situation is different for cases of materials containing discontinuities or different in thickness. In general, the absorption of radiation by a material depends on the effective thickness through which the radiations penetrate.

Discontinuities such as cracks, slag inclusions, porosity, lack of penetration and lack of fusion reduce the effective thickness of the material under test. Thus, the presence of such discontinuities causes radiations to experience less absorption as compared with those in areas with discontinuity. As a result, in areas containing discontinuities more radiations escape, recorded by the film and forming a dark image that represents the internal structure of the material.

The appearance of radiographic images depends on the type discontinuities encountered by the radiation. For example, cracks will produce a fine, dark and irregular line, whereas porosities produce dark round images of different sizes.

Some discontinuities present in a material such as inclusion of tungsten in steel has a higher density than its surrounding. In this case, the effective thickness that needs to be traversed by radiation is somewhat greater. In other words, more radiation is absorbed in this area as compared with other areas. As a result the intensity of radiation that escaped after traversing this area will be lesser than that for other areas giving a lighter image bearing the shape of tungsten inclusion inside the material.

2.5.8.4 Recommendations

X-Ray Test is used throughout the bridge inspection. Its capability to produce two-dimensional permanent images makes it. However, radiation used for radiography presents a potential hazard to radiographers as well as members of public. Due to its hazardous nature, the use of radiation, including for industrial radiography is strictly controlled by Regulatory Authorities.

X- Ray Tests are fast; they are capable of viewing interiors of concrete sections up to 3 ft (1 m) thick; they can generate radiation at the optimum energy level for a given specimen thickness; and they require minimal shielding since radiation emission can be terminated by turning off the electrical power supply. On the other hand, X-ray equipment is expensive and must be operated carefully because of high voltages and high radiation levels. Until recent miniaturization of X-ray generators such as gamma radiography equipment has generally been more portable. Gamma ray equipment is less expensive and

is easier to operate because electrical power is not necessary. The continuous radioactivity from the gamma ray sources requires more shielding and additional safety interlocks to prevent accidental exposures.

2.5.9 In-Situ Stress Measurement Test

2.5.9.1 Purpose

In-Situ Stress Measurement Test is used for measurement of stress in concrete. It has been derived from the mining industry where it is used to measure stresses in rock masses. The method is also widely used in metals or plastics. Its principle is simple: the strain field is relieved by coring or slotting the material; the change of the strain in the relieved area is measured and the stress is calculated taking into account the elastic properties of the material and the geometry of the cut.

2.5.9.2 Description of Techniques and Procedures

a) Strain Relief Method with Flat Jack Technique

In concrete, the modulus of elasticity varies with concrete mix, age, curing and environmental conditions. In order to eliminate the need to know the elastic properties of the concrete, a flat jack can be inserted into the slot and pressurized. As a result the initial strain field will be restored. The combination of the measured data on strain and pressure will give the value of the stress.

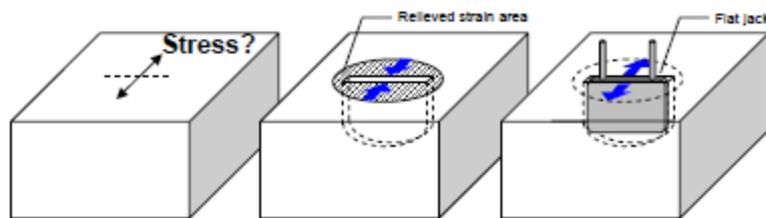


Figure 2.5.9-1 Principle of Strain Relief Method Combined with the Flat Jack Technique

The stress in the concrete is estimated from the value of the pressure that restores the initial distance between the 2 points:

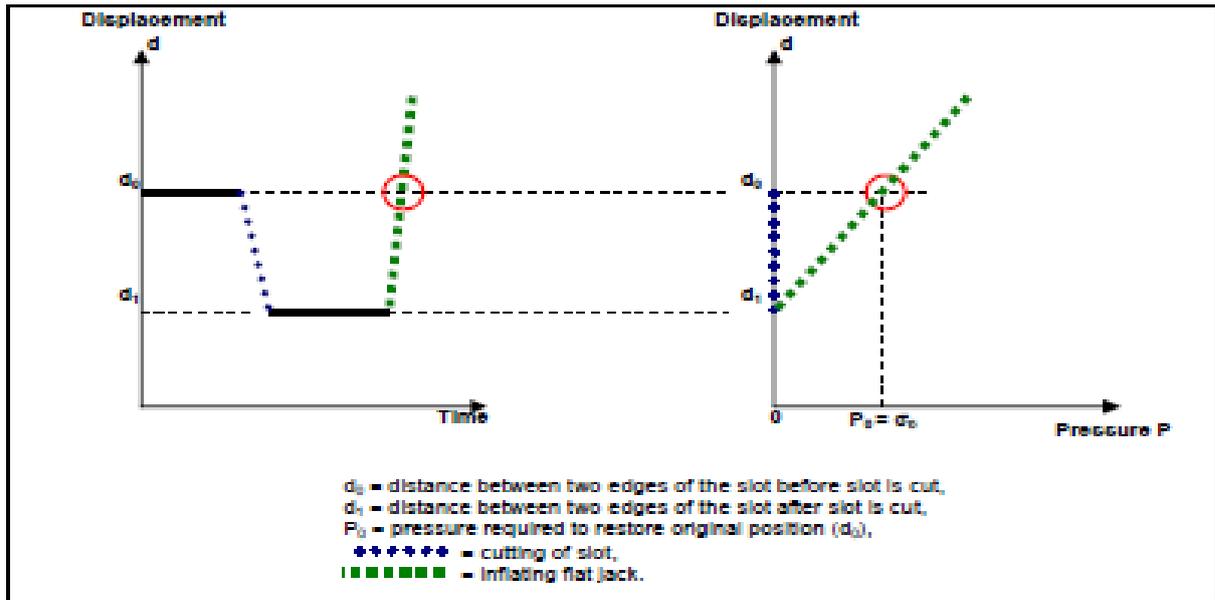


Fig.2.5.9-2 Basic Theory for Stress Calculation

When the concrete is in compression, the edges of the slot will move together and the pressure required to restore them to their original position corresponds to the local compressive stress perpendicular to the slot direction.

b) Hoop Stress-Releasing Technique

The hoop stress-releasing technique is similar to hole drilling stress-release method which is used to measure residual stress in metal. In this method, a 10mm long strain gauge is mounted in the center of a hoop marked on the concrete surface. The strain is then measured while the concrete surrounding the strain gauge is cut by a coring machine slowly.

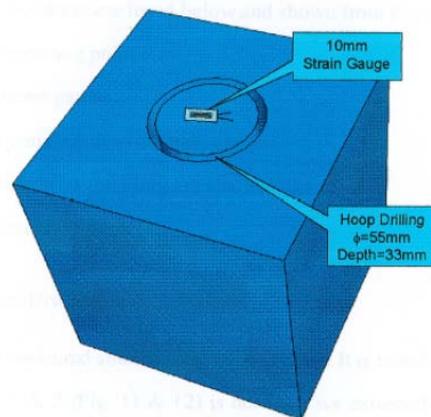


Figure 2.5.9-3 Diagram of Hoop Stress-Releasing Technique

The released strain is equal to the inverse of the original strain locked in the structure (say girder of the bridge). By measuring the released strain from the strain gauge, the original strain in the girder is obtained. The calculated axial force is equivalent to the effective pre-stress force acting on the section. The loss of pre-stress is then determined by comparing the original jacking force with the calculated axial force.



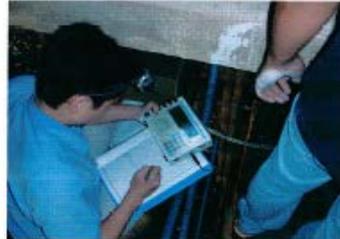
Step 1- Locate the measuring point



Step 2- Mount the strain gauge



Step 3 - Release stress by coring using drilling machine



Step 4 - Measure the released strain every 5mm core depth upto 33 mm

Photo 2.5.9-1 Procedures for In-Situ Stress Measurement Test Using Hoop Stress-Releasing Technique

2.5.9.3 Recommendations

Stress Measurement Test is recommended to be used in determining the effective pre-stress force acting on a structure subjected to damage by fire, overloading, rupture of pre-stressed tendons, and other causes.

The existing state of stress in a structural member as determined by Stress Measurement Test can be used by the engineer as reference in estimating extent of deterioration, updating of safety factors, forecasting the state of the structure and in optimizing the design of strengthening or repair solutions.

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9. Test Report on Katipunan Flyover Prestress Loss Measurement Project, Philippines; FF Cruz and Co., 2006

CHAPTER 3 ASSESSMENT

3.1 Scope

In bridge maintenance, the most important thing is to determine the cause of damage and to take countermeasures.

It is easy to identify the causes of damage in concrete bridges, such as cracking due to lack of load carrying capacity and traffic accidents. Condition Inspection can classify the defects by visual means.

However, it is difficult to predict deterioration due to carbonation and chloride. It is very important to determine “**prediction of deterioration**” or “**what has become of the current state of the bridge**”.

The aim of an assessment is to establish the safe load carrying capacity of the bridges. This is usually initially confined to the bridge superstructure which is regarded as the weakest element of most bridges, and then extended to bridge support and foundations.

Therefore, the focus of this chapter are: 1) to verify causes of damage based on results of condition inspection, 2) to identify/predict deterioration stage and bridge soundness based on results of NDT, 3) to determine appropriate repair methods for damages, and 4) to evaluate the overall assessment of bridges based on soundness of the bridge.

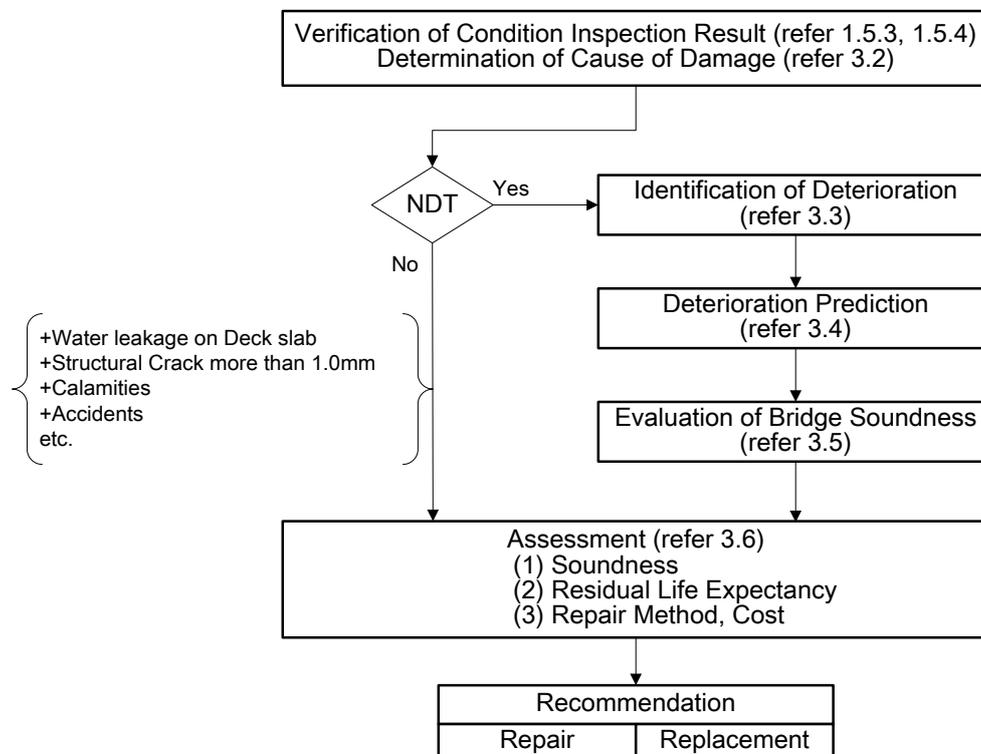


Figure 3-1 Flow Chart of Assessment for Bridge Engineering Inspection

3.2 Causes of Damage

3.2.1 Superstructure (Concrete Deck Slab and Girder)

3.2.1.1 Cracking

A crack is a linear fracture in concrete that extends partly or completely through the member. Cracks in concrete occur as a result of tensile stresses introduced in the concrete. All cracking in concrete members is caused by tensile stresses (concrete has a low tensile strength, but high compressive strength).

Structural cracks (load induced cracks) in concrete elements can be recognized as cracks with specific crack patterns related to each type of internal forces (bending, shear). Structural cracks might be a sign of structural deficiency. For reinforced concrete (RC) structures, cracks in most cases are not serious. RC is allowed to crack. The crack width and spacing will indicate whether there is something wrong or not.

Typical cracking patterns observed in reinforced concrete structures and the typical cause of each type of cracking are illustrated as follows:

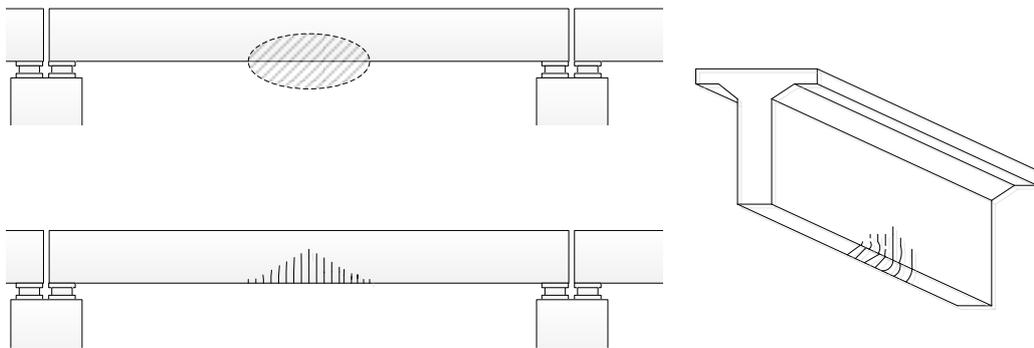


Fig. 3.2-1 Flexural Cracking at Midspan of Girder

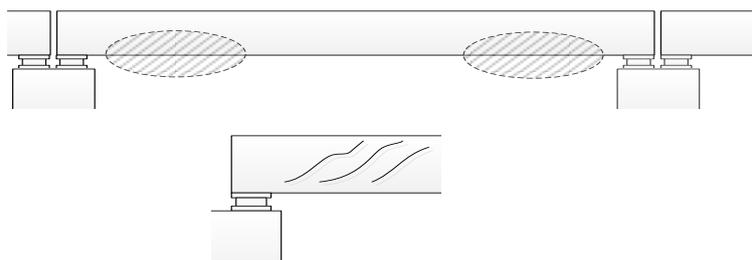


Fig. 3.2-2 Shear Cracking at L/4 of Span of Girder

For pre-stressed concrete, cracking is an indication of a potentially serious problem (overload, under-design, or design error). Normally no cracks should be visible.

From a structural point of view, it is important to distinguish structural cracks from the non-structural cracks caused by shrinkage, etc.



Photo 3.2-1 Shear Cracks



Photo 3.2-2 Bending Cracks



Photo 3.2-3 Shear Crack in Concrete Girder

a) Flexural Crack

For a beam subjected to bending, two different types of cracks will occur, refer to Figure 3.2-3. The first cracks to be formed emerge from the face under tension and extend to the neutral axis. They are called bending cracks.

When the bending moment is increased, new cracks will emerge from the face under tension to just beyond the main bars. These cracks are called tension cracks.

In a heavily reinforced beam with a depth (height) more than 0.4 m, these usually closer spaced tension cracks tend to join the bending cracks in the web forming a fork like crack pattern.

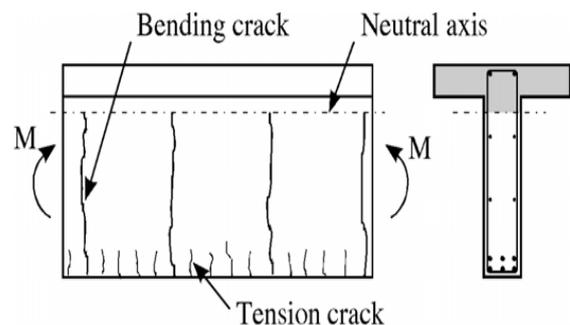


Figure 3.2-3 Flexural Cracks

The crack width of the bending cracks in the web above the main bars can be very high, if there is not sufficient longitudinal reinforcement in the web between the main bars and the compressive flange.

It must be noted that the bending cracks indicate the position of the neutral axis. This means that it is possible to compare this position with that found from calculations. Since the position of the neutral axis depends on the amount of reinforcement, measurements of

the actual position can be used as a check of the reinforcement. In the same way, the measured crack widths indicate the stress level in the bars, bearing in mind that normally the crack width measured neither corresponds to the maximum load nor the dead load, but lies somewhere in between.

Note that a systematic crack pattern in the surfacing on top of the bridge may be closely related to structural crack in the superstructure (for example bending cracks caused by a negative bending moment above an intermediate support for a continuous bridge).

b) Shear Crack

In beams and slabs subjected to shear (and bending), inclined shear cracks will occur in areas at the supports, refer Figure 3.2-4.

Close to a simple support, the angle between the inclined shear cracks and the beam axis will be approx. 45 degrees (maximum shear, low bending moment). Often, some of the usually fine tension cracks from the bending moment tend to join the shear cracks, as shown in Figure 3.2-4.

In the area between mid-span and support, the bending cracks will be more or less inclined by the shear force depending on the ratio between bending and shear, refer to Figures 3.2-4 and Figure 3.2-5.

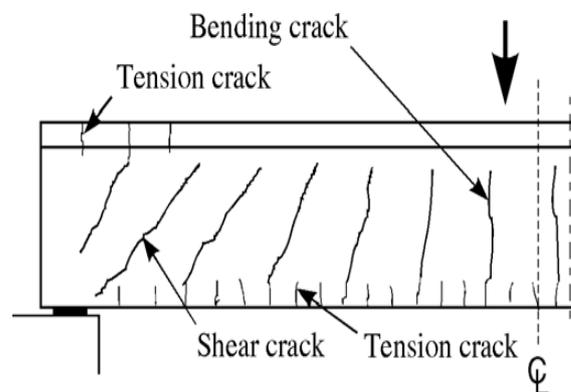


Figure 3.2-4 Shear Cracks

For T-girders, the stringer force in the compression flange will change to a tensile force above the support. This means that possible tension cracks tend to join the shear cracks resulting in more vertical cracks just above simple supports, refer to Figure 3.2-5.

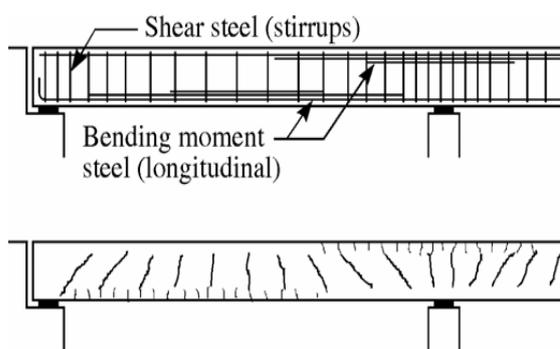


Figure 3.2-5 Shear and Bending Cracks

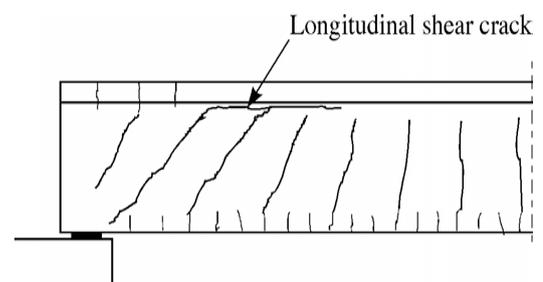


Figure 3.2-6 Longitudinal Shear Crack

The ordinary inclined shear cracks in a girder web may in some cases be connected to horizontal shear cracks, refer to Figure 3.2-6.

Shear cracks cross the total web thickness of a girder. But differently from tension cracks, shear cracks have limited strain development due to the main longitudinal rebars and the compression zone.

This means, that even if the yield stress may be reached in the stirrups when the cracks are formed, a new state of equilibrium is established between stringer forces, stirrups and concrete struts, which leads to lower stresses in the stirrups after cracking.

c) Bearing Cracks

Bearing cracks are defined as cracks, which occur in connection with bearings. Some typical examples of bearing cracks are shown in Figure 3.2-7.

In (a), the elastomeric pad is placed too close to the end of the girder and a kind of a splitting crack is formed between the cover and the bent main bars.

Generally, bent main bars in large dimensions give rise to bearing cracks, because the large bar size requires larger bending diameters. This means, that the edge of the end of the beam is not sufficiently reinforced, especially if horizontal forces due to e.g. temperature may occur too. This case is shown in (b).

A special variant of case (b) is shown in (c). Wrong concreting or un-removed polystyrene causes friction between girder and cap beam, which results in cracks behind the bearing.

In (d), a bearing crack in a cap beam is shown. This type of bearing usually occurs in connection with slab bridges with only asphaltic paper as bearing. The cracking is caused by friction due to horizontal forces (temperature) and the angle of rotation, which tend to move the reaction force to the outer edge.

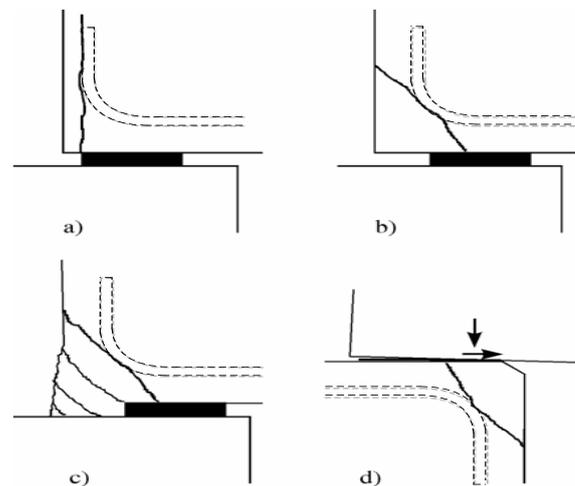


Figure 3.2-7 Bearing Cracks

3.2.1.2 Spalling/Scaling/Disintegration

Spalling is an area where concrete has broken off due to corrosion of reinforcing steel bars. A spall is a fragment, which has been detached from a larger concrete mass.

Vehicular or other impact forces on exposed concrete edges, deck joints or construction joints, may also result in the spalling or breaking off of pieces of concrete locally.

Scaling is the local flaking or loss of the surface portion of concrete or mortar by vehicle's tires.

Disintegration is the physical deterioration or breaking down of the concrete into small fragments or particles. The deterioration usually starts in the form of scaling and if allowed to progress beyond the level of very severe scaling is considered as disintegration.

The spalled area left behind is characterized by sharp edges

Vehicular or other impact forces on exposed concrete edges, deck joints or construction joints, may also result in the spalling or breaking off of pieces of concrete locally.



Photo 3.2-4 Spalling (At Bottom of Girder)



Photo 3.2-5 Disintegration



Photo 3.2-6 Scaling (Deck Slab)



Photo 3.2-7 Spalling (Deck Slab)

3.2.1.3 Delamination

Delamination is defined as a discontinuity in the surface concrete which is substantially separated but not completely detached from concrete below or above it. Visibly, it may appear as a solid surface but can be identified due to hollow sound by tapping. Delamination begins with the corrosion of rebar and subsequent cracking of the concrete. However, in the case of closely spaced bars, the cracking extends in the plane of the rebar parallel to the exterior surface of the concrete.



Photo 3.2-8 Delamination (Girder)

3.2.1.4 Rebar Exposure/Corrosion

Corrosion is the deterioration of rebar by electrolysis. The alkali content in concrete protects the rebar from corrosion but when moisture, air and/or chloride ions above a certain concentration are dissolved in water and penetrate through the concrete to rebar this protection breaks down and corrosion starts. In the initial stage, corrosion may appear as rust stain on the concrete surface. In the advanced stage, the surface concrete above the rebar cracks, delaminates and spalls off exposing heavily rusted rebar. Corrosion will happen more quickly when the concrete is in, or near salt water.



Photo 3.2-9 Rebar Exposure (Girder) Photo 3.2-10 Rebar Exposure (Deck Slab)

3.2.1.5 Honeycomb

Honeycomb is caused by the improper or incomplete vibration of the concrete that results in voids being left in the poured concrete where the cement mortar failed to completely fill the spaces between the coarse aggregate particles.



Photo 3.2-11 Honeycomb (Girder)

Photo 3.2-12 Honeycomb (Deck Slab)

3.2.1.6 Water Leakage

As a result of continuous progress of cracking, cracks penetrate slab thickness, and then water leakage start.



Photo 3.2-13 Water Leakage (Girder)



Photo 3.2-14 Water Leakage (Deck Slab)

3.2.1.7 Deformation/Sagging

Sagging happens due to structural or some initial error during construction. The following photograph shows the sagging of PC Deck Girder after project completion of four (4) spans of same type of girders. Sagging of the girder is suspected to be caused by some initial mistake in construction. The girders are now supported by steel columns.



Photo 3.2.1-15 Abnormal Sagging of PC Girders

3.2.1.8 Impact/Accident Damage

Railing posts and beams are totally shifted out of position by accident damage. The rebars are severely damaged.



Photo 3.2.1 -16 Impact/ Accident Damage

3.2.2 Superstructure (Steel Members)

3.2.2.1 Impact from Vehicles

The most common damage to truss bridges with interior passage is impact to the lateral braces from trucks either because of too low vertical clearance or because the total height of the vehicle exceeds the allowed limit.

The consequences may vary from almost harmless if only secondary braces are damaged to very harmful/collapse if the primary elements are damaged. Even though only the braces are hit, it may introduce problems for the main elements. If for an example the braces are connected to vertical compression members, the impact may create a permanent deformation of the compression members making these elements more or less useless (and introducing a redistribution of the load effects through the structure), refer to Figure 3.2-9.

Stability problems (buckling) in compression members - especially vertical - may also occur due to overload introducing a similar redistribution of the load effects as mentioned above. In both cases, a load capacity evaluation should be carried out to evaluate the risk of a possible collapse. Special attention should be paid to the connections/nodes between the different elements (tension/compression elements and horizontal beams elements), since they are predisposed for corrosion.

A common type of damage to plate girders (including plate girders in steel-composite structures) is impact from vehicles with heights exceeding the actual vertical clearance. Girders may be seriously affected by such an impact, which means that the load effects for some of the remaining girders will be increased. This is not an acceptable situation and remedy should be carried out.

3.2.2.2 Corrosion

Corrosion is the deterioration of steel members by chemical or electro-chemical reaction resulting from exposure to air, moisture, air-borne salts, industrial fumes and other chemical and contaminants in the environment in which it is placed. The term *rust* and corrosion are used interchangeably in this sense. Corrosion, or rusting, will only occur if the steel is not protected or if the protective coating wears or breaks off.

Rust on carbon steel is initially fine-grained, but as rusting progresses it becomes flaky and delaminates exposing a pitted surface. The process thus continues with progressive loss of section.

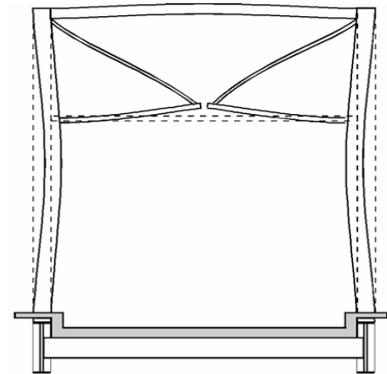


Figure 3.2 -8 Impact Damage to Truss Bridge

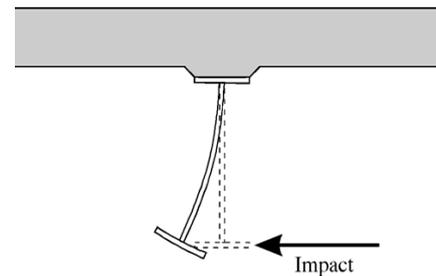


Figure 3.2-9 Impact Damage from Vehicle (Steel Girder)

Weathering steel, on the other hand, will form a relatively smooth rust layer, called a patina, which protects the underlying metal from further corrosion. However, in less than ideal circumstances, the patina may not form or may be penetrated and delaminated, resulting in progressive corrosion.

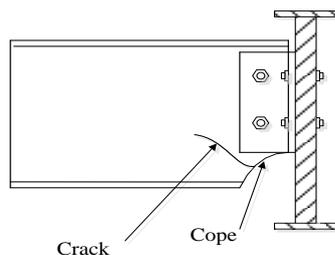
3.2.2.3 Fracture

Crack is a linear fracture in a steel member. Cracks are mainly produced due to defects, and can under certain conditions, lead to brittle fracture.

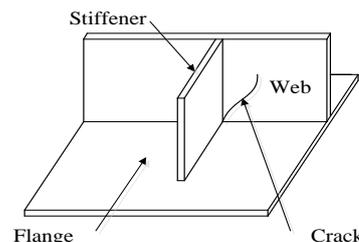
Brittle fracture is a crack completely through the member that usually occurs without prior warning or plastic deformation. Brittle fracture may occur at defect prone details after initial fatigue cracking. Fatigue prone details are those details that are susceptible to growth of fatigue cracks

The primary factors leading to fatigue cracking are: the number of applied stress cycles, which is a function of the volume of traffic; the magnitude of stress range, which depends on the applied live load; and the fatigue strength of the connection detail. Cracks caused by fatigue usually occur at points of tensile stress concentrations, at welded attachments or at termination points of welds. Cracks may also be caused or aggravated by overloading, vehicular collision or loss of section resistance due to corrosion. In addition, stress concentration due to the poor quality of fabricated details and the fracture toughness of materials used are contributing factors. Material fracture toughness will determine the size of crack that can be tolerated before fracture occurs.

Fracture critical components are components which are subject to tensile stresses in a single load path structure and whose failure could lead to collapse of structure. Any attachment having a length in the direction of tension stress greater than 100 mm and that is welded to the tension area of a fracture critical component shall also be considered as fracture critical.



Crack of Web near Gusset Connection



Crack of Web at Connection with Stiffener

Figure 3.2-10 Common Crack Locations in Steel Members

Welded details are more prone to cracking than bolted or riveted details. Grinding off the weld reinforcement to be smooth or flush with the joined metal surfaces improves fatigue resistance. Once cracking occurs in a welded connection, it can extend into other components due to a continuous path provided at the welded connection, and possibly lead to brittle fracture.

Many plate girders are welded. It is normal to find stress peaks at welds. The stress level

depends of the type of weld and the quality of the welding. If the stress range from the live load is considerable at a poor welding, there may be a risk of fatigue cracks. Therefore welded details, which may be critical in fatigue should be inspected carefully. This may for example be at a welding in the bottom flange of a girder, as shown in Figure 3.2-11 or at a welded connection between the horizontal wind truss and the bottom of the girders. Such details may at the same time also be critical to corrosion.

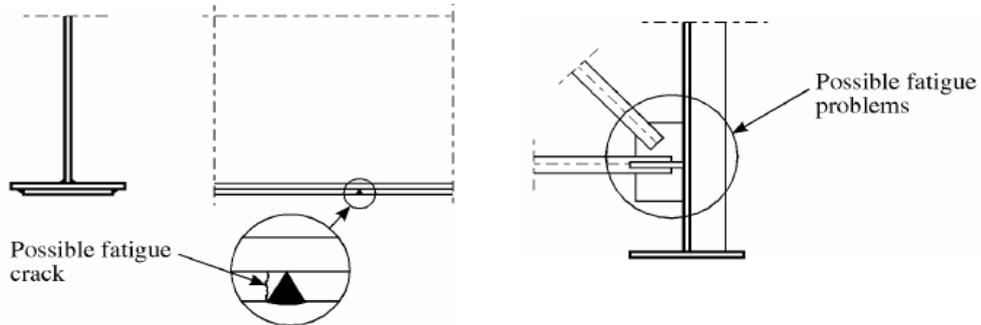


Figure 3.2-11 Fatigue Problems in Welded Steel Members

Bolted or riveted connections may also develop fatigue cracking, but crack in one component will generally not pass through into others. Bolted and riveted connections are also susceptible to cracking or tearing resulting from prying action, and by build-up of corrosion forces between the parts of the connection.

As cracks may be concealed by rust, dirt or debris, suspect surfaces should be cleaned prior to inspection.

Cracks that are perpendicular to the direction of stress are very serious, with those parallel to the direction of stress less so. In either case, cracks in steel should generally be considered serious, as parallel crack may for a number of reasons turn into a perpendicular crack. Any crack should be carefully noted and recorded as to its specific location in the member, and member structure. The length, width (if possible) and direction of crack should also be recorded.

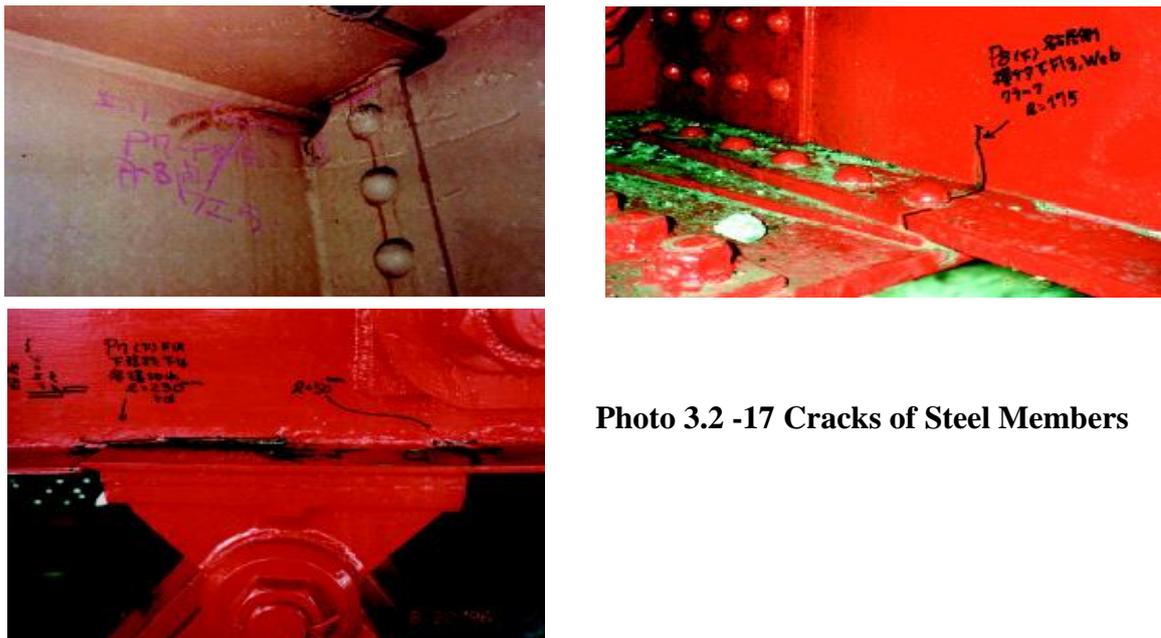


Photo 3.2 -17 Cracks of Steel Members

3.2.2.4 Loose Connections

Loose connections can occur in bolted or riveted connections. It may be caused by corrosion of the connector plates or fasteners, excessive vibration, over stressing, cracking, or failure of individual fasteners.

Loose connections may sometimes not be detectable by visual inspection. Cracking or excessive corrosion of the connector plates or fasteners, or permanent deformation of connections or members framing into it, may be indications of a loose connection. Tapping the connection with a hammer is one method of determining if the connection is loose or not.

Loosening or movement of the joint is the most common form of failure and may be accompanied by rattling or breaking of bolts, joint components and seating. The onset of some form of loosening can usually be detected by a crack developing between the joint and the adjoining surface and by increased noise under traffic. Eventually a series of cracks will develop in the surfacing itself. Adhesion or anchorage of the joint to the seating can usually be checked by the ringing sound when joint is tapped with a hammer. Loose parts of the joint thrown up by the passage of wheel can present extreme danger to traffic. The tightness of anchor bolts and nuts in bearings should be checked to ensure freedom from excessive vibration and proper restraint.

3.2.2.5 Permanent Deformations

Permanent deformations of steel members can take the form of bending, buckling, twisting or elongation, or any combination of these. Permanent deformations may be caused by overloading, vehicular collision, or inadequate or damaged intermediate lateral supports or bracing.

Permanent bending deformations normally occur in a direction perpendicular to the applied load and are usually associated with flexural members; however, vehicular impact may produce permanent deformations in bending in any other members.

Permanent buckling deformations normally occurs in a direction perpendicular to the applied load are usually associated with compression members. Buckling may also produce local permanent deformations of web and flanges of beams, plate girders or box girders.

3.2.2.6 Abnormal Vibration/Deflection

Abnormal vibration occurs on a bridge due to overloading (live loads), under-design (at design stage), under-sizing (at construction stage), defective or inadequate bearing and local and general scour due to undermining of foundation. Failure to arrest this problem could lead to structural problem which is catastrophic in nature. Excessive vibration may not be structurally damaging, unless the vibrations are setting up resonance, and hence, causing instability to the structure.

They can, however, be general indicators of distress within the structure, particularly if vibration and noise have become more evident since previous inspection. Vibration can be detected by standing on the mid span of the superstructure and/or on the road in the vicinity of the substructure when a vehicle passes.

Excessive deflection occurs on a bridge due to overloading (live loads), under-design (at design stage) and under-sizing (at construction stage). It is a difficult task to quantify and assess the excessive deflections with accurate precision. The presence of excessive deflections can be checked by standing on the mid span of the structure. The deformation of the structure or any part of it should not adversely affect its efficiency or appearance. Excessive deflection is one of the best visual indicators of the state of a structure. Small deflections due to heavy traffic loading can usually be detected visually, but longer term deformations, due to the structure's self-weight or repeated overloads, will require detection by leveling instruments. Most structural materials undergo substantial deflections under overload or conditions of distress before failure takes place.

3.2.3 Substructure (Concrete Piers, Abutments and Foundation)

3.2.3.1 Cracking

Cracking in substructures are caused by effect of salt water. Rebars inside were corroded and expanded to make cracks.



Photo 3.2-18 Cracking in Pier



Photo 3.2-19 Cracking in Abutment

3.2.3.2 Spalling

(See Section 3.2.1.2)

This spalling is caused by impact force on exposed concrete surface.



Photo 3.2-20 Spalling in Pier

3.2.3.3 Scaling

(See Section 3.2.1.2)

This scaling is caused by impact force during floods. This pier has rectangular section which is not good considering the river flow.



Photo 3.2-21 Scaling in Pier

3.2.3.4 Disintegration

(See Section 3.2.1.2)

3.2.3.5 Delamination

(See Section 3.2.1.3)

Delamination is caused by rebar corrosion



Photo 3.2-22 Delamination in Pier

3.2.3.6 Rebar Exposure/Corrosion

(See Section 3.2.1.4)

Rebar corrosion due to exposure to sea water caused delamination and spalling.



Photo 3.2-23 Rebar Exposure in Pier

3.2.3.7 Tilt/Settlement

Tilting normally occurs due to uneven settlement of foundation, displacement of pier (inclination) due to traffic impact or slip circle failure.

All foundations undergo slight movements, which if they remain small and uniform, cause no distress to the structure. Movements of large magnitudes, especially when they are differential movements, cause distress to nearly all structures, unless specific provisions have been made during the design stage. Such movements result from the unpredicted settlement or failure of foundation material, which is sometimes associated with mining subsidence, the developments of cavities in rocks, scour, and changes in water table. The onset of foundation movement may be difficult to detect, unless periodic leveling surveys have been carried out.

Usually, the first indication of trouble is a visual one caused by a change in the geometry of the structure, such as excessive or unusual movements at bearings and at expansion joints, cracking of abutments, wing walls and end of beams, or tilting of piers, abutments or wing walls.

Spread foundation settlement due to scouring.



Photo 3.2-24 Tilt/Settlement of Pier

Pile settlement due to impact force.



Photo 3.2-25 Tilt/Settlement of Pile

Abutment settlement due to earth pressure.



Photo 3.2-26 Tilt/Settlement of Abutment

3.2.3.8 Scouring

Scour is caused by the erosive action of running water, excavating and carrying away material from the bed and banks of waterways. Scour is one of the most frequent causes of bridge failures, mainly because it may develop to a very large extent within a short time.

If the level of the riverbed has changed significantly — in general or around piers/abutments — there is always reason to carry out more detailed investigations. Note that there may very well be problems, even if the erosion has not reached the level of the underside of the foundation. In many cases the carrying capacity of a direct foundation is dependent on the pressure (the weight) from the surrounding soil. And particularly, pile foundation depends on the surrounding soil.



Photo 3.2-27 Scouring (Concrete Pile at Right Side is Broken)



Photo 3.2-28 Scouring (Pier Foundation)

3.3 Identification of Deterioration

3.3.1 Prediction of Deterioration Mechanism

3.3.1.1 Principles of Deterioration Mechanism

Deterioration mechanism shall be identified using an appropriate procedure. When detecting defects during the condition inspection, it should be determined whether the defects are due to deterioration mechanism or not. The deterioration indicator evaluates progress and stage of deterioration, and indicators could be set as the measurement item at the Engineering Inspection. In the Engineering Inspection it is necessary to understand the cause and the stage of deterioration. It is possible to predict the deterioration mechanisms based on the relationship between the results of NDT and evaluation as given in Engineering Inspection.

In order to carry out appropriate maintenance of a bridge, it is vital to evaluate how performance of the bridge, its members, and the parts of the structure would vary under the environmental conditions to which they are exposed.

Table 3.3-1 Mechanism Related to Deterioration Factors, Indicators, and Characteristics of Defects

Mechanism	Major Factors	Characteristics of Defects
Carbonation	Carbon dioxide	Cracking in the direction of rebar axis, Delamination of concrete
Chloride Induced Deterioration	Chloride ions	Cracking in the direction of rebar axis, Rust exudation
Alkali-Silica Reaction	Reactive aggregate	Expansive cracking in restraining directions, Distributed cracking, White gel, discoloration
Fatigue	Traffic of large vehicles (weights in excess of the design load)	Lattice (Raft) cracking, Corner disintegration,
Corrosion	Oxygen and water	Section loss, Pitting corrosion

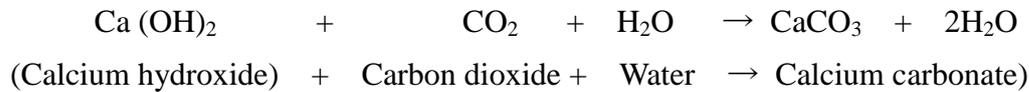
Deterioration mechanism should be identified by the environmental and usage conditions and characteristics of defects. Table 3.3-1 shows the relationship among deterioration factors, deterioration phenomenon, and, deterioration indicators. Deterioration mechanisms can be predicted in consideration with reference to section 3.4, by comparing with and examining each characteristic of deterioration mechanisms shown in the Table 3.3-1 and defects of the bridge.

3.3.1.2 Carbonation

Carbon dioxide in the atmosphere can be dissolved in moisture which enter into concrete pores, and react with calcium hydroxide in the cement paste to form a neutral calcium carbonate. Over a long period of time, the concrete gradually lowers the alkalinity in to the rebar, thus reducing the passive oxide layer around the steel and placing it in a more

acidic environment whereby it is susceptible to corrosion. Sound concrete is extremely alkaline with a value of pH 12 to 13.

Carbon dioxide (CO₂) in the air penetrates into concrete thru its pores from the surface, reacting with calcium hydroxide to form calcium carbonate.



The reaction shown above causes the pH in concrete to decrease. When the pH value goes below 11, the passive-state films in the reinforcing bars are destroyed, causing the bars to be oxidized or rusted. (As reinforcing bars rust, they expand 2.5 times.)



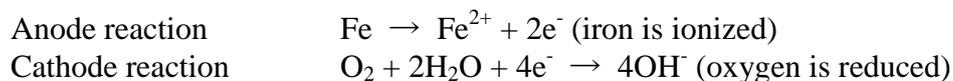
Photo 3.3-1 Carbonation

The depth of carbonation can be identified thru a bored sample from a concrete core, by spraying a phenolphthalein solution onto the surface of the cut portion and measuring the color development depth from the surface. (The color is red when concrete is high in alkaline and no color when the concrete is carbonated.)

If the water-cement ratio is high or if the covering depth is shallow, carbonation reaches the surface of the reinforcing bar in a shorter period. So, the water-cement ratio must be decreased (less than 60% at maximum) and a proper covering depth must be secured. So that corrosion of reinforcing bars is a less problem. (Unreinforced concrete is still subject to deterioration.)

3.3.1.3 Chloride Induced Deterioration

Chloride ions penetrate into concrete up to the surface of the reinforcing bars and destroying the passive-state film. When this destroyed film is subjected to oxygen and water, the reinforcing bars start to rust or begin to corrode.



For chloride ions that penetrate into concrete, there are specifically two types of salinity: an internal salinity contained in sea sand, mixing water, etc., used during concrete production and an external salinity, such as seawater, seawater splash, blown-in salinity, spray from anti-freezing agents, etc., after concrete solidification.

Counter Measures: Limit salt content, use rustproof reinforcing bars, allow sufficient covering depth for reinforcing bars, make densely packed concrete, surface linings, etc.

Concrete structures are usually repaired by first chipping the surface of the concrete to remove rust and salinity from the reinforcing bars then coating the reinforcing bars with a rust inhibitor and using a repair agent. Other repair methods described below are now introduced experimentally:

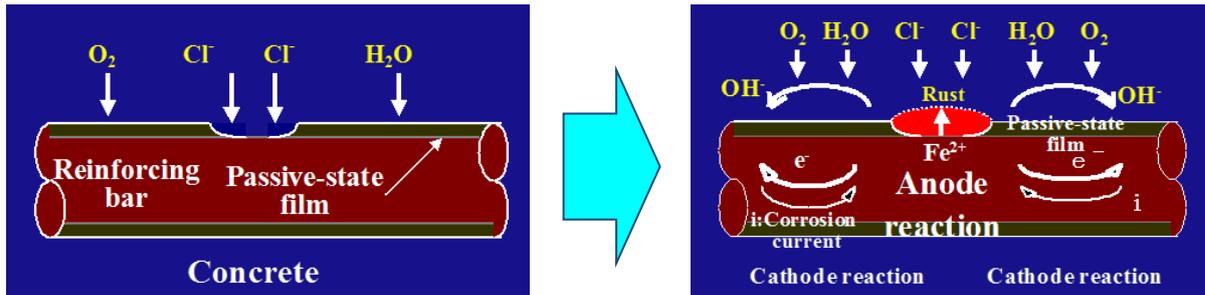


Figure 3.3-1 Anode Reaction



Photo 3.3-2 Chloride-Induced Deterioration

3.3.1.4 Alkali-Silica Reaction (ASR)

Some aggregates react adversely with the alkalis in cement to produce a highly expansive alkali-silica gel. The expansion of the gel under moist conditions leads to cracking and deterioration. The cracking occurs through the entire mass of the concrete. Alkali-silica reactions are generally slow by nature, and the results may not be apparent for 5-10 years.

Alkali-silica reaction: The reaction and expansion process do not proceed simultaneously. After the reaction period (7 to 10 days), the expansion process begins to develop.



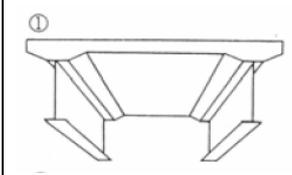
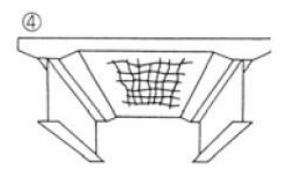
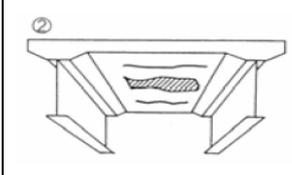
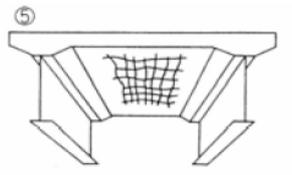
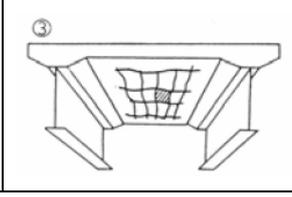
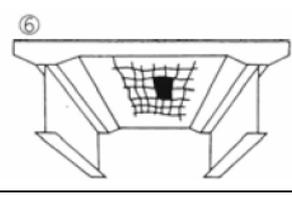
Photo 3.3-3 Alkali-Silica Reaction

The use of reactive aggregate or the presence of water and with extremely alkali substances encourages or develop cracking and deteriorates the concrete. Most cracks form a grid or hexagonal patterns. Cracks tend to develop along the laid reinforcing bars.

3.3.1.5 Fatigue

Drying shrinkage, incomplete execution of work, repeated application of loads, etc., cause cracks to pass-through to RC deck slabs, with the cracks developing like a grid. As these cracks develop, the concrete falls off and the other types of damage occur. Repeated application of loads and the effect of water cause the carrying capacity against shearing to decrease (fatigue strength can be one twentieth as small if the concrete is free from the effects of water). Development of cracking of RC deck slab is as follows:

Table 3.3-2 Development of Cracking in RC Deck Slab

			
① Good Condition		④ Alligator Cracks	
			
② Transverse Cracks		⑤ Increasing Number of Cracks/Water Penetration	
			
③ Orthogonal Cracks		⑥ Disintegration/Spalling	

3.3.1.6 Corrosion

Corrosion is the gradual deterioration of steel by chemical or electrochemical reaction with the environment. Mechanism of corrosion is shown Figure 3.3-2. There are two corrosion reactions that occur simultaneously; anode reaction and cathode reaction. Corrosion reaction proceeds with equal amount of reaction with each other. It means that, if the anode reaction is controlled, the cathode reaction is also controlled. Anode reaction occurs due to contact between steel and water. Also, the cathode reaction requires the presence of oxygen and water. Therefore, the basic countermeasure against corrosion is to cut off the supply of oxygen or water.

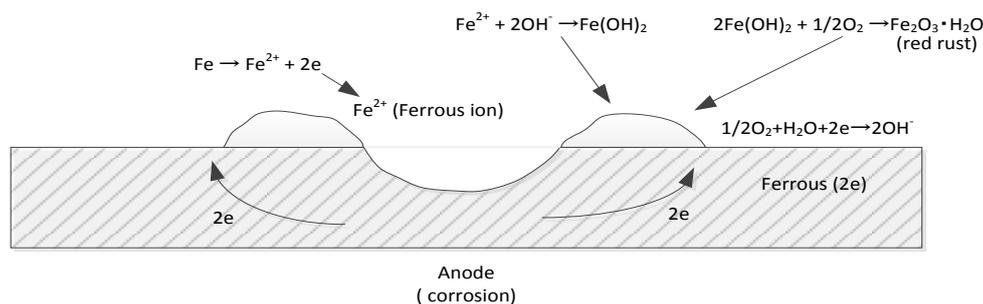


Figure 3.3-2 Mechanism of Corrosion in Steel

3.3.2 NDT for Identification of Deterioration Mechanism

If the defects are suspected to be caused by deterioration mechanism, the following NDT shall be conducted for identification of deterioration as follows:

Table 3.3-3 NDT Methods for Identifying Deterioration Mechanisms

Purpose of NDT	Deterioration Mechanism	Carbonation	Chloride-Induced Deterioration	Alkali Silica Reaction	Fatigue	Corrosion
	NDT Methods					
Carbonation Depth	N02 Carbonation Depth Measurement Test	**	**			
Rebar Corrosion	N04 Half-Cell Electrical Potential Test	**	**			
Concrete Strength	N01 Rebound Hammer Test	*	*	*	*	
	N07 Compressive Strength Test	*	*	*		
Rebar Detection	N05-01 Rebar Detection Test (Magnetic Type)	**	**		*	
	N05-02 Rebar Detection Test (Radar Type)	**	**		*	
Crack Depth	N03 Ultrasonic Pulse Velocity Test	*	*	*	*	
Corrosion of Metals	N08 Paint Thickness Test					*
	N09 Metal Thickness Test					**
Load Carrying Capacity	S02 Static Load Carrying Test	*	*	*	*	
	S03 Dynamic Load Carrying Test	*	*	*	*	

** : Required Test * : Test to be conducted depending on the recommendation of Advisory Team

3.4 Prediction of Deterioration

3.4.1 General

3.4.1.1 Prediction of Deterioration

Prediction of Deterioration mechanism based on condition/engineering inspection aims to determine the progress of subsequent deterioration and to incorporate the results in the evaluation and analysis of performance over time, planning of repair and strengthening, and the appropriate time for implementing such measures.

3.4.1.2 Methods and Accuracy of Prediction of Deterioration

Engineering Inspection provides detailed data on concrete qualities, reinforcement corrosion, geometry of the bridge and its members, reinforcing bar arrangement, and so

on. Practical and accurate process of deterioration should therefore be carried out using these data.

The five (5) main deterioration mechanisms are: Carbonation, Chloride-Induced Deterioration, Alkali-Silica Reaction, Fatigue and Corrosion.

Carbonation and Chloride-Induced Deterioration mechanisms cause rebar corrosion. For these mechanisms, there are relatively accurate process models up to the Initial stage, i.e., before the onset of rebar corrosion, but the process up to this stage generally relates to the initial inspection. At the time of Engineering Inspection, deterioration generally proceeds to the propagation and acceleration stages, in which the process of rebar corrosion is of importance. Though the process models for steel corrosion are generally less accurate than those for the Initial stage, it is advisable to improve the accuracy of the process by applying the inspection results of rebar corrosion.

Alkali-Silica Reaction proceeds to a certain extent by the time when cracks are recognized. Determining whether the reaction has finished or continues to proceed is of importance under this deterioration mechanism. In general, the test is conducted on concrete samples for such evaluation. However, if the reaction is found to continue, it is difficult to predict the subsequent rate of reaction.

With regards to Fatigue, subsequent progress of deterioration can be qualitatively determined based on the relationship between the time and the state of cracks, as the state of RC slabs crack shows changes from time to time.

Corrosion of steel is the most common factor leading to the deterioration of structural members and joints of a steel bridge. Corrosion process is related to the deterioration of paint coating. Therefore, paint thickness should be measured from time to time. Also, the location and extent of corrosion need to be recorded. This information can be used for analysis of the member and for taking preventive measures to minimize further deterioration

3.4.2 Carbonation

3.4.2.1 General

Carbonation is the phenomenon which lowers pH value of the pore solution because carbon dioxide which penetrates into concrete from the atmosphere and cement hydrate such as calcium hydroxide generates the carbonation reaction. According to this, the passivation film on the rebar surface is lost, and the corrosion progresses with the supply of water and oxygen. With the corrosion of the rebar, the generation of the crack and spalling of concrete cover and lowering of the load carrying capacity, etc. are generated.

The deterioration process of concrete bridges due to carbonation progress and corrosion of rebar is divided into four stages: “Initial Stage”, “Propagation Stage”, “Acceleration Stage” and “Deterioration Stage”. Since the effects of the deterioration phenomenon on the performance of the structure differ from one phase to another, each phase requires different deterioration process, evaluation and repair methods. For this reason, specific requirements for maintenance are hereafter described for each deterioration phase.

3.4.2.2 Deterioration Stage

The main effects of deterioration by carbonation is the corrosion of rebar, and there is no example of the serious deterioration of the bridges caused only by strength reduction or shrinkage due to carbonation. Therefore, carbonation induced deterioration of bridges is divided into four phases: the initial stage up to the time when carbonation reaches the threshold depth for beginning of corrosion ; the propagation stage from start of corrosion to beginning of corrosion induced cracking; the acceleration stage in which the rate of corrosion is increased by the presence of cracking; and the deterioration stage in which the progress of reinforcing bar corrosion causes significant losses in the load-carrying capacity and other qualities. Since each phase basically corresponds to a state (performance) of the structure, process of the progress of deterioration basically comprises process of the duration of each phase (Table 3.4-1).

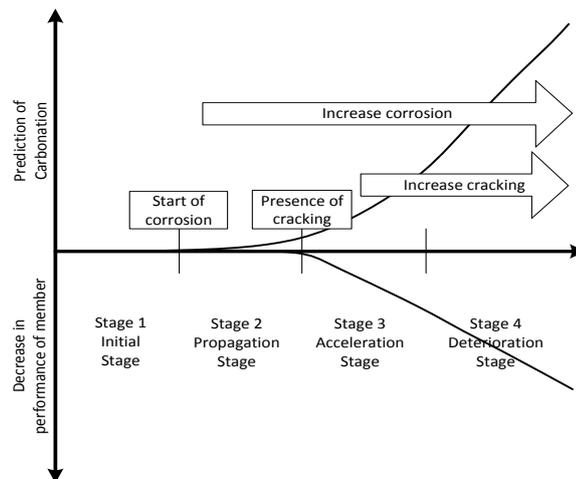


Figure 3.4-1 Process of Advancing Deterioration Due to Carbonation

Therefore, the length of the initial stage will be calculated from the carbonation speed, the length of propagation stage will be calculated from corrosion rate of rebar and corrosion quantity in the corrosion induced cracking, the length of acceleration stage will be calculated from the corrosion rate of rebar after the corrosion induced cracking, and the length of deterioration stage will be calculated from the corrosion rate of rebar after the corrosion induced cracking and corrosion quantity in which the load carrying capacity lowers.

Therefore, the length of the initial stage will be calculated from the carbonation speed, the length of propagation stage will be calculated from corrosion rate of rebar and corrosion quantity in the corrosion induced cracking, the length of acceleration stage will be calculated from the corrosion rate of rebar after the corrosion induced cracking, and the length of deterioration stage will be calculated from the corrosion rate of rebar after the corrosion induced cracking and corrosion quantity in which the load carrying capacity lowers.

From the above, the process object at the maintenance is the progress of the carbonation and progress of the rebar corrosion. The change (lowering) of various performance of the structure should become an object of the process. However, it is difficult to directly predict the performance deterioration based on the current level of technology. Therefore, the deterioration of concrete and rebar in the bridge is estimated, and various performance levels of the structure are evaluated by combining these results.

Table 3.4-1 Definitions of Deterioration Stages Due to Carbonation

Deterioration Stage	Definition
Stage I (Initial Stage)	Period when carbonation depth reaches the boundary depth for start of corrosion
Stage II (Propagation Stage)	Corrosion starting to induce cracking
Stage III-1, III-2 (Acceleration Stage)	Rate of corrosion is increased by the presence of cracking
Stage IV (Deterioration Stage)	Progress of corrosion causes significant losses in the load-carrying capacity

3.4.2.3 Process of Carbonation Progress

Process of carbonation progress should be made by using “Square Root Relationship between Time and Carbonation Depth”. Refer to Sub-Section 2.3.2.4, page 2-17 for details.

It is advisable to determine the carbonation rate coefficient from the carbonation depth measurements, utilizing the fact that the carbonation depth is proportional to the square root of the carbonation period. Where no inspection results are available, process should be made by using reliable carbonation rate equations, while adequately considering the materials and mixture proportions of concrete and the environmental conditions under which the structure is in service.

$$y = b\sqrt{t} \quad , \quad b = \frac{y}{\sqrt{t}}$$

where:

y: carbonation depth (mm)

b: carbonation rate coefficient (mm/ $\sqrt{\text{year}}$)

t: carbonation period (year)

$$D = C - y$$

where:

D: un-carbonated depth (mm)

C: cover (mm)

3.4.2.4 Process of Rebar Corrosion Progress

The starting time of corrosion of rebar is basically dependent on the depth of the un-carbonated concrete cover.

In general, the corrosion of rebar induced by carbonation begins before carbonation depth reaches the rebar. Also, there are many cases in which the starting time of the corrosion has been influenced by un-carbonated concrete cover depth, which is defined as a difference between carbonation depth and cover depth. It has been found that the corrosive case rapidly increases, when the un-carbonated depth becomes 10mm or less. There are only few examples of corrosion studies which show that there is a decrease in the function of the structure when the un-carbonated depth is smaller than 10mm. The beginning of carbonation induced corrosion may be assumed to be at the point when the un-carbonated depth is 10mm. The start of the corrosion is accelerated, when the chloride ions are included in concrete because chloride ions fixed in cement hydrate disassociate by the carbonation's progress and concentrate in the uncarbonated region. In the case of concrete containing chlorides such as those within coastal areas, the start of corrosion should be at the point when the un-carbonated depth is 25mm. Coastal areas as defined in this manual, are areas located within one (1) kilometer from the shoreline.

3.4.3 Chloride-Induced Deterioration

3.4.3.1 General

Chloride-induced deterioration is defined as the performance of deterioration phenomenon due to concrete cracking or spalling occurred by expansion of corrosion products and due to losses in steel cross-sectional area, as a result of corrosion of rebar accelerated by the existence of chloride ions.

The deterioration process of a concrete bridge due to chloride-induced corrosion of rebar is divided into four stages: “Initial Stage”, “Propagation Stage”, “Acceleration Stage” and “Deterioration Stage”. Since the effects of the deterioration phenomenon on the performance of the bridge differ from one stage to another, each stage requires different deterioration process, evaluation and repair methods.

3.4.3.2 Deterioration Stage

Chloride-induced performance deterioration of bridges are divided into four stages: the initial stage up to the beginning of rebar corrosion, the propagation stage from corrosion beginning to beginning of corrosion induced cracking, the acceleration stage in which the rate of corrosion is significantly increased by corrosion induced cracking, and the deterioration stage, in which substantial losses in the cross-sectional area of reinforcing steel cause significant reduction in the load-carrying capacity and other qualities as shown Figure 3.4-2

The factors determining each stage and its duration are presented in Table 3.4-2

The length of initial stage is calculated by diffusion of chloride ions and content of initially included chloride ions. The length of propagation stage is calculated by corrosion rate of rebar and corrosion amount of steel at the beginning of concrete cracks. The length of acceleration stage is calculated by corrosion rate of rebar with corrosion induced cracks and the length of deterioration stage is calculated by corrosion rate of rebar with corrosion induced cracks and corrosion amount at the beginning of reduction in load bearing capacity, respectively.

However, it is difficult to predict the performance deterioration directly with the bridge technical at present, various performances of bridges are evaluated by the combination of predicted concrete durability and corrosion of rebar.

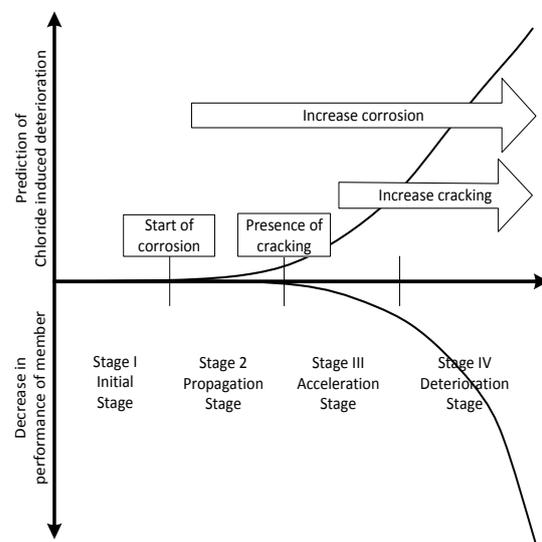


Figure 3.4-2 Process of Advancing Deterioration due to Chloride-Induced Deterioration

Table 3.4-2 Definitions of Deterioration Stages Due to Chloride-Induced Deterioration

Deterioration Stage	Definition
Stage I (Initial Stage)	Up to the time when the chloride ions content at the cover depth reaches the threshold value for corrosion of rebar.
Stage II (Propagation Stage)	From the beginning of corrosion of rebar to the onset of corrosion induced cracking.
Stage III-1,-2 (Acceleration Stage)	Period during which corrosion is accelerated by corrosion induced cracking
Stage IV (Deterioration Stage)	Period in which the load-carrying capacity is significantly reduced by the increase in corrosion

3.4.3.3 Process of Rebar Corrosion Progress

The beginning of corrosion of rebar is basically based on the depth of uncarbonated cover.

The start of the corrosion is advanced when the chloride ions are included in concrete because chloride ions fixed in cement hydrate dissociate by the carbonation's progress and concentrate in the uncarbonated region. In the case of concrete containing chlorides, it should be at the point when the uncarbonated depth is 25mm.

3.4.4 Alkali-Silica Reaction (ASR)

3.4.4.1 General

Alkali (Na_2SO_4 and K_2SO_4) contained in cement is dissolved into solution during the process of cement hydration, and makes the solution with strong alkalinity level (pH 13-13.5), by forming sodium and potassium hydroxides (NaOH and KOH). Extraordinary expansion and attendant cracks can be activated when aggregates containing certain siliceous minerals and carbonate rocks react with high alkaline solution in concrete. This reaction is called alkali-aggregate reaction.

At present, it is difficult to clearly divide the process of performance deterioration of bridges due to ASR into phases and predict the duration of each phase. However, based on the fact that the degree of performance deterioration changes with progresses of deterioration, the progress of deterioration is divided into four stages: initial stage, propagation stage, acceleration stage and deterioration stage. Since the effects of the deterioration phenomenon on the performance of the bridges differ from one stage to another, each stage requires different deterioration process, evaluation and repair methods.

3.4.4.2 Deterioration Stage

Process of the progress of ASR induced deterioration of bridges should be performed using the expansion of concrete and subsequent crack propagation as indicators.

Process of deterioration should be based on the results of condition inspection/engineering inspection. The expansion progress is classified by four expansion stages; each stage is described as follows:

Expansion Stage I (Initial stage): ASR progresses but expansion is not yet apparent. In this stage, an expansion potential is maximum and a residual expansion is maximum. Expansion Stage II (Propagation stage): ASR expansion significantly appears and expansion rate is at maximum. The residual expansion is still large.

Expansion Stage III (Acceleration stage): ASR itself converges and the expansion rate is reduced. Thereby, the remaining expansion is small.

Expansion Stage IV (Deterioration stage): ASR expansion settles down and residual expansion is almost zero.

The stages of the ASR-affected deterioration progress of bridge are defined as follows:

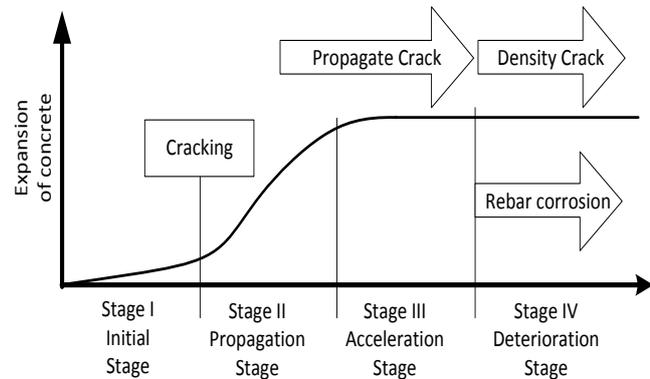


Figure 3.4-3 Process of Advancing Deterioration due to Alkali -Silica Reaction

Table 3.4-3 Definitions of Deterioration Stages Due to Alkali-Silica Reaction

Deterioration Stage	Definition
Stage I (Initial stage)	ASR progresses but expansion is not yet apparent.
Stage II (Propagation stage)	Expansion continuously progresses with steady supply of water and alkali, causing cracking.
Stage III (Acceleration stage)	Expansion due to ASR becomes significant with the almost maximum expansion rate, while cracking propagates.
Stage IV (Deterioration stage)	Crack density increases and steel corrosion progresses. Load-carrying capacities of members are affected by the loss of concrete strength and damage of reinforcing bar.

3.4.4.3 Process of ASR Progress

Process of the ASR progress shall be made with adequate consideration to the alkali-silica reactivity of aggregate, mixture proportions of concrete and effects of environmental conditions to which the structure is exposed.

The process condition of ASR is determined mainly by reactivity of aggregate (type and content of reactive mineral), concrete mixture (alkali content of cement and cement content of concrete) and environmental condition in the bridge (supply condition of water and alkali).

Prediction of ASR induced deterioration should be made from the results of inspection of the bridges.

If the aggregate used is reactive, then it can expand after a long period of service, even if no sign of ASR is observed during inspection. Moreover, it is difficult to accurately estimate the expansion of concrete in actual structures from the inspection results.

3.4.5 Fatigue

3.4.5.1 General

Based on case examples of serious damage in bridge deck slabs due to fatigue, the process of deterioration of deck slab is divided into four stages: “Initial stage”, “Propagation stage”, “Acceleration stage” and “Deterioration stage”. Since the effects of the deterioration phenomenon on the performance of the bridge differ from one stage to another, each stage requires different deterioration process, evaluation and repair methods.

3.4.5.2 Deterioration Stage

Process of fatigue is basically comprised of the following: process of the duration of “Initial Stage”, “Propagation Stage”, “Acceleration Stage” and “Deterioration Stage”, by utilizing the inspection results, such as changes in the condition of bottom surface of the slab.

Process of fatigue is basically required to be carried out qualitatively based on the conditions of the slab bottoms observed during inspection in consideration of the factors affecting the progress of defect. The changes in the state of slab bottom cracking due to defect are illustrated in Table 3.4-4. Moreover, performance deterioration due to defect is divided into four stages:

Stage I (Initial stage) in which one-directional flexural fatigue cracks develop on the bottom face of slabs

Stage II (Propagation stage) in which other cracks perpendicular to flexural cracks appear. Cracking occurs in two directions (longitudinal and transverse).

Stage III (Acceleration stage) in which crack width is increasing

Stage IV (Deterioration stage) in which water penetrates cracks and noticeable water leakages are observed which accelerates progress of reducing of the load-carrying capacity.

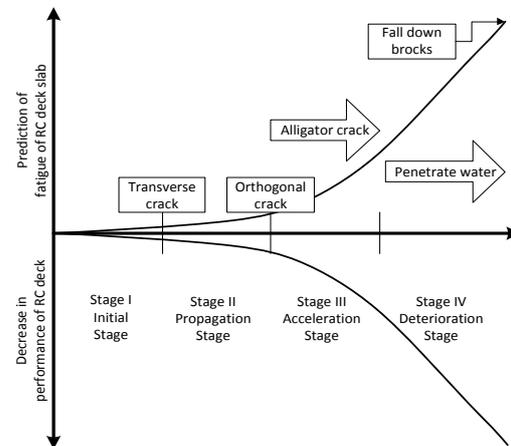


Figure 3.4-4 Process of Advancing Deterioration Due to Fatigue

Table 3.4-4 Change in the State of RC Deck Slab Bottom Cracking Due to Fatigue

Stage I Cracks in one direction	Stage II Cracks in two-direction	Stage III Cracks look like a mesh	Stage IV Water penetrates cracks; spalling

Fatigue can be relatively easily assessed and determined by observation of the state of cracking on the slab bottom. However, this does not express the period quantitatively, as crack density can be saturated. In such a case, the width and depth of cracks and crack behavior such as the opening-closing movement of working cracks and crack faulting should also be observed in addition to apparent defective events, as well as water leakage, free lime and condition of the road surfaces.

On the other hand, the rate of progress of deck slab defect depends on the design and construction conditions, such as the slab thickness, concrete strength and transverse reinforcement content, usage conditions, such as the traffic, passing over weight vehicles and wheel path positions and environmental conditions, such as rainwater falling on top of the RC deck slab.

Table 3.4-5 Definitions of Deterioration Stages of Fatigue

Deterioration Stage	Definition	Major Factors in Determining Duration
Stage I (Initial Stage)	A few cracks can be recognized in one direction along longitudinal bars due to drying shrinkage or loading. Depending on the restraining conditions of the main beams, transverse cracks also develop due to drying shrinkage and temperature changes in the main beam.	Applied design standard Slab thickness Transverse reinforcement content Slab span Execution Drying shrinkage
Stage II (Propagation Stage)	As the bending cracks in the axial directions develop, transverse cracks also begin to develop, forming map cracking. Despite the increase in the apparent crack density, the continuity of the reinforced concrete slab is retained.	Service conditions Traffic, Tonnage of vehicles Wheel paths
Stage III (Acceleration Stage)	Cracks are interconnected into fine networks, and they begin to open and close with rubbing motions. When cracks ravel, the resistance of the integrated concrete cross section is no longer expected. The load-carrying capacity of the reinforced concrete slab abruptly decreases.	In addition to the above, Environmental conditions Effect of water seepage Repair methods taken Slab waterproofing Repair/strengthening
Stage IV (Deterioration Stage)	The continuity of the slab is lost when penetrating cracks occur in the slab cross section. The slab begins to resist the wheel loads as a group of beams divided by penetrating cracks. The ultimate load-carrying capacity of the members is affected by the intervals of penetrating cracks, concrete strength and reinforcement quantity.	All of the above

3.4.6 Corrosion

3.4.6.1 General

Corrosion is caused by the contact of oxygen and water to the surface of the steel material. Paint is applied to prevent the ingress of oxygen and water. Deterioration of paint is one of the factors that cause rust, but it does not directly cause of the performance deterioration of the steel members. On the other hand, corrosion induces section loss gradually.

Therefore, the prediction of deterioration of corrosion for steel members should be performed.

3.4.6.2 Deterioration Stage

Corrosion performance deterioration of steel bridges is divided into four stages; the initial stage is up to the beginning of corrosion, this stage includes the deterioration of the coating. After deterioration of the coating film, corrosion starts to occur. The propagation stage starts from beginning of corrosion to beginning of section loss. This stage is when paint coating is lost completely. However, there is no section loss and decrease in performance of member has not yet occurred. The acceleration stage is the state in which the section loss is progressing, if the section loss ratio becomes 20% or more, it shifts to the deterioration stage.

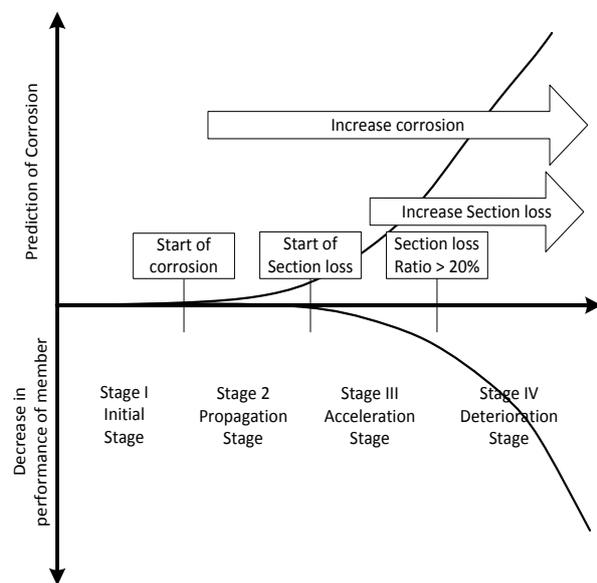


Figure 3.4-5 Process of Advancing Deterioration due to Corrosion

Table 3.4-6 Definitions of Deterioration Due to Corrosion

Stage of Appearance	State of Deterioration
Stage I (Initial Stage)	Deterioration of the coating. No corrosion
Stage II (Propagation Stage)	Point rust or rust occurs. Rust fluid and crevice of coated film are observed.
Stage III (Acceleration Stage)	Rust is in progress over the entire surface. Section loss begins to occur.
Stage IV (Deterioration Stage)	Section loss ratio more is than 20%.

3.5 Evaluation of Bridge Soundness

3.5.1 Carbonation

Performance degradation of a structure due to carbonation is basically caused by the corrosion of reinforcing bar, sufficient care should be taken when assessing whether the structure is in the initial, propagation, acceleration or deterioration stage. The method of evaluation is through appearance or/and non-destructive test result.

3.5.1.1 Evaluation of Deterioration

a) Appearance of Structure

The state of the appearance becomes important information for performance evaluation, when the performance of structure is not directly related to the steel or concrete condition. The stages of the appearance of structures involving carbonation induced performance degradation are given in Table 3.5-2.

If corrosion of rebar affects directly the performance of the bridges, it is important to directly measure existence of the corrosion, position, area, weight, pitting corrosion depth, etc. after the cover concrete was removed. Because if the quantitative data on the corrosion is detected, it becomes possible, to quantitatively evaluate deterioration of rebar, performance deterioration of the bridges, therefore, in consequence, it is desirable that the engineering inspection should be carried out.

Table 3.5-1 Stages of Defective Appearance Due to Carbonation

Stage of Appearance	State of Deterioration
Stage I (Initial Stage)	No apparent defects. Un-carbonated cover depth is greater than the threshold value.
Stage II (Propagation Stage)	No apparent defects. Un-carbonated cover depth is less than the threshold value. Corrosion occurs.
Stage III-1 (Early Acceleration Stage)	Corrosion induced cracking occurs.
Stage III-2 (Advanced Acceleration Stage)	Many corrosion cracks occur. Severe corrosion and partial peeling-off/spalling are observed. Corrosion amount increases.
Stage IV (Deterioration Stage)	Many corrosion cracks occur. Crack widths are large. Severe corrosion and peeling-off/spalling are observed.

b) Corrosion of Rebar

Corrosion of reinforcing bar directly affects the performance of the bridges and it is important to directly measure existence of the corrosion, positions, area, weight, pitting corrosion depth, etc. after the concrete cover is removed. Carbonation test and Half-cell test can indicate the stage of rebar corrosion. The stages of corrosion are classified into four (4) depending on the carbonation test and half-cell test results.

Table 3.5-2 Stages of Rebar Corrosion Due to Carbonation

Stage of Rebar Corrosion	State of Deterioration	Carbonation Test Degree of Damage	Half-Cell Test Degree of Damage
Stage I	No corrosion	I	I
Stage II	Rebar corrosion at few portions	II	II
Stage III-2	Rebar corrosion at many portions	---	III,IV

3.5.1.2 Determination of Deterioration Stage

Deterioration shall be made in considering the degree of performance deterioration for carbonation. In carbonated bridges, affected performance is different due to the deterioration process. Therefore, it is necessary to appropriately select the performance to be evaluated in every deterioration process. In the Initial stage, there is a possibility of corrosion of rebar by concrete carbonation; therefore, the performance which protects the reinforcement becomes an object of the evaluation. In the propagation stage, corrosion of rebar progresses with the supply of oxygen through the cover. Although the steel-protecting performance is not the direct performance of the structure, this performance is important because it affects structure performance over time. In the acceleration stage, after the start of corrosion, cracks are generated over a period of the accelerated corrosion of rebar, thereby lowering not only protection performance of the reinforcement, but also the serviceability performance such as rigidity and toughness of the member. Since more corrosion progresses in the deterioration stage than in the acceleration stage, it is necessary to check the safety performance such as the load carrying capacity in addition to the performance described above for the acceleration stage. When the reinforcement corrodes, the dynamic load may become a serious problem during the propagation stage.

The stages of appearance and corrosion of rebar are classified into Stages I to IV. The stage of soundness for carbonation shall be determined as the worst stage between appearance and corrosion of rebar.

Table 3.5-3 Stage of Soundness for Carbonation (Example)

Stage of Defective Appearance (Table 3.5-1)	Stage of Rebar Corrosion (Table 3.5-2)	Stage of Soundness (Worst Stage)
III-2	II	III-2

3.5.1.3 Countermeasure of Deterioration Process

When bridge soundness for carbonation is evaluated as worst stage, countermeasure of deterioration process to be taken shall be recommended as shown in Table 3.5-4.

Table 3.5-4 Recommended Countermeasures Against Deterioration Due to Carbonation

Recommended Action	Stage I (Initial Stage)	Stage II (Propagation Stage)	Stage III-2 (Advanced Acceleration Stage)	Stage IV (Deterioration Stage)
Monitor	*	*		
Repair			*	*
Strengthen				*

3.5.2 Chloride-Induced Deterioration

Performance degradation of a structure due to chloride-induced deterioration is basically caused by the corrosion of steel reinforcement. Sufficient care should be observed during assessment on whether the structure is in the initial, propagation, acceleration or deterioration stage.

3.5.2.1 Evaluation of Deterioration

The defective features of the appearance can provide valuable information for assessing the performance of the bridges. The stages of the appearance of structures involving chloride induced performance deterioration are given by Table 3.5-5

Detailed rating of bridge soundness due to carbonation and chloride should be evaluated based on results of three inspections for “defective appearance”, “rebar corrosion and un-carbonation depth”.

Table 3.5-5 Stages of Defective Appearance Due to Chloride-Induced Deterioration

Stage of Appearance	Bridge Not Repaired	Bridge Previously Repaired
Stage I	No apparent defects. Un-carbonated cover depth is greater than the threshold value.	
Stage II	No apparent defects. Un-carbonated cover depth is less than the threshold value; Corrosion occurs.	
Stage III-1	Rust stain appears on crack at concrete surface	---
	Rust stain appears on crack at concrete surface with spalling	Portion of repaired area develops same damage after 5 years
Stage III-2	Rust stain appears on crack at concrete surface with spalling at wide area	Portion of repaired area develops same damage within 5 years or whole repaired area develops same damage after 5 years
Stage IV	Rebar exposure	Entire repaired area develops same damage within 5 years

Table 3.5-6 Stages of Rebar Corrosion Based on NDT Results

Stage of Rebar Corrosion	State of Deterioration	Carbonation Test Degree of Damage	Half Cell Test Degree of Damage
Stage I	No corrosion	I	I
Stage II	Rebar corrosion at few portion	II	II
Stage III-2	Rebar corrosion	---	III,IV

3.5.2.2 Determination of Deterioration Stage

Determination of deterioration stage shall be made considering the degree of performance and importance of the bridges.

For bridges affected by chloride-induced deterioration, affected performances are different depending on which stage the structure is in. For this reason, it is necessary to select the performance for evaluation in each deterioration stage. In the initiation stage, the possibility of corrosion of rebar occurs due to presence of chloride ions. In the propagation stage, the corrosion of rebar occurs due to the oxygen supply passing through concrete cover, and the steel-protecting performance should be assessed. Although the steel-protecting performance is not the direct performance of structure, this performance is important because of its effect on the structure’s performance over time. In the acceleration stage, as the increase of corrosion amount is significant after the beginning of the corrosion induced cracking, evaluations should be made for deductions in the serviceability performance, such as on rigidity and ductility of members. In the deterioration stage, safety performance such as load-carrying capacity should be given particular attention. In the deterioration stage, safety performance, such as load-carrying capacity, particularly requires attention. In the case where concrete with corroded reinforcement is subjected to fatigue under dynamic loading, it can pose a problem even at a relatively early stage such as the propagation stage.

The stages of appearance and corrosion of rebar are classified into Stages I to IV. The stage of soundness for chloride-induced deterioration shall be the worst stage between appearance and corrosion of rebar.

Table 3.5-7 Stage of Soundness for Chloride-Induced Deterioration (Example)

Stage of Defective Appearance (Table 3.5-1)	Stage of Rebar Corrosion (Table 3.5-2)	Stage of Soundness (Worst Stage)
III-2	II	III-2

3.5.2.3 Countermeasures of Deterioration Process

When bridge soundness for deterioration is evaluated, countermeasures that satisfy the performance requirements shall be selected. When bridge soundness for chloride-induced deterioration is evaluated as worst stage, recommended countermeasure of deterioration process to be taken is as shown in Table 3.5-8.

Table 3.5-8 Recommended Countermeasures Against Chloride-Induced Deterioration

Recommended Action	Stage I (Initial Stage)	Stage II (Propagation Stage)	Stage III-1 (Early Acceleration Stage)	Stage III-2 (Advanced Acceleration Stage)	Stage IV (Deterioration Stage)
Monitor	*	*			
Repair			*	*	*
Strengthen				*	*

3.5.3 Alkali -Silica Reaction

Performance deterioration of a bridge due to ASR is induced by expansion and subsequent cracking of concrete, therefore, performances should be assessed with sufficient care as to which stage the structures is in: the Initial, propagation, acceleration or deterioration stage.

3.5.3.1 Evaluation of Deterioration

The stages of the appearance of structures damaged by Alkali-Silica Reaction (ASR) are given in Table 3.5-9

If a structure is affected by excessive expansion due to ASR, it is necessary to examine load-carrying capacity and rigidity of whole structure and/or member by loading test, to determine the reduction in compressive strength of concrete or damage of steel. An examination of defect on surface, such as crack and gel exudation should also be conducted. On places where dislocation and faulting of structure has occurred, it is necessary to check depth of cover, state of corrosion and existence of rupture of steel bar.

Table 3.5-9 Stages of Defective Appearance Due to ASR

Stage of Appearance	State of Deterioration
Stage I (Initial Stage)	ASR occurs, but no apparent defects.
Stage II (Propagation Stage)	Cracking occurs due to ASR-induced expansion. Discoloration and gel exudation (seepage and leakage) are observed.
Stage III (Acceleration Stage)	Cracking due to ASR propagates. Number and width of cracks increase.
Stage IV (Deterioration Stage)	Many cracks due to ASR occur. Dislocation and faulting of the structure are observed. Partial peeling-off/spalling occurs. Steel corrosion progresses with rust exudation (leakage) and large displacement/deformation.

3.5.3.2 Determination of Deterioration Stage

Where quantitative evaluation of the structure is difficult, a method may be applied as a practical solution, in which apparent events of the structure are staged in accordance with Table 3.5-9.

In the determination, the bridge should be estimated based on its deterioration stage i.e. initiation period and acceleration period, by condition/engineering inspection of crack developments.

In an ASR-affected structure, crack of concrete, displacement/deformation, peeling-off/spalling, reduction in strength, existence of yielding or rupture of steel bar must be evaluated on the basis of result of inspection and results of process. It has been reported that crack due to ASR develops on surface up to the cover depth, and few crack develops within the concrete, and the influence of ASR on load-carrying capacity is small when steel bars in concrete structure are properly arranged. In most cases, crack development within the concrete cover, reduction in serviceability performance, such as steel corrosion due to cracks, crack or discoloration should be considered. However, in case of recent research on structures experiencing excessive expansion due to ASR, reduction in concrete strength, reduction in bonding between steel bar and cover concrete and rupture of steel bar at bending parts etc. have been found out. In these cases, it is necessary to estimate load-carrying capacity of structure.

3.5.3.3 Countermeasure of Deterioration Process

When bridge soundness for deterioration is evaluated, countermeasures that satisfy the performance requirements shall be selected. When bridge soundness for ASR is evaluated as worst stage, recommended countermeasure against deterioration process shall be as shown in Table 3.5-10.

Table 3.5-10 Recommended Countermeasures Against Deterioration Due to ASR

Recommended Action	Stage I (Initial Stage)	Stage II (Propagation Stage)	Stage III (Acceleration Stage)	Stage IV (Deterioration Stage)
Monitor	*	*		
Repair			*	*
Strengthen				*

3.5.4 Fatigue

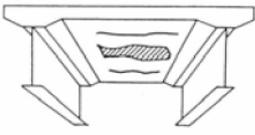
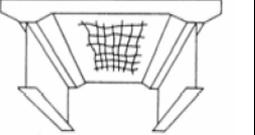
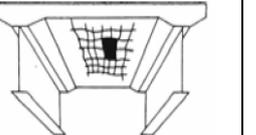
Performance deterioration due to fatigue should be assessed with sufficient care considering whether the slab is in the initial, propagation, acceleration or deterioration stage.

3.5.4.1 Evaluation of Deterioration

Evaluation of various performance of fatigue during condition/engineering inspection and for the end of the planned service period should be made by the method of Table 3.5-11.

Fatigue affects different performances of a reinforced concrete deck slab to a different degree depending on its deterioration phase. The deterioration phases can be qualitatively assessed from the state of deterioration at the bottom of the slab as shown in the table.

Table 3.5-11 Changes in the State of Slab Bottom Cracking Due to Fatigue

			
Stage I Development of cracks in one direction	Stage II Development of cracks in two directions	Stage III Cracks develop like a mesh	Stage IV Water penetrates cracks; spalling

The performance of slabs should be quantitatively assessed, but it is currently impractical. Its deterioration phase at the time of inspection or at the end of its service period is inferred from the state of the slab bottom and related to an appearance stage using Table 3.5-11.

3.5.4.2 Determination of Deterioration Stage

Determination of deterioration shall be made in considering the degree of performance degradation caused by fatigue, importance of the bridges.

The appearance stage of fatigue is related to typical performance degradation as given in Table 3.5-11.

3.5.4.3 Countermeasure of Deterioration Process

When bridge soundness for deterioration is evaluated, countermeasures that satisfy the performance requirements shall be selected. When bridge soundness for fatigue is evaluated as worst stage, countermeasure of deterioration process shall be undertaken as shown in Table 3.5-12.

Table 3.5-12 Recommended Countermeasures Against Fatigue

Recommended Action	Stage I (Initial Stage)	Stage II (Propagation Stage)	Stage III (Acceleration Stage)	Stage IV (Deterioration Stage)
Monitor	*	*		
Repair			*	*
Strengthen				*

3.5.5 Corrosion

The deterioration of the steel members is related to loss of paint coating and corrosion. The degree of progression of deterioration is caused by factors such as the natural environment, location and existing condition. Therefore, it is difficult to predict the deterioration.

The corrosion with section loss will be worsening in the future and will exert a significant impact on the performance of the structure. Therefore, when the inspector finds that there is corrosion, it should be repaired immediately.

3.5.5.1 Evaluation of Deterioration

There are two types of corrosion, one is surface corrosion and other is pitting corrosion. Surface corrosion causes uniform destruction of a relatively large surface of structural steel leading to reduction of cross-sections in the structural members. On the other hand, pitting corrosion occurs on small surface, developing deeply inside the steel and leading, in general, to the local concentration of the stresses.

Deterioration of Corrosion is divided into “Initial stage: Deterioration of Coating”, “Propagation Stage: Occurrence of Rust”, “Acceleration Stage: Rust with Section loss”, and “Deterioration Stage: More Section Loss”.

3.5.5.2 Determination of Deterioration stage

Evaluation of the deterioration due to Corrosion is shown Table 3.5-13.

Table 3.5-13 Stages of Defective Appearance Due to Corrosion

Stage of Appearance	State of Deterioration
Stage I (Initial Stage)	Paint deterioration is observed, but there is no rust.
Stage II (Propagation Stage)	(Surface Corrosion) Paint deterioration is progressing, rust of dot or rust occurs. (Pitting Corrosion) Rust fluid and crevice of coated film is observed. Film Painting is lost locally.
Stage III (Acceleration Stage)	(Surface Corrosion) Coating is almost lost. Rust is extended on the entire surface. Section loss occurs in steel member. (Pitting Corrosion) Coating cracking progresses. Section loss occurs in steel member.
Stage IV (Deterioration Stage)	Section loss is more than 20%.

3.5.5.3 Countermeasure of Deterioration Process.

When section loss occurs, the steel member will incur significant performance degradation. It is important to take countermeasures before the section loss occurs. Table 3.5-14 lists countermeasures against local corrosion and whole corrosion.

Table 3.5-14 Recommended Countermeasures Against Deterioration due to Corrosion and Paint Peel off

Recommended Action	Stage I (Initial Stage)	Stage II (Propagation Stage)	Stage III (Acceleration Stage)	Stage IV (Deterioration Stage)
Monitor	*	*		
Repair		*1	*2	*3
Strengthen				*4

*Monitor, *1: Touch-up paint, *2: Repainting, *3: Repainting + Add Plate, *4: Strengthen

3.6 Overall Assessment of Bridges

3.6.1 Overall Stage of Soundness

Overall stage of soundness is classified into four (4) stages (I, II, III, and IV). Overall soundness shall be evaluated based on the stage of soundness of each element.

Table 3.6-1 Overall Stage of Soundness

Overall Stage of Soundness	Carbonation	Chloride Induced Deterioration	Alkali-Silica Reaction	Fatigue	Corrosion
I	I	I	I	I	I
II	II	II	II	II	II
III	III	III-1	III	III	III
IV	IV	III-2, IV	IV	IV	IV

3.6.2 Residual Life Expectancy

It is difficult to determine the residual life expectancy of damaged bridges because there are many factors to consider such as: widening, aging, damage, accident, calamity, etc. There are some assessment models of residual life expectancy used in the world; however they are not accurate assessment models or residual life expectancy. Hence, it is impossible to determine residual life expectancy of bridges with accuracy.

Table 3.6-2 shows sample data on residual life expectancy of bridges in Japan.

Table 3.6-2 Residual Life Expectancy of Bridges in Japan

(a) Concrete Bridge (Reduced Serviceability, Damaged)

Construction Year	Average Life Expectancy	Remarks
1920-1930	60	
1931-1940	60	
1941-1950	60	World War II
1951-1960	60	
1961-1970	70	
1971-1980	100	
1981-1990	100	Shortage of data
1991-2000	100	Shortage of data

(b) Steel Bridge (Reduced Serviceability, Damaged)

Construction Year	Average Life expectancy	Remarks
1920-1930	50	
1931-1940	30	
1941-1950	40	World War II
1951-1960	60	
1961-1970	70	
1971-1980	70	
1981-1990	100	Shortage of data
1991-2000	100	Shortage of data

(Reference: Research on the Life Estimation of Highway Bridges, National Institute for Land and Infrastructure Management Ministry of Land, Infrastructure, Transport and Tourism, Japan, Dec. 2004)

The data can be referred to, in determining the residual life expectancy of bridges in the Philippines.

In Japan, bridge service life of 100 years is considered in the design of concrete and steel bridges to ensure structural safety and quality control as well as maintenance.

On the other hand, according to BMS in DPWH, the service lives are 50 years and 40 years for concrete bridges and steel bridges, respectively. Most of the bridges in the Philippines were constructed during World War II. Therefore, the service life of most bridges has already exceeded 50 years. However, if the bridges are maintained properly through repair and strengthening, their residual life expectancy can be prolonged.

It is therefore recommended that DPWH conduct a study on residual life expectancy of national bridges through data collection and analysis.

Furthermore, it is vital that bridges in the Philippines should be provided with countermeasures for extension of service life.

3.6.3 Repair Methods and Costs

The Inspectorate Team shall estimate the repair method and cost in accordance with the latest edition of the DPWH-JICA Bridge Repair Manual (BRM).

3.6.4 Overall Assessment

The Inspectorate Team shall determine the Overall Assessment of Bridge by the matrix of overall soundness of bridge and rating of review of condition inspection as shown in Table 3.5-2.

For instance, if overall condition rating of the subject bridge is Bad (3), evaluation of overall soundness of the same bridge is IV; overall assessment of the subject bridge shall be described as Bad-IV.

Each overall assessment shall be distinguished as follows:

Table 3.6-3 Overall Assessment Matrix

Overall Stage of Soundness	Overall Condition Rating from Condition Inspection Report	
	Poor (2)*	Bad (3)*
I,II	a) Damage condition (degree, area) may worsen. b) Progress of deterioration will be slow. c) To take countermeasures in the near future	a) Damage condition (degree, area) may worsen. b) Progress of deterioration will be slow. c) To take countermeasures as soon as possible
III	a) Damage condition (degree, area) will worsen soon. b) Progress of deterioration will worsen. c) To take countermeasures in the near future d) Residual life expectancy may be shortened	a) Damage condition (degree, area) will worsen rapidly. b) Progress of deterioration will worsen c) To take countermeasures as soon as possible d) Residual life expectancy will be shortened earlier than usual
IV	a) Damage condition (degree, area) will worsen soon. b) Progress of deterioration will worsen rapidly. c) To take countermeasures immediately. d) Residual life expectancy will be shortened earlier than usual	a) Damage condition (degree, area) will worsen rapidly. b) Progress of deterioration will worsen rapidly. c) To take countermeasures immediately d) Residual life expectancy will be shortened rapidly

* Refers to Assessment Indicators of Table 4-10: Guidelines in Assessment of Bridge Condition; Bridge Inspection Manual, Ver. 6.0; page 4-35

When determining the necessity for repair methods and selecting the type of such countermeasures, it is necessary to consider the maintenance category and to comprehensively investigate the remaining service period, life cycle cost, available budget, social impact of the structure, and ease of maintenance.

3.6.5 Recommendation

If after the overall assessment of bridge, it comes out that the total cost of bridge repair is more than 60 % of construction cost of a new bridge, the damaged bridge shall be recommended for replacement. If the total cost of bridge repair is less than 60% of construction cost of a new bridge, the damaged bridge shall be recommended for repair. The summary of bridge repair methods based on the Bridge Repair Manual is shown in APPENDIX C.

References

1. Standard Specifications For Concrete Structures-2001 “Maintenance”, Japan Society of Civil Engineers, 2005
2. Enhanced Bridge Management System JKR-BMS, Public Works Department, Kuala Lumpur, Malaysia, 2000
3. Chloride-induced Deterioration, Bridge Maintenance Manual (Draft), Committee on Countermeasures against Chloride-induced Deterioration of Bridges, 2008

APPENDIX A

BRIDGE ENGINEERING INSPECTION FORMS

(DPWH Letter Head)

(Date)

The Team Leader

Advisory Team for Bridge Engineering Inspection
DPWH-Central Office
Bonifacio Drive, Port Area, Manila

Subject: **Bridge Engineering Inspection Report**
(Name and location of bridge)

Sir:

Submitted herewith is the Bridge Engineering Inspection Report conducted last (date) at (Name) Bridge for your appropriate action.

Very truly yours,

(Name)

Team Leader
Inspectorate Team

Cc: File

Department of Public Works and Highways**BRIEF FOR ENGINEERING INSPECTION****FISCAL YEAR:** <insert budget year>**REGION:** VII**ENGINEERING DISTRICT:** <insert engineering district>

Road ID	
Section ID	
Road Name	
Bridge ID	
Bridge Name	
Type of Bridge (Bridge Code)	

Conclusions of Current Bridge Condition Report

Overall Condition of Bridge	
Recommended Action to Bridge	
Major Reason for Recommendation	
Estimated Remaining Bridge Life	

Previous Engineering Inspection Report

Prior Engineering Inspection report Available?	
Date of prior Engineering Inspection Report	
Type of Engineering Inspection Report Required	

DATE OF ISSUE: <insert date of printing from BMS>**Report BRE_01**

Department of Public Works and Highways

**ENGINEERING INSPECTION REPORT
SUMMARY SHEET**

BRIDGE INFORMATION

Bridge ID		Region	
Bridge Name		Engineering District	
Road Name		Province	
Road ID		Congressional District	
Section ID		Municipality	
Location		Barangay	

CONCLUSIONS FROM FINDINGS OF ENGINEERING INSPECTION

Recommended Work:		Overall Assessment	
None		Estimated Remaining Bridge Life (years)	
Major Maintenance		Comments on recommended work:	
Upgrading			
Replacement			
Estimated Cost of Recommended work			
Urgency of Recommended work			

DATE OF ISSUE:

<insert date of printing from BMS>

Bridge Name		Bridge ID		Date	
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**Reporting Form of Engineering Inspection Result
(Summary of Tests Conducted)**

For Concrete Bridge

Bridge Element	Attribute	Type of Material	Type of Test							
			N01	N02	N03	N04	N05-1	N05-2	N06	N07
Span 1	Deck Slab	Concrete								
	Main Member	Concrete Girder								
Span 2	Deck Slab	Concrete								
	Main Member	Concrete Girder								
Span 3	Deck Slab	Concrete								
	Main Member	Concrete Girder								
Abutment A	Main Structure	Concrete								
	Foundation	Concrete Pile								
Abutment B	Main Structure	Concrete								
	Foundation	Concrete Pile								
Pier 1	Main Structure	Concrete								
	Foundation	Concrete Pile								
Pier 2	Main Structure	Concrete								
	Foundation	Concrete Pile								

For Steel Bridge

Bridge Element	Attribute	Type of Material	NDT	
			N08	N09
Span 1	Deck	Steel		
	Main Member	Steel Girder/ Truss		
Span 2	Deck	Steel		
	Main Member	Steel Girder/ Truss		
Span 3	Deck	Steel		
	Main Member	Steel Girder/ Truss		

If the cause of defects is clearly visible or if the degree of progress of deterioration is obvious, it may not be necessary to conduct NDT (For example: Water Leakage on Deck Slab, Structural Cracks with more than 1.0mm width).

Special NDT (If necessary or if recommended)

Bridge Element	Attribute	Type of Material	Special NDT

**Reporting Form of Engineering Inspection Result
(Overall Stage of Soundness)**

Bridge Name		Bridge ID		Date	
Attributes	Carbonation (Table 3.5-3)	Chloride-Induced Deterioration (Table 3.5-7)	Alkali-Silica Reaction (Table 3.5-9)	Fatigue (Table 3.5-11)	Corrosion (Table 3.5-13)
Deck Slab					
Girder					
Pier					
Abutment					
Overall Stage of Soundness					

NOTE: For overall stage of soundness, select the worst stage

**Reporting Form of Engineering Inspection Result
Overall Assessment
(Based on Table 3.6-3)**

Bridge Name		Bridge ID		Date	
Overall Condition Rating from Condition Inspection Report	Overall Stage of Soundness		Overall Assessment		

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name		Bridge ID		Date						
Deck Slab	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction		Fatigue		Corrosion	
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Span No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)		Appearance (Table 3.5-11)		Appearance (Table 3.5-13)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)							
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)							
Stage of Soundness										

NOTE: For stage of soundness, select the worst stage

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name		Bridge ID		Date				
Girder	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction	Corrosion		
	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	Appearance (Table 3.5-13)		
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
Soundness (Table 3.5-3)		Soundness (Table 3.5-7)						
Span No. _ Girder No. _	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	Appearance (Table 3.5-13)		
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Span No. _ Girder No. _	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	Appearance (Table 3.5-13)		
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Span No. _ Girder No. _	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	Appearance (Table 3.5-13)		
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)					
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)					
Stage of Soundness								
<i>NOTE: For stage of soundness, select the worst stage</i>								

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name		Bridge ID	Date
-------------	--	-----------	------

Pier	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction	
	Pier No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)			
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)			
Pier No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)			
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)			
Pier No. —	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)	
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)			
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)			
Stage of Soundness						
<i>NOTE: For stage of soundness, select the worst stage</i>						

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name		Bridge ID		Date	
Abutment	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction
Abutment A-1	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)		
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)		
Abutment A-2	Appearance (Table 3.5-1)		Appearance (Table 3.5-5)		Appearance (Table 3.5-9)
	Rebar Corrosion (Table 3.5-2)		Rebar Corrosion (Table 3.5-6)		
	Soundness (Table 3.5-3)		Soundness (Table 3.5-7)		
Stage of Soundness					
<i>NOTE: For stage of soundness, select the worst stage</i>					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name		Bridge ID		Date	
Bridge Element		Attribute			
N01 Rebound Hammer Test <input type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings:					
Conclusions/Recommendations:					
N02 Carbonation Test					
Findings:					
Conclusions/Recommendations:					
N03 Ultrasonic Pulse Velocity Test					
Findings:					
Conclusions/Recommendations:					
N04 Half-Cell Electrical Potential Test					
Findings:					
Conclusions/Recommendations:					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input type="checkbox"/>					
Findings:					
Conclusions/Recommendations:					
N06 Infrared Thermal Image Test					
Findings:					
Conclusions/Recommendations:					
N08 Paint Thickness Test					
Findings:					
Conclusions/Recommendations:					
N09 Metal Thickness Test					
Findings:					
Conclusions/Recommendations:					

Inspection Record (Summary Sheet for Concrete Elements 1/4)

Bridge Name				Bridge ID		
Inspector		Date		Weather		
Bridge Element	Span	Attribute	Deck			
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Deck Slab	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth				mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking				mm	
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength				MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
	Transverse Rebar Size: __ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	D ≥ 25mm
II	D < 25mm

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	-200 < E
II	-200 ≤ E ≤ - 350
III	-350 < E < - 500
IV	E ≤ - 500

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	C/2 > D: (C=Conc. cover)
II	C > D ≥ C/2 (C ≥ 40mm)
III	C > D ≥ C/2 (C < 40mm)
IV	D ≥ C

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	F ≥ F _c
II	0.8F _c < F < F _c
III	F < 0.8F _c

Inspection Record (Summary Sheet for Concrete Elements 2/4)

Bridge Name				Bridge ID		
Inspector		Date		Weather		
Bridge Element	Span	Attribute	Main Member			
Span _ - Girder _	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth					
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking					
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test					
	Compressive Strength		MPa			
	Rebar Detection Test (Radar Type)					
	Longitudinal Rebar Size: ____ mm ϕ		Spacing	Minimum		mm
				Maximum		mm
	Stirrups Size: ____ mm ϕ		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Girder _	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth					
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking					
	Half Cell Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test					
	Compressive Strength		MPa			
	Rebar Detection Test (Radar Type)					
	Longitudinal Rebar Size: __mm ϕ		Spacing	Minimum		mm
				Maximum		mm
	Stirrups Size: ____mm ϕ		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						
Span _ - Girder _	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth					
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking					
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test /		Degree of Damage			
	<input type="checkbox"/> Compressive Strength Test					
	Compressive Strength		MPa			
	Rebar Detection Test (Radar Type)					
	Longitudinal Rebar Size: ____mm ϕ		Spacing	Minimum		mm
				Maximum		mm
	Stirrups Size: ____mm ϕ		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$
Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C $\geq 40\text{mm}$)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F_c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

Inspection Record (Summary Sheet for Concrete Elements 3/4)

Bridge Name				Bridge ID		
Inspector		Date		Weather		
Bridge Element	Pier	Attribute	Main Structure			
Pier No. ___	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth			mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking			mm		
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength			MPa		
	Rebar Detection Test (Radar Type)					
	Horizontal Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
	Vertical Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test		With extensive delamination				
Pier No. ___	Carbonation Test		Degree of Damage			
	Un-Carbonated Depth			mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking			mm		
	Half Cell Electrical Potential Test		Degree of Damage			
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage			
	Compressive Strength			MPa		
	Rebar Detection Test (Magnetic Type)					
	Horizontal Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
			Cover	Minimum		mm
				Maximum		mm
	Vertical Rebar Size: ___ mm Ø		Spacing	Minimum		mm
				Maximum		mm
		Cover	Minimum		mm	
			Maximum		mm	
Infrared Thermal Image Test						

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C ≥ 40mm)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F_c = Design compressive F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

Inspection Record

(Summary Sheet for Concrete Elements 4/4)

Bridge Name				Bridge ID			
Inspector			Date			Weather	
Bridge Element	Abutment	Attribute	Main Structure				
Abutment A1	Carbonation Test		Degree of Damage				
	Un-Carbonated Depth				mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage				
	Depth of Cracking				mm		
	Half Cell Test		Degree of Damage				
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage				
	Compressive Strength				MPa		
	Rebar Detection Test (Magnetic Type)						
	Longitudinal Rebar Size: ___ mm ϕ		Spacing	Minimum		mm	
				Maximum		mm	
	Cover		Cover	Minimum		mm	
				Maximum		mm	
	Transverse Rebar Size: ___ mm ϕ		Spacing	Minimum		mm	
				Maximum		mm	
Cover		Cover	Minimum		mm		
			Maximum		mm		
Infrared Thermal Image Test							
Abutment A2	Carbonation Test		Degree of Damage				
	Un-Carbonated Depth				mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage				
	Depth of Cracking				mm		
	Half Cell Test		Degree of Damage				
	<input type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage				
	Compressive Strength				MPa		
	Rebar Detection Test (Magnetic Type)						
	Longitudinal Rebar Size: ___ mm ϕ		Spacing	Minimum		mm	
				Maximum		mm	
	Cover		Cover	Minimum		mm	
				Maximum		mm	
	Transverse Rebar Size: ___ mm ϕ		Spacing	Minimum		mm	
				Maximum		mm	
Cover		Cover	Minimum		mm		
			Maximum		mm		
Infrared Thermal Image Test							

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C \geq 40mm)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

Inspection Record (Summary Sheet for Steel Elements)

Bridge Name		Bridge ID	
Inspector	Date	Weather	
Bridge Element	Attribute		
Girder <input type="checkbox"/> Upper Flange <input type="checkbox"/> Web <input type="checkbox"/> Lower Flange	Paint Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Metal Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Cross Beam <input type="checkbox"/> Upper Flange <input type="checkbox"/> Web <input type="checkbox"/> Lower Flange	Paint Thickness Test	
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Metal Thickness Test			
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Truss <input type="checkbox"/> Upper Chord <input type="checkbox"/> Lower Chord <input type="checkbox"/> Diagonal <input type="checkbox"/> Vertical <input type="checkbox"/> Cross Beam		Paint Thickness Test	
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Metal Thickness Test		
	Undamaged Point		mm
	Defective Point		mm
	Decrease Ratio		%
	Deck Slab <input type="checkbox"/> Deck Plate	Paint Thickness Test	
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%
Metal Thickness Test			
Undamaged Point			mm
Defective Point			mm
Decrease Ratio			%

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name		Bridge ID	
Inspector	Date:	Weather	

Sketches of Inspected Location

Inspection Result	
Test Point No. :	Actual Compressive Strength :
Bridge Element/Attribute :	Number of Test :

Test Results			

Inspection Record (Rebound Hammer Test 2/2)

Bridge Name		Bridge ID	
Inspector	Date	Weather	

Photo Records	
Photo-1 : General View	Photo-2 : Test Point
Photo-3 : Testing	Photo-4 : After Test (If Necessary)

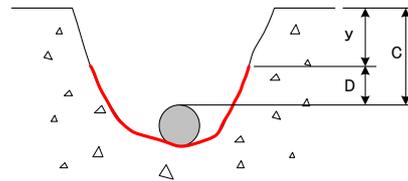
Inspection Record (Carbonation Test / Chipping-Off 1/2)

Bridge Name:		Bridge ID	
Inspector		Date	Weather

Sketch of Inspected Location

Inspection Result

Construction Year:	
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Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)

Inspection Record (Carbonation Test / Chipping-Off 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

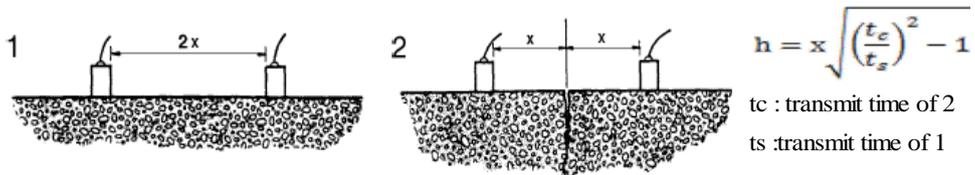
Photo Records	
Photo-1 : General View	Photo-2 : Test Point
Photo-3 : Testing	Photo-4 : After Test (If Necessary)

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 1/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Sketches of Inspected Location

Inspection Result



Test Point No.	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2 tc (ms)	Transmit time of 1 ts (ms)	Crack Depth h (mm)

Inspection Record

(Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name		Bridge ID	
Inspector	Date	Weather	

Photo Records	
Photo-1 : General View	Photo-2 : Test Point
Photo-3 : Testing	Photo-4 : After Test (If Necessary)

Inspection Record (Half-Cell Electrical Potential Test 1/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Sketch of Inspected Location

Inspection Result

Test Point No. ___
Equi-potential Contour Map

Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Photo Records

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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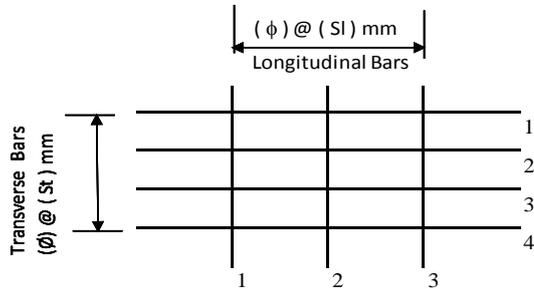
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Inspection Record (Rebar Detection Test / Magnetic Type 1/2)

Bridge Name		Bridge ID	
Inspector	Date	Weather	

Sketch of Inspected Location

Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Longitudinal Bars			Transverse Bars		
			Rebar Dia. (mm)	Spacing (mm)	Cover (mm)	Rebar Dia. (mm)	Spacing (mm)	Cover (mm)
Transverse	1							
	2							
	3							
	4							
Longitudinal	1							
	2							
	3							

Inspection Record (Rebar Detection Test / Magnetic Type 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Photo Records

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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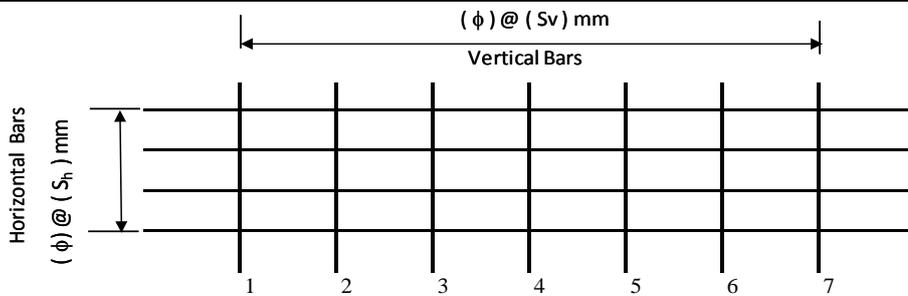
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Inspection Record (Rebar Detection Test / Radar Type 1/2)

Bridge Name		Bridge ID	
Inspector	Date	Weather	

Sketch of Inspected Location

Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Vertical Bars		Horizontal Bars	
			Spacing (mm)	Cover (mm)	Spacing (mm)	Cover (mm)
Vertical Bars						

Inspection Record (Rebar Detection Test / Radar Type 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Photo Records

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Infrared Thermal Image Test)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Sketch of Inspected Location:

Inspection Result

Test Point No.	Bridge Element / Attribute	Type of Structure

Visual Image	Thermal Image

Comment:	Comment:
-----------------	-----------------

Inspection Record (Paint Thickness Test 1/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Sketch of Inspected Location

Inspection Result																																							
<table border="1" style="margin: auto; border-collapse: collapse;"> <thead> <tr> <th style="width: 40%;">Test Point</th> <th style="width: 10%;">1</th> <th style="width: 10%;">2</th> <th style="width: 10%;">3</th> <th style="width: 10%;">4</th> </tr> </thead> <tbody> <tr> <td>Bridge Element/Attribute</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Type of Material</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Soundness Point * (μm)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Defect Point * (μm)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Decrease Ratio (%)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Defect Area (m²)</td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table> <p style="text-align: center; margin-top: 10px;">*: Average of more than 3 points</p>					Test Point	1	2	3	4	Bridge Element/Attribute					Type of Material					Soundness Point * (μm)					Defect Point * (μm)					Decrease Ratio (%)					Defect Area (m ²)				
Test Point	1	2	3	4																																			
Bridge Element/Attribute																																							
Type of Material																																							
Soundness Point * (μm)																																							
Defect Point * (μm)																																							
Decrease Ratio (%)																																							
Defect Area (m ²)																																							

Inspection Record (Paint Thickness Test 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Photo Records	
Photo-1 : General View	Photo-2 : Test Point
Photo-3 : Testing	Photo-4 : After Test (If Necessary)

Inspection Record (Metal Thickness Test 1/2)

Bridge Name		Bridge ID	
Inspector		Date	
		Weather	

Sketch of Inspected Location

Inspection Result																																							
<table border="1" style="margin: auto; border-collapse: collapse;"> <thead> <tr> <th style="width: 40%;">Test Point</th> <th style="width: 10%;">1</th> <th style="width: 10%;">2</th> <th style="width: 10%;">3</th> <th style="width: 10%;">4</th> </tr> </thead> <tbody> <tr> <td>Bridge Element/Attribute</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Type of Material</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Soundness Point * (mm)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Defect Point * (mm)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Decrease Ratio (%)</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Defect Area (m²)</td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table> <p style="text-align: center; margin-top: 10px;">*: Average of more than 3 points</p>					Test Point	1	2	3	4	Bridge Element/Attribute					Type of Material					Soundness Point * (mm)					Defect Point * (mm)					Decrease Ratio (%)					Defect Area (m ²)				
Test Point	1	2	3	4																																			
Bridge Element/Attribute																																							
Type of Material																																							
Soundness Point * (mm)																																							
Defect Point * (mm)																																							
Decrease Ratio (%)																																							
Defect Area (m ²)																																							

Inspection Record (Metal Thickness Test 2/2)

Bridge Name		Bridge ID	
Inspector		Date	Weather

Photo Records	
Photo-1 : General View	Photo-2 : Test Point
Photo-3 : Testing	Photo-4 : After Test (If Necessary)

APPENDIX B

EXAMPLE OF BRIDGE ENGINEERING INSPECTION REPORT



Republic of the Philippines
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
OFFICE OF THE REGIONAL DIRECTOR
REGION VII, Cebu City

28 February 2013

The Team Leader

Advisory Team for Bridge Engineering Inspection
DPWH-Central Office
Bonifacio Drive, Port Area, Manila

Subject: **Bridge Engineering Inspection Report**
Malaga Bridge, Dumaguete North Road, Km. 64 + 532
Malaga, Negros Oriental

Sir:

Submitted herewith is the Bridge Engineering Inspection Report conducted last 28-29 Nov 2012, 30 January -1 February 2013 and 20-21 February 2013 at Malaga Bridge for your appropriate action.

Very truly yours,

FELICIANO R. ESPINA

Team Leader
Inspectorate Team

Cc: File

Department of Public Works and Highways**BRIEF FOR ENGINEERING INSPECTION****FISCAL YEAR:** <insert budget year>**REGION:** VII**ENGINEERING DISTRICT:** Negros Oriental 1st DEO (NORFED)

Road ID	R00049NR
Section ID	S00418NR
Road Name	DUMAGUETE NORTH ROAD
Bridge ID	B00092NR
Bridge Name	MALAGA BRIDGE
Type of Bridge (Bridge Code)	Standard (RCDG)

Conclusions of Current Bridge Condition Report

Overall Condition of Bridge	Bad
Recommended Action to Bridge	Major Maintenance
Major Reason for Recommendation	Safety/Bridge Deterioration
Estimated Remaining Bridge Life	0 ~10 Years

Previous Engineering Inspection Report

Prior Engineering Inspection report Available?	None
Date of prior Engineering Inspection Report	None
Type of Engineering Inspection Report Required	New

DATE OF ISSUE: <insert date of printing from BMS>**Report BRE_01**

Department of Public Works and Highways

ENGINEERING INSPECTION REPORT SUMMARY SHEET

BRIDGE INFORMATION

Bridge ID	B00092NR	Region	REGION VII
Bridge Name	MALAGA BRIDGE	Engineering District	NEGROS ORIENTAL 1ST DEO
Road Name	DUMAGUETE NORTH ROAD	Province	NEGROS ORIENTAL
Road ID	R00049NR	Congressional District	1ST DISTRICT
Section ID	S00418NR	Municipality	BINDOY
Location	K0064+581	Barangay	MALAGA

CONCLUSIONS FROM FINDINGS OF ENGINEERING INSPECTION

Recommended Work:		Overall Assessment	BAD-IV
None		Estimated remaining bridge life (years)	0 ~ 10
Major Maintenance	<input checked="" type="checkbox"/>	Comments on recommended work:	
Upgrading		1.Undertake major maintenance immediately. 2.Damage condition will worsen rapidly particularly Pier No1 and Deck slab. 3.Progress of deterioration will worsen rapidly same as 2. 4. Residual life expectancy will be shortened rapidly. 5. Overall soundness: With regards to Deck slab, soundness is IV. Main cause may be over loading. With regards to Pier, soundness is IV, Main cause is the the bridge is near seashore.	
Replacement			
Estimated Cost of Recommended work	*		
Urgency of Recommended work			
<p style="text-align: center;">*: Based on Bridge Repair Manual</p>			

DATE OF ISSUE:

<insert date of printing from BMS>

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)

Form EI-02a

REVIEW FORM

INVENTORY DATA

Bridge ID	B00092NR	Bridge Name	MALAGA BRIDGE
Road ID	R00049NR	Road Name	DUMAGUETE NORTH ROAD
Region	VII	District office	NEGROS ORIENTAL 1ST ENGINEERING DISTRICT
Location	BINDOY, NEGROS ORIENTAL	(Near Coastal Area or Hot Spring)	✓

BRIDGE DESCRIPTION

Type of Bridge	RCDG	No. of Span	3
Type of Superstructure	RC Girder	No. of Abutment	2
Type of Substructure	Wall	No. of Pier	2
Type of Foundation	Not Known		

Overall Condition State (Based on Condition Inspection Result)	BAD
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Inspection Date	29-May-10
Inspection Date	20-Feb-13

Condition Inspection Engineering Inspection
--

No.	Type of Damage*	Findings from Condition Inspection Report (Type 2 Inspection)				Verification Damage Rating (based on ocular investigation by Inspectorate Team)	Evaluation Items						Recommended NDT (Refer to Table 1-1)													
		Bridge Element	Bridge Attribute	Type of Material	Damage Rating		(A) Structural	(B) Chloride	(C) Progress	(D) No Accessibility	(E) LC/AV	(F) Under-water	(A)	(B)	(C)	(D)	(E)	(F)								
1	Water Leakage	Span 1	Deck Slab	Concrete	Bad	✓	✓	✓	✓	✓							N01	N01	N01							
2	Water Leakage	Span 2	Deck Slab	Concrete	Bad	✓	✓	✓	✓									N02	N02	N02						
3	Water Leakage	Span 3	Deck Slab	Concrete	Bad	✓	✓	✓	✓									N03	N03	N03						
4	Spalling	Pier 1	Main Structure (Wall)	Concrete	Poor		✓	✓	✓	✓								N04	N04	N04						
Comments		Elements that are inaccessible (Partially inspected or not inspected by BI)																								
: Omitted NDT (It is necessary to replace immediately Deck Slab of Span 3 based on its appearance. Exposure and section loss of rebar at Pier 1, therefore N03 is omitted.)																										

LC/AV = Load Capacity/ Abnormal Vibration

*: Fill in damage with a condition state of Poor or Bad

BRIDGE ENGINEERING INSPECTION FORM (TYPE 3)		MALAGA BRIDGE				RCDG													
Bridge ID		Bridge Name		Type of Bridge		Recommended NDT (Refer to Table 1-1)													
B00092NR		MALAGA BRIDGE		Type of Bridge		Recommended NDT (Refer to Table 1-1)													
Findings from Condition Inspection Report (Type 2 Inspection)		Verification		Evaluation Items															
No.	Type of Damage*	Bridge Element	Bridge Attribute	Type of Material	Damage Rating	Damage Rating (based on ocular investigation by Inspectorate Team)	(A) Structural	(B) Chloride	(C) Progress	(D) No Accessibility	(E) LC/AV	(F) Under-water	(A)	(B)	(C)	(D)	(E)	(F)	
5	Delamination	Pier 1	Main Structure	Concrete	Poor	Poor		✓		✓	✓			N01					
6	Cracking	Span 1	Deck Slab	Conc	Poor	Poor	✓	✓	✓				N01	N01	N01				
7	Cracking	Span 2	Deck Slab	Conc	Poor	Poor	✓	✓	✓				N02	N02	N02				
8	Cracking	Span 3	Deck Slab	Conc	Poor	Poor	✓	✓	✓				N03	N03	N03				
9	Cracking	Span 1	Girdler	Conc	Poor	Poor	✓	✓	✓				N04	N04	N04				
10	Cracking	Span 2	Girdler	Conc	Poor	Poor	✓	✓	✓				N05	N05	N05				
11	Cracking	Span 3	Girdler	Conc	Poor	Poor	✓	✓	✓				N01	N01	N01				
Comments							Elements that are inaccessible (Partially inspected or not inspected by BI)												

*:Fill in damage with a condition state of Poor or Bad

LC/AV = Load Capacity/ Abnormal Vibration

Bridge Name	Malaga Bridge	Bridge ID	B00092NR	Date	20-Feb-13
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**Reporting Form of Engineering Inspection Result
(Summary of Tests Conducted)**

For Concrete Elements

Bridge Element	Attribute	Type of Material	Type of Test							
			N01	N02	N03	N04	N05-1	N05-2	N06	N07
Span	1 Deck Slab	Concrete	✓	✓	✓	✓	✓	-	-	-
	Main Member	Concrete Girder	✓	✓	✓	✓	-	✓	-	-
Span	2 Deck Slab	Concrete	✓	✓	✓	✓	✓	-	-	-
	Main Member	Concrete Girder	✓	✓	✓	✓	-	✓	-	-
Span	3 Deck Slab	Concrete	*	*	*	*	*	-	-	-
	Main Member	Concrete Girder	*	*	*	*	-	*	-	-
Abutment A	Main Structure	Concrete	-	-	-	-	-	-	-	-
	Foundation	Concrete Pile	-	-	-	-	-	-	-	-
Abutment B	Main Structure	Concrete	-	-	-	-	-	-	-	-
	Foundation	Concrete Pile	-	-	-	-	-	-	-	-
Pier	1 Main Structure	Concrete	✓	✓	*	✓	-	✓	✓	-
	Foundation	Concrete Pile	-	-	-	-	-	-	-	-
Pier	2 Main Structure	Concrete	-	-	-	-	-	-	-	-
	Foundation	Concrete Pile	-	-	-	-	-	-	-	-

For Steel Elements

Bridge Element	Attribute	Type of Material	NDT	
			N08	N09
Span	1 Deck	Steel	-	-
	Main Member	Steel Girder/ Truss	-	-
Span	2 Deck	Steel	-	-
	Main Member	Steel Girder/ Truss	-	-
Span	3 Deck	Steel	-	-
	Main Member	Steel Girder/ Truss	-	-

If the cause of defects is clearly visible or if the degree of progress of deterioration is obvious, it may not be necessary to conduct NDT (For example: Water Leakage on Deck Slab, Structural Cracks with more than 1.0mm width).

*:Omitted NDT

Special NDT (If necessary or if recommended)

Bridge Element	Attribute	Type of Material	Special NDT

Reporting Form of Engineering Inspection Result
(Overall Stage of Soundness)

Bridge Name	Malaga Bridge	Bridge ID	B00092NR	Date	20-Feb-13
Attributes	Carbonation (Table 3.5-3)	Chloride-Induced Deterioration (Table 3.5-7)	Alkali-Silica Reaction (Table 3.5-9)	Fatigue (Table 3.5-11)	Corrosion (Table 3.5-13)
Deck Slab	III-2	III-2	----	IV	----
Girder	II	II	----	/	----
Pier	IV	IV	----	/	/
Abutment	----			/	/
Overall Stage of Soundness	IV	IV	----	IV	----

NOTE: For overall stage of soundness, select the worst stage

Reporting Form of Engineering Inspection Result
Overall Assessment
(Based on Table 3.6-3)

Bridge Name	Malaga Bridge	Bridge ID	B00092NR	Date	20-Feb-13
Overall Condition Rating from Condition Inspection Report	Overall Assessment of Bridge Soundness		Overall Assessment		
BAD	IV		BAD-IV		

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name	Malaga Bridge	Bridge ID	B00092NR	Date	20-Feb-13
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Deck Slab	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction		Fatigue		Corrosion	
	Span No. 1	Appearance (Table 3.5-1)	III-1	Appearance (Table 3.5-5)	III-1	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-11)	IV	Appearance (Table 3.5-13)
Rebar Corrosion (Table 3.5-2)		III-2	Rebar Corrosion (Table 3.5-6)	III-2						
Soundness (Table 3.5-3)		III-2	Soundness (Table 3.5-7)	III-2						
Span No. 2	Appearance (Table 3.5-1)	II	Appearance (Table 3.5-5)	II	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-11)	IV	Appearance (Table 3.5-13)	---
	Rebar Corrosion (Table 3.5-2)	II	Rebar Corrosion (Table 3.5-6)	II						
	Soundness (Table 3.5-3)	II	Soundness (Table 3.5-7)	II						
Span No. 3	Appearance (Table 3.5-1)	III-1	Appearance (Table 3.5-5)	III-1	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-11)	IV	Appearance (Table 3.5-13)	---
	Rebar Corrosion (Table 3.5-2)	---	Rebar Corrosion (Table 3.5-6)	---						
	Soundness (Table 3.5-3)	III-1	Soundness (Table 3.5-7)	III-1						
Stage of Soundness	III-2		III-2		---		IV		---	

NOTE: For stage of soundness, select the worst stage

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name	Malaga Bridge	Bridge ID	B00092NR	Date	20-Feb-13			
Girder	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction		Corrosion	
	Appearance (Table 3.5-1)	I	Appearance (Table 3.5-5)	I	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-13)	---
Rebar Corrosion (Table 3.5-2)	II	Rebar Corrosion (Table 3.5-6)	II					
Soundness (Table 3.5-3)	II	Soundness (Table 3.5-7)	II					
Span No. _ Girder No. _	Appearance (Table 3.5-1)	II	Appearance (Table 3.5-5)	II	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-13)	---
	Rebar Corrosion (Table 3.5-2)	II	Rebar Corrosion (Table 3.5-6)	II				
	Soundness (Table 3.5-3)	II	Soundness (Table 3.5-7)	II				
Span No. _ Girder No. _	Appearance (Table 3.5-1)	II	Appearance (Table 3.5-5)	II	Appearance (Table 3.5-9)	---	Appearance (Table 3.5-13)	---
	Rebar Corrosion (Table 3.5-2)	II	Rebar Corrosion (Table 3.5-6)	II				
	Soundness (Table 3.5-3)	II	Soundness (Table 3.5-7)	II				
Stage of Soundness	II		II		-----		-----	
<i>NOTE: For stage of soundness, select the worst stage</i>								

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	Date	20-Feb-13
Pier	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction	
	Appearance (Table 3.5-1)	IV	Appearance (Table 3.5-5)	IV	Appearance (Table 3.5-9)	---
Pier No. 1	Rebar Corrosion (Table 3.5-2)	III-2	Rebar Corrosion (Table 3.5-6)	III-2		
	Soundness (Table 3.5-3)	IV	Soundness (Table 3.5-7)	IV		
Pier No. 2	Appearance (Table 3.5-1)	---	Appearance (Table 3.5-5)	---	Appearance (Table 3.5-9)	---
	Rebar Corrosion (Table 3.5-2)	---	Rebar Corrosion (Table 3.5-6)	---		
	Soundness (Table 3.5-3)	---	Soundness (Table 3.5-7)	---		
Stage of Soundness	IV		IV		-----	
<i>NOTE: For stage of soundness, select the worst stage</i>						

**Reporting Form of Engineering Inspection Result
(Stage of Soundness of Element and Attribute)**

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	Date	20-Feb-13
Abutment	Carbonation		Chloride-Induced Deterioration		Alkali-Silica Reaction	
Abutment A-1	Appearance (Table 3.5-1)	---	Appearance (Table 3.5-5)	---	Appearance (Table 3.5-9)	---
	Rebar Corrosion (Table 3.5-2)	---	Rebar Corrosion (Table 3.5-6)	---		
	Soundness (Table 3.5-3)	---	Soundness (Table 3.5-7)	---		
Abutment A-2	Appearance (Table 3.5-1)	---	Appearance (Table 3.5-5)	---	Appearance (Table 3.5-9)	---
	Rebar Corrosion (Table 3.5-2)	---	Rebar Corrosion (Table 3.5-6)	---		
	Soundness (Table 3.5-3)	---	Soundness (Table 3.5-7)	---		
Stage of Soundness	-----		-----		-----	
<i>NOTE: For stage of soundness, select the worst stage</i>						

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	SPAN -1	Attribute	DECK SLAB (Between G1 and G2)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings: Actual compressive strength, F = 17.0 MPa (Prior to 1957, design compressive strength Fc = 17.2 MPa)					
Conclusions/Recommendations: Since actual compressive strength is more than 0.80 Fc but less than Fc, the attribute is considered in poor condition. However, durability can be maintained by repair					
N02 Carbonation Test					
Findings: Concrete cover = 50mm, carbonated depth = 30mm, uncarbonated depth = 20mm					
Conclusions/Recommendations: Concrete is carbonated up to 60% of depth of cover Rebars maybe corroded					
N03 Ultrasonic Pulse Velocity Test					
Findings: Slab thickness = 200mm; Shear crack maximum width = 0.30mm Shear crack extended up to girder					
Conclusions/Recommendations: Rebars are corroded					
N04 Half-Cell Electrical Potential Test					
Findings: Readings range from -350mV to -500mV					
Conclusions/Recommendations: Rebars must be corroded					
N05 Rebar Detection Test Magnetic <input checked="" type="checkbox"/> Radar <input type="checkbox"/>					
Findings: Longitudinal bars = 12mm dia, spacing = 200mm on center, cover = 29mm Transverse bars = 16mm dia, spacing 150mm on center, cover = 28mm					
Conclusions/Recommendations: Check results with as-built plans, if available					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	SPAN -2	Attribute	DECK SLAB (Between G1 and G2)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings: Actual compressive strength, F = 17.56 MPa (Prior to 1957, design compressive strength, Fc = 17.2 MPa)					
Conclusions/Recommendations: Since actual compressive strength > 17.2 Mpa, attribute is considered still in good condition					
N02 Carbonation Test					
Findings: Concrete cover = 30mm, carbonated depth = 40mm					
Conclusions/Recommendations: Rebar may be corroded.					
N03 Ultrasonic Pulse Velocity Test					
Findings: Slab thickness = 200mm; Shear crack maximum width = 0.30mm Shear crack extended up to girder					
Conclusions/Recommendations: Rebars are corroded					
N04 Half-Cell Electrical Potential Test					
Findings: Readings range from -100mV to -300mV					
Conclusions/Recommendations: Rebars maybe partly corroded					
N05 Rebar Detection Test Magnetic <input checked="" type="checkbox"/> Radar <input type="checkbox"/>					
Findings: Longitudinal bars = 12mm dia. , spacing = 377mm on center, cover = 31mm Transverse bars = 16mm dia., spacing = 157mm on center, cover = 36mm					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	SPAN -1	Attribute	GIRDER (G2)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings: Actual compressive strength, F = 18.90 MPa (Prior to 1957, design compressive strength, Fc = 17.2 MPa)					
Conclusions/Recommendations: Since actual compressive strength > 17.20 Mpa, attribute is still in good condition;					
N02 Carbonation Test					
Findings: Concrete cover = 68mm, Carbonated depth = 35mm, uncarbonated depth =33mm					
Conclusions/Recommendations: Concrete is carbonated up to 51.47% of depth of cover Rebars will be corroded in near future					
N03 Ultrasonic Pulse Velocity Test					
Findings: G-1, G-2, G-3 (Thickness of Girder = 480mm) G-1: Shear crack max width = 0.30mm; crack has penetrated girder already G-2: Shear crack max width = 0.30mm; crack has penetrated girder already G-3: Shear crack max width = 0.30mm; crack max depth = 285.5mm					
Conclusions/Recommendations: Typical for G-1 to G-3 Rebars are corroded Conduct further investigation Repair is required					
N04 Half-Cell Electrical Potential Test					
Findings: About 10% of readings are below -200 mV					
Conclusions/Recommendations: Rebars not corroded; if corrosion occur, it may be due to water seepage on cracks					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input checked="" type="checkbox"/>					
Findings: Stirrups spaced @ 250, 300, 400, 450mm; cover = 68mm					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	SPAN -2	Attribute	GIRDER (G2)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings: Actual compressive strength, F = 14.80 MPa (Prior to 1957, design compressive strength Fc = 17.2 MPa)					
Conclusions/Recommendations: Since actual compressive strength is more than 0.80 Fc but less than Fc, the attribute is considered in poor condition. However, durability can be maintained by repair					
N02 Carbonation Test					
Findings: Concrete cover = 70mm, carbonated depth = 20mm, uncarbonated depth =50mm					
Conclusions/Recommendations: Rebars will be corroded in near future					
N03 Ultrasonic Pulse Velocity Test					
Findings: Girder width = 480mm Shear crack maximum width = 0.30mm; crack maximum depth = 327.6mm					
Conclusions/Recommendations: Rebars may be corroded					
N04 Half-Cell Electrical Potential Test					
Findings: 100% of readings are greater than -200 mV					
Conclusions/Recommendations: Rebars will be corroded in near future					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input checked="" type="checkbox"/>					
Findings: Stirrups spaced at 435mm, cover =70mm Longitudinal (main) rebar spaced at 410mm, cover =62mm					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	SPAN -3	Attribute	GIRDER (G1)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings: Actual compressive strength, $F = 17.1$ MPa (Prior to 1957, design compressive strength $F_c = 17.2$ MPa)					
Conclusions/Recommendations: Since actual compressive strength is more than $0.80F_c$ but less than F_c , the attribute is considered in poor condition. However, durability can be maintained by repair					
N02 Carbonation Test					
Findings: Concrete cover = 50 mm, carbonated depth = 36mm Uncarbonated depth = 14mm					
Conclusions/Recommendations: Concrete is carbonated up to 72% of total cover Rebar maybe corroded					
N03 Ultrasonic Pulse Velocity Test					
Findings: Girder is 480mm thk Shear crack max width = 286.3mm					
Conclusions/Recommendations: Rebars must be corroded					
N04 Half-Cell Electrical Potential Test					
Findings: Upper side readings between -200 to 350mV Lower side readings greater than -200mV					
Conclusions/Recommendations: Rebars near top of girder are may be partly corroded					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input checked="" type="checkbox"/>					
Findings: Stirrups spaced at 200, 300mm; cover = 68mm					

**Reporting Form of Engineering Inspection Result
(Narrative Summary of Test Results)**

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR	Date	20-Feb-13
Bridge Element	PIER 1	Attribute	MAIN STRUCTURE (WALL)		
N01 Rebound Hammer Test <input checked="" type="checkbox"/> / N07 Compressive Strength Test <input type="checkbox"/>					
Findings:					
Actual compressive strength, $F = 15.89$ MPa (Prior to 1957, design compressive strength $F_c = 17.2$ MPa)					
Conclusions/Recommendations:					
Since actual compressive strength is more than $0.80 F_c$ but less than F_c , the attribute is considered in poor condition. However, durability can be maintained by repair					
N02 Carbonation Test					
Findings:					
Concrete cover = 95 mm, Carbonated depth = 13mm Uncarbonated depth = 82mm					
Conclusions/Recommendations:					
Concrete is carbonated up to 14% of total cover Rebar will be corroded in the future					
N04 Half-Cell Electrical Potential Test					
Findings:					
Upper side readings between -200 to 500mV Lower side readings less than -500mV					
Conclusions/Recommendations:					
Rebars near bottom of pier wall are seriously corroded					
N05 Rebar Detection Test N05-01 Magnetic <input type="checkbox"/> N05-02 Radar <input checked="" type="checkbox"/>					
Findings:					
Vertical bars spaced at 500mm on center, cover = 80mm Horizontal bars spaced at 500mm on center, cover = 60mm					
N06 Infrared Thermal Image Test					
Findings:					
Infrared image indicate extensive delamination and spalling on wall facing Bindoy side					
Conclusions/Recommendations:					
Patching (refer to Bridge Repair Manual)					

Inspection Record (Summary Sheet for Concrete Elements 1/4)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	RAINY	
Bridge Element	Span	Attribute	Deck			
Span 1 - Deck Slab	Carbonation Test		Degree of Damage	II		
	Un-Carbonated Depth			20	mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage	IV		
	Depth of Cracking			95	mm	
	Half Cell Electrical Potential Test		Degree of Damage	III		
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	II		
	Compressive Strength			17	MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: 12 mm Ø		Spacing	Minimum	200	mm
				Maximum	200	mm
			Cover	Minimum	29	mm
				Maximum	29	mm
	Transverse Rebar Size: 16mm Ø		Spacing	Minimum	150	mm
				Maximum	150	mm
			Cover	Minimum	28	mm
Maximum				28	mm	
Infrared Thermal Image Test						
Span 2 - Deck Slab	Carbonation Test		Degree of Damage	II		
	Un-Carbonated Depth			0	mm	
	Ultrasonic Pulse Velocity Test		Degree of Damage	IV		
	Depth of Cracking			200	mm	
	Half Cell Electrical Potential Test		Degree of Damage	II		
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	I		
	Compressive Strength			15.47	MPa	
	Rebar Detection Test (Magnetic Type)					
	Longitudinal Rebar Size: 12mm Ø		Spacing	Minimum	377	mm
				Maximum	377	mm
			Cover	Minimum	31	mm
				Maximum	31	mm
	Transverse Rebar Size: 16mm Ø		Spacing	Minimum	147	mm
				Maximum	157	mm
			Cover	Minimum	36	mm
Maximum				36	mm	
Infrared Thermal Image Test						

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C ≥ 40mm)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

Inspection Record (Summary Sheet for Concrete Elements 2/4)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR
Bridge Element	Span	Attribute	Main Member (Girder)		
Span 1 - Girder 2	Carbonation Test		Degree of Damage	I	
	Un-Carbonated Depth		33 mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage	IV	
	Depth of Cracking		682.69 mm		
	Half Cell Test		Degree of Damage	I	
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	I	
	Compressive Strength		18.9 MPa		
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing	Minimum		mm
			Maximum		mm
	Size: _____ mm Ø	Cover	Minimum		mm
			Maximum		mm
	Stirrups	Spacing	Minimum	300	mm
			Maximum	200	mm
Size: _____ mm Ø	Cover	Minimum	68	mm	
		Maximum	68	mm	
Infrared Thermal Image Test					
Span 2 - Girder 2	Carbonation Test		Degree of Damage	II	
	Un-Carbonated Depth		50 mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage	IV	
	Depth of Cracking		327.6 mm		
	Half Cell Electrical Potential Test		Degree of Damage	I	
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	II	
	Compressive Strength		14.8 MPa		
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing (mm)	Minimum	410	mm
			Maximum	410	mm
	Size: _____ mm Ø	Cover (mm)	Minimum	62	mm
			Maximum	63	mm
	Stirrups	Spacing (mm)	Minimum	385	mm
			Maximum	435	mm
Size: _____ mm Ø	Cover (mm)	Minimum	64	mm	
		Maximum	70	mm	
Infrared Thermal Image Test					
Span 3 - Girder 1	Carbonation Test (Dry Run)		Degree of Damage	II	
	Un-Carbonated Depth		14 mm		
	Ultrasonic Pulse Velocity Test		Degree of Damage	IV	
	Depth of Cracking		685 mm		
	Half Cell Electrical Potential Test		Degree of Damage	None	
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	II	
	Compressive Strength		17.1 MPa		
	Rebar Detection Test (Radar Type)				
	Longitudinal Rebar	Spacing	Minimum		mm
			Maximum		mm
	Size: _____ mm Ø	Cover	Minimum		mm
			Maximum		mm
	Stirrups	Spacing	Minimum	200	mm
			Maximum	300	mm
Size: _____ mm Ø	Cover	Minimum	68	mm	
		Maximum	68	mm	
Infrared Thermal Image Test					

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C \geq 40mm)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

Inspection Record (Summary Sheet for Concrete Elements 3/4)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	FAIR	
Bridge Element	Pier	Attribute	Main Structure (Wall)			
Pier 1 - Main Structure (Wall)	Carbonation Test		Degree of Damage	I		
	Un-Carbonated Depth		82 mm			
	Ultrasonic Pulse Velocity Test		Degree of Damage			
	Depth of Cracking		mm			
	Half Cell Electrical Potential Test		Degree of Damage	IV		
	<input checked="" type="checkbox"/> Rebound Hammer Test / <input type="checkbox"/> Compressive Strength Test		Degree of Damage	II		
	Compressive Strength		15.89 MPa			
	Rebar Detection Test (Radar Type)					
	Horizontal Rebar Size: 20 mm Ø		Spacing	Minimum	500 mm	
				Maximum	500 mm	
	Vertical Rebar Size: 20mm Ø		Cover	Minimum	60 mm	
				Maximum	60 mm	
			Spacing	Minimum	500 mm	
				Maximum	500 mm	
			Cover	Minimum	80 mm	
Maximum				80 mm		
Infrared Thermal Image Test			With extensive delamination			

Carbonation Test Degree of Damage	Un-Carbonated Depth (D)
I	$D \geq 25\text{mm}$
II	$D < 25\text{mm}$

Ultrasonic Pulse Velocity Test Degree of Damage	Crack Depth (D in mm)
I	$C/2 > D$; (C=Conc. cover)
II	$C > D \geq C/2$ (C ≥ 40mm)
III	$C > D \geq C/2$ (C < 40mm)
IV	$D \geq C$

Half Cell Test Degree of Damage	Potential Difference Levels (mV) <E>
I	$-200 < E$
II	$-200 \leq E \leq -350$
III	$-350 < E < -500$
IV	$E \leq -500$

Rebound Hammer Test/ Compressive Strength Test Degree of Damage	F _c = Design compressive strength F = Actual compressive strength (in MPa)
I	$F \geq F_c$
II	$0.8F_c < F < F_c$
III	$F < 0.8F_c$

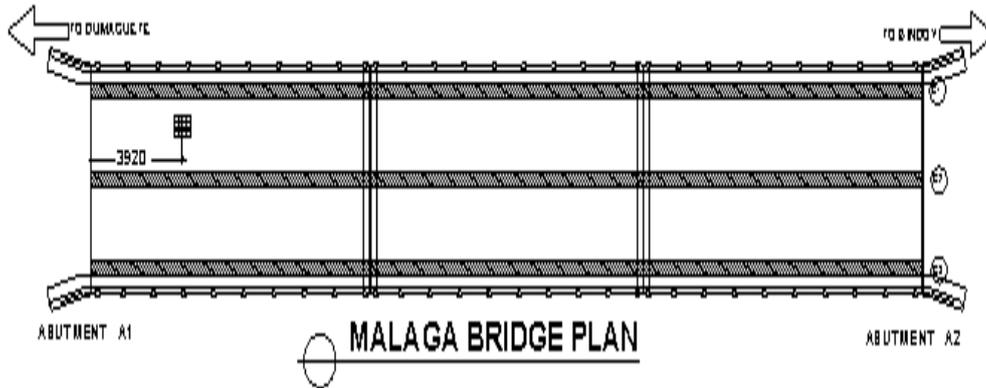
Inspection Records

Span 1 - Deck Slab

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date:	29-Nov-12
		Weather	FAIR

Sketches of Inspected Location : Span 1 - Deck Slab (Between G1 & G2)



Inspection Result

Test Point No. :	1	Actual Compressive Strength :	17.0 MPa
Bridge Element/Attribute :	Span 1/Deck Slab	Number of Test :	

Test Results

52	51.2	45.7	56.5
38.7	47.8	48.6	44.4
47.7	48.7	52.1	56.5
47.2	47.5	50.4	48.9
41.7	48.1	48.5	50

Standard Deviation : 4.3
Average Reading : 48.8 MPa

48.8×0.85 (factor from cube to cylinder strength) = 41.48
 41.48×0.41 (time factor) = 17.0 MPa

Inspection Record (Rebound Hammer Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records :Span 1 - Deck Slab (Between G1 & G2)

Photo-1 : General View	Photo-2 : Test Point
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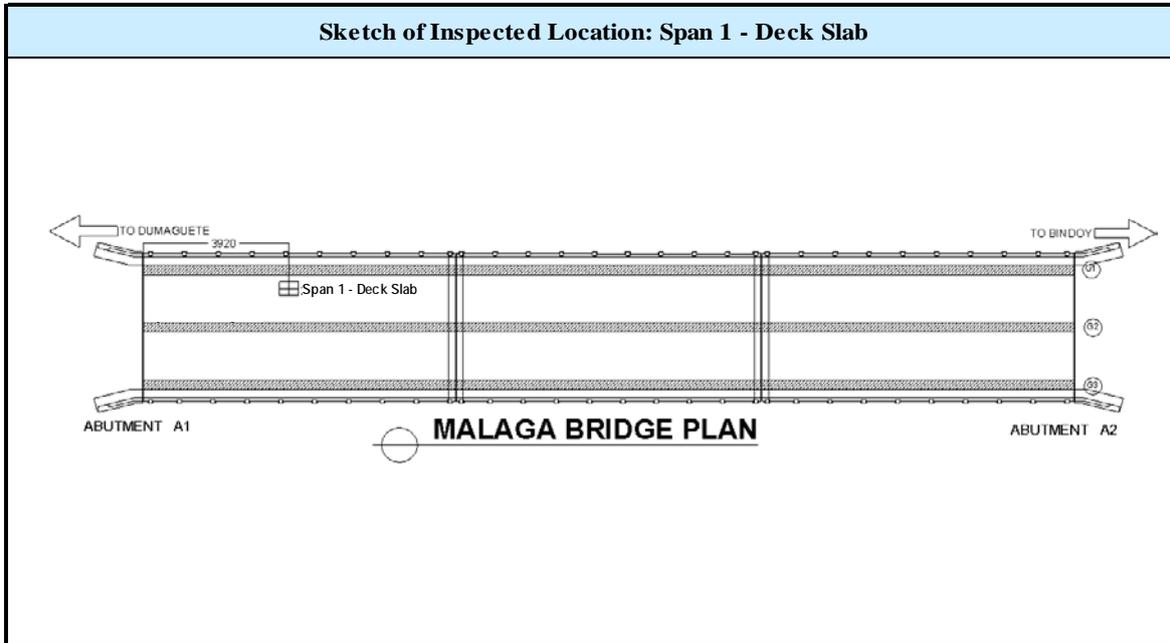


Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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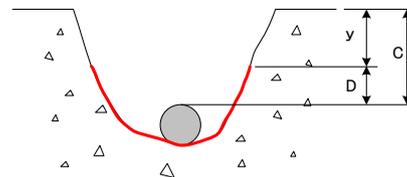
Inspection Record (Carbonation Test / Chipping-off 1/2)

Bridge Name:	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR



Inspection Result

Construction Year:	1957
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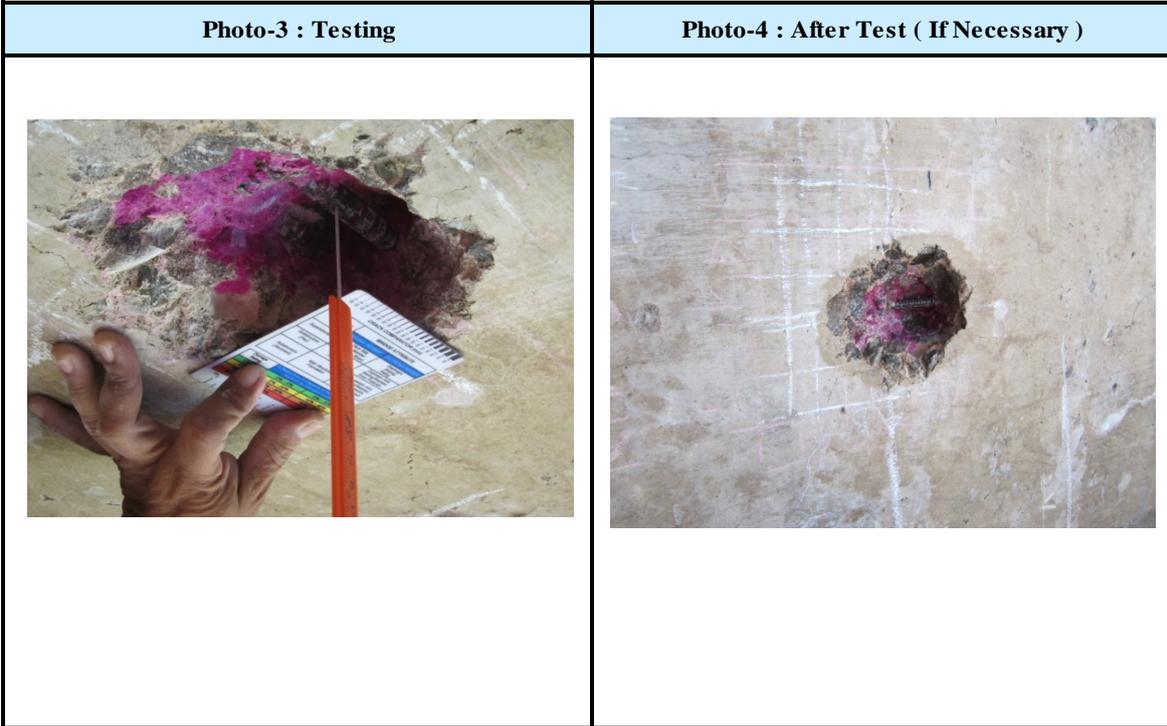
Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
1	Span 1 - Deck Slab	50	30	20

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 - Deck Slab

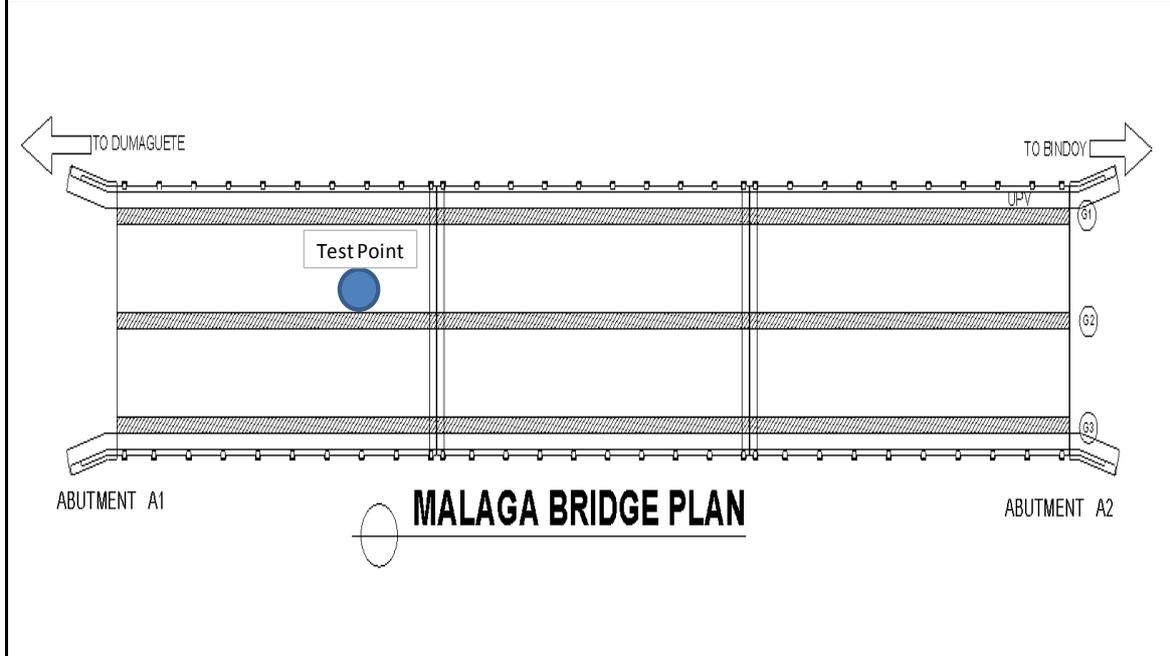
Photo-1 : General View	Photo-2 : Test Point
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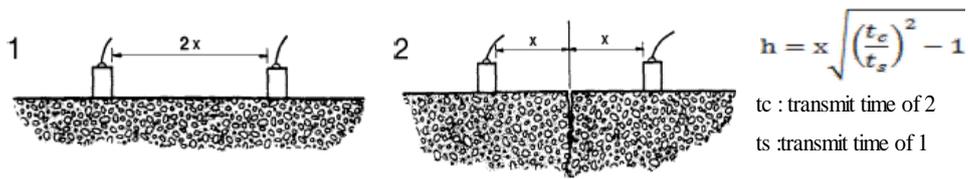
Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Inspected Location (Sketches : Span 1 - Girder 1 Upstream Side)



Inspection Result



Test Point No.	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2 t_c (ms)	Transmit time of 1 t_s (ms)	Crack Depth h (mm)
1	S1 - Deck Slab between G1&G2	50	145	67.6	95
2	S1 - Deck Slab between G1&G2	50	96	67.6	50
3	S1 - Deck Slab between G1&G2	50	104	67.6	58

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

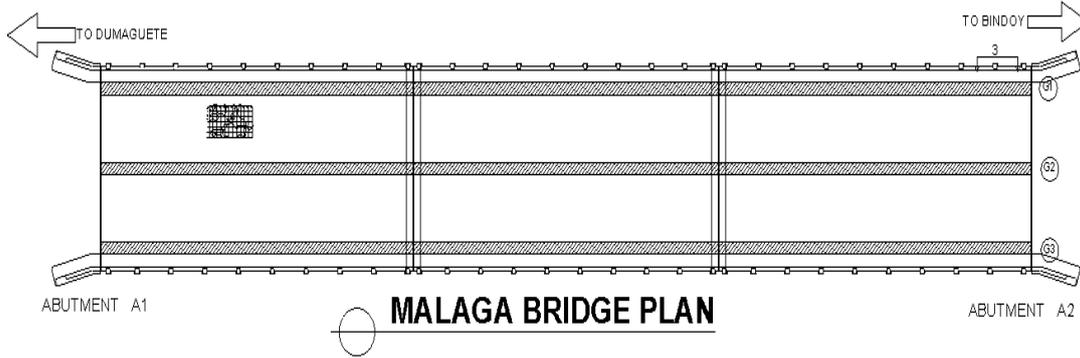
Photo Records : Span 1 -Girder 1

Photo-1 : General View	Photo-2 : Test Point
	
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	Empty space for Photo-4

Inspection Record (Half-Cell Electrical Potential Test 1/2)

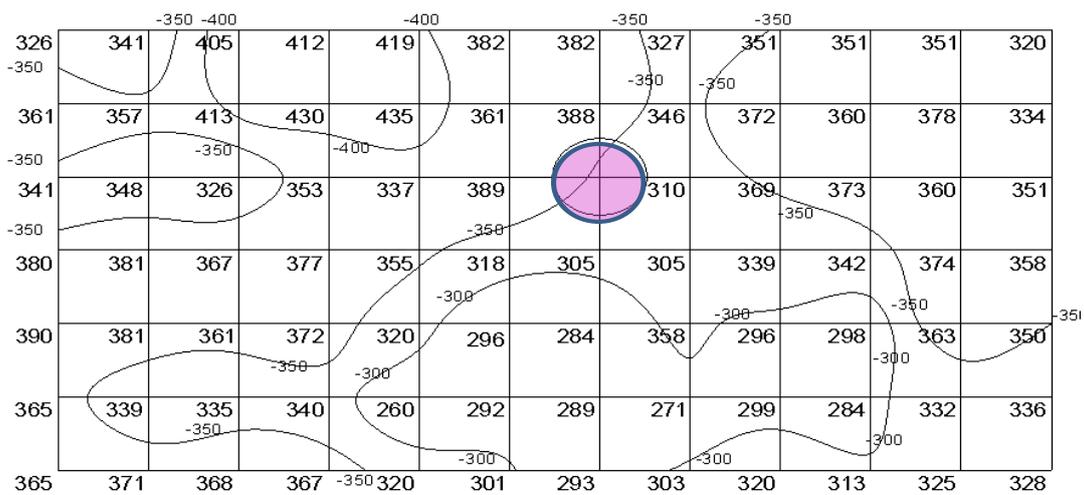
Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Sketch of Inspected Location: Span 1 - Deck Slab (Between G1 & G2)



Inspection Result

Test Point No. 1
Equipotential Contour Map



Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

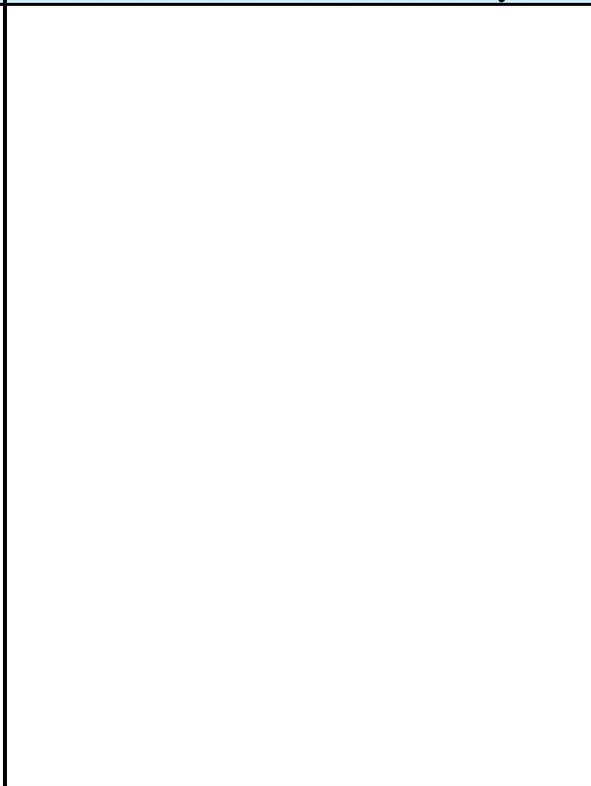
Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 - Deck Slab Between G1 & G2

Photo-1 : General View	Photo-2 : Test Point
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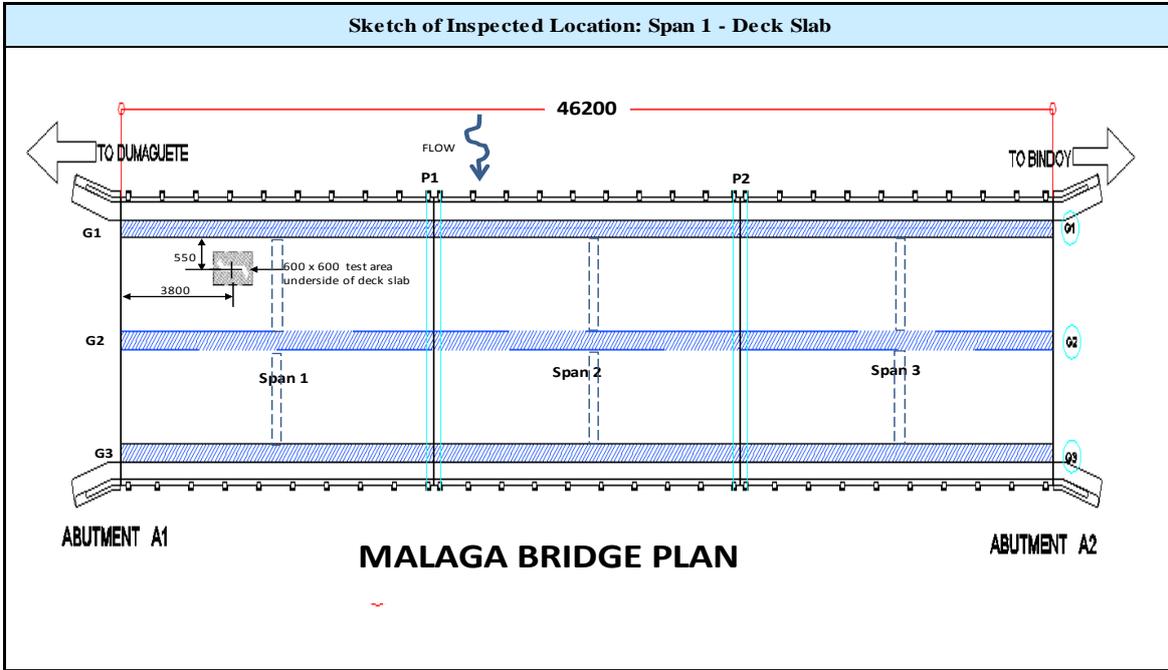


Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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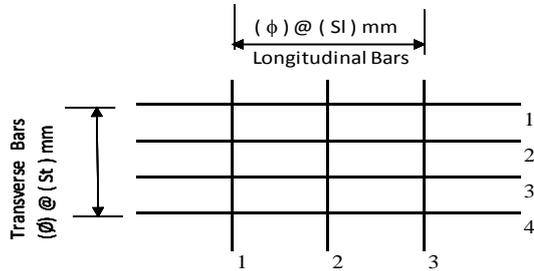


Inspection Record (Rebar Detection Test / Magnetic Type 1/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR



Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Longitudinal Bars			Transverse Bars		
			Rebar Dia. (mm)	Spacing (mm)	Cover (mm)	Rebar Dia. (mm)	Spacing (mm)	Cover (mm)
Transverse	1	Span 1 - Deck Slab				16	150	28
	2		16	150	28			
	3		16	150	28			
	4		16	150	28			
Longitudinal	1	Span 1 - Deck Slab	12	200	29			
	2		12	200	29			
	3		12	200	29			

Inspection Record (Rebar Detection Test / Magnetic Type 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 - Deck Slab

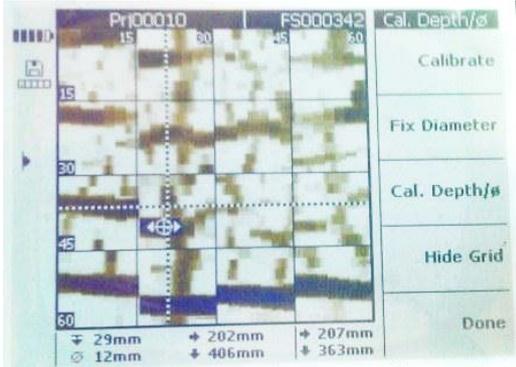
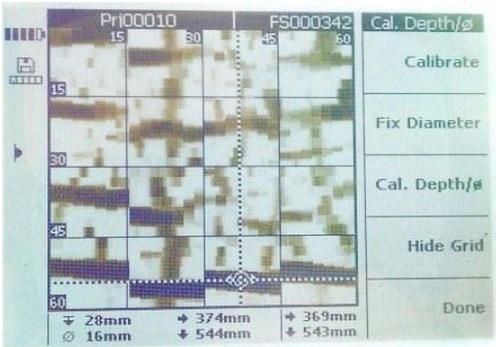
Photo-1 : General View	Photo-2 : Test Point
	

Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	

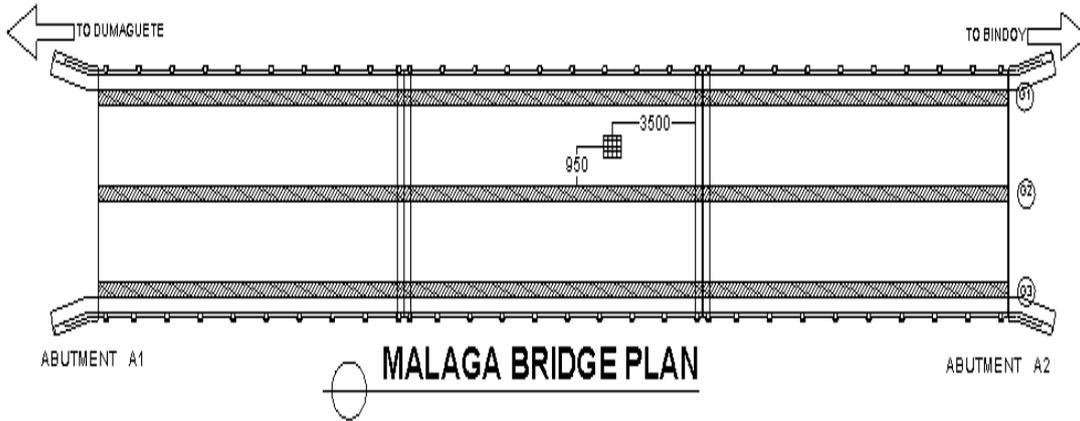
Inspection Records

Span 2 – Deck Slab

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date:	29-Nov-12
		Weather	FAIR

Sketches of Inspected Location: Span 2 - Deck Slab Between G1 & G2



Inspection Result

Test Point No. :	3	Actual Compressive Strength :	17.6 MPa
Bridge Element/Attribute :	Span 2 - Deck Slab	Number of Test :	

Test Results

51	48	46	52
52	52	52	54
50	50	52	54
48	48	50	49
53	51	46	50

Standard Deviation : No data
Average Reading : 50.4 MPa

50.4×0.85 (factor from cube to cylinder strength) = 42.84
 42.84×0.41 (time factor) = 17.56MPa

Inspection Record (Rebound Hammer Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 2 - Deck Slab Between G1 & G2	
Photo-1 : General View	Photo-2 : Test Point

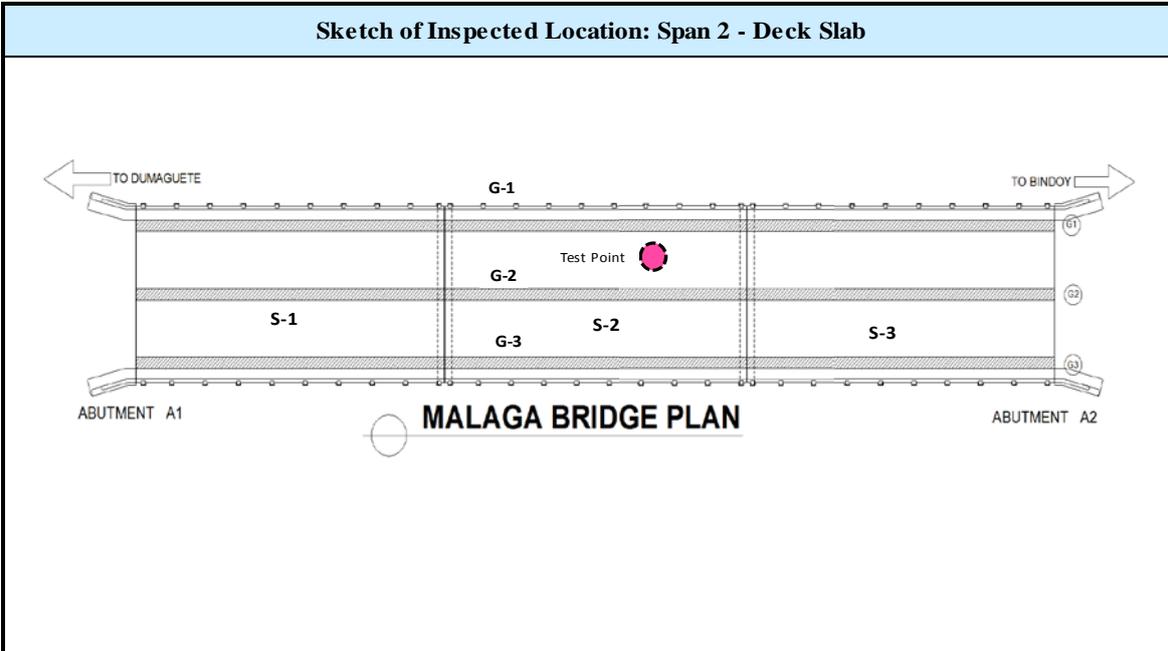


Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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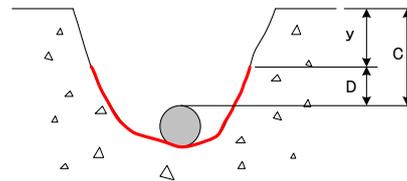
Inspection Record (Carbonation Test / Chipping-off 1/2)

Bridge Name:	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	CLOUDY



Inspection Result

Construction Year:	1957
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Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
1	Span 2 - Deck Slab	30	40	0

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13
		Weather	CLOUDY

Photo Records : Span 2 - Deck Slab

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing

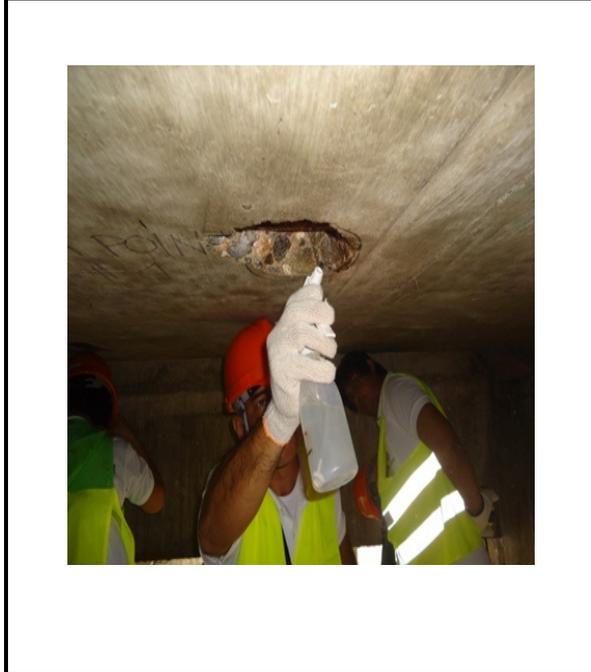


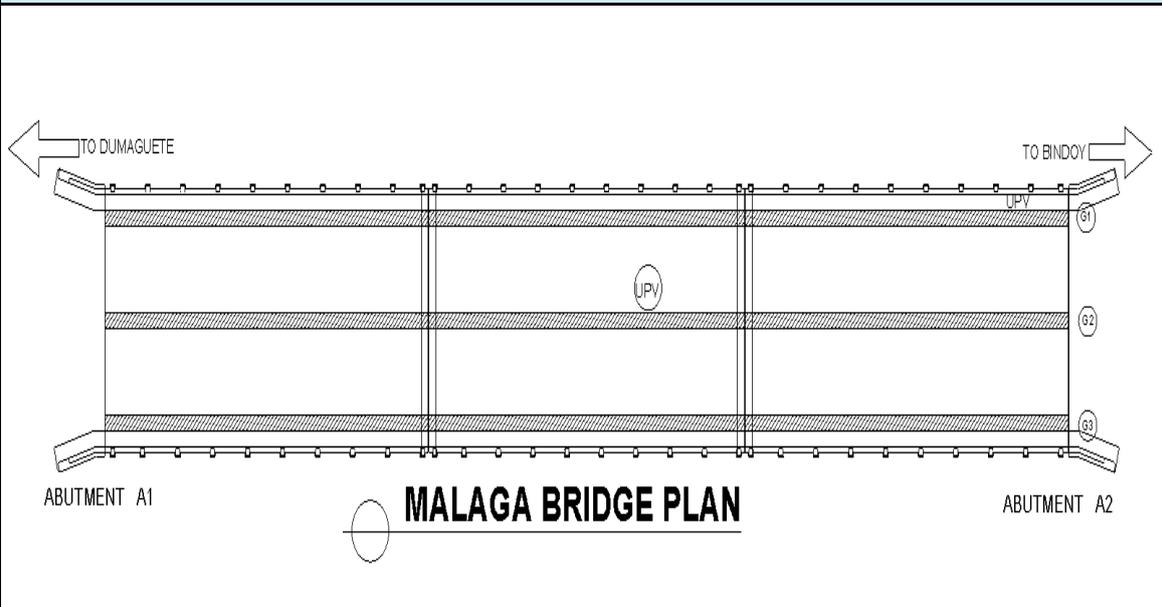
Photo-4 : After Test (If Necessary)



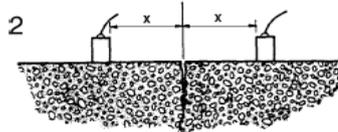
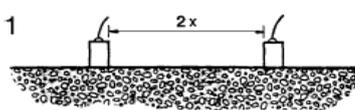
Inspection Record (Ultrasonic Pulse Velocity Test / Crack Dept 1/2)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Sketch of Inspected Location : Span 2-Deck Slab



Inspection Result



$$h = x \sqrt{\left(\frac{t_c}{t_s}\right)^2 - 1}$$

t_c : transmit time of 2

t_s :transmit time of 1

Test Point No.	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2 t_c (µs)	Transmit time of 1 t_s (µs)	Crack Depth h (mm)
1	S2 - Deck Slab between G1&G2	50	359	41.9	425.5
2	S2 - Deck Slab between G1&G2	50	165.4	41.9	190.40
3	S2 - Deck Slab between G1&G2	50	692	41.9	824.3

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR		
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Photo Records: Span 2 - Deck Slab

Photo 1- general View	Photo-2 : Test Point
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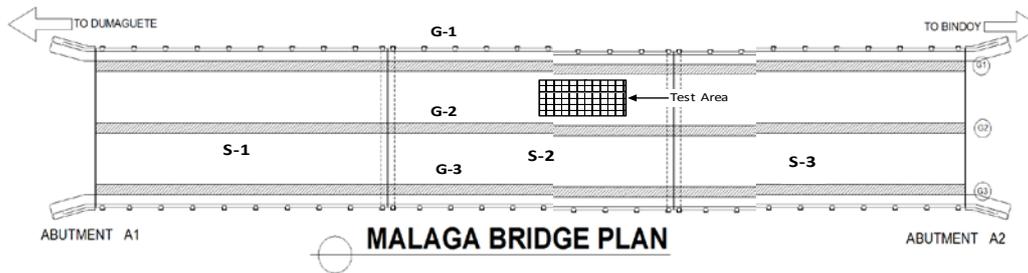
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Half-Cell Electrical Potential Test 1/2)

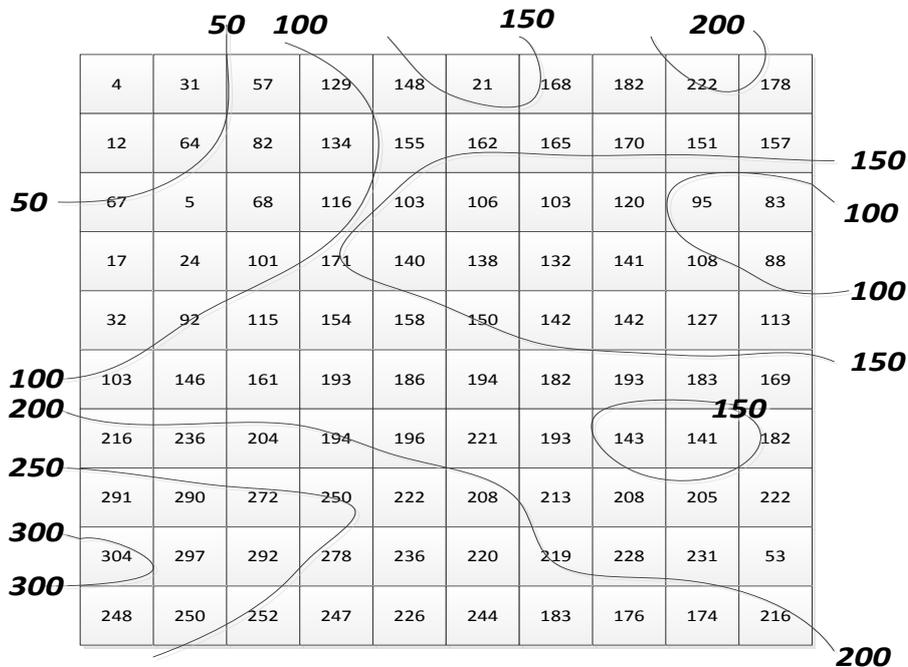
Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	CLOUDY

Sketch of Inspected Location: Span 2 - Deck Slab



Inspection Result

Test Point No. 1
Equi-potential Contour Map



Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13
		Weather	CLOUDY

Photo Records : Span 2 - Deck Slab

Photo-1 : General View



Photo-2 : Test Point



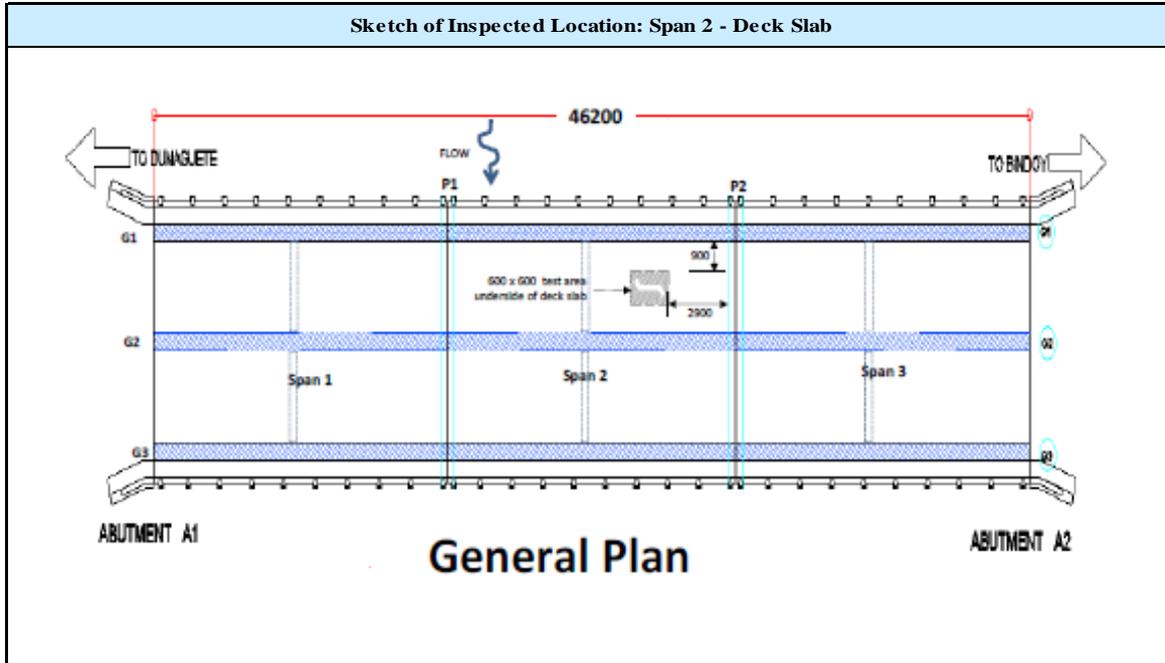
Photo-3 : Testing



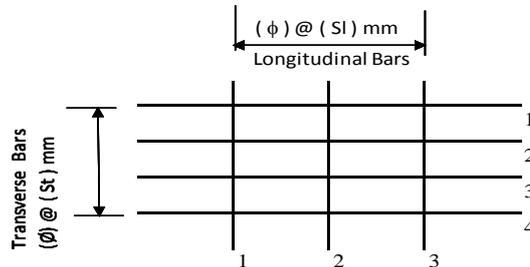
Photo-4 : After Test (If Necessary)

Inspection Record (Rebar Detection Test / Magnetic Type 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	30-Jan-13
		Weather	FAIR



Inspection Result

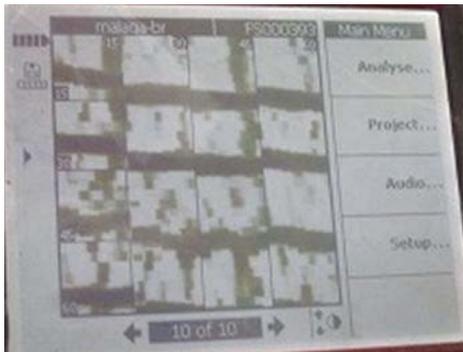


Direction	Bar No.	Bridge Element/Attribute	Longitudinal Bars			Transverse Bars		
			Rebar Dia. (mm)	Spacing (mm)	Cover (mm)	Rebar Dia. (mm)	Spacing (mm)	Cover (mm)
Transverse	1	Span 2- Deck Slab	-	-	-	16	-	36
	2		-	-	-	16	144	36
	3		-	-	-	16	157	36
	4		-	-	-	-	-	-
Longitudinal	1	Span 2 - Deck Slab	12	-	31	-	-	-
	2		12	377	31	-	-	-
	3		12	377	31	-	-	-

Inspection Record (Rebar Detection Test / Magnetic Type 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	30-Jan-13
		Weather	FAIR

Photo Records : Span 2 - Deck Slab

Photo-1 : General View	Photo-2 : Test Point
	
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	

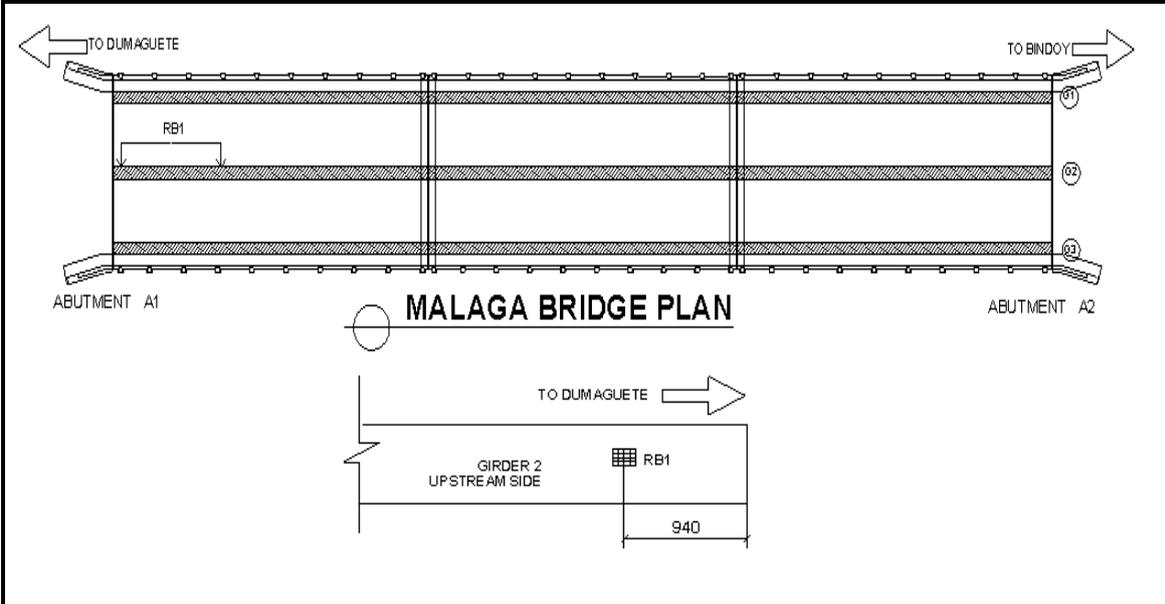
Inspection Records

Span 1 – Girders

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date:	29-Nov-12
		Weather	FAIR

Sketches of Inspected Location : Span 1 - Girder 2 (Upstream Side)



Inspection Result

Test Point No. :	2	Actual Compressive Strength :	18.9 MPa
Bridge Element/Attribute : Span 1 - Girder 2		Number of Test :	

Test Results

54.2	54.8	51.1	51.5
54.1	54.6	53.6	50.5
56.4	60.2	54.1	56
51.5	52.8	53.1	56.3
56.2	55.2	60.5	54.8

Standard Deviation : 2.1

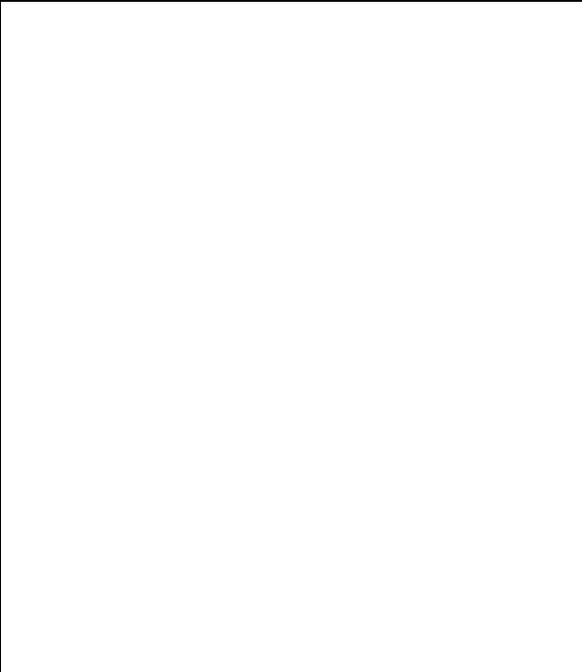
Average Reading : 54.5 MPa

54.5×0.85 (factor from cube to cylinder strength) = 46.30

46.30×0.41 (time factor) = 18.9MPa

Inspection Record (Rebound Hammer Test 2/2)

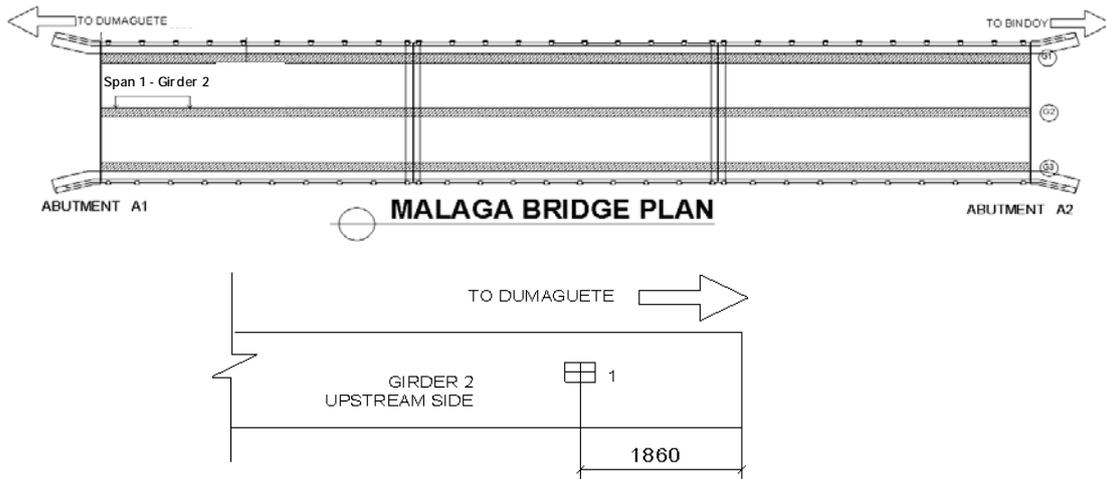
Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date 29-Nov-12	Weather FAIR

Photo Records : Span 1 - Girder 2 (Upstream Side)	
Photo-1 : General View	Photo-2 : Test Point
	
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	

Inspection Record (Carbonation Test / Chipping-off 1/2)

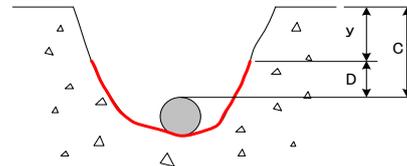
Bridge Name:	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Sketch of Inspected Location: Span 1 - Girder 2



Inspection Result

Construction Year: 1957



Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
2	Span 1 - Girder 2	68	35	33

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 - Girder 2

Photo-1 : General View	Photo-2 : Test Point
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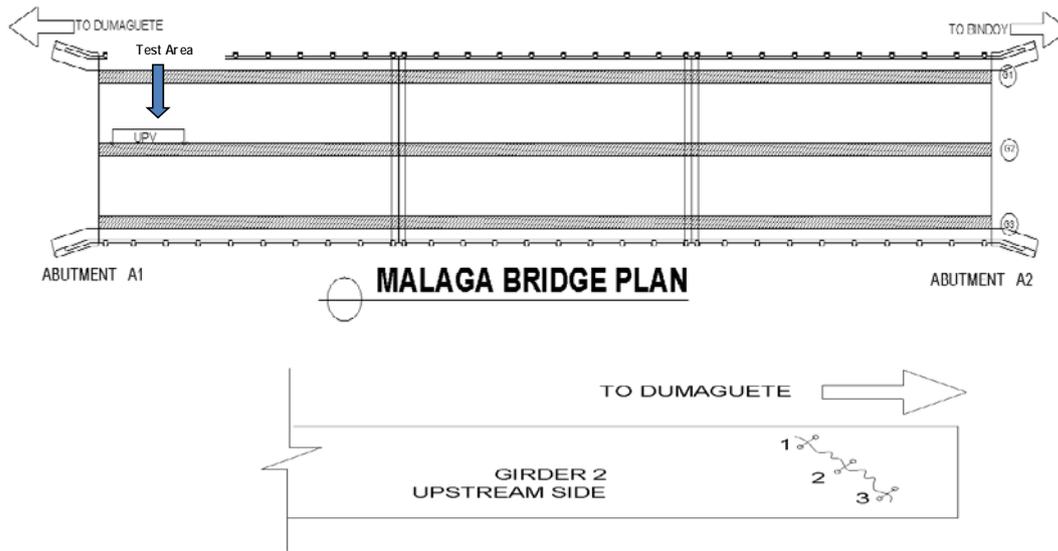
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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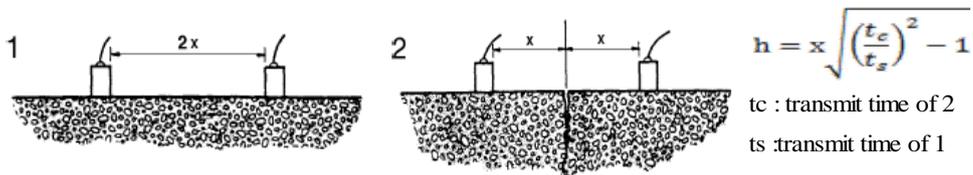
Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Inspected Location (Sketches : Span 1 - Girder 2 (Upstream Side))



Inspection Result



Test Point No.	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2 t_c (µs)	Transmit time of 1 t_s (µs)	Crack Depth h (mm)
1	S1 - G2 Upstream Side	50	559.3	50.4	552.6
2	S1 - G2 Upstream Side	50	690	50.4	682.69
3	S1 - G2 Upstream Side	50	360.4	50.4	354.2

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 -Girder 2

Photo-1 : General View	Photo-2 : Test Point
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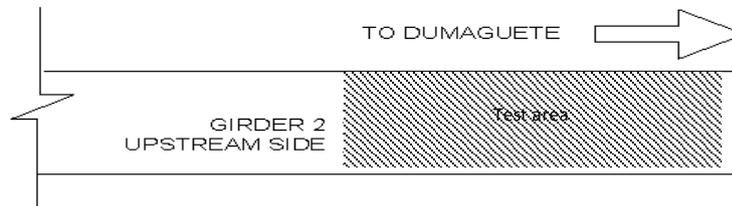
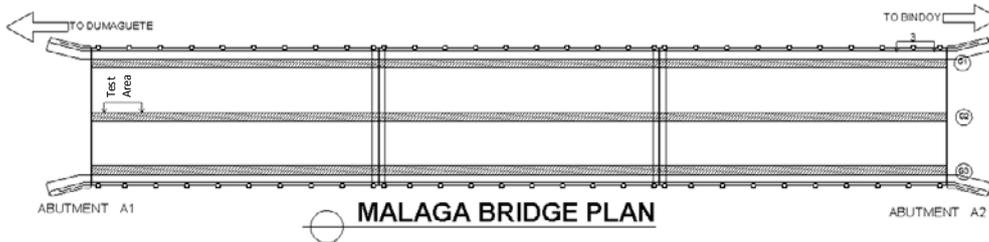
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Half-Cell Electrical Potential Test 1/2)

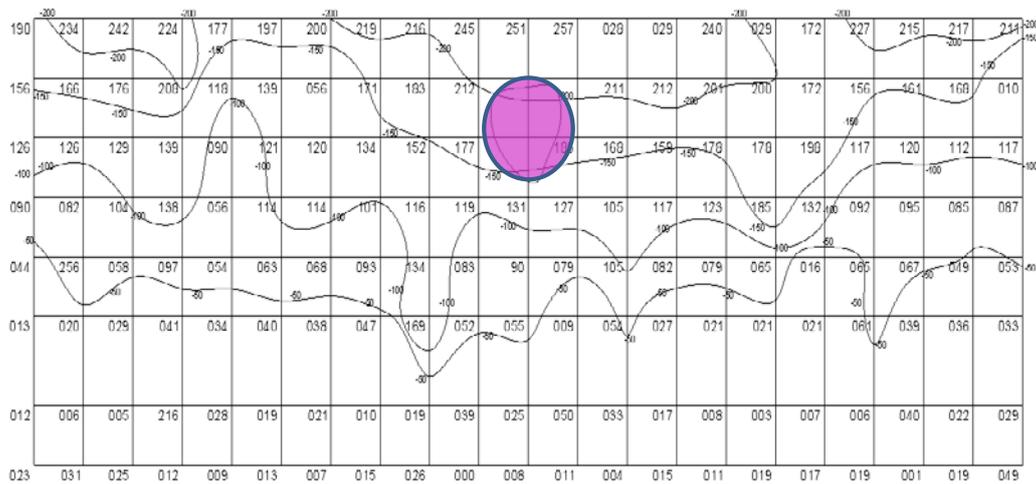
Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather
			FAIR	

Sketch of Inspected Location: Span 1 - Girder 2 (Upstream Side)



Inspection Result

Test Point No. 1
Equi-potential Contour Map



Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 1 - Girder 2 (Upstream Side)

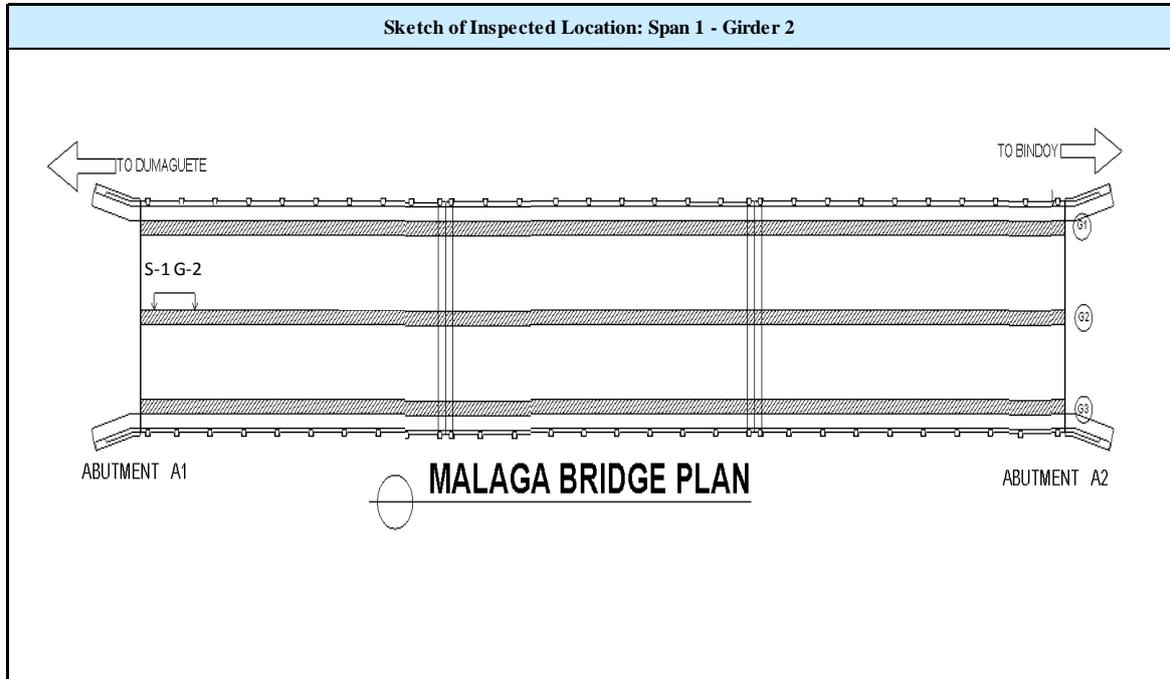
Photo-1 : General View	Photo-2 : Test Point
	

Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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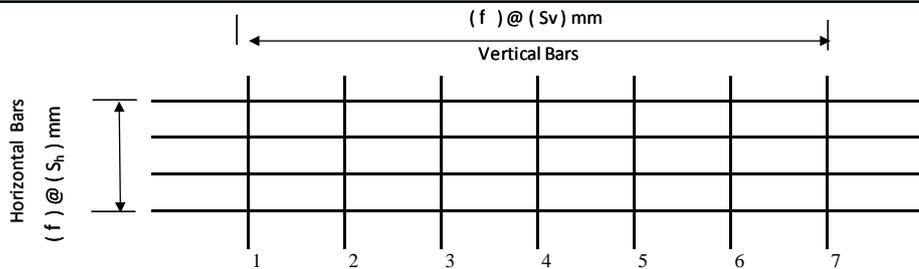
	
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Inspection Record (Rebar Detection Test / Radar Type 1/2)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR



Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Vertical Bars		Horizontal Bars	
			Spacing (mm)	Cover (mm)	Spacing (mm)	Cover (mm)
Vertical Bars	Stirup	Span 1-Girder 2	250/400/ 450	68		

Inspection Record (Rebar Detection Test / Radar Type 2/2)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Photo Records : Span 1 - Girder 2	
Photo-1 : General View	Photo-2 : Test Point
(No Picture)	(No Picture)
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
(No Picture)	(No Picture)

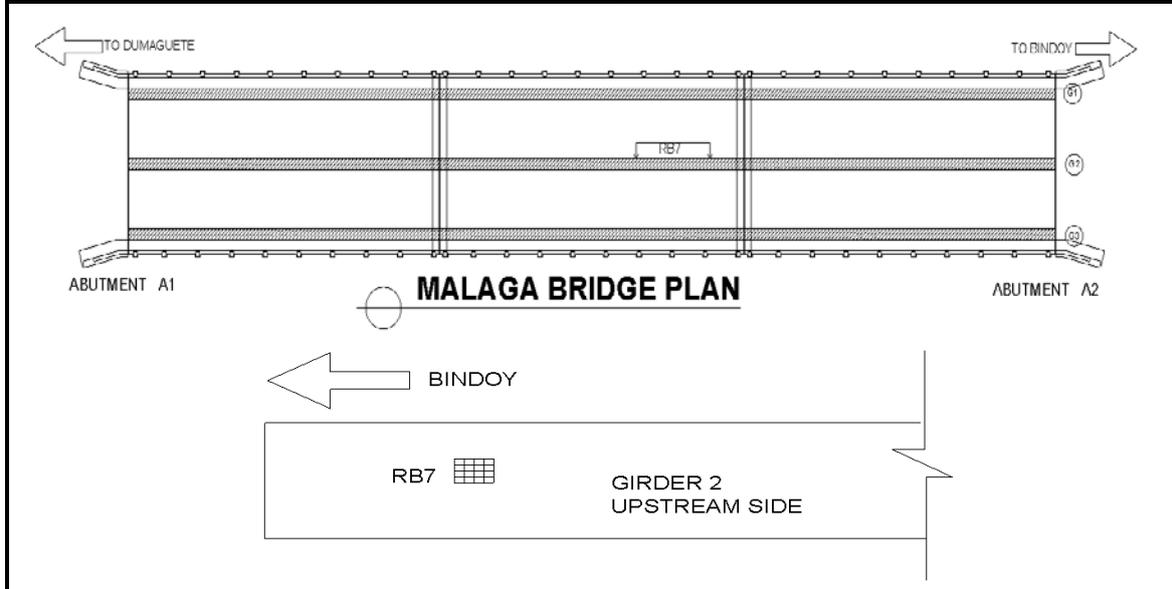
Inspection Records

Span 2 – Girder

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date:	29-Nov-12
		Weather	FAIR

Sketches of Inspected Location : Span 2 - Girder 2 Upstream side



Inspection Result

Test Point No. :	4	Actual Compressive Strength :	14.8 Mpa
Bridge Element/Attribute :	Span 2/Girder 2	Number of Test :	

Test Results

44.6	44.5	48.1	36
23.6	45.1	45.2	43.5
39.8	38.8	47	42.3
47.9	46.9	43.4	40.8
36.1	41.4	36.7	18.9

Standard Deviation : 7.7
Average Reading : 42.6 MPa

42.6 x 0.85 (factor from cube to cylinder strength) = 36.2
36.20 x 0.41 (time factor) = 14.8 MPa

Disregard the 2 readings marked X
Total of remaining 18 values = 767.2
Average reading = 767.2/18 = 42.60

Inspection Record (Rebound Hammer Test 2/2)

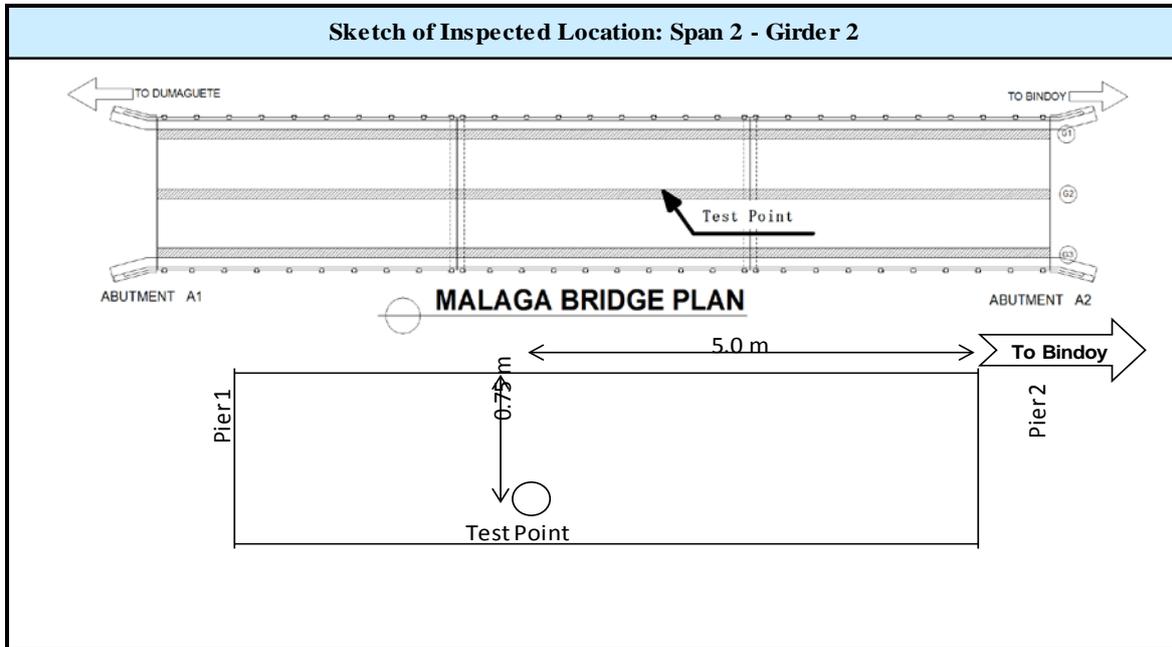
Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Span 2 - Girder 2 Upstream side

Photo-1 : General View	Photo-2 : Test Point
	
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	

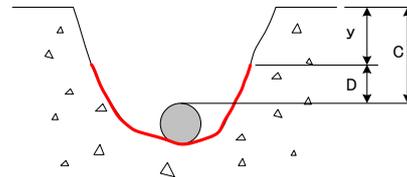
Inspection Record (Carbonation Test / Chipping-off 1/2)

Bridge Name:	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	CLOUDY



Inspection Result

Construction Year:	1957
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Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
2	Span 2 - Girder 2	70	20	50

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13
		Weather	CLOUDY

Photo Records : Span 2 - Girder 2

Photo-1 : General View	Photo-2 : Test Point
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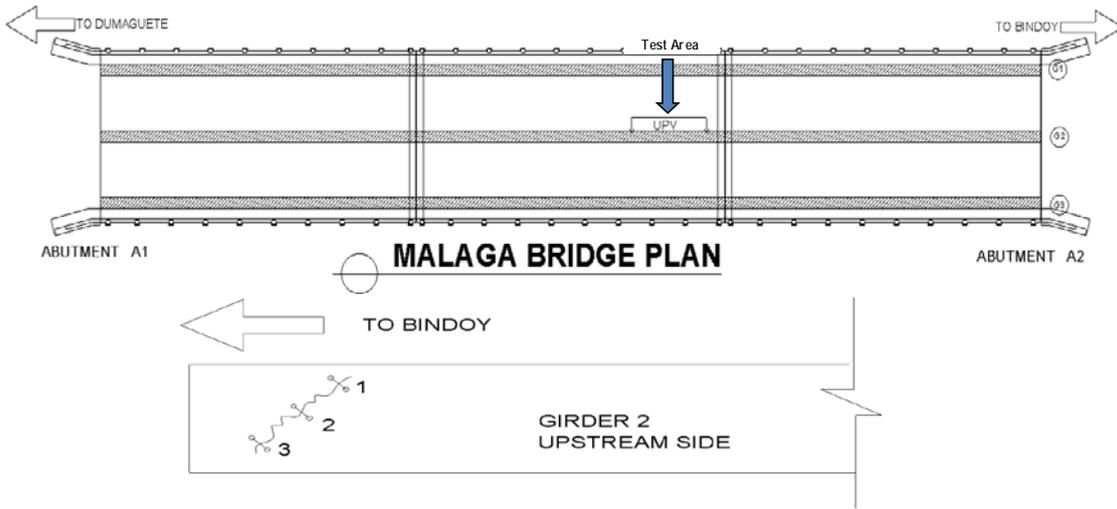
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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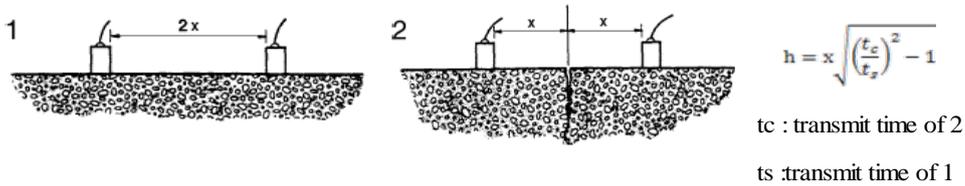
Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Sketch of Inspected Location: Span 2 - Girder 2 (Upstream Side)



Inspection Result



Test Point	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2	Transmit time of 1	Crack Depth
1	S2 - G2 Upstream side	50	296.6	50.7	288.2
2	S2 - G2 Upstream side	50	336	50.7	327.60
3	S2 - G2 Upstream side	50	208	50.7	198.1

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Photo Records : Span 2 - Girder 2 (Upstream Side)

Photo-1 : General View



Photo-2 : Test Point



Photo-3 : Testing

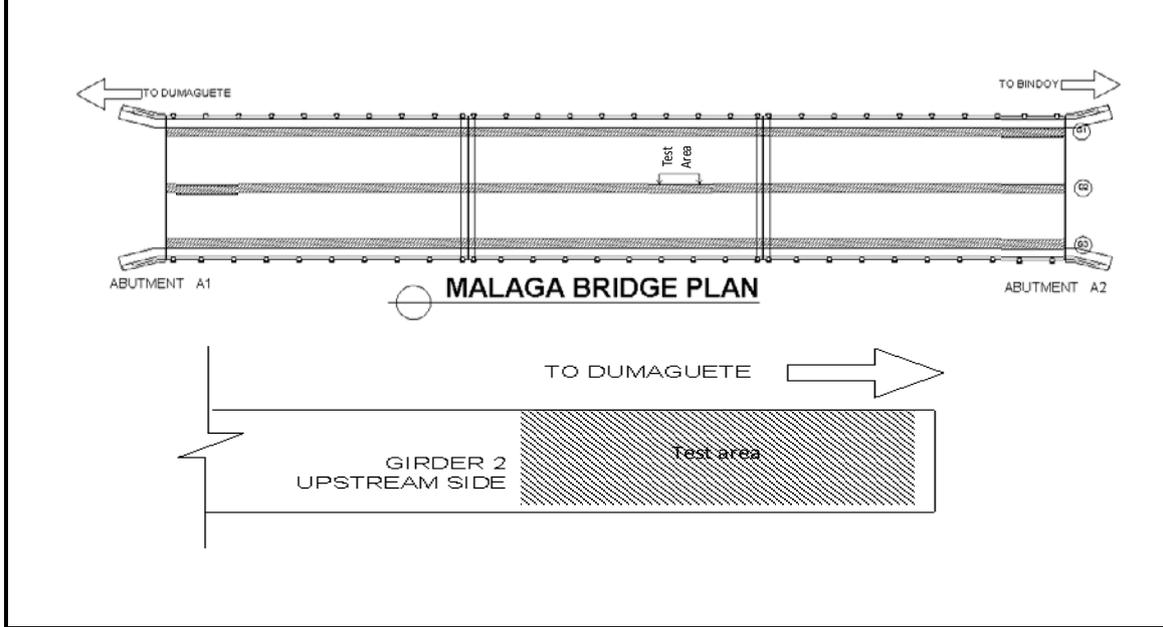


Photo-4 : After Test (If Necessary)

Inspection Record (Half-Cell Electrical Potential Test 1/2)

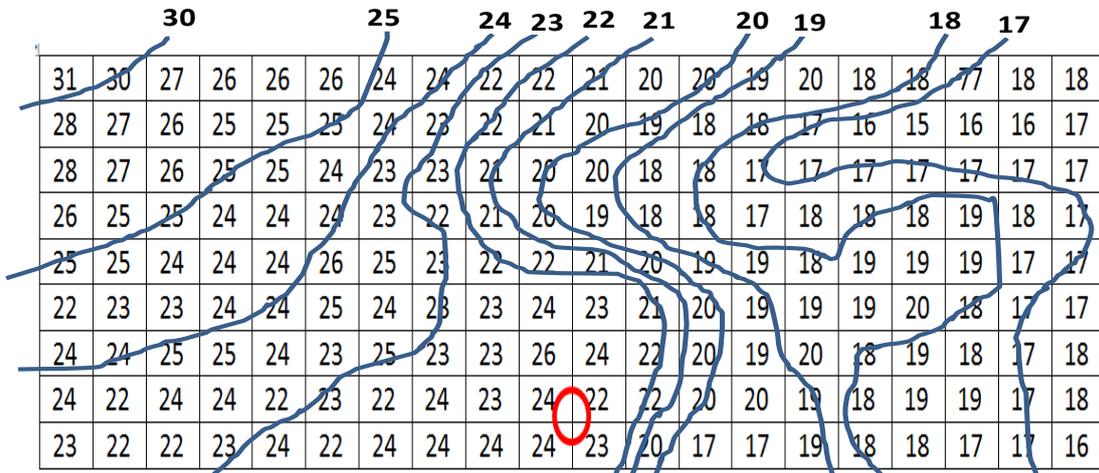
Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13	Weather	CLOUDY

Sketch of Inspected Location: Span 2 - Girder 2



Inspection Result

Test Point No. 1
Equi-potential Contour Map



Note: No probability of rebar corrosion

Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	20-Feb-13
		Weather	CLOUDY

Photo Records : Span 2 - Girder 2

Photo-1 : General View



Photo-2 : Test Point



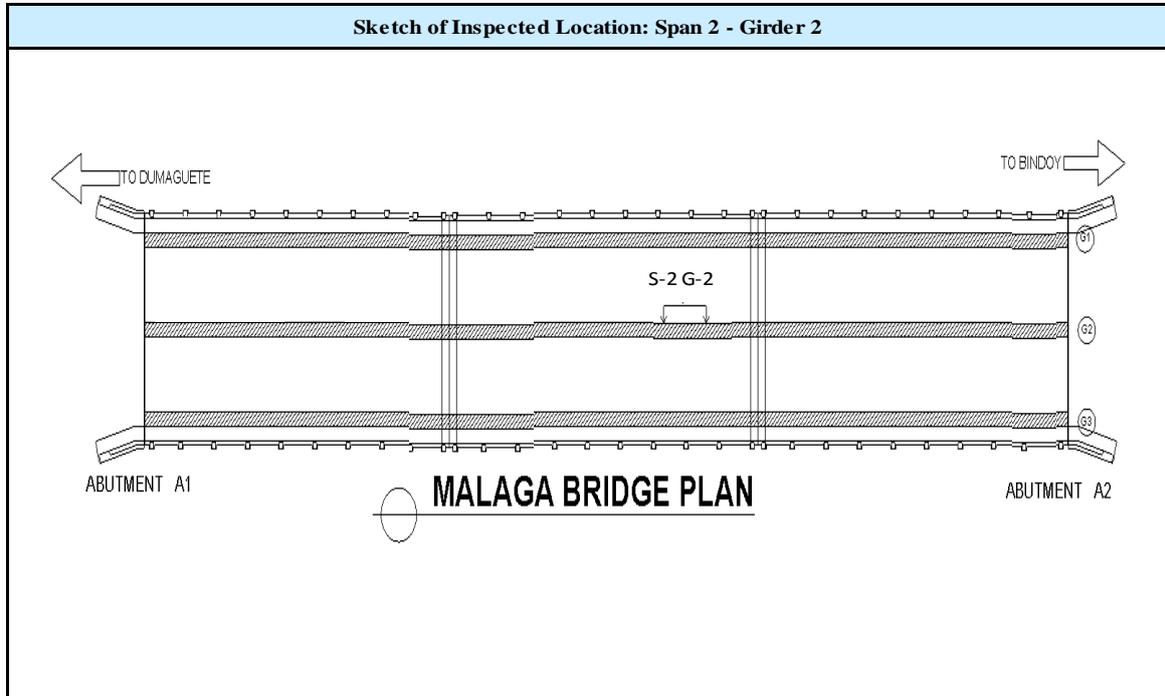
Photo-3 : Testing



Photo-4 : After Test (If Necessary)

Inspection Record (Rebar Detection Test / Radar Type 1/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13	Weather	FAIR



Inspection Result

$(\phi) @ (S_v)$ mm

Vertical Bars

Direction	Bar No.	Bridge Element/Attribute	Vertical Bars		Horizontal Bars	
			Spacing (mm)	Cover (mm)	Spacing (mm)	Cover (mm)
Vertical	Stirrup	Span 2-Girder 2		64	410	62
				64	410	63
				64		
				70		
				68		

Inspection Record (Rebar Detection Test / Radar Type 2/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13	Weather	FAIR

Photo Records : Span 2 - Girder 2

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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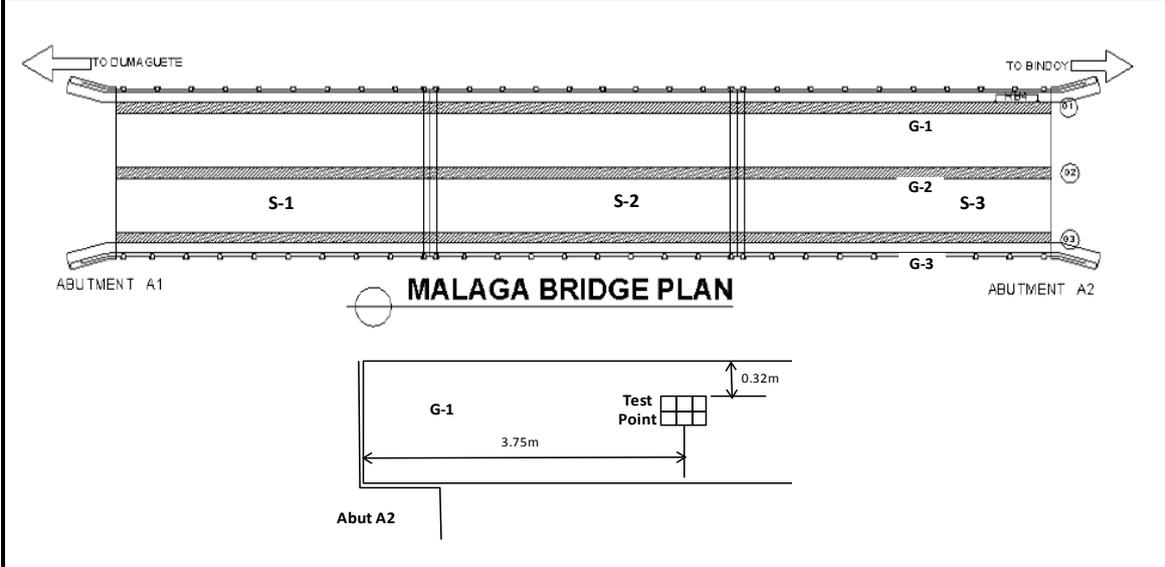
Inspection Records

Span 3 – Girder

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR		
Inspector	FELICIANO R. ESPINA	Date:	30-Jan-13	Weather	FAIR

Sketch of Inspected Location: Span 3 - Girder 1 (Upstream Side)



Inspection Result

Test Point No.	6	Actual Compressive Strength :	17.1 Mpa
Bridge Element/Attribute : Span 3-Girder 1		Number of Test :	1

Test Results

52	50.1	52.2	48
52.2	49.1	52.2	47
48.1	49	50	45.4
51	53	49.2	48
52.1	39.8	49	47

Standard Deviation : No data

Average Reading : 49.3 MPa

49.3×0.85 (factor from cube to cylinder strength) = 41.9

49.1×0.41 (time factor) = 17.1 MPa

Inspection Record (Rebound Hammer Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	30-Jan-13
		Weather	FAIR

Photo Records :Span 3 - Girder 1 (Upstream Side)

Photo-1 : General View	Photo-2 : Test Point
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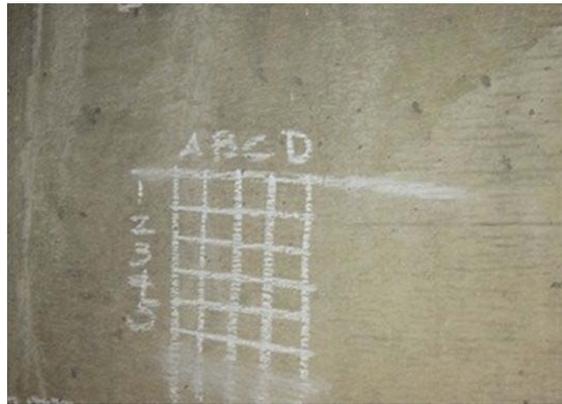


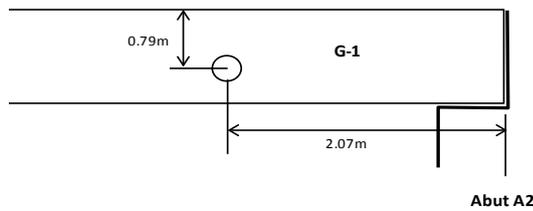
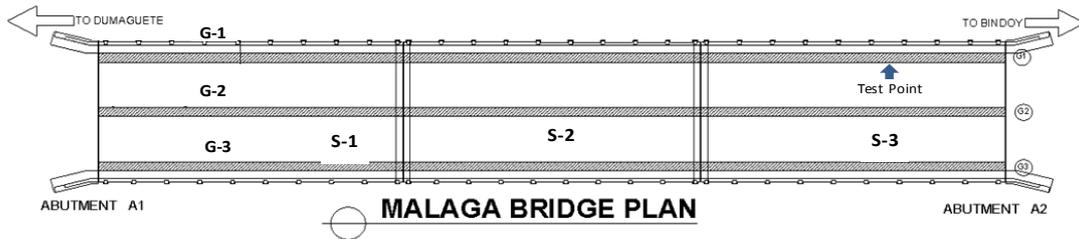
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Carbonation Test / Chipping-off 1/2)

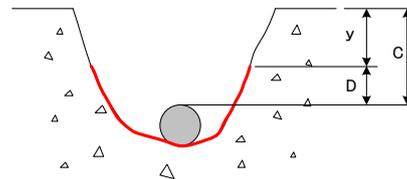
Bridge Name:	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13	Weather	FAIR

Sketch of Inspected Location: Span 3 - Girder 1



Inspection Result

Construction Year:	1957
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Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
1	Span 3 -Girder 1	50	$(31 + 41) / 2 = 36$	14

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13
		Weather	FAIR

Photo Records : Span 3 - Girder 1

Photo-1 : General View	Photo-2 : Test Point
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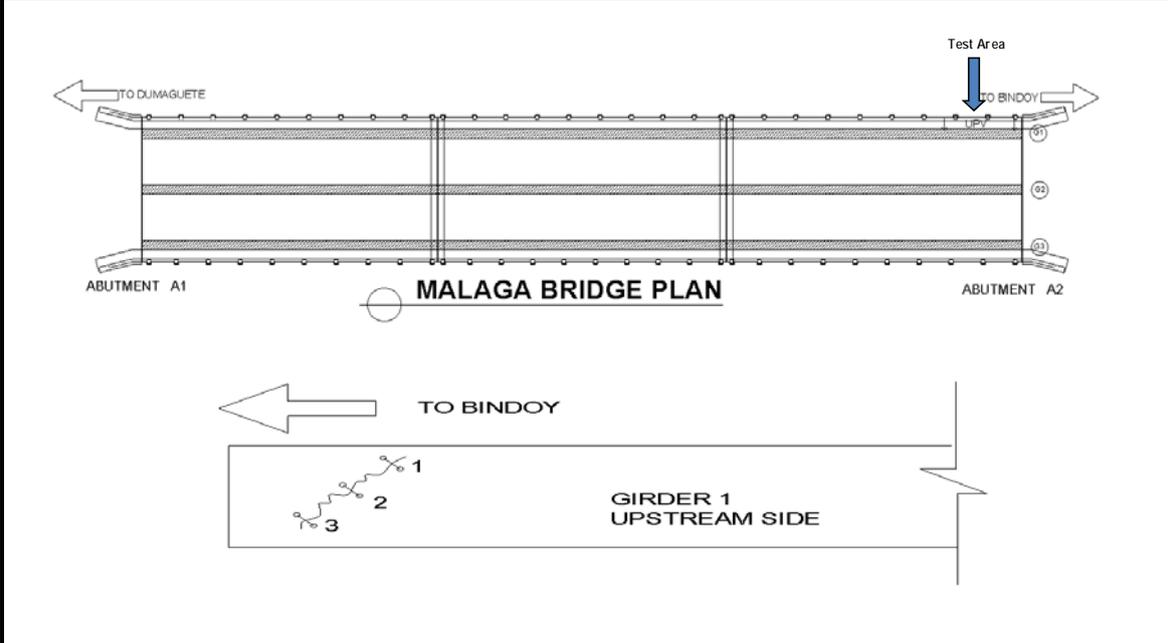
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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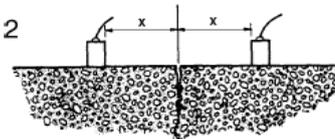
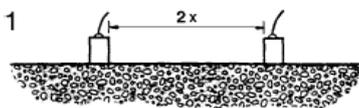
Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 1/2)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	28-Nov-12	Weather	FAIR

Sketch of Inspected Location: Span 3 - Girder 1 (Upstream Side)



Inspection Result



$$h = x \sqrt{\left(\frac{t_c}{t_s}\right)^2 - 1}$$

: transmit time of 2
ts : transmit time of 1

Test Point No.	Bridge Element/Attribute	Distance x(mm)	Transmit time of 2 tc (ms)	Transmit time of 1 ts (ms)	Crack Depth h (mm)
1	S3 - G1 Upstream side	50	284	96	139.2
2	S3 - G1 Upstream side	50	544	96	278.90
3	S3 - G1 Upstream side	50	558	96	286.3

Inspection Record (Ultrasonic Pulse Velocity Test / Crack Depth 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	28-Nov-12
		Weather	FAIR

Photo Records: Span 3 - Girder 1 (Upstream Side)

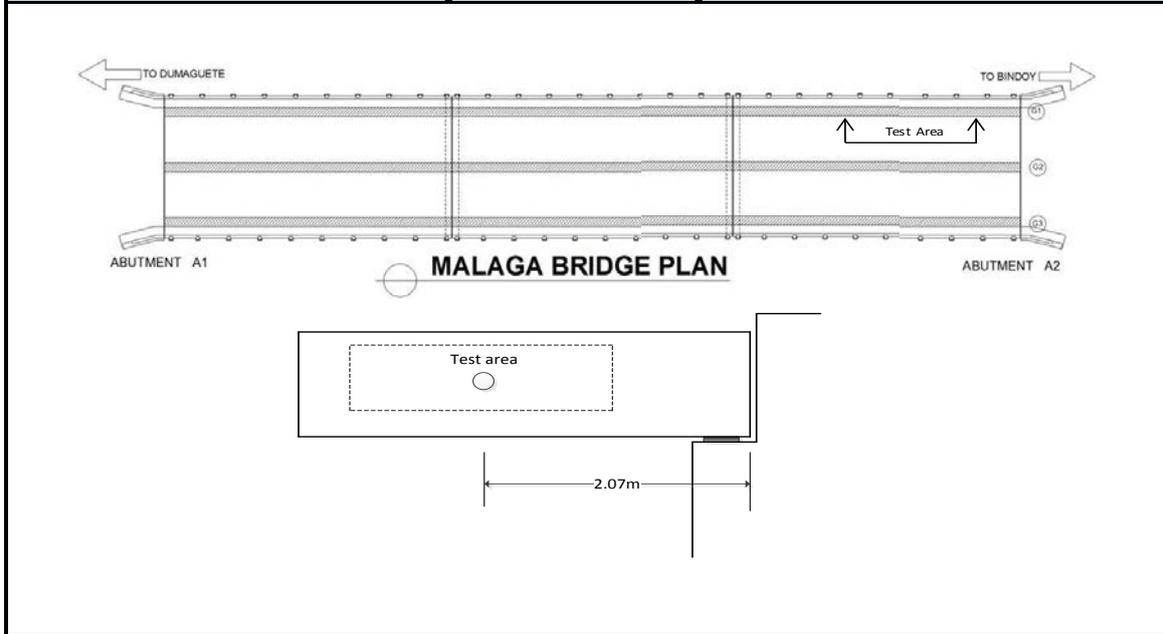
Photo-1 : General View	Photo-2 : Test Point
	

Photo-3 : Testing	Photo-4 : After Test (If Necessary)
	

Inspection Record (Half-Cell Electrical Potential Test 1/2)

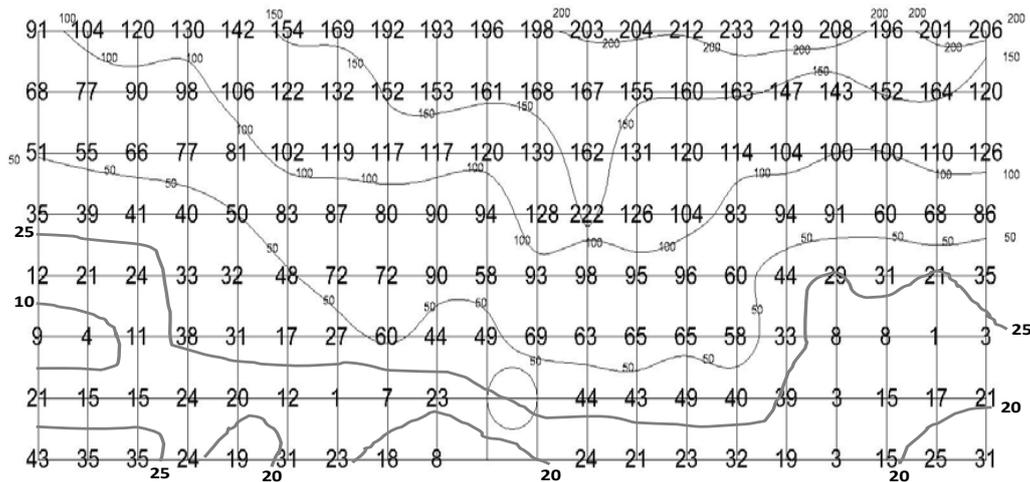
Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13	Weather	FAIR

Sketch of Inspected Location: Span 3 - Girder 1



Inspection Result

Test Point No. 1
Equi-potential Contour Map



Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	31-Jan-13
		Weather	FAIR

Photo Records : Span 3 - Girder 1

Photo-1 : General View	Photo-2 : Test Point
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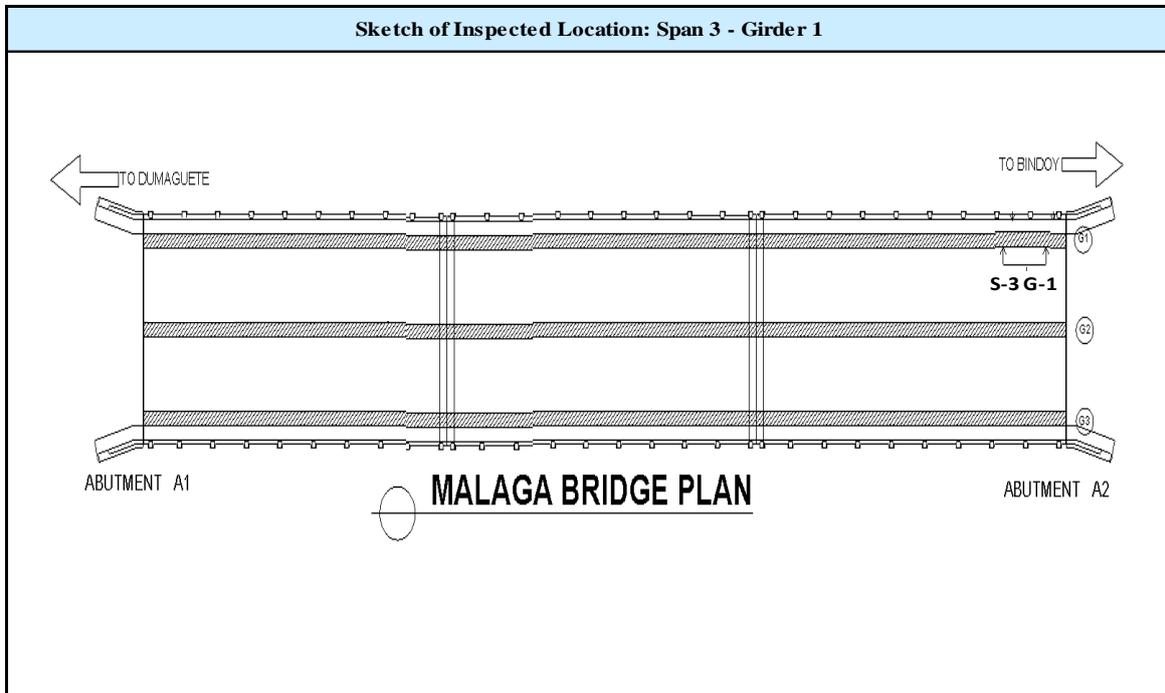


Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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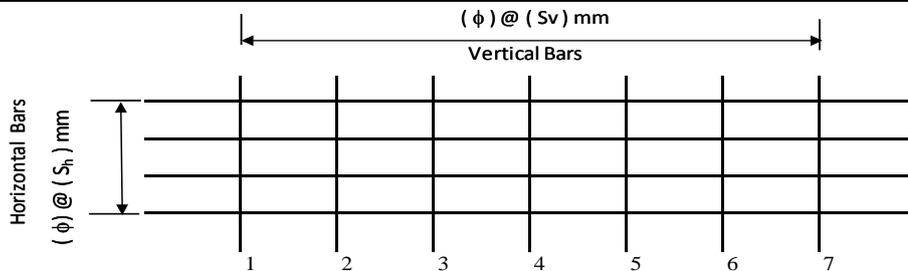


Inspection Record (Rebar Detection Test / Radar Type 1/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR



Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Vertical Bars		Horizontal Bars	
			Spacing (mm)	Cover (mm)	Spacing (mm)	Cover (mm)
Vertical Bars	Stirrup	Span 3-Girder 1	200/300	68		

Inspection Record (Rebar Detection Test / Radar Type 2/2)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Photo Records : Span 3 - Girder 1

Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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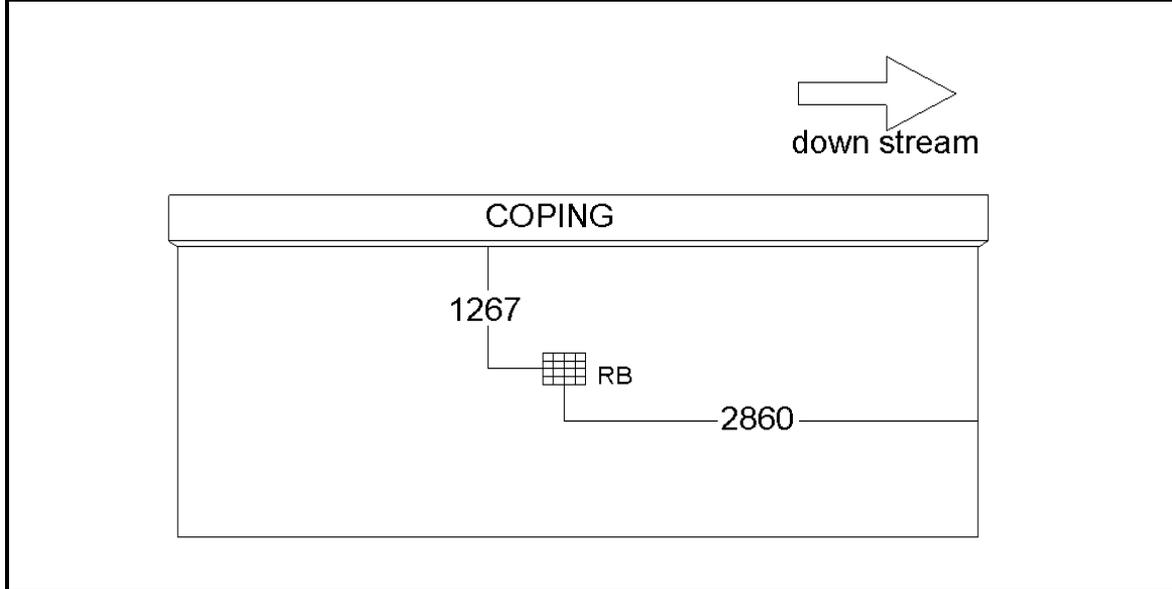
Inspection Records

Pier 1- Main Structure (Wall)

Inspection Record (Rebound Hammer Test 1/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date:	29-Nov-12
		Weather	FAIR

Skecthes of Inspected Location : Pier 1 - Main Structure (Wall facing Dumaguete)



Inspection Result

Test Point No. :	7	Actual Compressive Strength :	15.9 MPa
Bridge Element/Attribute : Pier 1 Main Struc. (Wall)		Number of Test :	

Test Results

45	46	47	45
40	46	44	44
46	42	49	42
47	44	48	48
47	46	48	48

Standard Deviation : No data
Average Reading : 45.6 Mpa

45.6×0.85 (factor from cube to cylinder strength) = 38.76
 38.76×0.41 (time factor) = 15.89 MPa

Inspection Record (Rebound Hammer Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Pier 1 - Main Structure (Wall facing Dumaguete)

Photo-1 : General View



Photo-2 : Test Point



Photo-3 : Testing

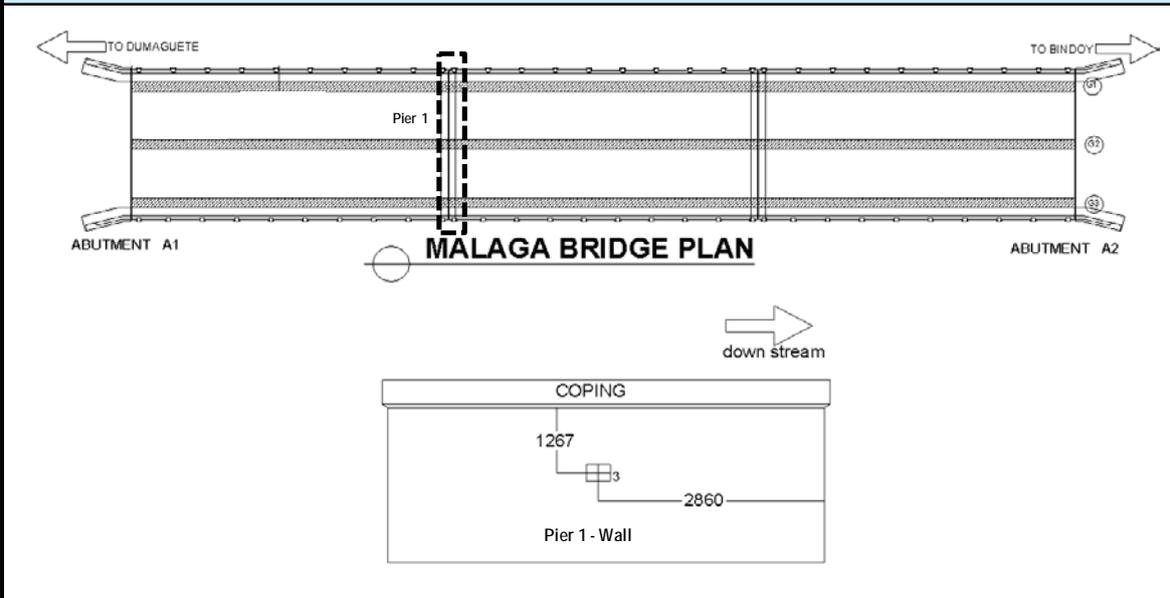


Photo-4 : After Test (If Necessary)

Inspection Record (Carbonation Test / Chipping-off 1/2)

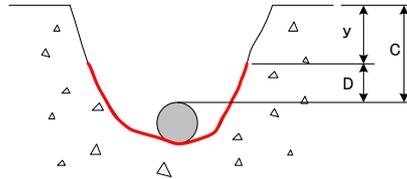
Bridge Name:	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Sketch of Inspected Location: Pier 1 - Main Structure (Wall)



Inspection Result

Construction Year:	1957
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Test Point No.	Bridge Element/Attribute	Cover Depth C (mm)	Carbonated Depth y (mm)	Un-carbonated Depth D (mm)
3	Pier 1	95	13	82

Inspection Record (Carbonation Test / Chipping-off 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Pier 1 - Main Structure (Wall)

Photo-1 : General View	Photo-2 : Test Point
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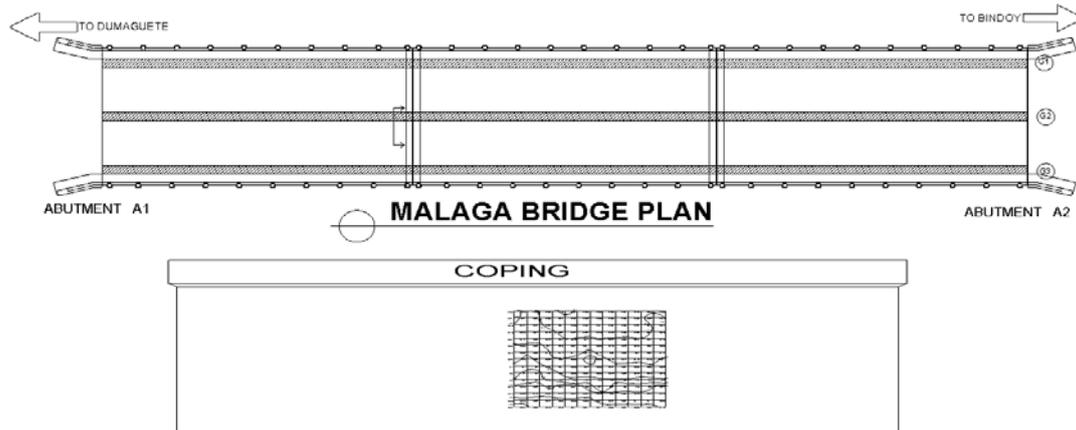
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Half-Cell Electrical Potential Test 1/2)

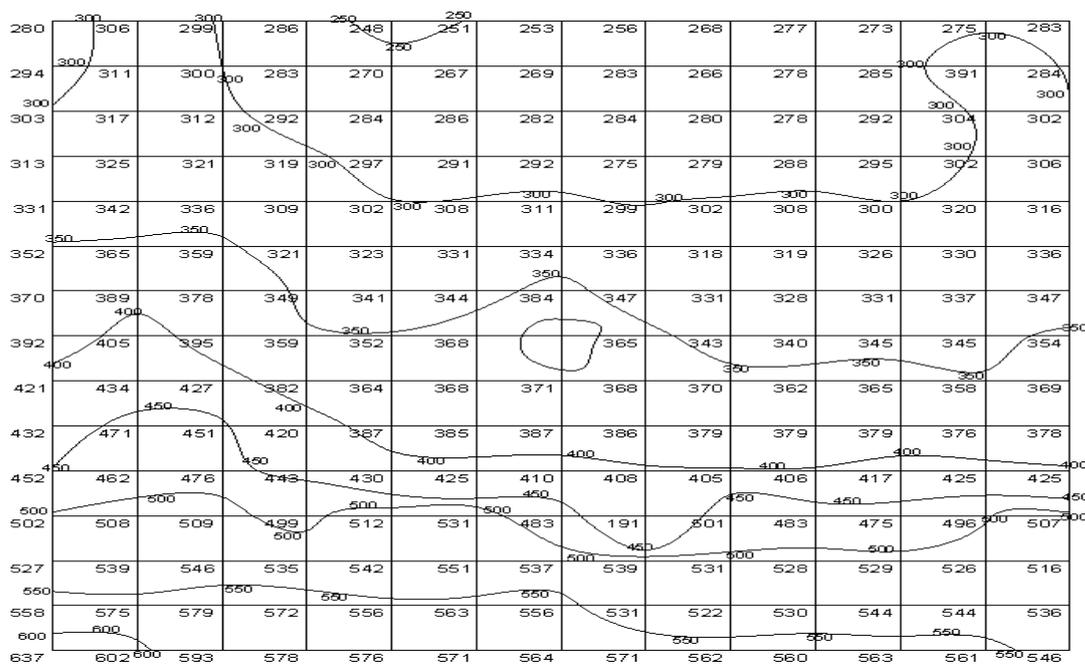
Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Sketch of Inspected Location: Pier 1 - Main Structure (Wall)



Inspection Result

Test Point No. 1
Equi-potential Contour Map



Unit of measure = mV

Inspection Record (Half-Cell Electrical Potential Test 2/2)

Bridge Name	MALAGA BRIDGE	Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12
		Weather	FAIR

Photo Records : Pier 1 - Main Structure (Wall)

Photo-1 : General View	Photo-2 : Test Point
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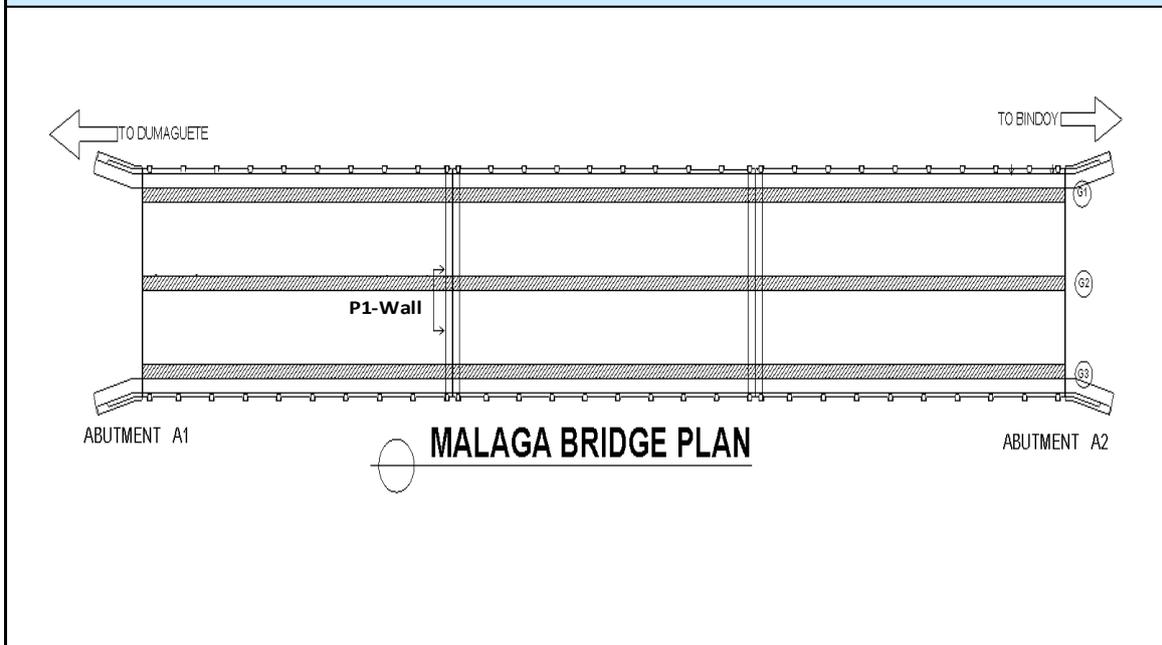
Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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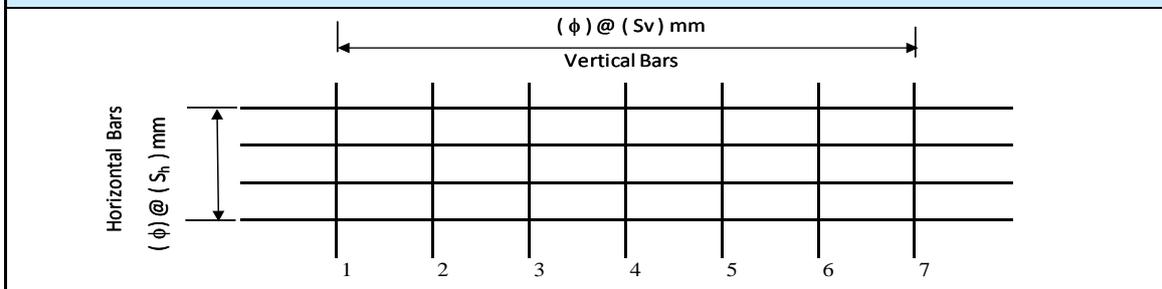
Inspection Record (Rebar Detection Test / Radar Type 1/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Sketch of Inspected Location: Pier 1 -Main Structure (Wall)



Inspection Result



Direction	Bar No.	Bridge Element/Attribute	Vertical Bars		Horizontal Bars	
			Spacing (mm)	Cover (mm)	Spacing (mm)	Cover (mm)
		Pier 1- Wall (Facing Dumaguete)	500	80	500	60

Inspection Record (Rebar Detection Test / Radar Type 2/2)

Bridge Name	MALAGA BRIDGE		Bridge ID	B00092NR	
Inspector	FELICIANO R. ESPINA	Date	29-Nov-12	Weather	FAIR

Photo Records : Pier 1 -Main Structure (Wall)

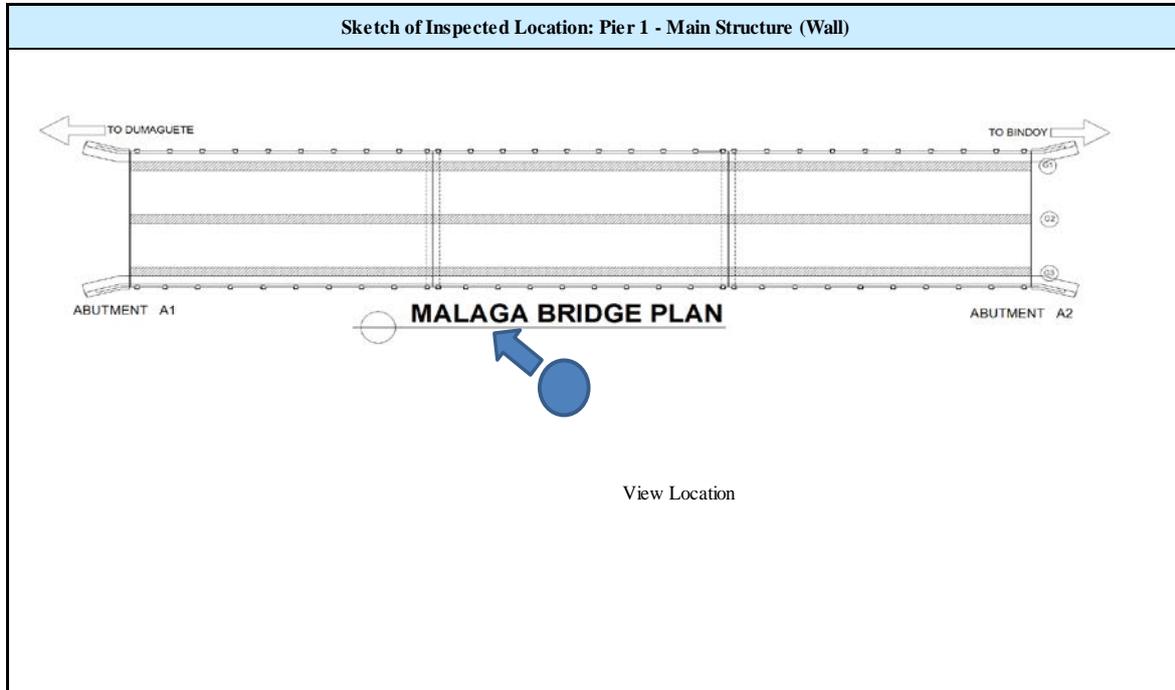
Photo-1 : General View	Photo-2 : Test Point
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Photo-3 : Testing	Photo-4 : After Test (If Necessary)
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Inspection Record (Infrared Thermal Image Test)

Bridge Name	MALAGA BRIDGE			Bridge ID	B00092NR
Inspector	FELICIANO R. ESPINA	Date	30-Jan-13	Weather	FAIR



Inspection Result

Test Point No.	Bridge Element / Attribute	Type of Structure
2	Pier 1/ Main Structure	Wall

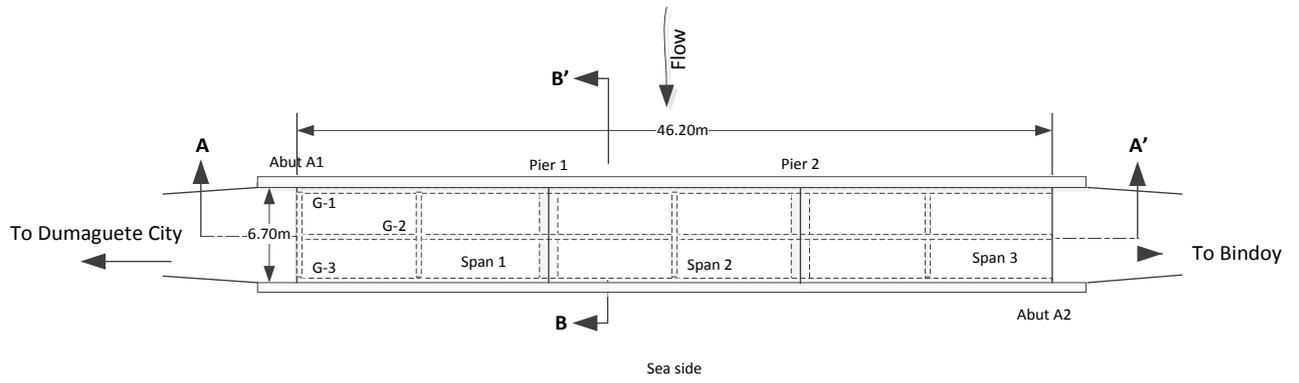
Visual Image	Thermal Image

Comment:
In this picture, spalling at the side of pier wall is marked with red circle while area marked with the green circle is the surface for investigation using this NDT to possibly detect the extent of delamination.

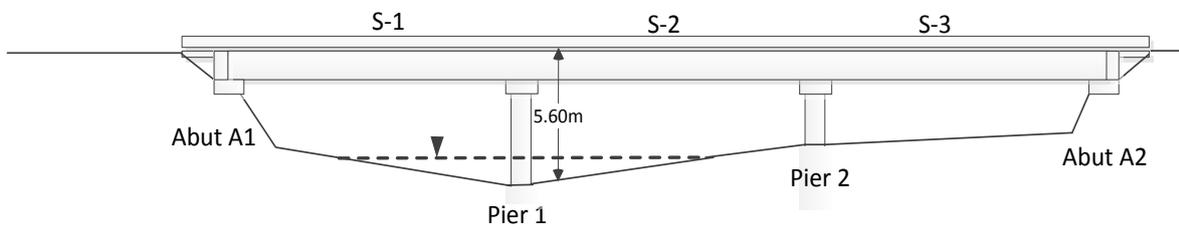
Comment:
In this picture, point marked by crosshair has a temperature of 29.8^o C (see at upper right side) with color yellow surrounded by green indicates that there is delamination.

Reference Charts And Diagrams

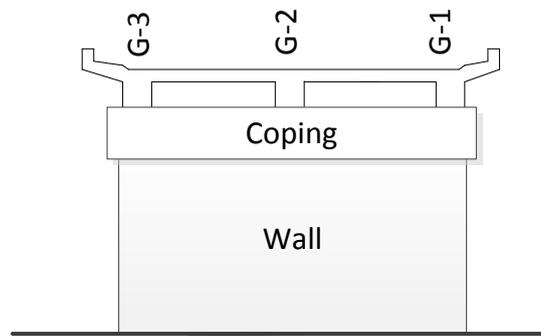
Malaga Bridge Engineering Inspection – Actual Test Location								
Element	Attribute	D0-1 Rebound Hammer	D0-2 Carbonation	D0-3 Ultrasonic Pulse Velocity	D0-4 Half-Cell Potential	D0-5 Rebar Detector		D0-6 Thermal Imager
						Hilti (Magnetic)	JRC (Radar)	
Span-1	Deck Slab	● G1-G2	● G1-G2	● G1-G2	● G1-G2	● G1-G2	-	
	Girder	● G2	● G2	● G2	● G2	-	● G2	
Span-2	Deck Slab	● G1-G2	● G1-G2	● G1-G2	● G1-G2	● G1-G2	-	
	Girder	● G2	● G2	● G2	● G2	-	● G2	
Span-3	Deck Slab	With water leakage X	With water leakage X	With water leakage X	With water leakage X		-	
	Girder	● G1	● G1	● G1	● G1	-	● G1	
Pier - 1	Wall	● Wall	● Wall	Rebar-exposure X	● Wall	-	● Wall	● Wall
 Conduct NDT								



General Plan



Section A-A' - Profile



Section B-B' - Elevation

Plan and Profile - Malaga Bridge

APPENDIX C

SUMMARY OF BRIDGE REPAIR METHODS

*(From DPWH-JICA Bridge Repair Manual, 2nd Edition;
Section 3.2- Procedure for Selecting Repair Method, pages 3-13 to 3-30)*

3-2 PROCEDURE FOR SELECTING REPAIR METHOD

3-2-1 Selecting Repair Method

The repair methods suggested in this manual are prepared based on the condition rating criteria of the BMS. The different repair methods are selected according to the type of defects found on the bridge structure. The selection of repair methods however gives priority to bridges that will remain structurally sound after the repair and to those with defects which can be practically repaired, considering local conditions in the Philippines. For major repairs where the bridge exhibits different combinations of defects, corresponding repair measures shall be simultaneously carried out using the appropriate repair methods.

Prior to using this manual, the following should be considered to determine the appropriate repair method:

(1) Necessary Strength Based on Original Design

Repair methods included in this manual aim to maintain the capacity of the bridge based on its original design and not to further enhance its serviceability strength to meet the latest design requirements. Nevertheless, it is noted that even if the full design load capacity is achieved, slight overstressing due to the defect may be accepted for certain serviceability limits. Engineering judgment is also vital in selecting the appropriate repair method in order to minimize cost. For example, repair of the full flange area of a simply- supported steel girder may not be necessary near its end span since bending moment at this location is expected to be small.

(2) Bridge Service Life and Characteristic of Defects

The proposed repair methods are limited to realizing the corresponding repair costs, and not the value of the bridge's life cycle cost. Engineering inspection team of the DPWH need to also consider the following factors in determining the appropriate repair methods:

- The service life of steel bridges is governed by the fatigue life of a steel section. It is not practical to spend largely on repairs if the bridge is close to its predicted fatigue life (50~60 years in Philippines), hence, replacement is a better option.
- In concrete structures, defects such as corrosion of reinforcement may not be visible. However, such defects are known to occur in the form of concrete cracking or spalling.
- It is not practical to spend on repairs for some localized defects if more extensive defects are expected to appear in the near future. For such case, engineering inspection should be conducted for the complete structure prior to implementation of costly repairs of local concrete defects.

(3) Detailed Evaluation/Assessment in Engineering Inspection

This manual does not provide assessment of the cost of repairs against its possible benefits, which could be the basis in deciding between repair and reconstruction. It should be noted that reconstruction is not covered in this manual. However, an engineering assessment for the necessity of reconstruction should also be undertaken by the engineering inspection team.

(4) Partial Replacement of Bridge

In case a primary bridge component exhibit defects that appear to spread widely and have continuously progressed, partial replacement of the component could be considered. This may involve replacement of damaged bracing members, stiffeners, stringers, cross girders, but not to the extent of completely replacing the components. (Total replacement of girder or deck slab is not included in this repair manual.)

(5) Total Repair Cost for Defects

The repair costs refer to the unit cost of each repair. The total cost for implementing repair should also include the following items for each bridge, with due consideration of the scale and location (terrain) of the bridge in concern:

- Transportation cost of materials and equipment
- Scaffolding, staging and other temporary facilities
- Detour road or detour bridge if necessary
- Management cost (Overhead and profit)

(6) Adverse Problems on the Structure and Traffic

Consider the following issues that may cause adverse problems during the implementation of the bridge Repair:

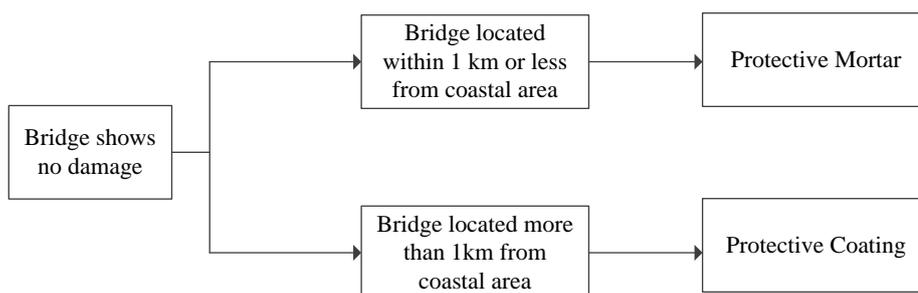
- Detour road or bridge for rerouting traffic flow.
- Reduction of traffic load clearances during overlay or jacking-up of girder
- Structure becomes more vulnerable to damage or deterioration during repairing
- Other potential defects observed

3-2-2 Preventive Maintenance

Although a bridge may not have any damage, it is recommended that the bridge members be protected from effects of carbonation and salt attack.

If the bridge is located more than one (1) kilometer from coastal area, protective coating made of acryl urethane based coating is applied to protect the bridge not only from carbonation but from weather/UV rays, chemical and oil damage. Refer to Sections 4-10, 5-8, and 6-6 for details of application of protective coating.

If the bridge is located less than one (1) kilometer from coastal area, protective mortar is applied to protect the bridge from effects of salt attack. Protective mortar is made from lithium nitrate polymer cement mortar. Refer to Sections 4-9, 5-7, and 6-5 for details of application of protective mortar



3-2-3 Cracking on Concrete Structure

Generally, fine and stable cracks with up to 0.3 mm width have no adverse effect if concrete cover to reinforcement is adequate. These cracks are generally harmless and need not be repaired.

Defective cracks with over 0.3 mm width develop due to carbonation, chlorination, alkali aggregate reaction, rebar corrosion, overloading of structure, foundation movement, and lack of adequate cover. It is important to identify the presence of these causes. All defective cracks must be repaired as shown in Figure 3-1 to Figure 3-3. The repair methods for cracks are classified according to crack width, based on the Condition Rating Criteria of the BMS.

(1) Deck Slab

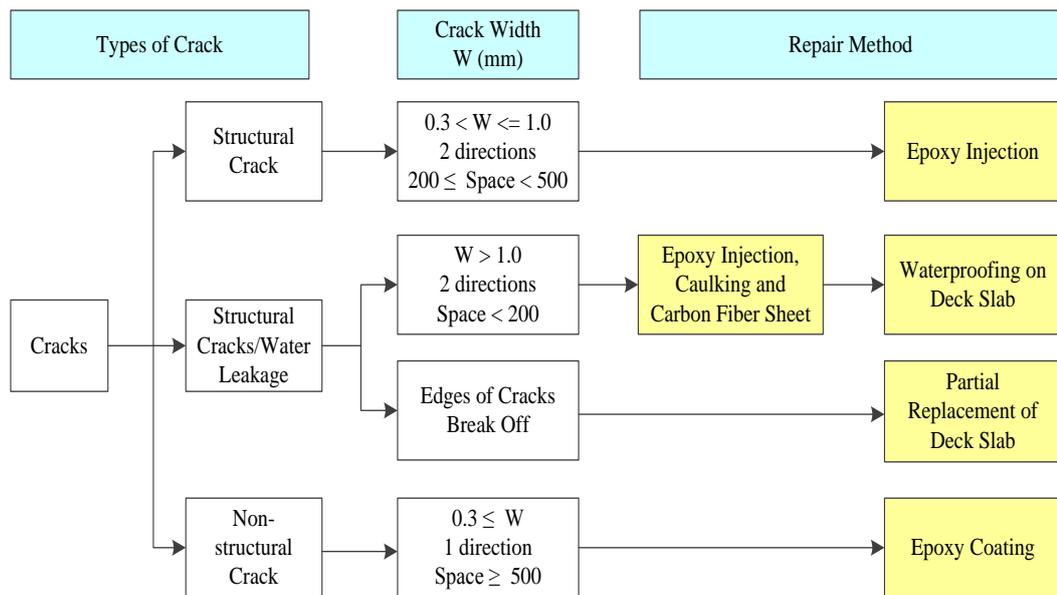


Figure 3-1 Selection Procedure of Repair Method for Deck Slab Due to Cracks

(2) Superstructure

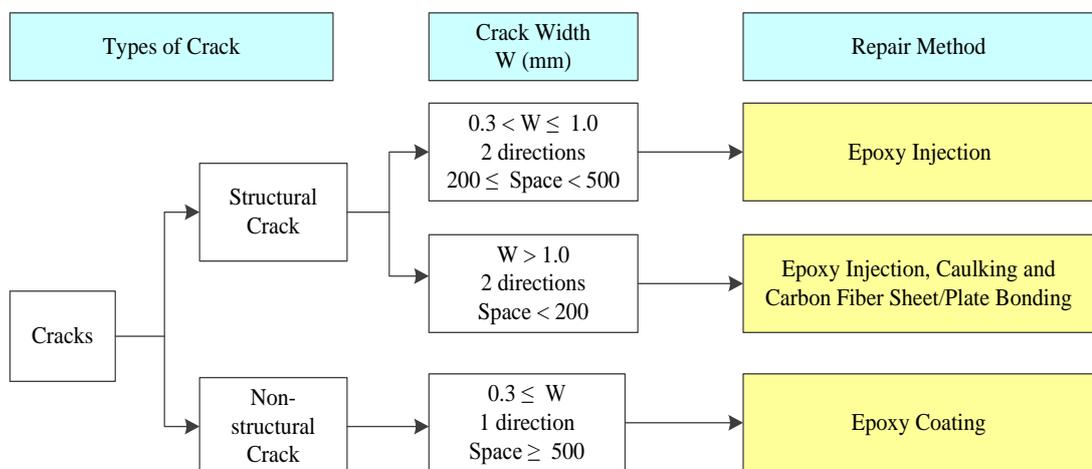
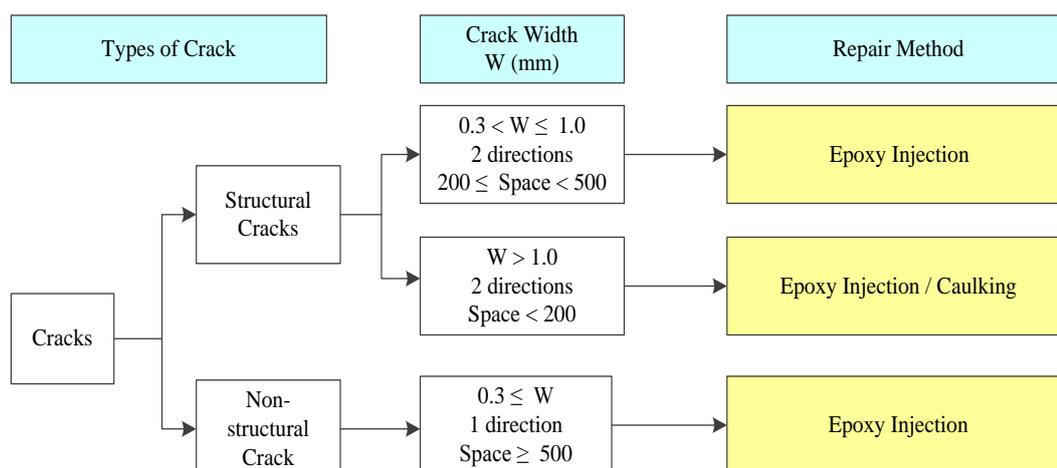


Figure 3-2 Selection Procedure of Repair Method for Girder Due to Cracks

(3) Substructure



Remark: Epoxy injection repair shall be similar to the epoxy injection for superstructure.

Figure 3-3 Selection Procedure of Repair Method for Substructure Due to Cracks

3-2-4 Spalling, Scaling and Disintegration on the Concrete Structure

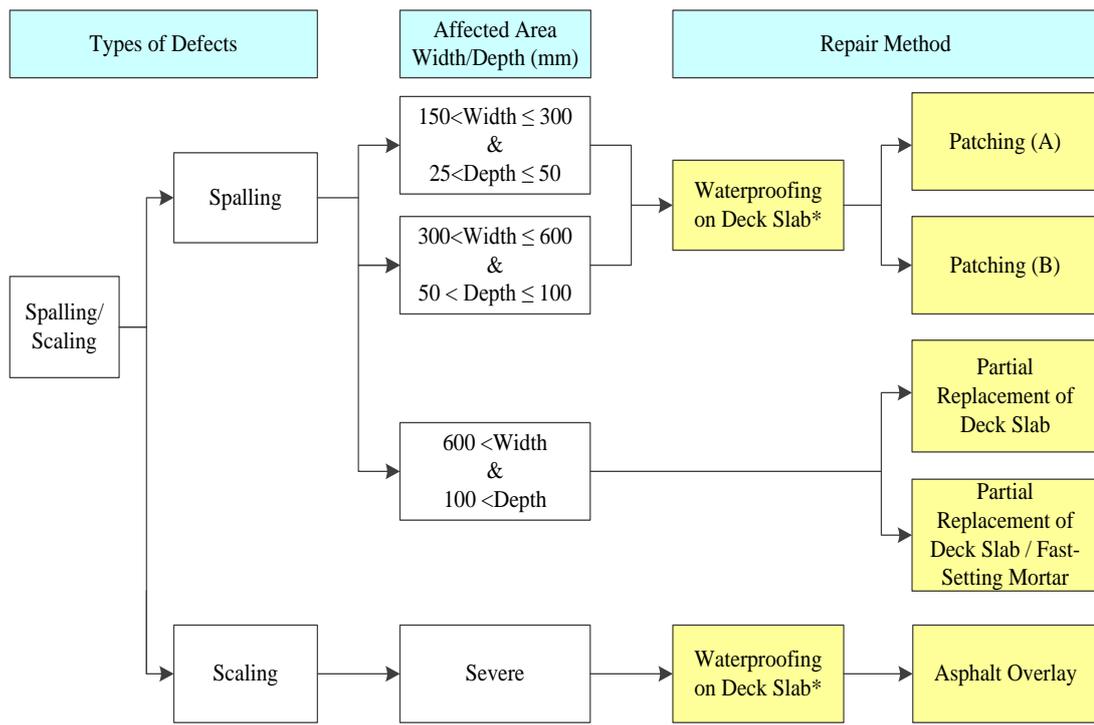
Spalling is defined as a depression resulting from detachment of a large fragment of concrete. On the other hand, disintegration is a depression resulting from detachment of small fragment. The major cause of spalling is expansion resulting from corrosion of reinforcement. Spalling caused by impact can weaken the structure locally and expose the reinforcement to corrosion.

Scaling of concrete surfaces is defined as local flaking or peeling away of concrete near the surface. As the deterioration continues, coarse aggregate particles are exposed and eventually become loose.

All defects must be repaired based on the flowchart shown in Figure 3-4 to Remark*: For pile bents, repair method is applied with concrete jacket

Figure 3-6. The repair method for spalling is classified according to width and depth damaged, while that for scaling is only according to the width of damage on the concrete. This is based on the Condition Rating Criteria of the BMS.

(1) Deck Slab



Remark*: Waterproofing shall be conducted after repair of damaged areas

Figure 3-4 Selection Procedure of Repair Method for Deck Slab due to Spalling and Scaling

(2) Superstructure

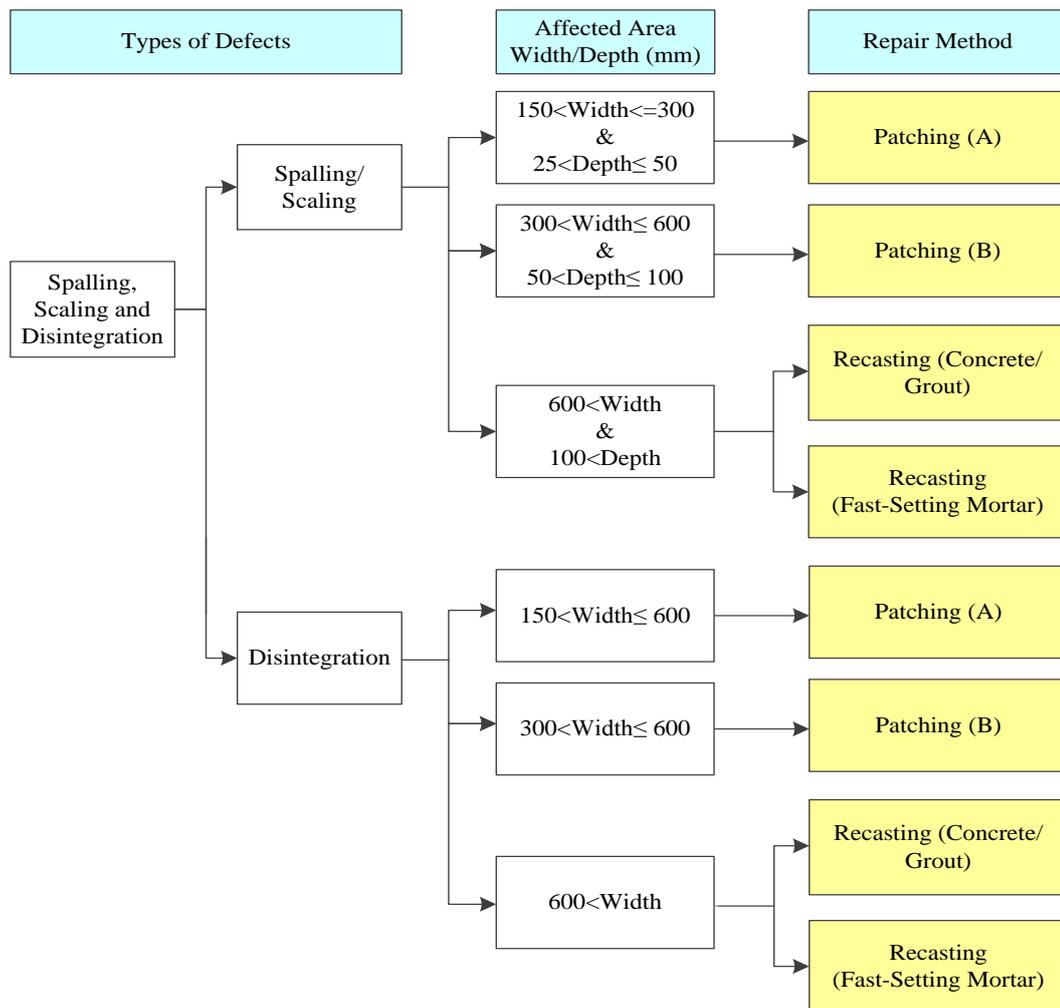
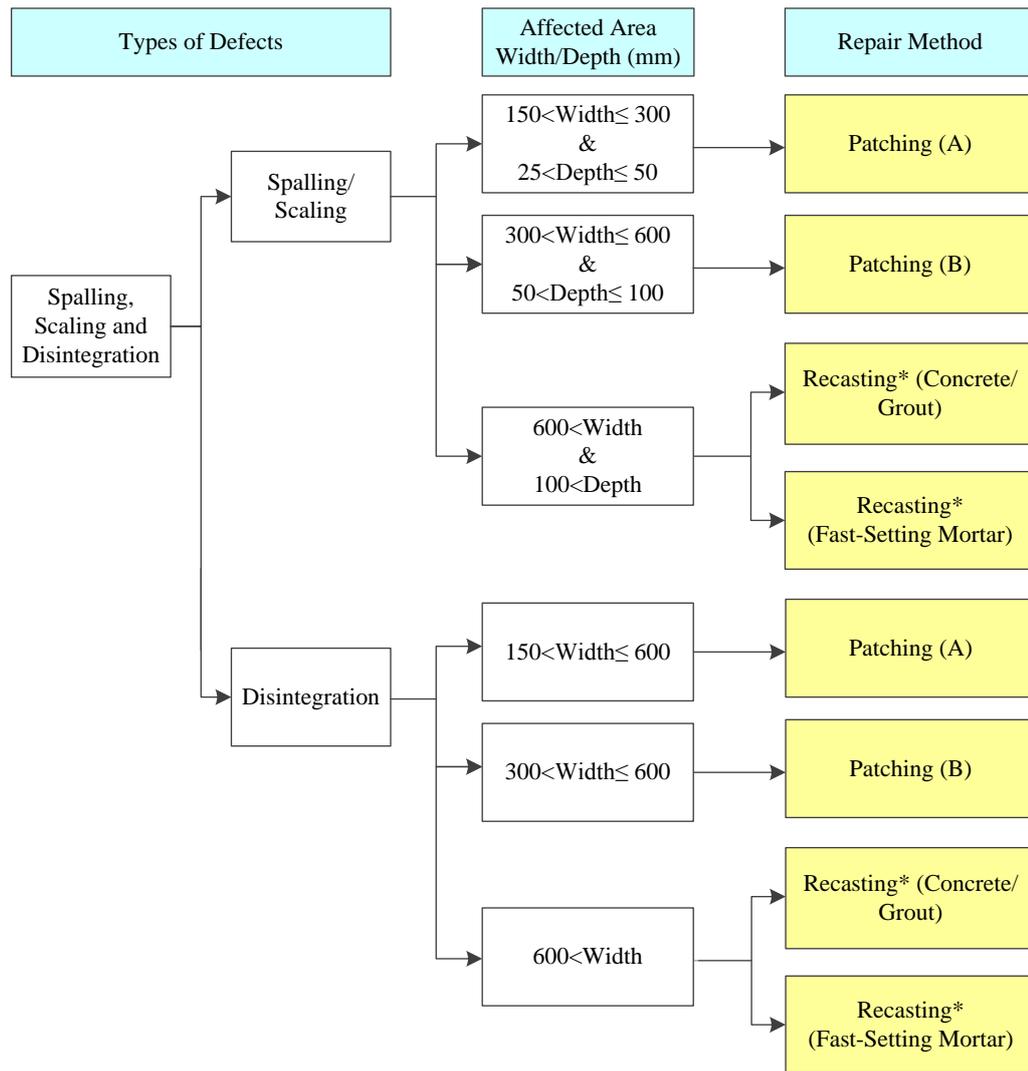


Figure 3-5 Selection Procedure of Repair Method for Superstructure Due to Spalling, Scaling and Disintegration

(3) Substructure



Remark*: For pile bents, repair method is applied with concrete jacket

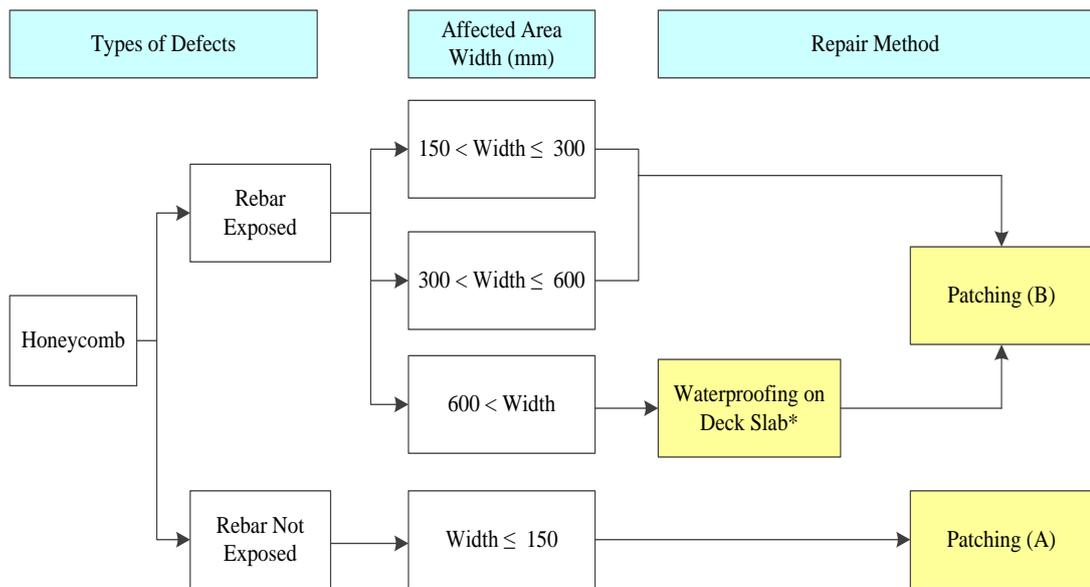
Figure 3-6 Selection Procedure of Repair Method for Substructure due to Spalling, Scaling and Disintegration

3-2-5 Honeycomb on Concrete Structure

Honeycomb is formed due to lack of mortar in the spaces between coarse aggregate particles. It is caused by insufficient compaction or vibration during the placement of concrete, resulting in a porous and weak reinforced concrete. The voids will allow ingress of water, oxygen and corrosive agents such as carbon dioxide, chlorides and sulfates that will eventually cause corrosion of the steel reinforcement.

Honeycomb defects must be repaired based on the flowcharts shown in Figure 3-7 to Figure 3-9.

(1) Deck Slab



* Water proofing shall be applied after deck repair

Figure 3-7 Selection Procedure of Repair Method for Deck Slab Due to Honeycomb

(2) Superstructure

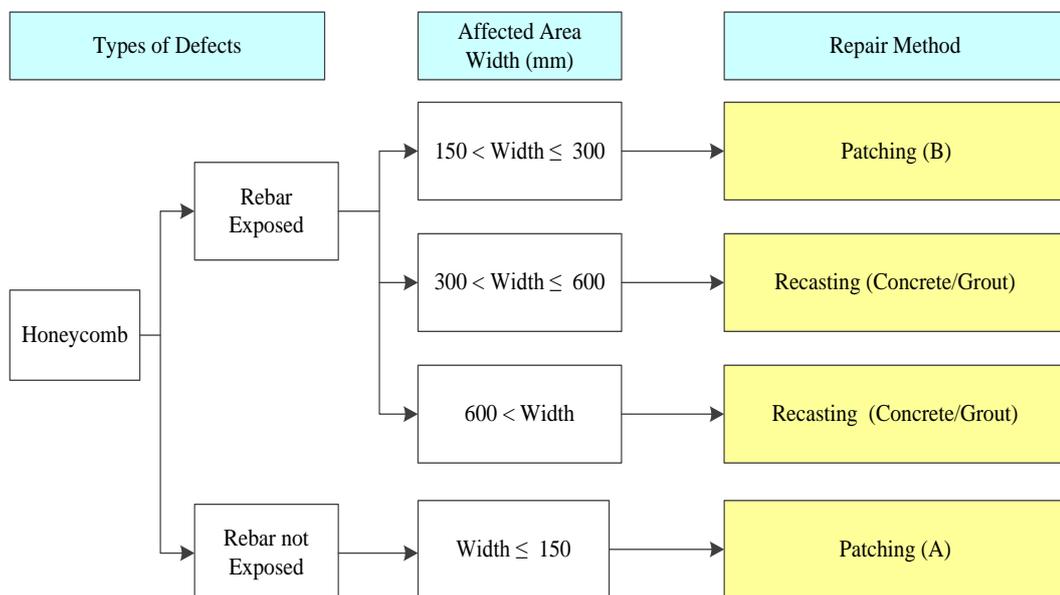


Figure 3-8 Selection Procedure of Repair Method for Superstructure Due to Honeycomb

(3) Substructure

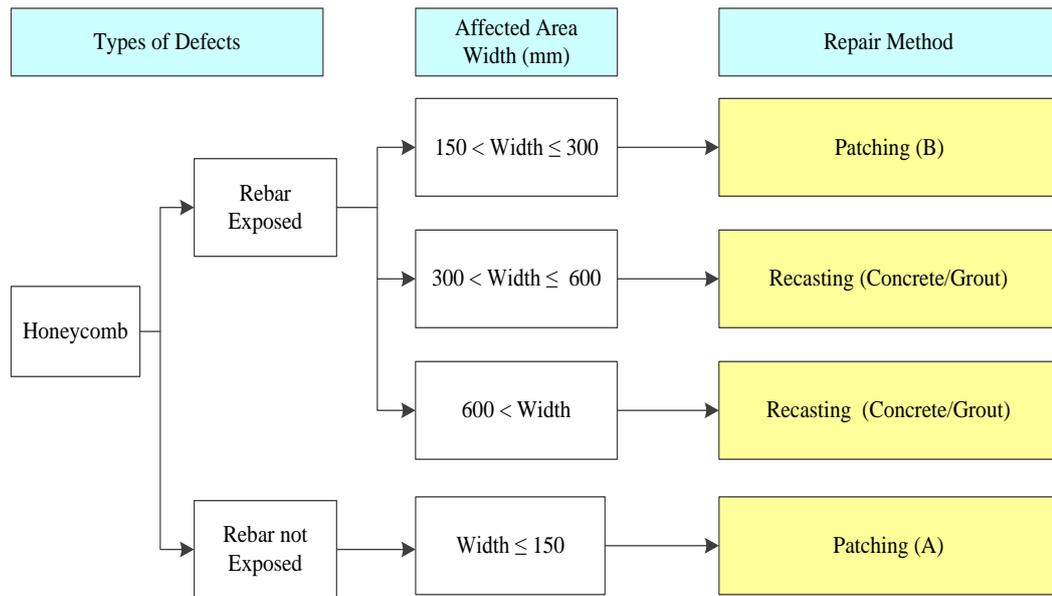


Figure 3-9 Selection Procedure of Repair Method for Substructure Due to Honeycomb

3-2-6 Rebar Exposure on Concrete Structure

During initial occurrence of defects such as spalling, scaling and delamination, exposure of rebars to air are limited, hence, does not significantly affect the strength and serviceability of the bridge structure. In severe cases where rebars are widely exposed and loss of section seem progressive strength and serviceability of the bridge is consequently reduced.

All defects must be repaired based on the flowchart shown in Figure 3-10 to Figure 3-12. The applicable repair method for rebar exposure is determined according to the width of defective area on the concrete.

(1) Deck Slab

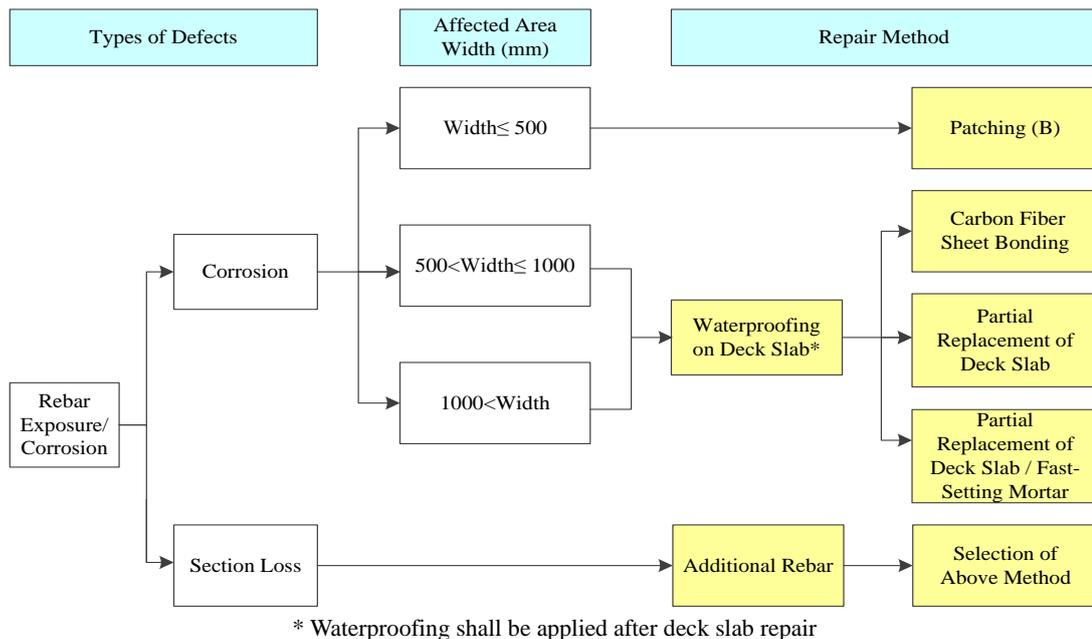


Figure 3-10 Selection Procedure of Repair Method for Deck Slab Due to Rebar Exposure

(2) Superstructure

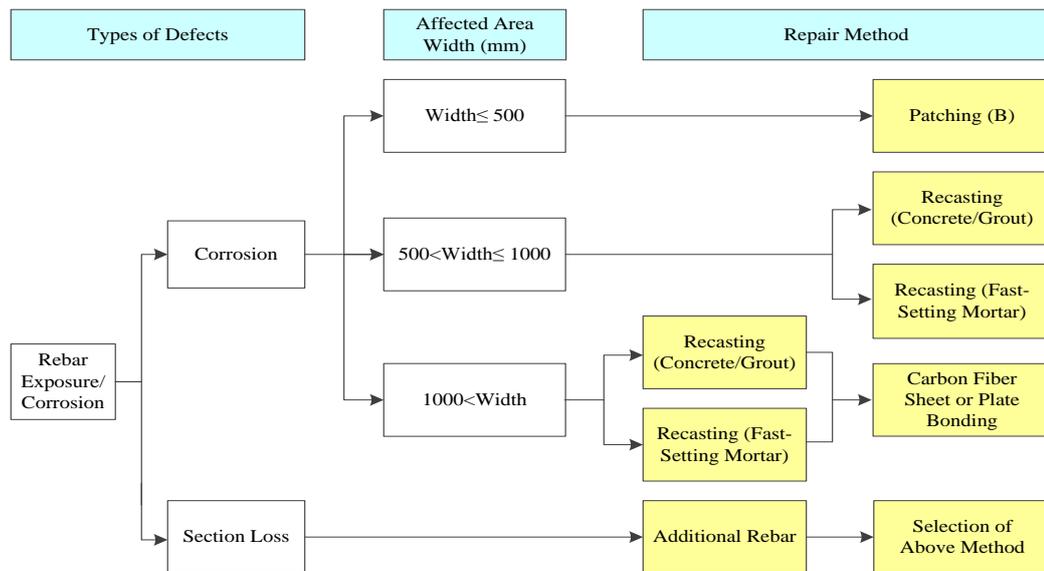
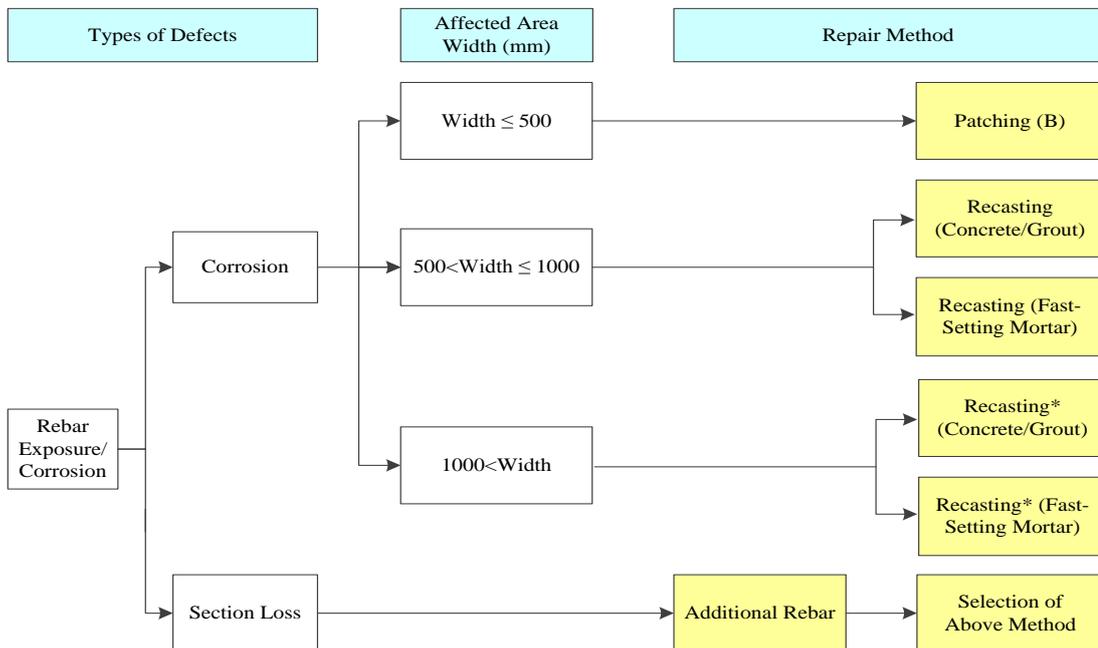


Figure 3-11 Selection Procedure of Repair Method for Superstructure Due to Rebar Exposure

(3) Substructure



*For pile bent, repair is by application of concrete jacket

Figure 3-12 Selection Procedure of Repair Method for Substructure Due to Rebar Exposure

3-2-7 Delamination on Concrete Structure

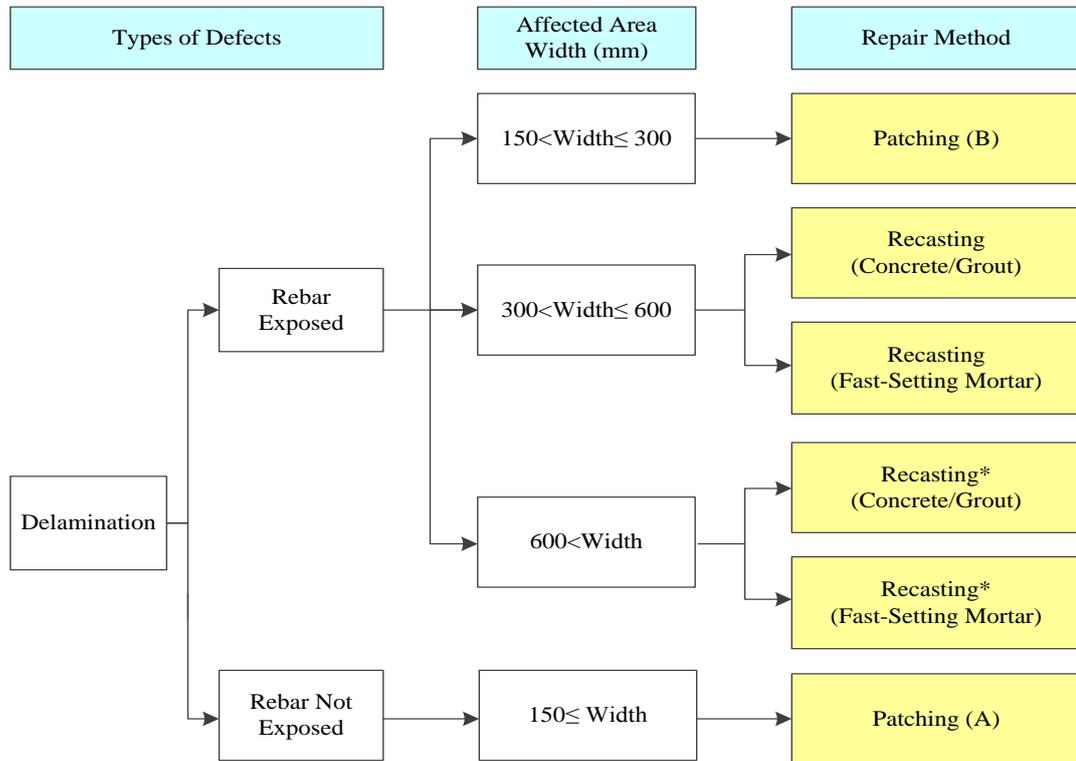
Delamination is the separation of concrete from the reinforcing steel. Delamination may not be identified visually since the defect is beneath the concrete surface. Sound tapping on the surface allows accurate identification of the affected area on the concrete surface. To confirm the delaminated

areas, hammer shall be used to break the inspected surface. It should be realized that if the corresponding repair is not successful, the concrete will eventually spall off.

All defects must be repaired based on the flowchart shown in Remark: If there is section loss of rebar, repair method with mark * is applied with carbon fiber sheet/plate bonding after recasting

Figure 3-13 and Figure 3-14.

(1) Superstructure



Remark: If there is section loss of rebar, repair method with mark * is applied with carbon fiber sheet/plate bonding after recasting

Figure 3-13 Selection Procedure of Repair Method for Delamination in Superstructure

(2) Substructure

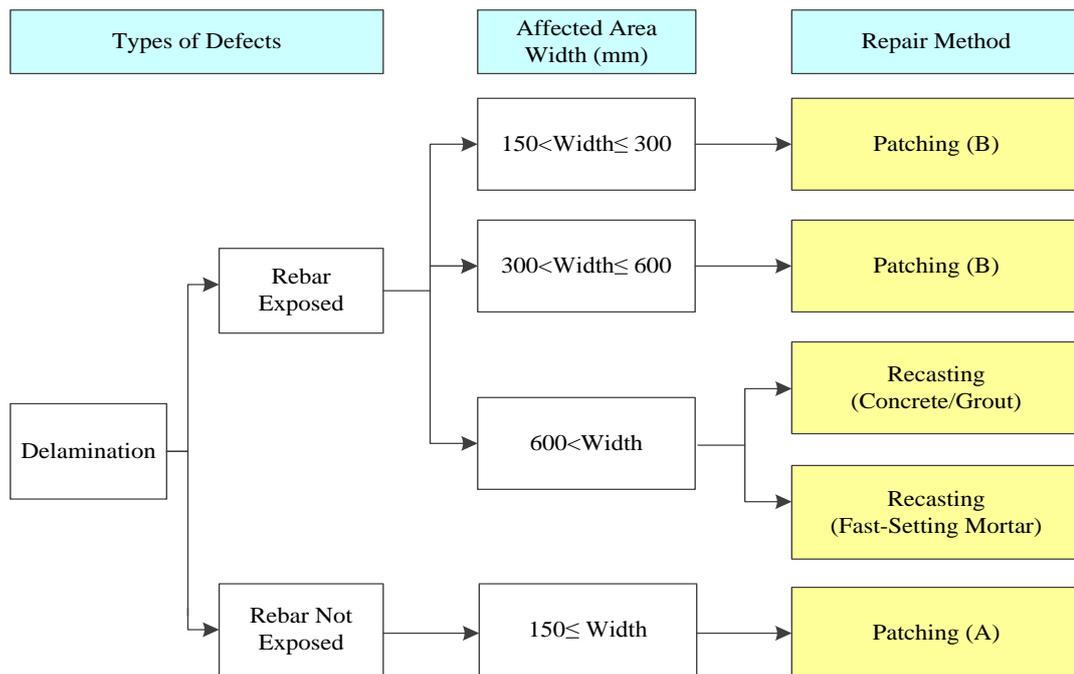


Figure 3-14 Selection Procedure of Repair Method for Delamination in Substructure

3-2-8 Corrosion on Steel Structure

Repainting may be the only repair measure required against rusting and corrosion of steel members. It may also be implemented to supplement other repair measures for strengthening section loss. The most effective method against corrosion is to apply abrasive blast cleaning prior to the proper application of a high quality paint system and touch-up painting as routine maintenance. Heavy corrosion with section loss often appears at the end of lower flanges, thus, special anti-corrosion paint should be applied to avoid progressive corrosion. The repair method for rust/corrosion is classified according to the severity of the condition and the percentage of section loss, based on the Condition Rating Criteria in BMS.

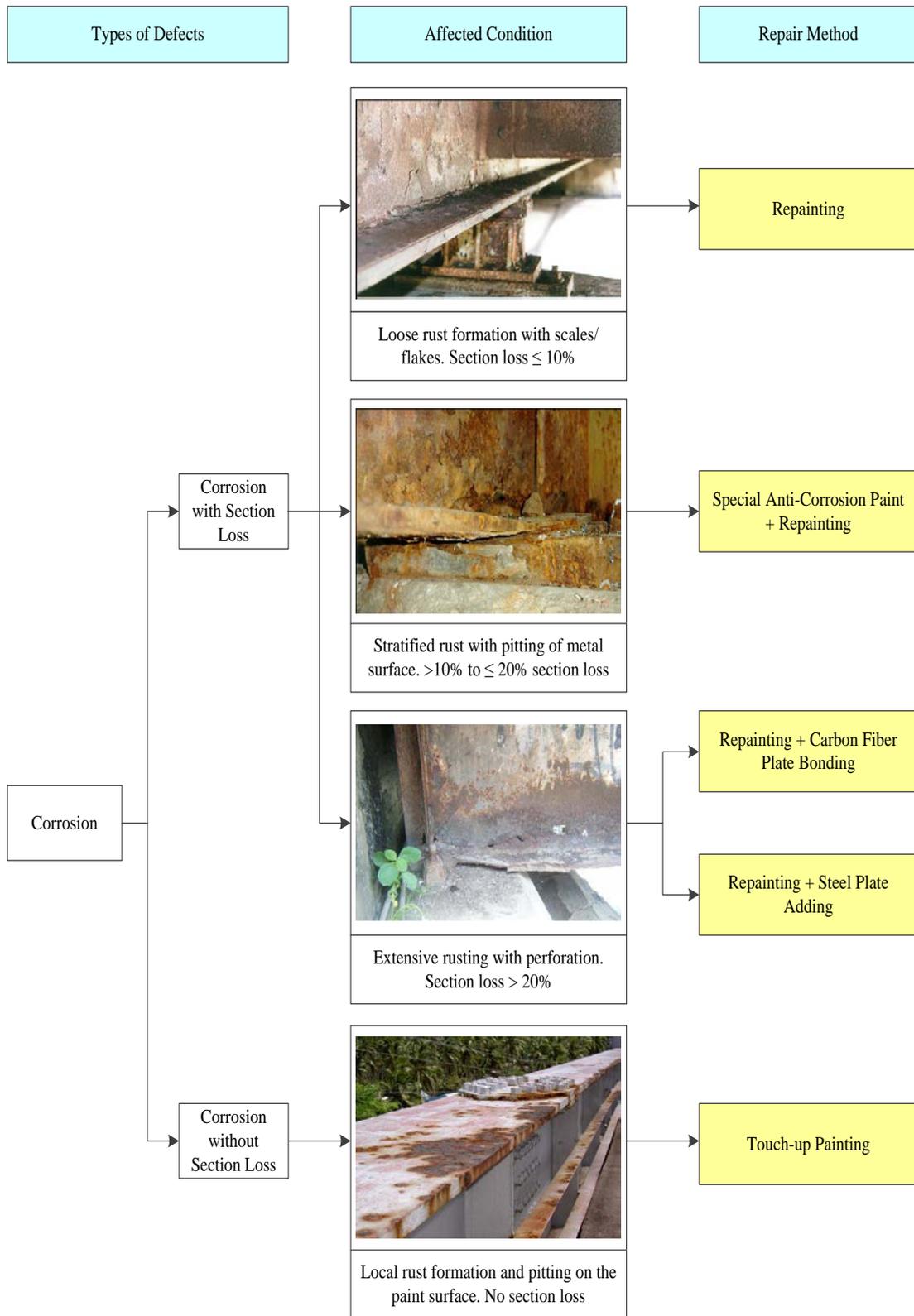


Figure 3-15 Selection Procedure of Repair Method for Steel Due to Corrosion

3-2-9 Paint Peel-off of Steel Structure

Paint peel-off on steel bridges is commonly found on bridges constructed in the 1980's. Touch-up painting is initially required to supplement the repainting repair for corrosion protection. This shall be applied to the steel structure, prior to final application of paint coating intended to restore the uniform appearance of the bridge. The repair method for paint peel-off is classified according to the percentage of affected surface area, based on the Condition Rating Criteria in BMS.

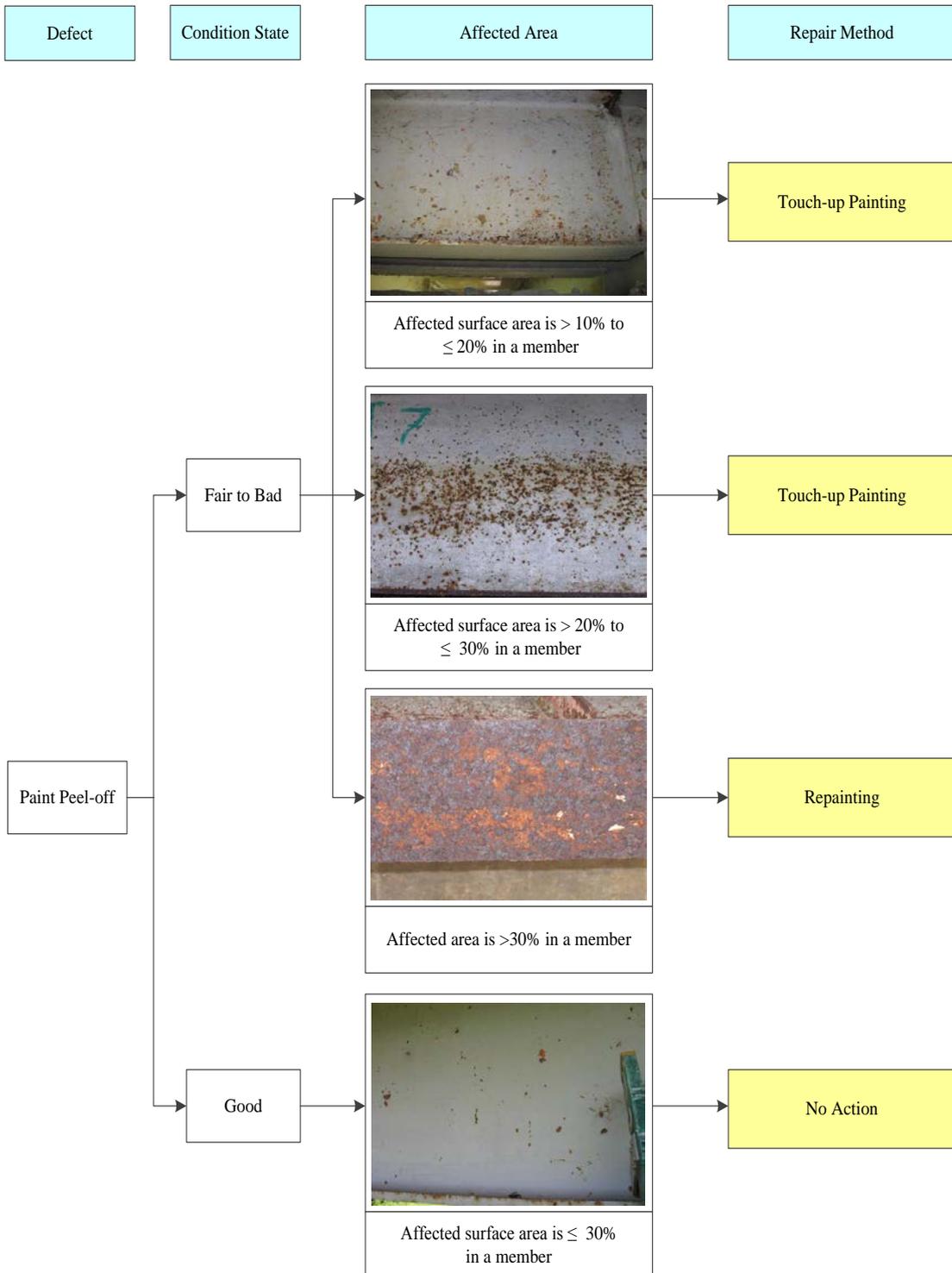


Figure 3-16 Selection Procedure of Repair Method for Steel due to Paint Peel-off

3-2-10 Loose Bolt Connection

Bolts at joints connecting the steel members are sometimes loosened or missing. There were cases where these are merely replaced with ordinary bolts instead of High Tension Bolts (HTB) or steel fasteners. Such conditions subject the bridge to risk of collapse. Repair for loose bolt connection is by resetting or retightening of HTB. The repair method for this defect is classified according to the percentage of loose fasteners and numbers of bolts that fell off from its connection, based on the Condition Rating Criteria in BMS.

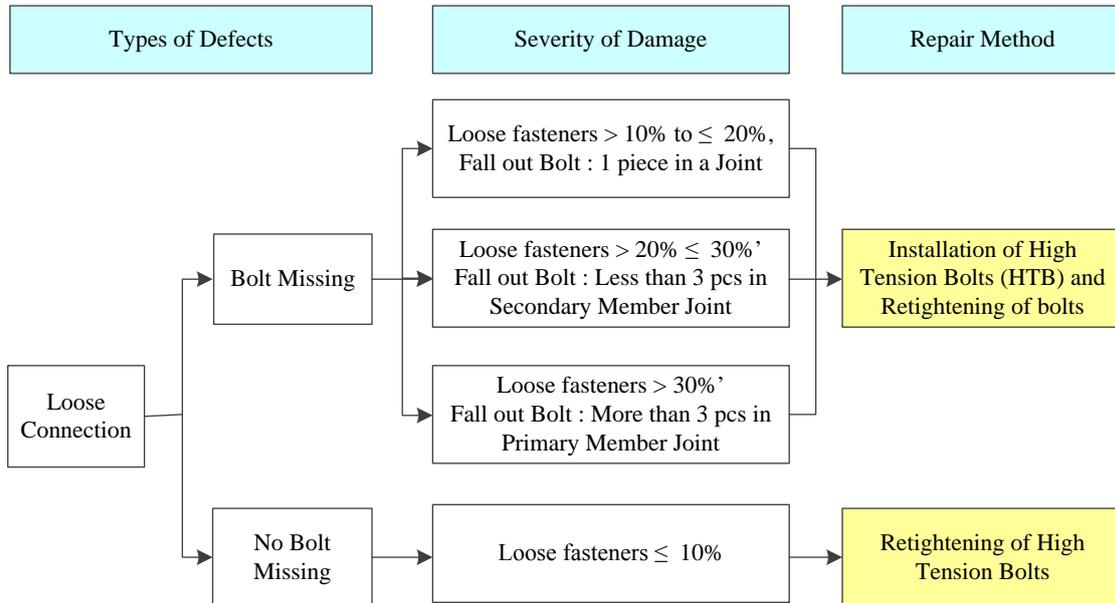


Figure 3-17 Selection Procedure of Repair Method for Steel Due to Loose Bolt Connection

3-2-11 Repair of Bridge Accessories

(1) Expansion Joint

The three major types of expansion joints used for bridges in the country include sealant, rubber and steel joints. These are often damaged because of rain water leakage. Said leakage causes many other damages on the bridge such as degradation of bearings and corrosion of steel girder. Damaged expansion joint should be replaced with seamless joint or a new type of rubber and steel joint with under drain function. The repair method for expansion joint is classified according to the extent of the damage, based on the Condition Rating Criteria in BMS.

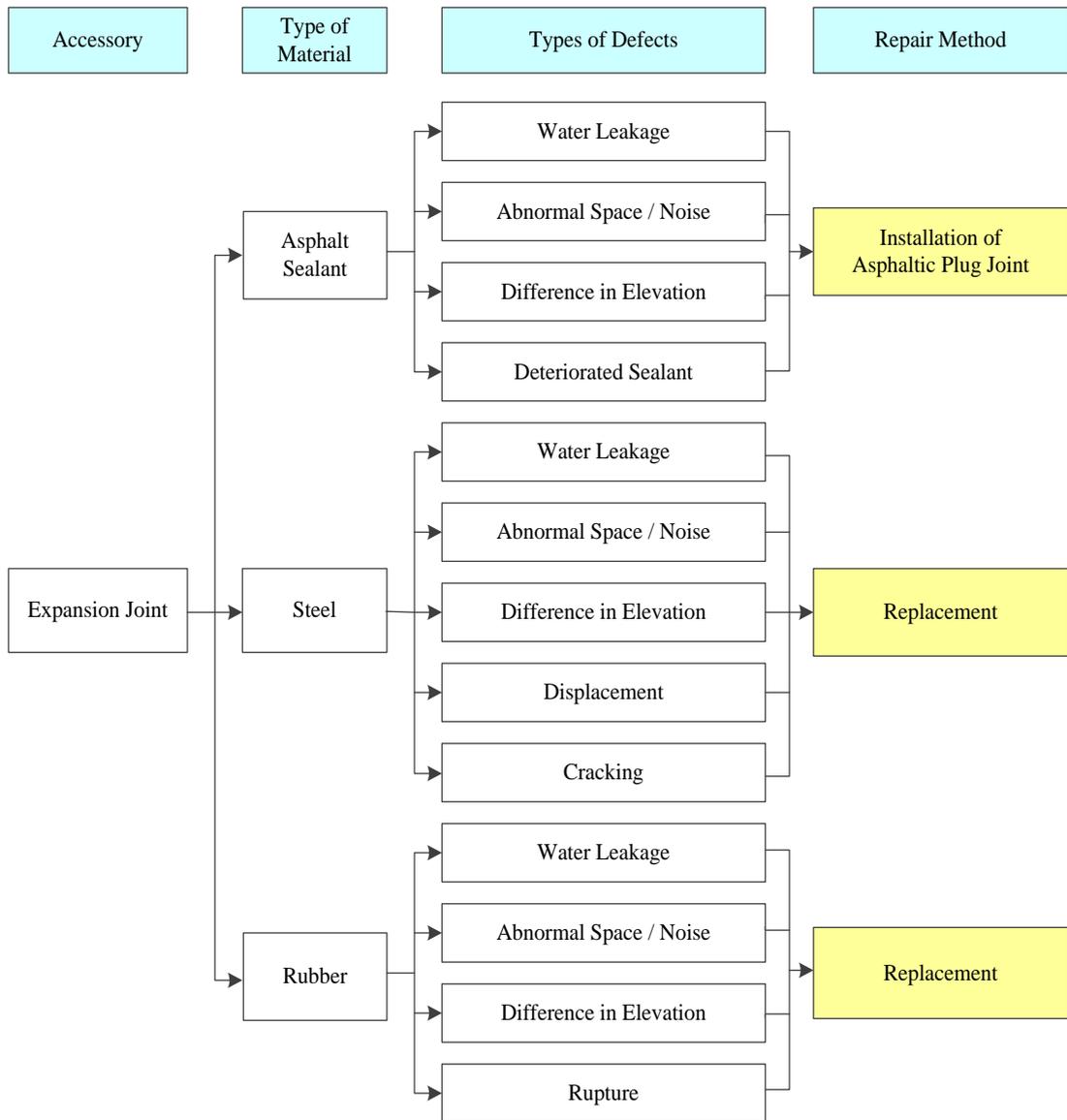


Figure 3-18 Selection Procedure of Repair Method for Expansion Joints

(2) Bearings

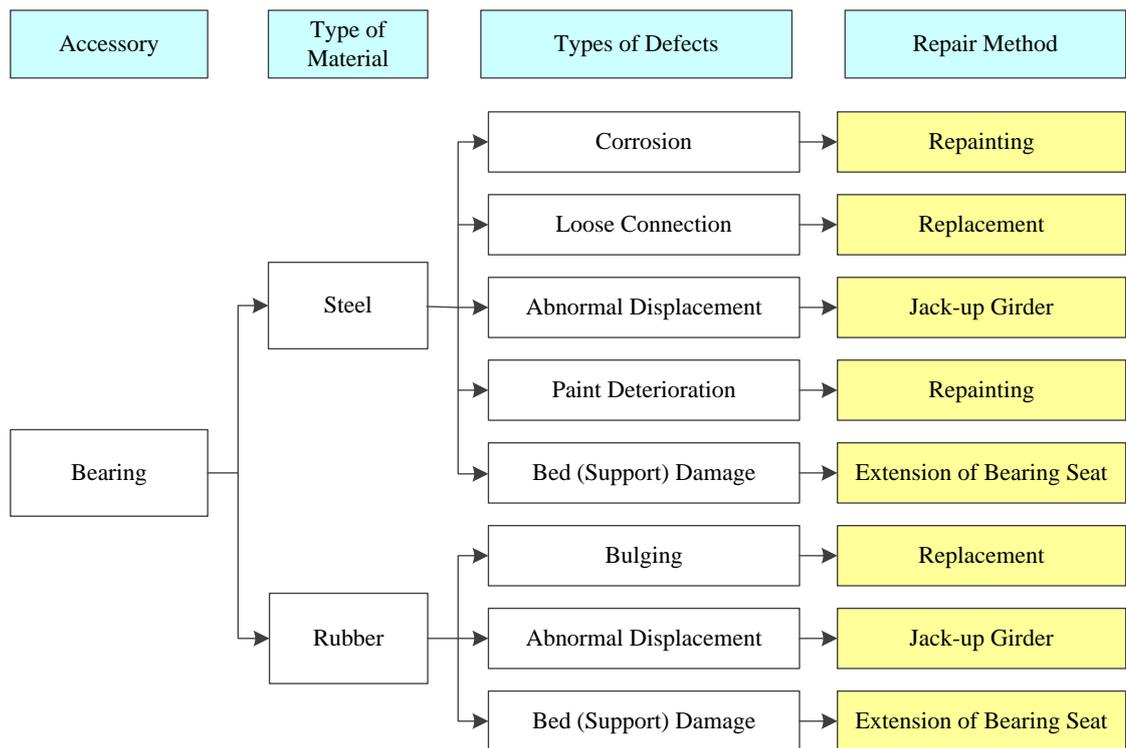
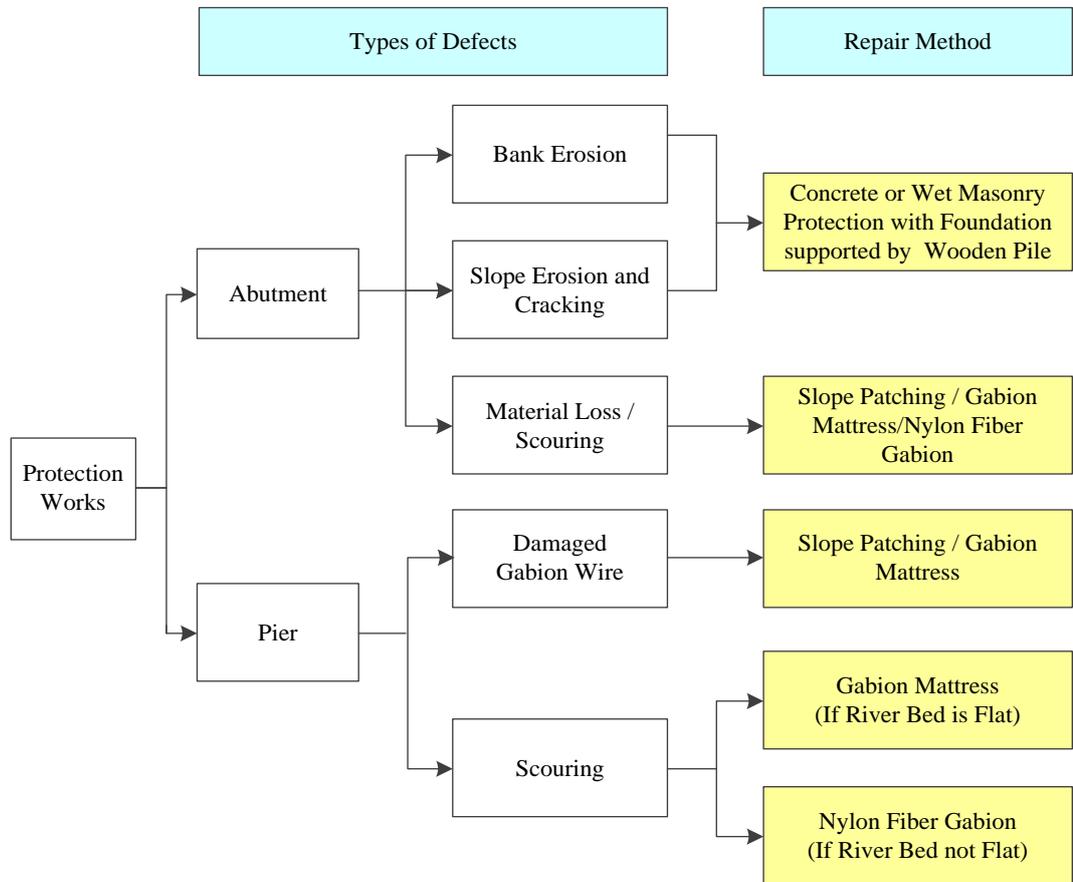


Figure 3-19 Selection Procedure of Repair Method for Bearings



Note: Each damage ranked with "Bad" in Condition of Rating Criteria will be repaired based on recommended Repair Method.

Figure 3-20 Selection Procedure of Repair Method for Protection Works