PRESTRESSED CONCRETE BRIDGES:
DESIGN AND CONSTRUCTION

Nigel R. Hewson

@Seismicisolation

Copyrighted material

Published by Thomas Telford Publishing, Thomas Telford Ltd,
1 Heron Quay, London E14 4JD.
URL: http://www.thomastelford.com

Distributors for Thomas Telford books are
USA: ASCE Press, 1801 Alexander Bell Drive, Reston, VA 20191-4400, USA
Japan: Maruzen Co. Ltd, Book Department, 3-10 Nihonbashi 2-chome,
Chuo-ku, Tokyo 103
Australia: DA Books and Journals, 648 Whitehorse Road, Mitcham 3132,
Victoria

First published 2003. Reprinted 2006

Also available from Thomas Telford Books
Manual of bridge engineering. Edited by M. J. Ryall, G. A. R. Parke and J. E. Harding.
ISBN 0 7277 2774 5
Bridge engineering: a global perspective. Leonardo Fernández Troyano. ISBN 0 7277 3215 3
Current and future trends in bridge design construction and maintenance 2. ICE.
ISBN 0 7277 3091 6

A catalogue record for this book is available from the British Library

ISBN: 0 7277 3223 4

© Thomas Telford Limited 2003

All rights, including translation, reserved. Except as permitted by the Copyright, Designs and Patents Act 1988, no part of this publication may be reproduced, stored in a retrieval system or transmitted in any form or by any means, electronic, mechanical, photocopying or otherwise, without the prior written permission of the Publishing Director, Thomas Telford Publishing, Thomas Telford Ltd, 1 Heron Quay, London E14 4JD.

This book is published on the understanding that the author is solely responsible for the statements made and opinions expressed in it and that its publication does not necessarily imply that such statements and/or opinions are or reflect the views or opinions of the publishers. While every effort has been made to ensure that the statements made and the opinions expressed in this publication provide a safe and accurate guide, no liability or responsibility can be accepted in this respect by the author or publishers.

Typeset by Academic + Technical, Bristol
Printed and bound in Great Britain by Antony Rowe Ltd, Chippenham, Wiltshire

@Seismicisolation

Dedication

To my wife Alison, who has visited many bridges over the years, and to my daughters Sarah and Laura, who may come to use them one day.

@Seismicisolation

Contents

Preface xiv

Disclaimer xvi

Acknowledgements xvii

List of figures xix

List of tables xxviii

1. Prestressed concrete in bridgeworks 1

Introduction 1
Principles of prestressing 4
Pre-tensioning 6
Post-tensioning 7
Brief history of prestressed concrete bridges 9
References 19

2. Prestressing components and equipment 20

Introduction 20
Proprietary systems 20
Wires 21
Strands and tendons 23
Bars 24
Anchorages 25
Tendon couplers 26
Ducting 28
Equipment for placing tendons 30
Stressing jacks 31
References 33

3. Durabilty and detailing 34

Introduction 34
Recent history of durability issues in the UK 34
Corrosion protection and ducting 36
Concrete 37

@Seismicisolation

Detailing 38
Access 40
Stray current protection 43
External tendon replacement 44
References 44

4. Grouting post-tensioned tendons 46

Introduction 46
Recent history of grouting 46
Grout material 47
Grout material tests 48
Grouting equipment 50
Vents and other details 51
Grouting trials 53
Pre-grouting test (pressure testing) of the ducting system 54
Air pressure testing 54
Water pressure testing 55
Leakages 56
Grouting procedure 57
Grease and wax grout 58
References 58

5. Prestress design 60

Introduction 60
General approach 60
Primary and secondary prestress effects 64
Prestress force and losses 67
Friction losses and tendon extension 67
Elastic shortening and strains 69
Relaxation of tendon steel 70
Creep losses 71
Shrinkage losses 72
Tendon eccentricity in ducts 73
Serviceability limit state stress check 73
Deflections and pre-camber 76
Vibrations and fatigue in tendons 76
Ultimate moment design 77
Shear design 79
Torsion design 81
Longitudinal shear 81
Partial prestressing 82
Construction sequence and creep analysis 83
Temperature effects 84
Concrete properties 87
Application of the prestress 88
Design procedures to BS 5400 88
Serviceability limit state stress check 89

@Seismicisolation

Ultimate moment resistance 89
Ultimate shear resistance 90
Ultimate torsion 91
Longitudinal shear 92
Partial prestressing 92
Design procedures to AASHTO standard specifications 93
Allowable stress design check 93
Flexural strength 94
Shear capacity 95
Torsion 95
Horizontal shear 96
References 96

6. Design of details 98

Introduction 98
Anchorages 98
Pre-tensioned strand 98
Post-tensioned tendons 99
Cast-in dead-end anchors for post-tensioned tendons 103
Anchor blisters or blocks 104
Anchor pockets 106
Couplers 107
Ducts 108
Diaphragms 110
Deviators 115
References 119

7. Concept design of prestressed concrete bridges 120

Introduction 120
Deck types 121
Selecting the deck arrangement 126
Articulation and span arrangements 128
Post-tensioning with internal or external tendons 129
Bridge costs 130
Material quantities 132
References 133

8. Analysis of prestressed concrete bridges 134

Introduction 134
Traditional approach to deriving forces, moments and shears 134
Dead load and applied loading analysis 135
Deriving the prestress forces and moments 137
Stage-by-stage and creep effects 140
Combining effects 141
Specialist software for the analysis of prestressed concrete bridges 141
General description 141

@Seismicisolation

Input data 143
Analysis of the structure 146
Output 150
Summary 150
References 150

9. Slab bridges 151

Introduction 151
Solid-slab bridges 152
Voided-slab bridges 152
Design of slab bridges 154
Reference 155

10. Beam-and-slab bridges 156

Introduction 156
General arrangement 160
Construction of *in situ* beam-and-slab decks 163
Casting and transportation of precast beams 165
Erection of precast beams 169
Casting of deck slab 171
Design of beam-and-slab decks 173
General design 173
Stress distribution through section 173
Precast beams in continuous and integral decks 176
Prestress and reinforcement 176
References 178

11. *In situ* multi-cell box girder decks 179

Introduction 179
General arrangement 180
Construction of *in situ* multi-cell box girders 181
Design of *in situ* multi-cell box girders 186
References 189

12. *In situ* single-cell box girder bridges 190

Introduction 190
General arrangement 191
Construction, span-by-span 194
Construction by balanced cantilever 197
Design of *in situ* single-cell box girders 201
Box behaviour 202
Prestress layout 202
Transverse prestressing of top slab 202
Deck articulation 204
Deck construction 205
References 205

@Seismicisolation

13. Precast segmental box girders **206**

Introduction 206
General arrangement 208
Casting of segments 210
Storage and transportation of segments 218
Segment erection, general 222
Segment erection by the balanced cantilever method 229
Segment erection by the span-by-span method 236
Segment erection by the progressive placement method 239
Design aspects associated with precast segmental decks 240
Prestress tendon layout 240
Shear keys at joint 240
Design with epoxy or concrete joints 243
Design with dry joints 244
Typical segment details 247
Deck erection 247
References 248

14. Precast full-length box girders **250**

Introduction 250
General arrangement 253
Casting and storage of the units 253
Transportation of the units 256
Erection of the units 258
Design of full-length precast box girder decks 261

15. Incrementally launched box girder bridges **262**

Introduction 262
General arrangement 264
Casting the deck 267
Launching the deck 271
Design aspects associated with launched box girder decks 276
Longitudinal design during launch 276
Transverse and local design during launch 278
Construction tolerances 279
Construction loading 279
Loads on supports during launching 279
References 281

16. Cable-stayed bridges **282**

Introduction 282
Cable-stayed bridge arrangements 284
Construction of concrete cable-stayed bridges 287
Box girder decks 289
Beam-and-slab decks 290
Design aspects associated with concrete cable-stayed bridges 295

@Seismicisolation

Analysis of cable-stayed bridges 295
Deck design and behaviour 298
Deck dynamic behaviour 302
Stays 303
Temporary loading 303
References 305

17. Other prestressed concrete bridge types 306

Introduction 306
Extra-dosed bridges 306
Fin-back bridges 308
Truss bridges 309
Arch bridges 310
Footbridges 312
References 313

18. Problems and failures 314

Introduction 314
Prestressing components 314
Wire, strand and tendon failures 314
Tendon extensions 315
Grouting and ducts 316
Corrosion 316
Concrete and reinforcement 317
Concrete cracks 317
Honeycombing 318
Concrete cover 320
Problems during construction 321
Failures due to design 321
Construction procedures 322
Structural behaviour problems 323
Problems after opening 324
Durability 324
Rehabilitation and modifications 324
References 324

Appendix A: Definitions 326

Appendix B: Symbols and notations used 328

Appendix C: Further reading and useful references 332

Introduction 332
Books 332
Design guides and technical reports 333
Articles 334
Standards, codes of practice and specifications 335
Websites 337

@Seismicisolation

Appendix D: Proprietary systems 339

Introduction 339
Multi-strand systems 339
External tendon systems 346
Flat-slab systems 348
Bar systems 350
Wire prestressing systems 354
Auxiliary equipment 357

Company and person index 361

Project index 362

Subject index 364

@Seismicisolation

Preface

This book is an expansion of the chapter on prestressed concrete bridges in the Manual of Bridge Engineering and seeks to give a wider coverage to the practical aspects involved in the design and construction of prestressed concrete bridge decks. Concrete remains the most common material for bridge construction around the world, and prestressed concrete is frequently the material of choice for bridge decks with spans greater than 25 m. As well as the more common highway and rail bridges, prestressed concrete has also been successfully used on some of the larger cable-stayed structures, major river crossings and urban viaducts.

Much has been learnt about prestressed concrete over the six decades since it was first used on bridgeworks and the current techniques employed in both their design and their construction have evolved greatly from those used by the early pioneers such as Freyssinet and Magnel. Higher strength concrete and improvements in the prestressing steels coupled with sophisticated design tools have given prestressed concrete a greater versatility.

There are many different ways to design and build prestressed concrete bridges and it is true to say every bridge is different in one way or another. All bridge designers have their own way of doing things and their own preferences in the design approach and the details to adopt, while individual contractors come up with a different solution to the same problem. No single publication can cover all the possible ways to design or build prestressed concrete bridges; however, this book presents the author's experiences, collected over 25 years in the industry.

Although there are several good publications covering general prestress concrete design, and many short articles and guidance notes on the different practical aspects of designing and constructing prestressed concrete bridges, there is little available bringing all this together. It is the aim of this book to combine all the aspects of prestressed concrete bridge decks into one volume.

Chapters 1 and 2 cover the general aspects of prestressing, its principles and the components that make up the prestressing systems. Chapters 3 and 4 consider durability issues, while Chapters 5 to 8 cover a range of general design issues. Chapters 9 to 17 discuss the design and construction of different deck forms and construction techniques. Chapter 18 looks briefly at some of the problems that have occurred in the past.

While reviewing the design and construction of the different types of prestressed concrete bridge decks and the prestressing systems used, this

@Seismicisolation

book assumes that the reader has a basic understanding of prestressed and reinforced concrete design, which can be applied to the specific application of bridges.

The author would welcome comments and dialogue on any of the subjects contained within this book, and he can be contacted by e-mail at nrh@tgp.co.uk

@Seismicisolation

Disclaimer

The information contained in this book is based on the experience of the author and his interpretation of current practice. All information and data contained should be checked and verified for use on any particular project.

@Seismicisolation

Acknowledgements

Where do you start when there have been so many friends and colleagues who have assisted me over the years, both in gaining the knowledge to write this book and in actively contributing to its contents? Perhaps I should start by thanking Graham Davenport, Keith Simm and Dick Thomas who gave me the initial encouragement and opportunity to become a bridge engineer, while all the people I have worked with have had some influence on this book's contents.

Many thanks to Tony Gee and Partners and Hyder Consulting Ltd without whom this book would not have been completed and for the many figures and photographs they provided.

Most of the drawings have been provided by Hyder Consulting, unless noted otherwise, with help from Ray Purvis and his team.

A special thanks also to Louise Smith for help with Chapters 3 and 4 on durability issues and grouting of tendons; to VSL, Freyssinet, DYWIDAG, BBR and McCall's for the information on their systems and for their permission to publish extracts from their brochures and to Sarah Hewson for help with some of the diagrams.

The author wishes to acknowledge and thank the following for the figures and drawings provided:

Andrew Barbour, Louise Smith, Alan Major, Francis Kung, Peter Fox, Martin Morris, Bill Hard, Tom Williams, Roger Knight, Jonathan Hiscock and all the others from Hyder who have provided photographs over many years.

Stephen Cardwell at VSL
Bernard Fortier at Campanon Bernard (now Vinci)
Flemming Pedersen and Robert Uthwatt at Cowi
Gordon Clark at Giffords
Paul Bottomley at Freyssinet
Dr Brenni at BBR
Bob Spackman and Stuart Brunton at TGP
Joe O'Donovan and Tony Dempsey
at Roughan & O'Donovan
Bijan Aalami at ADAPT
Ronald Yee at Yee and Associates
Jean-Philippe Mathieu at Bouygues

@Seismicisolation

Acknowledgements

Finally, a thank you to Brian Pope and Andy Hodgkinson for reading it all through to make sure it made sense.

I apologize should I have missed anyone who has helped or contributed to this book; it is not intentional but more a reflection on the time taken to bring it all together.

@Seismicisolation

List of figures

Chapter 1

Figure 1.1 Byker Viaduct, England
Figure 1.2 Ceirog Viaduct, Wales
Figure 1.3 River Dee crossing, Wales
Figure 1.4 Killarney Overbridge, Ireland
Figure 1.5 Viaduct constructed with precast beams
Figure 1.6 Prestressing building blocks
Figure 1.7 Change to stresses in beam
Figure 1.8 Precasting yard for the Bosporus crossing project, Turkey
Figure 1.9 Internal ducts and anchors prior to concreting
Figure 1.10 External tendons inside box girder deck
Figure 1.11 Stressing of post-tensioned tendon
Figure 1.12 Annet Bridge, France
Figure 1.13 Linn Cove Viaduct, USA
Figure 1.14 St James's Park Footbridge, England
Figure 1.15 M2 Medway Bridge, England
Figure 1.16 Dornoch Firth crossing, Scotland
Figure 1.17 Brotonne Bridge, France
Figure 1.18 Hung Hom Bypass, Hong Kong
Figure 1.19 Vasco da Gama Bridge, Portugal
Figure 1.20 Sunniberg Bridge, Switzerland
Figure 1.21 Longest prestressed concrete bridge span length *v* year
Figure 1.22 Sherbrooke Footbridge, Canada
Figure 1.23 Peace Footbridge, Korea

Chapter 2

Figure 2.1 Multi-strand tendon
Figure 2.2 Prestressing bar and anchor
Figure 2.3 Multi-strand tendon live-end anchor
Figure 2.4 Strand dead-end anchorage
Figure 2.5 Multi-strand tendon coupler
Figure 2.6 Prestress bar coupler
Figure 2.7 Steel ducts inside reinforcement cage
Figure 2.8 Corrugated plastic duct and coupler
Figure 2.9 HDPE duct

@Seismicisolation

Figure 2.10 Push-through placing of strand
Figure 2.11 Pull-through placing of strand
Figure 2.12 Jack for stressing single strand
Figure 2.13 Jack for multi-strand tendon
Figure 2.14 Jack for prestressing bar
Figure 2.15 Placing of large prestressing jack onto tendon

Chapter 3

Figure 3.1 Multi-strand tendon protection systems
Figure 3.2 Soffit drainage hole
Figure 3.3 Deck drainage
Figure 3.4 Bridge inspection unit
Figure 3.5 Soffit access hole
Figure 3.6 Abutment inspection gallery
Figure 3.7 Access through box girder diaphragms

Chapter 4

Figure 4.1 Flow-cone test
Figure 4.2 Inclined duct test
Figure 4.3 Grout mixing equipment
Figure 4.4 Anchor cap with grout inlet
Figure 4.5 Concrete capping to anchorage
Figure 4.6 Grout vents

Chapter 5

Figure 5.1 Precast beam prestressing
Figure 5.2 Prestressing bars used for segment erection
Figure 5.3 Typical post-tensioning layout
Figure 5.4 External tendons inside box girder
Figure 5.5 Prestress in continuous decks
Figure 5.6 Secondary effects due to built-in supports
Figure 5.7 Prestress stresses on section
Figure 5.8 Tendon-force profile
Figure 5.9 Relaxation losses in strands *v* initial stress
Figure 5.10 Stress distribution due to shear lag
Figure 5.11 Effective flange width
Figure 5.12 Ultimate moment of resistance
Figure 5.13 Contribution of haunched slab to shear resistance
Figure 5.14 Longitudinal shear
Figure 5.15 Creep redistribution of moments
Figure 5.16 Stresses generated from temperature gradient
Figure 5.17 Cube *v* cylinder strengths

@Seismicisolation

Chapter 6

Figure 6.1 Pre-tensioned strand at the end of a precast beam
Figure 6.2 End block design
Figure 6.3 End block reinforcement with internal ducts
Figure 6.4 Spread of stress into flanges
Figure 6.5 Dead-end anchorage reinforcement
Figure 6.6 Blister and anchor block design
Figure 6.7 Blister reinforcement
Figure 6.8 Three-dimensional modelling of anchor blister
Figure 6.9 Restraining curved ducts
Figure 6.10 Tendons in curved bottom slab
Figure 6.11 Ducts for external tendons
Figure 6.12 Diaphragm arrangements
Figure 6.13 Diaphragm hanging reinforcement
Figure 6.14 Vertical prestress in diaphragm
Figure 6.15 'U' beam diaphragm
Figure 6.16 Typical box girder intermediate diaphragm
Figure 6.17 Deviator arrangements
Figure 6.18 Concrete beam deviator
Figure 6.19 Concrete block deviator
Figure 6.20 Steel deviator
Figure 6.21 Three-dimensional finite element analysis of deviator

Chapter 7

Figure 7.1 Span ranges for different deck types
Figure 7.2 Precast beams
Figure 7.3 *In situ* box girder
Figure 7.4 Precast segmental box girder
Figure 7.5 Incrementally launched box girder
Figure 7.6 Cable-stayed bridge
Figure 7.7 Choice for single span decks
Figure 7.8 Choice for multi-span viaducts
Figure 7.9 Optimizing bridge costs
Figure 7.10 Typical quantities in prestressed concrete decks

Chapter 8

Figure 8.1 Grillage model for analysis
Figure 8.2 Longitudinal dead load moment from frame analysis
Figure 8.3 Frame model for transverse moment analysis
Figure 8.4 Full length three-dimensional finite element model
Figure 8.5 Tendon friction-loss spreadsheet
Figure 8.6 Prestress moments from influence coefficients
Figure 8.7 *ADAPT* menu window
Figure 8.8 *ADAPT* traveller definition
Figure 8.9 Graphical representation of the *ADAPT* model

@Seismicisolation

Figure 8.10 Stage-by-stage construction by *ADAPT*
Figure 8.11 Stage-by-stage construction with a traveller
Figure 8.12 *ADAPT* output

Chapter 9

Figure 9.1 Balbriggan Bridge, Ireland
Figure 9.2 M4 Overbridge, Ireland
Figure 9.3 Voided-slab deck section
Figure 9.4 Void formers
Figure 9.5 Weights holding void formers down

Chapter 10

Figure 10.1 Artist impression of the Bangkok SES viaduct
Figure 10.2 Analee Bridge, Ireland
Figure 10.3 Ghantoot Interchange Overbridge, United Arab Emirates
Figure 10.4 Precast beam bridge under construction over river
Figure 10.5 Precast beam bridge over railway
Figure 10.6 Typical precast beam arrangements
Figure 10.7 Typical precast beam deck section
Figure 10.8 Precast beam span range
Figure 10.9 *In situ* ladder beam arrangement
Figure 10.10 Beam continuity at pier
Figure 10.11 Precast beams on crosshead prior to casting diaphragms and deck slab
Figure 10.12 Falsework for *in situ* beams and deck slab
Figure 10.13 Overhead gantry for *in situ* beam and slab deck
Figure 10.14 Casting bed for pre-tensioned beams
Figure 10.15 Jacking frame at end of casting bed with inset of strand jack
Figure 10.16 Shutter with strands anchored at ends
Figure 10.17 Shutter for post-tensioned precast beam
Figure 10.18 Stressing post-tensioned beam
Figure 10.19 Beam transporter
Figure 10.20 Beam placed by crane
Figure 10.21 Gantry for placing beams
Figure 10.22 Formwork and reinforcement being placed
Figure 10.23 Deck slab being cast
Figure 10.24 Stresses in precast beam deck
Figure 10.25 Typical reinforcement and prestress arrangement
Figure 10.26 Typical deck slab reinforcement

Chapter 11

Figure 11.1 Kwun Tong Bypass, Hong Kong
Figure 11.2 Typical box cross-section
Figure 11.3 Multi-cell box girder bridge
Figure 11.4 Full-height scaffolding

@Seismicisolation

Figure 11.5 Formwork being installed
Figure 11.6 Reinforcement and tendon ducts
Figure 11.7 Bottom slab, webs and diaphragm cast
Figure 11.8 Stage-by-stage construction
Figure 11.9 Deck finishing machine
Figure 11.10 Balanced cantilever construction of multi-cell box
Figure 11.11 Three-dimensional finite element model of deck
Figure 11.12 Longitudinal tendon profile
Figure 11.13 Typical reinforcement arrangement

Chapter 12

Figure 12.1 MTR Island Line, Hong Kong
Figure 12.2 Malaysia Singapore Second Crossing
Figure 12.3 Typical single-cell box cross-section
Figure 12.4 Twin-box arrangement
Figure 12.5 Single-cell box with struts supporting side cantilevers
Figure 12.6 Full-height scaffolding from the ground
Figure 12.7 Gantry support for formwork
Figure 12.8 Reinforcement, ducts and web shutters being placed
Figure 12.9 Shutter for top slab between webs
Figure 12.10 Balanced cantilever construction
Figure 12.11 Balanced cantilever construction of deck
Figure 12.12 Pierhead and setting up of the traveller
Figure 12.13 Travelling form arrangement
Figure 12.14 Travelling form on deck
Figure 12.15 Balanced cantilever construction with bearings
Figure 12.16 Tendon layout for balanced cantilever construction
Figure 12.17 Transverse tendons profile and anchorage

Chapter 13

Figure 13.1 Byker Viaduct, England
Figure 13.2 Belfast Cross Harbour Links, Northern Ireland
Figure 13.3 Bangkok Second Expressway System, Thailand
Figure 13.4 Hung Hom Bypass and Princess Margaret Road Links, Hong Kong
Figure 13.5 Jahra-Ghazali Viaduct, Kuwait
Figure 13.6 General view inside casting yard
Figure 13.7 Casting cell schematic layout
Figure 13.8 Short-line casting bed
Figure 13.9 Inner shutter ready to slide into position
Figure 13.10 Counter-cast segment positioned
Figure 13.11 Segments on long-line casting bed
Figure 13.12 Long-line formwork
Figure 13.13 Reinforcement assembly jig
Figure 13.14 Reinforcement cage stored ready to be moved to casting cell
Figure 13.15 Inflated tubes in ducts during segment casting

@Seismicisolation

Figure 13.16 Segment setting out
Figure 13.17 Survey pins
Figure 13.18 Segments in storage yard
Figure 13.19 Segment on special carrier
Figure 13.20 Segments lifted by crane at storage yard
Figure 13.21 Cast-in lifting hooks
Figure 13.22 Segment positioned on rail-mounted bogey
Figure 13.23 Segment transported by barge
Figure 13.24 Precast segmental deck erection techniques
Figure 13.25 Epoxy being spread over joint
Figure 13.26 'O' rings around ducts at segment joints
Figure 13.27 Temporary prestress for segments
Figure 13.28 View inside box girder as segment is being positioned
Figure 13.29 Close-up of joint after segment erected
Figure 13.30 Dry-jointed segmental deck
Figure 13.31 Details of seal along top of dry joint
Figure 13.32 Spalling of concrete edges at joint between segments
Figure 13.33 Balanced cantilever erection using a gantry
Figure 13.34 Segment being positioned
Figure 13.35 Precast segments erected by crane
Figure 13.36 Precast segments erected by ground-level gantry
Figure 13.37 Precast segments erected by lifting frame
Figure 13.38 Prop and tie to stabilize cantilever
Figure 13.39 Stabilizing bracket around pier
Figure 13.40 Shutter for *in situ* stitch at midspan
Figure 13.41 Frame to align cantilevers
Figure 13.42 Overhead gantry for span-by-span erection
Figure 13.43 Segments hanging from gantry
Figure 13.44 Underslung truss for span-by-span erection
Figure 13.45 Lifting arm on underslung gantry
Figure 13.46 Progressive placing of segments with props
Figure 13.47 Prestress layout for balanced cantilever deck
Figure 13.48 Prestress layout for simply supported deck
Figure 13.49 Shear key arrangements
Figure 13.50 Reinforcement for large shear keys
Figure 13.51 Ultimate behaviour of beam with dry joints and external tendons
Figure 13.52 Typical segment arrangement
Figure 13.53 Web and bottom slab transition detail

Chapter 14

Figure 14.1 Saudi Arabia–Bahrain Causeway
Figure 14.2 Vasco da Gama crossing, Portugal
Figure 14.3 Singapore MRT
Figure 14.4 Confederation Bridge, Canada
Figure 14.5 Casting cell shutters
Figure 14.6 Pre-assembled reinforcement

@Seismicisolation

Figure 14.7 Unit being concreted
Figure 14.8 Completed precast unit stored
Figure 14.9 Beam with lifting gantry
Figure 14.10 Beam transporter
Figure 14.11 Beam being moved on to deck
Figure 14.12 Beam being transported by barge
Figure 14.13 Placing beams by gantry
Figure 14.14 Cantilevered deck being lifted into place
Figure 14.15 Deck on bearings
Figure 14.16 Stitch for deck continuity

Chapter 15

Figure 15.1 Dornock Firth viaduct, Scotland
Figure 15.2 Hasdel Viaduct, Turkey
Figure 15.3 Deck during launch over piers
Figure 15.4 Typical launched deck section
Figure 15.5 Tendon anchorage details
Figure 15.6 Casting area behind abutment
Figure 15.7 Typical casting and launching sequence for deck
Figure 15.8 Formwork arrangement
Figure 15.9 Outer formwork assembled
Figure 15.10 Casting cell looking towards deck
Figure 15.11 Concreting the deck slab
Figure 15.12 Temporary bearings for launch
Figure 15.13 Combined permanent and launch bearings
Figure 15.14 Push launching jack
Figure 15.15 Pulling arrangement
Figure 15.16 Guides fixed to piers and bearing plinths being prepared
Figure 15.17 Launching nose on deck
Figure 15.18 Bending moments in deck during launching
Figure 15.19 Bending moment range in deck during launching
Figure 15.20 Analysis of bottom slab and web during launch
Figure 15.21 Forces on substructure during launching

Chapter 16

Figure 16.1 Sunshine Skyway Bridge, USA
Figure 16.2 Vasco da Gama Crossing, Portugal
Figure 16.3 River Dee Crossing, Wales
Figure 16.4 Concrete deck arrangements for cable-stayed bridges
Figure 16.5 Multi-stay arrangements
Figure 16.6 Balanced cantilever construction
Figure 16.7 Deck cast partly on falsework and partly with traveller
Figure 16.8 Lifting deck segments into place
Figure 16.9 Vasco da Gama Crossing cable-stayed bridge layout
Figure 16.10 Yamuna cable-stayed bridge layout
Figure 16.11 Form traveller for casting deck

@Seismicisolation

Figure 16.12 Form travellers being lifted into position
Figure 16.13 Beam-and-slab deck under construction
Figure 16.14 Prefabricated reinforcement cage
Figure 16.15 Stays being installed
Figure 16.16 Simplistic analysis model
Figure 16.17 Three-dimensional analysis model
Figure 16.18 Deflected shape of cable-stayed deck
Figure 16.19 Dead load bending moment profile
Figure 16.20 Axial force in deck
Figure 16.21 Bending moments during deck cantilevering
Figure 16.22 Deck deflections during cantilevering
Figure 16.23 Stay anchorage arrangement

Chapter 17

Figure 17.1 Extra-dosed and fin-back bridge arrangements
Figure 17.2 Tsukuhara Bridge, Japan
Figure 17.3 Sunniberg Bridge, Switzerland
Figure 17.4 Barton Creek Bridge, USA
Figure 17.5 Bubiyan Bridge, Kuwait
Figure 17.6 Boulonnais Bridge, France
Figure 17.7 Vecchio Bridge, Corsica
Figure 17.8 Gladesville Bridge, Australia
Figure 17.9 Barelang Bridge, Indonesia
Figure 17.10 Footbridge, Singapore
Figure 17.11 Footbridge, Hong Kong
Figure 17.12 Kilmacanogue stressed-ribbon footbridge, Ireland

Chapter 18

Figure 18.1 Incomplete grouting
Figure 18.2 Spalling at deviator
Figure 18.3 Deviator failure
Figure 18.4 Honeycombing around anchors
Figure 18.5 Congested reinforcement
Figure 18.6 Honeycombing and voids in concrete
Figure 18.7 Precast beam dropped
Figure 18.8 Segments toppling in storage yard

Appendix D

Figure D1 VSL typical tendon and steel duct sizes
Figure D2 VSL PT-PLUS™ duct sizes
Figure D3 VSL anchorage type EC
Figure D4 VSL dead-end anchorage type H
Figure D5 VSL couplers types K and V
Figure D6 VSL jack details
Figure D7 VSL anchorage block-out and jack clearances

@Seismicisolation

Figure D8 VSL 'external' stressing anchorage type Ed
Figure D9 VSL 'external' stressing anchorage type Edm
Figure D10 Freyssinet slabstress tendons
Figure D11 DYWIDAG prestressing bars
Figure D12 DYWIDAG bar post-tensioning details
Figure D13 DYWIDAG bar anchor details
Figure D14 DYWIDAG bar coupler
Figure D15 DYWIDAG bar duct sizes
Figure D16 DYWIDAG bar jack data
Figure D17 BBRV wire tendons
Figure D18 BBRV standard wire tendons
Figure D19 BBRV wire anchorages
Figure D20 BBRV wire couplers
Figure D21 BBRV wire jack details
Figure D22 DYWIDAG strand-pushing equipment
Figure D23 DYWIDAG hydraulic pump data
Figure D24 DYWIDAG grouting equipment

@Seismicisolation

List of tables

Chapter 2

Table 2.1 Wire, strand and bar properties
Table 2.2 Multi-strand tendon sizes

Chapter 5

Table 5.1 Typical friction coefficients
Table 5.2 Tendon offset inside duct

@Seismicisolation

1

Prestressed concrete in bridgeworks

Introduction

Concrete is strong in compression but weak in tension; however, prestressing can be used to ensure that it remains within its tensile and compressive capacity under the range of loading applied. Prestressing in bridgeworks is normally applied as an external force to the concrete by the use of wires, strands or bars, and can greatly increase the strength of the concrete alone. This can result in longer or slender spans, which improves aesthetics while providing economy in the construction. There are many fine examples in the UK and around the world of bridges and viaducts with prestressed concrete decks, such as the Byker Viaduct in Fig. 1.1, built utilizing precast match-cast segments, the Ceiriog Viaduct box girder, in Fig. 1.2, which was launched into position, and the River Dee crossing in Fig. 1.3, with a cable-stayed deck. On motorways and other highway networks, numerous

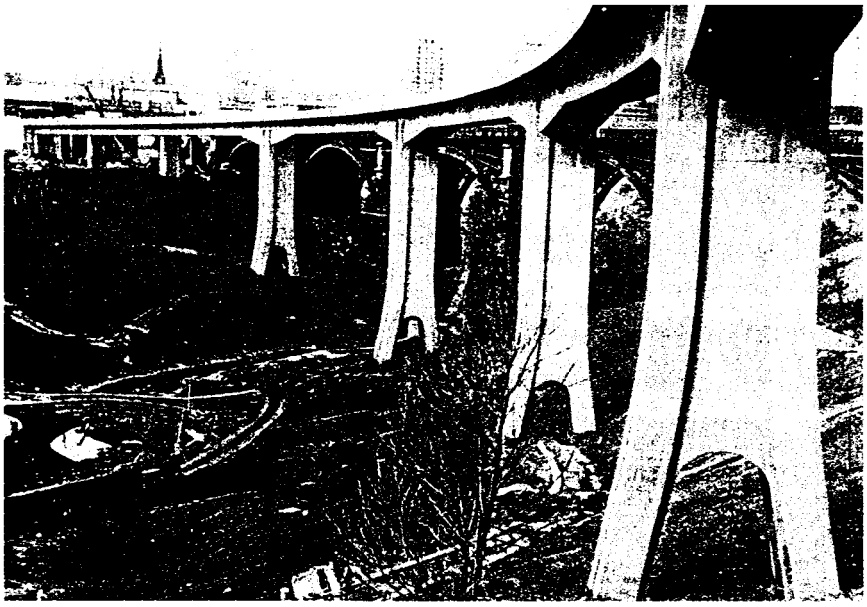


Figure 1.1. Byker Viaduct, England.

@Seismicisolation

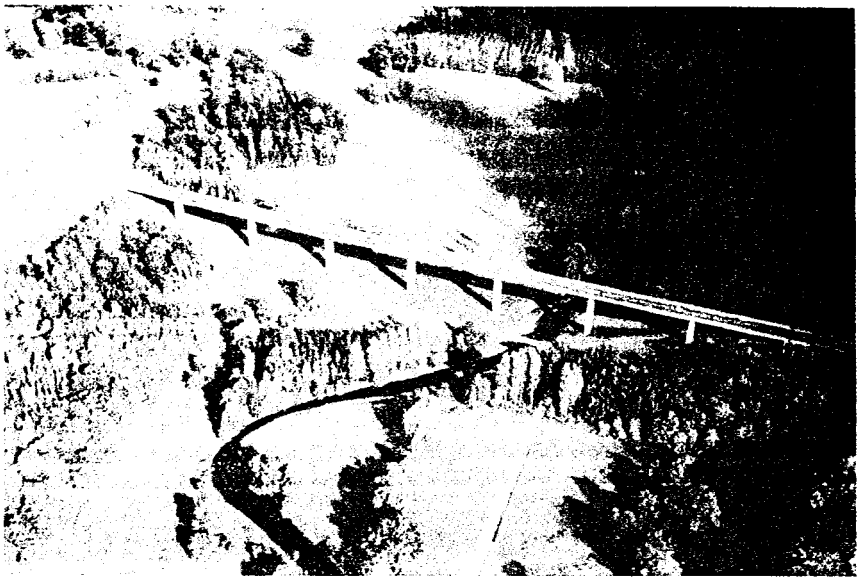


Figure 1.2. Ceiriog Viaduct, Wales. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

overbridges and underbridges, including the Killarney Overbridge in Fig. 1.4, have also utilized prestressing with either *in situ* or precast concrete beams.

Development of prestressed concrete bridges has given the bridge engineer increased flexibility in his selection of bridge form and in the construction techniques available. As a result, prestressed concrete is frequently the material of choice for bridges. Spans range from less than 25 m using precast beams, as in Fig. 1.5, to more than 450 m for cable-stayed bridges.

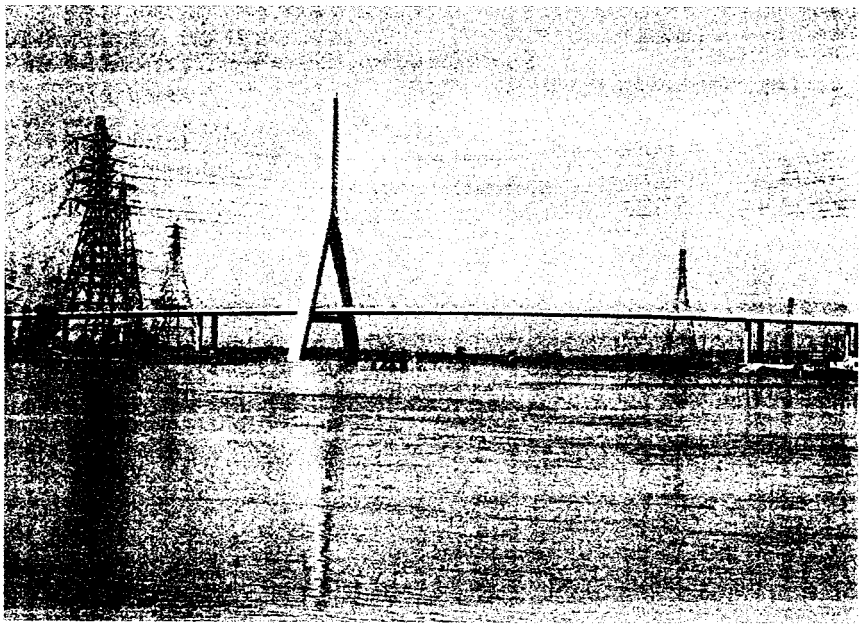


Figure 1.3. River Dee crossing, Wales. Reproduced courtesy of Gifford. Copyright reserved.

@Seismicisolation

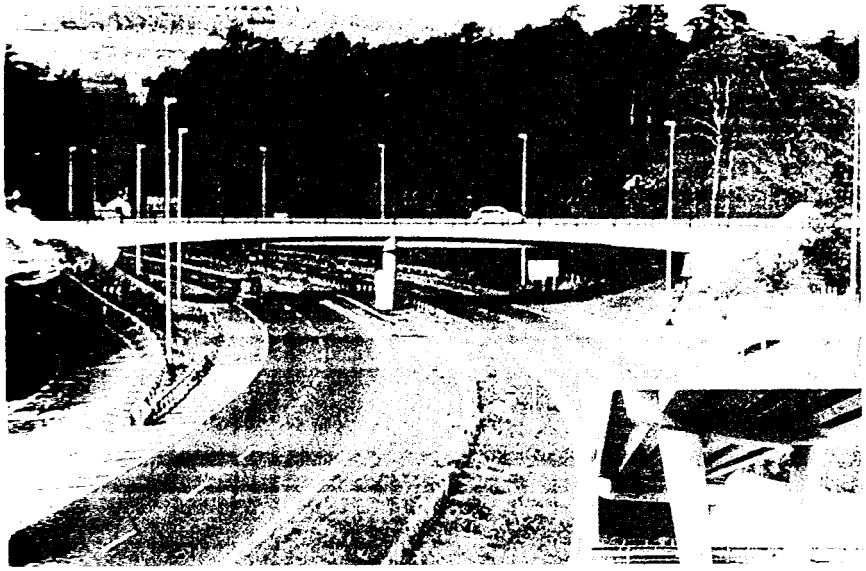


Figure 1.4. Killarney Overbridge, Ireland. Reproduced courtesy of Roughan & O'Donovan, Dublin. Copyright reserved.

Prestressed concrete bridges include a wide variety of different forms: from cast-in-situ to precast; from beams to box girders; and from simply supported to cable-stayed. Their functions range from the carrying of pedestrians and cyclists to road or rail traffic and they make up a significant proportion of the bridge stock in existence today.

The design of prestressed concrete bridges both greatly influences and is dependent on the construction process envisaged. The construction sequence and practical considerations in positioning the tendons influence the prestress layout much more than the desire to achieve a concordant profile. The deck layout and concrete shape is often dictated by the placement of the prestress

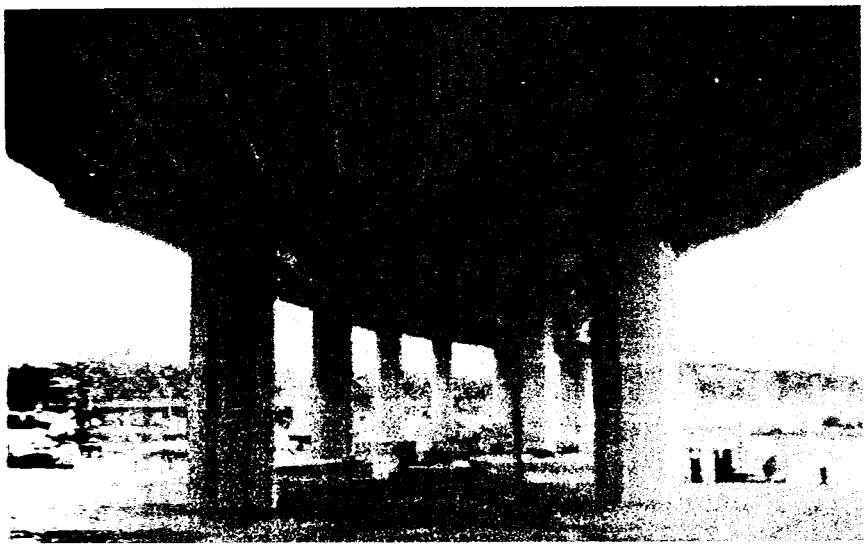


Figure 1.5. Viaduct constructed with precast beams.

@Seismicisolation

tendons and their anchorages, while the need for rapid construction or difficulties with access may dictate the type of structure and construction methodology adopted. On the other hand, the assumptions made in the analysis and design may govern the construction method and sequence adopted. Solutions using precast segments (see Chapter 13) are used over difficult ground or where rapid construction is needed. The deck may be launched (see Chapter 15) over a deep valley to reduce the need for falsework and to minimize the disruption and environmental impact on the landscape below. Precast beams (see Chapter 10) are used where access to the ground underneath is restricted, and stay cables (see Chapter 16) allow longer spans and reduce the number of piers. In all cases the design and the construction methodology must be developed together.

Durability and maintenance requirements feature prominently in any design produced today as lessons learnt from problems with existing bridges are fed back into the industry. Aesthetics are another important consideration, as designers strive to improve the appearance of bridges to fit into the environment. Today's bridge designer has to take into account all these factors in order to develop the optimum solution for any particular bridge project undertaken.

After deciding the type of bridge structure to be used and the construction approach to be adopted, the design of the prestressed concrete deck is carried out. This usually involves the use of specialist software, although hand calculations or spreadsheets are valid approaches that are still widely used by many designers. The analysis must accurately model the effect of the prestress and applied load on the structure, to ensure that all the strength and serviceability requirements are met. This often includes three-dimensional finite-element modelling, as well as stage-by-stage analysis with the creep and shrinkage behaviour included.

The many aspects that contribute to the design and construction of prestressed concrete bridge decks, including the materials and equipment used, the design requirements and procedures, and the construction techniques adopted, are covered in subsequent chapters. In this first chapter only the basic principles of prestressing are discussed, and a brief history of prestressing and prestressed concrete bridges presented.

Principles of prestressing

Force transfer between the prestress tendon and the concrete achieves prestressing of the concrete member. Tendons are pulled and stretched and then anchored against the concrete, with the tension in the tendon being balanced by compression in the concrete. In this way an external compressive force is applied to the concrete and is used to counter the tensile stresses generated under the bending moments and shear forces present.

The tendons are placed either within the concrete member as internal tendons, or alongside the concrete as external tendons, and can be unbonded or bonded to the concrete. They can be pre-tensioned or post-tensioned and consist of wires, strands or bars; however, their effect on the concrete, and the basic principles of design, are the same in all cases.

@Seismicisolation

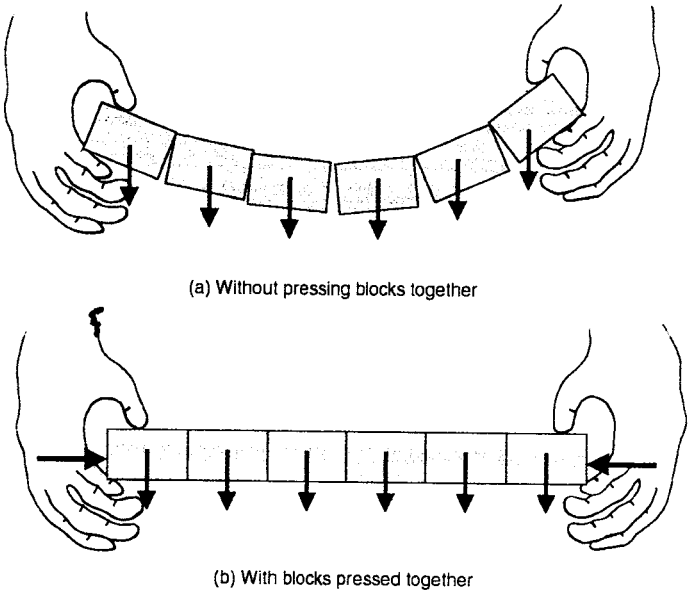


Figure 1.6. Prestressing building blocks.

The effect of prestressing can be demonstrated by attempting to pick up a row of children's building blocks, as depicted in Fig. 1.6. By pushing the ends together, and applying a 'prestressing' force to the blocks, it is possible to pick them up; however, if no force were applied to the ends then the row of bricks would fall apart. The joints between the blocks are unable to resist tension and, without the 'prestressing' force, they open up under their self-weight. With the 'prestressing' force applied the joints are kept in compression over their full depth.

The same principles apply to bridge decks where, in Fig. 1.7(a), the non-prestressed concrete beam would be subjected to tension along the

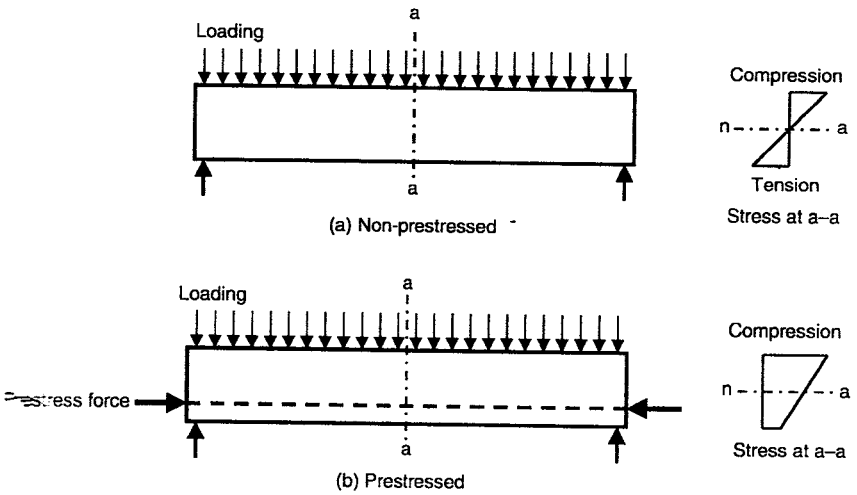


Figure 1.7. Change to stresses in beam.

@Seismicisolation

bottom fibre of the span due to the bending moments generated by the applied load. With a 'prestress' force applied as indicated in Fig. 1.7(b), the tension can be eliminated and the concrete kept in compression over the full section.

The concrete can be either fully prestressed, which ensures that the longitudinal stresses are always in compression, or partially prestressed, which allows some small tension to occur within the concrete under certain loading conditions.

Pre-tensioning

For pre-tensioning, the tendon is stressed by jacking against an anchor frame before the concrete is placed. The force is released from the anchor frame into the concrete when the concrete has obtained sufficient strength. Used primarily in precast beams, the tendons usually consist of 7-wire strand or individual wires.

The precasting beds can be hundreds of metres long and provide space to cast several beams in one go, or they can be arranged to cast one beam at a time. The beds have soffit and side shutters that are arranged around the prestressing tendons, which are tensioned before fixing the reinforcement and placing the concrete. Figure 1.8 shows a typical precasting bed arrangement, with the beam side shutters being prepared in the centre of the picture

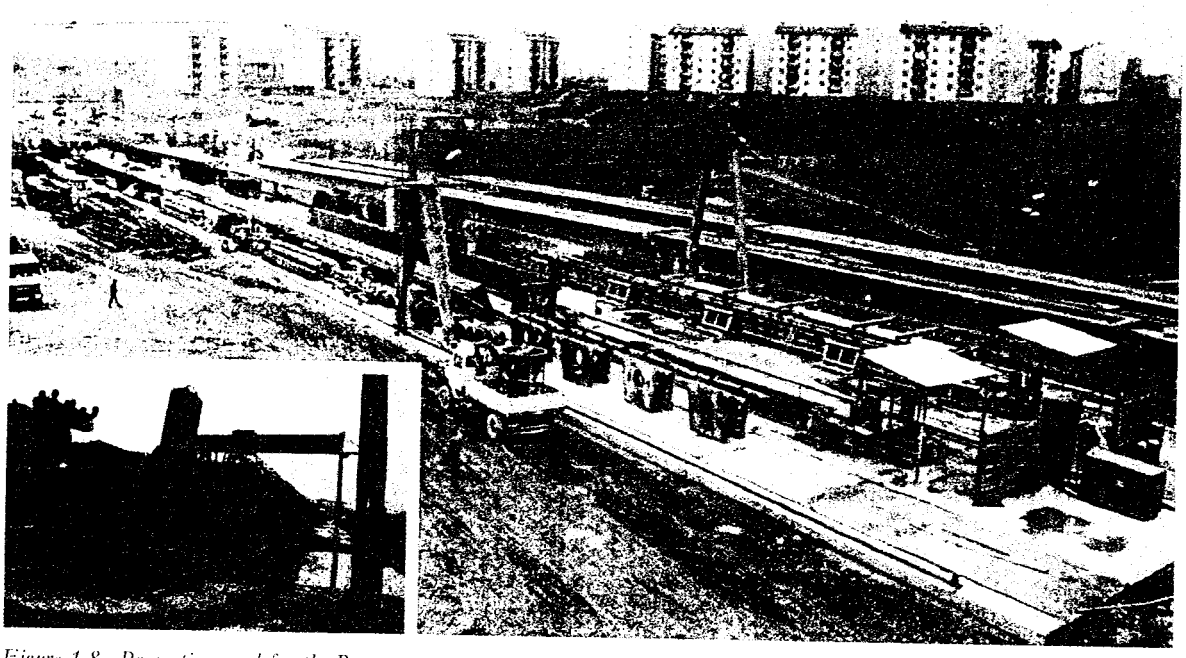


Figure 1.8. Precasting yard for the Bosporus crossing project, Turkey.

@Seismicisolation

and the jacking frame at the lower right corner. At the far end of the bed an anchor frame is needed to hold the strands when they are stressed, which is shown in the inset in Fig. 1.8.

After the concrete in the beams has reached the required transfer strength, the strands or wires are released from the jacking frames. The force in the strands transfers to the beams by the bond between the strand and the concrete over the anchorage length at the beam ends.

Post-tensioning

For post-tensioning the tendon is pulled and stretched using a jack and the resulting force is transferred directly on to the hardened concrete through the tendon anchor. The tendons consist of bars, single strands or multi-strands that can be arranged with a varying vertical and horizontal profile along the bridge deck, which allows for the most efficient arrangements of prestressing to be adopted. Post-tensioning can be used on many different types of structure, including precast beams, *in situ* or precast box girders and cable-stayed structures.

Internal tendons are placed inside a duct cast into the concrete and installed either before or after the concrete is placed, while external tendons are installed in ducts placed outside the concrete section after the concreting has been completed. Figure 1.9 shows internal prestressing ducts and the

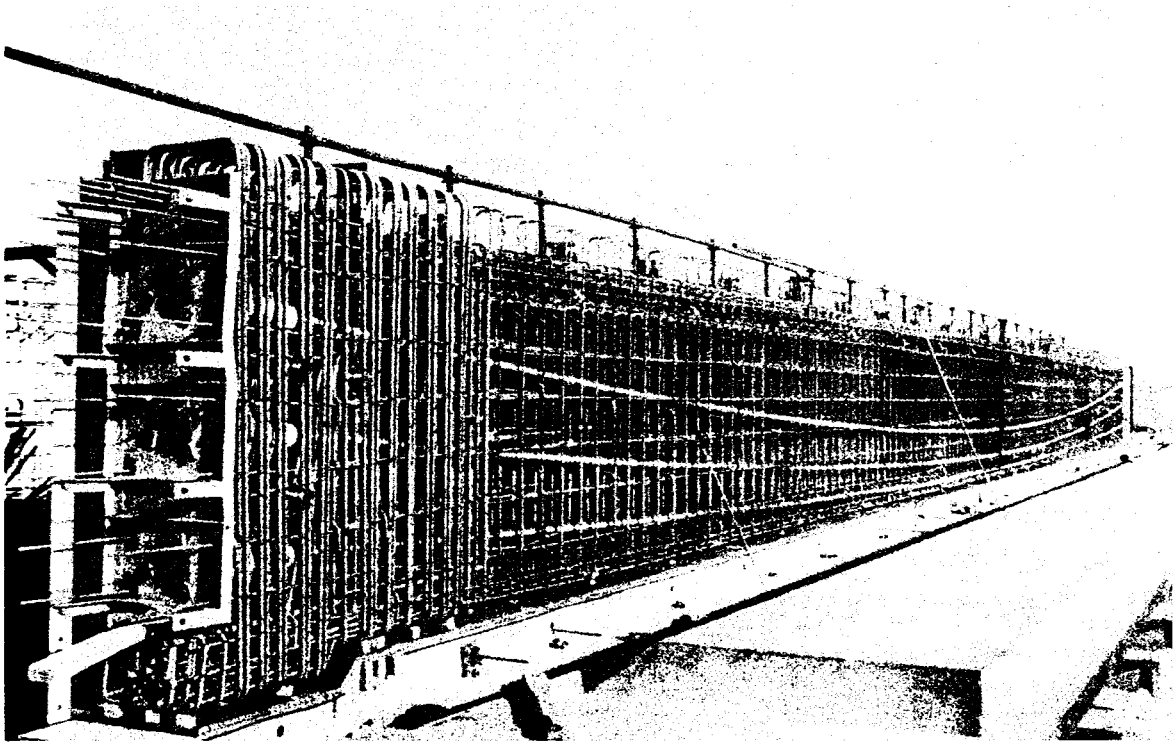


Figure 1.9. Internal ducts and anchors prior to concreting. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

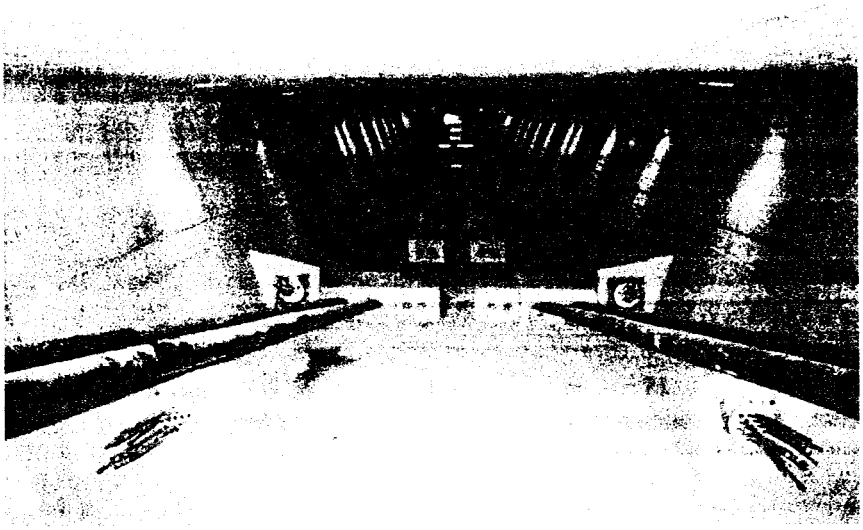


Figure 1.10. External tendons inside box girder deck.

prestressing anchorage arrangements installed in a beam prior to concreting, while Fig. 1.10 shows external tendons running inside a box girder adjacent to internal tendons anchored on blisters.

The force is generated in a post-tensioned tendon by the use of a stressing jack, as shown in Fig. 1.11 for a multi-strand tendon. The jack pulls the strands and transfers the force from the tendon on to the anchor plate and into the concrete. Reinforcement in the surrounding concrete resists the local tensile stresses around the anchor and assists in transferring the force into the deck.

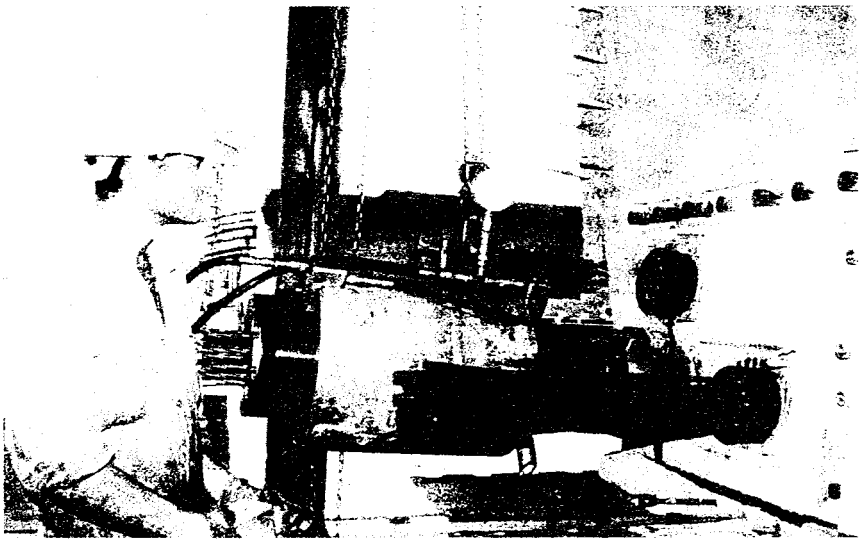


Figure 1.11. Stressing of post-tensioned tendon. Reproduced courtesy of Freyssinet International. Copyright reserved.

@Seismicisolation

Brief history of prestressed concrete bridges

Prestressing of concrete dates back to the early 1900s, when several engineers experimented with the technique, but it was Eugène Freyssinet (1879–1962) who first applied for a patent in 1928 covering the principle. Freyssinet is reported to have built an experimental arch in 1908, which incorporated prestressing tendons, and in 1930 he utilized prestressing during the construction of the Plougastel Bridge in France. In Germany, Dyckerhoff and Widmann used post-tensioned bars during the construction of the arch bridge at Alsleben in 1927.

During this time, much was still being learnt about prestressed concrete and the nature of time dependent effects. Engineers were experimenting with ways of using prestressed concrete in building and other applications. The first prestressed concrete bridges were built in the mid 1930s with the Oued Fodda Bridge in Algeria (1936), Aux Bridge in Saxony (1936) and the Oella Bridge in Germany (1938) leading the way, but it was not until after the Second World War that prestressed concrete became firmly established in bridge building.

Eugène Freyssinet is generally considered the father of modern-day prestressing, and it was his six bridges across the Marne, France, built between 1945 and 1950, which established the technique. The first of these, the Luzancy Bridge is a 55 m-long structure built as a 'frame' and integral with the abutments. The remaining five are each 73 m long and again built as a frame with the deck gently curving up towards the centre, where the deck depth is only 1 m. One of these, the Annet Bridge, is shown in Fig. 1.12. The decks are either a concrete box or concrete 'I' beam arrangement, with prestressing being used vertically in the webs as well as horizontally along the deck.

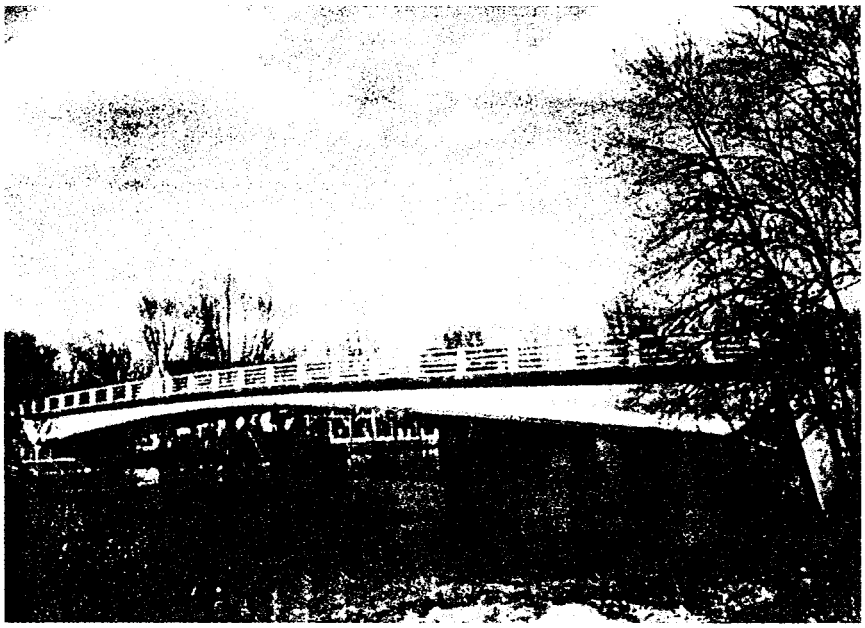


Figure 1.12. Annet Bridge, France. Reproduced courtesy of Freyssinet International. Copyright reserved.

@Seismicisolation

Prestressed concrete bridges

After the Second World War, with a shortage of steel in Europe, the use of prestressed concrete became popular in the reconstruction of bridges across the continent, and the 1950s saw prestressed concrete being used widely for bridgeworks. In this period Eugène Freyssinet continued to design many new prestressed concrete bridges, while Gustave Magnel developed the technique on several notable structures in Belgium. In the UK, L.G. Mouchel and Partners, in collaboration with the Prestressed Concrete Company, used prestressed concrete for a number of projects, including bridges. In Germany, Fritz Leonhardt was a leading exponent of prestressed concrete. The new material quickly caught on and its application was developed by many notable engineers of that time.

In 1939 Freyssinet developed and patented the first conical friction anchor. After 1945 other systems, such as those produced by Magnel-Blaton, BBRV and Lee-McCall, began to appear, as prestressing of concrete became popular. The early prestressing systems used were comprised of wires usually of 5 mm or 7 mm diameter, tensioned and anchored by a gripping device at the ends that transferred the load to the concrete. With the BBRV system, the wires were fitted with button heads at the ends to hold the individual wires against the anchor head, a system that is still used by BBR today. Towards the late 1950s, more systems using bars were developed; while the use of the wire systems evolved, with larger tendons employing wire-by-wire stressing and wedge anchors.

The very early attempts at prestressing concrete used normal steel as 'tie rods', and were not very successful as the low level of prestress in the steel was lost due to the shrinkage and creep in the concrete. By the time that prestressing was beginning to be used more widely in bridgeworks, higher strength steel wires were available with ultimate strengths of up to 1725 N/mm^2 and a yield limit of 1240 N/mm^2. Low relaxation grades of steel and higher strength wires, with ultimate strengths of up to 1860 N/mm^2, have now become standard in the industry.

By the early 1960s the wires were being assembled into strands and anchored by wedges onto an anchor cone cast into the concrete. This led to the development of the standard '7-wire' strand most commonly used today. Also at this time, large capacity jacks were developed that could tension the large multi-strand cables in one operation, and this type of tendon began to dominate, although bars and wires were still used for more specialist applications.

From the early 1950s, and over the next 40 years Jean Muller carried on with Freyssinet's work in developing the art of prestressed concrete bridge design and construction. He introduced the technique of match-cast precast segmental concrete deck construction on the Shelton Road Bridge, USA, in 1952. In 1962 Muller used the technique on a series of motorway overbridges around Paris, the first of which was the Choisy-le-Roi Bridge over the River Seine. He went on to use this approach on numerous viaducts and major bridges worldwide, with one of the most notable bridges being the Linn Cove Viaduct, shown in Fig. 1.13. This was built by a 'top down' approach to minimize the disruption to the environmentally sensitive area beneath. The deck was cantilevered out using precast segments and when the pier

@Seismicisolation

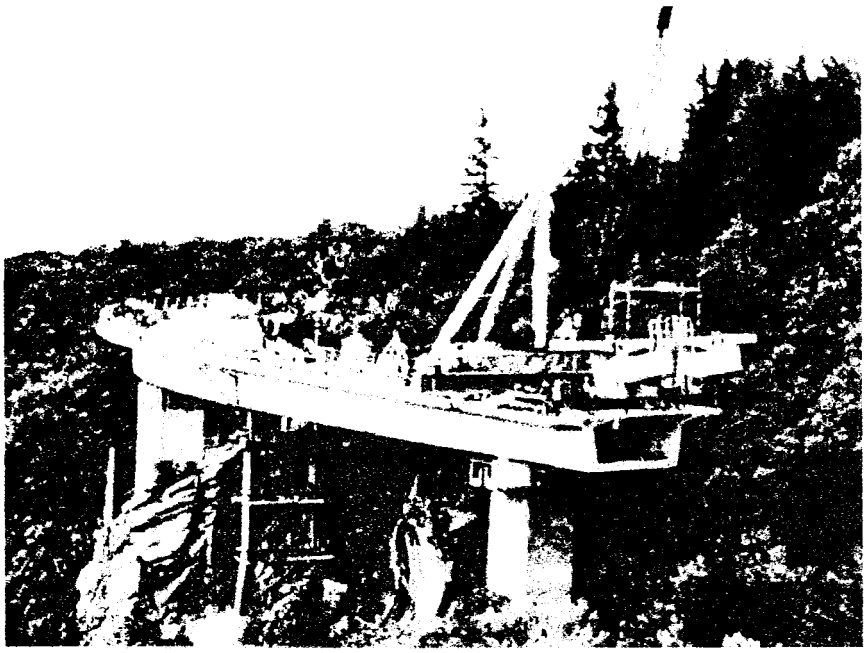


Figure 1.13. Linn Cove Viaduct, USA. Reproduced courtesy of VSL International. Copyright reserved.

position was reached, the precast substructure was placed from the deck and then used to support the deck as the construction continued.

In 1956 the 38 km-long Lake Pontchartrain Bridge, in the USA, was constructed with precast prestressed girders, each with a span length of 17 m, and is still one of the longest bridges in the world today.

The UK's first use of prestressed concrete for bridges involved precast prestressed beams manufactured and stockpiled at the beginning of the Second World War and used for the reconstruction of two bridges carrying roads over railways. Figure 1.14 shows the footbridge in St James's Park,

Figure 1.14. St James's Park Footbridge, England.

@Seismicisolation

London, that was built in 1947 as a slender beam with a span of 21 m and mid-span depth of 0·36 m and remains as a fine example of an early prestressed concrete structure. In 1948, Nunns Bridge in Lincolnshire became the first *in situ* post-tensioned concrete road bridge in the UK with a span of 22·5 m. This was followed by several other short-span bridges formed using precast beams and in 1954 the Northam Bridge in Southampton became the first major prestressed concrete bridge in the UK, with a total length of 148 m and spans up to 32 m. A detailed history of prestressed concrete road bridges in the UK is described in the book by Sutherland *et al.* (2001) [1] and paper by Sriskandan (1989) [2].

In 1960, the Mangfall Bridge in Germany was the first prestressed concrete truss bridge to be built and had a span of 108 m. In more recent years several truss decks using prestressed concrete have been built for multi-span viaducts, although this has not become a common form of structure for concrete bridges.

The first balanced cantilever bridge was built in reinforced concrete across the Rio de Peixe in Brazil in 1930, and it was not until 1950 that this form of construction was first used with prestressed concrete on the Lahn Bridge at Balduinstein, Germany. When the Medway Bridge in the UK, shown in Fig. 1.15, was opened in 1963 it was at the time the longest span in the world constructed as an *in situ* segmental concrete box girder using the balanced cantilever technique. The central span was 150 m long and included a 30 m-long suspended precast beam section in the centre. The approach spans of this 1000 m-long river crossing were constructed using precast prestressed concrete beams.

The Hammersmith Flyover, as described by Rawlinson and Stott (1962) [3], was a landmark structure in the UK. Completed in 1961, it was the first

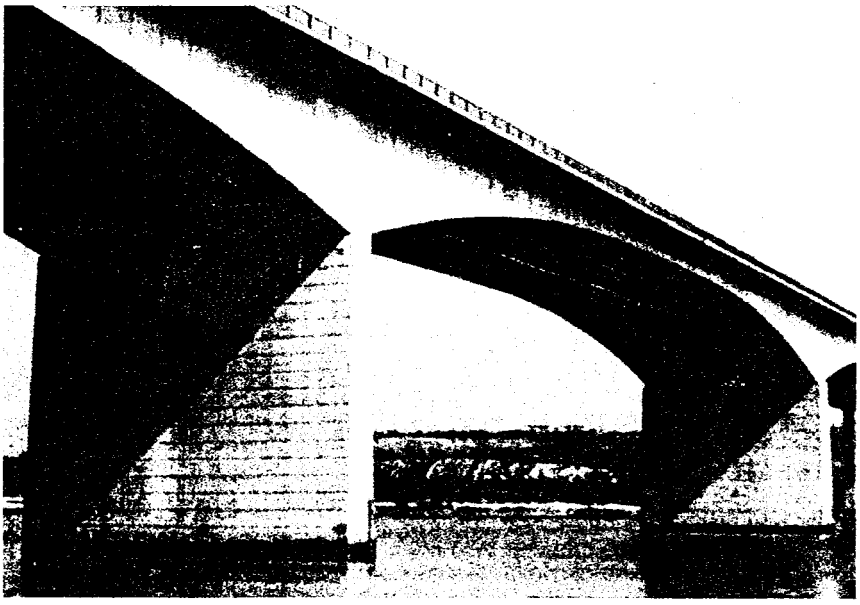


Figure 1.15. M2 Medway Bridge, England. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

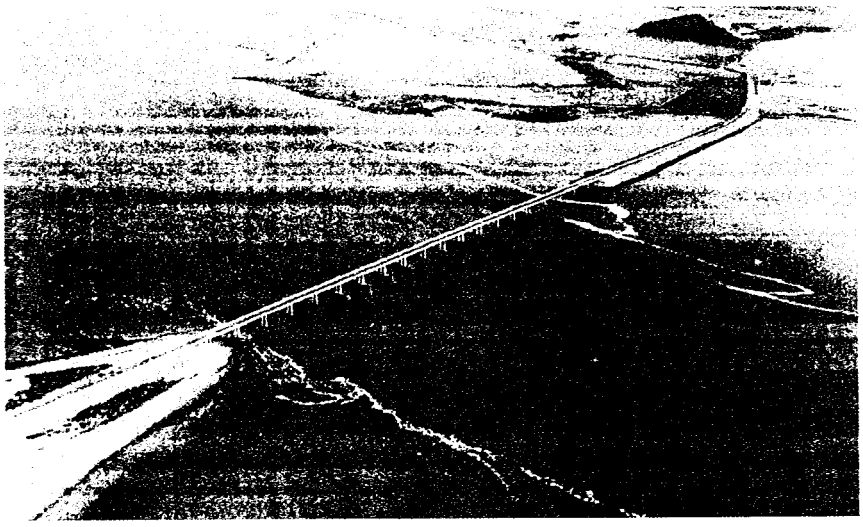


Figure 1.16. Dornoch Firth crossing, Scotland. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

modern elevated urban motorway. The continuous concrete box girder was built using precast segments erected on falsework and joined with a 75 mm *in situ* stitch before being prestressed. The centre spine for the deck was erected first and precast cantilevers added to give a total width of 18·6 m with a maximum span of 42·7 m and an overall length of 623 m.

Throughout the UK in the 1960s, the development of the motorway and bypass network led to the widespread use of precast beams for both over-bridge and underpass construction. The production of families of precast beams in factory conditions led to economic structures with the minimum of formwork required and rapid construction. The inverted 'T' beams, 'I' beams, box-beams, 'M' beams and pseudo-box beams were all popular and competed against each other.

The technique of incrementally launching a post-tensioned concrete box girder was first used on the bridge over the Rio Caroni in Venezuela in 1962, with the system commonly seen today first used on the Inn Bridge, Kufstein, Austria, in 1965. Both these bridges had the basic design carried out by Fritz Leonhardt and his partner Willi Baur. This technique has since been used on many bridges including several notable viaducts on the motorway systems in Germany, France and Spain, and is especially well suited for crossing over deep valleys or over difficult ground. Dornoch Firth crossing in Scotland, shown in Fig. 1.16, was opened in 1991 and was incrementally launched over the estuary during a ten-month period. With 20 spans it has a total length of 890 m.

Prestressed concrete was used in major cable-stayed bridges from the 1960s, with several notable structures built including the Rafael Urdaneta Bridge in Venezuela, with a 235 m main span. Refinements in the design and construction of concrete cable-stayed bridges have led to their common use for spans of up to 500 m. Another early example is the Brotonne Bridge in France, shown in Fig. 1.17. Built in 1976, it has a 320 m-long main span formed using a prestressed concrete box girder deck and a single plane of stays, which produced an elegant and striking appearance.

@Seismicisolation

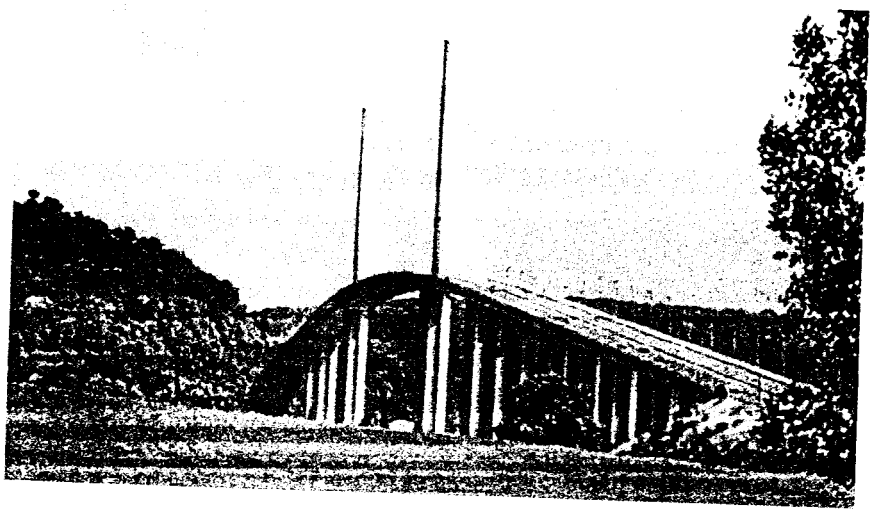


Figure 1.17. Brotonne Bridge, France. Reproduced courtesy of Freyssinet International. Copyright reserved.

From the mid-1970s the rapid growth of many major cities around the world, and the increase in the volume of highway traffic, led towards the development of long urban viaducts, with a need to minimize disruption during their construction, and to build them quickly and economically. Prestressed concrete, in the form of precast beams or precast segmental box girders, dominated this type of construction. In 1992 the 60 km of viaducts for the Bangkok Second Expressway System, Sectors A & C, were completed utilizing match-cast precast segmental box girders over poor ground and through congested streets. With 12 erection gantries, and a casting yard producing thirty-six segments a day, the viaducts were completed within a two-year construction period. More recently, the Hung Hom Bypass in Hong Kong, shown in Fig. 1.18, utilized precast match-cast segments to

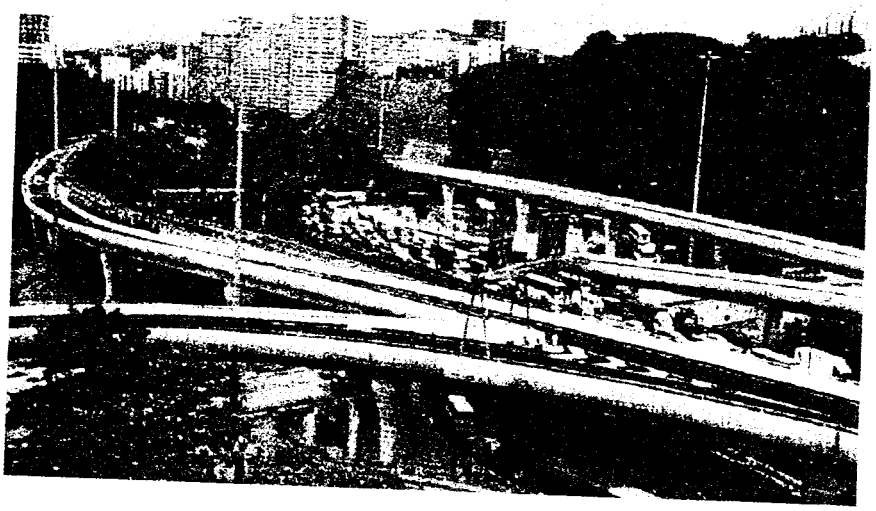


Figure 1.18. Hung Hom Bypass, Hong Kong. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

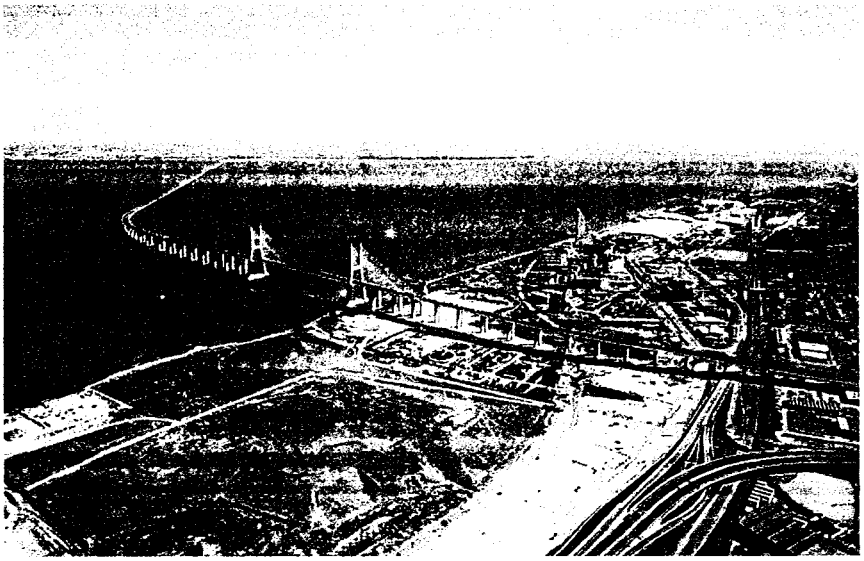


Figure 1.19. Vasco da Gama Bridge, Portugal. Reproduced courtesy of Yee Associates. Copyright reserved.

construct a viaduct crossing over several major highways, a mainline railway and part of the harbour.

Crossing the Tagus River, the Vasco da Gama Bridge in Portugal opened in 1998 and is shown in Fig. 1.19. It is an excellent example of modern prestressed concrete bridge design, with a wide range of construction techniques used on the 18 km-long crossing. Spanning the main navigational channel, the Main Bridge has an 830 m-long cable-stayed deck with a main span of 420 m utilizing *in situ* prestressed concrete edge beams and deck slab. The approach structures are divided into a number of different construction types. The Expo Viaduct and the 130 m navigation spans on the Central Viaduct were constructed using match-cast precast segmental box girder decks erected as balanced cantilevers, while the remainder of the Central Viaduct used precast full-length span elements weighing 2200 tonnes, lifted into place by a floating crane. The other major elements of the crossing are the Northern Viaduct which is an *in situ* ribbed slab cast on full height falsework and the Southern Viaduct which is a twin beam and deck slab arrangement cast *in situ* using formwork supported on a travelling gantry.

A variation to the cable-stayed bridge, the 'extra-dosed' arrangement, has short pylons and can be economic in the 200 m to 300 m span ranges. The Sunniberg Bridge in Switzerland, shown in Fig. 1.20, opened in 1998 and is an outstanding example of this form of structure. Extending the 'prestressing tendons above the deck gives an efficient structural arrangement, which can be used with either a box girder or a beam and slab form of deck.

The development of the different forms of prestressed concrete bridges is illustrated in Fig. 1.21, which shows the increase in span length achieved over the last 60 years. As concrete quality and strengths have increased, and prestressing systems improved, construction techniques have evolved to fully utilize the advantages inherent in prestressed concrete, thus allowing longer and more economic spans to be built.

@Seismicisolation

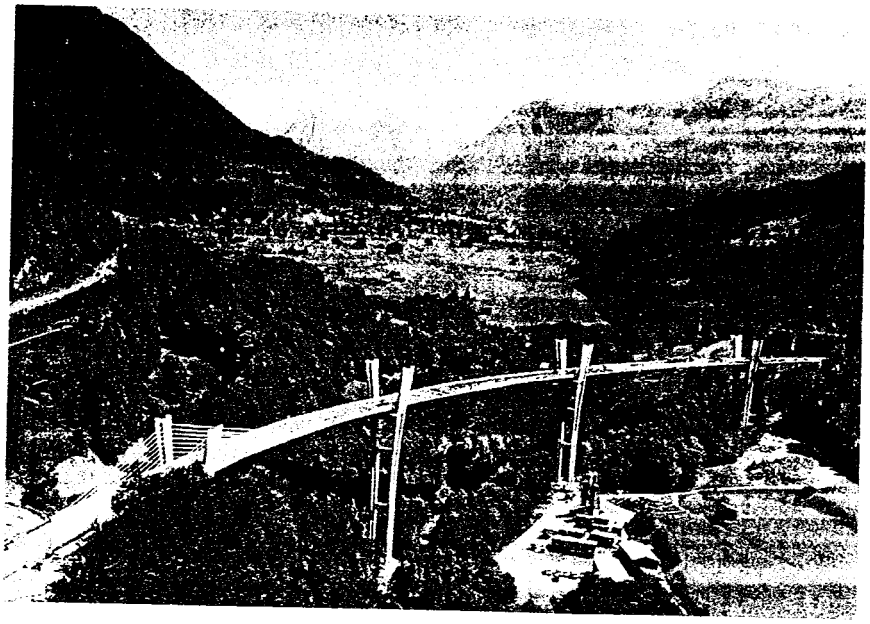
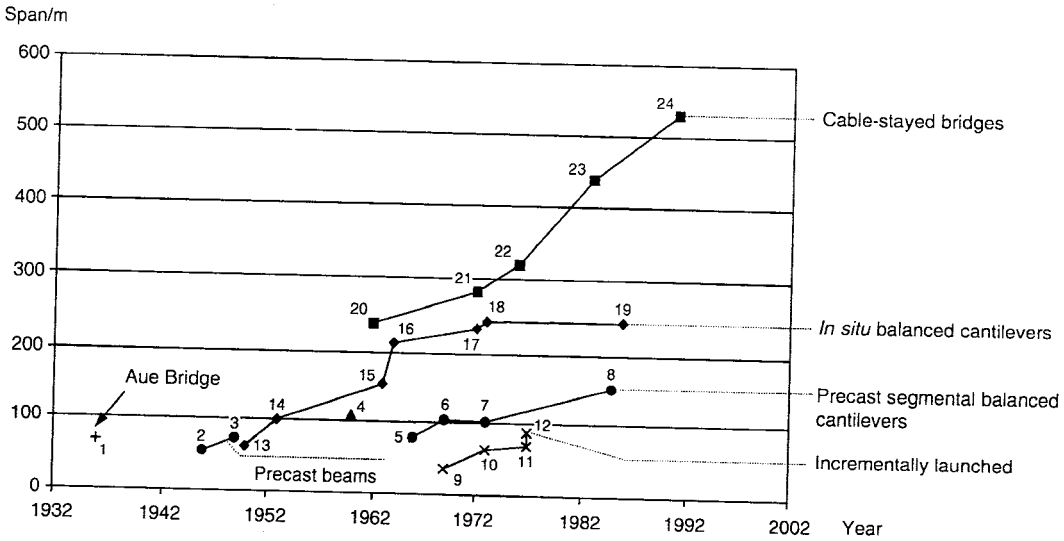


Figure 1.20. Stuoiberg Bridge, Switzerland. Reproduced courtesy of Tiefbauamt Graubünden. Copyright reserved.



1 Aue Bridge, Germany
2 Luzancy Bridge, France
3 Esbly Bridge, France
4 Mangfall Truss Bridge, Germany
5 Oleron Viaduct, France
6 Chilton Viaducts, Switzerland
7 St Cloud Bridge, France
8 Bahrain Causeway, Saudi Arabia
9 Pipeline Bridge, Italy
10 Horomoi Bridge, Japan
11 Necker Bridge, Germany
12 Leck Bridge, Germany
13 Lahn Bridge, Germany
14 Nibelungen Bridge, Germany
15 Medway Bridge, England
16 Bendorf Bridge, Germany
17 Urato Bridge, Japan
18 Hamana Bridge, Japan
19 Koror-Babelthaup Bridge, Pacific Islands
20 Rafael Urdaneta Bridge, Venezuela
21 Wadi Kuf, Libya
22 Brotonne Bridge, France
23 Barrios de Luna, Spain
24 Skarnsundet Bridge, Norway

Figure 1.21. Longest prestressed concrete bridge span length v year.

@Seismicisolation

The first designs for prestressed concrete bridges were based on experience gained from experiments and application of the technique in buildings and other structures as no codified rules existed. In the early days, Freyssinet and other prominent engineers published articles and gave lectures, forming the main source of exchanging information. In 1949, Abeles published his book [4], one of the first on the principles of prestressed concrete, followed by Magnel (1950) [5] and Guyun (1951) [6] a few years later. Design rules were published in France in 1953, followed in 1959 by the UK standard, CP115 published by the British Standards Institution [7]. Design codes evolved significantly as more knowledge was gained, especially in relation to the creep, shrinkage and temperature effects until the early 1980s when the codes we use today were developed.

The first prestressed concrete bridges tended to use post-tensioned tendons placed externally to the concrete. Prestressed wires were either left bare and open to the elements, or wrapped in some form of protection, or encased in concrete after stressing. Most of these forms of protection resulted in the external tendons having corrosion problems, which for many years led to post-tensioned designs usually adopting tendons placed internally.

There was some experimenting with external tendons in the 1960s and 1970s, but in the UK problems with durability occurred which effectively resulted in a UK-wide ban on the use of external tendons for several years. In France during the early 1970s external tendons were successfully used to strengthen several bridges, and prestressing strand was being used as stays on cable-stayed bridges. These adopted a robust protection system developed using high-density polyethylene ducts filled with cement grout or grease, which led to increased confidence in external tendons.

In the UK in 1985, a collapse of the Ynys-y-Gwas Bridge in Wales, due to corroded tendons, initiated a series of inspections and investigations. In 1992 this led to a ban on internal tendons, and all UK designs, at that time, had to use external tendons. This ban was partially lifted in 1996 when, with good detailing and construction practice, internal tendons were allowed, except for precast segmental construction where concerns over the water-tightness of the joints remained.

For the future, prestressed concrete is likely to continue to dominate as the material of choice for bridge spans in the range of 30 m to 200 m, while its use on longer span cable-stayed bridge decks should continue to grow. High-strength concretes are being developed and used in building works and are likely to become more common for bridgeworks which, when combined with prestressing, will enable longer and lighter structures to be adopted. Two recent bridges using high-strength concrete with prestressing are the Sherbrooke Footbridge in Canada and the Peace Footbridge in Korea.

The Sherbrooke Footbridge, completed in 1997, uses a special high-strength concrete product in its composition developed by the Bouygues' Research Department. The Reactive Powder Concrete (RPC) is made using small particles to give a dense mixture. Concrete used on the Sherbrooke Footbridge achieved a compressive strength of 200 N/mm^2 and tensile strengths of 7 N/mm^2 and 40 N/mm^2 under direct load and bending respectively. Shown in Fig. 1.22, the 3·5 m deep truss deck spans 60 m with

@Seismicisolation

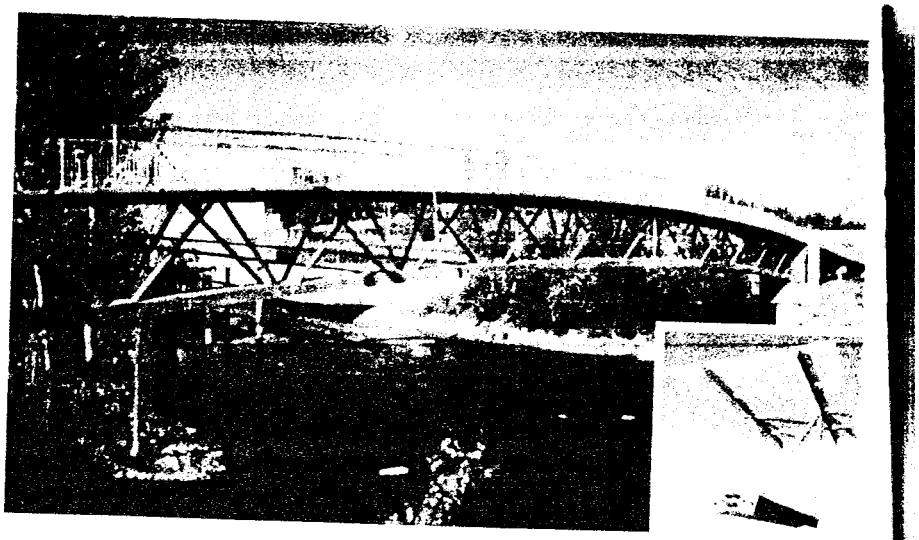


Figure 1.22. Sherbrooke Footbridge, Canada. Reproduced courtesy of VSL International. Copyright reserved.

a RPC top slab and bottom beam. The steel tube diagonal members are also filled with RPC concrete. The top slab is transversely prestressed, with external prestressing tendons used longitudinally.

Similar high-strength concrete has also been used on the Peace Footbridge completed in 2001. The fibre-reinforced high-strength concrete is prestressed both longitudinally and transversely for the beam and slab section that makes up the 120 m span arch seen in Fig. 1.23. The specially developed concrete enabled thin sections to be used, reducing the total weight of the deck while the dense concrete is expected to be highly durable.

Non-metallic prestressing tendons using carbon fibre reinforced polymers have been developed and tested in recent years. These have the advantage of having very high strength, and are not susceptible to corrosion; however,

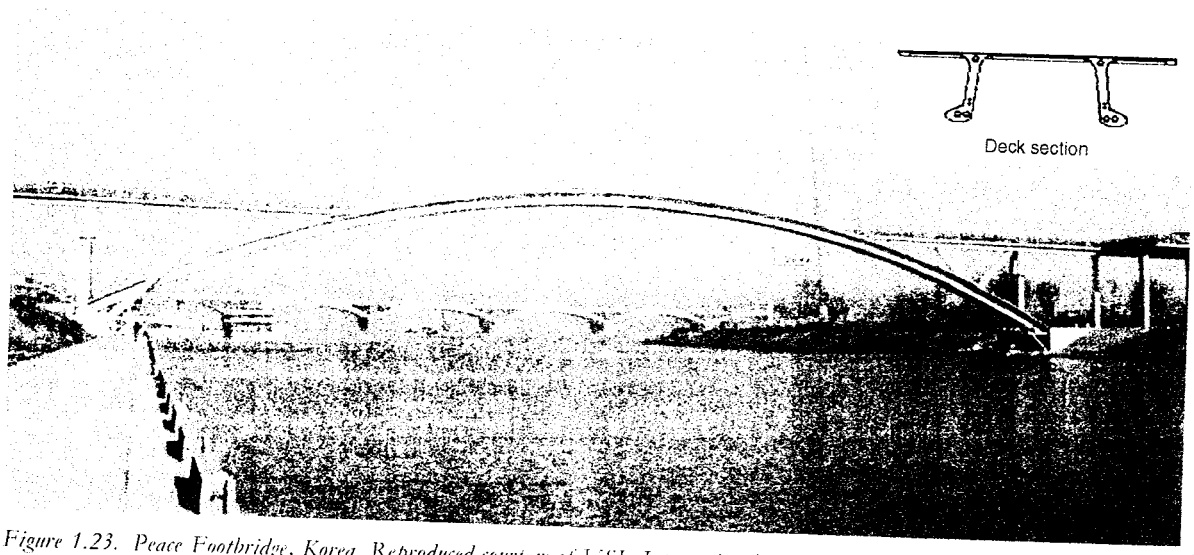


Figure 1.23. Peace Footbridge, Korea. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

they are currently much more expensive than steel tendons and have proved difficult to anchor effectively. As this technology develops their use is likely to become more common.

References

1. Sutherland, J., Humm, D. and Chrimes, M. (eds) (2001) *Historic Concrete – The Background to Appraisal*. London: Thomas Telford Publishing.
2. Sriskandan, K. (1989) 'Prestressed Concrete Road Bridges in Great Britain: A Historical Survey', *The Proceedings of the Institution of Civil Engineers*, **86**, April, 269–303.
3. Rawlinson, J. and Stott, P.F. (1962) 'The Hammersmith Flyover', *The Proceedings of the Institution of Civil Engineers*, **23**, December, 565–624.
4. Abeles, P.W. (1949) *The Principles and Practice of Prestressed Concrete*. London: Crosby Lockwood.
5. Magnel, G. (1950) *Prestressed Concrete*. London: Concrete Publications.
6. Guyun, Y. (1951) *Prestress Concrete*. Paris: Edition Eyrolles.
7. CP115, (1959) *The Structural Use of Prestressed Concrete in Building*. London: British Standards Institution.

@Seismicisolation

2

Prestressing components and equipment

Introduction

Prestressing systems are made up of a number of standard components. The basic prestressing element is the wire, strand or bar, while with post-tensioning the anchors, couplers and ducts form the embedded items. Specialist equipment for placing, stressing and grouting the tendon are required and proprietary systems have developed and become established worldwide. This chapter looks at the different components and equipment that make up the prestressing system. More details on the proprietary systems from individual suppliers are given in appendix D, while grouting equipment is included in Chapter 4.

Proprietary systems

The specialist nature of prestressing concrete has led to a number of proprietary prestressing systems being widely used throughout the industry, with several of the principal prestressing companies mentioned below. Most multi-strand prestressing systems utilize similar strand, either 13 or 15 mm diameter, while BBR also provide a parallel wire system and there are several bar systems available. When designing prestressed concrete members, the choice of which type of prestressing to be used is usually made by the designer, while the contractor chooses which particular system to adopt. However, the designer must ensure that the design and detailing adequately cater for the actual equipment and components to be used.

The VSL post-tensioning systems have been used throughout the world since 1956 and are protected by patents. They are used in every branch of prestressed concrete construction and although used primarily for bridges and buildings, they are used for numerous other applications as well, including soil and rock anchors and for lifting and sliding heavy loads. VSL originated in Switzerland and rapidly expanded, with their system being adopted internationally. In 1988, the group restructured under a new name, VSL International Ltd, to reflect their worldwide business, and today have subsidiaries or licensees in most parts of the world. For bridgeworks, VSL provide both multi-strand and stressbar systems.

Eugene Freyssinet was one of the pioneers of prestressed concrete: developing the materials and equipment, as well as the design philosophy. He took out several patents and worked closely with contractors in France

@Seismicisolation

to establish the techniques. In 1943, Edme Campenon established a company, Société Technique pour l'Utilisation de la Préconstrainte (STUP), to develop Freyssinet's ideas. STUP were involved with many early prestressed concrete structures and became established worldwide in both the design and construction of prestressed concrete bridges, as well as for other uses of the technique. In 1976, STUP changed its name to Freyssinet International (STUP), in order to honour Eugène Freyssinet and to develop the company on an international scale. Today, Freyssinet International provides multi-strand stressing systems.

Founded in 1865, DYWIDAG is the abbreviation of its mother company, Dyckerhoff and Widmann AG, which is one of the oldest construction companies in Germany. The first ever structure built with a prototype DYWIDAG Post-Tensioning System using bars was the arch-bridge at Alsleben (Germany) in 1927, followed by the three-span Aue Bridge (Germany) in 1936, where unbonded post-tensioned tendons were used.

DYWIDAG began licensing their system in 1950 on a worldwide basis and in 1979 formed the DSI group to co-ordinate its international business. DYWIDAG provide both multi-strand and prestressing bar systems for bridgeworks.

Bureau BBR Ltd, established in 1944, was originally a partnership formed by three Swiss civil engineers; M. Birkenmaier, A. Brandestini and M.R. Roš. Today it is a limited company registered in Zurich, Switzerland, and specializes in prestressing technology and related construction work. BBR operates on an international scale with numerous licensees, consultants, contractors and suppliers. BBR provides multi-strand and multi-wire prestressing systems for use in bridgeworks.

Macalloy is a registered trademark of McCalls Special Products Ltd, based in the UK, who produce high-tensile alloy-steel bars for prestressing concrete as well as a range of other applications.

Details of some of the above proprietary systems are given in appendix D; however, each proprietary system may have different details when used in different countries and the systems develop over time, so it is important to check the specifications with local prestressing suppliers for any particular project.

Wires

Individual wires are sometimes used in pre-tensioned beams but have become less common in favour of strand, which has better bond characteristics. The wire is cold-drawn from hot-rolled rods of high-carbon steel and stress-relieved to give the required properties. Wire diameters are typically between 5 mm and 7 mm, with a minimum tensile strength of between 1570 and 1860 N/mm^2 carrying forces up to 45 kN. Material properties for prestressing wire are specified in the British Standard 5896 (1980) [1]. Typical details for wires are given in Table 2.1.

Galvanized wire is available, although ungalvanized wire is used for most standard prestressing applications. Stainless steel wire can also be obtained, but this is not usually used for the prestressing of concrete.

@Seismicisolation

Table 2.1 Wire, strand and bar properties

		Nominal diameter (mm)	Nominal area (mm^2)	Nominal mass (kg/m)	Yield strength (N/mm^2)	Tensile strength (N/mm^2)	Minimum breaking load (kN)	Modulus of elasticity (kN/mm^2)	Relaxation[1] (class 2 or low relaxation)
7-wire strand low-relaxation									
13 mm (0·5″)	Euronorm 138–79, or BS 5896: 1980, Super	12·9	100	0·785	1580	1860	186	195	2·5%
	Euronorm 138–79, or BS 5896: 1980, Standard	12·5	93	0·73	1500	1770	164	195	2·5%
	Euronorm 138–79, or BS 5896: 1980, Drawn	12·7	112	0·89	1580	1860	209	195	2·5%
	ASTM A416-85, Grade 270	12·7	98·7	0·775	1670	1860	183·7	195	2·5%
15 mm (0·6″)	Euronorm 138–79, or BS 5896: 1980, Super	15·7	150	1·18	1500	1770	265	195	2·5%
	Euronorm 138–79, or BS 5896: 1980, Standard	15·2	139	1·09	1420	1670	232	195	2·5%
	Euronorm 138–79, or BS 5896: 1980, Drawn	15·2	165	1·295	1550	1820	300	195	2·5%
	ASTM A416-85, Grade 270	15·2	140	1·10	1670	1860	260·7	195	2·5%
Stress bars									
20 mm	BS 4486: 1980	20	314	2·39	835	1030	323	170/205	3·5%
25 mm	BS 4486: 1980	25	491	3·9	835	1030	505	170/205	3·5%
32 mm	BS 4486: 1980	32	804	6·66	835	1030	828	170/205	3·5%
40 mm	BS 4486: 1980	40	1257	10	835	1030	1300	170/205	3·5%
50 mm	BS 4486: 1980	50	1963	16·02	835	1030	2022	170/205	3·5%
Cold-drawn wire									
7 mm	BS 5896: 1980	7	38·5	302	1300	1570	60·4	205	2·5%
	BS 5896: 1980				1390	1670	64·3	205	2·5%
5 mm	BS 5896: 1980	5	19·6	154	1390	1670	32·7	205	2·5%
	BS 5896: 1980				1470	1770	34·7	205	2·5%

Notes

Reference should be made to the manufacturers' literature for the properties of individual wires, strands or bars being used.

[1] Relaxation after 1000 hrs, at 20 °C and 70% UTS.

@Seismicisolation

Strands and tendons

The most common form of prestressing is 7-wire strand, which is made up of individual cold-drawn wires with six outer wires twisted around an inner core wire. An example of a multi-strand tendon and its anchorage partly cut away is shown in Fig. 2.1. The strand is stress-relieved and is usually of a low-relaxation grade. Strands can be galvanized should greater protection be required, although this is unusual for normal prestressing applications. Epoxy-coated strands are also available and have been used for prestressing bridge decks, but this is not a common practice.

For post-tensioning, 13 mm or 15 mm diameter, 7-wire strand is used, either singly for pre-tensioning or in bundles to form multi-strand tendons. The most common post-tensioned tendon sizes utilize 7, 12, 19 or 27 strands to suit the

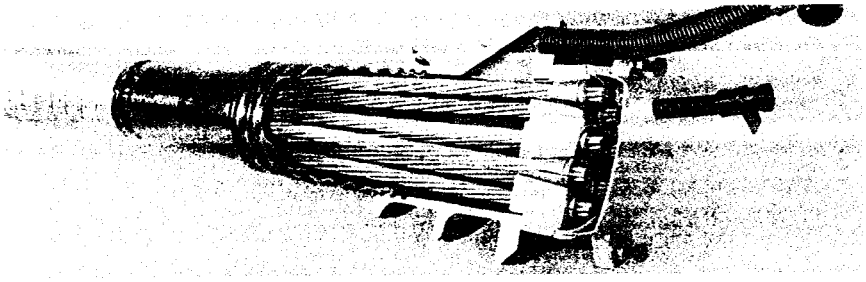


Figure 2.1. Multi-strand tendon. Reproduced courtesy of DYWIDAG-Systems International GmbH). Copyright reserved.

Table 2.2 Multi-strand tendon sizes

	13 mm (0·5″) strand				15 mm (0·6″) strand			
	Minimum breaking load (kN)		Typical duct sizes internal/external diameter		Minimum breaking load (kN)		Typical duct sizes internal/external diameter	
No. of strands	Euronorm 138–79 or BS 5896: 1980, Super	ASTM A416–85, Grade 270	Steel ducts (mm)	Plastic ducts (mm)	Euronorm 138–79 or BS 5896: 1980, Super	ASTM A416–85, Grade 270	Steel ducts (mm)	Plastic ducts (mm)
1	186	184	25/30		265	261	25/30	
2	372	367	40/45		530	521	40/45	
3	558	551	40/45		795	782	40/45	
4	744	735	45/50		1060	1043	50/55	
6	1116	1102	50/55		1590	1564	60/67	
7	1302	1286	55/60		1855	1825	60/67	59/73
12	2232	2204	65/72	59/73	3180	3128	80/87	76/91
18	3348	3307	80/87		4770	4693	95/102	
19	3534	3490	80/87	76/91	5035	4953	95/102	100/1
22	4092	4041	85/92		5830	5735	110/117	100/11
31	5766	5695	100/107	100/116	8215	8082	130/137	130/14
37	6882	6797	120/127	130/146	9805	9646	140/150	130/14
43	7998	7899	130/137		11 395	11 210	150/160	
55	10 230	10 104	140/150	130/146	14 575	14 339	170/180	

@Seismicisolation

standard anchor blocks available, although systems are available for tendons incorporating up to 55 strands when necessary. Stressing to 75 per cent UTS gives typical jacking forces of 140 kN or 199 kN for the 13 mm or 15 mm diameter strands respectively. The larger multi-strand tendons can carry forces up to 10 000 kN. Material properties for prestressing strand are specified in the British Standard 5896 (1980) [1] and by ASTM A416M-99 (1999) [4]. Table 2.1 gives typical details of the strand used, while Table 2.2 summarizes the most common tendon sizes and the associated minimum breaking loads for 'super strand' and ASTM grade 270 steel.

At the ends of tendons the strands are anchored either by splaying out the wires and encasing them in the concrete as a dead-end anchorage, or by passing them through an anchor arrangement and fixing them in to an anchor block for a live-end anchorage.

Bars

Prestressing bars, as shown in Fig. 2.2, are available in different diameters from 15 mm up to 75 mm and are used in post-tensioned construction. They typically have a minimum ultimate characteristic tensile strength of between 1000 N/mm^2 and 1080 N/mm^2, although a higher strength steel grade is available from some manufacturers. Jacking forces range from 135 kN to over 3000 kN. Bars are formed in straight lengths by the hot rolling of steel rods and are either smooth or deformed on the surface. Galvanized bars are available, but are not commonly used in standard post-tensioned concrete applications. Material properties for prestressing bars are specified in the British Standard 4486 (1980) [2] and in ASTM A722M-98 (1998) [5]. Typical details for bar post-tensioning are given in Table 2.1.

The bars are generally placed into ducts cast into the concrete between two anchor blocks located on the concrete surface. Bars are stressed by pulling from one end, using a stressing jack placed against the anchorage arrangement, and then held in place by a nut assembly.

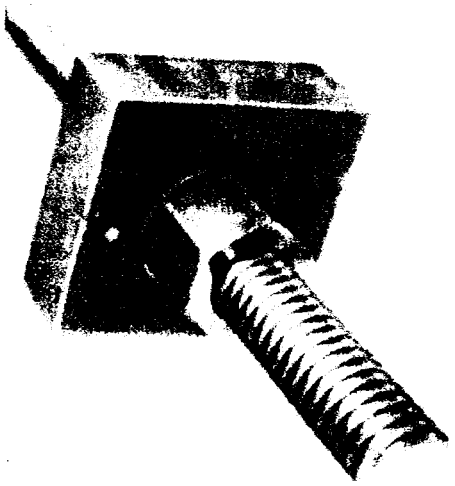


Figure 2.2. Prestressing bar and anchor. Reproduced courtesy of McCalls Special Products Ltd. Copyright reserved.

@Seismicisolation

Anchorages

At each end of a tendon the force is transferred into the concrete by an anchorage system. For pre-tensioned strands the anchorage is by bond and friction of the bare strand cast into the concrete, while for post-tensioned tendons anchorage is achieved by using anchor blocks or an encased dead-end anchor.

In Fig. 2.2 a typical anchor arrangement is shown for a post-tensioned bar. The bar is held in place by the threaded nut and the force from the bar is transferred through the threads and the nut and on to the anchor block that is cast into the concrete. The hole on the near side of the anchor block assists with the grouting of the duct and the anchorage arrangement after stressing has been completed.

Stressing of multi-strand tendons is undertaken using jacks placed over the anchorage and tendon. The jack grips each strand and pulls the tendon until the required force is generated. Wedges are then pushed into place around the strand and seated into the anchor block, so that on the release of the force by the jack the wedges grip the strand and transfer the force on to the anchorage and into the concrete. Figure 2.3 shows a typical live-end anchorage for a multi-strand tendon after the strands have been stressed and trimmed back to the wedges. The strands and wedges are seated into holes formed in the anchor block, which rests against the bearing plate and trumpet, cast into the concrete. The hole at the top of the anchor is for the grout to flow through, to ensure that the duct and anchor are fully filled during the grouting operation.

Strand tendons may also be anchored at the non-stressing end with a cast-in dead-end anchorage arrangement, as shown in Fig. 2.4, where the strands are spread out after emerging from the duct. In this arrangement, the tendon is installed before concreting, but is not stressed until after the concrete has attained the required transfer strength. As the strands emerge from the end of the ducting, a seal prevents the ingress of concrete into the duct and, where the strands spread out, a tension ring resists the splitting forces caused by the change in angle. At the end of the strands the wires are opened out to create a 'bulb' that, when encased within the concrete, ensures a good anchorage.

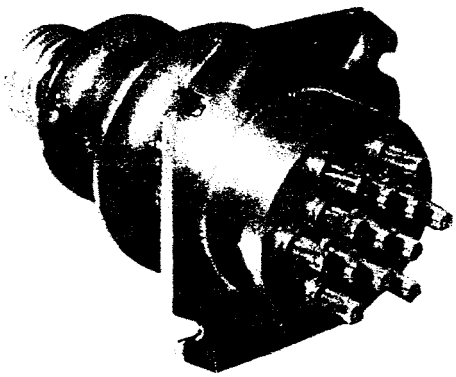


Figure 2.3. Multi-strand tendon live-end anchor. Reproduced courtesy of Freyssinet International. Copyright reserved.

@Seismicisolation

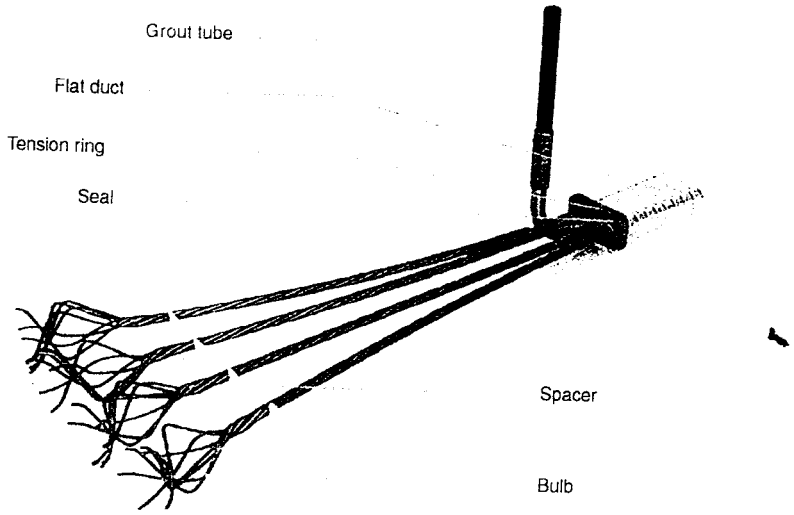


Figure 2.4. Strand dead-end anchorage. Reproduced courtesy of VSL International. Copyright reserved.

External, post-tensioned tendons should be removable and replaceable, and the detail of the anchorage is arranged to allow for this. Where cement grout is used, a lining is provided on the central hole of the anchor arrangement to ensure that the grout around the tendon does not bond to the anchor or adjacent concrete. This facilitates the cutting and pulling out of the tendon, if necessary. Alternatively, where grease or wax filler is used, the tendon can be de-stressed or re-stressed using a jack. To provide for this the strands must protrude far enough behind the anchor block to allow the jack to grip them. A special extended capping assembly is used to protect the strand, which is filled with the grease or wax to prevent corrosion.

All post-tensioned tendon anchor blocks should be detailed to allow them to fully fill up with grout and prevent any air being trapped during the grouting operation. This may be achieved by providing vents and holes through the anchor blocks, thus allowing the grout to flow freely.

Tendon couplers

Tendons can be coupled to extend their length during stage-by-stage construction. Figure 2.5 shows a coupler system for multi-strand tendons. A special anchor block and coupler arrangement is used to enable the tendon to be extended after it has been stressed. The standard bearing plate, shown on the right side of the figure, is cast into the first stage of concreting, with the tendon being placed and stressed as the construction progresses. During construction of the next stage and before concreting, the next length of tendon is positioned with the ends of the strands fixed into the coupling head, as shown in the centre of the figure. After concreting the next stage, the new length of tendon is stressed with the force being transferred through the coupler to the first tendon. This arrangement can simplify the tendon layout and save the cost of a separate anchor.

@Seismicisolation

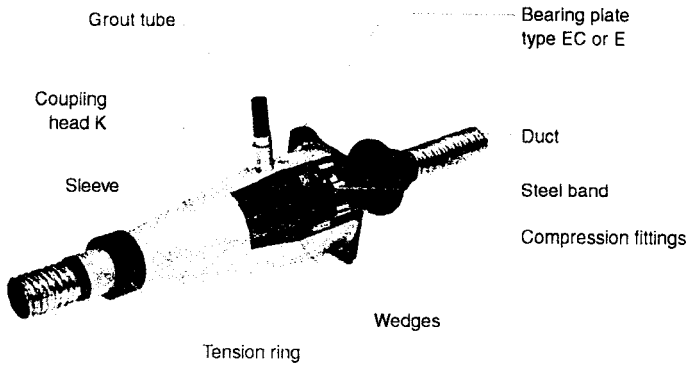


Figure 2.5. Multi-strand tendon coupler. Reproduced courtesy of VSL International. Copyright reserved.

Bars can also be joined together by using a simple threaded coupler, as shown in Fig. 2.6. The coupler is threaded on to the ends of the bars to transfer the load across the connection. Couplers enable long lengths of bars to be assembled and allow previously stressed bars to be extended into subsequent concrete pours.

Where the couplers are located, the concrete section has to be thick enough to adequately surround the arrangement, with sufficient room for the reinforcement and the concrete cover. If the duct runs through a thin web or slab then a thickened section is often needed at the location of the coupler. The thickening required can be significant on large multi-strand tendons, where the size of the coupler is much greater than the tendon duct.

Where couplers are used to extend cables or bars in stage-by-stage construction, the design must consider that although the 'anchor' side at the coupler may not carry much load in the final condition, it carries the full tendon load during the stage stressing. Therefore, the anchor part of the coupler and surrounding concrete must be designed to cater for this load in the temporary condition.

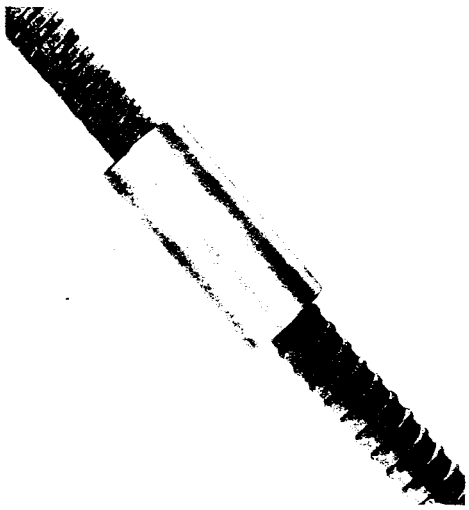


Figure 2.6. Prestress bar coupler. Reproduced courtesy of McCalls Special Products Ltd. Copyright reserved.

@Seismicisolation

Ducting

With internal post-tensioning a duct is used to form a void through the concrete into which the tendon is placed. Traditionally, ducts are made from corrugated steel, often from strips rolled on site into a helix and crimped together to form the duct. The requirements for steel ducts are covered by British Standard EN523 (1997) [6]. Recent concerns over the durability of prestressing tendons has led to the use of corrugated plastic ducts, which are made watertight along their complete length. Figure 2.7 shows typical steel ducts installed within a reinforcement cage, while Fig. 2.8 shows a typical plastic duct. Steel ducts usually have a wall thickness of 0·25 mm, while corrugated plastic ducts are normally 2·5 mm to 3 mm thick. Indicative duct sizes for different tendon systems are given in Table 2.2, with details of some of the proprietary systems given in Appendix D.

External tendons are usually placed inside high-density polyethylene (HDPE) ducts, shown in Fig. 2.9. These ducts have to be strong enough to withstand the abrasion from the strand as it is threaded and stressed, pressure

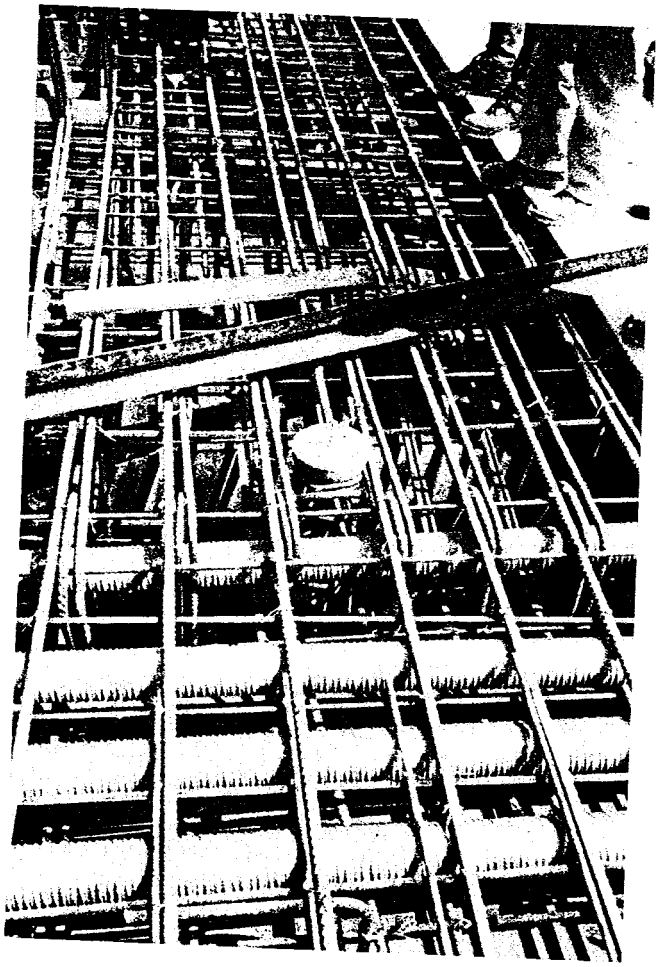
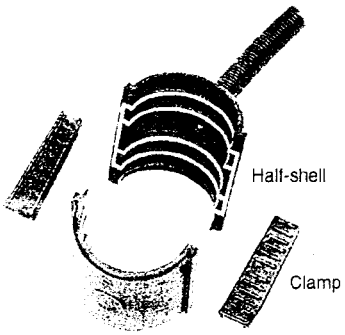


Figure 2.7. Steel ducts inside reinforcement cage.

@Seismicisolation

PT-PLUS plastic duct



PT-PLUS duct coupler

Figure 2.8. Corrugated plastic duct and coupler. Reproduced courtesy of VSL International. Copyright reserved.

from the strand as it goes around any curves in the tendon alignment, and the pressure from the grouting. As a result, the HDPE duct thickness is normally at least 6 mm.

Ducting is manufactured in convenient handling lengths, normally between 4 m and 6 m long and joined *in situ* to make up the full length required. Steel ducts are joined with a short coupler that goes around the

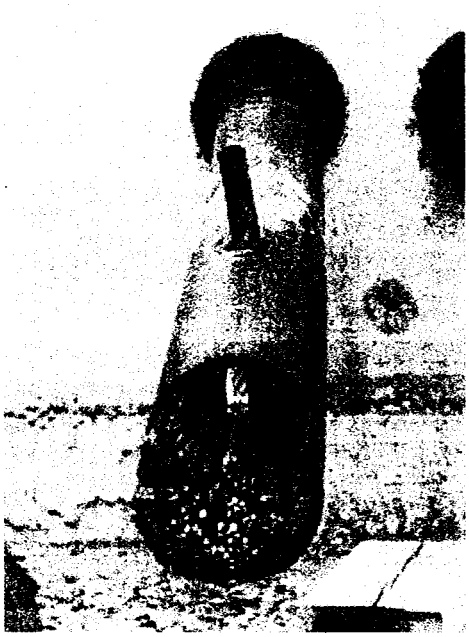


Figure 2.9. HDPE duct.

@Seismicisolation

outside of the ducts on both sides of a joint to provide continuity in the ducting system. The coupler is usually a slightly larger diameter duct, that fits tightly over the corrugations and is 'twisted' into position. Corrugated plastic ducts are made continuous and sealed with a coupler, which provides a watertight joint. A typical plastic coupler fitting is shown in Fig. 2.8.

HDPE ducts are joined together by either hot welding or sleeving. During hot welding, the carefully prepared ends of the ducts are positioned in a frame and a hot knife or plate used to soften the end faces of the HDPE. The duct ends are then brought together and held until the material has hardened again and a seal formed. When using sleeves, a short length of tight-fitting prefabricated HDPE duct is slid over the joint and sealed into position.

Grout inlets, outlets, vents and drainage tubes are needed along the ducts to remove any water that may accumulate inside and to facilitate in the grouting operation. For steel ducting, vents are formed by creating a hole in the duct and fixing a plastic saddle, onto which a plastic tube is connected leading to the concrete surface. Special adapters are used for corrugated plastic ducts, while for HDPE ducts the inlet or vent tube is 'welded' to the duct around a suitably sized hole. Examples of the different vent arrangements are given in Chapter 4.

Equipment for placing tendons

Bar tendons are usually placed by hand, by pushing the bars through the ducts. Hand placing can also be used for the shorter strand tendons and where a simple duct profile exists. For the larger and longer multi-strand tendons, special equipment is needed to position the strand.

Coils of 7-wire strand arrive on site usually weighing up to 3 tonnes. The coils are placed in a frame, as seen in Fig. 2.10, to enable the strand to be pulled out and made into the tendon. In a multi-strand tendon the strands

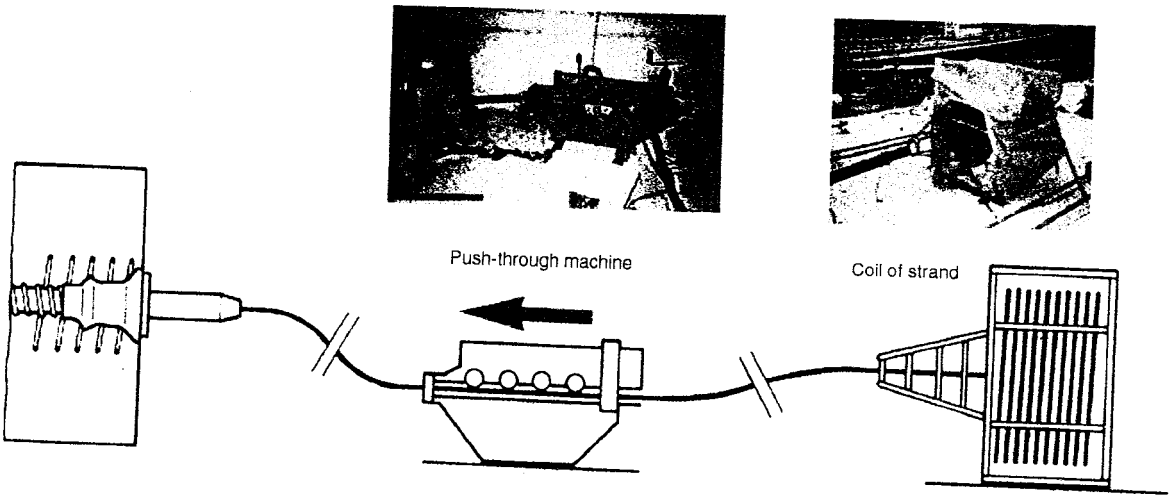


Figure 2.10. Push-through placing of strand. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

are positioned by either pushing or pulling the strand into place. The most common method of placing the strand is by the push-through method, either using a machine or by hand.

The push-through method involves pushing individual strands into the duct one at a time until the full number is in position as illustrated in Fig. 2.10. A push-through machine, also shown in Fig. 2.10, grips the strand as it is pulled out of the coil and feeds it into the duct.

The pull-through method is illustrated in Fig. 2.11. Strands are first cut to length and bundled into a complete tendon at one end of the duct. A steel pulling rope is thread through the duct and connected to a winch at one end. At the other end the rope is fixed to a cable sock, usually made from a wire mesh, which is put over the leading end of the tendon. When pulled, the sock grips the strands and pulls them all through the duct together.

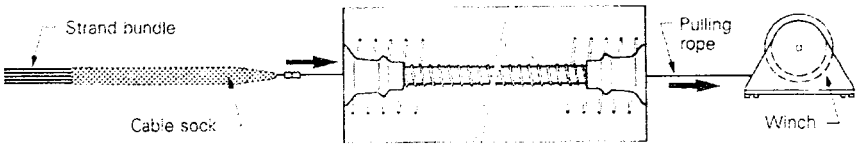


Figure 2.11. Pull-through placing of strand. Reproduced courtesy of VSL International. Copyright reserved.

Stressing jacks

Typical jacks for single-strand and multi-strand tendons are shown in Figs. 2.12 and 2.13 respectively. Figure 2.14 shows a typical jack for bar tendons. These jacks are hydraulically operated, with oil pumped into the piston to apply load to the tendon. The larger jacks can generate a pulling force in excess of 1200 tonnes.

The same jacks are also used to de-stress tendons where this is necessary. For de-stressing, special stools are used to move the jack back and to

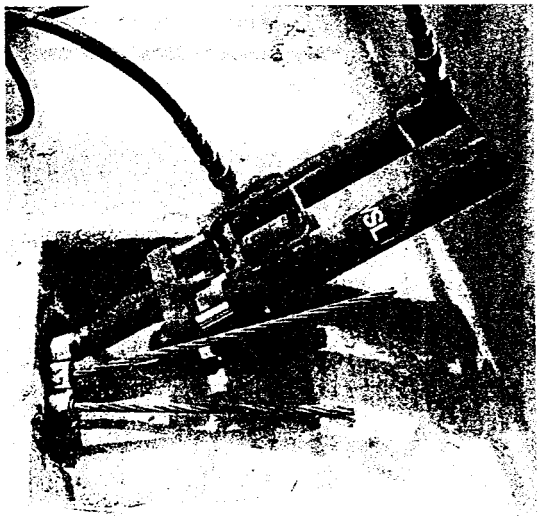


Figure 2.12. Jack for stressing single strand. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

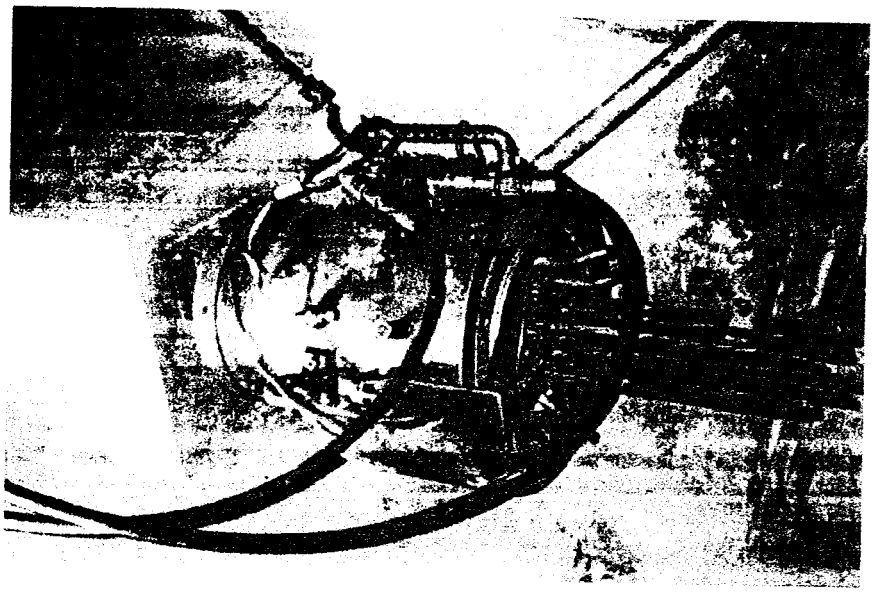


Figure 2.13. Jack for multi-strand tendon.

Figure 2.14. Jack for prestressing bar.

@Seismicisolation

Figure 2.15. Placing of large prestressing jack onto tendon.

give access to the anchor wedge or nut allowing them to be loosened or removed.

Jacks for single strands and small tendons, or smaller bars, weigh up to 250 kg, and can be easily handled and manoeuvred into position. For the larger tendons special lifting equipment, lifting frames or cranes, are required to move the jacks, which can weigh up to 2000 kg. Figure 2.15 shows typical jack-lifting equipment being used inside a box girder.

The design and detailing of the bridge deck should always take into account the access needed to set up and operate the jacks and associated equipment.

References

1. BS 5896 (1980) *High Tensile Steel Wire and Strand for the Prestressing of Concrete*. London: British Standards Institution.
2. BS 4486 (1980) *Hot Rolled and Hot Rolled and Processed High Tensile Alloy Steel Bars for the Prestressing of Concrete*. London: British Standards Institution.
3. BS 4447 (1973) *Specification for the Performance of Prestressing Anchorages for Post-tensioned Construction*. London: British Standards Institution.
4. ASTM A416M-99 (1999) *Standard Specification for Uncoated 7-wire Steel Strand for Prestressing Concrete*. Pennsylvania: ASTM International.
5. ASTM A722M-98 (1998) *Standard Specification for Uncoated Steel Bar for Prestressing Concrete*. Pennsylvania: ASTM International.
6. BS EN523 (1997) *Steel Sheaths for Tendons Terminology, Requirements, Quality Control*. London: British Standards Institution.

@Seismicisolation

3

Durabilty and detailing

Introduction

Discovery of corroded tendons in several bridges in the UK and internationally during the 1980s and early 1990s resulted in concern over the long-term durability of prestressed concrete bridges. Subsequent inspections and testing programmes have led to a better understanding of the factors that influence the good design and construction of these bridge decks. In the UK, a review of current practice and 'state of the art' research led to a report issued by the Concrete Society (1996) [1] and (2002) [2] giving guidance on some of the durability requirements for post-tensioned concrete bridges.

The prestressing elements in a bridge deck are a primary structural component and they need to be protected from risk of damage or deterioration. Good access to inspect as much of the prestressing tendon and other elements of the deck as possible is required for long-term maintenance, while for external tendons, provisions for their re-stressing or replacement need to be incorporated.

With well-designed details and good construction practices, prestressed concrete bridge decks provide durable and low-maintenance structures. This is clearly demonstrated by the large number of existing bridges that have performed well and are still in a good condition after many years in service.

This chapter discusses some of the aspects concerning durability and associated topics in more detail. The grouting of post-tensioned tendons is covered in Chapter 4.

Recent history of durability issues in the UK

Durability issues became the subject of much discussion and extensive research and development following the collapse of the Ynys-y-Gwas Bridge in December 1985.

The key features of the Ynys-y-Gwas Bridge collapse, as described by Woodward and Williams (1998) [3], were:

(a) The bridge collapsed without warning.
(b) The bridge was a highway structure built in 1953 and subjected to de-icing salts.

@Seismicisolation

c) The ducts were predominantly unlined, except at segment joints where they passed through ineffective short lengths of cardboard or steel cylinders, such that tendon protection was limited.
d) The segment joints were 25 mm wide and packed with mortar, resulting in permeable areas where water could easily find a path to a tendon.
e) The tendons were generally well grouted but, where grouting was inadequate, corrosion travelled along the length of the tendon instead of remaining focused at the segment joints, as was the case elsewhere.

The main lesson that has been learnt from this collapse was that the integrity of a post-tensioned structure relies on preventing corrosion of the tendons. This is best achieved by the application of multi-layer protection systems, allied to robust design details at construction joints, expansion joints, vents, anchorages and other key areas.

Following this collapse and the identification of defects in other prestressed concrete bridges in the UK, as outlined in the report by the Highways Agency and TRL (1999) [4], a review of key design details, specifications and construction techniques commenced. A moratorium was introduced by the UK's Department of Transport in September 1992 on the new design of post-tensioned bridges using internal tendons, to allow the issues to be fully researched. The moratorium was partially lifted in 1996 after the publication of the Concrete Society's report, TR47 [1], which specified improved detailing and grouting procedures. For precast segmental structures with match-cast joints, the ban was maintained due to concerns over the effectiveness of the protection to the tendons as they pass across the joints, where traditionally the ducts are not continuous.

Subsequently the 1990s witnessed a revival in the use of external tendons. They had become unpopular after a moratorium was introduced in October 1977, following the discovery of corroded wires in the Braidley Road Bridge, built in 1970. The industry had become wary of external tendons due to corrosion and anchorage problems caused by poor detailing and inadequately protected strand. The use of grease and wax for filling external tendon ducts was unreliable at that time as in several cases it leaked out, leaving tendons vulnerable to corrosion. Of the seven externally post-tensioned bridges built in the UK during the first phase of external tendon construction in the 1970s at least two that had strands individually sheathed in PVC had suffered from wire corrosion of such severity that the tendons have now been replaced. The second phase of externally post-tensioned structures in the 1990s saw improved detailing and better protection systems, including the use of HDPE ducts with either cement grout or grease to protect the tendons.

The consequences of corroded tendons are so serious that the reaction of the industry has been very vigorous. The number of bridges affected over the last 50 years has only been a small percentage of the total bridge stock, but even this was considered unacceptable. The numerous specifications, technical guidance, learned papers and other publications that now exist on the topics of grouting, tendon protection, ducting systems and best practice, bear testament to the importance of this subject. The multi-layer protection

@Seismicisolation

philosophy, along with the many refined design details developed, should result in more durable post-tensioned concrete structures in the future.

Guidance on the requirements to achieve durable post-tensioned concrete structures can be found in a number of publications, including the design manual published by the Highways Agency (2001) [5,6] and the book by the Comité Euro-International Du Beton (1992) [7].

Corrosion protection and ducting

Wires, bars and strand are generally used uncoated; however, to protect the tendons during storage or to reduce friction losses during stressing they can be coated with soluble oil which is washed off before grouting the duct. Galvanized and epoxy-coated bars and strand are also available, although neither has been commonly adopted in normal bridge prestressing works.

Pretensioned strand and wire in precast elements is fully encased in the concrete and is well protected from corrosion. The ends of the strands can be painted over where they finish on the concrete surface to provide a water-tight seal and prevent the ingress of moisture. As long as the concrete remains uncracked and sound, there is no direct path for any moisture to reach the strand, although long-term deterioration of the concrete can expose the strands to risk of corrosion, and this requires monitoring.

For post-tensioned arrangements, the different protection systems used for both internal and external post-tensioned tendons are illustrated in Fig. 3.1.

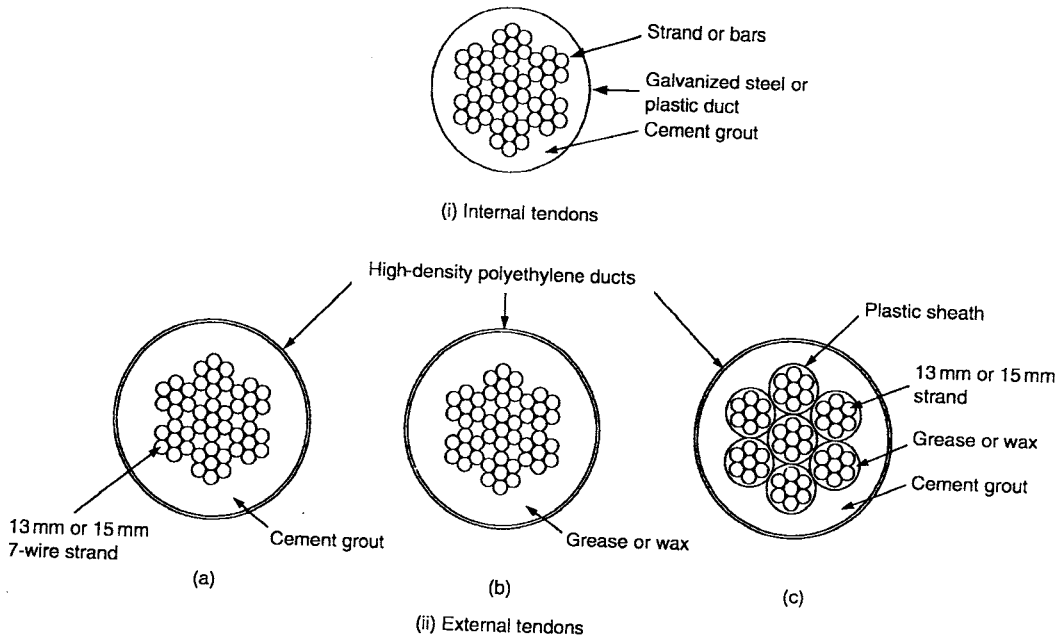


Figure 3.1. Multi-strand tendon protection systems.

@Seismicisolation

Post-tensioned tendons are placed inside ducts to allow them to be stressed after the concrete has hardened and to provide protection to the tendons.

The ducts for internal tendons are traditionally manufactured using galvanized mild-steel strips. In recent years, plastic ducting systems have been adopted to provide a watertight barrier around the tendon as protection against corrosion. Details of the ducting used are given in Chapter 2, 'Ducting'. Plastic or PVC ducting can be air-pressure tested before concreting to confirm its integrity and allow repairs if necessary.

After stressing internal post-tensioned tendons the duct is filled with cement grout that flows into the voids, expelling any water and air while providing a benign environment that discourages corrosion in the pre-stressing steel. Achieving full grouting of a duct can be difficult and requires experienced personnel and the adoption of good site practice. This aspect is discussed in more detail in Chapter 4.

HDPE ducts are used with external tendons. The HDPE ducts need to be strong enough to prevent deformation when pressurized during grouting and to resist the strand punching through at deviated positions during threading and stressing.

There are several different protection systems used with external tendons, as illustrated in Fig. 3.1(ii). The simplest is with the bare strand placed inside the HDPE duct and a cement grout used as detailed in (a). Alternatively, grease or wax filler can be used instead of the cement grout to fill the void as in (b). A third option (c) is for the individual strands to be placed inside a plastic tube with grease or wax filler injected at the factory. The strand and tubes are then placed inside the HDPE ducts and surrounded with cement grout on site.

HDPE provides long-term protection to the tendon, while the advantage of using grease as a filler is that it allows the tendons to be more easily de-stressed and re-stressed or replaced. This is an important feature for external tendons. Where cement grout is used re-stressing of the tendon is not possible, and removal of the external tendon involves cutting it up into short lengths and pulling it out of the deviators and anchor blocks. This requires careful detailing of the duct arrangements.

Concrete

To provide adequate protection to the prestressing steel the surrounding concrete must be well-compacted and free from defects. There must also be sufficient cover to prevent any deterioration in the concrete from penetrating to the level of the tendons during the design life of the structure. Durable concrete is largely achieved by good detailing and construction practices. The factors controlling the durability of concrete and the mechanisms of deterioration are well documented in the publication by the Comité Euro-International Du Beton [7].

Consideration should be given to the risk of cracks forming in the concrete next to the tendons, and the potential width and depths of possible cracks should be investigated. Where appropriate, additional reinforcement is provided to reduce the risk of cracks forming, or the tendons moved away from potential problem areas. The risk of cracking perpendicular to the

@Seismicisolation

prestress tendons can be reduced by applying some of the prestress to the concrete at an early age, in advance of any significant shrinkage or thermal strains occurring.

When designing and detailing the prestress layout and reinforcement arrangement there must always be provision for sufficient space to be available for placing and compacting the concrete. In congested areas it may be necessary to use concrete with small aggregate, and to detail the reinforcement with sufficient gaps for placing and vibrating the concrete.

The area around tendon anchorages can be highly congested, with the reinforcement and concrete subjected to high stresses making the need for dense and defect free concrete paramount. Special provisions, such as temporary access windows in the shutters, should be incorporated to allow easy inspection of the concreting. The windows can also be used to insert pokers to vibrate the concrete as it is placed, ensuring that a well-compacted material is achieved.

Detailing

Good detailing is essential to achieving a durable prestressed concrete structure. The quality, and hence eventual durability, of construction is improved by the use of section shapes and sizes that permit easy concreting. Careful detailing of the reinforcement and prestress tendons to simplify fixing, while maintaining sufficient concrete cover, contributes to long-term durability.

A primary requirement in the detailing is the provision of an adequate drainage system for the efficient removal of any rainwater. This minimizes the risk of water getting below the waterproofing layers or under the deck and causing deterioration of the concrete or reinforcement, or prestressing steel. Potential water paths to the prestressing tendons should be identified and then detailed to reduce the risk of water ingress. This may involve the provision of additional waterproofing or other protection systems.

For box sections, drainage holes, as illustrated in Fig. 3.2, should be provided through the bottom slab at all low points and next to all obstructions within the box, such as diaphragms and deviators. This enables any water entering the box to drain away, preventing ponding on the bottom slab. Drainage holes are particularly important if surface drainage or water mains are carried inside the box and they should be regularly inspected to ensure that they do not become blocked.

Potential leakage points, such as manholes or other penetrations through the deck slab, should be avoided if possible. Where drainage gullies are needed, the detailing of the interface between the gully and concrete deck slab should be such as to make the arrangement as watertight as possible. It is preferable for the surface-water drainage system not to penetrate the deck slab. For short- and medium-length bridge decks, side drains in the verges can be used to carry the water directly off the deck and collected behind the abutments. This is not practical for long bridge decks and drainage gullies are placed along the deck edges, feeding into a carrier pipe which is located either under the deck cantilevers, as illustrated in Fig. 3.3, or within the box section.

@Seismicisolation

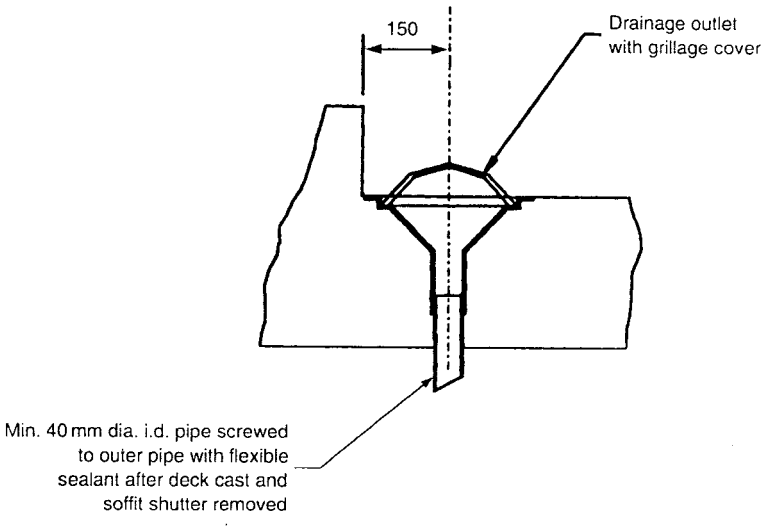


Figure 3.2. Soffit drainage hole. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

Tendon anchorages should be well protected and detailed to minimize the risk of water collecting around them. It is preferable to anchor tendons beneath the deck slab or inside a box section where they are protected, rather than in exposed areas near the top surface of a section where moisture could seep through the construction joints in the concrete.

The ends of a deck at an expansion joint are potentially vulnerable to the ingress of water. This poses a risk for any prestressing tendons anchored there, especially with post-tensioned arrangements. A drainage system

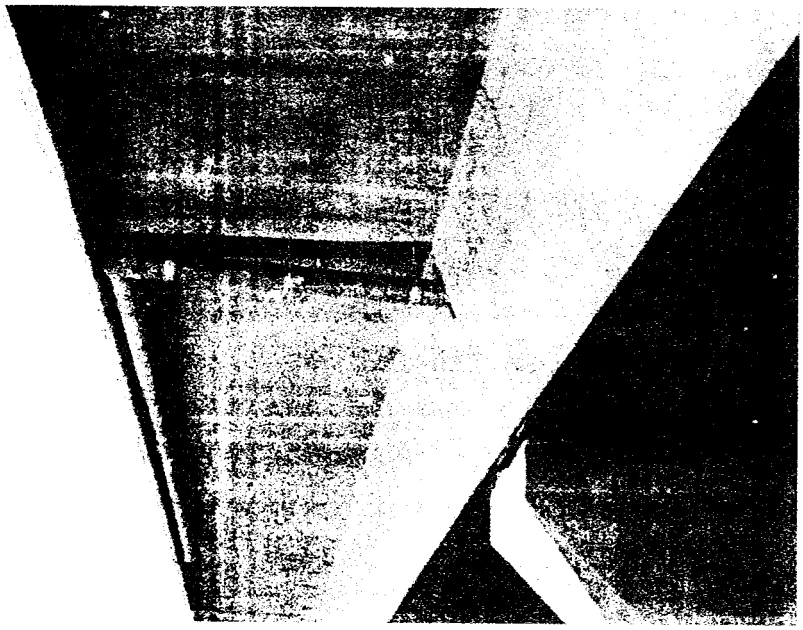


Figure 3.3. Deck drainage.

@Seismicisolation

should be provided under the expansion joint units to collect any water seepage. If possible, the anchors should be recessed away from the expansion joint and extra waterproofing layers applied to protect them.

Small drip channels should be used along the edges of all soffits potentially exposed to rainwater. These prevent the water from running across the soffit, protect the concrete and fixtures, and prevent unsightly staining of the surface.

Advice on detailing for all aspects of concrete bridges can be found in the report by the Concrete Society (2002) [2] and the CIRIA *Bridge Detailing Guide* (2001) [8].

Access

All bridges need good access for regular inspections and maintenance and this is of particular importance for critical items such as bearings, expansion joints and any exposed prestress tendons and anchorages.

Access to the outer surfaces of a deck can often be from the ground using movable scaffolding or working platforms. Where access from the ground is difficult, bridge inspection units as shown in Fig. 3.4 can give good access from the top of the deck. Using this type of plant gives ready access to the outside of the webs and to the soffits of the deck slabs and box, although getting good access between beams or boxes can still be a problem with the larger structures.

Box girder bridges need access into the voids. This is usually achieved at the abutments via the bearing shelf into the end of the deck and along the

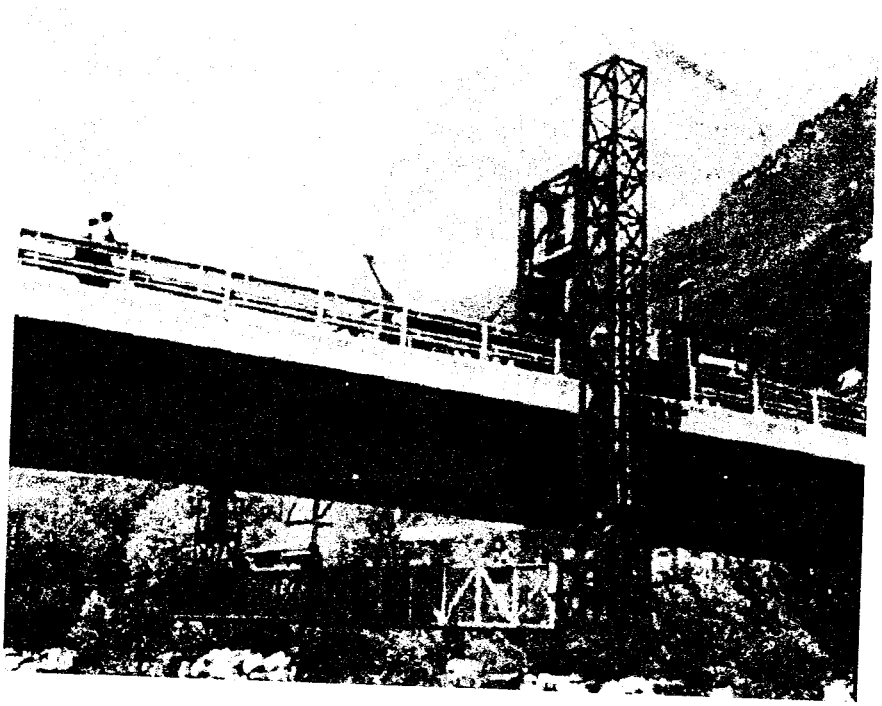


Figure 3.4. Bridge inspection unit. Reproduced courtesy of MOOG GmbH. Copyright reserved.

@Seismicisolation

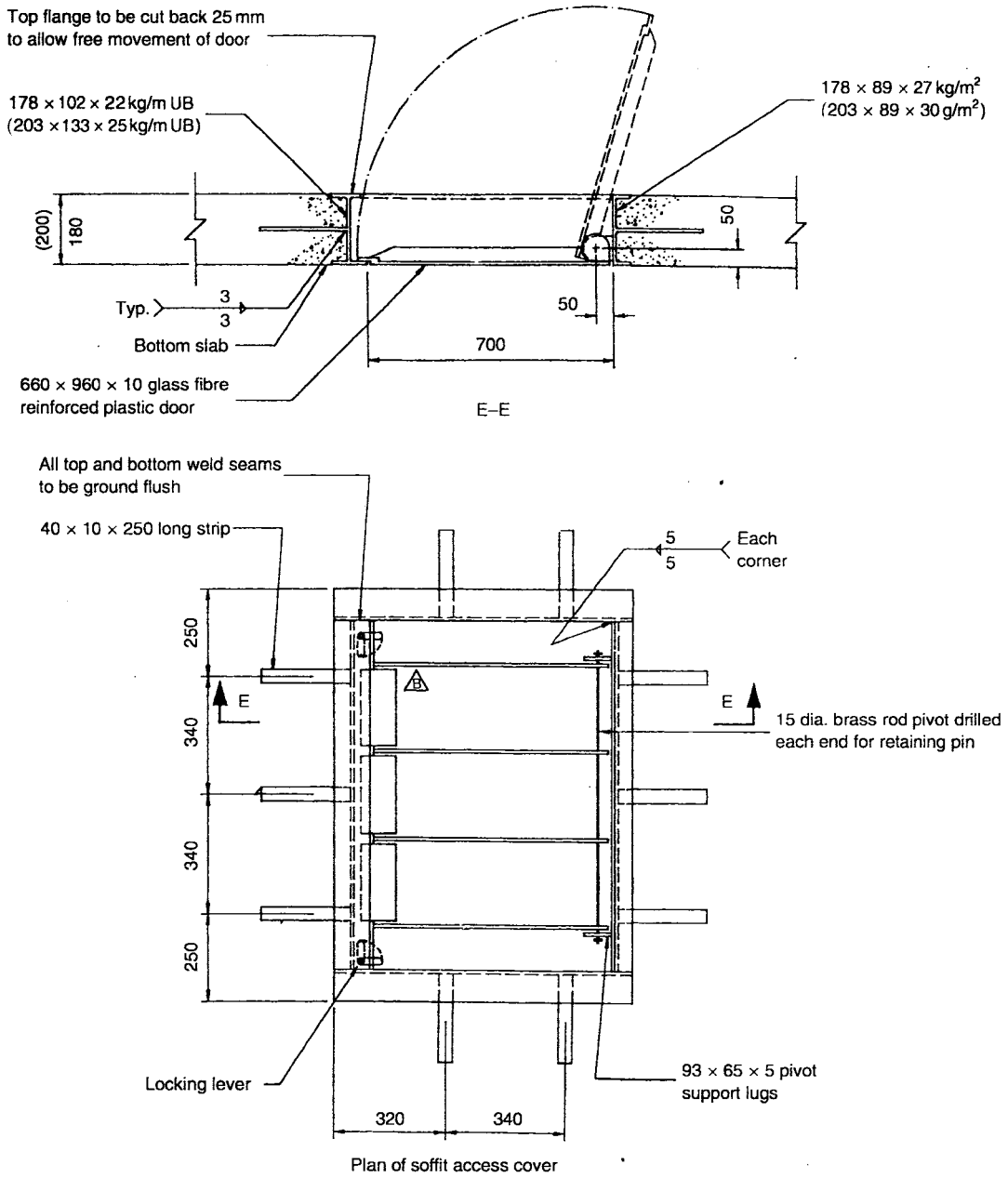


Figure 3.5. Soffit access hole.

length of the deck via soffit access holes as illustrated in Fig. 3.5. The soffit access holes are usually located towards the ends of a span and at locations where there is easy access from the ground. They are typically spaced between 150 m to 200 m apart, provided there is good access through

@Seismicisolation

any intermediate diaphragms. Lockable covers are used to give a secure arrangement.

At non-integral abutments for box-type decks, inspection galleries are provided between the end of the deck and the abutment backwall as illustrated in Fig. 3.6. Where expansion joints are located in the deck slab, the arrangement should allow easy access to all parts of the joint. At these locations there are often post-tensioned tendon anchorages which should also be accessible. Entrance to the inspection gallery is usually via a

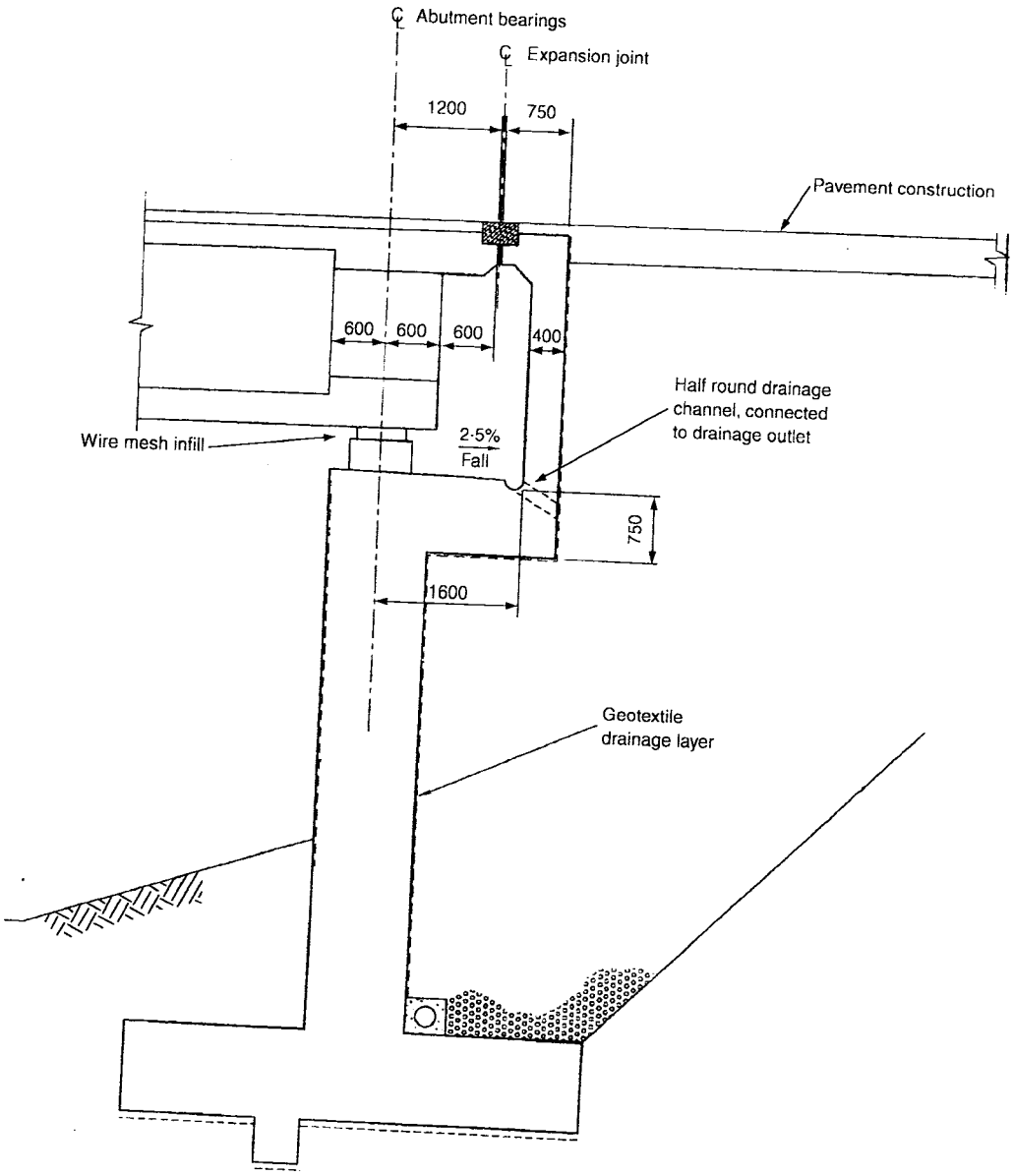


Figure 3.6. Abutment inspection gallery.

@Seismicisolation

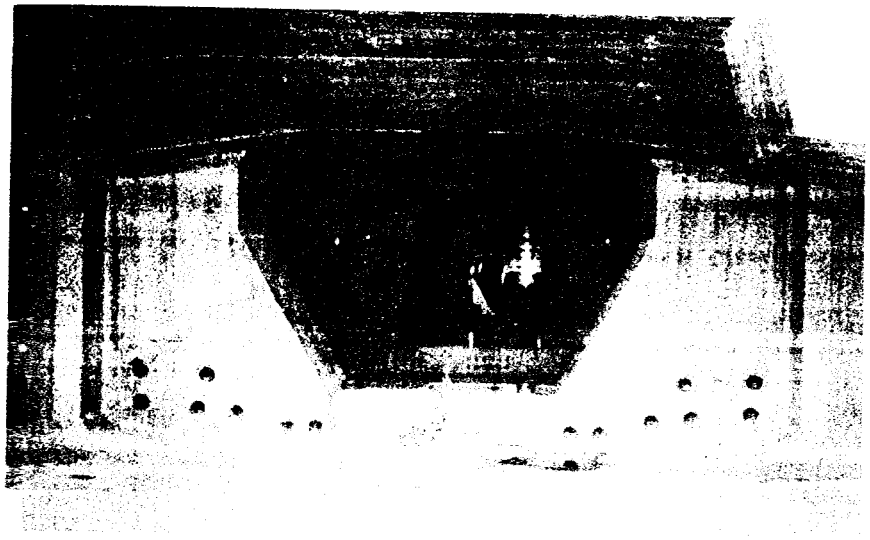


Figure 3.7. Access through box girder diaphragms.

secure door system built into the abutment side wall with a path constructed to provide access for transporting any equipment needed. The end diaphragm in the deck is detailed with a large hole through it to allow access into the box.

For cellular or box girder decks access holes are provided through the internal diaphragms as seen in Fig. 3.7. Multi-cell boxes usually have access holes through the internal webs so that all the voids can be inspected and maintenance carried out. Although these holes are normally small to minimize their impact on the design they should be made as large as possible to provide good access.

Stray current protection

For bridges supporting or near electrified railway systems, power lines or other sources of electrical currents, the prestress tendons and passive reinforcement should be earthed to protect them from the stray currents that can occur, which cause corrosion in the steel. DC-powered railway systems present the greatest corrosion risk, although it is also usual to protect structures where AC power supplies occur.

All the prestressing tendons and reinforcement bars should be connected to provide electrical continuity and to provide a path for the stray current to travel through the structure and down to the ground. Prestress tendons are good conductors of electricity in themselves and are normally connected to the reinforcement cage by a copper strip, or similar conducting material, that is fixed between the anchor block and the adjacent reinforcement. The reinforcement cage should have additional mild steel bars incorporated that are welded to the cages at appropriate locations to ensure good continuity in the connection. Alternatively, a special earthing cable can be provided along the complete length of deck and the tendons and reinforcement connected to it. At appropriate piers, or at the abutment, the deck reinforcement is connected to

@Seismicisolation

the substructure reinforcement by a copper strip spanning across the bearing gap. The stray current is then taken down to ground level and connected to earthing rods, formed from copper bars installed into the ground to provide a good earthing system.

External tendon replacement

One of the advantages with external tendons is the ability to re-stress or remove and replace them in the future if required. To enable this to be possible the detailing of the tendon at the anchorages and deviators has to allow replacement to take place.

The anchorage arrangement for post-tensioned bars is readily suited to re-fixing the jack and detensioning where grease or wax filler has been used instead of cement grout.

Each end of a multi-strand external tendon is anchored using a standard arrangement, similar to that shown in Fig. 2.3, but with special provisions incorporated to allow the tendon to be de-stressed and replaced without damaging the anchor components. If the duct is to be cement-filled this involves providing a double-sleeve arrangement inside the trumpet to allow the tendon to be pulled out, with typical arrangements illustrated in Appendix D. The strands at the stressing end of a tendon with wax-filled ducts are left with sufficient length extending behind the anchor block to allow a jack to be fitted to enable de-tensioning or re-stressing of the strands.

Sufficient space is required around the anchorages of external tendons, and along their length, to give access to remove the old tendon and to install a new one. Access to the structure also needs to be adequate for the stressing and grouting equipment to be handled and positioned. This involves using larger access holes and entrances compared with those needed for routine maintenance. Good access should be provided at each abutment, with full-size doors into the abutment-access chamber and large openings through the deck diaphragms. Where soffit access holes are provided they should provide a clear space of at least 1·0 m × 0·8 m.

Removal of the tendons first requires them to be de-stressed. Tendons in grease- or wax-filled ducts are de-stressed using a stressing jack to release the force at the anchor. The tendon is then pulled out of the duct and the grease or wax collected. Tendons in cement-grout-filled ducts have to be cut into sections to allow them to be removed. In this case, the ducting and grout are removed locally to expose the tendons in the area to be cut and the strands severed by cutting or burning, thus releasing the force. Another technique employed is to burn out the wedges at the anchor blocks, which releases the strands. Special precautions are needed to prevent a sudden release of the large forces that are in the tendons and to protect the operatives in the vicinity. De-stressing of tendons should be carried out by specialists with experience in this type of operation.

References

1. Concrete Society (1996) *Technical Report No. 47, Durable Bonded Post-Tensioned Concrete Bridges*. Slough: Concrete Society.

@Seismicisolation

2. Concrete Society (2002) *Technical Report No. 47, Durable Bonded Post-Tensioned Concrete Bridges*, 2nd edition. Slough: Concrete Society.
3. Woodward, R.J. and Williams, F.W. (1998) 'Collapse of Ynys-y-Gwas Bridge, West Glamorgan', *Proceedings of the Institution of Civil Engineers*, Part 1, **85**, 635–69.
4. Highways Agency and TRL (1999) *Post-tensioned Concrete Bridges Anglo-French Liaison Report*. London: Thomas Telford Publishing.
5. Highways Agency (2001) *(MDRB) Departmental Standard, BD 57/01 Design for Durability*. Norwich: HMSO.
6. Highways Agency (2001) *(MDRB) Advice Note, BA 57/01, Design for Durability*. Norwich: HMSO.
7. Comité Euro-International du Beton (1992) *Durable Concrete Structures*, 2nd edition. London: Thomas Telford Services Ltd.
8. CIRIA (2001) *Bridge Detailing Guide*. London: CIRIA.

@Seismicisolation

4

Grouting post-tensioned tendons

Introduction

The process of introducing grout into the duct of a post-tensioned tendon is a simple and effective way to improve the overall durability of the structure. The grout encases the strand or bar and acts as a protective layer to prevent corrosion. Cement-based grouts are the most common, although grease or wax fillers are sometimes used with external unbonded tendons.

The grouting process involves pumping the fluid grout into the duct at an inlet, which is at the lowest point and often part of an anchorage arrangement. As the grout flows through the duct it comes out at intermediate vents along the length of the tendon, until it finally emerges at the outlet at the far end of the tendon, having filled the duct. Cement-based grouts are pumped into the ducts on site after completion of the stressing operation, while grease or wax fillers are often injected into pre-assembled tendons at the factory.

The complicating factors include such issues as inadequate design details, insufficiently robust ducting systems, material variability, time constraints, equipment problems, meaningful testing, duct blockages and segment joints.

This chapter reviews the problems encountered in recent years and primarily discusses the process of grouting with cement-based grout. The procedures needed to achieve fully grouted ducts are described. Grease and wax grouts are briefly covered towards the end of this chapter.

Recent history of grouting

Recent years in the UK have seen a refinement of the requirements for the grouting of tendons, including more detailed and exacting specifications. The collapse of the Ynys-y-Gwas Bridge, in December 1985, highlighted historical deficiencies in the grouting process and related design and specification issues. It also emphasized the importance of effective grouting to prevent corrosion. Subsequent extensive research resulted in development of the specifications and procedures used for grouting, leading to the publication of the Concrete Society's report TR47 (1996) [1]. In the past, the effectiveness of grouting varied from project to project, as the general guidelines available did not fully address all the issues. Consequently, the quality achieved was largely a function of the experience

@Seismicisolation

and expertise of the individual design engineers, site supervisors and operatives.

The extensive programme of inspections of prestressed concrete bridges in the UK demonstrated that many early post-tensioned decks achieved well-grouted ducts; however, grouting defects and tendon corrosion were found in a significant number of cases. A small percentage of these bridges had unacceptably severe tendon corrosion. Consequently, new standards were adopted for all new projects in the UK. These standards are currently not used extensively outside the UK although this is likely to change as other countries work towards improving the effectiveness of grouting.

The current UK standard is the Highways Agency's *Manual of Contract Documents for Highway Works* [2], which gives the specifications for the grout material and procedures to be adopted. It can be adapted for individual projects to take into account specific requirements and latest developments in the industry.

Research since 1996 has resulted in the publication of the 2nd edition of the Concrete Society's report TR47 (2002) [3]. This introduces further tests for grout that are intended to improve the assessment of how the grout will perform in a duct under the particular conditions experienced in practice.

Grout material

Grout for filling post-tensioning ducts is traditionally a mix of water, cement and admixtures. Mixes are developed to limit certain detrimental properties, and designed to pass certain limiting tests such as specified in the Highways Agency's *Manual of Contract Documents for Highway Works* [2]. The 'design' or selection of such a mix on this basis could be inappropriate given the inadequate nature of the commonly specified tests that do not always fully reflect the performance of the grout in practical situations. Grout-mix designs need to be developed to suit the individual project requirements and to give the required end product of a fully grouted tendon.

Uniformity in the quality and properties of the basic ingredient materials is critically important to the consistent production of a grout. Due to the variability of the performance of grout made from ordinary cements, the UK industry is moving towards the use of special grouts, made from pre-bagged materials with only the water measured and added on site. It is also moving away from the traditionally specified grout material tests, with new performance tests being introduced.

The current European Standard for grout, BS EN 447 (1997) [4], covers the traditional approach to grouting and testing and should be used in conjunction with more recent guidance such as that given in TR47 [3]. Grout should always be made with Portland cement complying with BS12 [5] Class 42.5N or cement type CEM1 complying with BS EN 197-1 [6] or similar standards.

Admixtures are supplied in either a liquid or powder form and can have expanding or non-expanding properties. Proprietary admixtures may also include fluidifying, water reducing and super-plasticizing agents. As the use of special grouts becomes the norm, admixtures should become available pre-measured to the correct dosages.

@Seismicisolation

Trials should be carried out to ensure that the mix performs in the intended way, fully filling the duct and encasing the tendon. Commonly these take the form of small-scale 'suitability' trials that test the material properties, and large-scale grouting trials where tendons and duct details replicating those in the works are grouted following the proposed procedures. The effectiveness of the grouting is determined by cutting up the completed tendons in the large-scale trials and visually inspecting the grouted ducts. These trials are discussed in more detail later in this chapter.

During winter it may be necessary to heat the water for the grout as well as storing the constituent materials in controlled temperatures. In concrete box girders the grouting location temperature can be maintained above 5 °C utilizing electric heaters inside the deck, and suspended sheeting to cordon off work sections.

In the summer, the hotter environment reduces the fluidity of the grout and shortens its working life, which can create problems during grouting. This can be countered by adding ice to the water supply or installing air-conditioning units in the material storage facilities. The normal specification requirement is to wash out grouting plant every three hours. This should be carried out more frequently in hot weather to prevent setting grout blocking critical areas of the grouting plant and tendon ducts.

From the provision of the basic ingredients to the mixing and injection of the grout, the quality control of the material is critical to the successful completion of the grouting process.

Grout material tests

Traditionally the fluidity of the grout after mixing and before installation in the works is measured by using a flow cone of known dimensions, as illustrated in the Concrete Society's report [3] and shown in Fig. 4.1. Additional tests for volume change, homogeneousness, sedimentation and grout strength are also often carried out.

The flow-cone test measures the time for a known quantity of grout to pass through the cone and is a gauge of the grout's behaviour; however, this does not necessarily indicate how the grout will perform when passed through the small spaces in a tendon duct or when pumped under pressure. Thixotropic grouts, used in special circumstances, flow when pumped but come to an equilibrium when not agitated and therefore cannot be tested by the flow-cone method.

The bleed properties of the grout used to be measured using traditional 'jam jars', where the bleed and the volume change are measured at fixed times against a height of grout standing in the test cylinder. The bleed is simply the emergence of water from the mixed grout. This test gives no overall picture of how the grout will perform under pressure, in a profiled duct or passing through restrictions. A trial of new tests, seen in Fig. 4.2, was carried out on vertical and inclined tubes, and with and without prestressing strands inserted. These tests showed that the presence of the strands increased the bleed from the grout.

@Seismicisolation

Figure 4.1. Flow-cone test. Reproduced courtesy of Louise Smith. Copyright reserved.

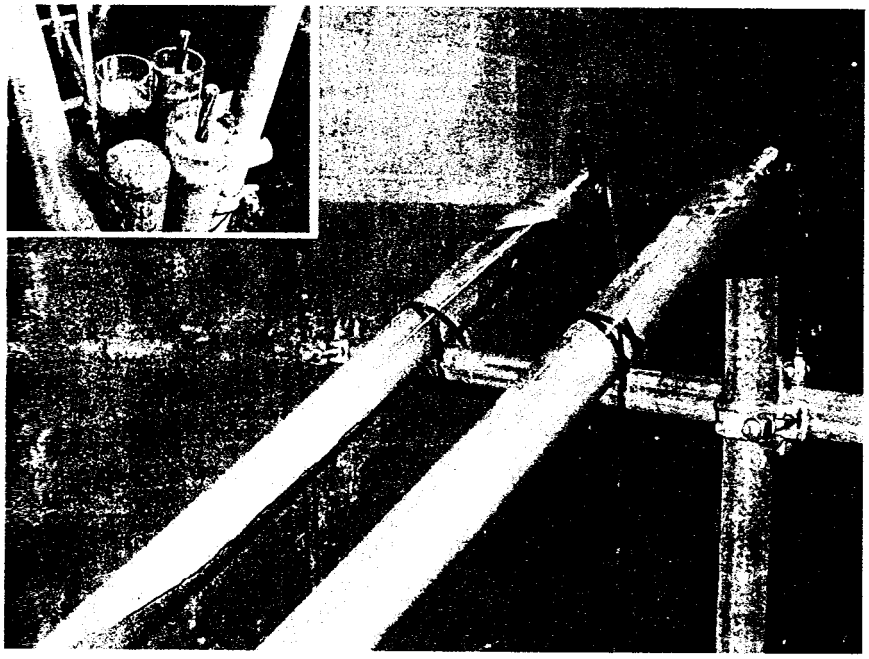


Figure 4.2. Inclined duct test. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

The Institution of Engineers, Malaysia
Bangunan Ingenieur, [illegible]
46720 Petaling Jaya, [illegible] Selangor, Malaysia.

A 'no bleed/no shrinkage' grout is the perfect choice of material but, when gauged against the traditional bleed test, no true picture of its properties can be seen. Bleed water emerges from the grout under pressure, travelling along individual strands, which act like wicks, to high points. The resultant void, filled with air and water, provides a perfect environment for corrosion.

The second edition of TR47 [3] addresses this issue by introducing a new test for bleeding. The grout is poured into a 1·5 m-high vertical duct with prestressing strands inside, and the height of the grout and amount of bleed water measured over a period of time.

'Flowmeters', which measure the amount of air in a fluid grout, have also undergone trials, giving operatives the possibility of using such measuring devices to watch grout flow into a duct and to determine the percentage voids in the duct whilst the grout is still fluid.

Grout material tests should be carried out using the same material, mix design, plant and operatives as proposed for the permanent works. The environmental conditions should also be similar to those expected, with the temperature and humidity controlled.

Grouting equipment

Grout mixing on site is carried out using special mixing pans as shown in Fig. 4.3. These are used to mix the cement, water and any admixtures required to give a homogeneous grout. For long tendons, which require large quantities of grout, several mixing pans are needed; or a larger holding tank for mixed grout can be used to store two or three batches of grout. This prevents delays during the grouting which can cause blockages if the grout is allowed to stand for too long.

A grout pump, often combined with the mixing pans, takes the fluid grout from the pans and pushes it into the duct via the inlet. The grout is usually pumped with a pressure up to 1 N/mm^2, although if grouting is carried

Figure 4.3. Grout mixing equipment. Reproduced courtesy of Louise Smith. Copyright reserved.

@Seismicisolation

Bangunan Ingenieur, Lots 60/62, Jalan 52/4,
46720 Petaling Jaya, Selangor Darul Ehsan, Malaysia.
(LIBRARY)

out above the pump the additional head can increase the working pressure up to 3 N/mm^2.

All grouting equipment should be duplicated on site to ensure that back-up is always available in case of breakdowns. The use of easily replaceable fittings allows quick resolution of problems encountered during the grouting operation.

Vents and other details

The ability of the grout to flow round the strand over the whole tendon length is influenced by certain duct and vent parameters. Guidance on duct diameter is normally recommended by the suppliers of the prestressing system but is also given in other technical publications such as that by the FIP (1990) [7] and clause 9.26.4 of AASHTO [8]. The area of the duct should be at least twice the steel area of the tendon inside.

Anchorages are usually treated in one of two ways. The most straightforward method is to trim the protruding strand to the minimum permissible length, usually a strand diameter, and then place a cap over the anchor head to restrain the grout, as shown in Fig. 4.4. This cap normally has a small vent hole with the grout pumped in via an inlet tube attached to the anchor trumpet, although some post-tensioning systems have a large hole in the bearing plate to allow the grout to be pumped in via the anchorage cap. The second method is to provide a capping of concrete to encase the anchor head and protruding strand as seen in Fig. 4.5. This provides a watertight seal for the grouting operation with an inlet tube cast into the concrete to allow the grout to be pumped into the duct.

Figure 4.4. Anchor cap with grout inlet. Reproduced courtesy of Louise Smith. Copyright reserved.

@Seismicisolation

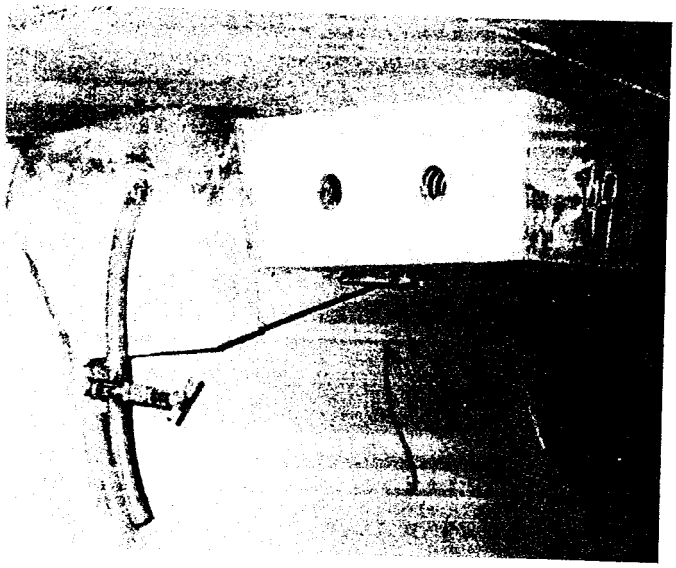
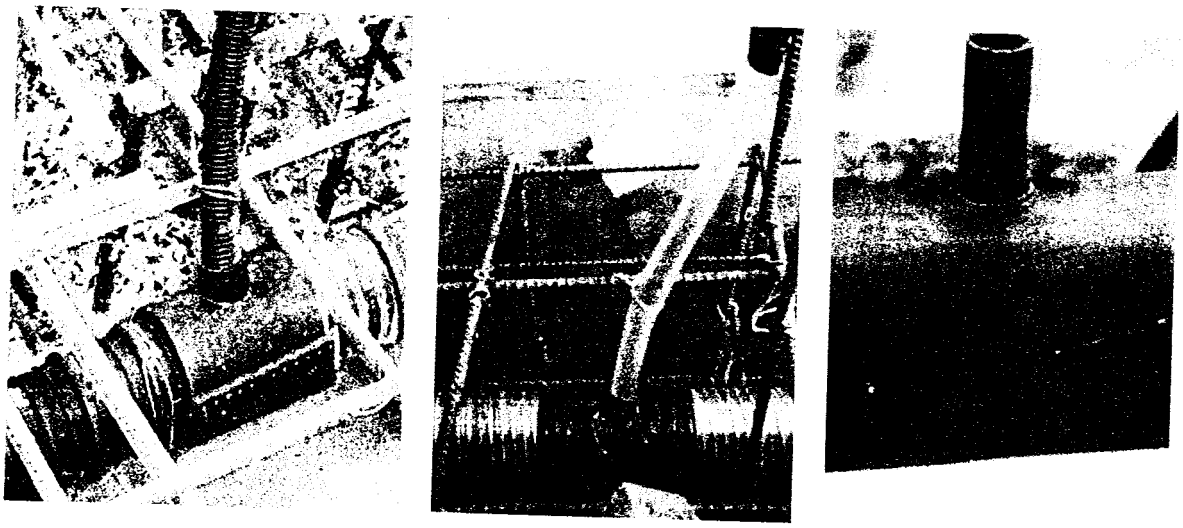


Figure 4.5. Concrete capping to anchorage.

Inlet tubes are needed to allow the grout to be pumped into the duct and should have a minimum internal diameter of 25 mm. The inlet is provided at the lowest anchorage point allowing grout to be pumped 'uphill' into the duct, as this helps to expel any air along the length. Outlet tubes are required to vent off the air and to ensure that the ducts are full of grout. These can be 12 mm diameter, but it is normally recommended to make these 25 mm diameter so that they can also be used to pump grout into the duct should the need arise due to blockages elsewhere.



(i) Plastic duct (ii) Steel duct (iii) HDPE external duct

Figure 4.6. Grout vents.

@Seismicisolation

Vents are provided at each low point, both to drain out any standing water and to draw off any trapped air. Vents should also be provided at, and just after, each high point to allow air and water to be removed from the crest during the grouting operation, and after by re-opening these vents. A vent or outlet tube is provided at the 'far' end of the duct or far anchorage to ensure that the duct is filled along its full length. Typical arrangements for vent connections with plastic, steel and HDPE ducts are shown in Fig. 4.6.

Vents should be provided at regular intervals along ducts with long 'flat' profiles to remove trapped air that can occur as the grout runs along the bottom of the duct. The maximum vent spacing is typically less than 15 m.

Difficulties can occur with external tendons where the duct has a crest through a wide deviator or diaphragm such that the high point is hidden within the concrete and therefore inaccessible. To prevent air being trapped and forming a void in the duct, vents are positioned as close to the crest as possible on either side of the diaphragm. Although this does not always prevent a void, it should be small and the grout should still provide cover to the strand. Alternatively a small diameter tube can be inserted through the top of the duct, with the connection positioned at the crest and the free end of the tube leading outside the concrete. This method is described by Smith and Wood (2001) [9].

Grouting trials

Grouting trials are used to demonstrate the acceptability of the proposed grouting method, grout mix, equipment, operatives and vent arrangement before application in the permanent works. Typically, a 40 m to 75 m-long test bed is constructed incorporating provision for several different tendon profiles. The tendon profiles chosen for the grouting trial should replicate the tendon profiles proposed in the permanent works.

After installation within the duct, the tendon should be sufficiently stressed to cause it to follow the required profile. The anchor should then be capped and the tendon duct pressure-tested using air or water as appropriate before carrying out the grouting. The same operatives and supervisors that are proposed for the grouting of the permanent works should also undertake the grouting trial.

The grouting of the trial length should proceed without any delays in the process and without any blockages or leakages in the equipment. The trial can indicate where procedural refinements are required as well as providing useful information about the performance of different types of duct, vent fittings and grout material.

After the grout has set, the tendon and duct are cut or cored to allow inspection at critical sections. These should include the crests in the profile and at locations near the anchorages and vent positions. The grouted duct should not contain excessive voids likely to put the tendon at risk. Voids with dimensions of less than 5 per cent of the duct diameter are often considered acceptable, especially in areas away from the strands.

@Seismicisolation

If the initial trials are unsuccessful, further trials should be carried out until all the problems are resolved. Lessons learned from the trial should be incorporated into the design and grouting procedures.

Pre-grouting test (pressure testing) of the ducting system

There is a need to prove the integrity of the duct system before grouting commences to ensure that leaks do not occur. For internal tendons with plastic ducts this is carried out before concreting, and for external tendons or internal tendons with corrugated steel ducts before grouting. Traditionally this is proved by blowing air through the duct or by water testing, which involves the filling of the ducts with pressurized water to aid visual determination of any leaks. In many circumstances the more rigorous investigation with air pressure testing gives better results, and is now a mandatory requirement for most work in the UK.

Pressure testing is both labour intensive and time-consuming, even for fully accessible external post-tensioning systems. Benefit would be gained from further development in the design and manufacturing of the ducting, to ensure that all joints become more positive and reliable.

Much work has been done recently to develop a closed system of ducting to provide a fully watertight enclosure. These duct systems are manufactured using PVC or plastic for internal tendons and HDPE for external tendons. They are the only forms of duct currently permitted in the UK. Elsewhere in the world it is still common to use corrugated steel ducting for internal tendons. Manufactured from steel strips and crimped together, corrugated steel ducts result in a non-watertight arrangement.

Air pressure testing

Air testing under pressure is used primarily on plastic and HDPE ducts to check the integrity and potential water-tightness of the duct and fittings. The testing process can also indicate, by opening all the inlets and vents and allowing the air to flow through, that the vents, ducts, anchorages and other areas are all free from blockages. However, the passage of air is no guarantee that grout will also be able to flow freely.

Testing in two phases can be useful. First, the unvented system is tested to locate major leaks in the duct itself; and second, testing is carried out with all vents and associated fittings added to determine the soundness of the vent work. The fittings for some plastic duct systems may not be able to sustain high pressures, even though the duct may still give an overall satisfactory performance, and this needs to be considered when choosing the system to be used.

Leak repair techniques normally involve patching, by placing a section of duct over a discontinuity and 'welding' it in place, or welding and sealing a discontinuity by applying a length of hot bead from a polyethylene strip.

Either of the above techniques, when undertaken carefully, produces repairs that should pass air testing; however, there may still be a risk of

@Seismicisolation

leaks during grouting. Alternatively, easily replaceable fittings, including prefabricated saddles and rubber collars, can be used. Duct discontinuities can then be resolved quickly and easily before concreting or grouting starts, as appropriate.

Leakages in an area of duct that lies within a deviator or anchorage location can prove difficult to resolve for external tendons. In these locations the duct is effectively hidden behind concrete and therefore excluded from any standard repair technique. If the duct is air tested before the tendon is stressed it is possible to remove it from the deviator or anchorage and make a repair before re-installing. If the tendon has already been stressed then it would need to be either de-stressed to allow the duct to be removed and repaired or repaired *in situ* if this is feasible.

The following issues are important during air testing:

(a) Construction sites are noisy. The air pressure selected should be high enough to allow rapid audible location of leaks.
(b) With regard to the issue of reasonable percentage loss in pressure, TR47 suggests that 10 per cent of 0·01 N/mm^2 over 5 minutes is acceptable. Air loss through couplings in airlines may easily exceed this limit and should be taken into account when assessing the acceptability of any test. On large-scale grouting operations there is the opportunity to investigate acceptable loss levels and develop suitable criteria for the specific project parameters.
(c) Application of soap solutions to all couplers, vents and caps clearly indicates any leaks by producing bubbles where air emerges through small gaps.
(d) Micro-cracking of concrete in the anchorage zone is a feature of post-tensioning. Leakage through such cracks cannot be identified by post-concrete pressure testing of anchorage components before post-tensioning. However, tendons thus affected, which could not hold air pressure, could be grouted successfully. Making this judgement is a matter of experience.
(e) Air-testing pressure losses are not necessarily an indication of behaviour during grouting.

In summary, air testing does not necessarily indicate that water egress will occur at connections during grouting or that there is any risk of water ingress at connections later; however, the test is useful for determining the integrity of the ducting and in identifying potential problem areas.

Water pressure testing

This test is often used to identify leaks in internal corrugated steel ducts and is carried out before or after installing the tendons. It has been a common practice to water-test internal tendons in precast segmental construction, where the 'O' ring seals around the ducts at the segment joints are a potential weakness. It is also used in non-segmental construction and cast *in situ* segmental construction. Water is pumped through the ducts and out of all the vents and outlets to ensure that an even flow can be achieved. The vents and outlets are then closed and the water pressurized to a level

@Seismicisolation

similar to that required during the grouting process. The pressure drop over a period of time is monitored as an indication of the integrity of the ducts and the seriousness of any leaks.

During water testing, locations where water emerges on the inside or outside faces of the concrete are easily identified and repaired. Cross-linking between tendons is also identified by the emergence of water from the vents of an adjacent duct.

After completion of the water testing, all residual water is removed from the duct by blowing air through and by providing drainpipes or vents at all low points where appropriate.

Leakages

Grout leakages can occur with any form of deck construction, although the pre-grouting tests described above should minimize their occurrence. For simple structures and straightforward prestress tendon layouts the risk of leaks is small, but for structures with more complex tendon arrangements the risks are significant.

In balanced cantilever construction, top tendons are commonly anchored on the end faces of the segments. Face anchors located at the top of the webs or in the top flange are usually contained within a thin-walled 'box' formed during concreting of the segment. This box is very prone to leaks and cross-linking with other anchor boxes.

It is desirable to grout tendons including their anchors in one go, feeding the grout through one anchor and letting it out at the other anchor. This allows the grout to be forced through all the interstices of the strands, filling all small holes to give a product that has minimum voids. However, it is common that the weakest area for holding grouting pressure is the face anchor box. This can be identified during pressure testing. Remedies include large temporary patching where the leak is visible on the concrete, and breaking out the wall of the box on the inside of the segment to effect a repair. The latter is clearly not recommended, as breaking out concrete adjacent to live tendons can be hazardous.

Another solution is to fill the box with grout to seal the problem area first and then grout the rest of the duct in a separate operation. This is best achieved by using a special thixotropic grout, which can be poured into the anchor box through the box vent, emerging through the top flange or the top of the web during the first stage of the grouting operation. The thixotropic grout, if not agitated, will fill the anchor box without passing through into the duct and jeopardizing the second-stage grouting operation. Monitoring the quantity of grout poured in gives warning of any leaks. The disadvantage of this method is that the interstices of the strand will not be completely filled with grout in the anchorage zone.

Precast segmental concrete box girder bridges typically utilize relatively thin members for the top and bottom slabs and walls. Ducts pass through the ends of segments at standard positions to minimize the effect on the segment mould bulkhead. In the top flange at pier locations and the bottom flange at mid span, tendons are usually closely spaced. In addition to the risk associated with the

@Seismicisolation

tendons in close proximity, stripping and handling damage of the concrete can easily produce a leak or cross-linking with another tendon.

Good detailing and construction practice is vital to prevent cross-linking and grout leaking in segmental decks. Ducts should be as widely spaced as other design considerations permit. For pre-cast match-cast segmental construction, effective 'O' ring seals must be used around each duct at the segment joint and the epoxy filler between the segment ends must be evenly spread over the faces to fill up all gaps.

If cross-linking between ducts occurs, this can be overcome by simultaneously grouting the affected ducts. Identification of the potential cross-linking by air or water testing and planning of the grouting process to control the flow of grout between the ducts is essential to minimize the risk of air becoming trapped.

The above problems with leakages and cross-linking largely explain why there is still a moratorium on the use of internal tendons in pre-cast match-cast segmental construction in the UK.

Grouting procedure

Once pressure testing has been carried out successfully, grouting of the duct can take place. Where possible, groups of tendons should be grouted together so as to minimize any risks of leakage between adjacent ducts and to simplify the overall grouting process.

If there is any water in the ducts this should be removed as much as possible before the start of grouting. It is not always practical to remove all the water from the ducts and small quantities present do not usually present a problem, as it is expelled during the grouting process.

Grout is introduced via the inlet at a slow, even rate, maintaining a pressure of 0–1 N/mm^2 and grout is allowed to emerge sequentially from intermediate vents before they are sealed. For external tendons the rate of progress of the grout along the tendon should be established audibly by tapping of the duct.

At intermediate vents and the anchorage outlet at the end of the tendon, a quantity of grout is allowed to vent before slowly closing the tap. All vents should be closed in sequence in the direction of grout flow, with the outlet being closed last. It is useful to hold the end of the vent pipe below the surface of the collected grout in a bucket. Any air being pushed out of the duct is seen as a series of bubbles in the bucket of grout. Grouting should continue until the air bubbles are no longer visible. The outlet grout should be collected and subjected to a fluidity test using the flow cone discussed earlier to ensure that it still complies with the specified requirements.

The grout in the duct system should be pressurized for a short period, allowing any air voids to rise up to the crests in the duct profile. The vents at the crests are then reopened one at a time to release any air present. Finally, with all vents closed again, the system is re-pressurized and held for a specified period before finally being sealed.

Experienced operatives should be used to undertake the grouting, and all procedures regularly checked as part of the quality-control requirements.

@Seismicisolation

Good communications are needed between the operatives, particularly where clear lines of sight between all involved are not possible.

Following grouting, vibration on the structure should be prevented for 24 hours to allow the grout to set and harden.

Post-grouting checks should include inspection of vent pipes to ensure that they remain full of grout. For external tendons, tapping of the duct can indicate if any air voids are present when a hollow sound is recorded. Investigations involving drilling into ducts should be carried out with care to avoid damage to the tendons.

Specifications usually require provision to be made for adequate flushing out of a duct with water if there is a problem, such as equipment failure or grout blockage. The specification usually requires 'complete removal of the grout'; however, this can be very difficult to achieve. The washing-out water tends to flow over the top of the grout without actually disturbing it. Even the addition of compressed air into the water only affects a localized area. It is debatable whether flushing out should be attempted in favour of a later re-grouting exercise.

Grease and wax grout

Grease and wax grout is used with external prestressing to enable easy de-stressing or replacement of the tendons. The grease or petroleum wax is injected into the duct at between 80° to 90 °C, before cooling to a soft, flexible filler. Additives are often included in the wax to help penetration through the wires and to inhibit corrosion.

With pre-assembled tendons or where tendons are individually sheathed, the wax is injected at the factory in controlled conditions. For larger and longer tendons it is often necessary to inject the wax on site, requiring specialist equipment for storage and heating of the wax. Wax grouting follows a similar procedure to cement grouting except that injection pressures are typically 0·2 N/mm^2.

Where external tendons are required to be re-stressable, sufficient length of strand is left protruding to allow a stressing jack to be re-attached. An elongated cap covers the protruding strand so that the length of tendon immediately behind the anchorage can be protected by grease or wax infill.

References

1. Concrete Society (1996) *Technical Report No. 47, Durable Bonded Post-Tensioned Concrete Bridges*. Slough: The Concrete Society.
2. Highways Agency (1998) *Manual of Contract Documents for Highway Works, Volume 1, Specification for Highway Works*. Norwich: HMSO, Clause 1711.
3. Concrete Society (2002) *Technical Report No. 47, Durable Bonded Post-Tensioned Concrete Bridges*, 2nd edition. Slough: The Concrete Society.
4. BS EN 447 (1997) *Grout for Prestressing Tendons – Specification for Common Grout*. London: British Standards Institution.
5. BS12:1991 (1996) *Specification for Portland Cement*. London: British Standards Institution.

@Seismicisolation

6. BS EN 197-1 (2001) *Part 1, Composition, Specification and Conformity Criteria for Common Cements*. London: British Standards Institution.
7. Fédération International de la Préconstraint (FIP) (1990) *Guide to Good Practice, Grouting of Tendons in Prestressed Concrete*. London: Thomas Telford Publishing.
8. AASHTO (1996–2002) *Standard Specification for Highway Bridges, 16th Edition*. Washington, DC: American Association of State Highway and Transportation Officials.
9. Smith, L.J. and Wood, R. (2001) 'Grouting of External Tendons – A Practical Perspective'. *Proceedings of the Institution of Civil Engineers*, February, Issue 1, **146**, 93–100.

@Seismicisolation

5

Prestress design

Introduction

The design of the prestress for a bridge deck is integral to the overall design process, and it must take into account the behaviour of the structure as a whole under the applied loads and during the construction stages. Application of the prestress affects the structural behaviour, and the design process requires a clear understanding of the interaction between them in order to achieve a safe and efficient structure.

The general principle with prestressed concrete design is to ensure that the concrete is either in compression or within acceptable tensile stress limits under service load conditions. Prestress also enhances the shear and ultimate moment capacity of the concrete section.

This chapter describes the different aspects that are considered during the design of a prestressed concrete bridge. The initial sections deal with general design procedures while the particular requirements of the UK design standard BS 5400 [1] and the USA standard specifications by AASHTO [2] are given toward the end of this chapter. Codes and Standards are continually being developed and updated, and reference should always be made to the latest requirements and the bridge owners' particular specifications before commencing a design.

The design of individual details and the analysis of prestressed concrete decks are discussed in Chapters 6 and 8 respectively, while the design of particular deck types is covered in Chapters 9 to 17.

Where formulae are used in this chapter the units are in N and mm.

General approach

The structural form of the deck and the construction methods to be employed will influence the choice of prestress type and layout. Precast beams use pre-tensioned strands, wires or post-tensioned tendons, with typical arrangements as illustrated in Fig. 5.1. Pre-tensioned strands or wires are usually straight, but can be deflected to give a draped profile. Post-tensioned tendons are draped to reflect the bending moments generated in the beam.

Prestressed bars are considered easier to handle and install, and are often used where they simplify construction. Such applications include the temporary prestress during the erection of precast segmental decks, as shown in Fig. 5.2, or during the launching of box girder decks. Bars placed

@Seismicisolation

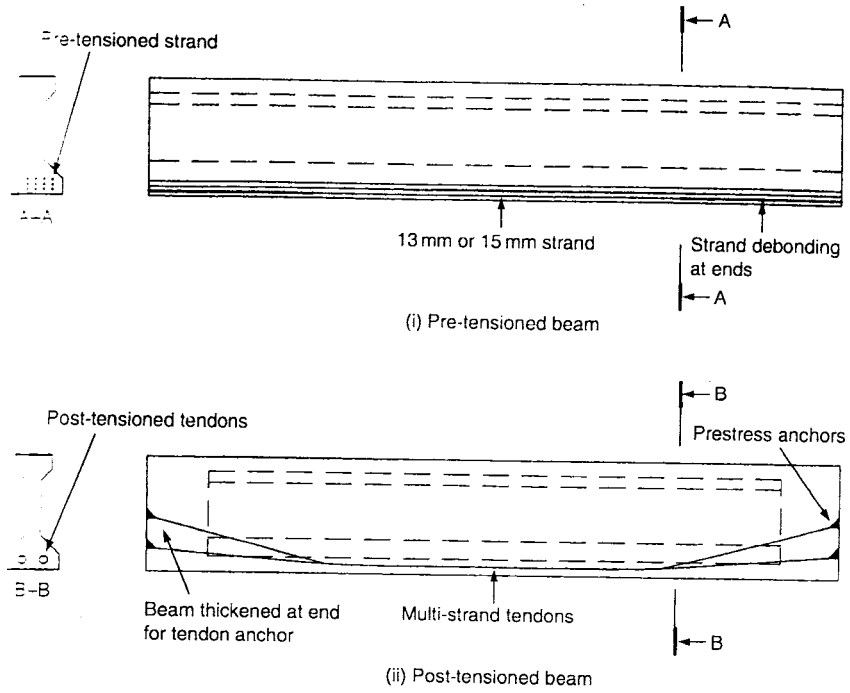


Figure 5.1. Precast beam prestressing.

ducts within the concrete are usually used straight between anchor points; however, large radius curved alignments can also be achieved.

Post-tensioned tendons are installed on site and have the advantage of being able to achieve complex tendon layouts and very high stressing

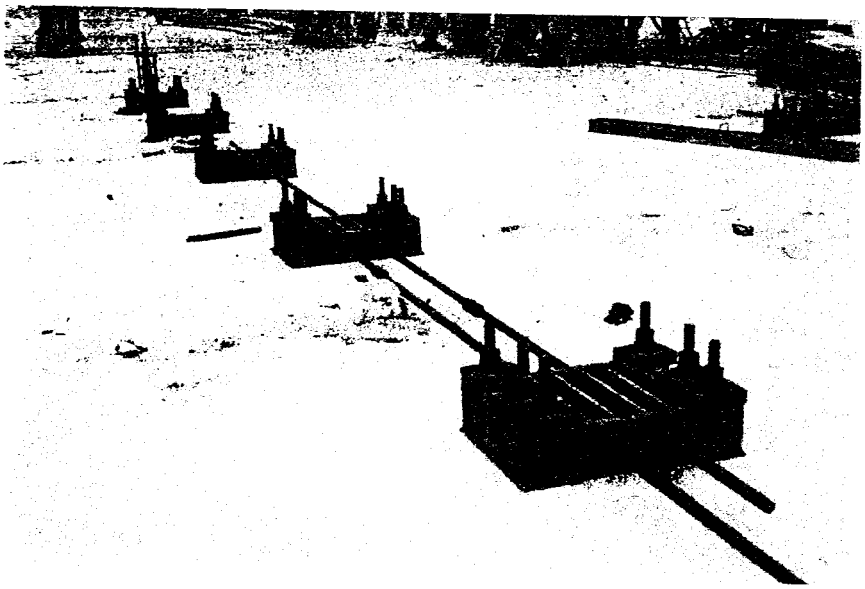


Figure 5.2. Prestressing bars used for segment erection.

@Seismicisolation

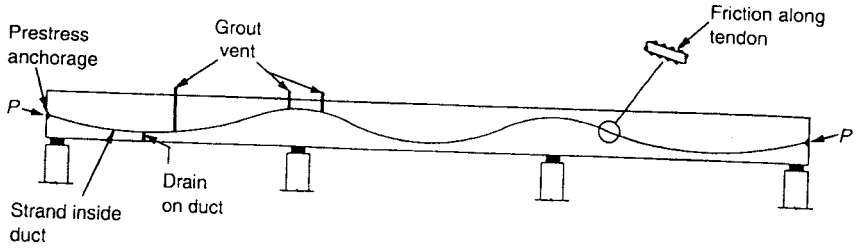


Figure 5.3. Typical post-tensioning layout.

forces. A typical layout for a continuous deck is illustrated in Fig. 5.3. Jacking systems used normally require a tendon length of more than 5 m to control losses during lock-off. The friction losses become excessive for internal post-tensioned tendons longer than 120 m, even when double-end stressed. External post-tensioned tendons have much lower friction losses and are installed with lengths of over 300 m. In stage-by-stage construction couplers are sometimes used to extend post-tensioned tendons across a construction joint.

External tendons, as seen in Fig. 5.4, are used extensively for bridge strengthening as well as for new construction, giving designs where the tendons are readily inspectable and replaceable. External tendons are anchored at the diaphragms or on special anchor blocks along the span. The tendons are deflected at deviators to give a draped profile.

Prestress tendon layouts, for most types of bridge structures, are now well established. New designs normally start by considering a tendon arrangement similar to that used on previous structures of the same type, for example: straight strand in the bottom of precast beams; cantilever tendons and

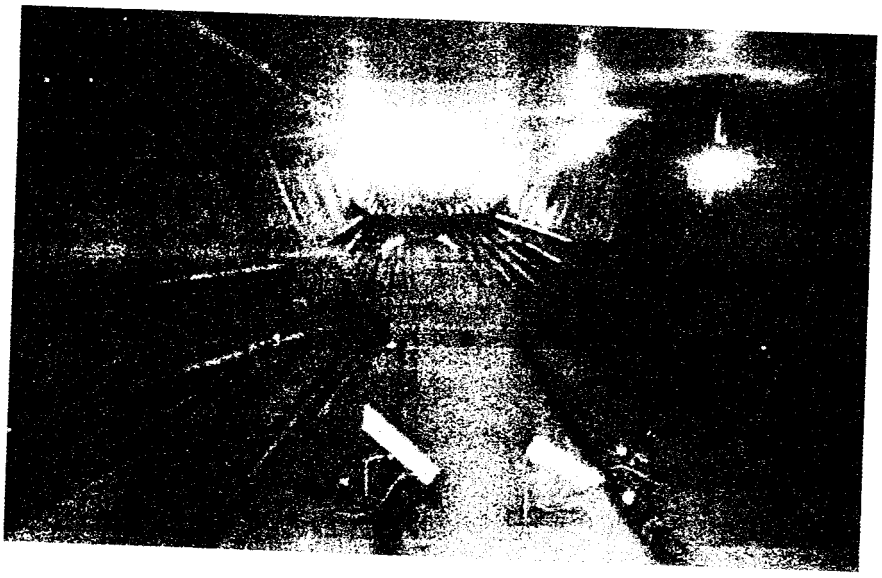


Figure 5.4. External tendons inside box girder.

@Seismicisolation

continuity tendons for balanced cantilever construction; or simple draped tendons for *in situ*, continuous decks.

For most types of deck, the prestress requirement is governed by the serviceability stress check. The number of tendons required at critical sections, such as mid-span and over supports, is derived by calculating the stresses on the section due to the applied loads and construction effects, and then estimating the prestress required to keep these stresses within the allowable limits based on an initial estimate of the secondary moment. From this initial estimate of the number of tendons needed a preliminary prestress layout can be drawn up. The difficulty at this stage is that the prestress secondary moments are not known, and therefore, after estimating the prestress layout required, it is necessary to calculate the actual secondary moments generated and to compare these with the values used in the initial estimate. It may take several iterations of estimating the secondary moment and adjusting the prestress arrangements before the required prestress stresses are obtained and the actual secondary moment matches the assumed secondary moment, to give the final design. After this the ultimate moment is checked at the critical sections and, if necessary, additional tendons added to ensure that adequate resistance can be mobilized. Where additional tendons are required a recheck of the serviceability stresses must be undertaken to ensure that they are still within the acceptable limits.

For structures with external, unbonded tendons the quantity of prestress required can be governed by the ultimate limit state, especially where the predicted increase in stress in the tendon at ultimate bending failure is small and there is no non-prestressed reinforcement available. This behaviour is described in the section 'Ultimate moment design', later in this chapter. In this case the number of tendons required at critical sections is determined from the ultimate moment check, and the serviceability check then carried out to ensure that the stresses are within acceptable limits.

The magnitude of the secondary moments in statically indeterminate structures is dependent on the prestress layout. They can also vary greatly with any type of structural form and are influenced by the construction techniques used. For bridge decks the secondary moment is usually sagging which reduces the design hogging moments over the piers and increases the design sagging moments in mid-span. For spans in the range of 35 m to 40 m, the prestress secondary moments for a typical box girder arrangement can be at least 5000 kN m at internal piers, while for heavily prestressed structures and for longer spans the secondary moments can be significantly higher.

Several countries now recommend that allowance should be made for the future installation of an additional 10 per cent or more of prestress tendons when designing new post-tensioned decks. This can easily be incorporated using external tendons. The need for this extra provision has come about due to excessive loss of prestress or large deflections occurring on some existing structures and because of the requirement to be able to upgrade or strengthen the structure in the future to take heavier loads. It would seem prudent to make provisions for future additional tendons on all new post-tensioned bridges.

@Seismicisolation

Primary and secondary prestress effects

When the prestressing tendons apply load to the structure the resultant forces and moments generated can be considered as a combination of primary and secondary, or parasitic effects, as illustrated in Fig. 5.5.

The primary effects are the moment, shear and axial forces generated at each section by the direct application of the force in the tendon at that section. The axial force, P, is the force in the tendons at the section being considered after all applicable losses have been taken into account. The primary moment is the tendon force multiplied by the distance of the force from the neutral axis, i.e. $P \times e$, and the shear is the vertical force due to the inclination of the tendon.

Secondary effects occur when the structure is statically indeterminate and restraints to the structure prevent the prestressed element from freely deflecting when the prestress is applied. For statically determinate and simply supported decks the structure is free to deflect when prestressed and hence no secondary effects are set up. With continuous decks the intermediate supports restrain the deck from vertical movement and secondary moments, and shears occur when the prestress is applied.

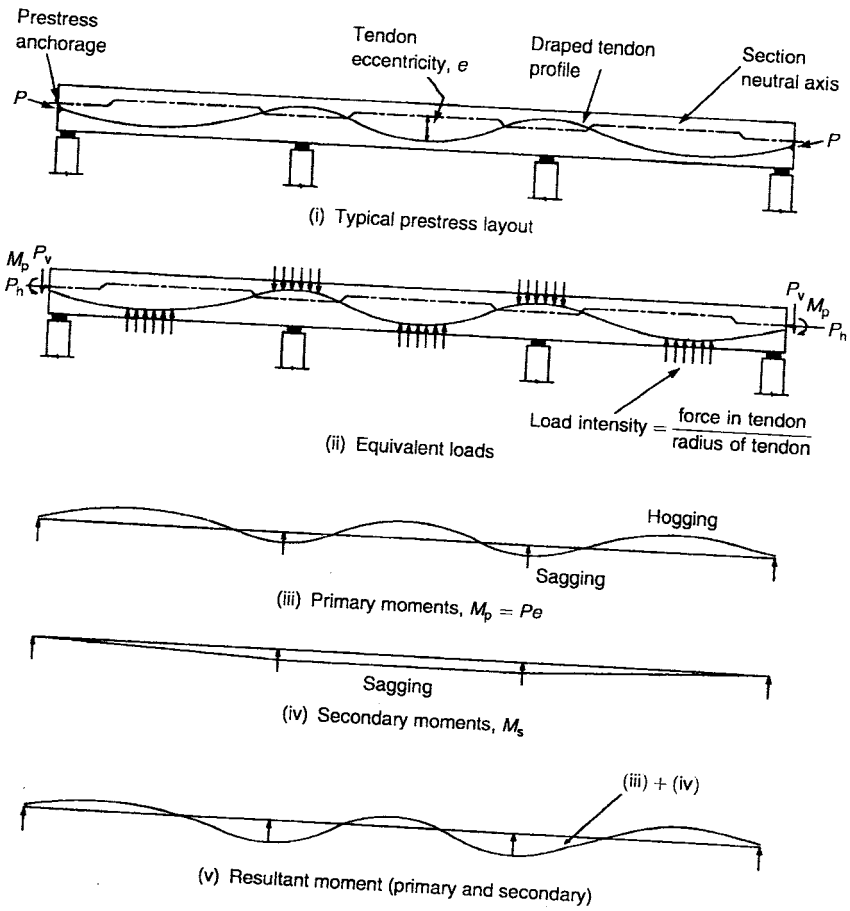


Figure 5.5. Prestress in continuous decks.

@Seismicisolation

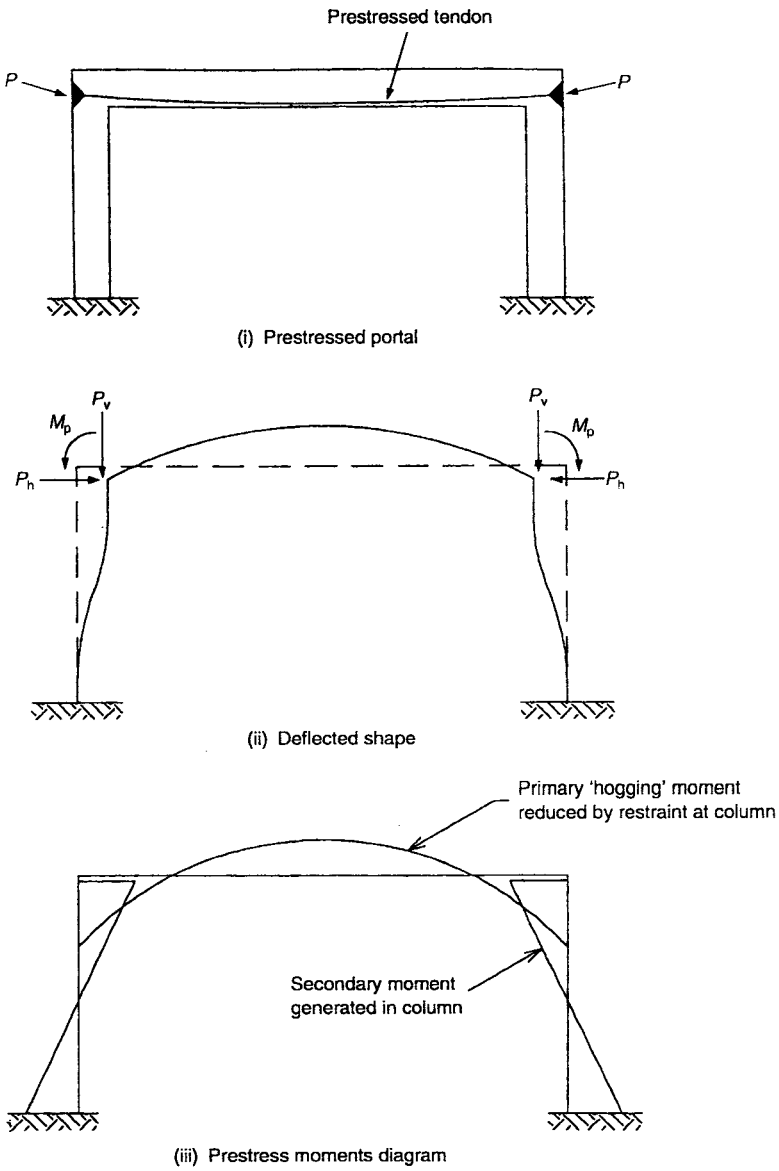


Figure 5.6. Secondary effects due to built-in supports.

Where decks are built integrally with abutments or piers the restraints generate secondary effects from the prestressing, which must be taken into account in the design. An example of this is shown in Fig. 5.6. The prestress compresses the portal beam causing it to shorten. This shortening is resisted by the stiffness of the columns, generating secondary moments and 'tensile' forces in the beam. Similarly, the prestress tends to bend the portal beam under its primary effects causing the beam to deflect, and the columns again resist this deflection, generating secondary effects.

The secondary moment set-up with post-tensioned tendons is dependent on the structural arrangement at the time of stressing. For bridges constructed in

@Seismicisolation

stages the structural arrangement changes as the deck is built out and each stage of prestressing will act on a different structural system. The final secondary moment in the deck is based on the combination of the secondary moments set up at each stage. This is complicated further by creep of the concrete and long-term losses in the prestressing. These cause a redistribution of the staged secondary effects with the changes acting on the modified structure.

Pre-tensioned strands installed in precast beams are stressed when the beam is 'simply supported' and no secondary moments are generated. However, if the beam is made 'continuous' or built into the supports after installation, creep of the concrete under the prestress load and redistribution of forces due to long-term stress losses in the prestressing steel will result in the development of secondary moments and shears.

Prestress secondary moments can be derived by a number of methods, as described in Chapter 8. One popular way is by applying the equivalent loads to a frame model. Other techniques include numerical methods such as using influence coefficient, or specialist analysis programs that combine the prestress with the structural analysis.

When the equivalent load method is used, a combination of both the primary moments and secondary moments are produced; however, this approach becomes considerably more complex when the variations in prestress force along the tendon are included. The Influence Coefficient method is suited to the use of spreadsheets which simplify the calculation and have the advantage that the prestress force profile is easily incorporated and the calculation can be quickly adjusted during the design process. Proprietary software programs are the easiest and quickest way to derive the combined primary and secondary effects on a structure, although many of those currently available do not take into account all the different aspects that must be considered during the design.

Primary and secondary moments both generate longitudinal bending stresses on the section, as indicated in Fig. 5.7. The resultant stresses are taken into the serviceability stress check. The secondary moments and shears are taken into the ultimate limit state checks.

Post-tensioned tendons in horizontally curved decks generate torsions due to the secondary effects of the prestress. The behaviour of curved box

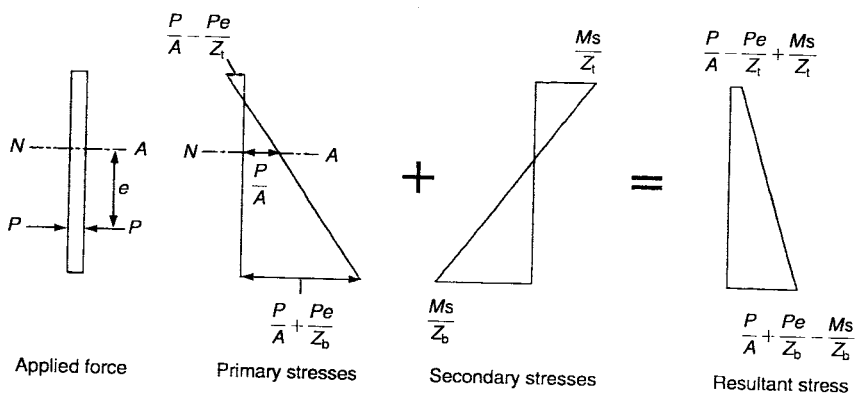


Figure 5.7. Prestress stresses on section.

@Seismicisolation

structures is described in the paper by Garrett and Cochrane (1970) [3]. Intermediate supports along a curved structure resist the decks' tendency to twist, and, where the prestress generates secondary moments along the deck and vertical reactions on the supports, a complementary torsion is present.

For curved box girder decks with a total horizontal angle change of less than 20° between the torsional restraints an estimate of the torsion is derived by computing the secondary moments, assuming that the bridge is straight, and dividing these by the horizontal radius of the deck. This 'M/R' diagram is then applied as a load to the structure, and the shears generated along the deck are the torsions that will be present in the curved structure. Where the angle change is over 20° the approximations made in this approach may not be valid. This method is presented in the paper by Witecki (1969) [4].

Provided the tendons are symmetrical about the vertical axis of the section, the primary effects of the tendons are not affected by deck curvature; however where this is not the case, such as with external tendons running straight between anchor and deviator points, transverse bending and further torsions can be generated.

In tightly curved box girders and continuous multiple-beam decks on a curved alignment the behaviour of the structure becomes complex as the beams twist and the deck slab distorts transversely. With these arrangements a three-dimensional finite element analysis should be carried out to determine the overall structural behaviour and the effects of the prestress.

Prestress force and losses

During stressing a tendon experiences losses in the force along its length due to friction with the duct. Further losses occur after lock-off, over the long-term, due to elastic shortening, creep and shrinkage within the adjacent concrete and by relaxation of the tendon steel. The long-term stress in the prestressing steel is given by:

$$f_{pc} = f_p - l_E - l_r - l_c - l_s$$

Tendons are often stressed up to 80 per cent UTS (ultimate tensile strength), although some codes limit the jacking force to 75 per cent UTS, and the maximum force in the tendon after release by the jack to a maximum of 70 per cent or 75 per cent UTS for post-tensioning and pre-tensioning respectively.

External tendons are not bonded to the concrete, and the elastic shortening, creep and shrinkage losses are estimated by considering the total concrete deformation between the tendon anchorages or other fixed points, and this is taken as being distributed uniformly along the tendon length.

Friction losses and tendon extension

Prestressing bars and pre-tensioned wires or strands are usually straight and encounter little friction losses. Multi-strand tendons that are placed inside ducting experience friction losses as the tendon is stressed. These losses can

@Seismicisolation

Table 5.1. Typical friction coefficients

	k	μ
Bare strand in metal ducts	0·001 to 0·002	0·2 to 0·3
Bare strand in UPVC ducts	0·001	0·14
Bare strand in HDPE ducts	0·001	0·15
Greased and plastic sheathed strand in polyethylene ducts	0·001	0·05 to 0·07

be significant when the tendons follow a curved profile, and cause the strands to press against the sides of the duct.

When the jack pulls a profiled multi-strand tendon, the movement of the strand is resisted by friction against the duct. The associated loss in the force transmitted along the tendon is expressed in terms of μ, the friction coefficient acting on the angle change in the tendon profile and k, the wobble coefficient acting over the length of the tendon. The wobble coefficient is a measure of the unintentional local misalignment of the duct, which results in the tendon rubbing against the duct surface, causing additional friction losses. The force F at any point along the tendon, x metres from the stressing anchor, is given by:

$$F = F_o e^{-(\mu\theta + kx)}$$

The friction coefficients are obtained from the manufacturers' literature for the system being used. Typical values are given in Table 5.1. The actual value achieved on any particular project depends on a number of factors, including the surface condition of the steel, the type, diameter, condition and support of the duct and the installation method adopted.

For an external tendon, bar or pre-tensioned strand, $k = 0$ over its 'free' length. Where external tendons pass through deviators or diaphragms, friction losses occur although wobble is often ignored over the short lengths concerned.

The tendon force F is sometimes expressed as $F = F_o e^{-\mu(\theta + kx)}$ with the value of k being adjusted accordingly.

When the jack releases a multi-strand tendon a short length of reverse friction is set up as the strand pulls in and the wedges grip to lock the strand in place. The lock-off pull-in can be up to 7 mm, depending on the system being used. With a prestressing bar anchorage is achieved by tightening a nut against the anchor plate after stressing the bar to the required force. Bar systems with fine threaded anchorages have minimal lock-off pull-in, while those with coarse threads can pull-in a small amount.

Figure 5.8 shows a typical force profile along an internal multi-strand tendon stressed from one end. The force loss along the tendon is dependent on the tendon profile, and for tendons longer than 40 m and with large angle changes excessive losses can occur. This can be compensated for by double end stressing, where the tendon is jacked from both ends. The maximum economic length for internal multi-strand tendons is usually about 120 m, above which the friction losses become too high reducing the effectiveness of the tendon. With external tendons the friction loss occurs only where

@Seismicisolation

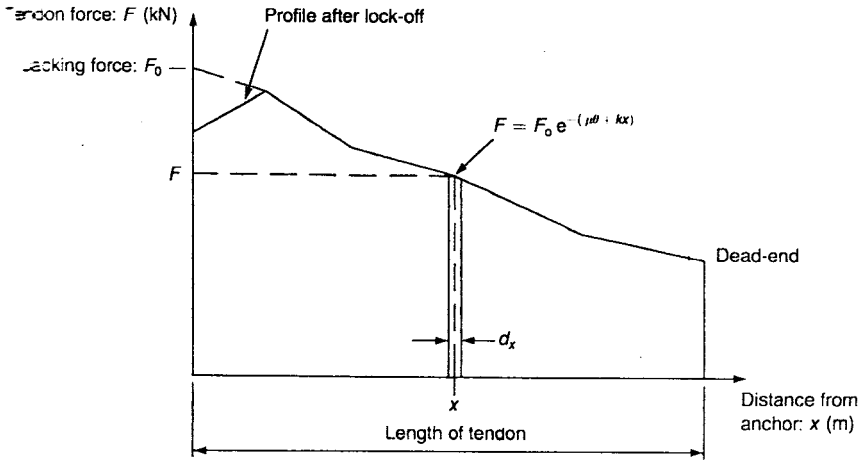


Figure 5.8. Tendon-force profile.

the tendon passes through a deviator and it is possible to use very long tendons without significant losses.

The extension of the tendon during stressing can be calculated from the force profile:

$$\text{Extension} = \sum F d_x / A_t E_t = \text{area under force profile} / A_t E_t$$

This gives the extension between the faces of the anchors which is compared with the values measured on site during the stressing to confirm the satisfactory tensioning of the tendon. The measured extension must be adjusted to take into account any pull-in of the wedges at the dead-end of the tendon, which typically is between 2–4 mm, and also the extension in the strand between the measuring point behind the stressing jack and the face of the anchor. For cast-in dead-end anchors, shown in Fig. 2.4, the predicted tendon extension is usually calculated up to a point midway between the end of the duct and the bulb at the end of the strands.

The E_t value used is determined from tests carried out on samples taken from the batch of strands actually used on the project being considered. With 7-wire strand, short lengths are tested by being held between fixed jaws in a testing rig. These tests give higher E_t values than those achieved with long lengths of strand where the effect of the 'spiral' outer wires reduces the effective stiffness. The effective E_t value is also reduced in multi-strand tendons where the strands are often twisted around each other during installation and straighten out during stressing. For long multi-strand tendons these effects reduce the E_t value by up to 5 per cent compared to the values obtained by testing a short length of strand.

Elastic shortening and strains

Elastic shortening of the concrete during the stressing of the tendons causes stress losses in any tendons installed previously. The loss in stress is calculated by the following formula:

$$f_E = \sigma_c (E_t / E_c)\ \text{N/mm}^2$$

@Seismicisolation

where σ_c is taken as the increase in stress in the concrete adjacent to the tendon, occurring after the tendon has been stressed.

In pre-tensioning where all the strands are anchored simultaneously the full prestress force will generate elastic shortening of the concrete and give a similar loss in each tendon at the time of force transfer. In post-tensioning each tendon at a section will suffer a different loss depending upon when it is stressed in relation to the other tendons. It is usually sufficient in the design to calculate an average loss and apply this to all the tendons with σ_c taken as half the total final stress in the concrete, averaged along the tendon length.

As the deck is loaded it deflects, causing changes to the strain in the concrete with any bonded tendons being subjected to a corresponding change in strain and stress. The tendons are usually near the 'tension' face of the concrete and the stress in the tendons increases under further loading; however, this effect is small and often neglected when considering the overall prestress design.

Relaxation of tendon steel

After the force is applied to the prestressing steel and the tendon anchored, relaxation occurs in the steel which results over time in a reduction of the force in the tendon. Typical values of relaxation after 1000 hours for tendons stressed to 70 per cent UTS are given in Table 2.1. The amount of relaxation of the stress in the prestressing steel depends on the initial stress present and steel characteristics as indicated in Fig. 5.9.

Magura *et al.* (1964) [5] proposed the following equation for calculating the tendon stress relaxation under constant strain;

$$f_{pt}/f_p = 1{\cdot}0 - (\log(t)/c)(f_p/f_y^* - 0{\cdot}55)$$

$c = 10$ for stress relieved strand with expected relaxation of 7 per cent after 1000 hours when stressed to 70 per cent UTS.

$c = 45$ for low relaxation strand with expected relaxation of 1·5 per cent after 1000 hours when stressed to 70 per cent UTS.

Up to 50 per cent of the 1000-hour relaxation percentage can occur within the first 24 hours. For low relaxation steel the total relaxation

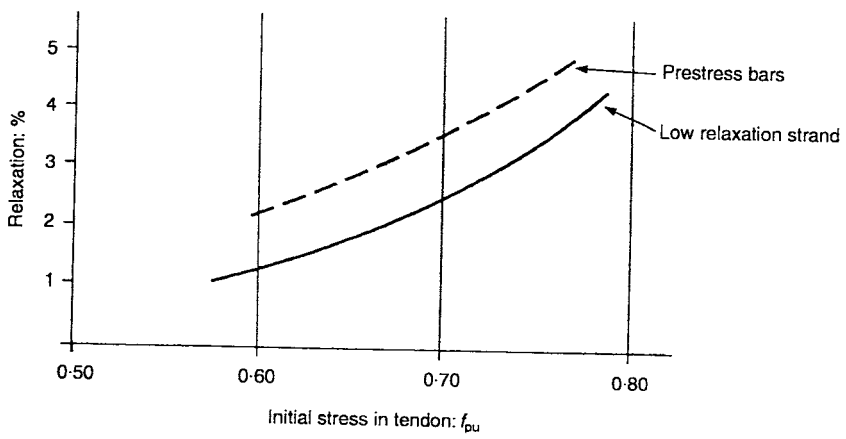


Figure 5.9. Relaxation losses in strands v initial stress.

@Seismicisolation

during a structure's life can be greater than three times the 1000-hours value. When the initial stress in the strand is less than $0{\cdot}55f_y^*$ the relaxation losses become small. Near the stressed end of a tendon the initial stress after lock-off can be greater than 70 per cent, while further along the tendon the stress may well be less than 50 per cent UTS if significant friction losses occur.

BS 5400 sets the loss of force to be taken in the design as the maximum relaxation after 1000 hours for a jacking load equal to that imposed at transfer. This may be conservative in some areas but may underestimate the long-term losses in other situations. The normal practice in the UK is to estimate the average relaxation that is likely to occur in all the tendons and apply this value throughout.

AASHTO [2] gives guidance on the relaxation losses to be taken in the design as follows:

Pre-tensioned members:

For stress relieved strand, $l_r = 137{\cdot}9 - 0{\cdot}4l_E - 0{\cdot}2(l_c + l_s)\ \mathrm{N/mm^2}$

For low-relaxation strand, $l_r = 34{\cdot}5 - 0{\cdot}10l_E - 0{\cdot}05(l_c + l_s)\ \mathrm{N/mm^2}$

Post-tensioned members:

For stress relieved strand, $l_r = 137{\cdot}9 - 0{\cdot}3l_f - 0{\cdot}4l_E - 0{\cdot}2(l_c + l_s)\ \mathrm{N/mm^2}$

For low-relaxation strand, $l_r = 34{\cdot}5 - 0{\cdot}07l_f - 0{\cdot}1l_E - 0{\cdot}05(l_c + l_s)\ \mathrm{N/mm^2}$

where l_f = friction-loss stress reduction in the tendon at the point being considered expressed as an amount below the 70 per cent UTS level.

For prestressing bars, $l_r = 20{\cdot}7\ \mathrm{N/mm^2}$

Again, this is conservative in some cases but may underestimate the long-term losses in other situations.

Creep losses

As concrete shortens due to creep under the compressive stresses, the change in concrete strain causes a corresponding change of strain and stress in the tendons. The loss of stress in the tendon due to concrete creep is given by:

$$l_c = \phi\sigma_c E_t/E_c\ \mathrm{N/mm^2}$$

The creep factor of the concrete is ϕ and is primarily dependent on the concrete mix, the environmental conditions, the member thickness and the maturity of the concrete at the time of loading. In normal bridge construction, ϕ is in the range 1·6 to 2·5.

BS 5400 part 4, Appendix C, provides suitable charts to derive ϕ taking into account the different parameters, while section 6 of the same document gives the following estimates for the creep per unit length in the concrete, expressed in terms of the stress in the concrete adjacent to the tendons:

@Seismicisolation

For pre-tensioned strand in humid or dry exposure:

with $f'_{ci} \geq 40\,\mathrm{N/mm^2}$ creep $= 48 \times 10^{-6}$ per $\mathrm{N/mm^2}$
with $f'_{ci} < 40\,\mathrm{N/mm^2}$ creep $= (40/f_{ci}) \times 48 \times 10^{-6}$ per $\mathrm{N/mm^2}$

For post-tensioned strand in humid or dry exposure and tensioned between 7 and 14 days:

with $f'_{ci} \geq 40\,\mathrm{N/mm^2}$ creep $= 36 \times 10^{-6}$ per $\mathrm{N/mm^2}$
with $f'_{ci} < 40\,\mathrm{N/mm^2}$ creep $= (40/f_{ci}) \times 36 \times 10^{-6}$ per $\mathrm{N/mm^2}$

where the stress in the concrete at transfer is $>0{\cdot}33f_{ci}$ the above estimated creep should be increased pro-rata up to 1·25 times when the stress is $0{\cdot}5f_{ci}$.

With AASHTO [2], for either pre-tensioned or post-tensioned members, the creep losses can be estimated as:

$$l_c = 12\sigma_{ci} - 7\sigma_{cd}\ \mathrm{N/mm^2}$$

The actual creep that occurs depends on many factors and, as the design and construction of prestressed concrete decks become more sophisticated, the creep effects need to be assessed as accurately as possible. The estimated losses given above should be used only for simplistic designs that are not sensitive to small changes in prestress forces.

Shrinkage losses

Shortening of the concrete due to shrinkage causes corresponding shortening and stress reduction in the tendons. Shrinkage losses are given by:

$$l_s = \Delta_{cs} E_t\ \mathrm{N/mm^2}$$

The shrinkage strain within the concrete that has occurred since the tendons were installed is Δ_{cs}, and is dependent on similar parameters to those for the creep factor above.

BS 5400 part 4, Appendix C gives guidance on deriving Δ_{cs}, while section 6 provides an estimate of the shrinkage strain, based on the relative humidity (RH) and age of concrete at transfer, as follows:

	Humid exposure (90% RH)	Normal exposure (70% RH)
Pre-tensioned strand with transfer between 3 and 5 days	100×10^{-6}	300×10^{-6}
Post-tensioned tendon with transfer between 7 and 14 days	70×10^{-6}	200×10^{-6}

With AASHTO, shrinkage losses are estimated as:

Pre-tensioned members, $l_s = 117{\cdot}21 - 1{\cdot}034\,\mathrm{RH}\ \mathrm{N/mm^2}$

Post-tensioned members, $l_s = 0{\cdot}80(117{\cdot}21 - 1{\cdot}034\,\mathrm{RH})\ \mathrm{N/mm^2}$

where RH is expressed as a percentage.

@Seismicisolation

As with creep losses, the above estimates should be used only for simplistic designs that are not sensitive to small changes in prestress forces. For most cases the shrinkage losses should be estimated based on the actual concrete mix and environmental conditions expected using the guidance given in BS 5400 part 4, Appendix C, or similar.

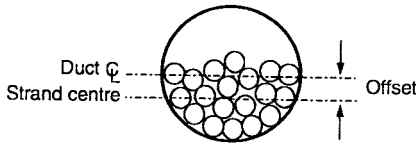
Tendon eccentricity in ducts

Strand tendons with a profiled layout will move to the inside face of any curved duct. This usually results in the tendon sitting at the bottom of the duct over the piers and at the top of the duct along the span. This offset between the duct centreline (C/L) and the tendons can be significant and depends on tendon and duct sizes. Typical offset values to be taken into account when calculating the tendon eccentricity in the design are given in Table 5.2.

Table 5.2. Tendon offset inside duct

No. of 13 mm strands	Internal diameter of duct (mm)	Offset from duct C/L to strand centre (mm)	No. of 15 mm strands	Internal diameter of duct (mm)	Offset from duct C/L to strand centre (mm)
1	20	4	1	25	5
7	55	8	7	60	10
12	65	10	12	80	15
19	80	12	19	100	17
27	95	14	27	110	20
31	100	15	31	130	22
37	120	25	37	140	25
42	130	25			
55	140	25			

Note: C/L = centreline



Serviceability limit state stress check

Longitudinal stresses in a prestressed concrete deck change at each stage of construction and throughout the structure's design life. The stress in the concrete must be kept within allowable limits for both compression and tension. It is usual to keep the concrete in compression across the full section under permanent loads and normal live loading with tensions of between 2 and 3 N/mm^2 permitted with unusual loading conditions or under the influence of temperature differentials. Typical allowable stresses in the concrete under service load are given later in this chapter when designing to BS 5400 or AASHTO [2] respectively.

@Seismicisolation

In precast segmental construction with match-cast joints the longitudinal reinforcement is not continuous, and it is normal to ensure that the joints remain in compression under all loading conditions both during construction and in the permanent structure. For precast construction, with elements connected using unreinforced concrete or mortar infill, the complete area of the connection is kept in compression with a minimum stress of 1·5 N/mm^2 to ensure that the joint is held together.

The stress levels along the length of a deck are checked at all the critical stages in the structure's life. This would normally include:

(a) at transfer of the prestress to the concrete;
(b) during each stage of construction, with temporary loads applied;
(c) at bridge opening, with full, live load;
(d) after long-term losses in the prestress and full creep redistribution of moments have occurred.

The unfactored nominal force in the tendons, after all associated losses are considered, is used when carrying out the serviceability limit state checks.

Where external tendons are used the design of the deck should be checked for the situation when any two of the tendons are removed should it be necessary to replace them in the future. In this case it is normal to consider a reduced live loading to reflect traffic management systems that may be implemented.

For a beam or box subjected to bending the cross-section is assumed to remain planer with a linear variation in stress over the section depth and a constant stress at any one level. When flanged beams are subjected to high shear, such as adjacent to supports, the bending in the section can result in a non-uniform distribution of longitudinal stresses across the flanges, as illustrated in Fig. 5.10. This effect, where the stresses in the flanges are higher over the webs than elsewhere, is known as shear lag. With prestressed concrete decks, the moments due to prestress and dead load tend to balance each other out and hence reduce the magnitude of any shear lag that occurs. For this reason shear lag is generally not significant in prestressed concrete bridge decks of standard dimensions.

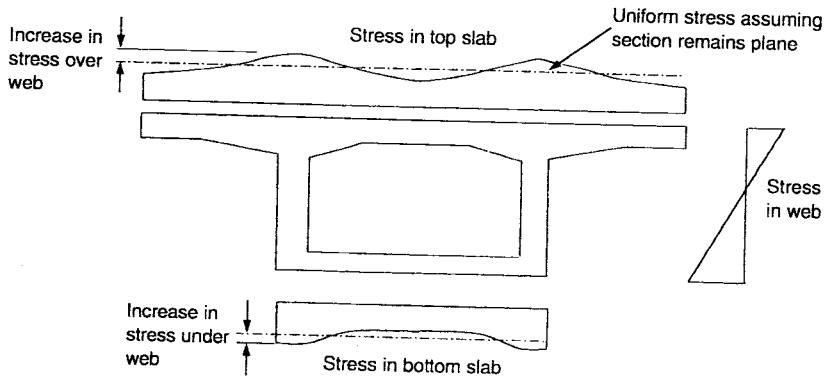


Figure 5.10. Stress distribution due to shear lag.

@Seismicisolation

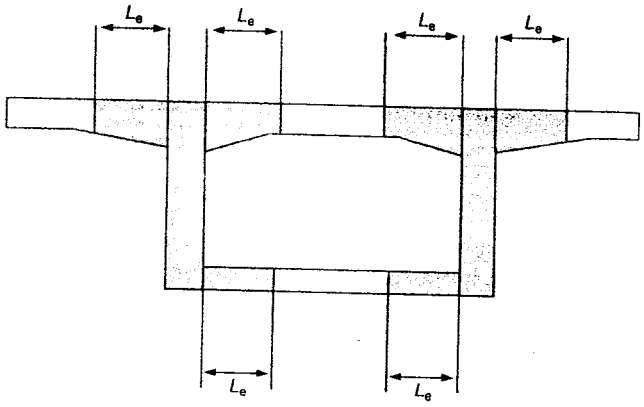


Figure 5.11. Effective flange width.

When shear lag becomes significant the change in stress distribution around the section is quantified using a three-dimensional finite element analysis, or as described in the Cement and Concrete Association (C&CA – no longer in existence) Technical Report for box girders by Maisel and Roll (1974) [6].

When analysing the structure to derive forces and deflections the effects of shear lag are usually neglected although, with unusual arrangements such as very wide decks with widely spaced webs or long-edge cantilevers, shear lag may become more significant and should be considered in the analysis.

Both BS 5400 and AASHTO [2] include the concept of using reduced flange widths in the design of a section to simulate the effects of shear lag. By applying the moments and forces to a section with a reduced flange width the derived stresses are increased, similar to shear lag behaviour. The 'effective flange width' L_e, as illustrated in Fig. 5.11, is given in BS 5400 as the lesser of 0·1 × distance between points of zero moment or the actual flange width either side of the webs. With continuous beams this results in a much smaller effective flange width near the support compared to mid-span, where the effects of shear lag are much less.

In AASHTO [2], the effective flange width is given as:

(i) For precast beam and *in situ* slab, width taken as the lesser of:

Total flange width <0·25 × span length of beam
Flange overhang <6 × flange thickness or <0·5 clear distance between webs

(ii) For precast beam and *in situ* slab, with flange on one side only, width taken as the lesser of:

Total flange width <0·083 × span length of beam
Flange overhang <6 × flange thickness or <0·5 clear distance between webs

(iii) For box girders,

With normal arrangements full flange widths are taken
With unusual proportioned boxes shear lag to be determined by analysis.

One of the problems with using effective flange widths is the difficulty in accurately accounting for the prestress effects. With post-tensioned construction, in the region beyond the anchor zone, the axial force will act on the

@Seismicisolation

complete deck section; while the primary and secondary moments will act to counter the dead load and may be applied to the reduced area.

Warping of thin-walled box girder decks under torsional and distortional behaviour causes redistribution of the longitudinal bending stresses at a cross-section. This behaviour is described in the C&CA Technical Report by Maisel and Roll (1974) [6], which presents several methods to quantify the effect. Under asymmetrical and torsional loading the box section distorts and the cross-section undergoes out-of-plane displacements which cause additional longitudinal stresses. When a full-length three-dimensional finite element model is used this behaviour is automatically included in the analysis.

Deflections and pre-camber

Deflections of the concrete deck occur due to self-weight and prestress, and from the weight of the permanently applied loads. Additional movements occur due to long-term creep of the concrete and losses in prestress. The deflections due to the prestress are often greater than, and in the opposite direction to, the dead-load deflections. This results in the deck deflecting upwards upon application of the prestress and removal of the formwork. Subsequent application of the superimposed dead load and long-term losses in the prestress force reduce this upwards deflection or can result in a net sag deflection.

The deck should be cast and erected so that the required profile is achieved upon completion. Concrete creep will cause the deck to change its profile over its design life and it is usual to aim to achieve the desired alignment at the end of construction or at the time of bridge opening, although the long-term changes in deflections should be checked to ensure that they are not excessive.

The adjustment made to the profile during casting to achieve the desired shape is called the pre-camber, and it is governed by the overall design, the construction sequence and the concrete properties.

The balance between the dead loads and prestress usually results in small net deflections. Deflections are generally not significant for normally proportioned prestressed concrete decks designed with stresses within the allowable limits and do not need to be checked other than to confirm the pre-camber values to be catered for.

From a visual aspect it is desirable for a span to appear to be 'hogging' rather than 'sagging'. To ensure this on straight decks a small upwards profile is often built into the deck along each span by adjusting the pre-camber. The upward profile imposed is usually only a few centimetres at mid-span.

Vibrations and fatigue in tendons

Vibrations from traffic or wind seldom create a problem for most prestressed concrete decks and it is not normally necessary to consider this in detail. The fluctuation in the direct stress in the tendons due to live and other loading is also very small and fatigue is generally not critical. However, where external tendons are used, in certain circumstances vibrations of the individual tendons can occur due to traffic or deck movements.

@Seismicisolation

The details and arrangements of external tendons should be developed so that they are not subjected to vibrations that are likely to give rise to fretting of the strand or bending stresses that could cause fatigue problems. This is achieved by ensuring that the frequency of the free length of tendons, between anchors or deviator points, is not close to the natural frequency of the deck or of the traffic using the bridge. Approximate values for frequencies can be estimated as follows:

$$\text{Frequency of the external tendon} = (\pi/L_T)\sqrt{(F/m_t)}\ \text{Hz}$$

$$\text{Frequency of the deck} = (k_f^2/(2\pi L_{sp}^2))\sqrt{(EI/m_d)}\ \text{Hz}$$

k_f depends on span continuity and is π for simply supported spans and between 3 and 4·5 for continuous deck.

Vibration frequency of highway traffic is often taken as being between 1 and 3 Hz. For rail traffic the frequency depends on train speeds and axle spacing along with the bogey design.

To prevent vibrations it is usually sufficient to limit the free length of the tendon to 12 m or less, which should result in the tendon frequency being different from that of the deck and traffic.

Ultimate moment design

As the bending moment on a prestressed beam increases, the compression on one side goes up while on the other side the concrete goes into tension. When the tensile strength of the concrete is exceeded, cracking will occur and the load is transferred either to the prestressing tendons or to any non-prestressed reinforcement present. As the moment increases further, the cracks in the concrete open and propagate across the section with a pure couple set up between the compression in the concrete, and tension in the tendon and the non-prestressed reinforcement. The maximum moment is reached when either the concrete or tendon and reinforcement fail.

When considering the ultimate moment of resistance the full width of the flanges are taken into account to resist the compressive forces generated. Shear lag effects are neglected.

The moment of resistance, or capacity, at a section is derived by comparing the balance of the tensile force in the tendons and reinforcement with the compressive force in the concrete. The strain distribution is assumed linear across the section, with the point of zero strain being the effective neutral axis.

On the compressive side, away from the neutral axis the stress in the concrete builds up rapidly until it reaches a peak of the compressive strength in bending, taken as $0{\cdot}67f_{cu}$, before reducing slightly with increasing strain. A maximum strain in the concrete of 0·0035 is taken at failure, with this occurring at the extreme compression fibre.

On the tensile side, the stress in the tendons and passive reinforcement is a function of its distance from the neutral axis, while the concrete is assumed to have no tensile strength. Where several different layers of tendons and reinforcement are present the strain, and hence the stress, in each is different.

@Seismicisolation

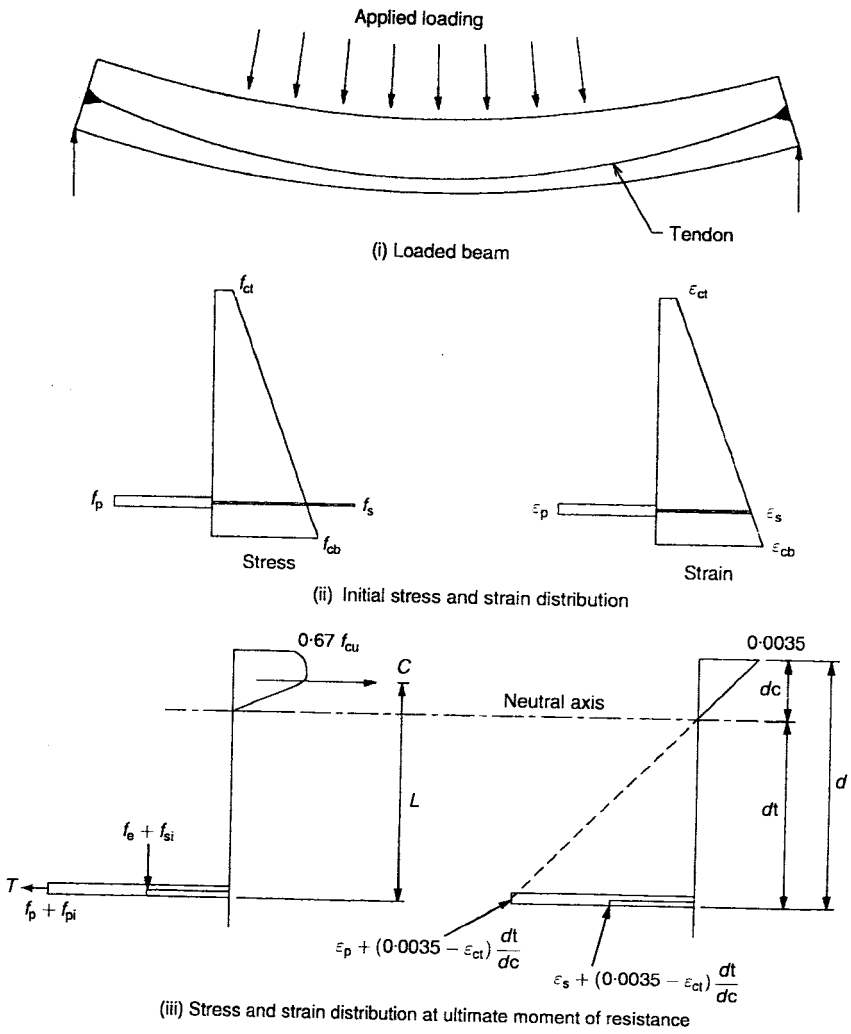


Figure 5.12. Ultimate moment of resistance.

For the design of the completed structure the stress in the tendon, f_p, is considered after all long-term losses have occurred to give a 'worst case' situation.

The stress–strain relationships of concrete, prestress tendons and reinforcement are all well established. Typical curves for prestress tendons and reinforcement can be obtained from the material suppliers, while the behaviour of concrete is described in various publications, such as the book by Neville (1995) [7]. Idealization stress–strain curves for design purposes are given in BS 5400 part 4, Figs 1, 2 and 3.

The theoretical stress and strain distributions across a section with internal, bonded tendons when subjected to bending are illustrated in Fig. 5.12. The section is assumed to remain planer under bending. The initial stress and strain in the tendons, reinforcement and concrete is that generated by the prestress and permanent loading. As the applied loading increases, the stresses and strains change until they reach the ultimate design state indicated in Fig. 5.12(iii).

@Seismicisolation

To determine the ultimate moment of resistance or strength capacity of the section an iterative approach is used, with a first estimate of d_c taken, from which C and T are calculated. d is then adjusted and C and T recalculated until $C = T$, giving the ultimate moment of resistance, $M_r = T \times L$.

The moment of resistance of any section along the deck must exceed the bending moment generated by the applied loads to give a sufficient factor of safety against failure. This is achieved by applying appropriate factors to reduce the calculated capacity of the section and to increase the effects of the applied loads.

With external tendons, unbonded from the concrete over long lengths, the change in the steel stress under ultimate loads is directly related to the increase in the tendon length between its fixed points as the deck deflects. The increase in strain in the unbonded tendon may be small depending on the position of the fixed points and the tendon layout. This limits the increase in stress in the tendon and ultimate failure is usually governed by the compression in the concrete which can give rise to a brittle failure mechanism. Estimate of the ultimate moment of resistance follows the same procedure for internal tendons as discussed above, although for long tendons installed over several spans $f_{pi} = 0\,\mathrm{N/mm^2}$ is taken when the average strain increase is small. For short tendons anchored within one span f_{pi} depends on the tendon layout and deck arrangement. For very short tendons, crossing the section being considered, the increase in stress can be up to $450\,\mathrm{N/mm^2}$ which is similar to the increase that can be expected for internal, bonded tendons.

When a deck with external tendons deflects, the tendons will remain straight between their restraints and, at a section away from the anchors or deviators, the tendons move towards the neutral axis. This has the effect of reducing the strain in the tendon while also reducing the lever arm, L, decreasing the moment of resistance. This can become significant if the free length of external tendons is greater than a quarter of the span length and the tendons are unrestrained near the critical sections.

As with the serviceability stress check, a deck containing external tendons should have adequate capacity to carry a reduced live loading with any two of the tendons removed.

It is usually sufficient to check only the critical points along the deck, such as over each of the intermediate piers and at each mid-span.

A more detailed description on checking the ultimate moment and flexural strength to BS 5400 [1] and AASHTO [2] is given later in this chapter.

Shear design

The vertical loads on the deck impart a shear onto the section that must be catered for in the design. Prestressing tendons assists in resisting the shear in two ways; first by enhancing the shear capacity of the concrete and second by providing a vertical component of force that counteracts the shear when the tendon is inclined in the appropriate direction. The prestress force considered in the design of the completed structure is that which remains after all long-term losses have occurred to give a worst-case scenario.

@Seismicisolation

There are two types of shear failure associated with prestressed concrete beams: with the section either uncracked or cracked in flexure. Where the section is uncracked in flexure shear failure is caused by web cracks forming where the principal stresses exceed the tensile capacity of the concrete. Where sections are cracked in flexure it is possible for the shear to combine with the flexural crack to result in failure. The capacity of the section under both types of failure is considered with the lowest value taken for the design.

Shear is only considered at the ultimate limit state: when the shear resistance of the section must exceed the shear forces applied to give an adequate factor of safety. The entire shear on a section is assumed to act on the webs, with only the web resistance taken into account in the design. The maximum allowable shear stress on a section is limited to keep the principal stress in the concrete to acceptable values, and this limit often governs the minimum width required for the webs.

Shear capacity at any section is the sum of the ultimate shear resistance of the concrete V_c, and the ultimate shear resistance of the reinforcement present, V_s. This can include both the non-prestressed reinforcement and any bonded prestressing tendon not being utilized at the ultimate limit state for other purposes.

Shear in the deck is checked adjacent to each pier and at regular intervals along each span with reinforcement provided to give adequate resistance. Shear reinforcement is normally provided in the form of links or stirrups.

In long span box girders, where the deck is haunched, the longitudinal bending in the deck gives rise to compression in the bottom slab which acts parallel to the soffit, as illustrated in Fig. 5.13. This compression has a vertical component that acts against the shear forces and can be deducted from the applied shear force, V.

Allowance is made for the reduction in effective web width where internal post-tensioned tendons are placed within ducts in the webs. Before grouting the tendons the full duct width is deducted from b, while after grouting two-thirds of the duct diameter is deducted.

When checking the ultimate shear capacity at the ends of pre-tensioned beams allowance is made for the loss of prestress over its anchorage length l_t, as defined in Chapter 6. It is normally sufficient to consider the prestress

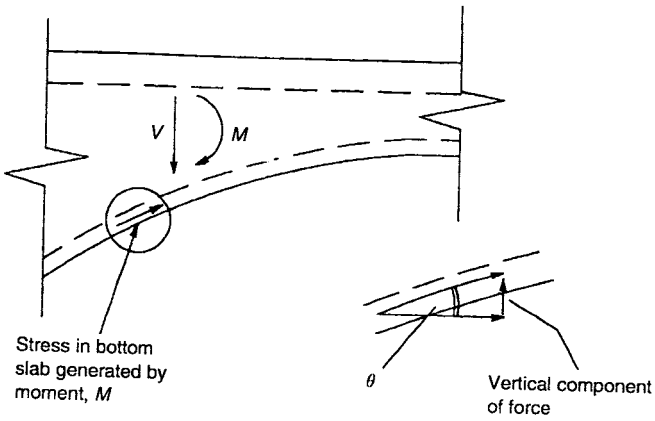


Figure 5.13. Contribution of haunched slab to shear resistance.

@Seismicisolation

force as varying linearly over this length when deriving V_c. If the shear resistance based on the beam being simply reinforced, and ignoring the prestress, is greater than that calculated, assuming a prestressed beam, then the higher value can be taken.

With external tendons the shear capacity at any section should be adequate to carry a reduced live loading with any two tendons removed, to allow for future tendon replacement if necessary.

Later in this chapter the design procedures are described for shear when using BS 5400 [1] and AASHTO [2].

Torsion design

If the deck is subjected to non-symmetrical loading or is on a horizontal curve the twisting that can occur sets up a torsion within the prestressed concrete members. Torsion is normally considered at the ultimate limit state and does not generally govern the dimensions of the concrete section, but may require additional reinforcement to be provided.

In the webs the torsional stresses in the transverse plane can be added to the vertical shear stresses. The resultant stresses and associated force is designed for the ultimate limit state following the procedure for shear but with additional longitudinal reinforcement provided, to cater for the longitudinal component of the torsion. In the flanges the torsional stresses are calculated and both transverse and longitudinal reinforcement provided as appropriate.

For rectangular sections, the torsional shear stress, v_t, is given by:

$$v_t = 2T_u/(b_{min}^2(b_{max} - b_{min}/3))$$

For box sections the stress is different where the width of webs and slabs varies, and at any particular point around the box is given by:

$$v_t = T_u/(2b_{wo}A_o)$$

With precast segmental decks, where no reinforcement passes through the joints, the concrete should be in compression at all times and the longitudinal tensile stresses from torsion need to be overcome by a residual longitudinal compression from the prestress equal to $T_s/(2b_{wo}A_o)$.

The procedure for checking the torsional stresses and calculating the reinforcement requirements to BS 5400 [1] is given in the section 'Ultimate Torsion', later in this chapter. The AASHTO [2] standard specification does not directly cover this aspect; however, guidance is given in the AASHTO specifications for segmental bridges (1999) [8] and details of this are given in the section 'Torsion', in this chapter.

Longitudinal shear

As the bending moments change, the flow of stress through the section gives rise to longitudinal shear forces within the concrete which are checked at the slab–web interfaces, as indicated in Fig. 5.14.

The longitudinal shear acting on section A is equal to the difference in the forces acting on the two ends of area A_L over the length considered. The

@Seismicisolation

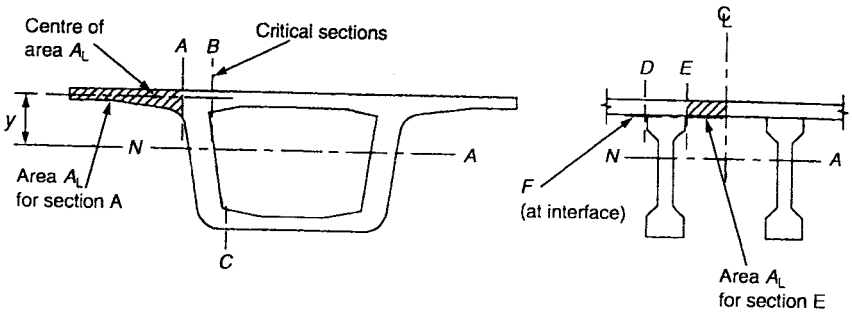


Figure 5.14. Longitudinal shear.

difference in force is equal to the area multiplied by the change in longitudinal stress acting on the section, which is itself a function of the change in bending moment. This can be expressed in terms of the vertical shear force, with the ultimate longitudinal shear force per unit length given by:

$$V_L = V(A_L y/I)$$

where A_L and y refer to the area of concrete outside the section being considered.

Where the concrete is monolithic, or is able to transmit shear across the critical section, it generates some resistance to the longitudinal shear with reinforcement provided to give the remaining resistance.

The requirements for reinforcement to cater for the longitudinal shear when designing to BS 5400 or AASHTO [2] are given in the relevant sections later in this chapter.

Partial prestressing

The design of fully prestressed structures is based on the concrete being uncracked under service loading: either with the stresses in compression under all loading conditions or with small tensions allowed under transient loading which are kept below the tensile strength of the concrete. For partially prestressed structures the principle is to allow the concrete to crack under service loading and to limit the crack widths to the normal allowable values for reinforced concrete. This is often related to a hypothetical allowable tension in the concrete of up to 6 or 7 N/mm^2 when carrying out the longitudinal stress check. Several countries, including Denmark and Australia, have successfully adopted this approach into their bridge design standards.

The ULS or capacity checks of the moment and torsion are carried out in the normal manner; while for shear the concrete resistance is calculated as cracked in flexure.

A fatigue check of the prestress is carried out to ensure that the service stress fluctuation in the strand is not critical for the level of stress present. A check to ensure that the stress range is below 120 N/mm^2 is normally sufficient.

The advantages of partial prestressing include reduction in the quantity of prestress, full utilization of non-prestressed reinforcement, smaller deflections

@Seismicisolation

due to prestress and reduced creep in the concrete. The disadvantages include the need for more non-prestressed reinforcement and a possibility that the durability of the structure will be reduced in harsh environments; although this has not currently been noted where partially prestressed structures have been used.

Construction sequence and creep analysis

The way a bridge is built affects the moments and shears generated in the structure and this is taken into account during the design process. The structure is checked for strength, stability and the serviceability requirements at each stage of construction. The final moments and shears derived should reflect the construction sequence followed. Figure 5.15 illustrates the dead load bending moments in a four-span deck constructed in stages as a series of balanced cantilevers. At the end of the deck construction the bending moment diagram follows the cantilever moments with large hogging at the piers and little moment at mid-span. If the deck is built 'instantaneously' the balance between the hogging at the piers and the sagging at mid-span is more even. After completion of the construction the creep of the concrete adjusts the moments with the final moments being between the as-built moments and the moment if the deck was built instantaneously.

When the statical system of a concrete structure changes during construction creep of the concrete will modify the as-built bending moments and shear forces towards the 'instantaneous' moment and shear distribution; the amount of the change being dependant on the creep factor, ϕ, of the concrete which is defined as:

$$\phi = \text{creep strain/elastic strain}$$

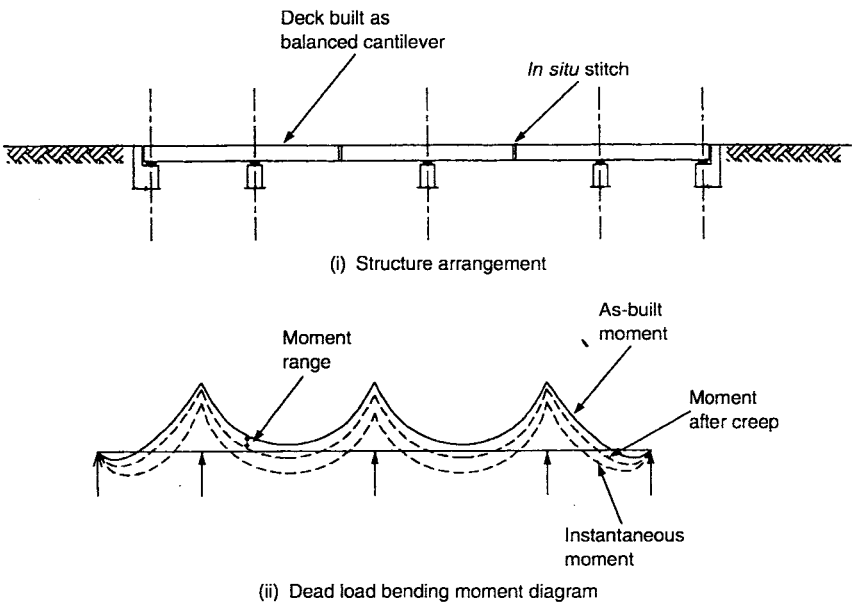


Figure 5.15. Creep redistribution of moments.

@Seismicisolation

The value of ϕ depends on the concrete constituents, mix proportions, section details, environmental conditions and the age of concrete, and typically range from 1·3 to 3 or more. For precast construction ϕ is normally around 1·6, while for *in situ* construction ϕ would normally be between 2·0 and 2·5. BS 5400 part 4, Appendix C.2, gives the derivation of ϕ in more detail.

Where the change to the statical system is sudden, such as when connecting balanced cantilevers with a mid-span stitch or jacking up the deck, the modification to the moments is:

$$M_{\text{final}} = M_{\text{as-built}} + (1 - e^{-\phi})(M_{\text{inst}} - M_{\text{as-built}})$$

Where the change is gradual, such as in the differential shrinkage between precast beams and *in situ* top slab or differential settlement between adjacent piers, the modification to the moments becomes:

$$M_{\text{final}} = M_{\text{a}}(1 - e^{-\phi})/\phi$$

Shears are modified in a similar way.

The effect of creep can be significant as indicated by the following comparison between the ϕ factors:

ϕ	$(1 - e^{-\phi})$ sudden change	$(1 - e^{-\phi})/\phi$ gradual change
1·6	0·80	0·50
1·8	0·83	0·46
2·0	0·86	0·43
2·2	0·89	0·40
2·4	0·91	0·38

Considering *in situ* concrete with a ϕ equal to 2·2 and using balanced cantilever construction with a 'sudden change' when the cantilevers are stitched together, the final moments will have crept from the as-built condition 89 per cent of the way towards the instantaneous condition. Even with a gradual change, such as in long-term settlement of the piers, the final moments and shear built up in the deck are only 40 per cent of the moments and shears that would have been generated if the change had occurred instantaneously.

Both the dead load and prestress secondary effects are affected by the stage-by-stage build-up of the deck and by the creep that occurs during the construction period and afterwards.

Temperature effects

Changes in the effective temperature of the deck will cause it to expand or contract, while differential temperature gradients through the concrete cause stress variations that must be considered in the design.

@Seismicisolation

Where a deck is free to expand or contract the overall change in effective temperature will not give rise to any forces in the structure; although the movement must be catered for in the bearing and expansion joint design. When a restraint exists that restricts free movement, such as integral decks or multiple fixed piers, forces will be generated throughout the structure. These forces can be determined by modelling the change in effective temperature as a strain differential in a frame analysis program.

Under a uniform temperature change, the change in deck length, $\Delta_L = \Delta_t \alpha L$.

The co-efficient of thermal expansions, α, is taken as $12 \times 10^{-6}/°C$ with most aggregates or $9 \times 10^{-6}/°C$ with limestone aggregates.

Differential temperature effects are generated through the concrete section as the outer surface heats or cools more quickly than the rest. This was extensively researched by the TRL as reported by Emerson (1977) [9], and from this profiles of the temperature gradient through a deck in different situations were developed. BS 5400 part 2, Fig. 9 [10] defines a number of simplistic temperature gradients that can be used with different structural types.

When a rapid increase in temperature occurs, for example during sunrise, the outer skin of the concrete heats up more quickly than the concrete core. As the outer skin tries to expand it is restricted by the core which results in compressive stresses building up at the edges of the section and tensile stresses in the centre. Where there is a rapid decrease in temperature, such as at sunset, the outer concrete cools more quickly than the inner core, resulting in tensile stresses at the edges of the concrete and compressive stresses in the centre.

A typical simplified temperature gradient through a section is indicated in Fig. 5.16(i). The profile of the gradient through a section is similar with both a positive or negative temperature change. The different temperatures through the concrete section cause different strains and therefore different stresses to be generated.

The stresses can be converted into equivalent forces and give rise to an out-of-balance axial force and moment as indicated in Fig. 5.16(iv) (a) and (b).

Provided the ends of the deck are not restrained, the force and moment will relieve at the free ends of the structure, as indicated in Fig. 5.16(v), and the stresses adjusted accordingly along the deck to give the resultant stress distribution across the section. If the ends of the deck are restrained, for example with integral structures, the moments are prevented from fully relieving.

The stress in the section is the equivalent stress from the temperature change, less the stress from the relieved equivalent axial force, plus the stress from the relieving of the out of balance moment. These stresses are included in the design when considering the serviceability stress check of the prestressed concrete.

The relieving of the out of balance moment, illustrated in Fig. 5.16(v), generates shear forces in the spans and these are considered during the ultimate shear design process.

@Seismicisolation

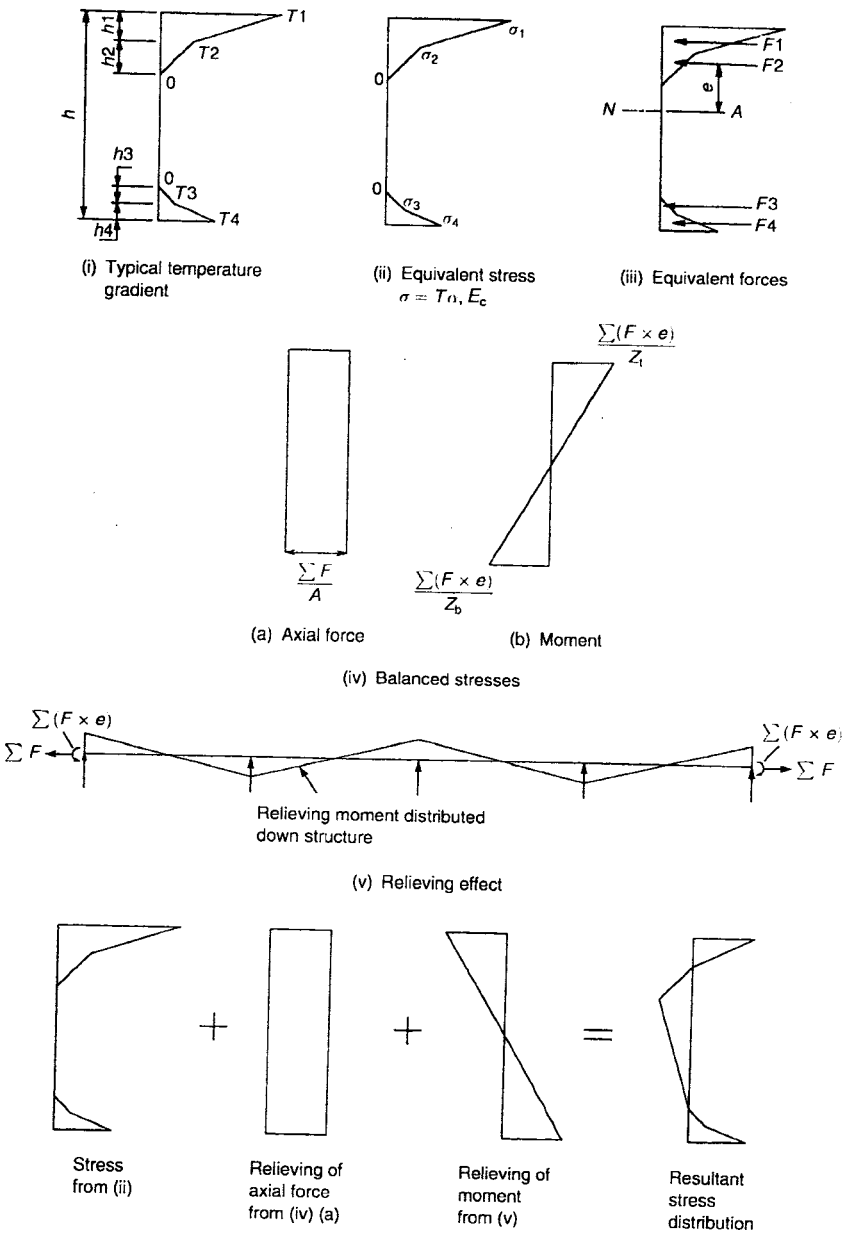


Figure 5.16. Stresses generated from temperature gradient.

Traditionally only the differential temperature across the vertical plane, i.e. between the top and bottom of the section, is taken into consideration in the design. Depending on the orientation of the deck and the position of the sun, differential temperatures can also occur between the two sides of a deck, giving rise to differential horizontal effects. This has not been investigated in detail and no guidance is given in the design codes. Further research is required to quantify the true significance of this effect.

@Seismicisolation

Concrete properties

The strength of concrete is specified in terms of its cube strength, f_{cu}, or its cylinder strength, f'_c. Cube strengths are determined from crushing 150 × 150 × 150 mm samples while with cylinders a 150 mm diameter by 300 mm long sample is crushed.

The characteristic strength of the concrete is related to its crushing strength 28 days after casting; however for prestressed concrete the strengths earlier than this and in the longer term are also important. Many factors affect the development of strength within the concrete as described by Neville (1995) [7] but for design purposes the strength at any time is often taken as, $f_{ci} = \exp\{0{\cdot}25[1 - \sqrt{(28/t)}]\} f_{cu}$ where t is the concrete age in days.

Some codes, such as the British Standards, base their design rules on the cube strength of the concrete, while others, such as the American AASHTO [2] standards and European standards, use the cylinder strength of concrete. The relationship between cube and cylinder strength varies depending on the concrete strength and its properties but, as an approximate guide, the comparison given in Fig. 5.17 can be taken.

The required strength of the concrete is determined by the compressive stresses generated in the concrete by the prestress and applied forces. The strength must be sufficient to ensure that the allowable stress limits exceed the stresses present. A strength, f_{cu} of 45 N/mm^2, or f'_c of 40 N/mm^2, is typical for prestressed concrete; however, it is becoming more common to use higher strengths. Concrete with f_{cu} up to 65 N/mm^2, or f'_c of 60 N/mm^2 is frequently produced, while even higher values have been achieved on some projects.

The rate of gain of strength of the concrete is important as this governs the time at which the prestress can be applied. At transfer of the prestress force to the concrete it is normally required that the concrete strength f_{ci} be a minimum of 30 N/mm^2 (f'_{ci} of 25 N/mm^2). This requirement can vary depending on the tendon and anchor arrangement and the magnitude of force applied.

High early concrete strengths are required for precast segmental construction to be able to achieve a daily casting cycle for the segment production cell. Typical concrete strengths of 12 N/mm^2 are achieved after 10 hours to allow

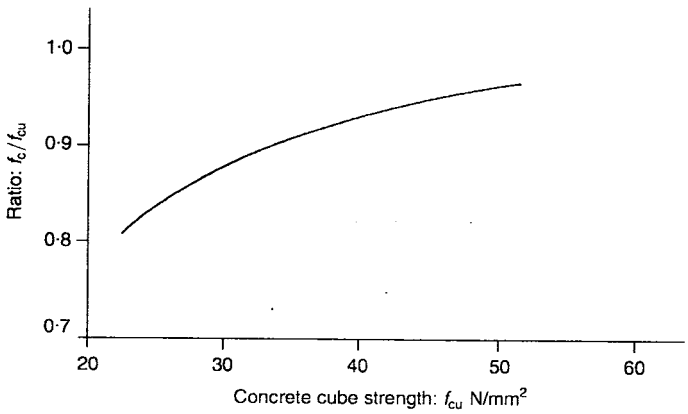


Figure 5.17. Cube v cylinder strengths.

@Seismicisolation

stripping of the shutters with 25 N/mm^2 achieved after 36 hours to allow the segments to be lifted and transported to the storage yards.

The modulus of elasticity of the concrete is related to the concrete strength as well as being affected by the material properties of the aggregates used. The modulus also reduces as the stress on the concrete increases. For normal concrete an approximate design value for the modulus can be taken as $4800\sqrt{f_{ci}}$. Typical values range from 28 000 to 36 000 N/mm^2 as the concrete strength, f_{ci}, increases from 30 to 60 N/mm^2, although these can vary by up to ±20 per cent.

To minimize creep and shrinkage losses in the prestressing the cement content and water/cement ratio of the concrete should be kept to a minimum, compatible with the high concrete strengths required. Admixtures are often used to improve the workability of the concrete and allow the water/cement ratio to be reduced.

Application of the prestress

With pre-tensioning the full prestress is applied to the concrete at an early age, while with post-tensioning the application of the prestress is often in stages and may occur over a period of some months.

Where possible, when using post-tensioning, it is advantageous to apply part of the prestress soon after concreting. The minimum concrete age when the prestress can be applied is restricted by the strength of concrete required to enable the anchorages to be loaded. However, stressing some of the tendons as soon as possible can help to limit the tendency for early thermal or shrinkage-induced cracking to occur. Early prestressing can be achieved by partially stressing some of the tendons to, say, 50 per cent of their design load. By only applying part of the load to the anchor it is possible to apply the prestress with the concrete at a lower strength than that needed to apply the full load.

When a deck is prestressed it normally 'lifts up' along the span, reducing any load on the falsework supporting the concrete. In stage-by-stage construction, if the deck is cantilevering over a pier when it is prestressed the end of the cantilever is often deflected downwards and increases the load on the falsework beneath. To avoid overstressing the falsework the prestress can be applied in stages. The first stage prestress should be just adequate to support the concrete dead load without causing any significant deflections. The falsework can then be removed after which the rest of the prestress can be installed.

Design procedures to BS 5400 [1]

The following section highlights specific design requirements when using BS 5400, supplementing the sections above. BS 5400 is based on a limit state approach with the structure checked against criteria at both the serviceability limit state (SLS) and the ultimate limit state (ULS).

At SLS the factored nominal forces and unfactored behaviour of the structure are considered, while at ULS load factors and material factors are used to ensure that an adequate overall factor of safety is achieved.

@Seismicisolation

The load factors, γ_{f1} applied for both the SLS and ULS under different load combinations, are given in BS 5400 part 2 (1990) [10] and modified in the UK by the Highways Agency's standard BD 37/01 (2001) [11]. An additional analysis factor, γ_{f3}, is applied to increase the loading effects and is 1·0 for the SLS and 1·1 for the ULS when elastic methods are used in the analysis of the structure.

Design to BS 5400 [1] is based on the 28-day concrete cube strength, f_{cu}.

Serviceability limit state stress check

In the UK, prestressed concrete bridge decks are usually designed to Class 1 under permanent loads and normal live loading and to Class 2 for all other load combinations.

The allowable stresses in the concrete under service load are:

Compressive stresses: $0{\cdot}5f_{ci} \leq 0{\cdot}4f_{cu}$ with triangular stress distribution or
$0{\cdot}4f_{ci} \leq 0{\cdot}3f_{cu}$ with uniform stress distribution

Tensile stresses: 0 N/mm^2 with permanent loads and normal live load
$0{\cdot}45\sqrt{f_{cu}}$ with full loading and other effects on pre-tensioned members
$0{\cdot}36\sqrt{f_{cu}}$ with full loading and other effects on post-tensioned members

Allowable tensile stresses during construction are:

1 N/mm^2 during erection with prestress and co-existent dead and temporary loads
$0{\cdot}45\sqrt{f_{cu}}$ for pre-tensioned members under any loading
$0{\cdot}36\sqrt{f_{cu}}$ for post-tensioned members under any loading

The allowable stress in the prestressing steel after anchoring the tendon is $0{\cdot}7f_{pu}$ for post-tensioned tendons and $0{\cdot}75f_{pu}$ for pre-tensioned tendons; however, the jacking force may be increased to give stresses up to $0{\cdot}8f_{pu}$ during stressing.

Ultimate moment resistance

The stress in the concrete, prestress tendons and reinforcement is reduced by a partial factor for strength, or material factor, γ_m, which is 1·5 for concrete and 1·15 for prestress tendons and reinforcement. Referring to Fig. 5.12(iii), the stress in the concrete becomes a maximum of $0{\cdot}45f_{cu}$ and the stresses in the tendons and reinforcement are divided by 1·15.

When calculating the ultimate moment of resistance in the case of rectangular sections, or flanged sections with the neutral axis in the flange, the compressive stress in the concrete is taken as an average of $0{\cdot}4f_{cu}$ over the full depth of compression, with the concrete strain limited to 0·0035 at the outermost fibre. The increase in stress in the tendon, f_{pi}, and non-prestressed reinforcement, f_{si}, is derived by considering the change in strain caused by the bending within the beam and is obtained by reference to the tendon stress/strain curves given in Figs 2 and 3 in BS 5400 part 4 [1], with $\gamma_m = 1{\cdot}15$.

@Seismicisolation

To determine the ultimate moment of resistance, the compressive force in the concrete, C, and the tensile force in the tendons and reinforcement, T, are calculated as follows:

$$C = 0{\cdot}4 f_{cu} d_c b_s$$

$$T = A_t(f_p + f_{pi})/1{\cdot}15 + A_p(f_s + f_{si})/1{\cdot}15$$

As described in an earlier section, the position of the neutral axis is determined to give the situation where the forces C and T balance with the ultimate moment of resistance being the moment generated between them.

Where the neutral axis lies within the web of a flanged section the compressive force is calculated by considering the components of force generated in both the flange and the web above the neutral axis. The compressive stresses are based on the stress–strain curves with the material factor applied.

If the ultimate moment of resistance is less than 1·15 times the applied ultimate moment then the strain in the tendon, at the centroid of the outermost 25 per cent of the total tendon area, should be at least $0{\cdot}005 + f_{pu}/(E_s \gamma_m)$. This is to ensure that the section is 'under-reinforced' with the tendons yielding significantly before failure of the concrete in compression.

Ultimate shear resistance

With internal tendons, the ultimate shear resistance of the concrete V_c is the lesser of the ultimate shear resistance of a section uncracked in flexure, V_{co}, and the ultimate shear resistance of a section cracked in flexure, V_{cr}.

$$V_{co} = 0{\cdot}67 bb\sqrt{(f_t^2 + f_{cp} f_t)} + V_p$$

$$V_{cr} = 0{\cdot}037 bd\sqrt{f_{cu}} + (M_{cr}/M)V$$

where

$$M_{cr} = (0{\cdot}37\sqrt{f_{cu}} + f_{pt})I/y$$

The values for V_p, f_{cp} and f_{pt} are based on the prestressing forces after all losses have occurred and have been multiplied by the partial safety factor, $\gamma_{fL} = 0{\cdot}87$. Where the vertical component of prestress, V_p, resists shear at the section being considered, it is ignored when calculating V_{cr}.

The maximum shear force V is limited to $0{\cdot}75\sqrt{f_{cu}}bd$, but no greater than $5{\cdot}8\,\mathrm{N/mm^2}$, to prevent excessive principal stresses occurring in the concrete. With the use of concrete of strength significantly higher than $60\,\mathrm{N/mm^2}$ the limit of $5{\cdot}8\,\mathrm{N/mm^2}$ appears to be conservative, although further research and testing is needed to justify increasing this limit.

BS 5400 part 4 [1] requires that minimum shear reinforcement is provided such that:

$$(A_{sv}/S_v)(0{\cdot}87 f_{yv}/b) = 0{\cdot}4\,\mathrm{N/mm^2}$$

When V exceeds V_c shear reinforcement in the form of vertical links is required such that

$$(A_{sv}/S_v) = (V + 0{\cdot}4bd_t - V_c)/(0{\cdot}87 f_{yv} d_t)$$

@Seismicisolation

Longitudinal reinforcement is provided in the tensile zone with:

$$A_{sl} \geq V/2(0{\cdot}87f_{yl})$$

This quantity can include both the non-prestressed reinforcement and any bonded prestressing tendon not being utilized at the ultimate limit state for other purposes.

The shear reinforcement requirements are based on a truss analogy with the longitudinal reinforcement or prestress needed to form the tensile chord in the truss. Clark [12] published a paper in 1984 arguing that the above formula for the longitudinal reinforcement was in error and showing that it should be:

$$A_{sl} \geq \cot\theta V/2(0{\cdot}87f_{yl})$$

where θ is the angle of the assumed included strut in the concrete. As the point being considered gets closer to the support, θ increases and the required A_{sl} reduces. With prestressed concrete decks, where the prestress tendons are usually anchored on the ends of simply supported decks or continue fully over the intermediate supports for continuous decks, there is normally no need for additional longitudinal reinforcement for the shear action.

In the UK, the Highways Agency's standard BD 58/94 [13] requires that sections with unbonded external tendons be designed as reinforced concrete columns subjected to an externally applied load. The shear stress on the section is given by:

$$v = (V - 0{\cdot}87V_p)/(bd) \text{ where } v \text{ should not exceed the lesser of } 0{\cdot}75\sqrt{f_{cu}} \text{ or } 5{\cdot}8\,\text{N/mm}^2$$

The reinforcement required A_{sv} is given by:

when $v \leq \xi_s v_c$ $\quad A_{sv} \geq 0{\cdot}4bs_v/0{\cdot}87f_{yv}$

when $v > \xi_s v_c$ $\quad A_{sv} \geq bs_v(v + 0{\cdot}4 - (1 + 0{\cdot}05(0{\cdot}87P)/A_c)\xi_s v_c)/0{\cdot}87f_{yv}$

v_c and ξ_s are given in BS 5400 part 4 [1], Tables 8 and 9 respectively.

The resulting reinforcement requirements using this method appear overly conservative when compared to other international design codes and further testing and research is needed to justify the application of these formulae with external tendons.

Ultimate torsion

For torsion it is necessary to calculate the torsional shear stresses generated by the ultimate loads and where these stresses exceed $v_{t\,min}$ as given in Table 10 of BS 5400 part 4 [1], reinforcement is provided by means of transverse links and longitudinal bars. The stresses generated by torsion and shear are added together and checked to ensure that the sum does not exceed $0{\cdot}75\sqrt{f_{cu}}$ and is less than $5{\cdot}8\,\text{N/mm}^2$.

For rectangular sections, torsional reinforcement is provided such that:

$$A_{st}/S_v \geq T_u/(1{\cdot}6x_i y_i(0{\cdot}87f_{yv}))$$

$$A_{sl}/S_L \geq (A_{st}/S_v)(f_{yv}/f_{yl})$$

@Seismicisolation

Where T and I sections are used the torsion is considered as acting on the individual rectangular elements, with the section divided up to maximize the sum of $(h_{max}h_{min}^3)$ of each rectangle. Each rectangle is then designed to carry a proportion of the torsion based on its value of $(h_{max}h_{min}^3)$ in relation to the sum of the values for all the rectangles. The reinforcement is determined as for normal rectangular sections and detailed to tie the individual rectangles together.

For box sections, reinforcement is provided such that:

$$A_{st}/S_v \geq T_u/(2A_o(0{\cdot}87f_{yv}))$$

$$A_{sl}/S_L \geq (A_{st}/S_v)(f_{yv}/f_{yl})$$

Where a part of the section is in compression the compressive force may be used to reduce the A_{sl} required.

This reduction is given by: $A_{sl}/S_L = (f_c b_{wo})/(0{\cdot}87f_{yl})$

Longitudinal shear

To cater for the longitudinal shear stresses at the critical sections, BS 5400 requires that sufficient reinforcement cross the plane to ensure that V_L is not greater than the lesser of either:

(a) $k_1 f_{cu} L_S$

or

(b) $v_L L_S + 0{\cdot}7A_r f_y$

Values for v_L and k_1 are given in BS 5400 part 4 [1], Table 31. For precast beam-and-slab construction a minimum area of reinforcement of 0·15 per cent of the contact area is required across the interface between the beam and slab. BS 5400 part 4 allows reinforcement that is provided for other purposes to be utilized to resist the longitudinal shear, as long as it is fully anchored beyond the section being considered.

Partial prestressing

Classified in BS 5400 part 4 [1] as Class 3, partial prestressing of concrete is not usually adopted on bridgeworks in the UK.

When using BS 5400 to design a partially prestressed deck, the hypothetical SLS stresses are given in part 4, Table 25. The ultimate moment and torsion design is carried out in the normal manner and for ultimate shear the concrete resistance is calculated as cracked in flexure, with:

$$V_{cr} = (1 - 0{\cdot}55(f_{pc}/f_{pu}))v_c bd + M_o V/M$$

where $M_o = f_{pt}I/y$.

Any vertical component of prestress is ignored.

f_{pc} and f_{pt} are based on the prestressing force after all losses have occurred, and then multiplied by the partial safety factor $\gamma_{fl} = 0{\cdot}87$.

f_{pc} should be $\leq 0{\cdot}6f_{pu}$.

v_c is given in BS 5400 part 4 [1], Table 8, in which the value of A_s is the total area of prestressed and non-prestressed reinforcement in the tensile zone.

@Seismicisolation

Design procedures to AASHTO standard specifications [2]

The following section highlights specific requirements for the design when using AASHTO [2], and it supplements the earlier sections. The AASHTO [2] standard specification provides the basic requirements but is supplemented by other documents such as the *Guide Specification for the Design and Construction of Segmental Concrete Bridges* [8].

AASHTO [2] is based on a load factor design (LFD) and allowable stress design (ASD). In ASD the nominal forces and unfactored behaviour of the structure are considered, while with LFD load factors and strength capacity reduction factors, ϕ, are used to ensure that an adequate overall factor of safety is maintained. The load factors for the different load combinations are given in section 3 of AASHTO [2]; while the strength reduction factors are given in the individual design sections.

A new bridge design specification, the Load and Resistance Factor Design (LRFD) [14] was issued in 1998, and it will be fully adopted in the year 2007 when the AASHTO [2] standard specifications will be superseded.

In the analysis and design the concrete section is assumed to behave with a linear elastic strain distribution over its depth and with stresses linearly proportional to the strains. After the concrete has cracked the tension in the concrete is ignored.

Design to the AASHTO [2] specifications is based on the 28-day cylinder strength of concrete, f'_c.

Allowable stress design check

Allowable stresses in the concrete under service load are:

Temporary stresses before losses due to creep and shrinkage;

Compression:

Pre-tensioned concrete	$\leq 0{\cdot}60 f'_{ci}$
Post-tensioned concrete	$\leq 0{\cdot}55 f'_{ci}$

Tension:

Pre-compressed tensile zone	none specified
Tension areas with no bonded reinforcement	$1{\cdot}379\ \mathrm{N/mm^2}$ or $\leq 0{\cdot}249\sqrt{f'_c}$

where tensile stress exceeds this, bonded reinforcement to be provided, with the maximum tensile stress $\leq 0{\cdot}623\sqrt{f'_c}$.

Stress at service load after losses have occurred:

Compression:

All load combinations	$\leq 0{\cdot}60 f'_c$
except:	
Prestress + Dead load	$\leq 0{\cdot}40 f'_c$
Live load + 50% (Prestress + Dead load)	$\leq 0{\cdot}40 f_c$

Tension:

Pre-compressed tensile zone:	
members with bonded reinforcement (includes bonded prestressing strands)	$0{\cdot}498\sqrt{f'_c}$

@Seismicisolation

severe exposure conditions	$0{\cdot}249\sqrt{f'_c}$
members without bonded reinforcement	0

The allowable stress in the prestressing steel is as follows:

Pre-tensioned members, prior to transfer;

Low-relaxation strands	$0{\cdot}75f'_s$
Stress-relieved strand	$0{\cdot}70f'_s$

Post-tensioned members

At anchorage after lock-off	$0{\cdot}70f'_s$
At end of lock-off loss zone	$0{\cdot}83f^*_y$
For short periods prior to lock-off	$0{\cdot}90f^*_y$

At service load after losses $0{\cdot}80f^*_y$

Flexural strength

AASHTO [2] requires that the factored moment at a section, M, be greater than the nominal moment strength, M_r, reduced by the appropriate strength capacity reduction factor, i.e.

$$M \leq \phi M_r$$

where $\phi = 1{\cdot}0$ for precast concrete and 0·95 for *in situ* concrete.

The guide specification [8] gives further values of ϕ for the different types of segmental construction as follows:

Fully bonded tendons without dry joints between segments	$\phi = 0{\cdot}95$
Unbonded tendons without dry joints between segments	$\phi = 0{\cdot}90$
Unbonded tendons with dry joints between segments	$\phi = 0{\cdot}85$

By varying the different ϕ values, AASHTO [2] recognizes the comparative performances between internal, fully bonded and external tendons and with the use of dry joints on precast segmental construction.

The nominal moment strength of a section can be calculated following the procedure outlined earlier in the chapter. Alternatively AASHTO [2] gives an estimate of the moment strength for rectangular sections or flanged sections with the neutral axis in the flanges:

$$M_r = A_t(f_p + f_{pi})d(1 - 0{\cdot}6(A_t/bd)(f_p + f_{pi})/f'_c)$$

$(f_p + f_{pi})$ is the total stress in the prestressing steel under ultimate loading.

This is similar to the approach previously described, with the first part of the formula giving the force in the prestressing steel and the equation in the bracket giving an approximation in terms of d for the lever arm between the centre of compression in the concrete and the centre of tension in the steel. An iterative approach is used to derive f_{pi} accurately corresponding to a maximum value of M_r.

AASHTO [2] contains further estimates for deriving the flexural strength with flanged sections where the neutral axis is outside the flange and for the case where non-prestressed reinforcement is present and taken into account.

@Seismicisolation

Shear capacity

The factored shear force at a section V should be less than $\phi(V_c + V_s)$, with V_c and V_s the nominal shear strengths of the concrete and of the reinforcement respectively.

V_c is the lesser of the capacities of the concrete cracked or uncracked in flexure, with the shear strengths given as:

Uncracked in flexure $V_{cw} = bd(0{\cdot}29\sqrt{f_c'} + 0{\cdot}3f_{pc}) + V_p$
Cracked in flexure $V_{ci} = 0{\cdot}05bd\sqrt{f_c'} + V_d + (M_{cr}/M)V$

where

$d \geq 0{\cdot}8h$ and $V_{ci} \geq 0{\cdot}22bd\sqrt{f_c'}$

$M_{cr} = (I/y_t)(0{\cdot}5\sqrt{f_c'} + f_{pe} - f_d)$

M and V are the factored moment and co-existent shear from the load combination causing maximum moment at the section.

$V_s = A_{sv}f_{yv}d/S_v$

with $V_s \leq 0{\cdot}644\sqrt{f_c'}bd$ and $d \geq 0{\cdot}8h$.

The strength reduction factor, ϕ, is taken as 0·9.

The guide specification [8] gives further values for the different types of segmental construction as follows:

Fully bonded tendons without dry joints between segments	$\phi = 0{\cdot}90$
Unbonded tendons without dry joints between segments	$\phi = 0{\cdot}85$
Unbonded tendons with dry joints between segments	$\phi = 0{\cdot}85$

AASHTO treats shear design in the same way for both internal and external tendons when calculating V_c and the shear reinforcement to be provided. With external tendons the vertical component of the prestress is considered as an applied load and reduces the applied shear where the deviators are in the outer one-third of the span.

When checking for the ultimate shear failure of the concrete uncracked in shear, V_{cw}, the vertical component of external prestress, V_p, is considered unfactored, i.e. $V_u - V_p < \phi(V_{cw} + V_s)$, where V_u is the ultimate applied shear and V_s is the ultimate shear resistance of the reinforcement present. When checking for the ultimate shear failure of the concrete cracked, V_{ci}, the vertical component of prestress is ignored.

Torsion

The AASHTO [2] standard specifications do not directly cover the design for torsion other than to recommend that for segmental box girders the increase in web shear is taken into account in the shear design. Guidance is given in the guide specification [8] as follows: where T_u is $<0{\cdot}33\phi T_c$ then torsional effects may be ignored. This relates to a very low level of stress set up by the torsion, with the stress comfortably carried by the concrete. The ϕ value is that relating to shear as noted above.

@Seismicisolation

$$T_c = 0{\cdot}166K\sqrt{f'_c}(2A_o b_c)$$

with $K = \sqrt{(1 + (f_{pc}/0{\cdot}166\sqrt{f'_c}))}$ but $<2{\cdot}0$. $K < 1{\cdot}0$ when coexistent longitudinal stress is greater than $0{\cdot}498\sqrt{f'_c}$ in tension.

When T_u is above this value the torsional stress in the webs is added to the vertical shear stress and the concrete checked and reinforcement provided in accordance with the shear requirements.

Transverse reinforcement in the top and bottom slabs, A_{st}, is provided such that:

$$A_{st}/S_t = T_u/(2A_o f_{yv})$$

Longitudinal reinforcement, A_{sl}, per metre around the perimeter is provided such that:

$$A_{sl} = T_u/(2A_o f_{yl})$$

For solid beam elements, A_o may be taken as 85 per cent of the area enclosed by the transverse torsion reinforcement and b_c may be taken as the concrete area divided by the perimeter of the section.

Horizontal shear

Equivalent to 'longitudinal' shear discussed above, the horizontal shear is checked at LFD with V_L not exceeding ϕV_{nh}. The ϕ factor is that for shear as noted in the section 'Shear capacity'.

The concept of shear friction is used to predict the capacity of the critical sections as follows:

Without additional reinforcement provided, V_{nh} is $0{\cdot}55b_v d$ when the contact surface is clean, free of laitance and intentionally roughened.

With a minimum reinforcement provided of $0{\cdot}34b_v s/f_y$, V_{nh} is $0{\cdot}55b_v d$ when the contact surface is clean, free of laitance but not intentionally roughened and $2{\cdot}41b_v d$ when the contact surface is intentionally roughened.

Additional reinforcement is then provided to achieve the required value of V_{nh}, which is increased by $0{\cdot}004b_v d$ for each percentage of reinforcement above the minimum value noted above.

The guide specification [8] defines the sections to be checked at the vertical planes of the web/slab interfaces and where dissimilar materials join.

References

1. BS 5400 (1990) *Steel, Concrete and Composite Bridges, part 4. Code of Practice for the Design of Concrete Bridges*. London: British Standards Institution.
2. AASHTO (1996–2002) *Standard Specification for Highway Bridges*, 16th Edition. Washington, DC: American Association of State Highway and Transportation Officials.
3. Garrett, R.J. and Cochrane, R.A. (1970) 'The analysis of prestressed concrete beams curved in plan with torsional restraints at the supports', *The Structural Engineer*, **3**, March, 128–32.
4. Witecki, A.A. (1969) 'Simplified method for the analysis of torsional moment as an effect of a horizontally curved multispan continuous bridge', *American Concrete Institute*, First International Symposium on Concrete Bridge Design, pp. 193–204.

@Seismicisolation

5. Magura, D.D., Sozen, M.A. and Siess, C.P. (1964) 'A Study of Stress Relaxation in Prestressing Reinforcement', *PCI Journal*, 9, **2**, April.
6. Maisel, B.I. and Roll, F. (1974) *Methods of analysis and design of concrete box beams with side cantilevers*. C&CA Technical Report, London.
7. Neville, A.M. (1995) *Properties of Concrete*, 4th edition. London: Longman.
8. AASHTO (1999) *Guide Specification for Design and Construction of Segmental Concrete Bridges*, 2nd edition. Washington, DC: American Association of State Highway and Transportation Officials.
9. Emerson, M. (1977) *TRRL Laboratory Report 765, Temperature Difference in Bridges: Basis of Design Requirements*. Crowthorne: Transport and Road Research Laboratory.
10. BS 5400 (1990) *Steel, Concrete and Composite Bridges, part 2. Specification for Loads*. London: British Standards Institution.
11. Highways Agency (2001) *(MDRB) Departmental Standard, BD 37/01, Loads for Highway Bridges*. Norwich: HMSO.
12. Clark, L.A. (1984) 'Longitudinal shear reinforcement in beams', *Concrete*, **18**, **2**, February, 22–3.
13. Highways Agency (1994) *(MDRB) Departmental Standard, BD 58/94, The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing*. Norwich: HMSO.
14. AASHTO (1998–2002) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, 2nd edition. Washington, DC: American Association of State Highway and Transportation Officials.

@Seismicisolation

6

Design of details

Introduction

Prestressed concrete bridge decks have a number of general features such as the anchorages, couplers and tendon ducts, while other details such as deviators and diaphragms are common occurrences. This chapter looks at the design and detailing of these different features.

Where formulae are used in this chapter the units are in N and mm.

Anchorages

All prestressing tendons are anchored against or within the concrete. These anchorages exert large concentrated forces and give rise to high local stresses that must be catered for in the design. Anchorage of pre-tensioned strand is achieved by bond between the wires and the concrete, while for post-tensioning the anchorage is with anchor blocks bearing against the concrete or by embedment of cast-in dead-end anchors within the concrete.

Pre-tensioned strand

When pre-tensioned strand is released from the jacking frame the force transfer is achieved through bond between the wires and the concrete. At the end of the beam, as seen in Fig. 6.1, the strand slips into the concrete and the force is transferred as the bond gradually builds up away from the end face until the concrete takes up the total force in the strand. The distance over which the force transfer takes place is the transmission length; l_t. The slip-in at the end is typically between 2 mm and 3 mm.

When $F_o \leq 75$ per cent UTS and $f_{ci} \geq 30\,\text{N/mm}^2$, BS 5400 part 4 [1] defines the transmission length as $l_t = k_t D_t / \sqrt{f_{ci}}$.

k_t depends on the type of wire or strand as follows:

for plain, indented or crimped wire with a wave height of less than $0{\cdot}15D_t$, $k_t = 600$
for crimped wire with a total wave height greater than $0{\cdot}15D_t$, $k_t = 400$
for 7-wire standard and super-strand, $k_t = 240$
for 7-wire drawn or compacted strand, $k_t = 360$

For a typical 15·2 mm diameter 7-wire strand with a stress of 70 per cent of UTS and a concrete strength of 35 N/mm^2 the transmission length is 617 mm.

@Seismicisolation

Figure 6.1. Pre-tensioned strand at the end of a precast beam.

With AASHTO standard specifications [2] the development length is defined as:

$$l_t = 0{\cdot}145 D_t (f_{su}^* - 0{\cdot}67 f_{pe})$$

No reinforcement is needed around the strand to counter the local stresses; however, depending on the overall distribution of the strand around the section, it may be necessary to provide equilibrium reinforcement in a way similar to the anchors of post-tensioned tendons.

Post-tensioned tendons

When post-tensioned tendons, either strands or bars, are anchored they apply a large concentrated force to the concrete which must be contained. The guide published by CIRIA (1976)[3] describes the behaviour and design of post-tensioned concrete anchor blocks for simple arrangements. The force from the tendon is first transferred directly to the anchor plate and then into the adjacent concrete. The region of concrete immediately behind the anchor block is highly stressed and reinforcement is needed to prevent the concrete splitting or spalling. The force flows from the anchor out into the surrounding concrete, as indicated in Fig. 6.2(ii), until, at a distance away from the end face, the stress distribution matches that derived from simple bending theory. The distance taken to reach this 'normal' stress distribution

@Seismicisolation

is usually taken as equal to the depth of the anchor block, although the distance can be shorter than this with multi-anchor arrangements spread out on the end face.

Behind each anchor splitting forces and tensile stresses occur which are resisted by reinforcement as indicated in Fig. 6.2(iii) and quantified in BS 5400 part 4 [1] as follows:

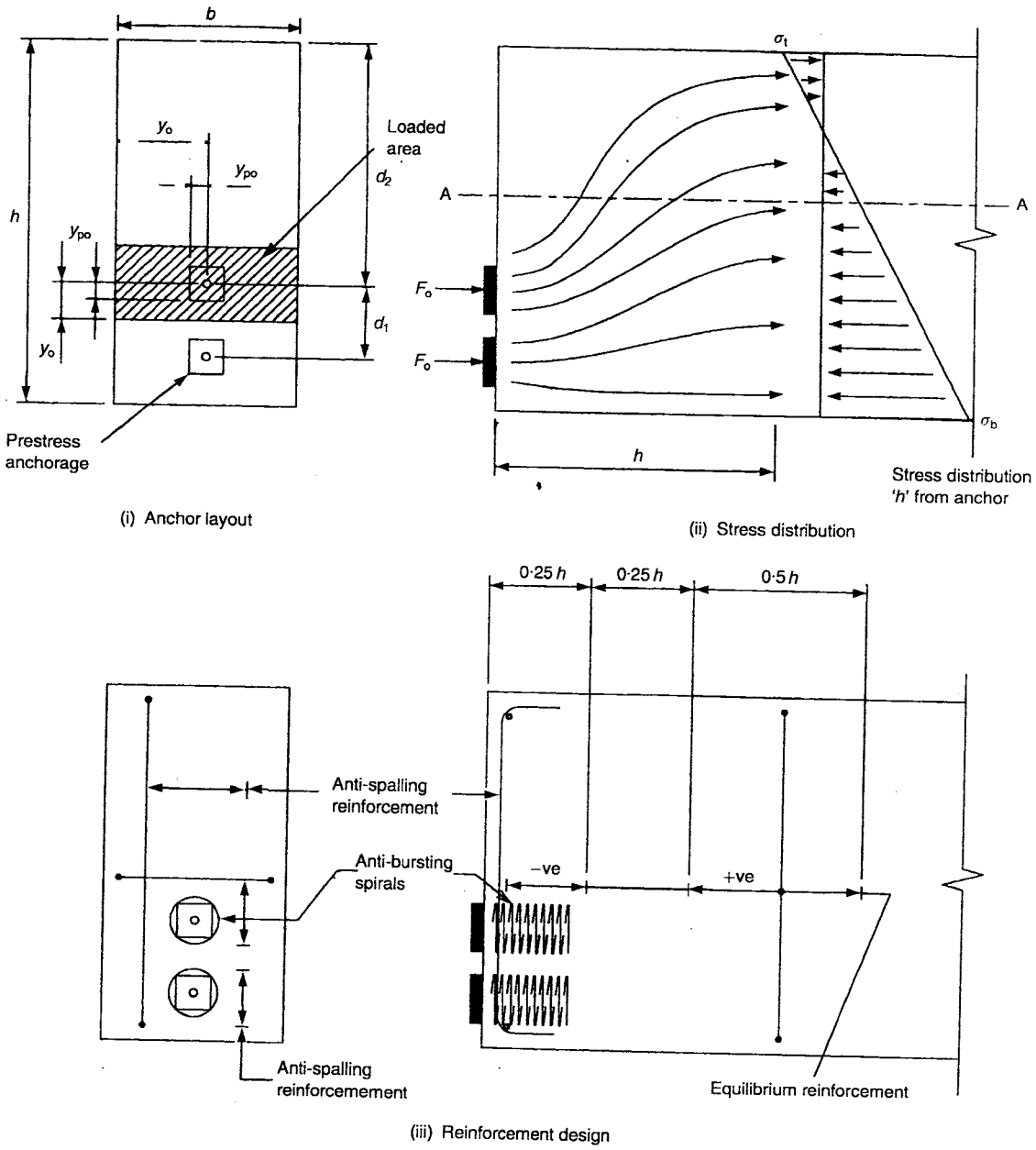


Figure 6.2. End block design.

@Seismicisolation

Bursting reinforcement: provided as a spiral or series of links around each individual anchor to counter the bursting forces set up perpendicular to the line of the tendon, with:

$$A = F_{bst}/(0{\cdot}87f_y)$$

The bursting force, F_{bst} depends on the end-block arrangement and is given by:

y_{po}/y_o	0·3	0·4	0·5	0·6	0·7
F_{bst}/F_o	0·23	0·20	0·17	0·14	0·11

The force on the anchor, F_o, varies from the jacking force, reducing as the tendon is locked off and suffers long-term losses. $0{\cdot}87f_y$ is often replaced by a stress of 200 N/mm^2 or less to control cracking in the concrete; although higher stresses are usually acceptable for the short-term loading before tendon lock-off.

Bursting reinforcement is provided as a spiral or links uniformly spread out over a distance between $0{\cdot}2y_o$ and $2y_o$ from the anchor face and is detailed to enclose a cylinder or prism with dimensions 50 mm larger than the face of the anchor plate. The bursting forces, in two directions perpendicular to the tendon, are determined and reinforcement provided to suit. Where spirals or links are used, the area of reinforcement resisting the bursting is taken as the total area crossing the plane being considered.

The loaded area used for determining y_o is taken as symmetrical about the anchor extending to the nearest edge of the concrete or the midpoint between adjacent anchors, as illustrated in Fig. 6.2(i).

Figure 6.3 shows the bursting reinforcement provided on each of the three tendons by spirals placed just behind the anchor-block face, on the left side of the picture.

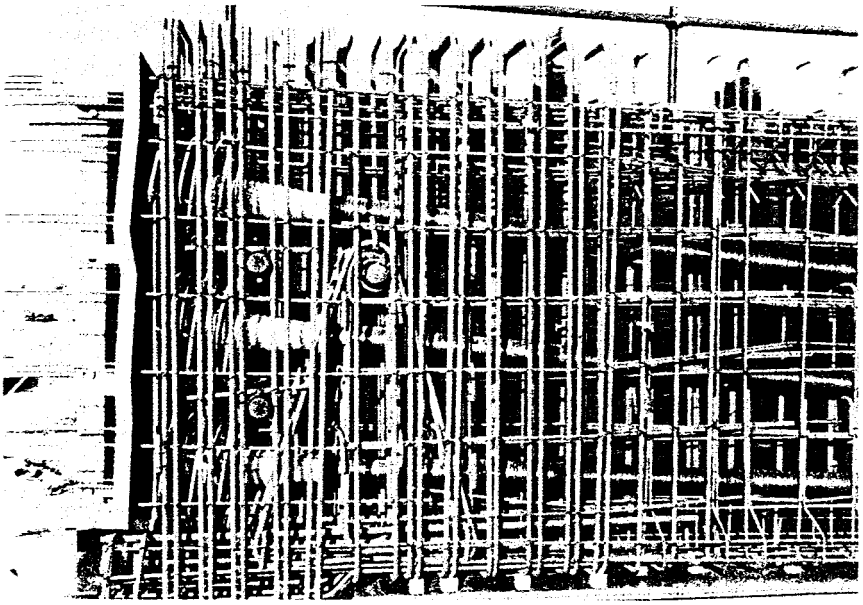


Figure 6.3. End block reinforcement with internal ducts.

@Seismicisolation

Spalling reinforcement: provided to prevent spalling of the end-face concrete around the anchor plate, with

$$A = 0{\cdot}04F_o/(0{\cdot}87f_y)$$

To control cracking in the concrete, the stress in the reinforcement should not exceed 200 N/mm^2, while the bars should be placed as near to the end face as possible and anchored around the concrete edges. Corners and edges of the concrete anchor block are particularly vulnerable and the spalling reinforcement should be placed with its minimum cover in these areas.

Where the anchor is positioned non-symmetrically on an end face additional spalling stresses are set up and additional reinforcement is provided in the unsymmetrical face such that;

$$A^1 = 0{\cdot}2((d_1 - d_2)/(d_1 + d_2))^3(F_o/0{\cdot}87f_y)$$

Equilibrium reinforcement: provided to maintain the overall equilibrium of the end block.

The force from the anchor block is assumed to have fully spread out at a distance h from the anchor face as indicated in Fig. 6.2(ii). By consideration of the concrete block with the anchor force on one side and the stress distribution on the other, the equilibrium of any horizontal plane, such as A–A, is checked and vertical reinforcement provided to resist the out-of-balance moment and shear. Based on the method presented in the CIRIA guide (1976) [3] reinforcement is provided to resist a force equal to the moment divided by a lever arm of $h/2$. The reinforcement is then placed over the distance $0{\cdot}25h$ or $0{\cdot}5h$ as indicated in Fig. 6.3(iii) depending on the direction of the out-of-balance moment.

The shear on each horizontal plane is checked to ensure that the shear stress does not exceed $(2{\cdot}25 + 0{\cdot}65\rho f_y)$ N/mm^2, where ρ is the ratio of the reinforcement crossing the plane and is equal to A_s/bh.

Where flanged box or beam sections are used, the stress behind the anchor has to spread out over the width of the flange. The situation where the tendons are anchored in the webs is indicated in Fig. 6.4. The flow of stresses into the flanges is checked for 'equilibrium' effects and reinforcement provided as described above. For complex concrete shapes and anchor block arrangements it is necessary to carry out three-dimensional finite element analysis of the end block to determine the 'equilibrium' requirements with reinforcement being provided to counter any tensile stresses set up.

AASHTO [2] gives guidance resulting in similar reinforcement being provided allowing either elastic stress analysis, strut and tie models or the results from experimental tests to be used to derive the forces and reinforcement required.

When several tendons are anchored at the same section the bursting, spalling and equilibrium requirements change as each tendon is stressed. The order of stressing of the tendons should be specified by the designer and the design checked for the situation after each tendon is installed.

The reinforcement in the anchorage zone has to be carefully detailed to ensure that all the necessary reinforcement is included without causing congestion, which would hinder placing of the concrete. Bursting reinforcement

@Seismicisolation

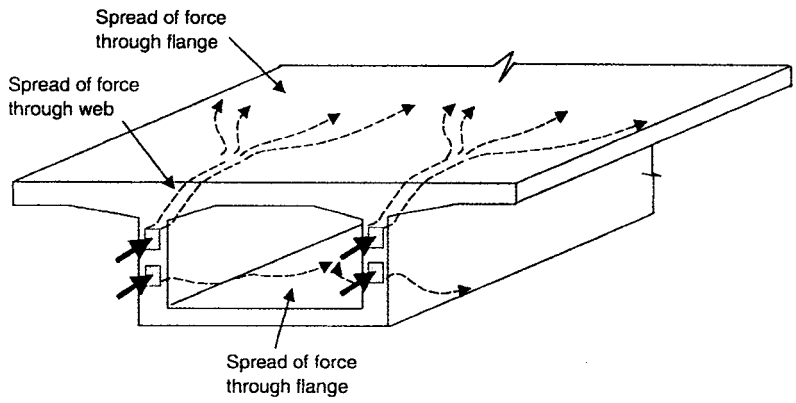


Figure 6.4. Spread of stress into flanges.

is provided by a spiral or links placed around the tendon close to the anchor face and inside the other spalling and equilibrium reinforcement near the concrete surfaces. The spalling reinforcement normally consists of small diameter bars placed as close to the end face as possible and around all vulnerable corners. Equilibrium reinforcement is placed around the perimeter of the section over the appropriate region in the end block. Figure 6.3 shows a typical reinforcement arrangement for the end block of a precast concrete beam. As well as the anti-bursting spirals behind the anchors, the spalling reinforcement can be seen with horizontal and vertical bars near the end face. The closer spacing of the vertical links indicates the equilibrium reinforcement.

To ensure that the tendon force is applied squarely to the anchor block the tendons must be accurately positioned within the anchorage area and be straight for a minimum distance behind the anchor plate. The minimum distance required is a function of the tendon size or force in the tendon, and is typically 800 mm for a 7 × 15 mm strand tendon, 1000 mm for a 19 × 15 mm strand tendon and 1500 mm for larger tendons.

Cast-in dead-end anchors for post-tensioned tendons

Where dead-end anchors are encased in concrete as 'blind anchors', illustrated in Fig. 2.4, the force is transferred to the concrete by the bond between the strand and concrete and by the wire 'bulbs' at the end. The tension ring around the tendon as it emerges from the duct resists the outward forces generated by the angle change in the individual strands as they fan out. Figure 6.5 shows an alternative dead-end anchor arrangement for a multi-strand tendon with the strands looped at the end to assist in the anchorage. In this case, spiral reinforcement is provided to counter any potential bursting forces. The overall reinforcement requirements are governed by the arrangement of the strand and are usually specified by the suppliers of the system being used.

Depending on the concrete shape and arrangement of the dead-end anchors, it may be necessary to provide equilibrium reinforcement similar to that described above and to tie the anchor into the concrete behind it to prevent tension cracks forming.

@Seismicisolation

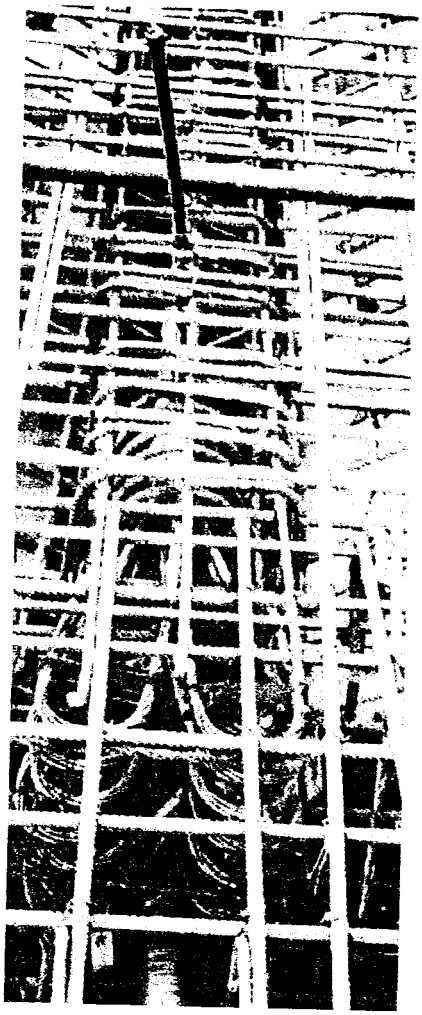


Figure 6.5. Dead-end anchorage reinforcement.

Anchor blisters or blocks

Anchorage blisters or blocks are used mainly in concrete box girder decks, where they are cast on the side of the concrete member as seen in Figs 2.13 and 2.14. As well as the usual bursting and spalling reinforcement, additional bars are needed to tie the blister or block into the main body of concrete, illustrated in Fig. 6.6(i).

Bursting and spalling reinforcement quantities are calculated as above and are shown for a top anchor blister in Fig. 6.7. The additional tie-back reinforcement is required to prevent cracks occurring behind the anchor due to the tensile forces generated to achieve strain compatibility in the concrete. It has been traditional to provide tie-back reinforcement to cater for a force equal to 50 per cent of F_o, although finite element analysis can show that significantly less reinforcement than this is needed in some cases. AASHTO [2] specifies a requirement for a minimum of 25 per cent of F_o for the tie-back force.

@Seismicisolation

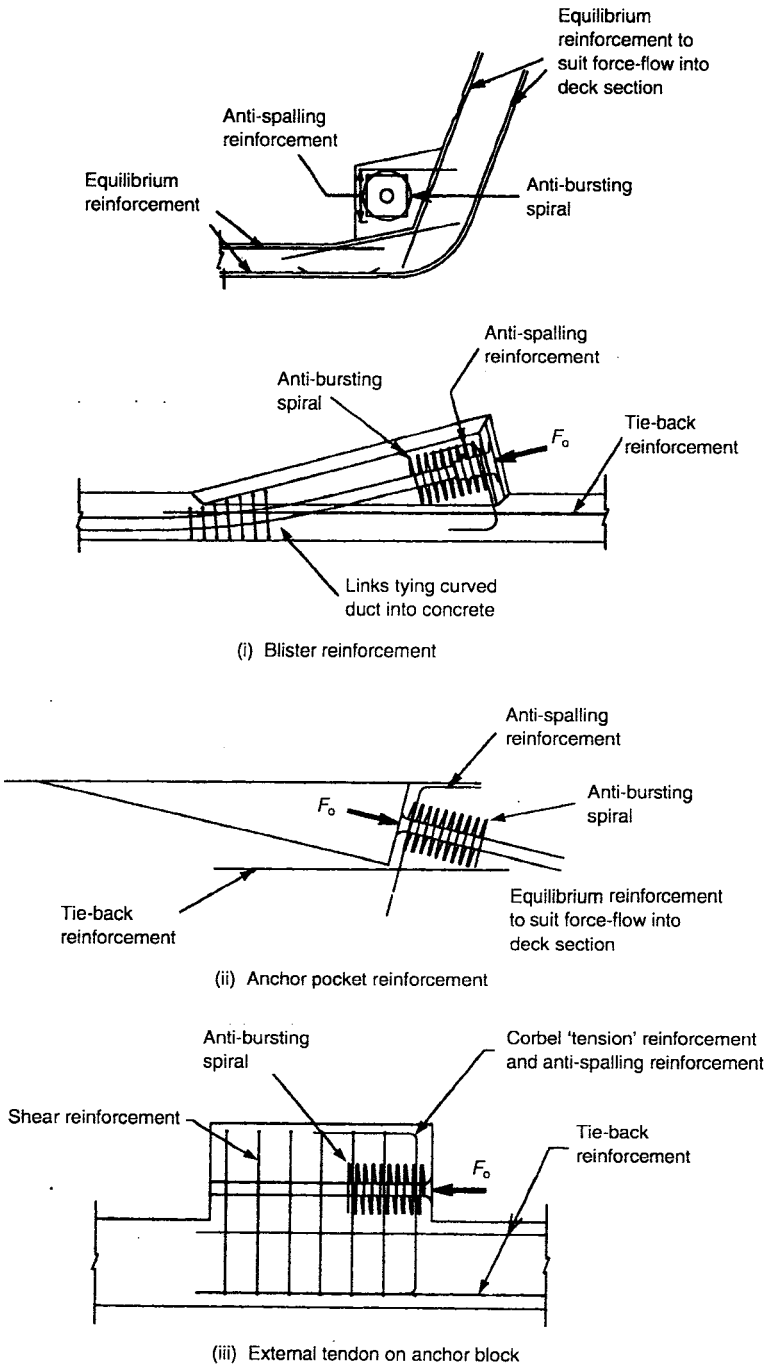


Figure 6.6. Blister and anchor block design.

The blister is normally kept as small as possible to reduce the interference with the shutters used to form the inside of the box or sides of the web. The anchors are positioned at the interface between the webs and slabs close to the concrete faces and with the minimum clearance that is needed to position

@Seismicisolation

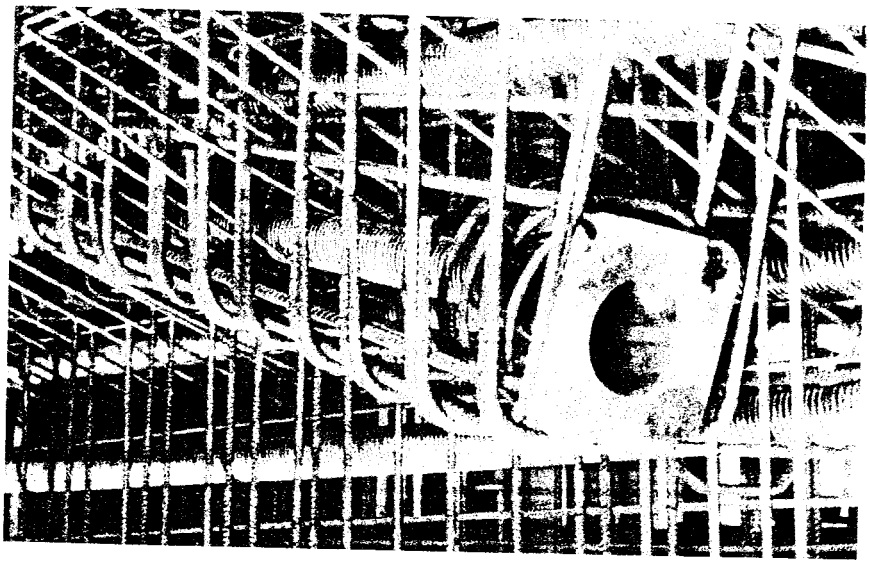


Figure 6.7. Blister reinforcement.

the stressing jack. The blister is sized to give the minimum edge distance to the anchor plate.

With blisters the force from the anchor follows the tendon, flowing directly into the concrete section. The tendon is normally anchored at between 10° and 20° to the web or slab surface. As it passes from the blister into the concrete web or slab it sweeps through a tight curve. Over the length of this curve the tendon is tied into the concrete with links as described in the section on ducts, later in this chapter.

Anchor blocks for external tendons carry the load from the anchor into the web or slab concrete. Depending on their size and layout, they act either as a corbel or as a cantilever and are designed accordingly as indicated in Fig. 6.6(iii).

Equilibrium effects occur for both blister and anchor block arrangements as the force distributes across the section and into the webs and slabs, requiring reinforcement that can be quantified as described in 'Post-tensioned tendons', earlier in this chapter. Local to the blister, transverse tensile stresses can occur in the web and slabs due to the inclination of the tendon, and it is necessary to provide additional reinforcement to cater for this.

The spread of stresses throughout the concrete section can be determined using three-dimensional finite element modelling with solid elements, as illustrated in Fig. 6.8 which shows a short slice of a longer model set up to analyse a blister in a box girder deck. From this analysis, the tensile stresses and forces at each section are determined and reinforcement provided where needed.

Anchor pockets

Figure 6.6(ii) illustrates an arrangement where a recess or pocket is provided in the concrete to anchor the tendons. The large anchor forces tend to cause cracking of the concrete behind the recesses and similar tie-back

@Seismicisolation

reinforcement should be provided as for the anchor blisters, as well as the normal bursting and spalling reinforcement. Equilibrium reinforcement is needed in the webs and slabs to counter the tensile stresses as the force spreads out.

Couplers

Where the tendons are joined together using couplers, normally between one section of deck already concreted and the next section to be built, the local design in front of the coupler is similar to a post-tensioned anchor block. With the arrangement illustrated in Fig. 2.5 the normal anchor plate, as seen on the right side, is cast into the first stage of concrete and, once the tendon is installed and stressed, the reinforcement requirements around the anchor block for bursting, spalling and equilibrium are as defined in 'Post-tensioned tendons' above.

When the tendon in the next section of deck is installed it is coupled to the back of the first tendon and stressed. The force on the anchor plate is reduced as the forces in the tendons either side become balanced. The governing criterion for the design of the reinforcement is during the first stage with

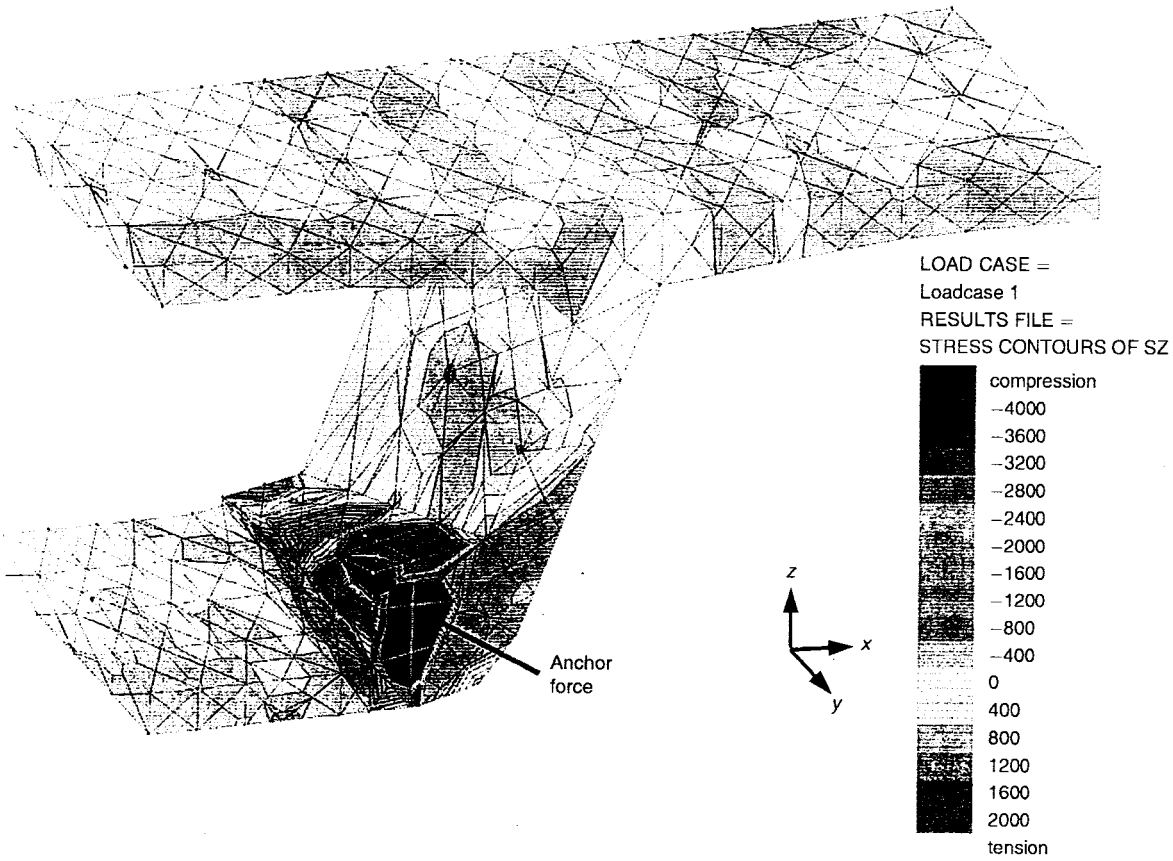


Figure 6.8. Three-dimensional modelling of anchor blister.

@Seismicisolation

only the first section of tendon anchored. There is normally no requirement for any additional reinforcement in the second-stage concrete behind the coupler as there are no forces imparted locally to the surrounding concrete.

Couplers require the concrete section to be large enough to house the arrangement and still maintain sufficient concrete cover to the surrounding reinforcement. This is seldom a problem with bar couplers, as illustrated in Fig. 2.6, due to their compact size; however, with multi-strand tendons the concrete section often needs thickening around the coupler.

Ducts

Duct sizes are governed by the practicalities of having sufficient space to thread the tendon through and to allow the grout to flow freely around the strands or bars. The internal area of a duct is not normally less than twice the tendon area. Typical duct sizes are indicated in Table 2.2, which shows the different standard systems available.

Internal tendon ducts, as shown in Fig. 6.3, must be supported and held in place during the concreting operation. This is to prevent the ducts from being dislodged or 'floating' up in the fluid concrete. Reinforcing bars are normally placed beneath and around the duct and fixed to the rest of the reinforcement cage to provide a rigid support. The required spacing of these supports depends on the duct size and stiffness, and is normally between 0·5 m and 1·0 m.

To ease concrete placing, to prevent local crushing of the concrete and to ensure that a dense concrete surround is achieved, ducts cast into the concrete should have a minimum clear spacing between each other of at least:

(a) Duct internal diameter
(b) Aggregate size + 5 mm
(c) 35 mm

The concrete cover to the duct should not be less than 50 mm, although this depth may need to be increased when the ducts are curved either perpendicularly or in the plane parallel to the concrete surface. Table 36 of BS 5400 part 4 [1] gives an indication of the minimum cover required for different duct radii. Below these values, reinforcement may be needed to prevent the tendon from bursting out of the duct.

With curved tendons, the strand bears against the edge of the duct and exerts a pressure on the concrete causing splitting forces that must be considered in the design. The tendency to split the concrete depends on the force in the tendon and the radius of the curve. The force per unit length on the concrete is equal to the force in the tendon divided by the duct radius, in radians. When the radius is large enough, no additional reinforcement is needed as the tensile strength of the concrete is sufficient. With tighter curves reinforcement should be provided to restrain the tendon and transfer the force back into the surrounding concrete, as indicated in Fig. 6.9.

Curved tendons running parallel to each other in the plane of curvature are at risk of bursting into the adjacent duct. If the radius of the ducts is large and there is sufficient spacing between the ducts, as noted above, the concrete

@Seismicisolation

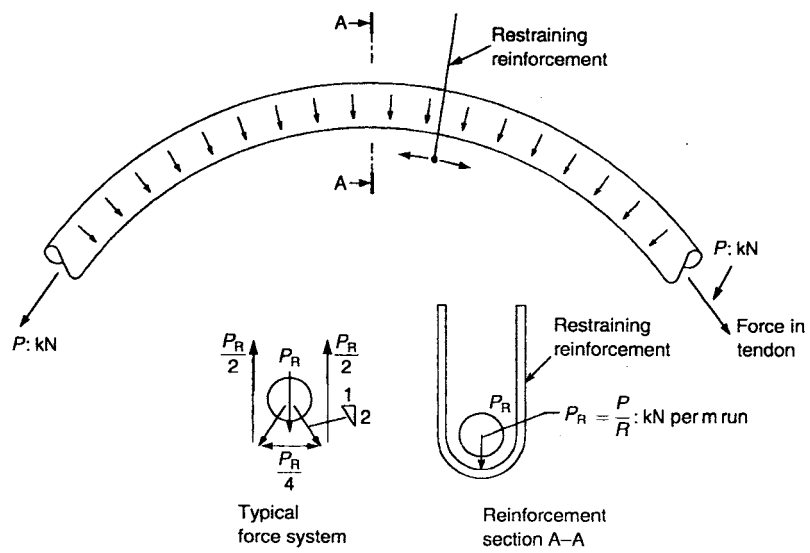


Figure 6.9. Restraining curved ducts.

tensile strength should be adequate to prevent any bursting. Table 37 of BS 5400 part 4 [1] gives an indication of the minimum radii and spacing of the ducts below which reinforcement may be needed.

Where a curved tendon exerts a force on the concrete section it is sometimes necessary to provide additional reinforcement. With a tendon placed inside the bottom slab of a haunched box girder a resultant downward force is generated, as indicated in Fig. 6.10. This will produce bending and shears in the bottom slab and tension in the webs.

Multi-strand tendons can follow a profile with tight curves, the minimum radii being governed by the ability of the duct to bend during fixing. Individual strands can be bent to radii less than 0·5 m. With tendons the minimum radius varies from 3 m for a 4 × 15 mm strand system, 5 m for 12 × 15 mm strands and up to 8 m for a 31 × 15 mm strand system.

External tendon ducts are easily accessible between the deviators and anchorage points, but they should be checked during the design to ensure

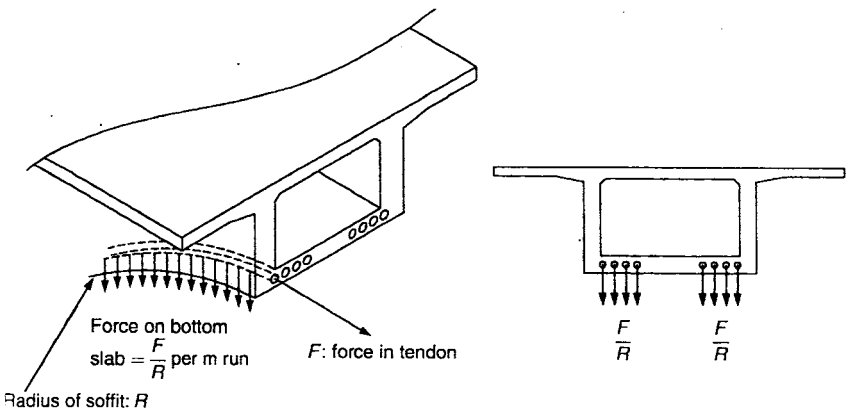


Figure 6.10. Tendons in curved bottom slab.

@Seismicisolation

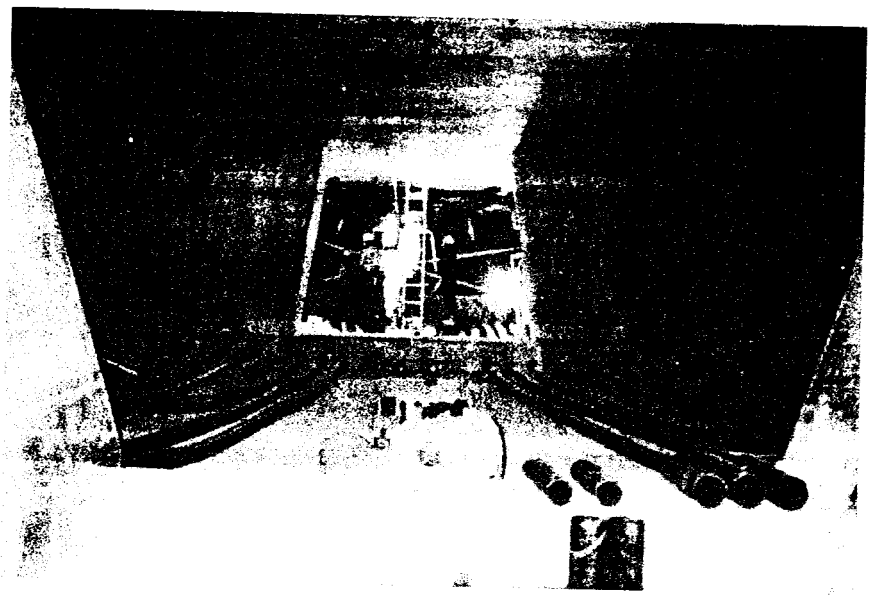


Figure 6.11. Ducts for external tendons.

that they do not clash with each other or push against the concrete surface as they follow their intended alignment. Figure 6.11 shows a series of external ducts being laid out before fixing in position and threading the tendons. Each duct is well clear of its neighbour, allowing easy inspection, and having wide spacing as they pass through the deviator to give plenty of room to fix the reinforcement around each hole.

Diaphragms

Diaphragms are generally used in the deck at points of support to transfer the vertical load from the webs into the bearings below and to provide transverse stiffness to the deck section. Typical diaphragm-wall arrangements for box girders and I beam decks are illustrated in Fig. 6.12. For the arrangement shown in Fig. 6.12(i), a truss analogy is normally used to model the force transfer from the webs into the bearings. The force is resisted by a 'tie' across the top of the diaphragm and a strut towards the bearing. In Figs 6.12(ii) and 6.12(iii) the diaphragms behave as a beam and bending moments and shears can be determined, and reinforcement provided in the normal way.

Where a web is positioned over a bearing the vertical load goes directly into the support. Elsewhere 'hanging reinforcement' may be needed to take the force from the web up to the top of the diaphragm 'beam' to ensure that the flow of force accurately reflects the assumed structural behaviour.

When a bearing is offset from the web by less than half the deck depth the hanging reinforcement can be proportioned to carry just the shear force in the webs below the line A–A, indicated in Fig. 6.13(i). The remainder of the force is transferred through the concrete using shear friction or corbel action. Where the bearing offset is greater than half the deck depth hanging reinforcement is provided to carry the full shear force on the diaphragm.

@Seismicisolation

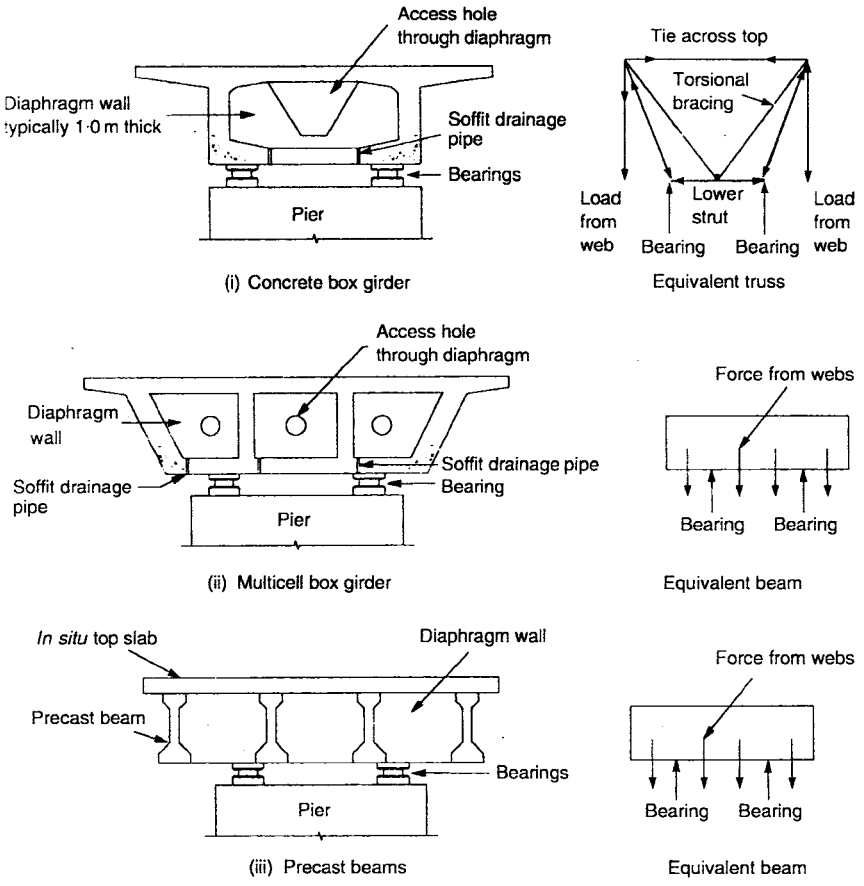


Figure 6.12. Diaphragm arrangements.

Hanging reinforcement is provided in the form of vertical bars distributed over part of the diaphragm and webs, as indicated in Fig. 6.13(ii). The reinforcement provided in the webs for vertical shear can be utilized as part of the hanging requirement, while draped tendons running along the deck can be used to carry some of the vertical load up to the top of the diaphragm.

Where significant torsion exists in a box girder deck, it is necessary to consider the horizontal forces present in the top and bottom slabs adjacent to the diaphragm and to transfer these forces into the bearings. Reinforcement should be provided across the diaphragm to resist these forces and to tie the slabs together.

As an alternative to providing reinforcement for the 'hanging' and tie forces, vertical and transverse prestressing tendons can be used. Using prestressing to resist the forces in the diaphragm reduces congestion of the reinforcement, making concreting easier as well as reducing potential cracking by keeping the concrete in compression. Figure 6.14 shows an arrangement where vertical bar tendons were used for a diaphragm in a precast segmental box girder deck.

@Seismicisolation

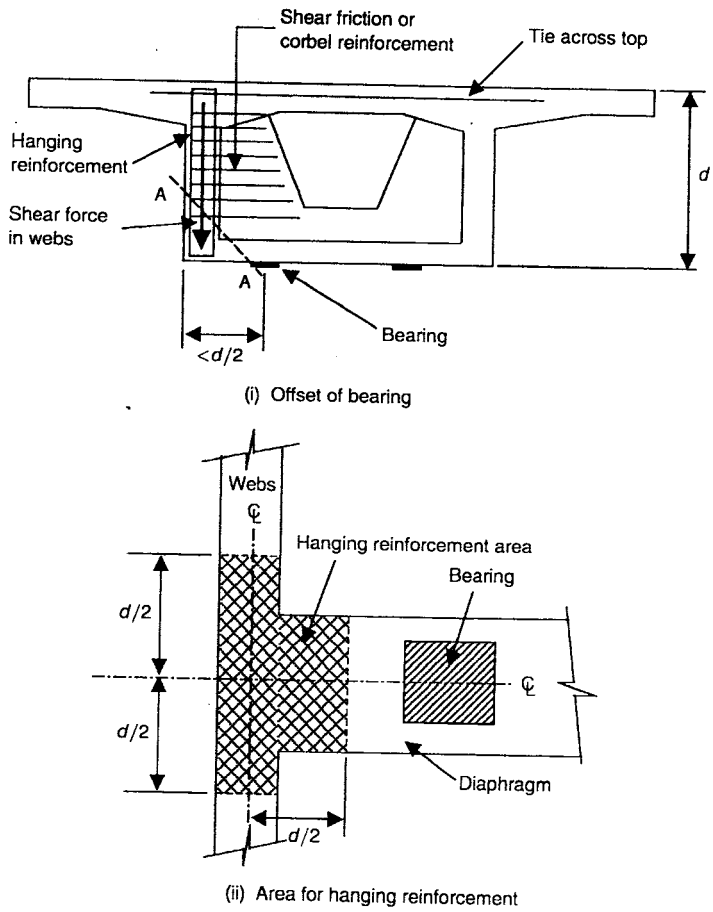


Figure 6.13. Diaphragm hanging reinforcement.

Where diaphragms are incorporated into the deck over bearings they should be designed with provisions for temporary jacks to be installed to allow the replacement of the permanent bearings in the future.

The concrete above both the permanent and temporary bearings is designed in a manner similar to that for an anchor end block, with spalling and bursting reinforcement being provided accordingly. With the temporary bearings it is usually assumed that a reduced live load would occur when the bearing replacement takes place.

Figure 6.15 shows a half-depth diaphragm arrangement for a precast beam deck. In this case each of the beams is supported directly by the bearings on the pier crosshead and the small half-depth diaphragm provides transverse restraint and rigidity but is also used for jacking up the deck should the bearings need replacing. This simple arrangement provides an efficient structural system. The governing criterion for the design is the situation with temporary jacks placed under the diaphragm during replacement of the bearings.

Transverse diaphragms are often used for precast beam decks with longer spans at 'quarter' or 'third' points in the span to connect the beams transversely

@Seismicisolation

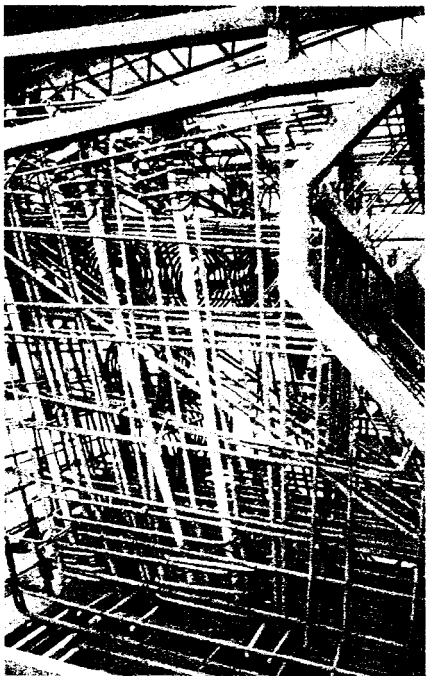


Figure 6.14. Vertical prestress in diaphragm.

and improve the distribution of the applied loads across the deck. These intermediate diaphragms are usually of minimal thickness and lightly reinforced, and the design forces, moments and torsions are derived from a grillage or three-dimensional analysis of the deck.

In box girder decks, diaphragms are usually provided at each end support and over each intermediate pier, with a typical arrangement shown in

Figure 6.15. 'U' beam diaphragm.

@Seismicisolation

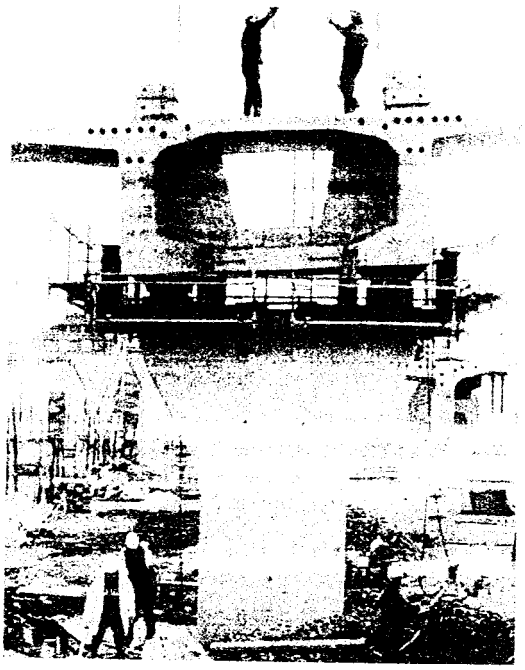


Figure 6.16. Typical box girder intermediate diaphragm. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

Fig. 6.16. The diaphragm walls are typically 1·0 m to 1·5 m wide to extend over the width of the bearings plus an allowance for any relative movement between the deck and the support. The position of the temporary jacks during bearing replacement is usually just inside the permanent bearings, beneath the diaphragm wall and on top of the pier.

Transverse diaphragms are often used within the span for multi-cell box girders to provide transverse rigidity to the box arrangement and to distribute the applied loading across the complete box section.

Diaphragms are used to anchor external tendons. The thick diaphragm walls provide an ideal location for the anchorages and with suitable reinforcement can easily accommodate the high forces imposed. The anchors should be placed as near to the deck webs or slabs as possible, to minimize the bending generated in the diaphragm, which acts as a beam or wall spanning between the webs and slabs. Access holes through the diaphragms result in uneven load distributions from the external tendons to the deck section. As a simplistic approach to the design, the anchor load can be considered to spread out across the diaphragm at 45° towards the 'supports', with the load in each direction proportioned depending on the concrete arrangement. Reinforcement is then provided in each direction to cater for the associated bending moments and shears.

If the diaphragm thickness is more than 50 per cent of its minimum span between slabs or webs it behaves as a deep beam and reinforcement is provided as described in the CIRIA Guide No. 2 (1977) [4].

More refined analysis and design are undertaken using a detailed three-dimensional finite element model which is used to derive the distribution of forces and stresses from the anchors to the deck section.

@Seismicisolation

Deviators

Deviators are used to deflect external tendons to give the desired profile and are subjected to large applied forces. These forces from the tendons must be tied into the deck section. Figure 6.17 illustrates several different types of deviators that are commonly used. The concrete beam is the most common arrangement and can cater for very high loads from the deviated tendons with the force being transferred into the deck section either by a 'strut-and-tie' action or by bending and shear through the beam. The capacity of the concrete-block arrangement is less than with the beam arrangement as the force from the tendon is carried locally into the adjacent concrete with bending in the slabs and webs. Steel deviator arrangements are often used on existing structures where external tendons are being retrospectively fitted to provide strengthening. For new construction it is usually easier to design and construct concrete deviators, while for existing structures it is often easier to install steel deviators.

Concrete beam and block deviators are shown in Figs 6.18 and 6.19 respectively. The external tendons pass through holes or tubes left in the

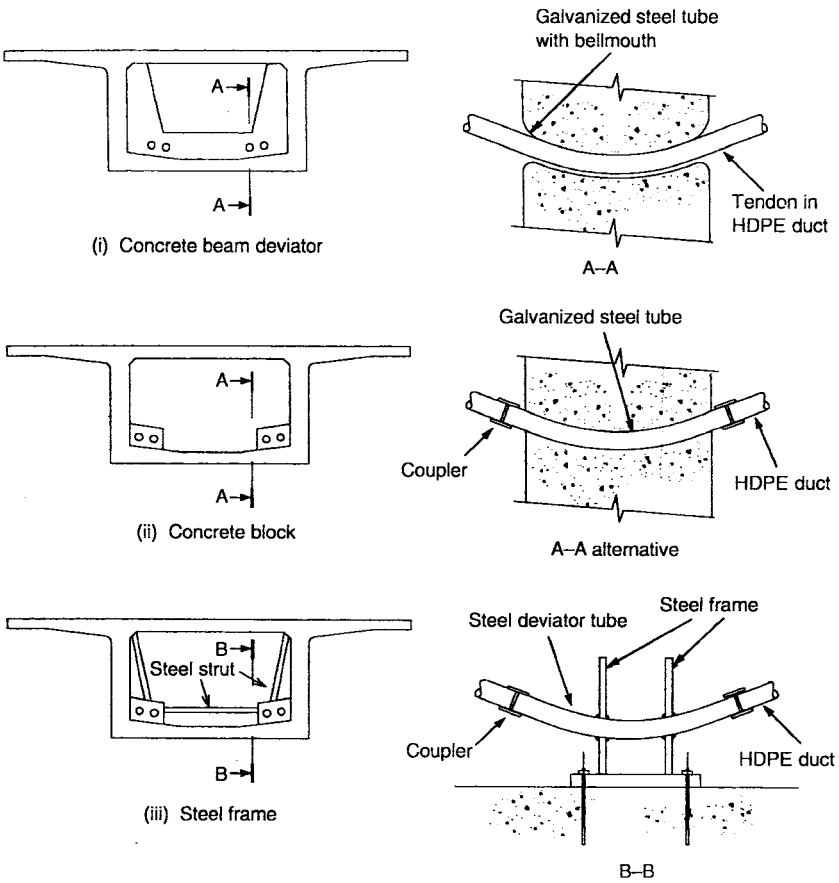


Figure 6.17. Deviator arrangements.

@Seismicisolation

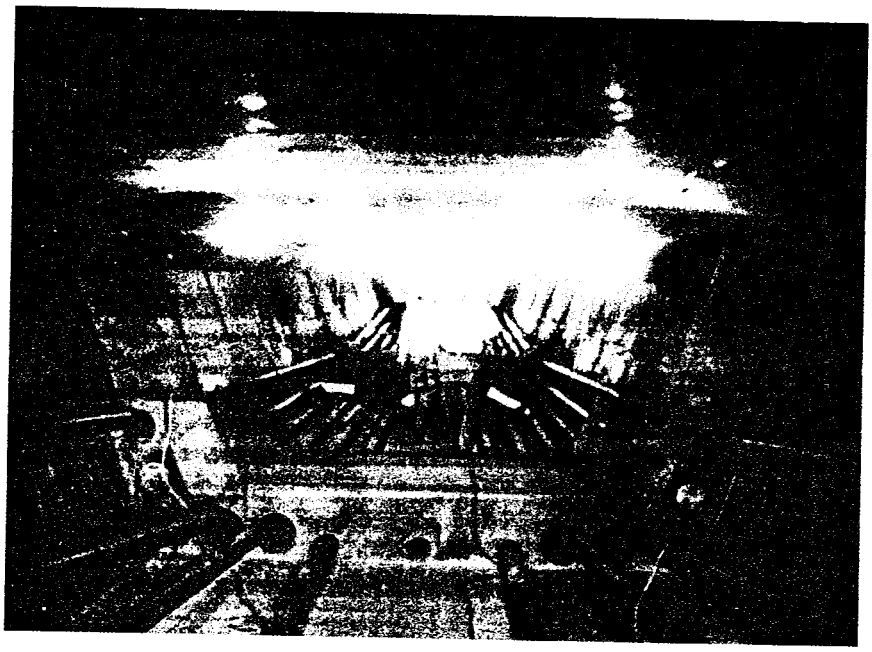


Figure 6.18. Concrete beam deviator.

concrete that guide the tendons through the required alignment. The radii of the tendons through the deviator typically range from 2·5 m for a 7 × 15 mm strand tendon, 3·0 m for a 19 × 15 mm strand tendon and up to 5·0 m for 27 × 15 mm strand tendons. In Fig. 6.18 the beam deviator has also been used to anchor some of the external tendons, while the hole in the centre of the beam is for a future additional tendon should this be required.

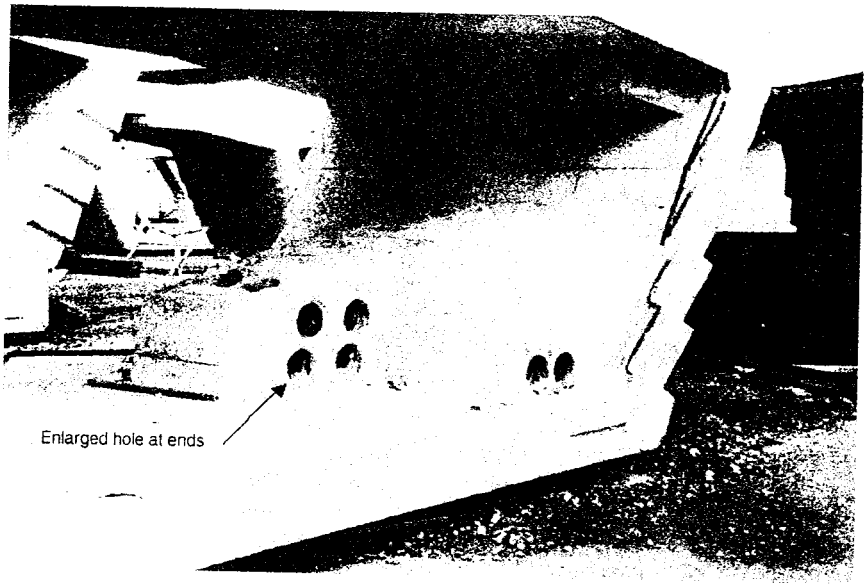


Figure 6.19. Concrete block deviator.

@Seismicisolation

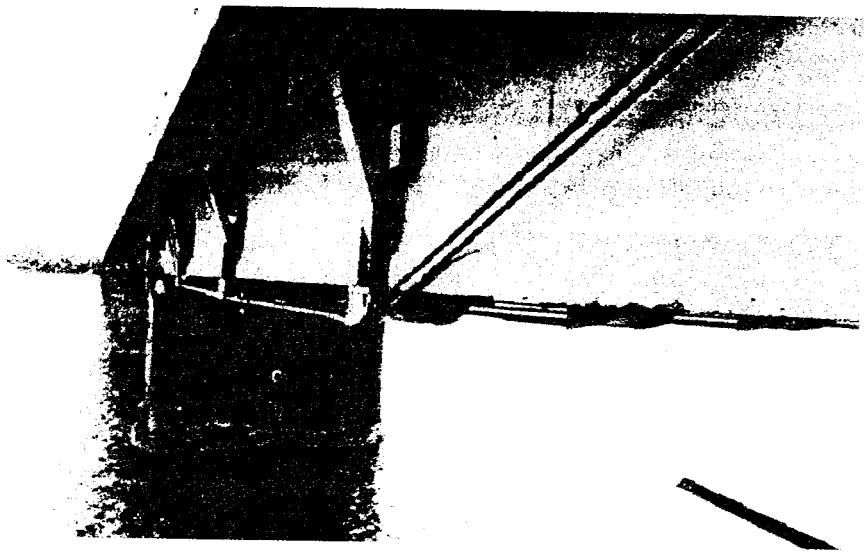


Figure 6.20. Steel deviator. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

Figure 6.20 shows a steel deviator clamped to the underside of a prestressed concrete beam, used during the strengthening of an existing bridge. In this example the steel deviator transfers the load from the deflected tendon into the concrete webs via a concrete nib; alternatively the steel could be fixed to the concrete using prestressing bars.

The holes in the deviators and the continuity of the tendon ducting can be arranged in several different ways. The most common method is to concrete-in oversized steel or stiff plastic sleeves through which the external tendons are threaded while still inside their own protective duct system, as illustrated in Fig. 6.17(i). This has the advantage of allowing the external tendon duct to be continuous over its full length. The problem with this arrangement is in lining up the hole correctly when casting the concrete, and 'misalignments' are common occurrences. Any misalignments result in the tendon bearing against the concrete edge where it emerges from the deviator causing kinks in the tendons and spalling of the concrete. This effect can be reduced by enlarging the hole over the end sections as seen in Fig. 6.19 or by including a small radius on the ends of the sleeves so as to move the point of contact back from the end face and reduce the risk of kinks in the tendon.

An alternative arrangement is to place a steel tube into the deviator and to join the external tendon ducting to this on either side as illustrated in Fig. 6.17(ii) and (iii). The disadvantage of the tube method is again the difficulty in achieving the required tendon alignment. When the deviators are being constructed the external tendons are not in place, and setting out the sleeve to the correct angle and position at both ends is difficult.

On some projects the holes through deviators have been formed directly in the concrete without sleeves or tubes, as was the case in Fig. 6.18. A shaped void former is used to create the hole in the concrete during casting and is removed soon after the concrete begins to harden. A bell-mouth arrangement at the ends of the deviator hole allows the tendons to deviate through the

@Seismicisolation

required angle and provides some tolerance should small misalignments occur.

The design force on the deviator is equal to the force in the tendon multiplied by the 'angle change' (in radians). In the UK, where the Highway Agency design standard BD 58/94 [5] is used, the design is carried out at the ultimate limit state with the applied ultimate force based on the characteristic strength of the tendon. Alternatively the design of the deviators is based on the maximum load carried by the tendon with the stresses in the reinforcement limited to permissible stress levels and low enough to minimize the risk of cracks forming in the concrete.

The tendons generate a 'radial' force that is tied into the concrete by links positioned around the hole and extending back into the deck section. The force on the deviator generates bending and shears in the adjacent deck section, which must be designed for and reinforcement provided as necessary.

Where concrete deviators are cast against the sides of webs and flanges, the forces from the deflected tendons flow into the adjacent concrete both longitudinally and transversely. For more complex arrangements it may be necessary to set up a three-dimensional finite element model as illustrated in Fig. 6.21, to determine the distribution of force into the concrete. By using solid elements and modelling a sufficient length of the structure the principal

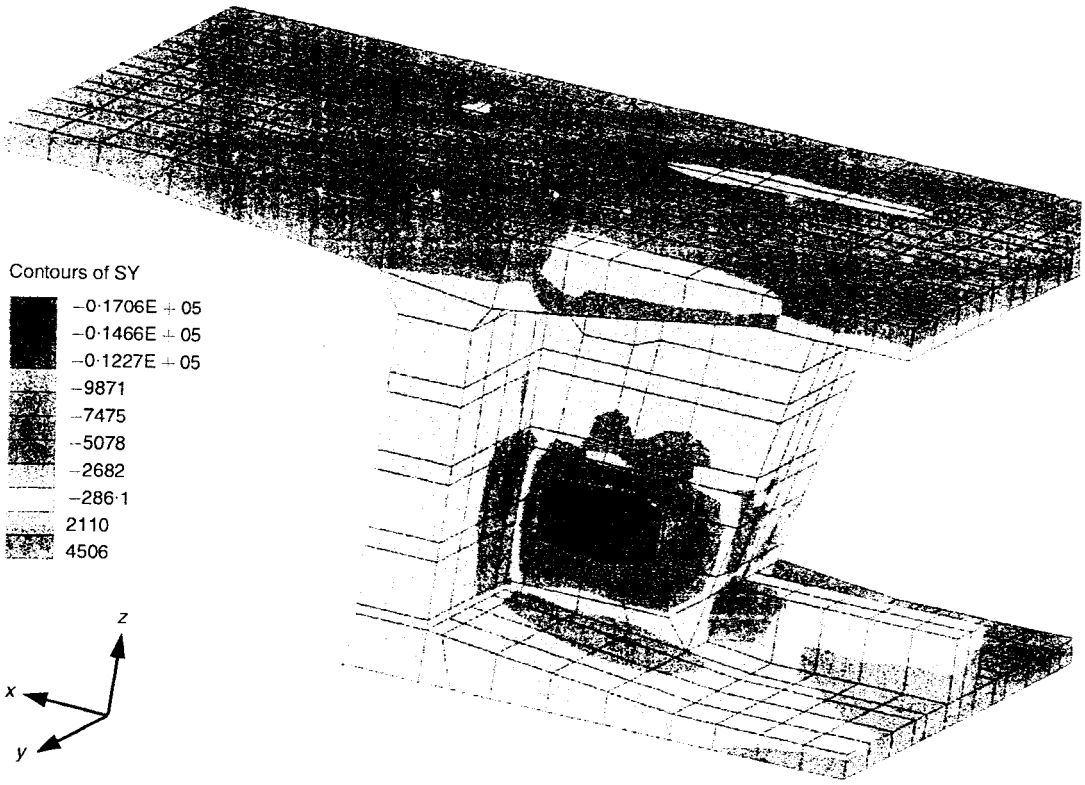


Figure 6.21. Three-dimensional finite element analysis of deviator.

@Seismicisolation

stresses can be derived and the reinforcement provided to cater for any tensile forces that occur.

References

1. BS 5400 (1990) *Steel, Concrete and Composite Bridges, part 4. Code of Practice for the Design of Concrete Bridges*. London: British Standards Institution.
2. AASHTO (1996–2002) *Standard Specification for Highway Bridges, 16th edition*. Washington, DC: American Association of State Highway and Transportation Officials.
3. CIRIA (1976) *Guide No. 1. A Guide to the Design of Anchor Blocks for Post-tensioned Prestressed Concrete Members*. London: CIRIA.
4. CIRIA (1977) *Guide No. 2. The Design of Deep Beams in Reinforced Concrete*. London: CIRIA.
5. Highways Agency (1994) *(MDRB) Departmental Standard, BD 58/94, The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing*. Norwich: HMSO.

@Seismicisolation

7

Concept design of prestressed concrete bridges

Introduction

There are many factors affecting the choice of bridge type, span arrangement and general layout. Prestressed concrete decks include a wide range of construction forms with span lengths ranging from 25 m for single spans to over 400 m in cable-stayed bridges. For spans below 25 m, reinforced concrete is likely to be more economic, while for spans above 400 m, steel or composite cable-stayed decks are more likely to be used to reduce the deck weight. Between these two extremes prestressed concrete often gives an economic, aesthetic and simple solution. Figure 7.1 indicates the typical span range for different types of concrete deck construction.

The choice of deck form is influenced by the particular site constraints, and the advantages and disadvantages of each type need to be carefully considered to determine the optimum solution.

This chapter briefly reviews the different types of prestressed concrete bridges and considers the factors affecting the concept design for an individual project.

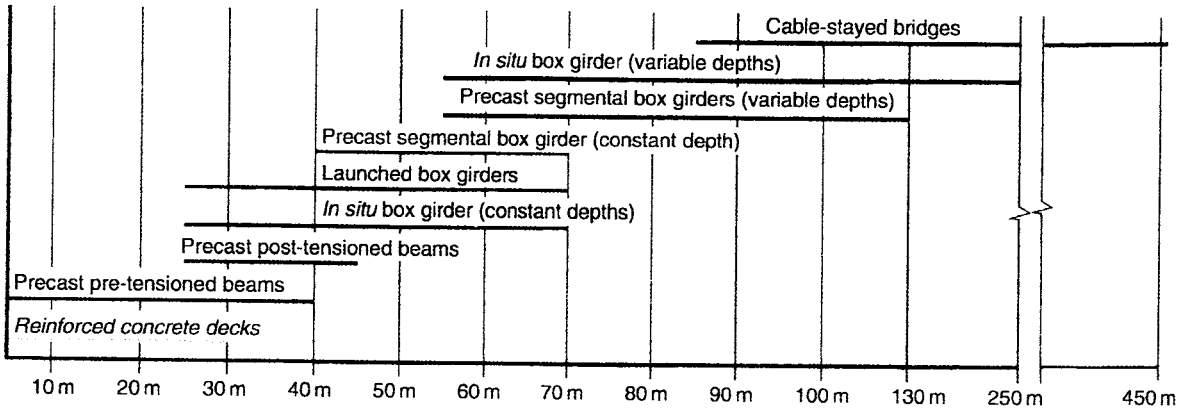


Figure 7.1. Span ranges for different deck types.

@Seismicisolation

Deck types

Prestressed concrete is used in a wide variety of ways in the construction of bridge decks. It is formed as beams or boxes, cast *in situ* or precast, lifted or launched into place and it utilizes either pre-tensioned or post-tensioned tendons. As well as simply supported or continuous span arrangements, concrete decks are also used on cable-stayed structures.

For short spans, *in situ* voided slabs are sometimes used, with the void formers reducing the deck weight and the section area to improve the efficiency of the prestress. The tendons are placed in the concrete 'webs' between the voids or in the slabs. For short spans, precast beams are usually more economic than voided slabs. This has resulted in few voided slab types being constructed in recent years.

Precast beams with an *in situ* concrete deck slab are popular for short and medium span bridges, especially where existing precasting facilities result in cost savings being achieved. For spans up to 30 m precast beams compete favourably on cost with *in situ* reinforced concrete or steel beam construction. Beams utilizing either pre-tensioning or post-tensioning have been developed with a range of different shapes, including 'I', 'T' and 'U' beams. For long multi-span viaducts, as illustrated in Fig. 7.2, precast beams offer cost savings due to the repetition and speed that is achieved during their construction.

Single or multi-cell concrete box girders are well suited for medium- to long-span bridges. They are ideal for curved decks due to their torsional rigidity and are able to cope with complex geometric requirements. They provide efficient solutions and combine with a range of different construction techniques to suit most site conditions.

Construction depths greater than 1200 mm are needed to give the height necessary to construct the boxes, although the need for reasonable access

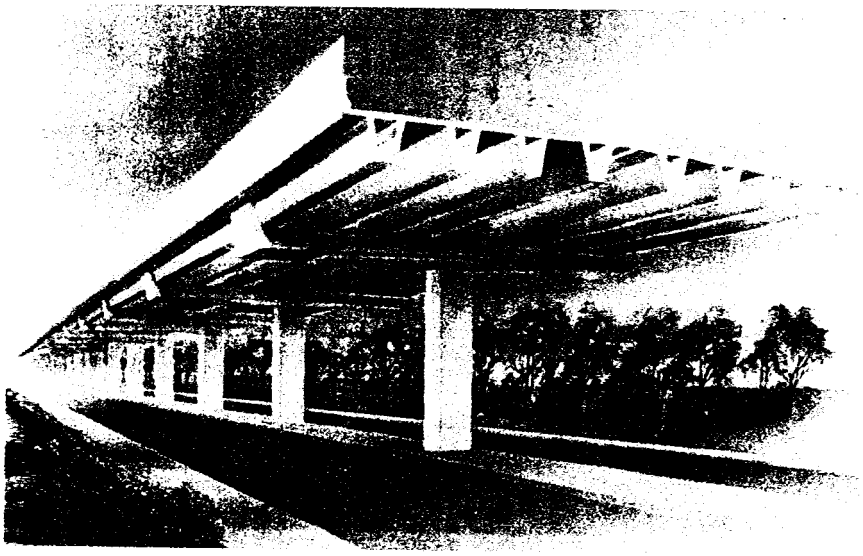


Figure 7.2. Precast beams. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

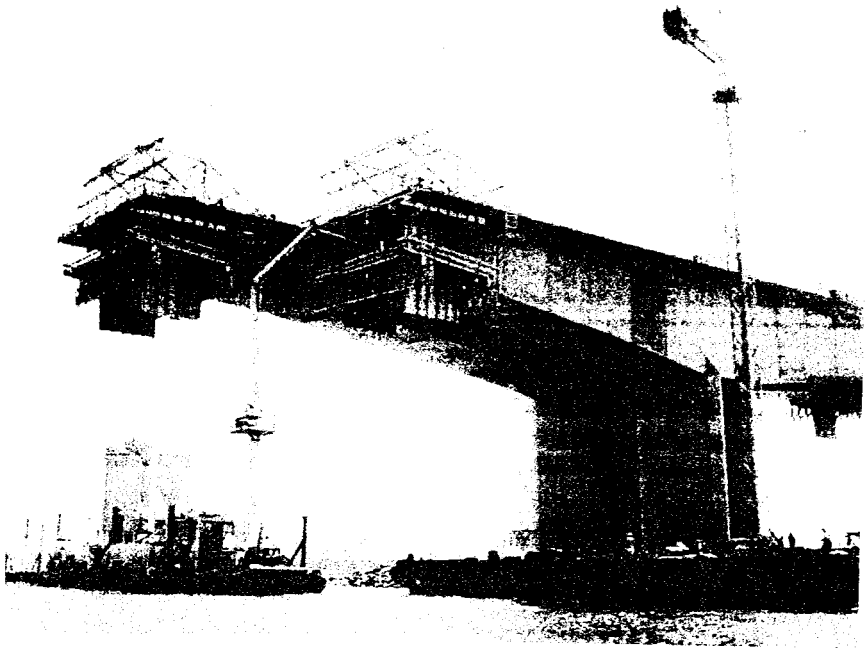


Figure 7.3. In situ *box girder.*

into the box for inspection and maintenance requires a depth of 1800 mm or more. This tends to limit the use of box girders to spans greater than 35 m. Spans less than 35 m are more likely to be constructed using precast beams, *in situ* reinforced concrete or steel beam construction.

In situ box girder decks with spans up to 50 m are usually built on falsework or launched into position. For longer spans the concrete is cast *in situ* as a balanced cantilever, using form travellers as shown in Fig. 7.3.

Precast single- or twin-cell box girders are cast in short segments or in full span lengths. The cost of precasting, transporting and erecting the segments requires a significant length of deck to be constructed for the technique to be economic. Full-length precast units are very heavy and require large equipment to transport and position them; however, they offer an efficient form of construction for long viaducts where savings are achieved through repetition in the construction process.

Short precast segments with match-cast joints, as shown in Fig. 7.4, have become popular where rapid construction and minimum site disruption is a requirement. The financial investment in setting up a casting yard and in the purchase of transportation and erection equipment generally requires deck lengths greater than 2 km before this form of construction becomes economic.

Where access is difficult beneath a bridge deck, or environmental concerns restrict the construction options, box girders are launched into position, as seen in Fig. 7.5. The deck is cast in sections behind one of the abutments and incrementally launched into position. Design requirements of the deck during the launch phase usually result in more concrete, reinforcement and prestress than for normal *in situ* construction. This is offset by savings in

@Seismicisolation

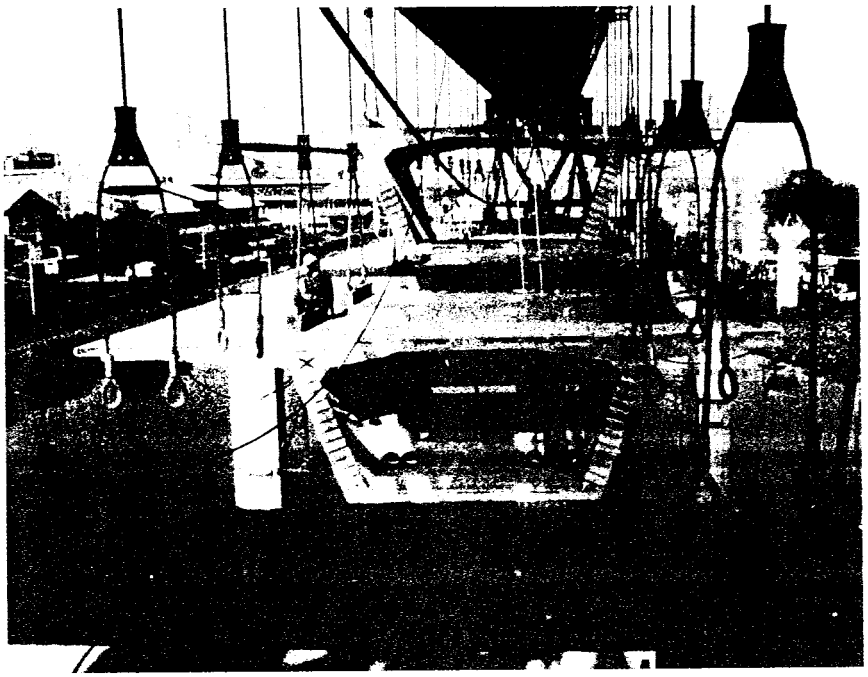


Figure 7.4. Precast segmental box girder.

the temporary works and equipment when compared with other forms of construction.

For longer spans the depth of a concrete box girder can become excessive. Box girders built by the *in situ* balanced cantilever technique have been used

Figure 7.5. Incrementally launched box girder. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

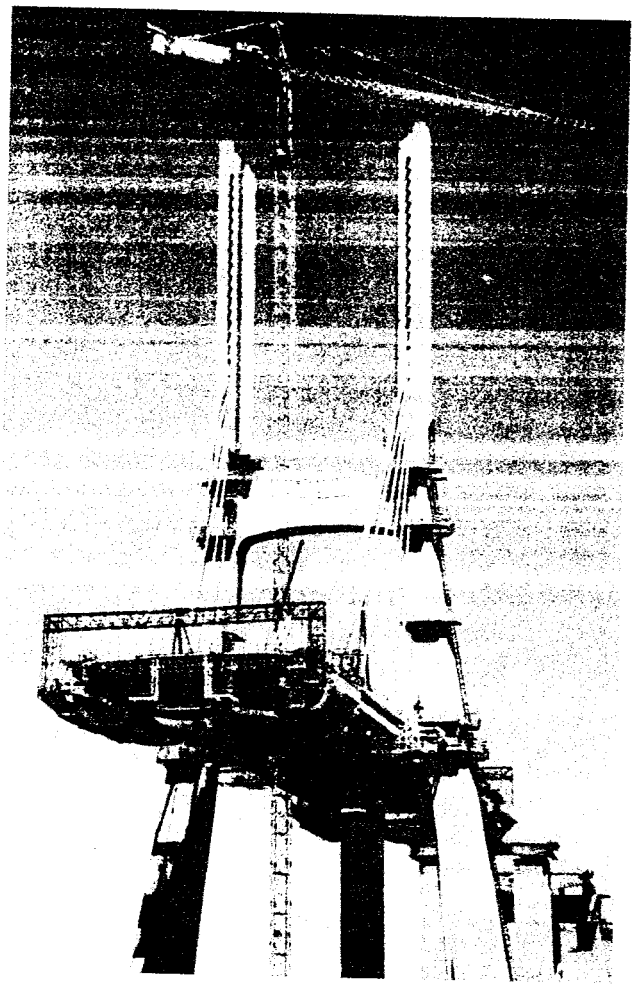


Figure 7.6. Cable-stayed bridge. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

for spans up to 250 m. At this span deck depths exceed 12 m and it is difficult to achieve an aesthetically pleasing solution.

Only relatively shallow decks are required with a cable-stayed arrangement, as seen in Fig. 7.6. Prestressed concrete spans over 500 m long have been built using this form of construction. For spans shorter than 200 m a cable-stayed deck is usually more expensive than alternative deck types although where a shallow deck is needed for clearance or aesthetic reasons cable-stayed arrangements produce elegant solutions. For twin pylon cable-stayed bridges with spans over 500 m steel or composite construction usually offer more economic solutions in the current market.

Prestressed concrete is also used in other types of construction, such as stressed ribbon decks, trusses, extra-dosed and arch bridges. These less common types of deck solutions are usually adopted due to particular site constraints, project requirements or aesthetic considerations.

The advantages and disadvantages of the main deck types are summarized as follows:

@Seismicisolation

Deck type	Advantages	Disadvantages
Slab	• simple construction	• inefficient section • limited to short spans
Voided slab	• simple construction	• inefficient section • limited to short spans
Precast beams	• minimum site disruption • rapid construction • economic for spans 20–35 m • repetitive • reduced on-site work • beams cast in factory conditions	• good access to site needed • heavy lifting equipment • complex to achieve continuous construction • less aesthetically pleasing • limited use with curved decks
In situ multi-cell box girder	• efficient deck section • aesthetically pleasing • simple construction • economic for short lengths and medium spans • can cope with complex geometry	• labour intensive • significant temporary works • slow construction • good access needed
In situ single-cell box girder	• efficient deck section • suitable for tight curves and complex geometry • aesthetically pleasing • simple construction • economic for medium to long spans	• labour intensive • significant temporary works • slow construction
Precast segmental box girder	• efficient deck section • suitable for tight curves and complex geometry • rapid construction • aesthetically pleasing • minimal disruption to site • segments cast in factory conditions • economic for long viaducts	• expensive casting yard and equipment costs • high level of technology required
Precast full-length span box girder	• efficient deck section • rapid construction • aesthetically pleasing • minimal disruption to site • segments cast in factory conditions • economic for long viaducts	• expensive casting yard and equipment costs • high level of technology required • not suitable with difficult access
Incrementally launched box girder	• minimum site disruption • minimal environmental impact • repetition • simple construction • minimum temporary works	• higher concrete, reinforcement and prestress quantities • deeper box section
Cable-stayed deck	• good aesthetic appearance • only option for very long spans	• more expensive than other deck types for short spans • design more complex • sophisticated construction

@Seismicisolation

The typical arrangements and construction techniques for these different deck types are discussed in more detail in Chapters 9 to 17.

Selecting the deck arrangement

It may appear that the designer is presented with a wide choice of bridge arrangements to select from when he considers a particular bridge site, but in practice the choice is usually more restricted once all the restraints and local factors are taken into account.

As an indication of the construction choices for different span lengths, Fig. 7.7 shows the typical deck options for a single span prestressed concrete bridge. Two or three span 'overbridges' would follow a similar choice. Where a deck crosses a railway a common solution is to use precast beams in order to minimize any disruption and reduce the construction risk to the railway system. Similarly, for short river spans precast beams minimize the impact on the river and do not require falsework. For longer spans, box girder construction is usually the most efficient arrangement while for short lengths of deck these are most likely to be built *in situ*. For longer spans over a railway or river the box girder can be incrementally launched into position or built with precast segments; however these are relatively expensive forms of construction for short lengths of deck.

For long multi-span bridges and viaducts the span lengths are usually greater than 30 m for aesthetic and economic reasons. These can be built

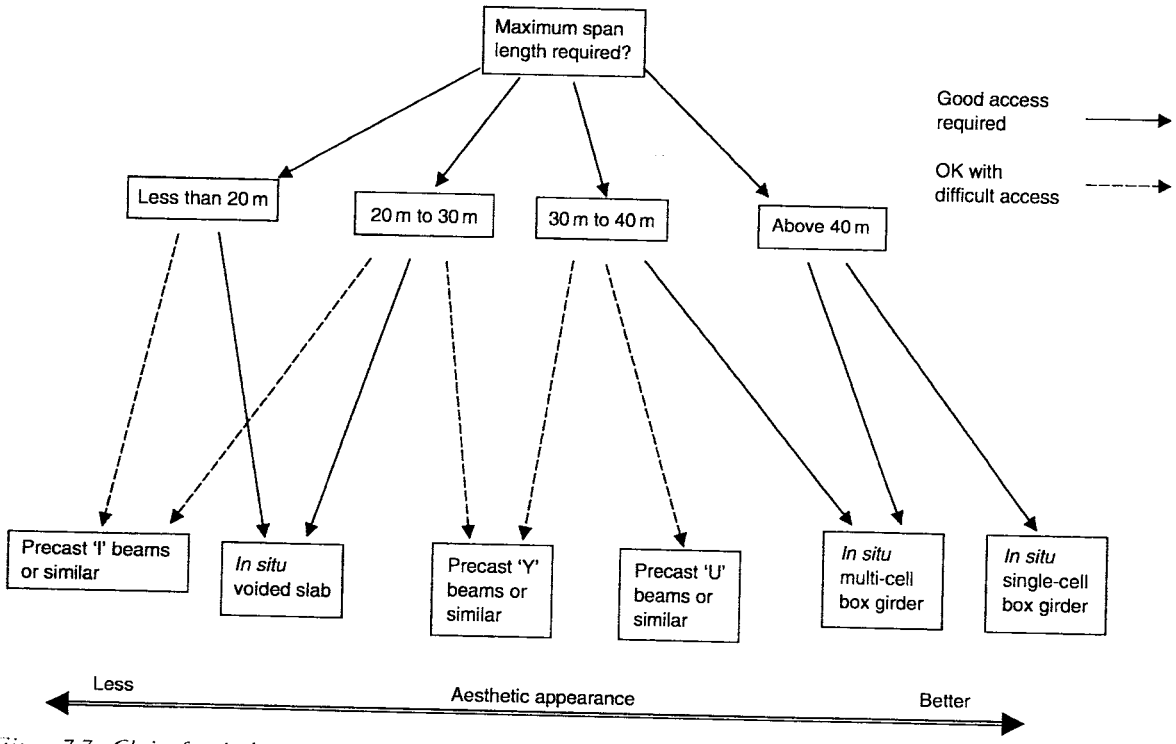


Figure 7.7. Choice for single-span decks.

@Seismicisolation

using precast beams or with box girders and Fig. 7.8 indicates the most common options used. Precast beams are generally used only for spans less than 40 m as above this their heavy weight requires specialist lifting equipment. Launched box girders offer good solutions where a deep valley or other obstruction is to be crossed while precast segments provide a versatile form of construction but usually need at least 2 km of deck to be economic.

When initially planning a structure and sizing up the concrete section, a good starting point is to select a suitable deck layout, cross-section and concrete thickness based on previous similar projects. These can then be developed to take into account the different loading, design codes and ground conditions of the particular site being considered. The examples in Chapters 9 to 17 may provide a good basis for selecting an initial arrangement.

For precast beams, the manufacturers usually provide guidance on the span range and beam spacing for the different types of beams available. For other types of deck construction there are many papers and articles in the professional journals and other publications available. The Internet is also building up a growing collection of papers and project reports which can provide useful data. Appendix C contains references to several useful websites and other sources of information on prestressed concrete bridges.

Aesthetics, durability and buildability all influence the deck arrangement and details adopted. Bridges should fit into their surroundings and present a pleasant appearance; with good practice presented in the publication by the Highway Agency (1996) [1]. The issues of durability are discussed in Chapter 3 and are often associated with particular details of a deck rather than affecting the overall choice of arrangement. Bridge decks often present

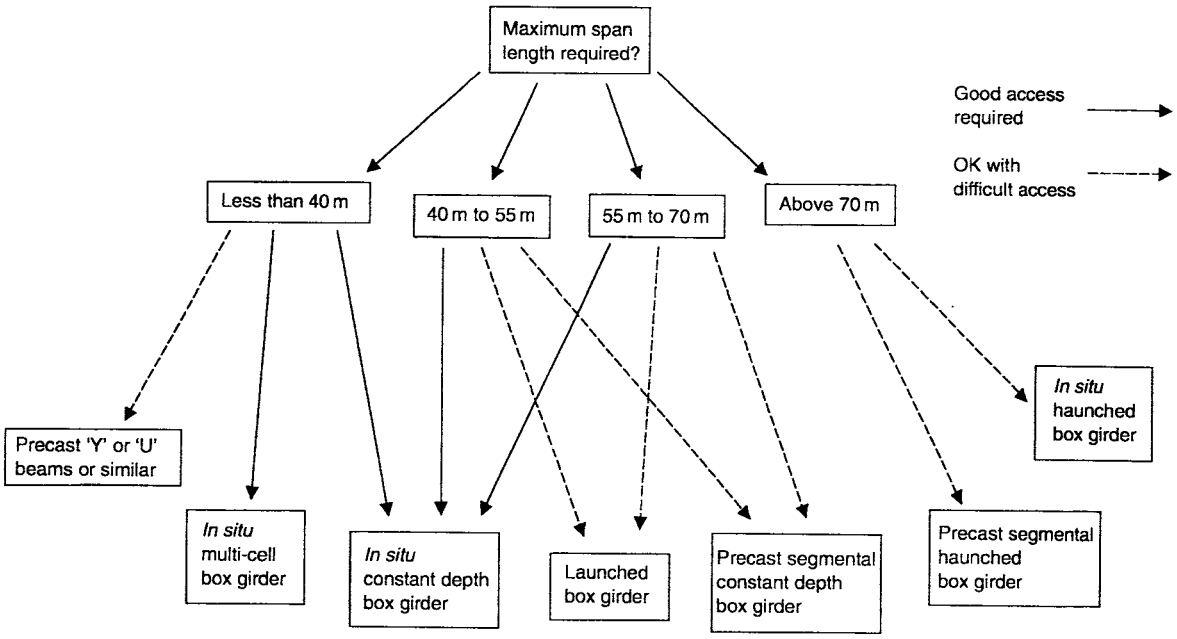


Figure 7.8. Choice for multi-span viaducts.

@Seismicisolation

challenges in their construction due to their locations and complexity, and buildability is often a governing factor in the choice of structure. Many of the issues of buildability are covered in the other chapters, while guidance is given in the CIRIA guide (1996) [2].

Articulation and span arrangements

Where bridges have an overall length less than 60 m it is becoming common to build the deck continuous and integral with the piers and abutments. This reduces the number of expansion joints and bearings, and it eliminates the maintenance problems that can occur with these elements. For longer structures the deck should be made continuous for as long as practically possible, which depends on the structural form. Concrete box girders have been constructed with continuous lengths up to 1·7 km and concrete cable-stayed bridges with decks up to 800 m in length between expansion joints.

Expansion joints in the deck should be positioned above a pier or at an abutment where the ends of the deck can be fully supported. The arrangement must be detailed to give good access to all parts of the expansion joint. Half joints in the deck should be avoided, if possible, unless there is sufficient space for access to the bearings and expansion joints for inspection, maintenance and replacement. Half joints, hinges or other discontinuities in the span should also be avoided for the reason that they allow long-term deflections from creep and prestress losses to adversely affect the profile of the deck.

The need for bearings between the deck and supports depends on the structural form and stiffness of the substructure. Launched box girder decks require bearings to facilitate the launch process, while precast segmental construction usually incorporates bearings to simplify erection. *In situ* concrete decks can be constructed on bearings or built integrally with the substructure provided the deck movements do not generate excessive forces in the piers and foundations.

Bridge decks with horizontal curvature give rise to articulation problems that need to be addressed in the design. A bridge deck will always try to expand or contract towards its effective 'fixed' point, and with curved decks this can set up unusual horizontal load distributions in the guided bearings if they are orientated parallel to the deck above. For this reason some designers prefer to align guided bearings towards the point of fixity; however, for long decks or tight curves this can cause problems due to the relative transverse movements that occur across the expansion joints.

Curved alignments can generate significant torsion and overturning effects in the deck structure. This requires the bearings to be offset transversely from the structure centreline or the deck tied down to counteract the effect.

Span arrangements are usually governed by the nature of the obstruction being bridged, but if possible the spans should be arranged to suit the type of deck being proposed. With precast beams it is preferable to keep a constant beam length throughout to standardize the construction equipment. For continuous box girder decks the piers should be arranged to give an end span that is approximately 70 per cent of the length of the adjacent internal span in order to create a balance between the hogging and sagging

@Seismicisolation

moments. Where a deck is built by the balanced cantilever method the length of end spans should be reduced to approximately 60 per cent of the internal spans to minimize out-of-balance moments on the end pier cantilever. If the end span of a continuous deck is less than the above recommendations there is a possibility that uplift will occur at the end bearings, in which case a tie-down arrangement will be required.

The minimum span length and height of deck above the ground is usually dictated by the clearance requirements of the road, rail or river passing underneath. A ratio of span length to deck height above the ground of approximately three is considered optimum to achieve a good visual balance, although this is not always practical with very high or very short piers. For shorter deck lengths where only a few spans are required a balanced arrangement of the crossing with either one, three or five spans is considered to give a better visual appearance than an even number of spans.

Guidance on the best arrangements from aesthetic considerations is given in the publication by the Highways Agency (1996) [1].

Post-tensioning with internal or external tendons

In the past, both internal and external post-tensioned tendons have suffered problems with their design, construction and durability. These problems have been well documented and current practices in detailing and construction have established a basis on which both systems can be used successfully. As part of the overall design concept a designer must choose between internal or external tendons, or a combination of both.

The advantages of external tendons over internal tendons are as follows:

- The casting of segments is simplified by eliminating the presence of ducts in the webs and flanges. The formwork, fixing of reinforcement and concreting are all made easier.
- Reduced deck web thickness due to the absence of ducts results in a saving on deck weight and substructure costs.
- Allowing the use of dry joints with precast segmental construction, where appropriate. This makes deck erection simpler and faster as well as reducing the cost of construction.
- Installation is made easier by having access to all parts of the duct. Placing of the strands is more straightforward and not prone to the blockages of the ducts that can be associated with internal tendons.
- Tendon layouts are simplified.
- Grouting of tendon is easier due to the better access available and because individual tendons can be grouted without the leakage or blockage problems that can occur with internal tendons in some forms of construction.
- Reduced friction losses within the tendon results in a higher effective force in the strand.
- It is easier to inspect tendons during construction and for long-term maintenance.
- The tendons are replaceable in the future.

@Seismicisolation

- Provisions to add tendons later to upgrade or strengthen the deck are easily incorporated.

The disadvantages of external tendons over internal tendons are as follows:

- They are protected by HDPE ducts which are fully cement grouted or wax filled, which result in a higher initial material cost for the prestress system.
- Reduced tendon eccentricity at critical points increases the prestress quantity required.
- Under ultimate bending conditions external tendons require more prestress to generate the same moment of resistance.
- They rely on anchorage integrity to maintain the prestressing force. If the anchor fails the tendon 'fails' along its complete length.
- Anchorage points and deviators are subjected to high concentrations of forces which need to be tied into the deck structure.
- Fatigue or fretting can be a problem if the free length of the tendon is too long, making the tendon susceptible to vibrations.
- The exposed tendons are more vulnerable to accidental damage.
- Better access is required to the tendons, deviators and anchorages for installation, future inspection and replacement of the tendons if necessary.

Internal and external tendons both produce durable and efficient structures and the choice is often influenced by local requirements and practices. Many recent bridges have used a combination of both internal and external tendons to combine the advantages of both systems.

Bridge costs

In general the shorter the span length the less cost per m^2 of deck. Prestressed concrete bridges are unlikely to be economic for spans less than 20 m where plain reinforced concrete structures would be preferred. Precast prestressed beams offer suitable economic solutions for shorter spans if access is limited, such as when crossing a river or railway. As the deck span gets longer the quantities of concrete, reinforcement and prestressing steel used per m^2 of deck increase causing a rise in the cost of construction. If significant temporary works are required this also increases the overall cost.

Where complex or expensive foundations are needed, such as over poor ground or to avoid obstructions below, the substructure costs become a higher proportion of the overall cost and adjust the balance in favour of longer spans. Other factors influencing the cost of a bridge include the location, ground conditions, design, labour costs, material costs and site restrictions.

In the UK a typical single-span highway overbridge or underpass costs approximately £850 per m^2 of deck (at time of writing) with the abutments making up a significant portion of the overall cost. For a typical three-span overbridge with spans up to 20 m the cost is approximately £700 per m^2 (at the time of writing), as the substructure costs are spread out over a greater deck area. As the span lengths increase the cost per m^2 of deck increases.

@Seismicisolation

A typical multi-span viaduct with 40 m spans could cost approximately £850 per m^2 of deck, while with 70 m spans the cost could increase to approximately £1200 per m^2. For cable-stayed bridges the costs can be over £2500 per m^2. Footbridges tend to cost more per m^2 due to the smaller deck area and relatively high abutment and support costs. (All prices are as at the time of writing.)

When developing the concept design for a bridge it is usual to carry out a cost comparison for different deck forms and span arrangements. Preliminary estimates of deck and substructure quantities and construction costs for the different options give an indication of the most cost-effective solution; although other considerations such as aesthetics and environmental issues may influence the final choice.

With long multi-span viaducts, the choice for the optimum span length to give the most cost-effective solution is a balance between the deck and substructure costs. With longer spans the deck quantities and cost increase, but, by reducing the number of piers, savings can be made in the substructure. Deck costs are usually directly proportional to the span length. The cost of the substructure, expressed as a cost per m^2 of deck, generally reduces as the span gets longer, but, where piles are used, steps in the cost occur as another pile is added to take the increase in load. The cost of the deck and the substructure can be plotted on to a graph, as illustrated in Fig. 7.9, thus enabling the optimum span length to be selected.

External tendons cost more per tonne than the equivalent internal tendons, but this is offset against savings elsewhere. With the tendons outside the concrete section the concrete section sizes, particularly the webs, are reduced in thickness. In addition reinforcement fixing and concrete placing

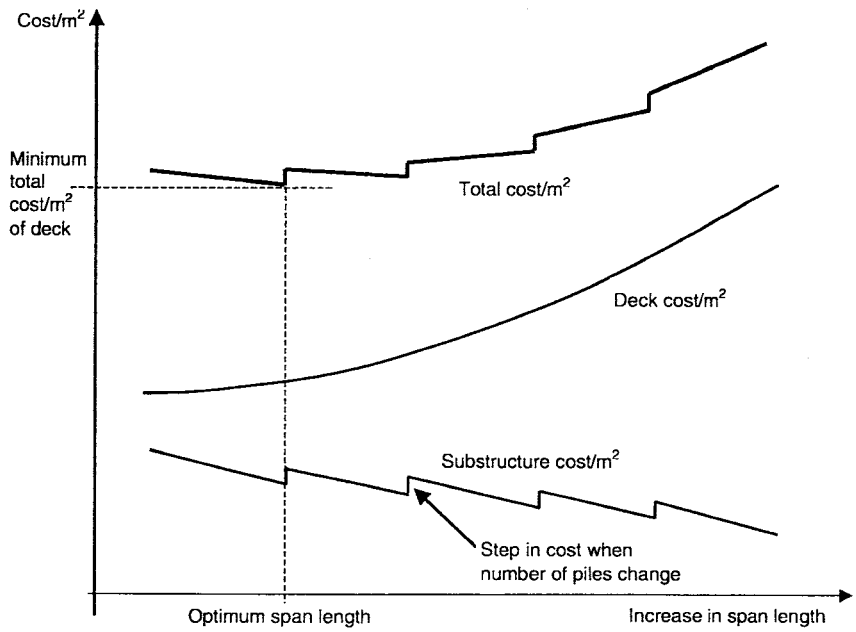


Figure 7.9. Optimizing bridge costs.

@Seismicisolation

are made easier. The use of external tendons with segmental construction in conjunction with span-by-span erection has resulted in rapid assembly of long viaducts, giving overall cost savings by reducing the construction period.

Material quantities

The quantity of concrete, reinforcement and prestressing used in a deck is normally in a reasonably well-defined band for given span lengths and deck types; although every bridge is different and the deck width, live loading and other aspects all influence the final quantities achieved. During the design process it is useful to compare the material quantities for the deck with those experienced on similar projects as an indication of the efficiency of the arrangement proposed. Figure 7.10 indicates the typical range of material quantities that can be expected in a deck for spans between 30 m and 150 m.

Precast segmental construction usually produces savings in concrete volume when compared with *in situ* construction, due to thinner sections being more practical to achieve in the factory casting conditions. Incrementally launched box girder decks usually have slightly higher concrete volume than the other forms of deck construction due to the design requirements of the launching.

The overall reinforcement quantity per m^2 of deck is significantly less in precast segmental construction and precast beams, when compared to other forms of construction, due to reduced longitudinal reinforcement as early thermal effects and differential shrinkage are less; whereas launched box

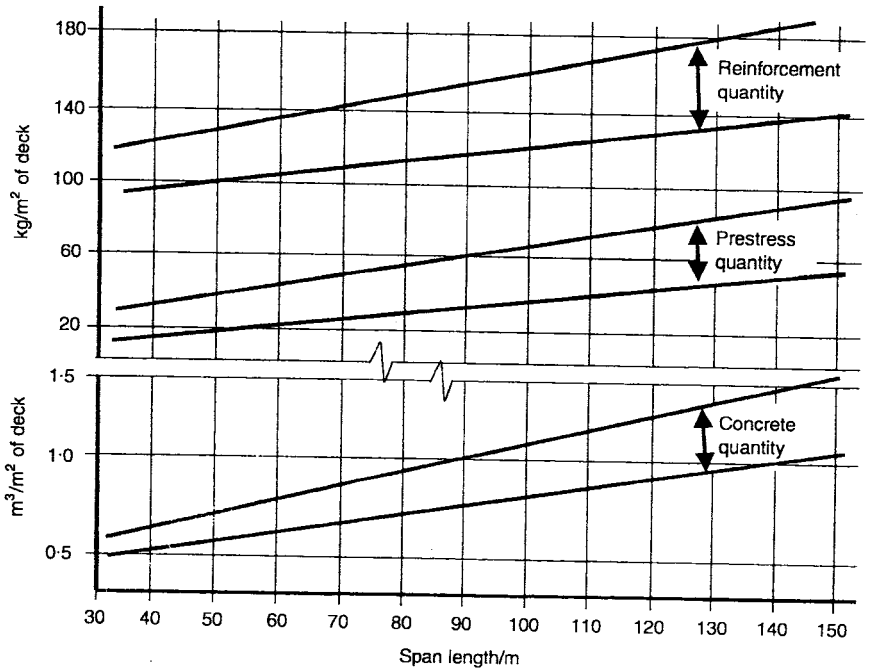


Figure 7.10. Typical quantities in prestressed concrete decks.

@Seismicisolation

girders have more reinforcement to cater for the temporary loading and structural requirements during the launch.

Prestressed beams usually require the minimum prestress quantities while precast segmental decks produce savings compared to *in situ* decks by reducing the creep in the deck and long-term losses in the tendons. Incrementally launched box girders need additional prestress to cater for the launch conditions. If external tendons are used instead of internal tendons the prestress quantity can increase due to a reduction in efficiency in resisting the bending effects, but this is offset to some extent by a reduction in friction losses.

The different design codes and loading criteria significantly affect the material quantities in a deck. Standard rail loading is heavier than highway loading, requiring more prestress while the lighter loads on footbridges reduce the quantities. Bridge decks designed using BS 5400 [3] tend to result in higher reinforcement and prestress quantities when compared with equivalent structures designed to AASHTO [4].

References

1. Highways Agency (1996) *The Appearance of Bridges and Other Highway Structures*. London: HMSO.
2. CIRIA Report 155 (1996) *Bridges – Design for Improved Buildability*. London: CIRIA.
3. BS 5400 (1990) *Steel, Concrete and Composite Bridges, Part 4. Code of Practice for the Design of Concrete Bridges*. London: British Standards Institution.
4. AASHTO (1996–2002) *Standard Specification for Highway Bridges, 16th edition*. Washington, DC: American Association of State Highway and Transportation Officials.

@Seismicisolation

8

Analysis of prestressed concrete bridges

Introduction

When analysing prestressed concrete bridges the different aspects discussed in Chapter 5 must all be taken into account. The self-weight, applied loading and prestress have to be combined in the overall structural behaviour and the time-dependent effects, such as creep, shrinkage and loss of prestress force, included. Traditionally these effects are considered separately and then combined to give the overall design requirements although, in recent years, computer software packages have been developed to combine the effects within a single analysis.

The principles behind structural analysis and their application in bridge design are well documented and fully covered in numerous publications, such as Shanmugam and Narayanan (2000: Chapter 3) [1] and Hambly (1991) [2]. The following sections review the way that prestressed concrete bridge decks are usually analysed and the practical problems that may be encountered.

This chapter begins by describing the traditional approach to analysing prestressed concrete bridge decks, and it goes on to review one of the specialist software packages available to show how the approach to analysis is evolving.

Traditional approach to deriving forces, moments and shears

It has been traditional in bridge design to consider longitudinal and transverse designs, different loading conditions and long-term effects separately. They are then combined in order to check the design requirements. Forces and moments are derived from analysis using either two- or three-dimensional line beam or grillage models. Applied loading is directly applied within the model with prestress effects generated by using equivalent loads or calculated independently.

In recent years the availability of three-dimensional finite-element software packages has enabled the longitudinal and transverse effects to be determined within a single analysis model; although it is still usual for the design in the two directions to be carried out separately and the requirements combined on the drawings.

@Seismicisolation

Dead load and applied loading analysis

To analyse multiple beam structures a grillage model is established to derive the distribution of moments and shears under the different applied loading conditions. The deck is modelled as a series of discrete members both longitudinally and transversely, as illustrated in Fig. 8.1. The moments and shears on each member are extracted from the grillage model and fed into the design process.

Longitudinally it is simplest to provide a grillage member along the centre line of each beam or web. The section properties of each member should include a portion of the top slab up to midway between adjacent beams or webs and, similarly, a portion of the bottom slab where 'U' beams or box-beams are used. With box structures represented by individual grillage members for each web, the torsional stiffness of the box is calculated and a quarter of this is assigned to each web with the remaining half of the torsional stiffness assigned to the transverse members.

Transversely, grillage members are positioned at each diaphragm location with section properties to match the diaphragm beam and at regular intervals between, with section properties to match the length of top and bottom slabs represented. The location of the transverse members should give a relative spacing of longitudinal beams to transverse beams of between 1 : 1 and 1 : 2 to achieve reasonably accurate structural behaviour.

For single-beam-type structures the force and moment distributions in the longitudinal direction are derived from a line beam analysis. Figure 8.2 illustrates a typical two-dimensional longitudinal analysis showing the dead load moments for a three-span frame structure. From this type of analysis the bending moments, shears and axial loads in each of the members are derived for the dead load, superimposed dead load and live load, and input into the design process. Differential settlement of the supports is modelled by varying the support levels, and temperature variations are included by applying strain variations on to the model. Torsional effects are calculated manually using simple beam theory.

The transverse analysis for box-type structures is carried out on a two-dimensional frame model, with a typical output for dead-load moment illustrated in Fig. 8.3. By placing supports under the webs and applying the different loadings imposed on the section the forces and moments around the box are obtained. Supports to the box section vary from 'rigid', where the webs sit directly on the bearings, to more flexible at mid-span, where the webs can deflect relatively to each other and are supported by the longitudinal structural action of the deck.

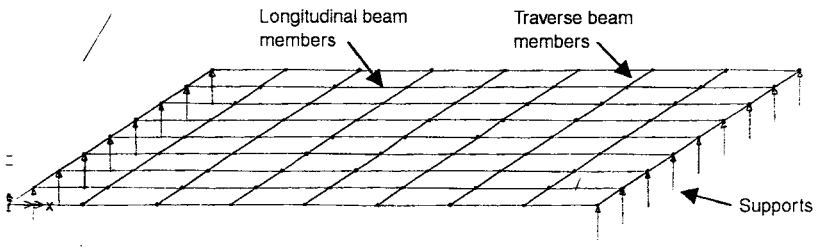


Figure 8.1. Grillage model for analysis.

@Seismicisolation

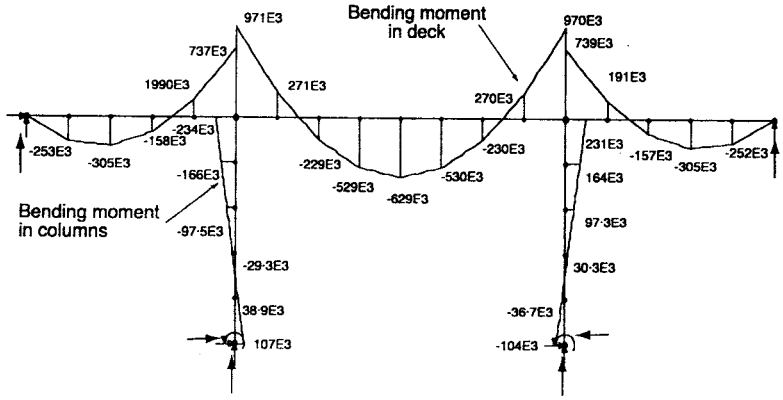


Figure 8.2. Longitudinal dead load moment from frame analysis.

This difference in effective support stiffness results in variations to the distribution of forces and moments around the box. To represent this behaviour the analysis is carried out with rigid supports and again with 'spring' supports; with the worst case taken at each section around the box. The value of the spring support is derived from the longitudinal analysis model, and it is a function of the longitudinal deflection of the beam under a unit load.

Modern computer capacity and speed have made the use of three-dimensional finite element analysis software more common when designing structures. A single model, as illustrated in Fig. 8.4, can be used to analyse the complete structure and derive both the longitudinal and transverse effects. The advantage of a full finite element analysis is that the distribution of the load in both the longitudinal and transverse directions is more accurately modelled, resulting in a more refined and economic design. With this type of analysis the stresses or forces in the elements in all three directions are obtained directly. The stresses are input into the design to determine the prestress requirements, and the shears and moments are used to check the reinforcement requirements.

With grillage and line-beam analysis, effects such as shear lag and warping of the section are not included; however, a full-length three-dimensional finite element model includes these effects automatically in the overall structural behaviour.

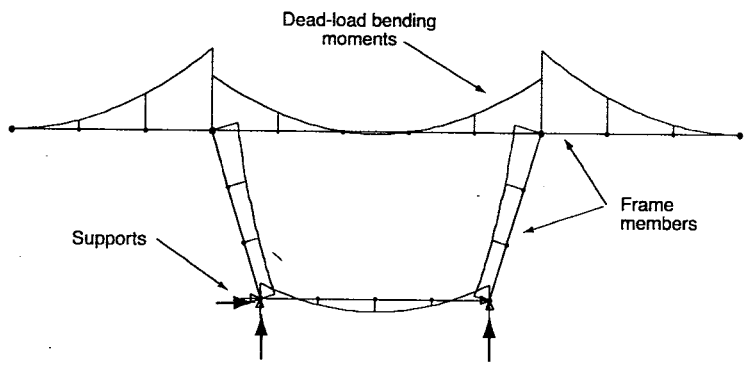


Figure 8.3. Frame model for transverse moment analysis.

@Seismicisolation

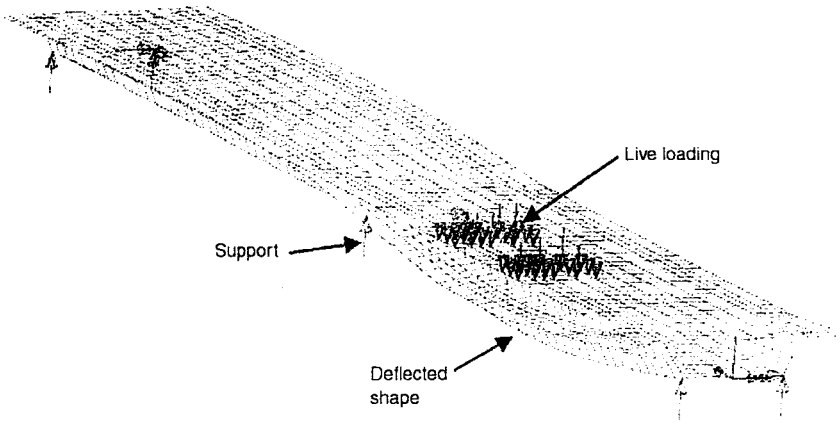


Figure 8.4. Full length three-dimensional finite element model.

Deriving the prestress forces and moments

Design of the prestress is an iterative process that involves balancing the stresses generated by the self-weight and applied loading with those from the prestress. The secondary moments are usually a significant part of the prestress effect and the design process involves estimating the prestress required and then determining the primary and secondary effects so that these can be used to check the resulting stress levels. If these stresses are outside the allowable limits, further adjustments are made to the prestress arrangement until the final stresses are acceptable. It is important that the modelling and analysis of the prestress effects are set up to easily incorporate any subsequent adjustments to the prestress layout that may be necessary.

There are two stages in deriving the prestress effects. The first is to calculate the force in each of the tendons and the second is to establish the primary and secondary effects.

There are proprietary computer programs available that determine the force profile along a tendon or, alternatively, a simple spreadsheet can be established using the formula in Chapter 5, 'Friction losses and tendon extension'. The spreadsheet in Fig. 8.5 illustrates the force profile for a single end stressed tendon during stressing, and after lock-off. The expected extension during stressing is also calculated. The force profiles for each tendon are combined to give the total prestress axial force and primary moment at any section along the deck.

The secondary effects can be derived using a number of different techniques, the most common being the equivalent load method where the forces from the tendons are applied directly to a grillage or simple line beam model as illustrated in Fig. 5.5(ii). The equivalent load approach is described in papers by Aalami (1990) [3] and Catchick (1978) [4].

The geometry of the model must closely match the structure's layout, and the prestress load precisely applied to ensure that the structural behaviour is accurately represented. If the tendon is inclined at the anchor, both a vertical and horizontal load is applied to the structure as well as the moment generated when the tendon is offset from the neutral axis. At each change in angle of the

@Seismicisolation

SPREADSHEET FOR CALCULATION FOR POST-TENSIONING FORCE IN TENDONS

TENDON REF. 1

STRESSING END NODE No. = 1

Po = 202.5 kN

μ = 0.2

k = 0.00700

Area = 150 mm^2

Anchor Pull-in = 6 mm

E = 195000 N/mm^2

Node No.	Distance (m)	Vertical angle	Horiz. angle	Total angle(rad)	Force Px (kN)	Draw-in Dist. (mm)	Extension (mm)		Distance (m)	Reverse force	Total force after lock-off
1	0	0	0	0.0000	203	0.00	0.0		0.00	172	172
2	1	0	0	0.0000	202	0.010	6.9		1.0000	172	172
3	2	0	2	0.0349	201	0.183	6.9		2.0000	174	174
4	3	0	2	0.0698	199	0.469	6.8		3.0000	175	175
5	8	10.5	0	0.2532	190	3.664	33.3		8.0000	184	184
6	13	7	0	0.3754	184	7.895	32.0	187	10.7605	187	184
7	18	0	0	0.3754	183		31.4				183
8	20	0	0	0.3754	183		12.5				183
9	24	0	0	0.3754	182		24.9				182
10	28	3.5	0	0.4365	178		24.6				178
11	30	0	0	0.4365	178		12.2				178
12	33	0	2.2	0.4749	176		18.1				176
13	35	0	2.2	0.5133	174		12.0				174
14	38	6.8	0	0.6321	169		17.6				169
15	43	0	0	0.6321	168		28.8				168
16	48	0	0	0.6321	167		28.6				167
17	57.5	0	0	0.6321	165		53.8				165

TOTAL EXTENSION 351 mm

Lock-off profile

Maximum force in tendon

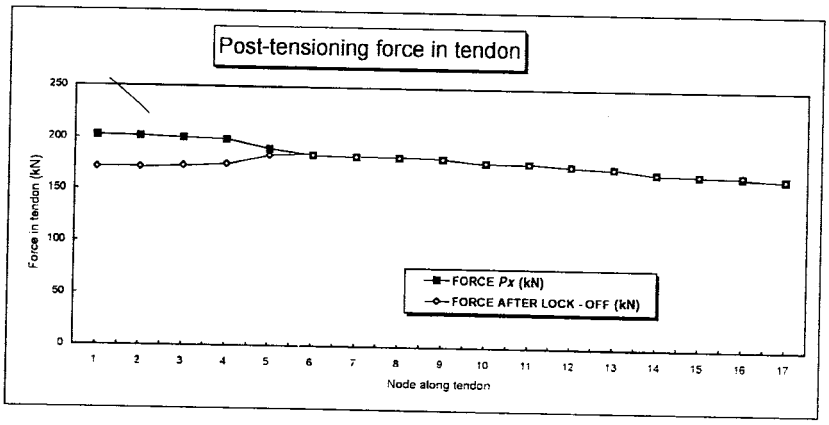


Figure 8.5. Tendon friction-loss spreadsheet.

@Seismicisolation

tendon an equivalent load equal to the force in the tendon multiplied by the angle change (in radians) is imposed on the structure. For a curved tendon the equivalent radial force per metre along the curve is equal to the force in the tendon divided by the radius of the curve.

The member layout in the model must accurately follow the neutral axis of the structure to ensure that the change in prestress primary moment is generated when the deck section changes. Friction losses in the tendons are incorporated by applying 'equivalent' forces and moments on the members along the model.

The advantages of the equivalent force method are that it is suitable for a wide range of structural arrangements and that it gives the combined primary and secondary effects. The disadvantages are that it is not always straightforward to establish accurately the analysis model and equivalent loads, and modelling the prestress losses can be complex.

An alternative approach to deriving the secondary effects is the influence or flexibility coefficient method, which readily takes into account changes in section and prestress losses. With a prestress layout as shown in Fig. 8.6(i), and by using the theory of least work, the following equations can be established:

$$f_{11}X_1 + f_{12}X_2 = -U_1$$

$$f_{21}X_1 + f_{22}X_2 = -U_2$$

where

$$f_{11} = \int_s \frac{m_1^2}{EI}\,ds$$

$$f_{12} = \int_s \frac{m_1 m_2}{EI}\,ds = f_{21}$$

$$f_{22} = \int_s \frac{m_2^2}{EI}\,ds$$

$$U_1 = \int_s \frac{m_1 M_p}{EI}\,ds$$

$$U_2 = \int_s \frac{m_2 M_p}{EI}\,ds$$

In Fig. 8.6(ii) M_p is the free moment generated by the prestress on the beam. The moments for unit release at the intermediate supports are m_1 and m_2. If the deck is considered as a series of individual simply supported spans, when M_p is applied the beams deflect causing relative rotations on either side of the supports. The moments required at the ends of the spans to counter the relative rotations are the secondary moments in the beam and are derived from the above equations.

By solving the equations X_1 and X_2 are determined: which are the secondary moments generated at their respective support. This approach can be extended to derive secondary moments for structures with many

@Seismicisolation

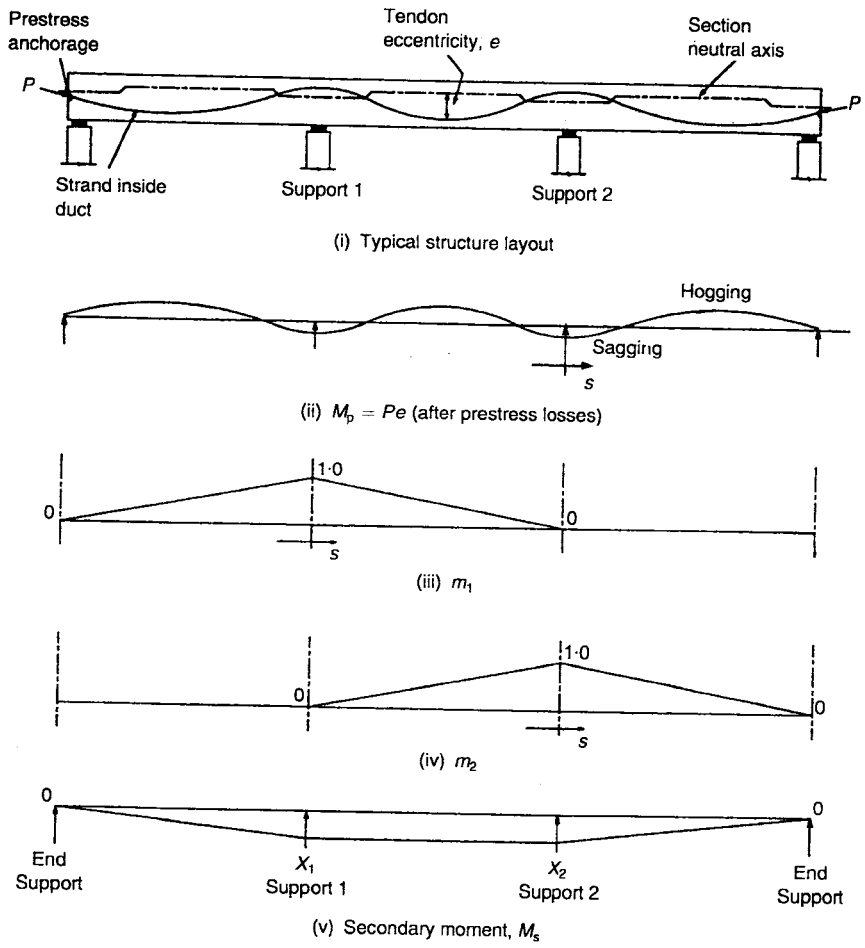


Figure 8.6. Prestress moments from influence coefficients.

spans and is simplified by the use of spreadsheets. The 'secondary' shear forces and reactions at the supports are determined by considering the changes in the secondary moments along the structure.

After the derivation of both primary and secondary effects they are combined to obtain the total prestress effect. These are initially calculated with no long-term losses in the prestressing force. The losses due to elastic shortening, relaxation, creep and shrinkage are taken into account by estimating the percentage loss and reducing the primary and secondary effects accordingly.

Stage-by-stage and creep effects

Prestressed concrete bridges are usually built in stages with the resulting stage-by-stage forces and moments being locked into the structure and subsequently modified by creep behaviour, as discussed in 'Construction sequence and creep analysis', Chapter 5. Both dead-load and prestress moments and forces are affected by the stage-by-stage build-up of the structure.

@Seismicisolation

Traditionally, each stage of the construction is modelled and analysed separately, using the techniques described above, to give the dead-load and prestress effects for that stage. The results from each stage analysis are combined with the results from all the previous stages to give the total moment and force distribution in the structure at the stage being considered. The moments, shears and prestress force from earlier stages are modified to take into account the long-term losses in the prestress and the creep redistribution of moments and shears.

In the final condition the 'instantaneous' dead-load and prestress secondary moments are calculated for the completed structure after all long-term prestress losses have occurred, and these values used in the creep assessment to determine the final design.

Combining effects

Once all the moments and forces have been derived for dead load, imposed loads and the prestress effects they are combined to enable the design to be checked for the serviceability and ultimate-limit state requirements.

When using the traditional design approach, as described above, the effects are usually combined using spreadsheets to organize the data and output from each of the different analyses carried out at each stage of the construction.

Using standard structural analysis software and the traditional approach is time-consuming and complex, although it does give the designer a good understanding of the structural behaviour under the different effects. An alternative is to use specialist software as described below.

Specialist software for the analysis of prestressed concrete bridges

Many firms have developed their own specialist in-house software over the years to deal with analysing prestress effects, but frequently these only cater for some of the effects and the designer is left to combine the results from other parts of the design process. Several proprietary software packages are now available which will combine most of the aspects to be considered when designing prestressed concrete bridges. The following sections show how these proprietary packages can be utilized. The software program *ADAPT-ABI* [5] is used as an example to demonstrate their versatility. *ADAPT-ABI* is currently a two-dimensional analysis but three-dimensional software is becoming more widely available.

General description

The main characteristic of specialist software programs like *ADAPT-ABI* is the ability to combine the basic structural analysis with the time-dependent and load-history analysis that is crucial in the design of prestressed concrete bridges. Many programs model each individual prestress tendon and will analysis the construction of composite structures with precast and *in situ* members. They also include the segmental and stage-by-stage construction of both pre-tensioned and post-tensioned concrete decks.

@Seismicisolation

Bridges that are built by the balanced cantilever method, launched into position, built span by span or cabled-stayed can all be analysed at each stage in their construction with the prestress losses, creep, shrinkage, concrete aging and creep recovery taken into account for the time that has elapsed since the start of construction. It is possible to model the travelling forms, launching nose or other temporary works in the construction sequence to closely match the actual conditions that will be experienced on site.

By taking all the different aspects into consideration, and using the ability of the computer to handle large quantities of information and complex structural behaviour, it is possible with programs like *ADAPT-ABI* to analyse a bridge in greater detail. More accurate modelling of the concrete behaviour is possible than when using the traditional techniques discussed above. The rapid analysis provided by specialist software allows the designer to easily look at the sensitivity of the design to changes in the different parameters, which is useful during both the design and construction phases of a project.

The structure is usually modelled as a system of frame or finite-element members connected at nodes with supports and other restraints imposed as appropriate. Prestressing tendon elements and stay cable elements are connected to the frame elements at the nodes. Time is divided into a number of steps to reflect the construction sequence and schedule. The program models the build up of the structure and calculates the structural response at the end of each time step, or stage, incorporating the changes that have occurred since the previous step.

In common with other similar software programs, *ADAPT-ABI* has a series of modules that make up a complete suite for carrying out the different phases of the analysis. The *Basic* module allows the structure to be defined and analysed, while *ABI-Gen* enables the structure to be broken down into groups of spans or columns to assist in the design process. *ABI-Gen* also allows the application of different load patterns and provides load combinations and envelopes for input into the design. The moving load module permits a user-defined train of loads to be run across the structure to extract the maximum effects or to generate influence lines. Forces and extensions of the prestress tendons are extracted from the analysis with the friction and elongation module.

ADAPT-ABI input and output data and the graphics interface are all accessed through *ABI-Shell*, which integrates the different operations into a single menu, as shown in Fig. 8.7.

The menu gives access to create and edit the different input files and to run the different stages of the analysis. It also gives access to the graphical display where the user can view the input model and the different parts of the output.

The output from *ADAPT-ABI* combines the self-weight and applied loading with the prestress; including concrete creep and shrinkage effects, prestress losses and the stage-by-stage build-up of forces and moments. The moments, forces, stresses or deflections are presented in graphical format or in a text file ready for use in the design.

@Seismicisolation

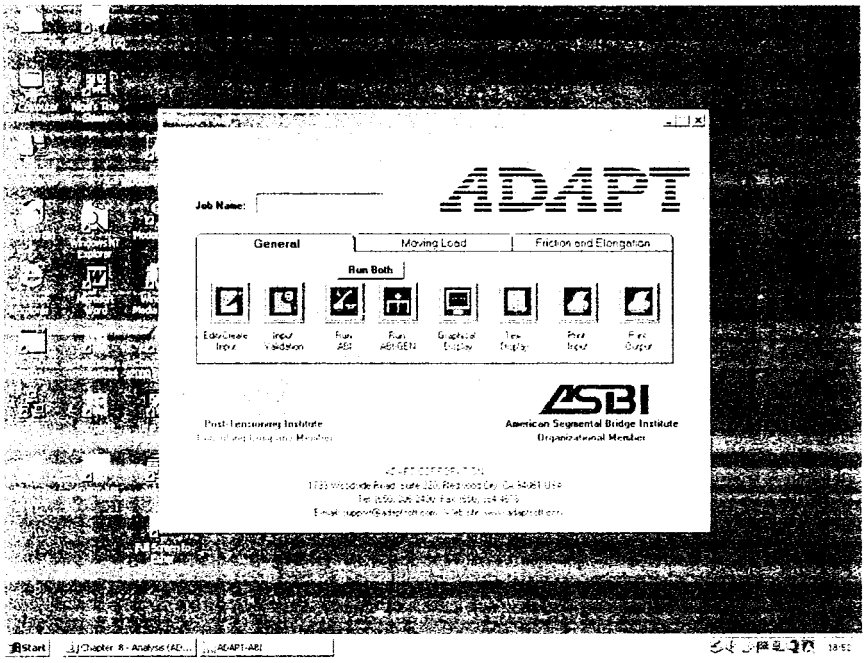


Figure 8.7. ADAPT *menu window. Reproduced with permission.*

Input data

The input data for the *Basic ADAPT-ABI* module is arranged in a series of sections as follows:

TITLE
UNITS
ACTIVATE EXTRACT
CONCRETE PARAMETERS

MESH INPUT
- NODES
- SEQUENCE
- CONCRETE PROPERTIES
- MILD STEEL PROPERTIES
- OFFSET DATA
- ELEMENTS
  - FRAME
  - SPRINGS
- PRESTRESSING STEEL
- TENDON GEOMETRY
- STAY ANALYSIS
- TRAVELERS

MESH COMPLETE

SET

@Seismicisolation

```
CHANGE STRUCTURE
        BUILD (Frame elements)
        RESTRAINTS

        REMOVE (Elements)
        STRESS (Tendons)
        DE-STRESS (Tendons)
        MOVE (Traveler)
CHANGE COMPLETE

LOADING

SOLVE Day =
OUTPUT
CAMBER

STOP
```

Many of these are self-explanatory and demonstrate the different factors that are taken into account in the analysis.

The CONCRETE PARAMETERS define the E-value, shrinkage and creep of the concrete and the change to these over time. It is possible to call up predetermined parameters that reflect the BS 5400 or ACI codes or to define the parameters to suit the user's requirements. With *ADAPT-ABI* the analysis is able to incorporate a more refined and accurate model for the time-dependent behaviour of the concrete than is possible by hand calculations.

CONCRETE PROPERTIES describe the compressive strength, unit weight and thermal expansion co-efficient of the different concrete mixes being used.

By defining the MILD STEEL PROPERTIES the reinforcement present in the concrete is included in the structural analysis.

OFFSET DATA allows the structural members to be offset from the defined node. This allows a single node to be used to represent the ends of a number of elements which are at different levels in the structure. This simplifies the input for composite structures such as beam-and-slab decks which have elements at different levels and which have been built at different times.

The ELEMENTS are either frame or spring elements. The frame elements are defined by their area, moment of inertia, concrete type and the distance from their centroid to their extreme fibres. The spring elements are fixed between two nodes and have stiffness along their length, perpendicular to their axis and rotational at the nodes.

PRESTRESSING STEEL allows the properties of the prestressing strand or bars to be specified. The tendons' modulus, ultimate stress, friction, relaxation coefficients and their coefficient of thermal expansion are all defined.

In the TENDON GEOMETRY section the profile of each individual tendon is defined along with the tendon area. The tendon profile either is input as a series of straight lines between the structure nodes and offset from them, or it is generated as a parabolic profile between defined points. The tendons are utilized as either internal or external prestress by selecting appropriate properties and suitably installing them into the structure.

[redacted]
@Seismicisolation

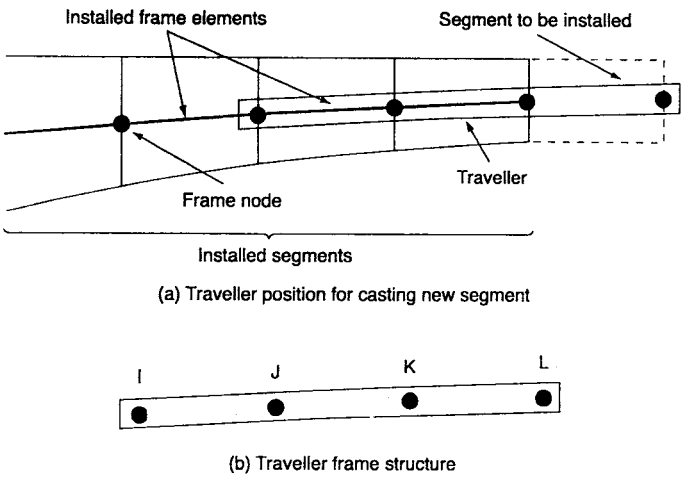


Figure 8.8. ADAPT *traveller definition. Reproduced with permission.*

The STAY ANALYSIS command allows the analysis to include the non-linearity of the behaviour of stay cables, which are defined as similar to tendons but identified as stays.

TRAVELER is a generic term within *ADAPT-ABI* used for an auxiliary structure employed during the construction of the bridge. Form travellers and other temporary works are defined and used in the stage-by-stage construction. A traveller is modelled as a frame structure, connected to the previously completed deck, as illustrated in Fig. 8.8. The weight and stiffness of the traveller is taken into account so that the temporary load is applied to the structure while the traveller supports the 'wet' concrete. The program allows the traveller to be moved at different stages in the construction and removed at the end of its use.

The elements, prestress and temporary works having been defined, the structure is built in a series of stages in a manner similar to that actually carried out on site. The CHANGE STRUCTURE section of the input allows individual elements to be 'built' and removed later if necessary. Each tendon is installed and stressed, re-stressed or de-stressed as required, while a form traveller can be placed, moved or removed as the construction proceeds.

LOADING allows external loads to be applied to the structure at any stage in the construction as well as on the completed bridge. The point loads and uniformly distributed loads are applied or removed to represent temporary construction loading or the superimposed dead load and live loading on the final structure.

The SOLVE command is set at specific stages and dates in the construction sequence, and it causes the program to update the analysis incorporating any changes that have occurred since the last SOLVE command. This command is also used to progress the time within the analysis, with 'Day =' defining the day, or age, at which the analysis is carried out.

When OUTPUT is included after any SOLVE command the program records the forces, stresses, tendon forces, displacement and all the other

@Seismicisolation

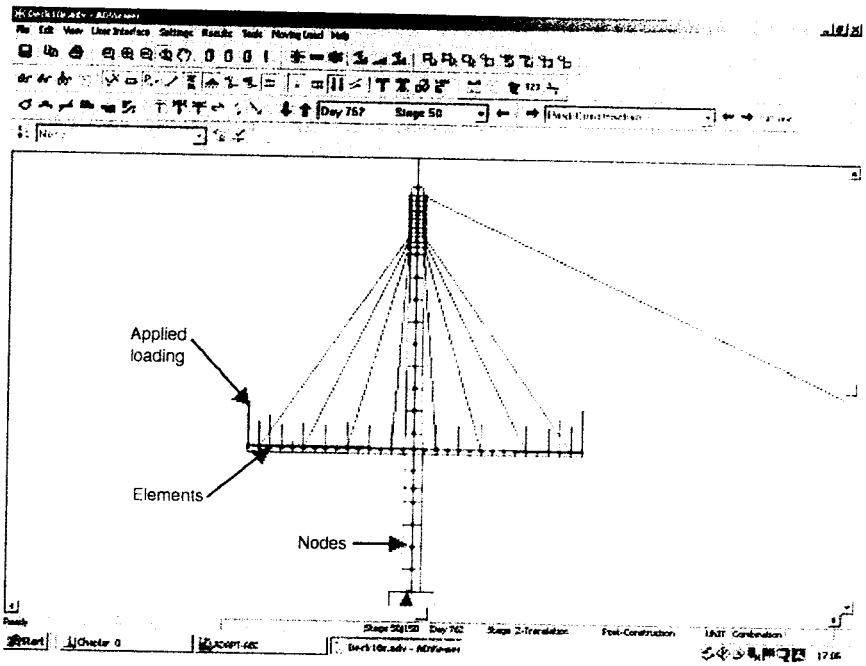


Figure 8.9. Graphical representation of the ADAPT *model. Reproduced with permission.*

output data at that stage for inclusion into an output text file or for use in the graphical presentation of the results.

CAMBER calculates the precamber required to be applied to the elements during construction to give the required profile of the structure at the specified date.

When all the input has been completed the program runs a check on the data and gives a graphical representation of the structure's arrangement and the applied loads, as illustrated in Fig. 8.9. From this the model can be quickly checked to ensure that it is built up in the way intended and includes all the loads to be taken into account.

Analysis of the structure

The structure is analysed in a stage-by-stage process with the program building up the structure in the defined sequence and time. Each element is built or installed, and the tendons are stressed in the same order and period as expected on site. At the beginning of each time-step the stress, strain and displacement within the structure is known and this equilibrium state is used as the starting point for carrying out the analysis of the next time-step. The changes in the structure that occur during the next step are then superimposed on to the structure with the time-lapse taken into account when the time dependent effects of the concrete and prestress losses are included.

Figure 8.10 illustrates three consecutive stages during the construction of a launched box girder deck. The concrete segment is first cast, followed by installation of the prestressing tendons and then launching of the deck into position.

@Seismicisolation

Because of the non-linearity of the time-dependent solutions, an iterative procedure is adopted in finding the solution to each time-step, with convergence criteria set to achieve sufficient accuracy. At the end of the time-step the new stress, strain and displacement distribution are stored ready for use in the next step of the analysis. The concrete parameters are also updated, including the current 'E' value and residual creep, shrinkage and relaxation.

For composite construction, such as a precast beam with *in situ* deck slab, the section is modelled as two or more parallel elements positioned at their centroid. In the analysis, the separate elements act together compositely, being held together at the nodes. The redistribution of forces and moments due to differential shrinkage and creep between the two elements are automatically taken into account at each time step.

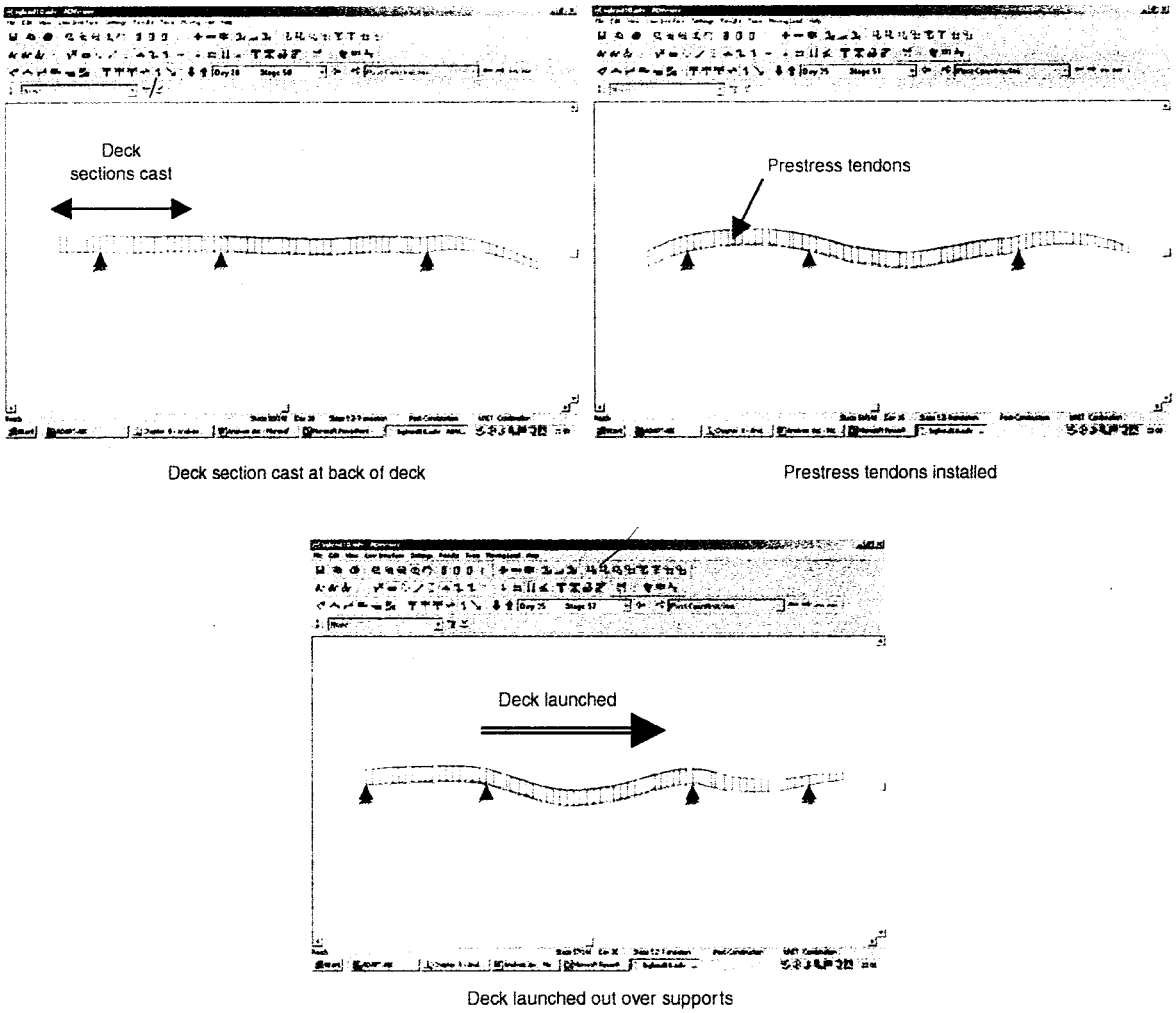


Figure 8.10. Stage-by-stage construction by ADAPT. *Reproduced with permission.*

@Seismicisolation

The boundary conditions at any node can be defined or changed at any time during the step-by-step analysis. Displacements can be applied to the structure to match those that might occur on site. In this way, the supports and restraints to the structure are installed, removed or adjusted as the construction proceeds.

The differential temperature effects on the structure are analysed by defining the ambient temperature and temperature gradient across each element. The expansion or contraction and bending of the element is then imposed onto the structural model.

When TRAVELERS is used in the model, its self-weight and the weight of the new element being lifted or supported are carried by the traveller back to its point of fixitation on the completed structure. When the traveller is moved it releases the weight of the new element, which is then carried through its own stiffness back to the rest of the structure. The weight of the traveller is also removed from the structure and placed at its new location. Figure 8.11

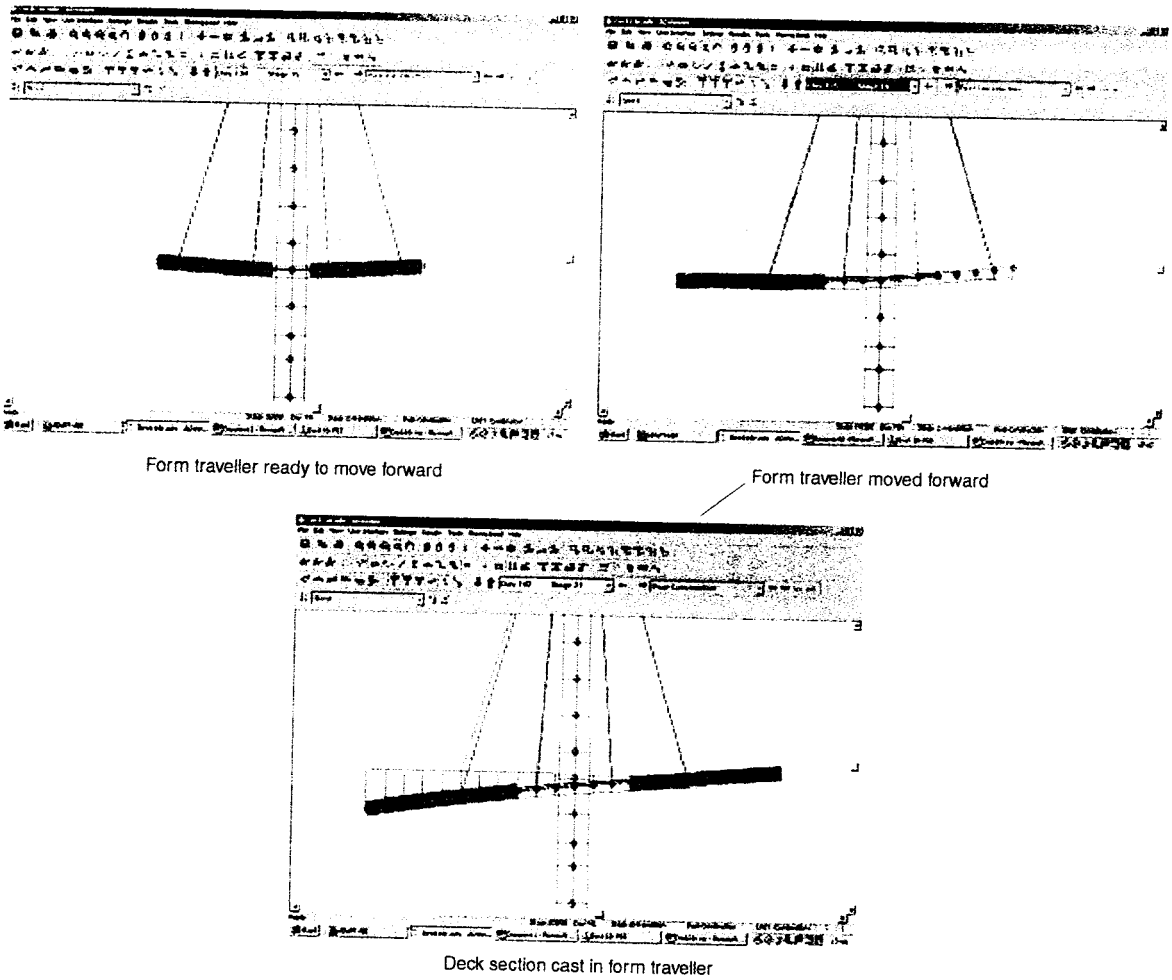


Figure 8.11. Stage-by-stage construction with a traveller. Reproduced with permission.

@Seismicisolation

illustrates the affect of the traveller on the deflections of a deck section. The weight and stiffness of the traveller has a significant influence on the final deflected shape of the bridge.

The program takes into account the friction losses and anchor lock-off effect when calculating the prestress force at the time of installing the tendon. When subsequent tendons are stressed the forces in the previously stressed tendons are adjusted due to the elastic deformations within the structure. Subsequent losses in the tendon force due to creep and shrinkage of the concrete are automatically incorporated while the relaxation within the tendon over the time-step elapsed is calculated within the analysis and imposed onto the structural system.

The analysis of cable-stayed bridges takes into account the effect of catenary sag on the stays' behaviour. With long stays or stays stressed to a low force there are significant sags in their profile, which reduces their effective 'E'

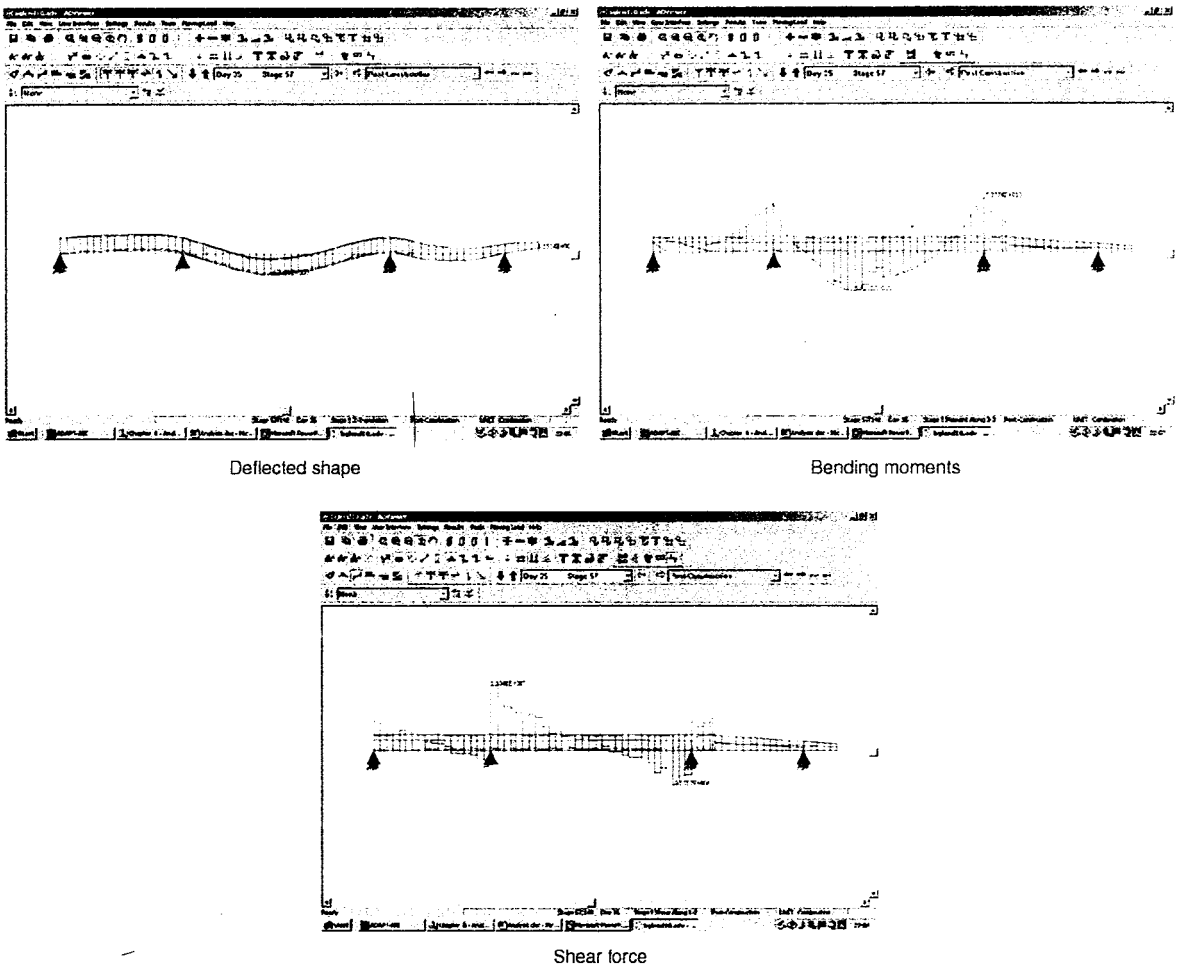


Figure 8.12. ADAPT *output. Reproduced with permission.*

@Seismicisolation

value in the structural model. This effective 'E' value changes depending on the force in the stay.

Output

ADAPT-ABI prepares the output in both text and graphical format. At every step in the analysis it is possible to output the nodal displacements, element forces, support reactions and spring forces, element stresses and tendon forces. It is also possible to output the required camber to which the structure must be cast so that the structure achieves its required profile at the end of the construction. Typical examples of the graphical output showing the deflected shape, bending moments and shear forces are given in Fig. 8.12.

The *ABI-Gen* module can be used to split the structure into groups of elements and to create load combinations and load patterns which simplifies the post-processing work needed. The *ABI-Moving Load* module allows the force and moment envelope and influence lines to be produced thus identifying the critical loading conditions.

Summary

Proprietary software, such as *ADAPT-ABI* bring together all the different aspects that are considered in the analysis of prestress concrete bridges and simplify the design process. This has also made the analysis and design more accurate when the time dependent effects such as creep, shrinkage and prestress loss are considered; previously this had only been an informed estimate at best.

References

1. Shanmugan, N.E. and Narayanan, R. (2000) *Manual of Bridge Engineering*, Chapter 3, 'Structural analysis'. London: Thomas Telford.
2. Hambly, E.C. (1991) *Bridge Deck Analysis*. 2nd edition. London: Chapman and Hall Ltd.
3. Aalami, B.O. (1990) 'Load Balancing: A Comprehensive Solution to Post-tensioning', *ACI Structural Journal*, Nov.–Dec.
4. Catchick, B.K. (1978) 'Prestress analysis for continuous beams: some developments in the equivalent load method', *The Structural Engineer*, **2, 56B:** June 29–36.
5. ADAPT Corporation, *ADAPT Structural Concrete Software System*. www.adapt.com

@Seismicisolation

9

Slab bridges

Introduction

Solid or voided prestressed concrete slabs are used for short spans and where good access is available for their construction. Using slabs simplifies the formwork and concreting operations, although the decks are relatively heavy and this makes prestressing inefficient. Slab decks were regularly used in the early period of prestressed bridges and are still used on appropriate projects. Figure 9.1 showing the Balbriggan Bridge in Ireland is a recent example where this type of deck has been used.

Solid slabs are used only for short spans. With spans above 15 m the deck is likely to be voided to reduce the deck section and dead load. For short span lengths either plain reinforced concrete decks or prestressed beam-and-slab arrangements usually result in a more efficient and economic solution. For these reasons prestressed solid and voided slabs are not widely used in bridgeworks. However, prestressing of the concrete improves the structure's durability compared with a reinforced concrete design and many consider a slab arrangement to be better aesthetically compared with a beam-and-slab deck. Figure 9.2, showing an overbridge on the M4 in Ireland, illustrates a

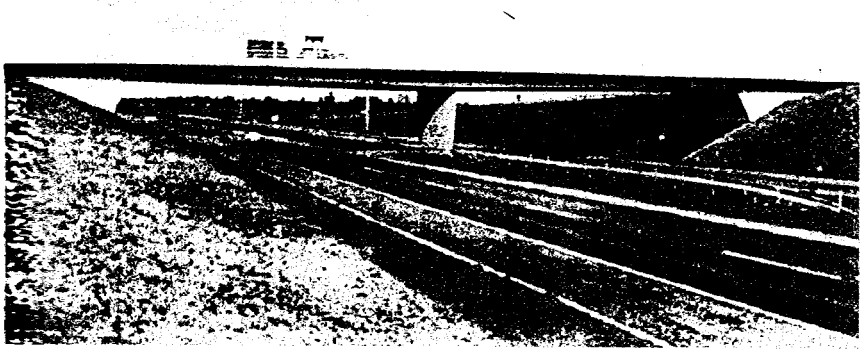


Figure 9.1. Balbriggan Bridge, Ireland. Reproduced courtesy of Roughan and O'Donovan, Dublin. Copyright reserved.

@Seismicisolation

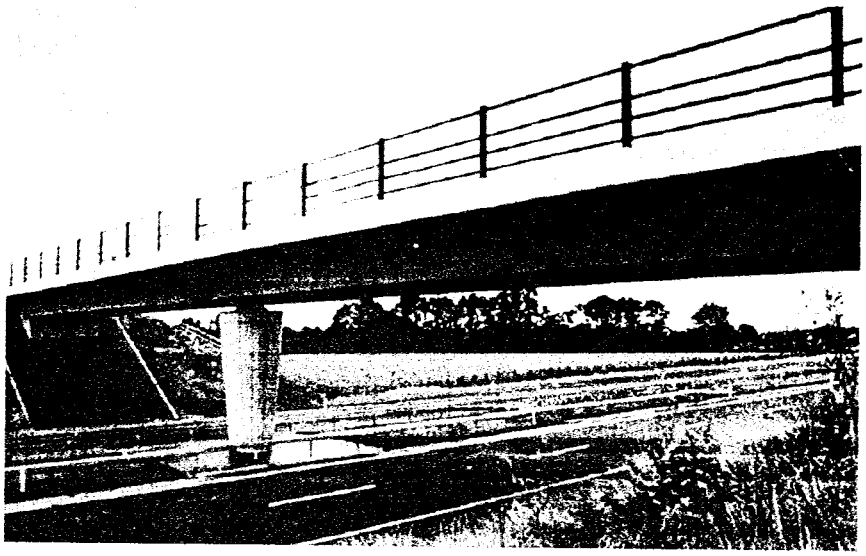


Figure 9.2. M4 Overbridge, Ireland. Reproduced courtesy of Roughan and O'Donovan, Dublin. Copyright reserved.

typical arrangement for this type of construction and demonstrates the pleasing appearance that is achievable.

Solid-slab bridges

Solid-slab *in situ* concrete decks are cast on full-height falsework with a simple formwork, reinforcement and prestress layout. The slabs are post-tensioned with either bar or multi-strand tendons that are installed after the concrete has attained sufficient strength. This arrangement is used for short spans where good access is available. The advantages include straightforward construction and simple formwork layout while the disadvantages are that prestressing a solid concrete element is inefficient and dead load becomes excessive as span lengths get longer.

Several short-span bridges have used sections of precast concrete slabs, lifted into place on to temporary supports and then prestressed together with post-tensioned tendons. The connections between the sections are made either with a narrow mortar or concrete stitch, or as a glued match-cast joint.

Precast pre-tensioned beams placed closely together and topped with *in situ* concrete to form a solid-slab construction are suitable for very short spans. By placing the beams side-by-side no formwork is required for the *in situ* concrete, resulting in a rapid and robust construction.

Voided-slab bridges

In situ voided-slab decks were commonly used during the early period of prestressed concrete bridge development within the span range of 25 m–35 m and with span-to-depth ratios of up to 20 : 1. A typical cross-section is illustrated in Fig. 9.3. The voids usually extend over only the centre portion of the span with a solid section used over the supports. In this way the dead-load

@Seismicisolation

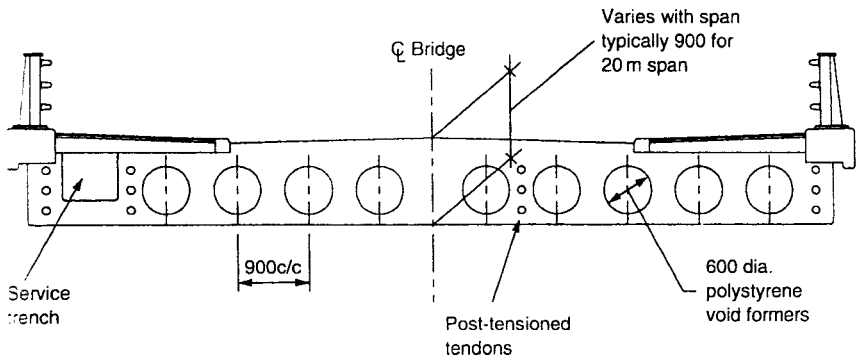


Figure 9.3. Voided-slab deck section.

moments are reduced while the full shear capacity is maintained at each end of the span.

The prestress tendons usually extend the full length of the deck, anchored in the end faces. For continuous decks extending over several spans, multi-strand tendons follow a draped profile within the concrete 'webs' between the voids.

The concrete is cast within formwork supported by falsework built up from the ground. The voids are formed using either round- or rectangular-shaped polystyrene blocks with bevelled corners, as seen in Fig. 9.4. The

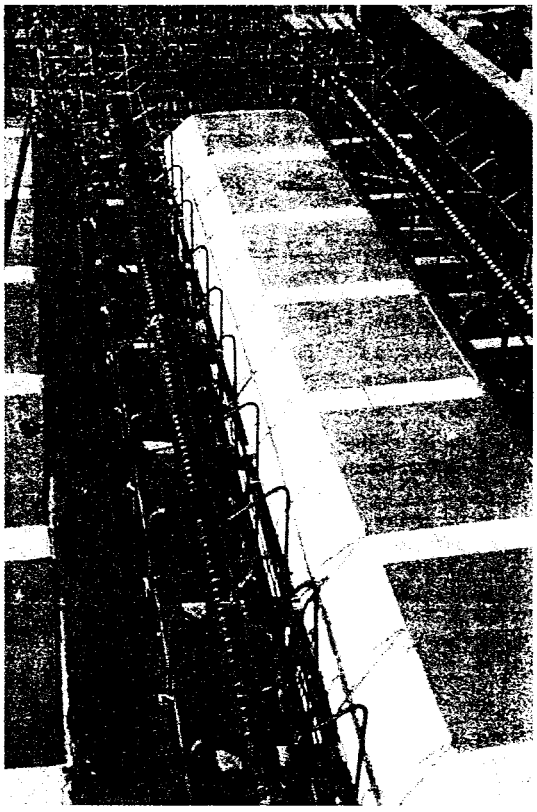


Figure 9.4. Void formers.

@Seismicisolation

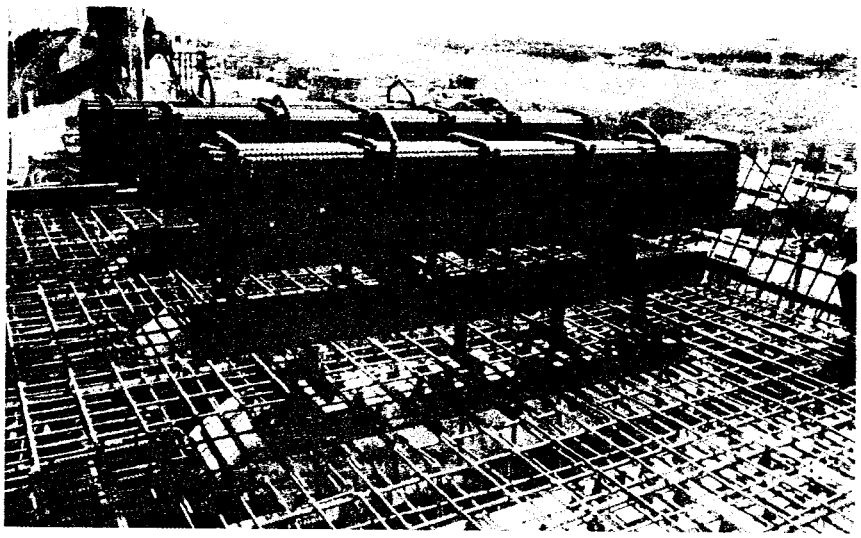


Figure 9.5. Weights holding void formers down.

void-former shape allows the concrete to flow around the polystyrene and to fill the space underneath. A minimum concrete thickness of 150 mm is provided above and below the void for the reinforcement and concrete cover. Drainage pipes are provided at the low point in each of the voids to prevent any build-up of water during and after the curing of the concrete.

As the polystyrene void formers weigh less than the fluid concrete, they tend to float up as the concrete is placed and vibrated. To keep the void formers in place they are held down by ties through the soffit formwork to the falsework below or by placing large weights above the deck supported on the polystyrene, as in Fig. 9.5. The weights are removed after the initial set of the concrete beneath the voids, allowing the top concrete surface to be completed and finished off.

The advantages of voided-slab decks include simple construction and straightforward formwork arrangement, while the disadvantages include the requirement to hold down the voids during concreting, and a heavy deck section. In recent years this has resulted in beam-and-slab or multi-cell boxes being preferred on most projects.

Design of slab bridges

Analysis of the slab, with or without voids, is carried out with sufficient accuracy using a grillage model. The longitudinal grillage members match the layout of the prestress tendons or groups of tendons and the transverse members are placed over the supports and at suitable spacing between to give an evenly balanced grid pattern.

The section properties of both the longitudinal and transverse members are based on the deck width to mid-way between the members with the resultant voided section taken longitudinally and the section at the centre of the void taken for the full width transversely. This is normally found to give sufficiently accurate results for undertaking the design. When assigning the

@Seismicisolation

torsional stiffness to the grillage members it is usually sufficient to give both longitudinal and transverse members the same value of twice the 'I' value of the longitudinal members.

Hambly (1991) [1] gives a detailed description of the behaviour and analysis of slab bridges. The slab arrangement is efficient in distributing the applied live load transversely across the section.

The longitudinal prestress design is based on the bending moments and shear forces extracted from the grillage analysis output. The prestress is designed to balance the serviceability limit state stresses, with checks carried out to ensure adequate ultimate limit state strength as described in Chapter 5.

Transversely, the slab is designed as a reinforced concrete element, with reinforcement placed in the top and bottom of the section to take the transverse bending moments derived from the grillage analysis. Additional reinforcement is placed around the void to tie together the top and bottom sections of concrete and to control cracking in the thinner sections of the slab.

Reference

1 Hambly, E.C. (1991) *Bridge Deck Analysis*, 2nd edition. London: Chapman and Hall.

@Seismicisolation

10

Beam-and-slab bridges

Introduction

Beam-and-slab arrangements are the most common form of prestressed concrete bridge deck and utilize either precast or *in situ* concrete beams with the deck slab usually cast *in situ*. This type of deck is well suited for small- or medium-span bridges where the beam weights are small enough to use readily available cranes and for projects where repetition in the construction gives cost savings. However, beam-and-slab arrangements are generally considered less attractive than the box girder form of construction for the longer spans.

The artist impression of a multi-span viaduct, in Fig. 10.1, illustrates the typical beam-and-slab deck arrangement used. The beams span between piers or crossheads and support a thin deck slab, which provides the platform for the users above. The deck slab and the beams work together as a composite section to resist the superimposed and live loading. The efficient arrangement of the concrete and prestressing results in less overall material being used than in other types of prestressed concrete construction.

With cast *in situ* beams, haunched arrangements are possible, as illustrated in the Analee Bridge in Fig. 10.2. The haunched profile of the beams provides an efficient structural arrangement and an elegant short-span crossing over the river. *In situ* concrete beams are more suited for smaller, 'one-off' bridges where it is difficult to justify setting up a pre-casting yard, if one does not

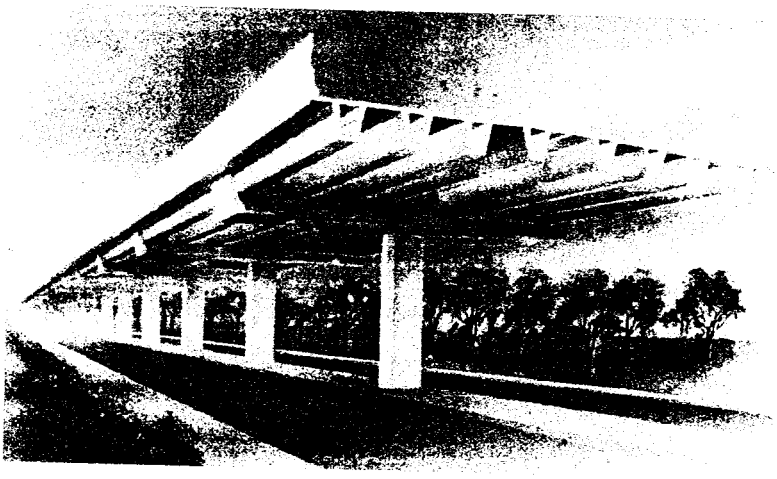


Figure 10.1. Artist impression of the Bangkok SES viaduct, Thailand. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

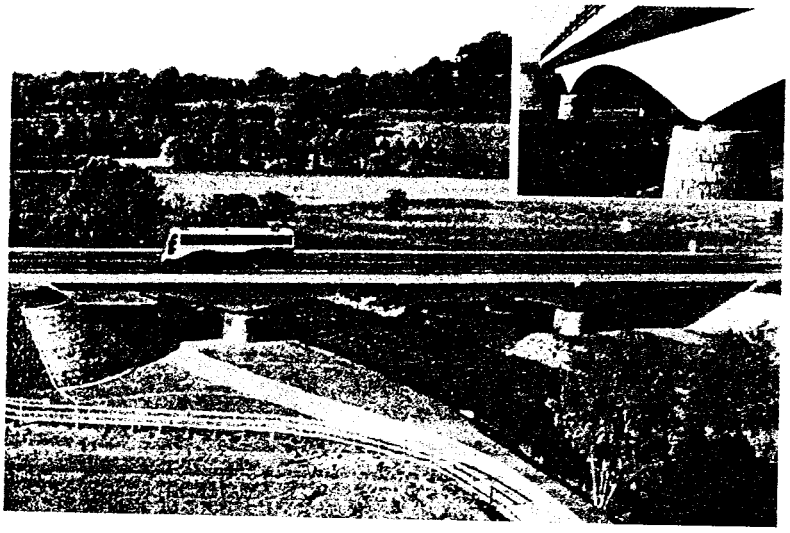


Figure 10.2. Analee Bridge, Ireland. Reproduced courtesy of Roughan and O'Donovan, Dublin. Copyright reserved.

already exist. They are also well suited for bridges in undeveloped countries where the simple construction techniques often give the best results.

Precast beams are usually of constant depth along their length and are frequently used for underpasses or overbridges on highway schemes, as shown in Fig. 10.3. In countries where pre-casting yards are already established beam-and-slab decks provide a quick and economic form of construction. Many different beam shapes have been developed, with the choice of which to use left to the designer. On large projects, with a long length of viaduct or with many bridges to construct, the cost savings from the repetition associated with precast beams may justify the construction of a special casting yard if there is not an existing one close by.

When building over obstructions, such as a river, the use of precast beams reduces the need for temporary works. Figure 10.4 shows a bridge under construction, on the A55 in Anglesey, where the precast beams spanning

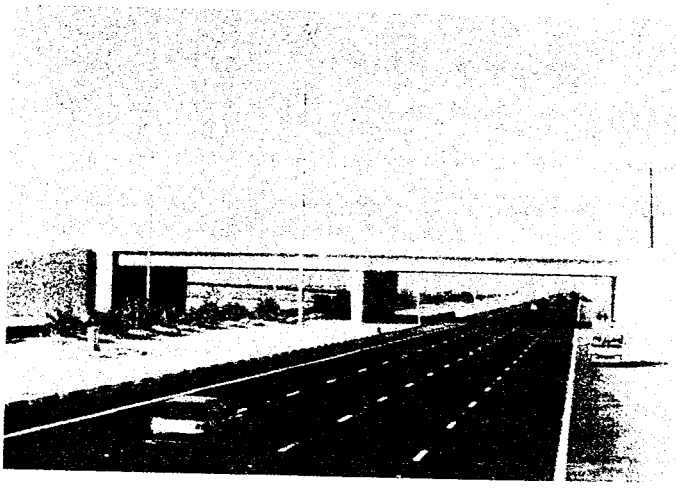


Figure 10.3. The Ghantoot Interchange Overbridge, United Arab Emirates. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

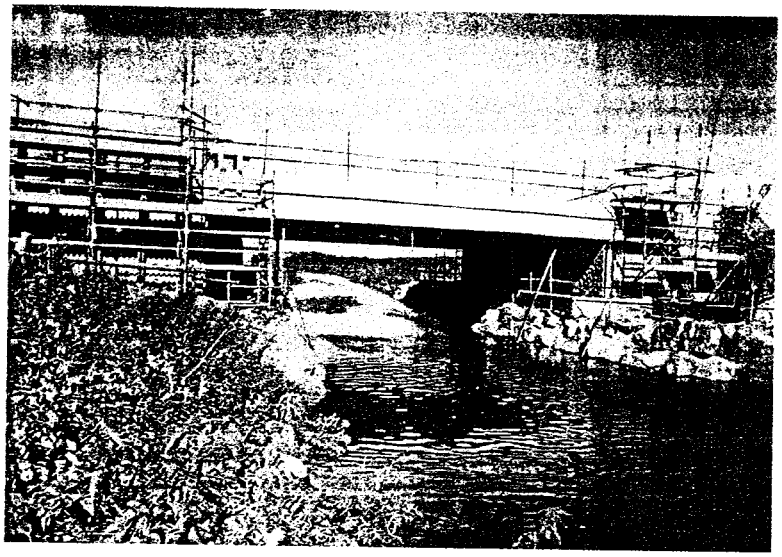


Figure 10.4. Precast beam bridge under construction over river. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

the river were placed by crane. The resulting reduction in work carried out at the site, and the shorter construction period, minimized the impact on the environment.

Bridges constructed over railways have their own set of requirements, including restricted possession periods in which the deck is built. Rail operation safety considerations limit the construction options and the use of precast beams is an established solution for a deck in this situation. The single-span road-over-rail crossing shown in Fig. 10.5 has precast beams lifted into place by a crane during a short possession period, with the deck completed by casting an *in situ* concrete deck slab over the top.

The span lengths of deck using beam-and-slab arrangements range from 25 m up to 40 m. In exceptional cases spans of up to 60 m have been

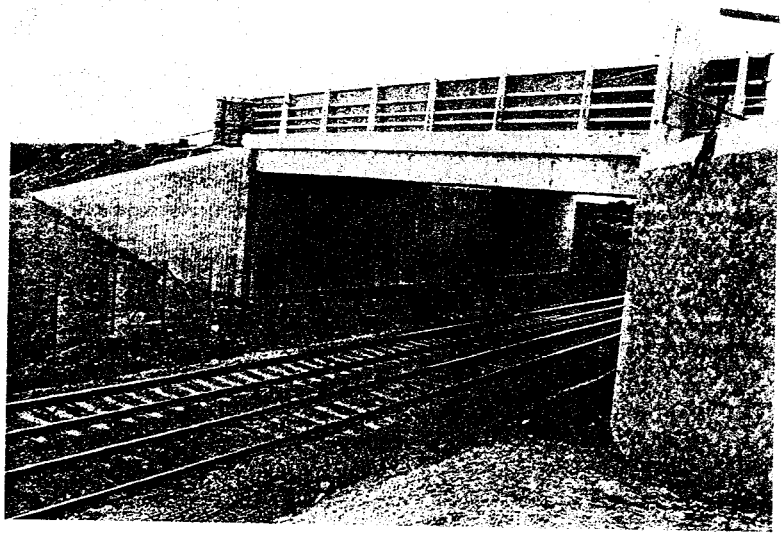


Figure 10.5. Precast beam bridge over railway. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

constructed; although for spans greater than 40 m the box girder form of construction is more efficient. For short spans *in situ* reinforced concrete decks are more economic; although over rivers or railways precast beams are usually preferred in order to overcome the access problems.

Precast beams are more common than *in situ* beams especially where existing pre-casting facilities exist. Precast 'M' beams and box beams are used for shorter spans and minimize the area of deck slab formwork needed. 'I' and 'Y' beams are the most common type for medium-length spans while the 'U' and 'T' beams are used for the longer spans. Recently in the UK, 'super Y' or 'SY' beams have been developed for the longer span range and they are similar to the original 'Y' beams except for being deeper and more slender. Figure 10.6 illustrates some of the different beam

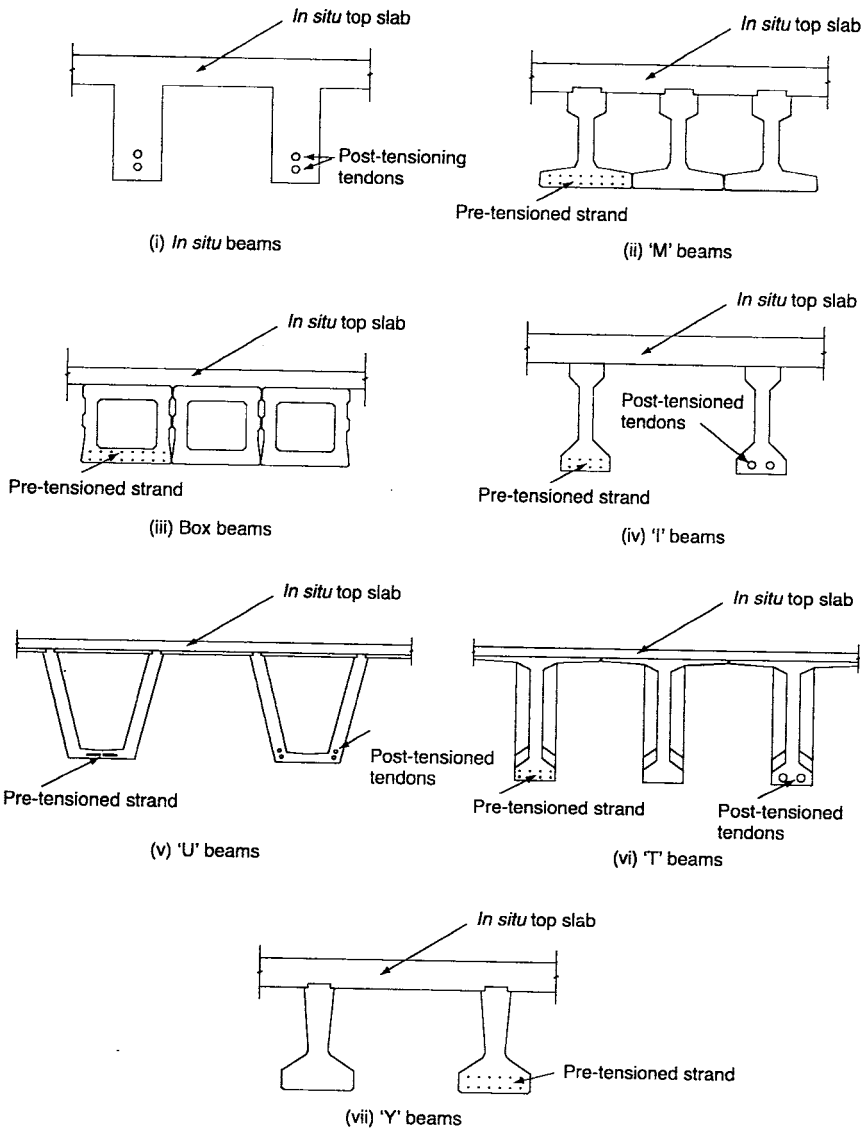


Figure 10.6. Typical precast beam arrangements.

@Seismicisolation

shapes. The history of precast beams in the UK is described in the paper by Taylor (1998) [1].

The advantages of using precast beams include economies from repetition and rapid construction, as well as minimizing the on-site work. The disadvantages include the need for good access to site and for heavy-lifting equipment for the longer beams.

General arrangement

A typical section through a deck with precast 'Y' beams is illustrated in Fig. 10.7. The precast beams span longitudinally and support an *in situ* concrete deck slab, usually cast using permanent formwork supported from the beams themselves. Special edge beams with a plain outer face are used to improve the appearance of the deck. The deck slab is provided with a concrete upstand along both sides of the deck to support the parapet and finish off the verge and waterproofing detail.

The spacing of the beams depends on the type and capacity of the beam used. The spacing is usually less than 3 m to allow standard systems of permanent formwork to be used for the deck slab construction, although it may be wider than this if necessary to suit the design and construction requirements. The deck slab is typically between 200 mm and 300 mm thick which is governed by the beam spacing and the live loading carried. When the overall width of the deck slab varies along the span length the beams are 'fanned' out giving a variation in beam spacing over the span.

Standard beam shapes have developed in countries where precast beams are established and permanent casting yards exist. In the UK, 'M', 'Y' and 'SY' beams are the most widely used with 'inverted T', 'box', and 'U' beams also available. The USA tend to use widely spaced 'I' beams or 'bulb Ts'. In Australia the 'I', 'U', 'T' and 'super T' beams are common arrangements. The 'super T' is shaped similar to a deep 'U' beam with side cantilevers extending out from the top of the webs on either side. It was first used in southern Europe in the 1980s. Where dedicated precasting facilities are set up for a particular project the beam type and shape is developed to suit the project's requirements.

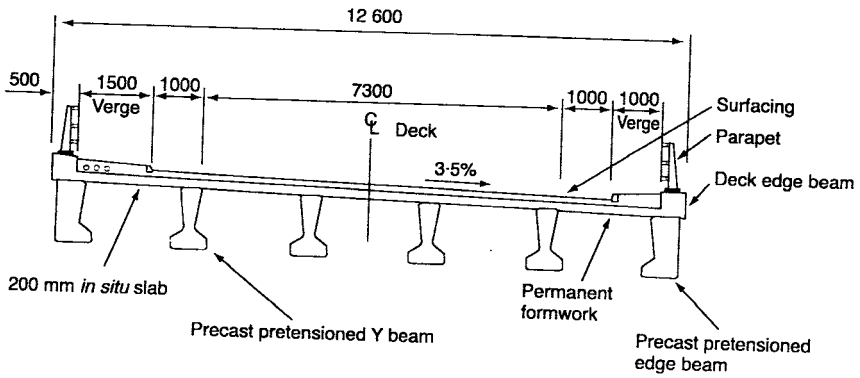


Figure 10.7. Typical precast beam deck section.

@Seismicisolation

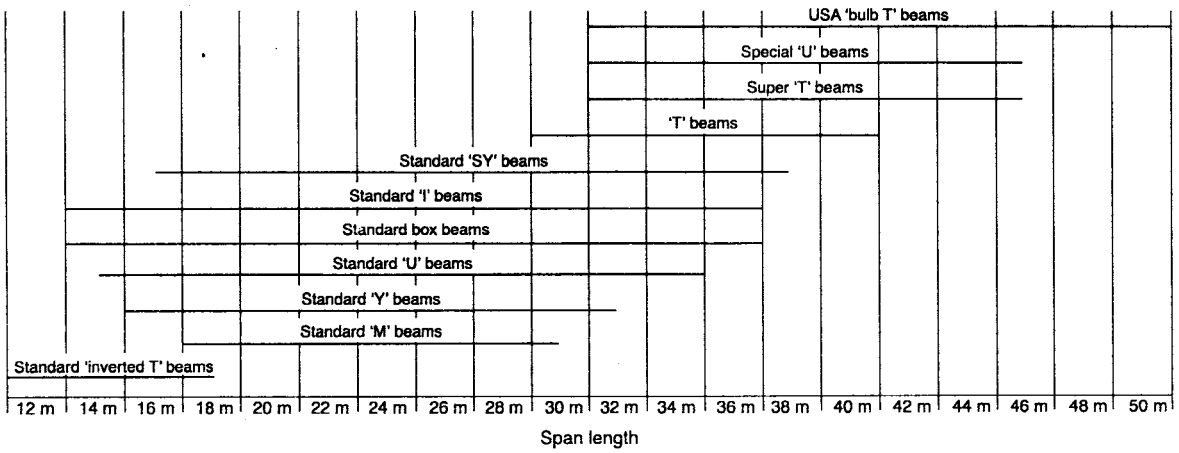


Figure 10.8. Precast beam span range.

The choice of which precast beam shape to use is partly governed by the span length and loading but is often based on the designer's preference.

The standard precast beam types consist of families covering a range of different depths and strand arrangements. The length that each beam is able to span depends on its depth and the beam spacing. The deeper and more closely spaced beams are able to support heavier loads over longer spans. The typical span ranges for the different types of precast beams are given in Fig. 10.8 with the typical span-to-depth ratios of the deck usually between 16:1 and 20:1.

The layout for cast *in situ* beams depends on the deck arrangement required. For short- or medium-span decks the beams are often cast closely spaced, as illustrated in Fig. 10.6(i). Where *in situ* beams are used on longer spans and wide decks an efficient solution is achieved with a 'ladder-beam' arrangement where larger longitudinal edge beams and smaller transverse crossbeams support the deck slab. The arrangement illustrated in Fig. 10.9 spanned

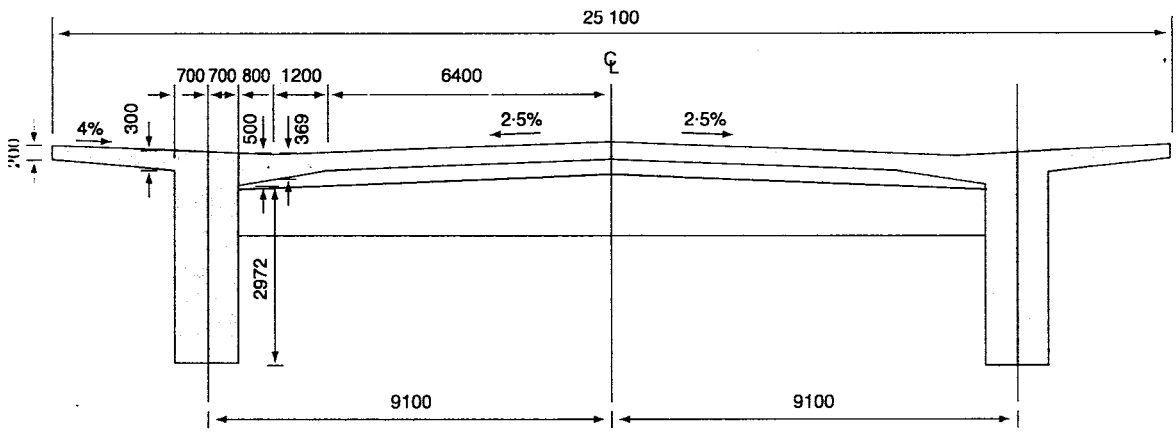


Figure 10.9. In situ *ladder beam arrangement. Reproduced courtesy of Cowi. Copyright reserved.*

@Seismicisolation

60 m with the 3·5 m-deep edge beams supporting shallower crossbeams at 5 m centres and with a 250 mm thick deck slab. Both the edge beams and crossbeams were post-tensioned with multi-strand tendons.

For precast beams the prestressing is achieved with pre-tensioned strand or post-tensioned tendons positioned to take the sagging moments, as indicated in Fig. 5.1. Most standard precast beams, produced in established casting yards, use pre-tensioned strand while post-tensioning is often adopted for the very long beams and when a project-specific casting yard is set up.

At abutments the beams are either supported on bearings or built integral with the abutment wall. At intermediate piers the beams are simply supported or made continuous as illustrated in Fig. 10.10. With a fully continuous arrangement, as shown in Fig. 10.10(ii), the beams are connected together longitudinally with an *in situ* diaphragm over the pier. An arrangement often used is to make the deck slab continuous and the beams simply supported, as shown in Fig. 10.10(iii). This arrangement simplifies the

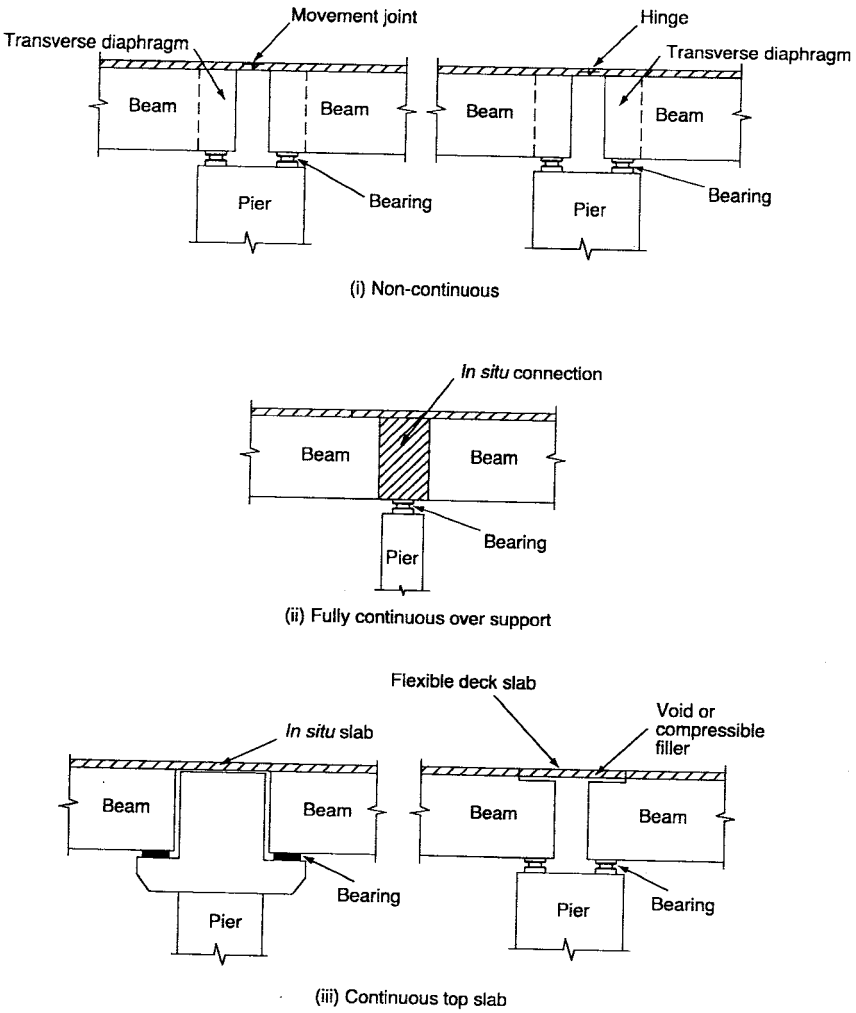


Figure 10.10. Beam continuity at pier.

@Seismicisolation

Figure 10.11. Precast beams on crosshead prior to casting diaphragms and deck slab.

construction while reducing the need for expansion joints in the road surface. The non-continuous arrangement shown in Fig. 10.10(i), is generally not preferred due to the susceptibility of movement joints to leak and the difficulties for inspection and maintenance.

At the supports the beams are connected transversely by an *in situ* concrete diaphragm to provide rigidity to the deck structure. The beams are either individually supported by bearings on a crosshead arrangement at the pier, as shown in Fig. 10.11, or use the transverse diaphragms to transfer the beam load to the bearings, as illustrated in Fig. 6.12(iii). The height of the transverse diaphragms may be reduced to provide additional space above the crosshead to insert temporary jacks, should it become necessary to replace the bearings in the future.

The beams in Fig. 10.11 have a half-joint arrangement at the ends to reduce the overall construction depth of the deck and crosshead at the pier. This reduces the visual impact of the crosshead although the beam construction is made more complex. Where half-joints are used, sufficient space must be provided around the bearings to allow for inspections and maintenance.

Transverse diaphragms are sometimes incorporated in the span to improve the distribution of live load between the individual beams. These intermediate diaphragms have become less common in recent years as designers usually prefer to simplify construction by omitting them and designing the beams accordingly.

Construction of *in situ* beam-and-slab decks

Where the beams are cast *in situ* the formwork is supported on full height falsework or trusses spanning between the piers. The full height falsework shown in Fig. 10.12 was used where good access and suitable ground

@Seismicisolation

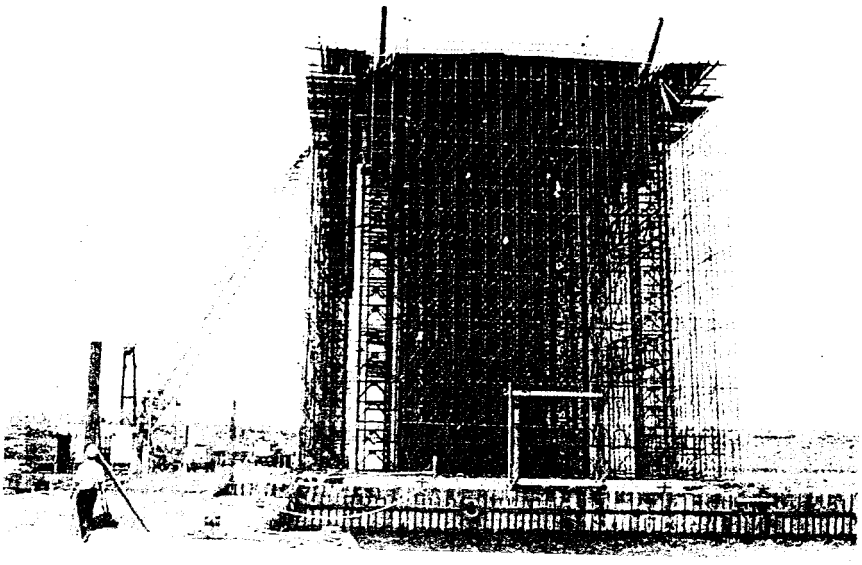


Figure 10.12. Falsework for in situ *beams and deck slab.*

conditions were available. Although it is time-consuming and labour intensive to erect and dismantle the full-height scaffolding the equipment is usually available locally, and the system is versatile, being able to cope with a wide variety of different deck arrangements and site conditions. If sufficient scaffolding and formwork is available several spans may be constructed at the same time which speeds up the overall rate of construction.

Span-by-span construction using a self-launching truss, as shown in Fig. 10.13, is used where access is difficult and a sufficient length of deck justifies the high cost of the truss system. In this example the formwork is hung from the truss, which is supported off the piers and the previously completed deck. Alternatively, the truss may be placed underneath the deck and directly

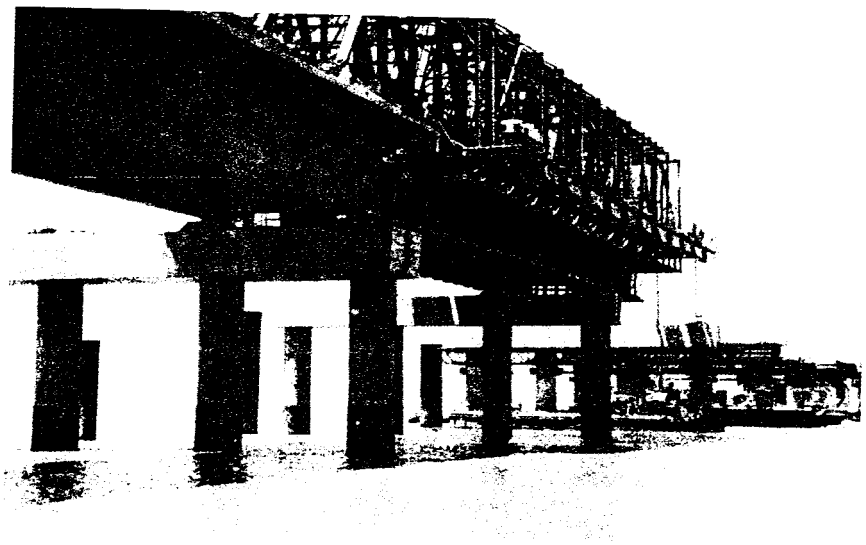


Figure 10.13. Overhead gantry for in situ *beam-and-slab deck. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.*

@Seismicisolation

support the formwork when sufficient headroom is available. Construction usually starts at one end of the bridge and progresses one span at a time, using the completed deck for access. A typical construction cycle would achieve one span every four to six weeks.

The beams and deck slab are usually cast in one pour to give a monolithic construction and to eliminate differential shrinkage effects. After the concrete has hardened the post-tensioning tendons are threaded through the ducts cast into the concrete and stressed, lifting the deck off the formwork. The falsework and formwork are then removed or the truss launched forward to cast the next section of deck.

Casting and transportation of precast beams

Precast beams are either pre-tensioned or post-tensioned and are made in special casting yards set up to produce large quantities of beams in factory conditions.

Casting beds consist of a soffit form, side shutters and internal forms. With pre-tensioning it is usual for a bed to be long enough to cast several beams at a time while with post-tensioning each beam is cast in an individual bed.

Figure 10.14 show a casting yard for pre-tensioned precast 'U' beams where up to five beams were cast on the bed during each cycle. The reinforcement is seen placed on the soffit shutter ready to be fixed in position. Pre-tensioned strands extend between anchor frames at each end of the bed. The side shutters, shown in the insert, and the internal shutter move down the bed to cast each beam on a daily cycle. The completed beams are seen at the top of the picture in the storage area.

With pre-tensioned beams, jacking frames, as seen in Fig. 10.15, are located at either end of the casting bed to provide an anchorage for the strands to be stressed against. These frames are subjected to high forces from the prestressing

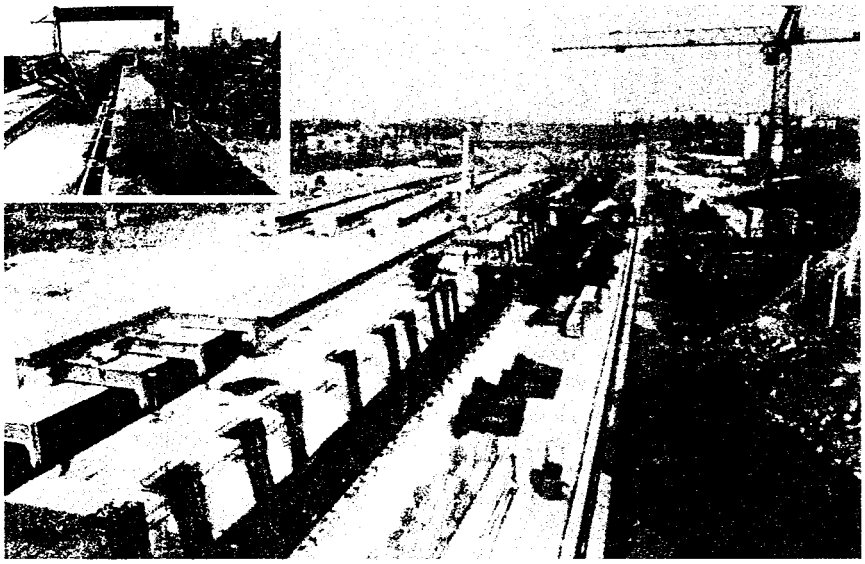


Figure 10.14. Casting bed for pre-tensioned beams.

@Seismicisolation

Figure 10.15. Jacking frame at end of casting bed with inset of strand jack.

strand, especially for the larger and longer beams, and have to be anchored into the ground to stop them sliding forward or overturning.

The strands are individually stressed using a mono-jack, as seen in the insert to Fig. 10.15. After the beams have been concreted the strands are released using a large jack placed within the frame. This jack contracts to allow the back section of the frame to move forward releasing all the strands at the same time. Where several beams are cast on a single bed the strands between the beams are cut and trimmed back flush with the concrete surface.

An alternative to using jacking frames is to anchor the pre-tensioned strand on to the shutters, as seen in Fig. 10.16. The strands are tensioned against a

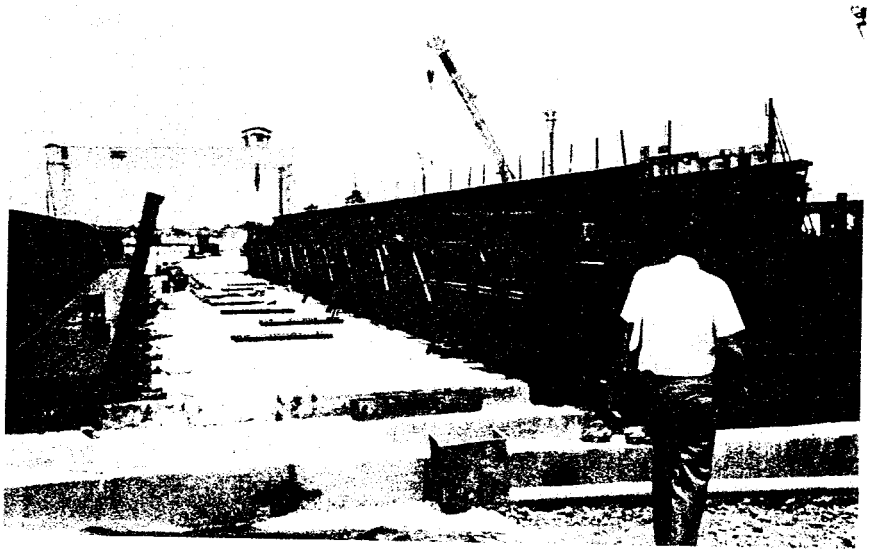


Figure 10.16. Shutter with strands anchored at ends.

@Seismicisolation

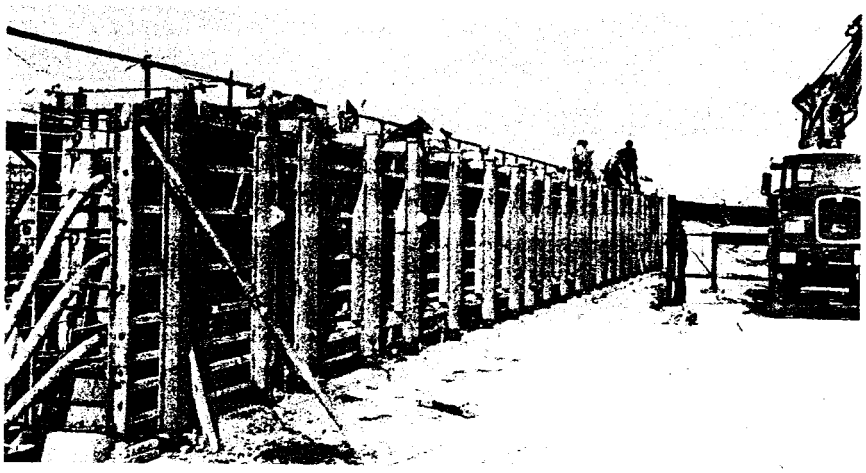


Figure 10.17. Shutter for post-tensioned precast beam. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

steel frame at each end of the beam, with the forces transmitted through the shutter to balance each other. The advantage of this is that the jacking frames and foundations are dispensed with; however, the individual shutters have to be designed to take the high compressive forces, and this adds to their cost.

After installation of the strand, the reinforcement and shutters are assembled and the concrete cast. The side shutters are usually stripped when the concrete has reached a strength of 12 N/mm^2. A concrete strength of at least 35 N/mm^2 is normally required before the strands are released from the jacking frames and the force transferred into the beam. The release of the force in the strands should be done gradually as a sudden release increases the draw-in and anchorage transmission length of the strand.

Steam curing of the concrete is used to accelerate the strength gain for earlier transfer of the prestress and to shorten the cycle time between casting and removing the beams.

To complete the beams the pre-tensioned strands are cut flush with the concrete surface at the ends of the beam, and a water resistant sealant is applied over the strand ends to keep moisture out and reduce the risk of corrosion occurring.

For post-tensioned precast beams the casting cell layout is simpler, as a jacking frame is not needed. A typical arrangement for a precast 'I' beam is shown in Fig. 10.17 where the shutters are set up ready for concreting. The beams are assembled with ducts and tendon anchorages cast in, as seen in Fig. 1.9. This allows the tendon to be threaded through and stressed after the concrete has gained sufficient strength. The ducts are stiffened and protected from the possible ingress of concrete by inserting and inflating a pneumatic tube.

The stressing of the post-tensioned tendons is carried out with standard stressing equipment, as shown in Fig. 10.18. Tensioning all the tendons with only the beam weight present may overstress the concrete, and with post-tensioning the stressing is often carried out in several stages. Sufficient prestress is initially applied to take the beam weight and later the deck slab

@Seismicisolation

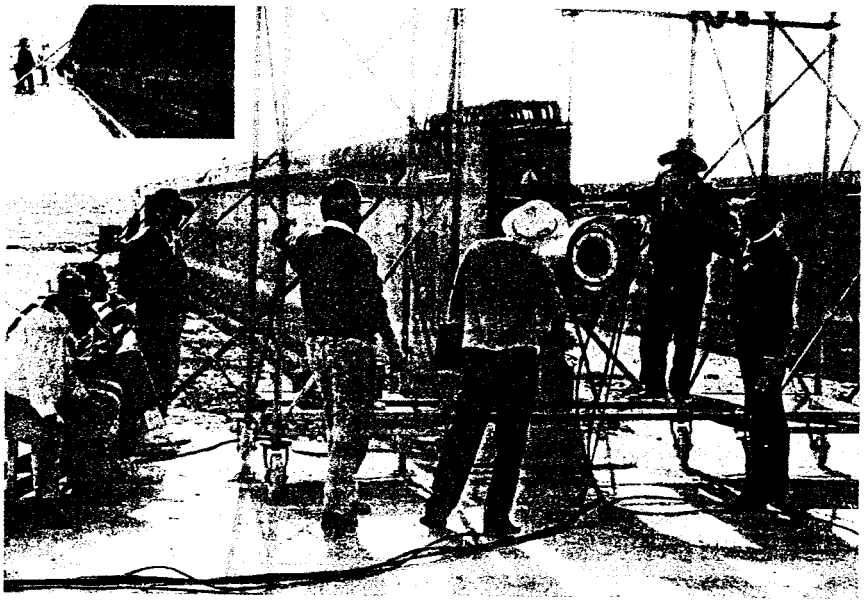


Figure 10.18. Stressing post-tensioned beam. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

weight. After casting the deck slab the remaining prestress is applied to cater for the imposed and live loading.

Post-tensioned tendons are normally more expensive than pre-tensioned strand when taking into account all the costs of installation and grouting. For this reason pre-tensioning is generally preferred unless other factors influence the choice, such as when existing precasting facilities are not available and only a small number of beams are needed. In this situation the cheaper casting set-up makes post-tensioning economic.

For simply supported beams the applied dead-load moment reduces away from the centre to zero at the supports. If a constant prestress force is applied along the bottom of the beam throughout its full length this can generate high compression in the bottom and tension in the top over a significant length at the beam ends. This is overcome in post-tensioned beams by draping the tendons, with the anchors raised up at the ends.

With pre-tensioned beams the prestress force at the ends is reduced by debonding some of the strands with a plastic tube. Alternatively, some of the pre-tensioned strands may be deflected upwards at the ends by means of an anchor frame positioned at $\frac{1}{4}$ or $\frac{1}{3}$ points along the beam length. Deflecting the strand complicates the casting bed arrangement and is not commonly done.

During the transfer of prestress to a beam, the beam tends to lift up along its length as the prestress counteracts the dead load. As the beam lifts off the bottom shutter, as seen in the insert in Fig. 10.18, its weight is shed back to the ends of the beam. If this weight acts through a short length at each end of the beam the concentrated load can cause local overstressing and spalling of the concrete. To prevent this a short length of soft bearing material is placed under the beam soffit at the ends to help spread out the load.

@Seismicisolation

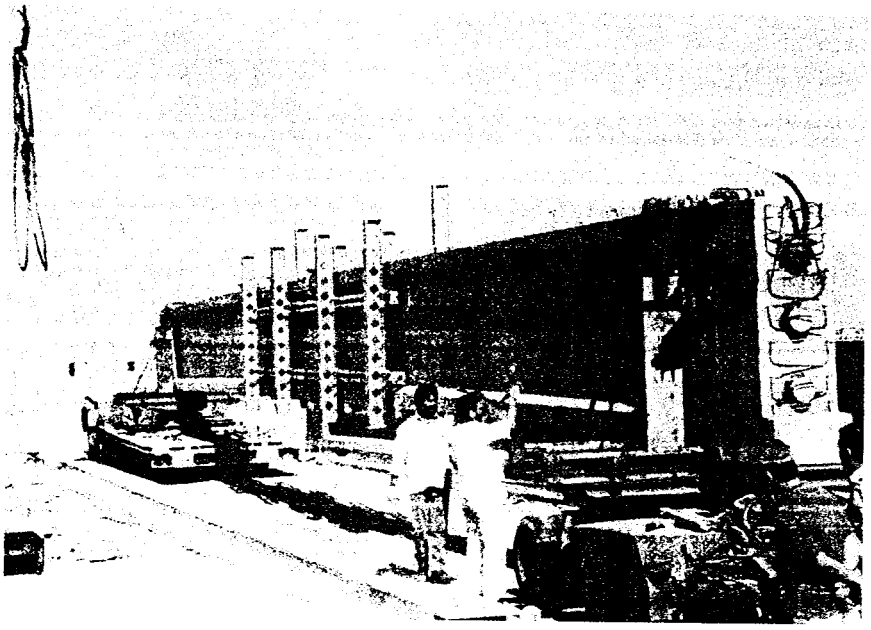


Figure 10.19. Beam transporter. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

After the beam has been prestressed it is lifted off the casting bed using a crane or gantry and moved to a storage area. Moving the beams short distances around the casting or storage areas is often done by using specialist equipment such as overhead lifting frames or transporters. When moving beams long distances and over public highways, specialist bogey arrangements, as seen in Fig. 10.19, are used. Smaller beams are often placed on a standard low-loader for transporting.

The longer and more slender beams, such as the 'SY' type, have little transverse strength and when lifted or transported they are susceptible to lateral bending and buckling, and they may require temporary support frames.

Beam weights vary greatly depending on the type of beam and its length. A typical 16 m long 'Y' beam weighs approximately 12 tonnes while similar beams spanning 30 m would weigh approximately 40 tonnes. For the long-span 'U' beams the weights can exceed 150 tonnes and specialist heavy lifting and transporting equipment is required.

Beams are lifted using hooks cast into the concrete or by inserting a lifting bar through holes formed in the ends of the beam. The lifting points are placed as near to the permanent support positions as possible to set up similar bending moments in the beam.

Erection of precast beams

The placing of precast beams into position is normally done by crane; however, for long multi-span bridges or where ground access is poor an erection gantry is used to lift the beams and carry them into place.

The precast beams are lifted individually and placed on to the bearings or supports. Figure 10.20 shows an edge beam being lifted into position by a

@Seismicisolation

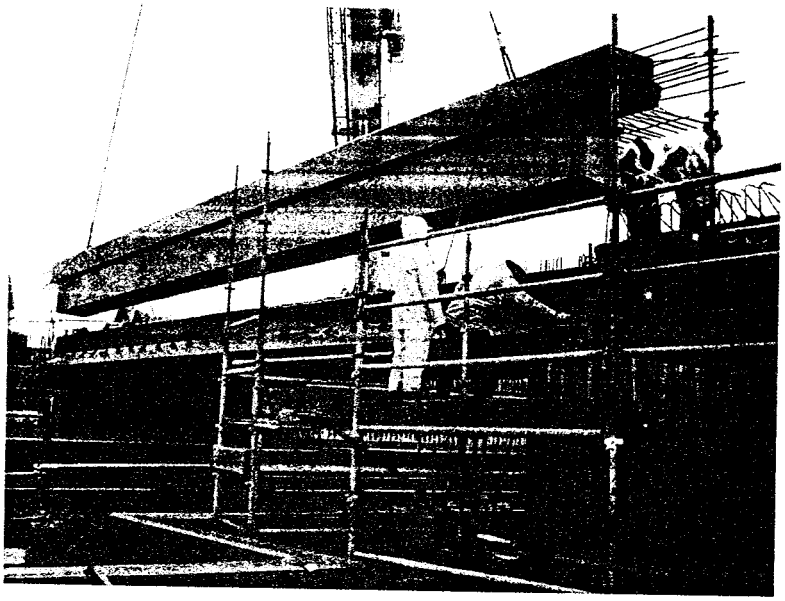


Figure 10.20. Beam placed by crane. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

crane. The beams are spanning a small river and the crane was able to reach the deck from a position behind the abutment. When selecting the beams to be used the weight of each should be considered, to ensure that there is a suitable crane available with the capacity to lift the beams at the radius required.

With long and heavy beams it is sometimes necessary to use two cranes; although in some countries this is discouraged due to the high safety risks involved. With a crane at each end of the beam careful control is needed to ensure that they are evenly and correctly loaded at all times.

Precast beams are usually placed directly on to the bearings at the supports; although where they are built into a diaphragm or made continuous over the support they are placed on temporary supports while the *in situ* connection is made.

Where bearings are used under each beam they are installed on the bridge piers or abutments prior to lifting in the precast beams. Rubber or laminated bearings are either placed directly on to the prepared concrete support or bedded on a thin layer of cement mortar. Pot or mechanical bearings are placed on a cement grout bedding with dowels fixing the bearing to the concrete below.

With rubber bearings the beams rest directly on to the bearing surface or are bedded on a thin layer of cement mortar. If pot bearings are used the beams are held above the bearing leaving a small gap which is filled with a cement grout.

Erection gantries are used on long viaducts where there are many spans, and the cost of the gantry is spread out over many uses. The gantry sits on the pier crossheads or abutment, as shown in Fig. 10.21 and is self-launching, moving from pier to pier as the deck progresses. Gantries operate in one of two ways depending on how the beams are delivered. If the beams are delivered along the completed deck they are picked up by the back of the

@Seismicisolation

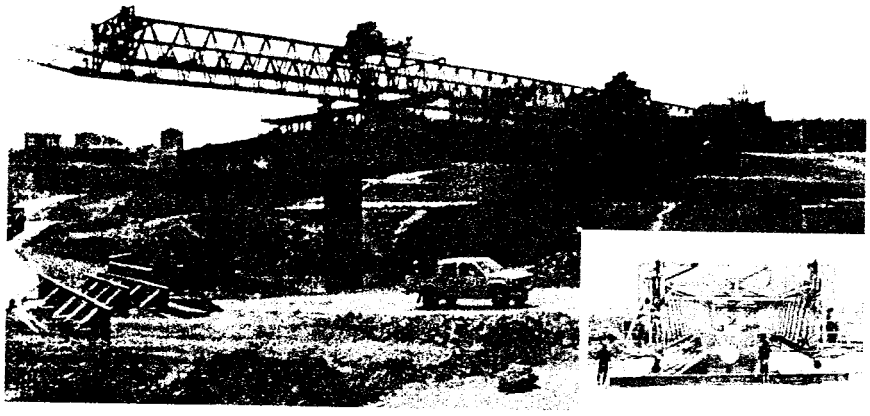


Figure 10.21. Gantry for placing beams.

gantry and moved out over the span before being lowered into position. If the beams are delivered at ground level, the gantry reaches down and raises them up to deck level, placing them on the bearings.

When the beams are placed on the bearings they are held in position by temporary frames or struts, which prevent them from sliding off or from toppling over. The beams may also need holding in place during the concreting of the deck slab, although the deck formwork often provides sufficient support.

Casting of deck slab

After positioning the precast beams the formwork is placed to allow the top slab and diaphragms to be cast. With closely spaced beams only minimal formwork is needed, while with widely spaced beams the formwork and falsework may be significant.

Where there is good access from the ground the deck slab formwork may be supported by full-height scaffolding. As the deck slab weight is not transferred to the beams until the slab concrete has gained strength, it has the advantage of improving the stress distribution across the composite section in the completed structure.

It is common to use permanent formwork between the beams, while for the deck side cantilevers and parapet beams the formwork is supported off the outer beam or from the deck slab between the beams if this is completed first.

Permanent formwork is supported on the edges of the beams, spanning across the gap. Glass-fibre reinforced cement (GRC) panels are used where the beams are closely spaced, with ribbed glass reinforced plastic (GRP) panels used for spans up to 4 m. Precast concrete planks, either ribbed or a constant depth, are used for a wide range of beam spacing and act compositely with the deck slab. The panels are bedded down on the edge of the beam and sealed to prevent grout loss.

@Seismicisolation

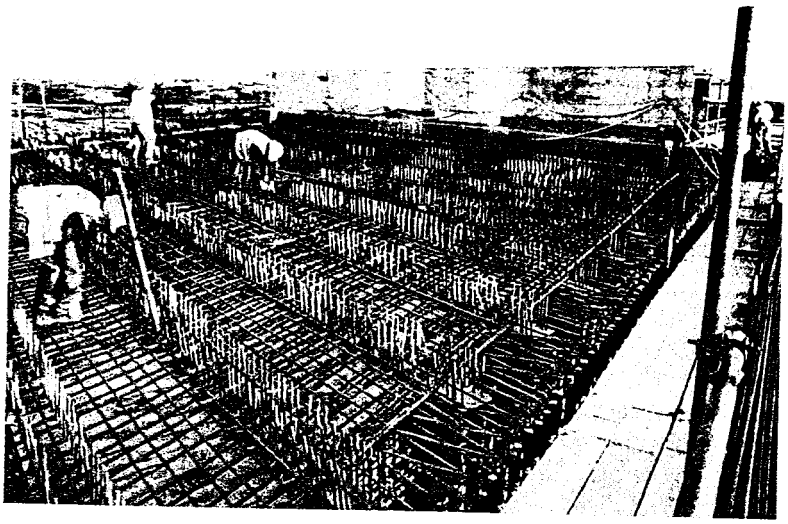


Figure 10.22. Formwork and reinforcement being placed. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

The formwork arrangement for the edge cantilevers is simplified by casting the deck width in stages, although this needs to be considered during the design to cater for the differential shrinkage effects. In the first stage the deck slab is cast across the beams omitting the edge cantilever section. The formwork for the edge cantilever is then supported off the completed deck slab. Alternatively, the edge cantilevers may be precast and lifted into position before being connected to the deck with an *in situ* stitch.

With the beams and deck formwork in place the deck slab reinforcement is fixed, as seen in Fig. 10.22. The bars are detailed with a simple layout for easy positioning and fixing. Holes are left through the webs of the beams at the transverse diaphragm positions to feed bars through and provide the transverse connection.

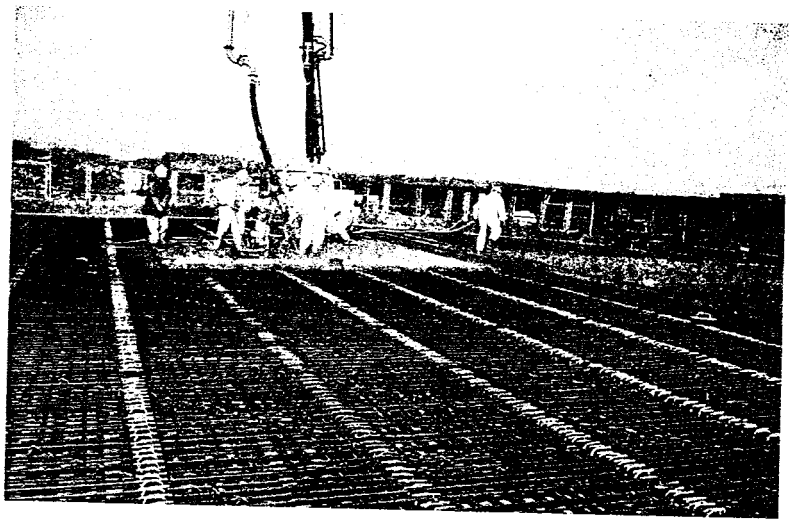


Figure 10.23. Deck slab being cast. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

The deck slab concrete is placed by skip or with a pump, as shown in Fig. 10.23. The deck is divided into bays to help control the concreting and to establish the finished levels required. On large areas of deck a finishing machine is often used to help level and compact the concrete surface and to give access across the concrete to apply the surface finish and curing. A typical finishing machine is shown in Fig. 11.9.

Design of beam-and-slab decks

Design of beam-and-slab decks involves the superimposing of many different effects to build up the overall design state, as described in Chapter 5. The structural behaviour of beam-and-slab decks is described by Hambly (1991) [2].

General design

Longitudinal and transverse effects in the deck slab under superimposed dead load and live load are derived from a grillage analysis. This gives satisfactory results for most standard beam-and-slab arrangements. Recommendations for setting up grillage models are given by West (1973) [3].

Where unusual beam-and-slab arrangements occur or secondary effects, such as distortions or transverse bending in the beams, become significant a three-dimensional finite element model should be used to derive the load effects. Shell elements are used to model the webs, slabs and diaphragms of the three-dimensional structure. The dead, superimposed and live loads are applied directly to the model to give the forces and moments at the critical sections for the beam-and-slab design.

Serviceability design considering the stresses in the deck is described in the next section. The design of the deck for ultimate moment, shear and torsion is carried out as described in Chapter 5. The check of the longitudinal shear along the interface between the slab-and-beam must include the forces generated by the differential shrinkage and creep effects.

During lifting and transporting, precast beams are subjected to different loading conditions causing a change to the bending moments and shears, which must be considered in the design. The beams must be handled carefully to minimize any impact or dynamic loading, while lifting and temporary support positions should be located as close to the permanent support position as possible. With long slender beams, transverse instability may occur either during lifting or when placing the concrete for the top slab, and suitable restraints may be needed. This effect is described in the paper by Burgoyne and Stratford (2001) [4].

Stress distribution through section

The prestress and dead-load stress distribution through the beam-and-slab and the time-dependent effects of creep, shrinkage and relaxation of the prestress are all taken into account in the design. Differential shrinkage between the precast beams and the *in situ* deck slab also affects the stress distribution in the section.

@Seismicisolation

The effect of the construction and stressing sequence and the time dependent behaviour are considered in the design by building up the stress profile through the deck, superimposing the different effects at each stage in the construction and during service.

When the precast beams are cast they are supported by the formwork along their full length, and on the application of the prestress they tend to hog upwards between the ends. On moving to the storage yard they are supported only at the ends of the beams, giving a typical stress profile at mid-span as indicated in Fig. 10.24(i). At this stage the beams are subject to their own dead weight and the applied prestress, with the resulting stresses at the top and bottom of the beam derived from simple bending theory. With the full prestress present and only the self-weight of the beam the problem is to

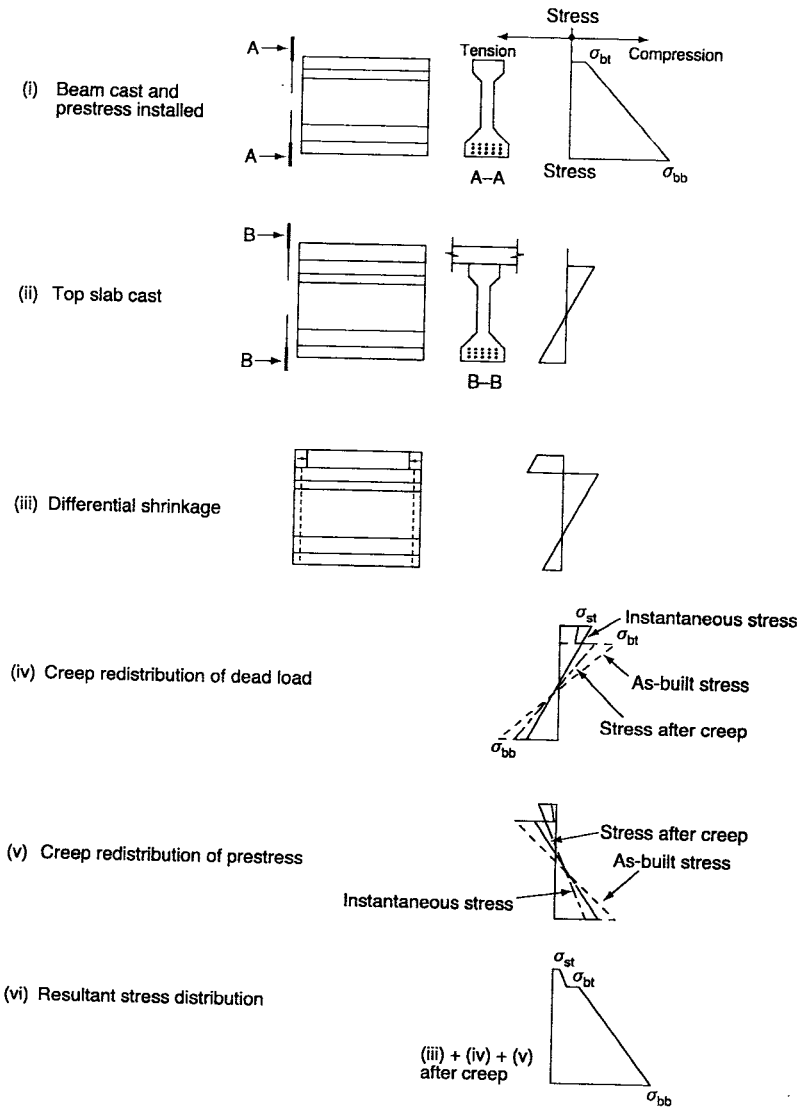


Figure 10.24. Stresses in precast beam deck.

@Seismicisolation

ensure that the compression in the bottom and tension in the top of the beam do not exceed the allowable limits.

After erecting the precast beams, if the deck slab is cast on permanent formwork the weight of the slab is carried by the beam section alone to give a stress distribution, as indicated in Fig. 10.24(ii). If the deck slab is cast on full-height scaffolding when the scaffold is lowered the slab's weight is carried by the beam-and-slab's combined section.

After the deck slab concrete has hardened, creep of the concrete redistributes the dead load and prestress stresses within the section while differential shrinkage between the beam and deck concrete sets up additional stresses in the section. The superimposed and live loads are carried by the combined beam and deck slab section.

The creep redistribution of the dead load and prestress stresses is estimated by comparing the as-built condition with the 'instantaneous' state. In the as-built condition the entire dead load and prestress force is carried on the beam and the stresses are a combination of Fig. 10.24(i) and (ii). In the theoretical 'instantaneous' condition the full dead load and prestress force is assumed to be applied to the composite section. The stresses will creep from the as-built condition towards the instantaneous condition with the final stress at any level in the section being given by:

$$\sigma_{\text{final}} = \sigma_{\text{as-built}} + (1 - e^{-\phi})(\sigma_{\text{inst}} - \sigma_{\text{as-built}})$$

The loss of prestress forces due to relaxation in the strand, and the concrete creep and shrinkage affect the stresses in both the slab and beam. These long-term prestress losses should be considered in two phases. For the first phase, before the deck slab is cast, the prestress losses are estimated and the prestress force in the beam reduced accordingly. In the second phase, the remaining losses that occur after casting the slab are estimated and applied as a 'tensile' force to the composite section at the position of the centroid of the strand or tendons. The resulting stresses are then combined with the other stresses in the section.

When the top slab is cast, the concrete in the beam will have already completed a large proportion of its shrinkage, and differential shrinkage occurs between the two. The deck slab tries to shrink more than the beam and is restrained by it, creating tension in the slab and a combined compressive force and sagging moment in the beam. The equivalent differential shrinkage force assuming the top slab was fully restrained is estimated from:

$$\text{force, } F_s = \Delta_s \cdot A_s \cdot E_c \frac{1 - e^{-\phi}}{\phi}$$

with $(1 - e^{-\phi})/\phi$ reducing the effect due to creep of the concrete.

The stresses in the composite section are derived by adding the stress of the 'fully restrained' slab to the stress due to the released force applied to the composite section, i.e.

(a) tensile stress from force F_s acting on slab alone applied at the centroid of slab, plus

@Seismicisolation

(b) stresses generated from a compressive force F_s acting on the composite section, applied at the centroid of the slab.

This gives a stress profile similar to Fig. 10.24(iii).

The stresses generated at the top and bottom of the slab and top and bottom of the beam are checked at each stage of construction as well as in the permanent condition after opening and again after the long-term effects have occurred.

Precast beams in continuous and integral decks

When the beams are fully connected longitudinally over the piers to make the deck continuous, as illustrated in Fig. 10.10(ii), the dead load and prestress effects are redistributed due to creep of the concrete. The dead-load sagging moment is reduced at mid-span and a hogging moment set up at the pier. The superimposed and live loads will also cause a hogging moment at the connection. The creep of the concrete under the prestress force causes the beam to hog at mid-span generating a sagging restraint over the pier. This may lead to cracks in the deck soffit above the pier if the effect is not fully considered and suitable reinforcement provided.

The final moment at the pier due to the creep of the dead load and prestress may be derived by a similar approach to that described in Chapter 5, with

$$M_{\text{final}} = M_{\text{as-built}} + (1 - e^{-\phi})(M_{\text{inst}} - M_{\text{as-built}})$$

ϕ is based on the residual creep left in the beam at the time of casting the connection and $M_{\text{as-built}}$ is zero where the connections are made after erecting the beam and casting the top slab. The moment, M_{inst}, calculated as if the deck was built continuous 'instantaneously' includes the prestress secondary moment as well as the prestress primary moment and dead-load moment.

The differential shrinkage between the top slab and beam generates a secondary hogging moment in the deck at the pier after the connection has been made, which must also be considered in the design.

Reinforcement is provided across the connection to cater for the hogging and sagging moments in the deck above the pier. The section is designed as a reinforced concrete element, with reinforcement placed in the deck slab and between the bottom of the beams. With larger beams it is possible to provide post-tensioned tendons in the webs which run through the connection to prestress the *in situ* concrete.

A similar behaviour occurs when precast beams are built integral with the abutments. Reinforcement is provided to connect the deck slab and beam with the abutment to take the moments' set up. The hogging moments may also set up tensile stresses in the top of the precast beam and extra reinforcement is provided to control any cracking that may occur.

Prestress and reinforcement

Post-tensioned tendons usually follow a draped profile and need additional reinforcement at the beam ends around the anchorages, as described in Chapter 6 and shown in Fig. 6.3. Reinforcement is provided along the beam to cater for shear and torsion, and the early thermal effects within the concrete.

@Seismicisolation

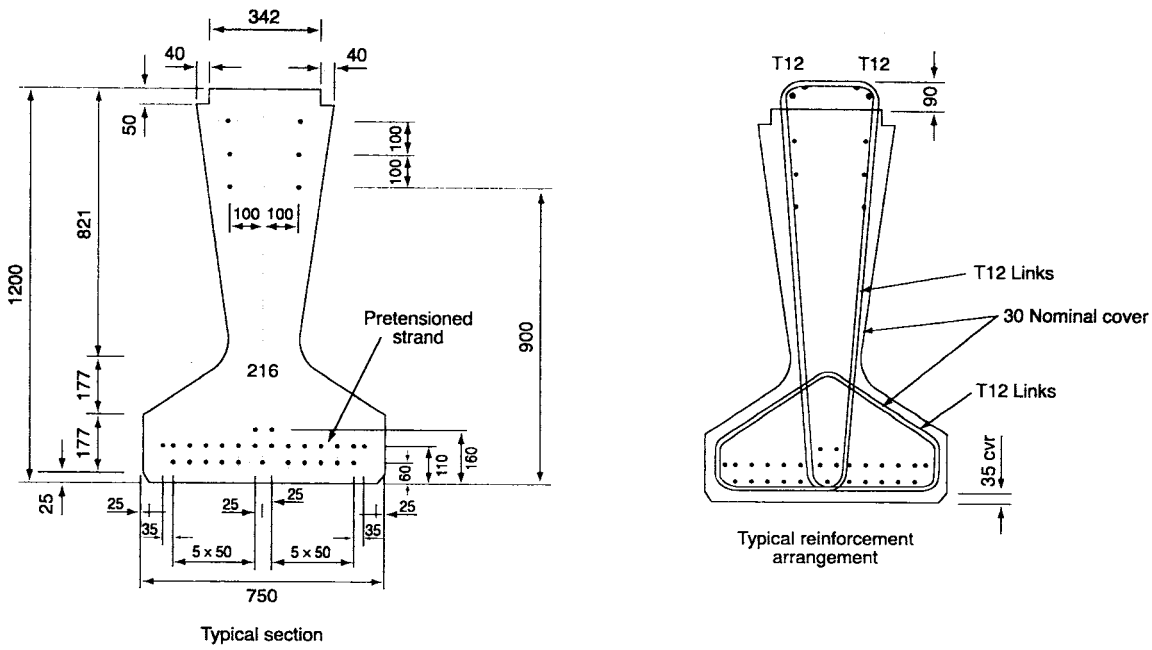


Figure 10.25. Typical reinforcement and prestress arrangement.

With pre-tensioned beams longitudinally the strands are sufficient with only nominal additional reinforcement required in the top of the beam as illustrated in Fig. 10.25. The main prestressing strands are spread out evenly over the bottom of the beam with a minimum spacing of 50 mm and a concrete cover similar to that normally provided for reinforcement. Additional strands are sometimes provided in the top of the beam to overcome tension in the temporary condition before the beam is fully loaded.

The reinforcement in the deck slab is designed to cater for the moments, shears and torsions acting on the elements as derived from the grillage analysis. The design moments used to calculate the reinforcement requirements are derived by combining the direct bending and torsion as described in the paper by Wood (1968) [5].

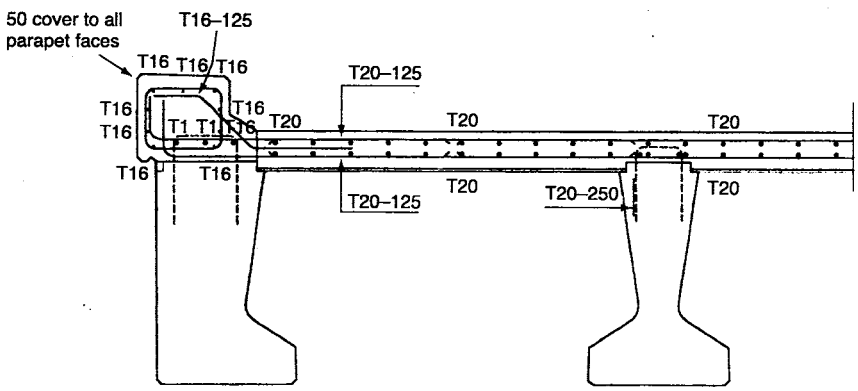


Figure 10.26. Typical deck slab reinforcement.

@Seismicisolation

The quantity of reinforcement required depends on the depth of deck slab, beam spacing and the live load carried. A typical arrangement for the deck slab reinforcement is illustrated in Fig. 10.26.

Details of the standard precast beams are found in manufacturers' literature and on the appropriate websites noted in Appendix C.

References

1. Taylor, H.P.J. (1998) 'The precast concrete bridge beam: the first 50 years', *The Structural Engineer*, **76/21**, November, 407–14.
2. Hambly, E.C. (1991) *Bridge Deck Analysis*, 2nd edition. London: Chapman and Hall Ltd.
3. West, R. (1973) *Recommendations on the Use of Grillage Analysis for Slab and Pseudo-slab Bridge Decks*. London: C&CA/CIRIA.
4. Burgoyne, C.J. and Stratford, T.J. (2001) 'Lateral instability of long-span prestressed concrete beams on flexible bearings', *The Structural Engineer*, **79/6**, March, 23–6.
5. Wood, R.H. (1968) 'The reinforcement of slabs in accordance with a pre-determined field of moments', *Concrete*, February, 69–76.

@Seismicisolation

11

In situ multi-cell box girder decks

Introduction

In situ multi-cell concrete box girder bridges are frequently used in the medium span range from 30 m up to 50 m. They provide a versatile arrangement suitable for simple highway bridges or longer viaducts while able to cope with complex geometry, tight curvature and variable deck widths. The box shape gives a good aesthetic appearance as well as being efficient in its design. Figure 11.1 shows an example of a bridge with this type of deck built as part of a highway interchange scheme in Hong Kong.

Below spans of 30 m precast beams or voided slab decks are more suitable while above 50 m a single-cell box arrangement is usually more economic.

The advantages of this type of construction include the efficient use of the concrete and prestress, simple construction where access is easy and the ability to cope with complex geometry. The techniques used are well established and experienced labour and equipment are, usually, readily available locally.

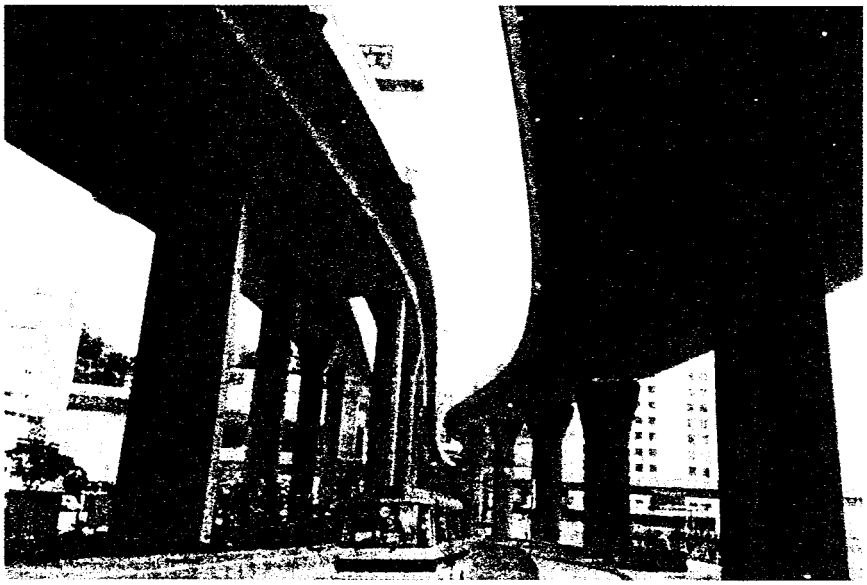


Figure 11.1. Kwun Tong Bypass, Hong Kong.

@Seismicisolation

The disadvantages include the need for extensive labour activities on site, slow construction and complex falsework if access is difficult.

A variation of this type of deck is the two- or three-cell box girder which is also used for longer spans, utilising the techniques as described in Chapters 12 and 13. This chapter concentrates on the multi-cell arrangements used for the medium-length spans when cast on full-height falsework or temporary support trusses.

General arrangement

The deck arrangement is similar to a voided slab, but with the voids occupying a larger proportion of the deck area and usually being rectangular in section. A typical deck section is illustrated in Fig. 11.2. The outer webs are often sloped and the side cantilevers made longer to improve the appearance.

The web thickness is governed by the shear requirements, but they must be wide enough to provide space for the reinforcement and concrete to be placed around the prestressing ducts. This usually requires a minimum web thickness of 300 mm, but may need to be wider if large tendons are used. The deck slab size is governed by the web spacing and the live loading carried and is typically between 200 mm and 300 mm thick. There is very little loading on the bottom slab with a thickness between 150 mm and 200 mm being sufficient.

Transverse diaphragms are provided across the full width of the box at each of the support locations. The diaphragms provide rigidity to the box and assist in transferring the loads in the webs to the supports. Intermediate diaphragms are often placed at $\frac{1}{4}$ or $\frac{1}{3}$ points along the span to stiffen up the box and to help distribute the loading between the webs.

Access into the box cells is achieved through soffit access holes of a minimum of 600 mm diameter, and are located near the abutments. Similar sized holes are provided through each of the diaphragms and webs, as required to give access into each section of the deck. Small drainage holes, typically 50 mm diameter, are provided through the bottom slab at the low point in each section of deck to ensure that water cannot collect inside the box cells.

Concreting and construction restraints dictate a minimum deck depth of 1200 mm; although for reasonable inspection and maintenance access a depth of at least 1800 mm is needed. With an optimum span-to-depth ratio

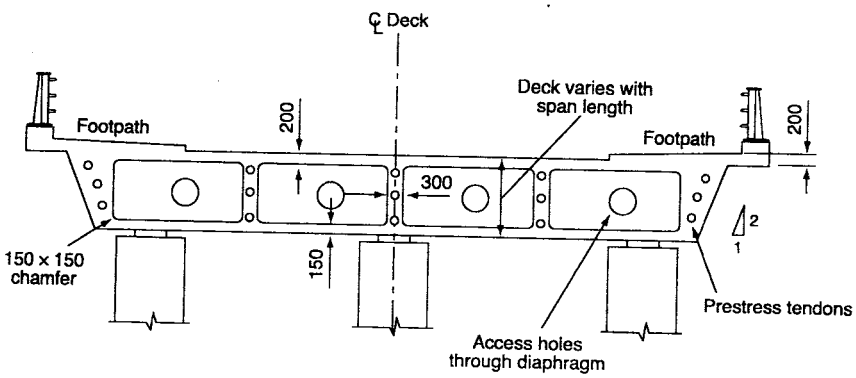


Figure 11.2. Typical box cross-section.

@Seismicisolation

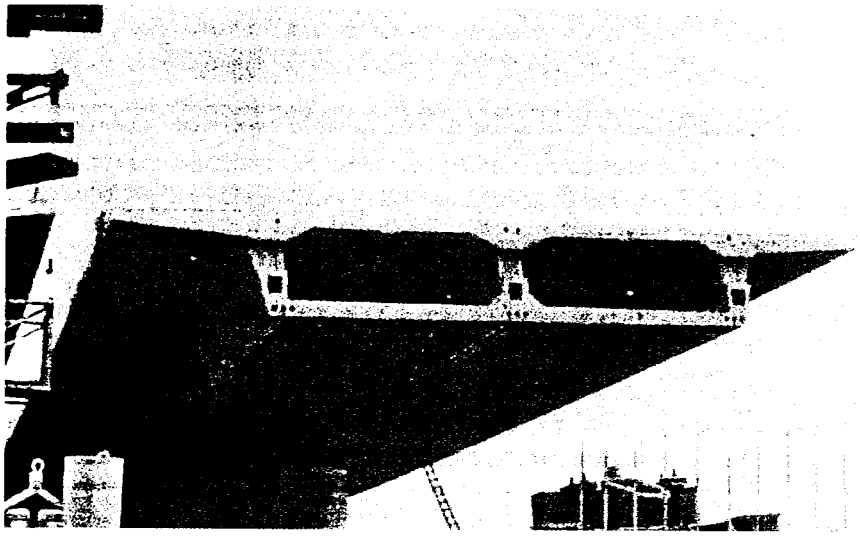


Figure 11.3. Multi-cell box girder bridge. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

of between 18 : 1 and 25 : 1, the preferred span lengths are usually greater than 30 m.

Multi-strand tendons are used following a draped profile, and are located in the bottom of the webs in mid-span and at the top of the webs over the supports. For decks with an overall length less than 80 m and fully cast before applying the prestress, the tendons would usually extend over the full deck length and be anchored on the end diaphragms. Longer decks are cast in stages on a span-by-span basis, with the prestress tendons anchored on the webs at the construction joint, as seen in Fig. 11.3. The tendons are then continued into the next stage of deck by using couplers.

Construction of *in situ* multi-cell box girders

Most *in situ* multi-cell box girders are cast on full-height scaffolding built up from the ground, as seen in Fig. 11.4. Where good access exists this form of construction provides flexibility in the construction sequence and deck layout. Obstructions under the deck, such as live roads, railways or small rivers, are overcome by spanning with temporary works to support the falsework. Guidance on the design and construction of falsework is given in the reports by Bragg *et al.* (1975) [1] and the Concrete Society and IstructE (1971) [2].

After erecting the scaffolding the formwork is placed to the required shape and profile. Timber formwork, consisting of a plywood facing supported by timber studding, is shown being installed in Fig. 11.5. Steel forms are used when long lengths of deck are to be cast in stages and the shutters are re-used many times. With timber forms it is easier to have square or angled corners and flat faces while steel forms are able to incorporate curved corners and sides.

Casting the deck section in several stages simplifies the formwork. This also makes the concreting operations much simpler and easier to control. The

@Seismicisolation

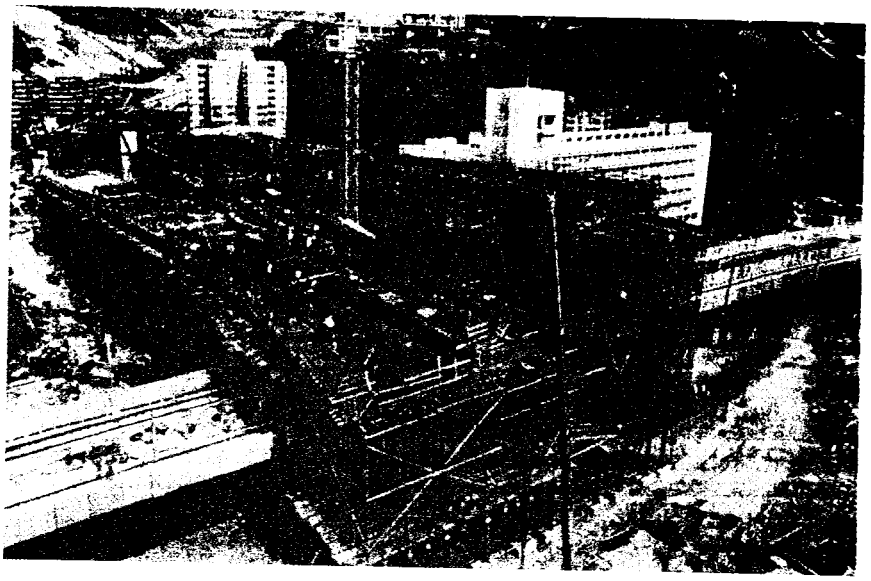


Figure 11.4. Full-height scaffolding. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

bottom slab, outer webs and diaphragms are cast first, followed by the inner webs and top slab soon after. The time delay between castings should be kept to a minimum to reduce any early thermal and differential shrinkage effects.

It is preferable to cast the outer webs with the bottom slab so that the construction joint is at the top of the web and hidden in the corner with the top slab. A construction joint between the bottom slab and webs is difficult to hide on the concrete surface and, although this is not important for the inner webs, it mars the appearance on the outer webs. The formwork

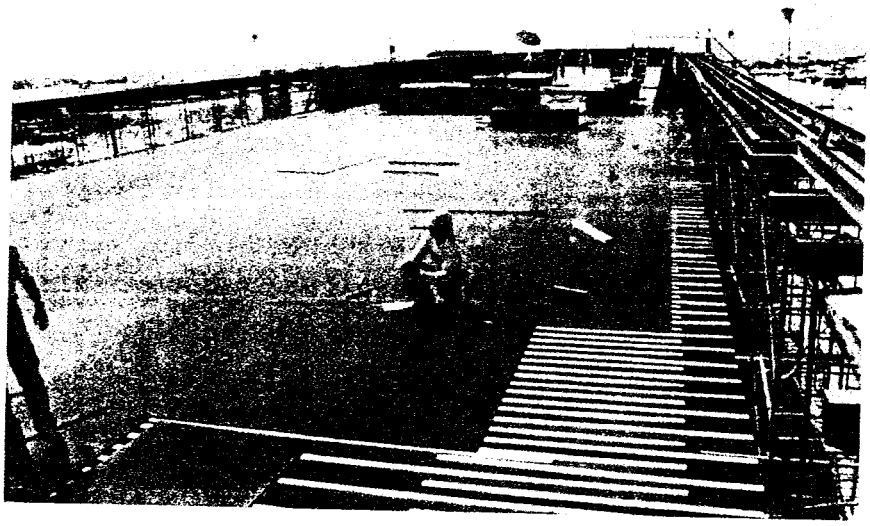


Figure 11.5. Formwork being installed.

@Seismicisolation

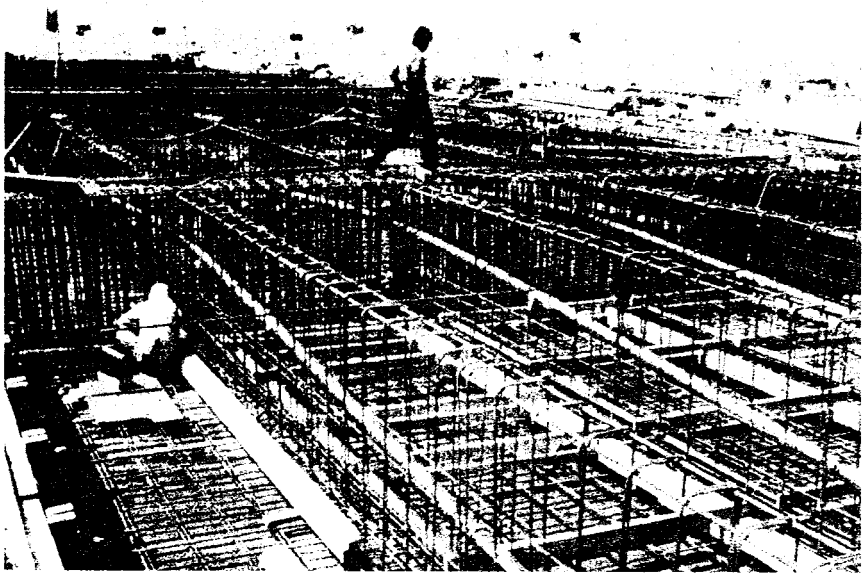


Figure 11.6. Reinforcement and tendon ducts.

for the inner webs and top slab is supported off the bottom slab concrete, simplifying the overall arrangement.

With the formwork in position, the next activity is the fixing of the reinforcement, prestressing ducts and anchorages. Figure 11.6 shows the reinforcement and ducts in place for concreting the bottom slab and outer webs. Short shutters are being installed along the bottom of the webs to form kickers when the webs are cast in the next stage.

Without the inner web and top slab formwork in place the access for placing, compacting and finishing the concrete in the bottom slab is improved. The subsequent concreting of the inner webs and top slab is done from above the deck without needing access to the void.

Figure 11.7 shows a multi-cell box after the bottom slab and webs have been cast and before installing the top slab formwork and reinforcement.

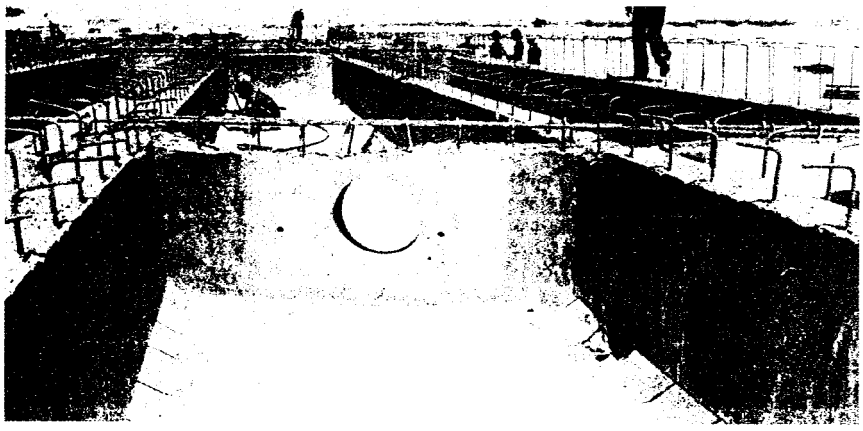


Figure 11.7. Bottom slab, webs and diaphragm cast.

@Seismicisolation

At this stage the deck is still fully supported by the falsework; which remains in place until the concreting is completed and the tendons installed.

Either permanent formwork panels or removable table forms are used between the webs to support the wet deck slab concrete. The removal of formwork from inside the voids, after the deck is completed, requires it to be broken down into small sections and passed out through the access holes in the diaphragms and bottom slab. Alternatively, a larger temporary access hole is left in the top slab at one end of the deck which is concreted after the rest of the formwork has been removed.

Longer bridge decks, extending over several spans, are usually cast in sections on a span-by-span basis, as illustrated in Fig. 11.8. This has several benefits including reducing the size of concrete pours to a more manageable quantity, optimizing the length of the prestress tendons and permitting the maximum re-use of the falsework and formwork. The first section cast is a complete span plus part of the adjacent spans to give short cantilevers. This moves the construction joints away from the highly stressed region at the pier and helps to balance the moments in the deck in the temporary and permanent situations. To optimize the overall moment distribution the construction joint is placed between the $\frac{1}{4}$ or $\frac{1}{3}$ points of a span. Subsequent sections of deck extend out from the construction joint over the next pier with a short cantilever, as before. This process is continued until the end of the deck is reached.

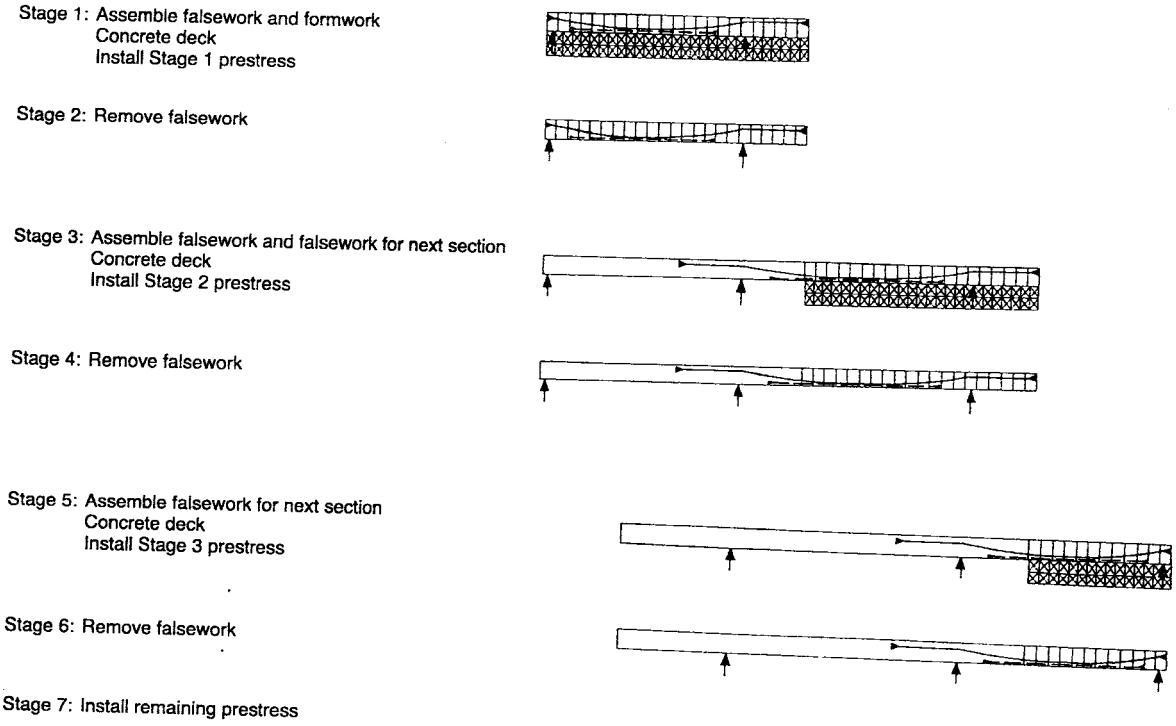


Figure 11.8. Stage-by-stage construction.

@Seismicisolation

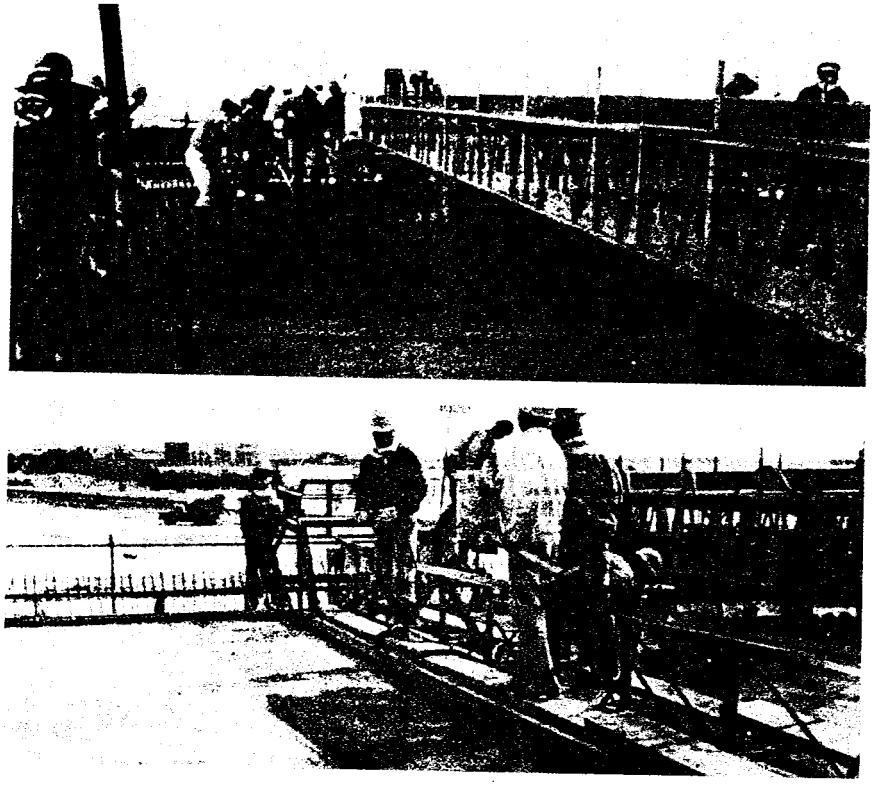


Figure 11.9. Deck finishing machine.

During concreting of the deck slab the level and finishing of the top surface has to be carefully controlled. On smaller decks this is achieved by placing levelling timbers on the reinforcement and screeding the concrete to the top of these. For larger areas of slab a finishing machine, as seen in Fig. 11.9, is used to assist in accurately levelling the top surface. The finishing machine runs along rails preset to the correct levels down either side of the deck. The machine is used to compact and float off the top surface of the concrete as well as to provide a working platform to give access over the completed deck for curing and covering the concrete.

When the concrete has attained the required strength the prestress tendons are installed and stressed. The deck tends to lift up along its span and reduce the load on the falsework as the prestress is applied. The falsework is removed after sufficient tendons have been stressed to carry the deck dead load.

If some parts of the deck deflect downwards when the prestress is applied this causes additional load on the falsework. This situation occurs when the deck is cast in stages using the span-by-span approach. The cantilever extending beyond the pier in the direction of construction tends to deflect down as the previous span is prestressed and lifts up. The falsework must be designed to take the increased load. The increase may be minimized by suitably arranging the prestress layout and stressing only the minimum number of tendons required to support the deck dead load before removing the falsework.

@Seismicisolation

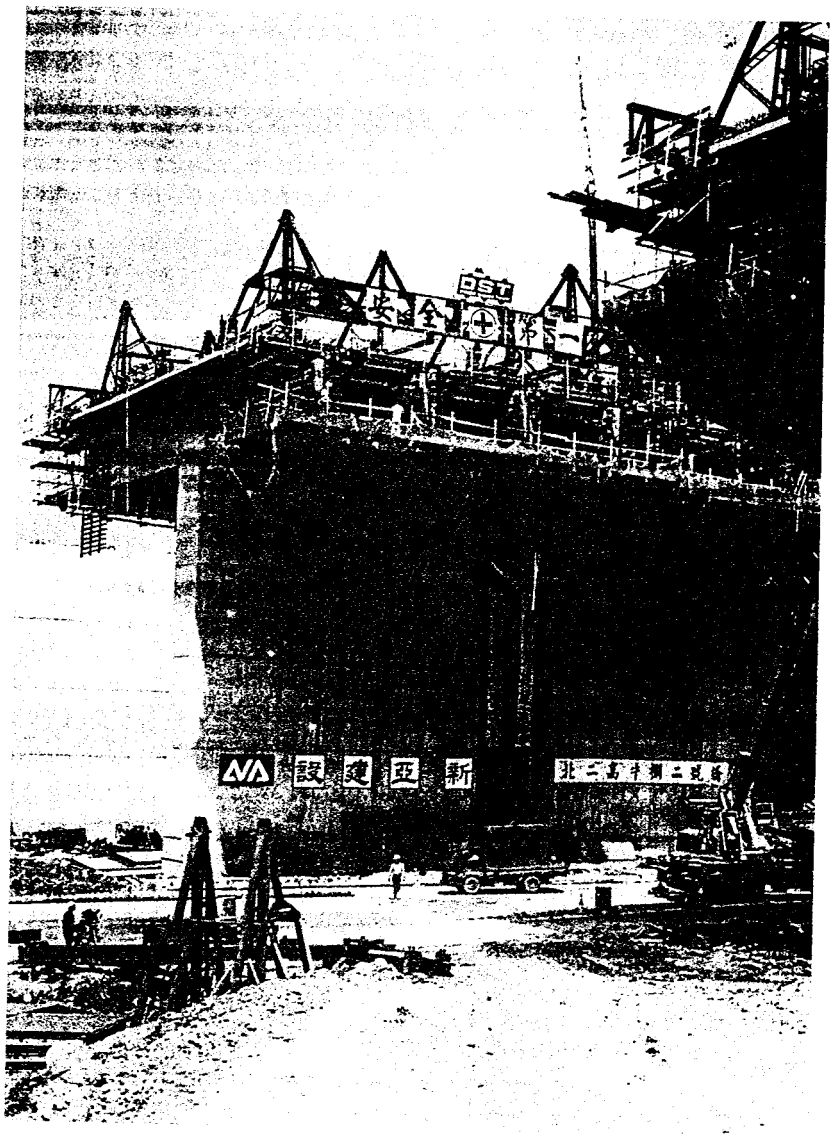


Figure 11.10. Balanced cantilever construction of multi-cell box. Reproduced courtesy of DYWIDAG-Systems International GmbH. Copyright reserved.

Where access is difficult, or the deck is at a high level, the formwork is supported from a truss spanning between the piers, similar to Fig. 10.13, or the deck built with a form traveller as a balanced cantilever as shown in Fig. 11.10.

A truss system is usually self-launching to move itself between the piers. To simplify the truss design, deck construction is on a span-by-span basis with the construction joint just beyond the pier diaphragm.

Design of *in situ* multi-cell box girders

Multi-cell box structures are analysed using a grillage model to derive the forces and moments in the webs and slabs, as discussed in Chapter 8. The

@Seismicisolation

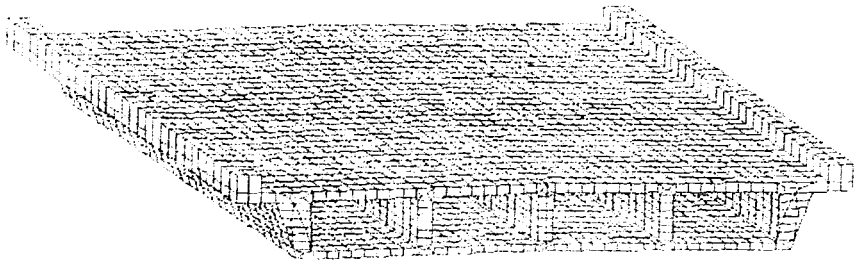


Figure 11.11. Three-dimensional finite element model of deck. Reproduced courtesy of Andy Hodgkinson. Copyright reserved.

local bending moments in the deck slab from wheel loads are often derived using Pucher charts (1976) [4] or similar design guides. The structural behaviour of this type of deck is described by Hambly (1991) [3].

Thin webs and slabs can significantly deform and distort under the imposed loading, and in this case the deck analysis is most accurately carried out using a three-dimensional finite element model with shell elements for each web and slab member, as illustrated in Fig. 11.11. The moments and shears in each element in each direction are taken directly from the analysis, and are used for both the longitudinal prestress design and the transverse reinforcement design.

For the longitudinal design the deck section is divided up into a series of 'I' beams consisting of a web and associated top and bottom slabs up to the mid-point of each cell. Prestress layouts are designed for each web to balance the stresses derived from the finite element or grillage analysis and to provide sufficient ultimate strength.

The prestress for long multi-cell box girder decks constructed in stages is arranged as illustrated in Fig. 11.12, with similar tendon profiles and arrangements in each web. In this example, the deck construction is shown starting with the span between piers 3 and 4, progressing outwards. Construction joints are approximately at the $\frac{1}{4}$ points along the spans.

The tendons at each stage are anchored in the webs at the 'free' end of the deck. Where the new section of deck meets the previously cast section at the construction joint the prestress is joined together using couplers, as described in Chapter 2. Alternatively, the prestress tendons are 'lapped' and anchored in blisters inside the deck.

Transverse reinforcement is provided in the section to cater for the transverse moments and shears expected in the webs and slabs. Additional reinforcement is required for the longitudinal shear in the webs and torsion in the box section. A typical layout of reinforcement for a multi-cell box is illustrated in Fig. 11.13.

With the deck fully prestressed longitudinally only nominal longitudinal reinforcement is required in the permanent condition. Early thermal effects within the concrete and differential shrinkage between the concrete pours may require additional reinforcement to control cracking in the temporary condition before the prestress is applied.

The diaphragms over the supports are designed as described in Chapter 6, while the intermediate diaphragm design is based on the forces and moments

@Seismicisolation

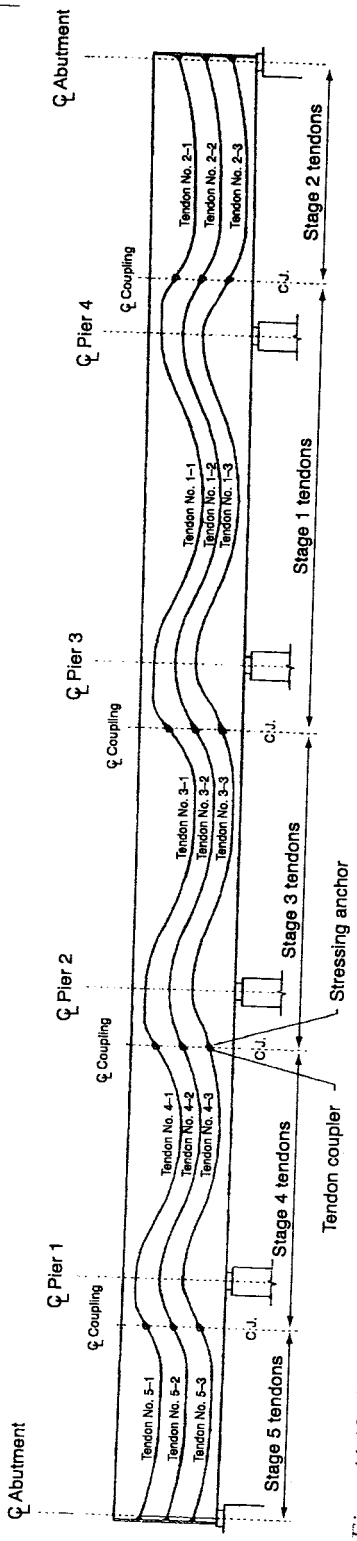


Figure 11.12. Longitudinal tendon profile.

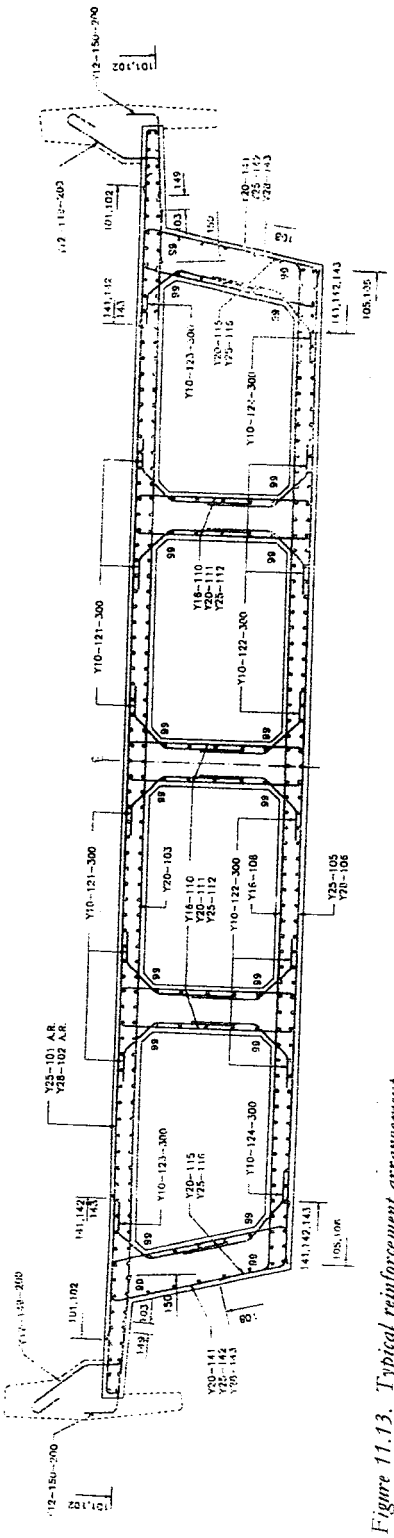


Figure 11.13. Typical reinforcement arrangement.

@Seismicisolation

generated from the grillage or finite element analysis, with reinforcement provided based on a reinforced concrete beam section.

References

1. Bragg, S.L., Ahm, P., Bowen, F.M., Champion, S., Kemp, L.C., Mott, J.C.S., Wilshere, C.J., Greville, W.E.J. and Hume, I.J. (1975) *Final Report of the Advisory Committee on Falsework*. London: HMSO.
2. The Concrete Society and IStructE (1971) *Technical Report TRCS 4, Falsework*. London: The Concrete Society.
3. Hambly, E.C. (1991) *Bridge Deck Analysis, 2nd edition*. London: Chapman and Hall Ltd.
4. Pucher, A. (1976) *Influence Surfaces of Elastic Plates, 5th edition*. Wien, Austria: Springer-Verlag.

@Seismicisolation

12

In situ single-cell box girder bridges

Introduction

Single-cell box girders cast *in situ* are used for spans ranging from 40 m up to 270 m. The box arrangement is considered to give a good aesthetic appearance; with the web of the box in shadow producing a slender appearance when combined with a slim parapet profile.

Single box arrangements are efficient for both the longitudinal and transverse designs, and they produce an economic solution for most medium- and long-span structures. The box structure is well suited for curved decks, and it is able to cope with complex highway geometry.

This type of deck is constructed span-by-span, using full-height scaffolding or trusses, or as a balanced cantilever using form travellers. The post-tensioned concrete box girder spans for the Hong Kong MTR Island Line, shown in Fig. 12.1, were cast *in situ* on falsework with span-by-span construction.

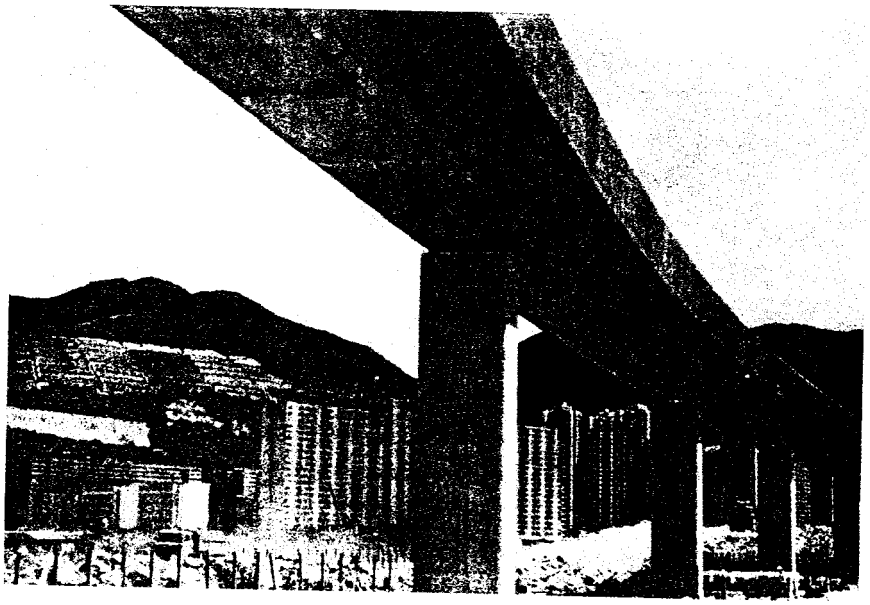


Figure 12.1. MTR Island Line, Hong Kong.

@Seismicisolation

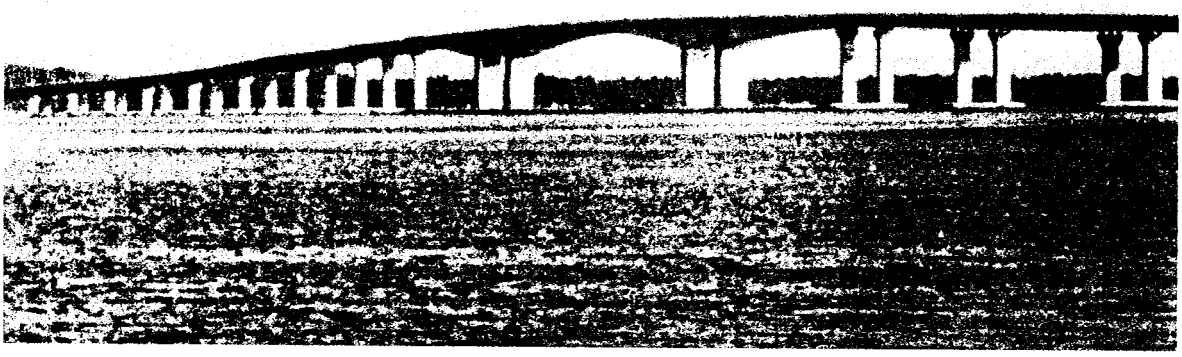


Figure 12.2. Malaysia–Singapore Second Crossing. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

Balanced cantilever construction is suitable for the longer spans and where access is restricted. For the Malaysia–Singapore Second Crossing, shown in Fig. 12.2, the 165 m long main navigational span and adjacent side spans were cast *in situ* as balanced cantilevers while the shorter spans, on the approach viaduct, utilized precast segments. Precast segmental construction produced the most economic solution for the repetitive spans of the approach viaduct, whereas the 'one-off' nature of the longer main span favoured *in situ* construction.

The advantages of *in situ* single-cell box girders are their efficient use of concrete, reinforcement and prestress, flexibility in span arrangements and good aesthetics. Their disadvantages include the labour-intensive site activities and the long construction times needed for the larger structures.

General arrangement

Constant depth sections are adopted on spans up to 70 m and with a span-to-depth ratio of approximately 20 : 1. For spans above 50 m a haunched profile is common with a typical span-to-depth ratio of 16 : 1 at the piers, increasing to 45 : 1 at mid-span. Between 50 m and 70 m the deck is either haunched or a constant depth throughout, depending on the site constraints and the designer's preference. The publication by Podolny and Muller (1982) [1] contains many early examples of concrete box girder arrangements used around the world.

Using a constant depth section throughout allows the deck details to be standardized, simplifying the construction. With a haunched profile the shallower deck at mid-span reduces the dead-load bending moments and shears in the deck, while the deeper section at the pier is more efficient resulting in an overall reduction in concrete and prestress quantities.

@Seismicisolation

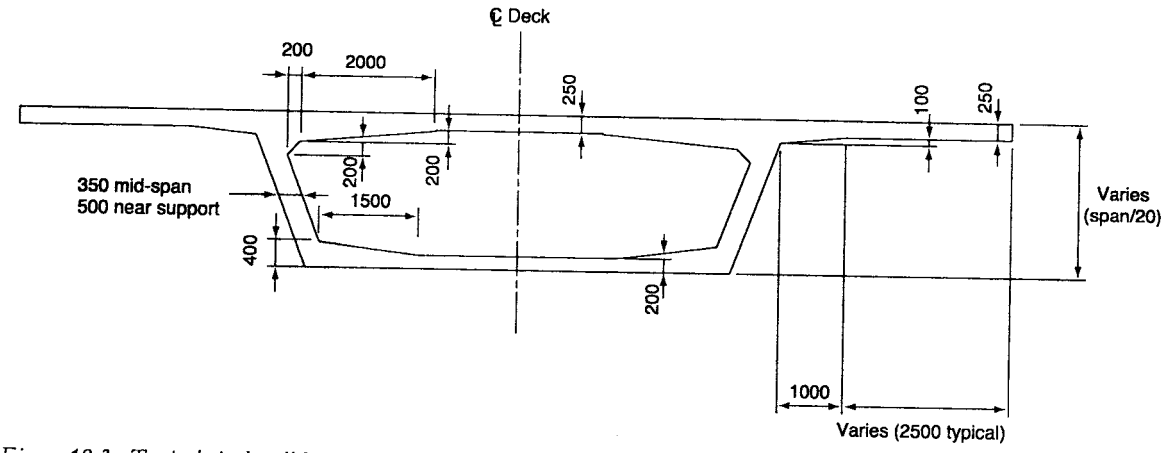


Figure 12.3. Typical single-cell box cross-section.

Figure 12.3 illustrates a typical section for a single-cell box girder. The width of the top slab is governed by the user's requirement in terms of highway, rail or footway layout. The side cantilevers are usually between 2 m and 3·5 m long, above which the slab design becomes less economic. It is preferable to slope the webs to improve the appearance of the box while the width of the bottom slab should be sufficient to provide transverse stability to the deck when overturning about the bearings is being considered.

Local bending and punching shear design under live load governs the top slab thickness. The punching shear from wheel loads generally requires a minimum slab thickness of 200 mm while transverse bending requirements dictate a deeper slab near the webs.

The web thickness is governed by the shear design from the longitudinal analysis. A minimum thickness of 350 mm is usually adopted for ease of construction, with the webs thickening adjacent to the piers to suit the ultimate shear requirements.

The bottom slab is usually a nominal thickness of between 160 mm and 200 mm over the mid-span region, but it may need to be thicker near the supports to cater for the compressive stresses generated by the longitudinal hogging moments and the torsional stresses in the box. Where large pre-stressing tendons are placed in the top or bottom slabs sufficient concrete depth is needed to provide adequate cover to the ducts.

With a single-cell box the deck width is typically in the range of 6 m to 16 m. Below a width of 6 m the resulting narrow box is difficult to construct and causes problems with stability of the deck at the supports. Above a width of 16 m the top slab arrangement becomes less efficient. For wider decks a multi-cell box, as described in Chapter 11, or a twin-box arrangement with the inner cantilevers joined together, as illustrated in Fig. 12.4, is often used.

Wider decks are achievable with a single-cell box by introducing additional support to the deck slab. Providing transverse ribs under the deck slab allows longer side cantilevers and wider spacing of the webs but requires a more complex shutter arrangement. The arrangement illustrated in Fig. 12.5, where struts are used to support the deck slab and transfer the load back to

@Seismicisolation

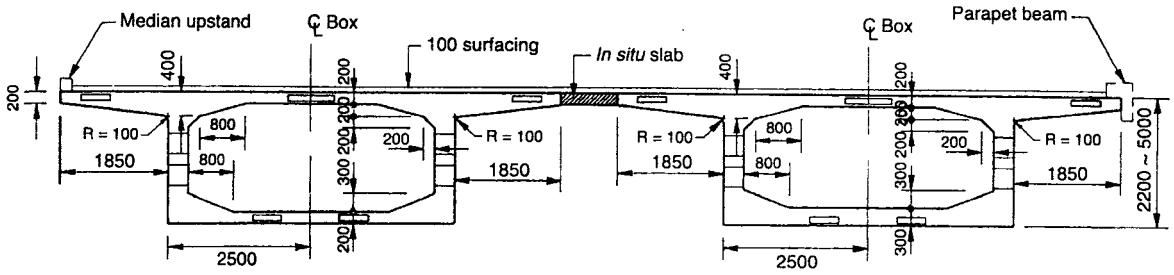


Figure 12.4. Twin-box arrangement.

the webs, has been used successfully on a number of projects in the UK and worldwide.

In the UK it is usual to design the top slab as a reinforced concrete element, while in Europe and elsewhere the use of transverse prestressing is common. Flat slab-type tendons are anchored in the cantilevers and extend over the full width of the top slab. This improves the durability of the structure by reducing the crack widths associated with a reinforced concrete design. However, transverse prestressing is usually only economic for wider decks.

Longitudinal prestressing consists of post-tensioned tendons, usually with multi-strand systems, but occasionally with bar systems as well. Both internal and external prestress tendons are widely used with the choice dependent on particular project requirements and the designer's preference. The advantages and disadvantages of internal and external tendons are discussed in 'Post-tensioning with internal or external tendons', Chapter 7.

With both span-by-span and balanced cantilever construction the deck is either supported on bearings at the supports or built integrally with the piers. Building the deck integrally eliminates the need for bearings and in the case of balanced cantilever construction reduces the need for temporary

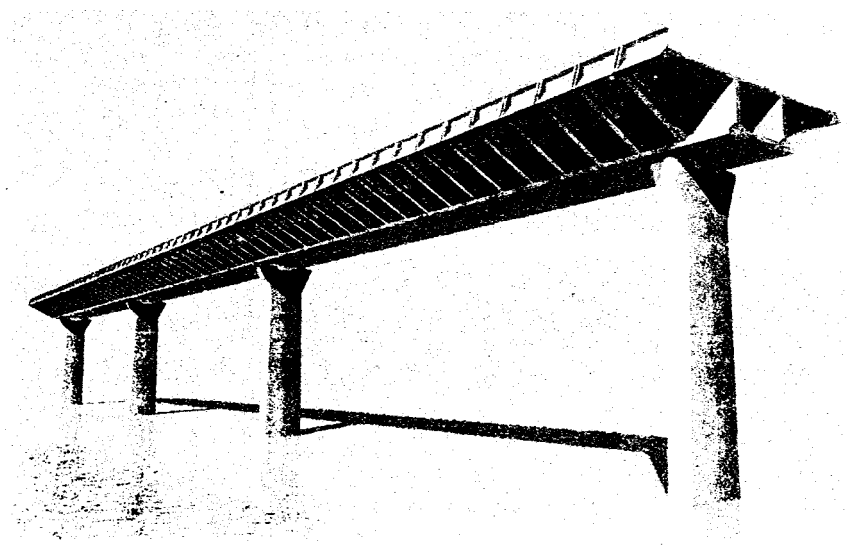


Figure 12.5. Single-cell box with struts supporting side cantilevers. Reproduced courtesy of Dissing and Weitling. Copyright reserved.

@Seismicisolation

supports. Integral construction is not always possible with long decks as the movements in the deck due to creep, shrinkage and temperature change may generate excessive forces in the substructure at the ends of the structure.

Many of the early balanced cantilever bridges built had hinges at the mid-span points to leave the deck acting as a series of propped cantilevers in its final form. However, several of these structures developed problems, with significant sag occurred in the spans. The use of hinges is seldom seen in modern bridges, with designers preferring to join the cantilevers with *in situ* stitches.

Construction, span-by-span

Simply supported spans are constructed full length before applying the prestress. For continuous decks, span-by-span construction involves casting a length of deck between vertical construction joints positioned in each span.

A typical span-by-span construction sequence for a continuous deck is similar to that for a multi-cell box illustrated in Fig. 11.8. The first stage cast consists of a full span plus a short cantilever, with subsequent stages extending out over the adjacent piers. The length cast is usually between the $\frac{1}{4}$ or $\frac{1}{3}$ points in adjacent spans to optimize the bending moments and prestress requirements during construction and in the final structure. It is also preferable to place transverse construction joints away from the piers where the maximum moments and shears occur. After concreting a span the prestress is installed allowing the falsework to be lowered and moved forward ready for the next span to be constructed.

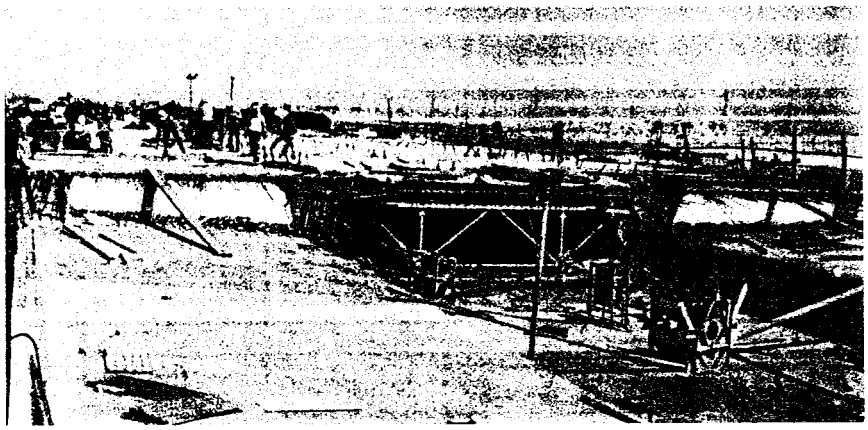
Where good access is available beneath the deck, full-height scaffolding is used to support the formwork, as shown in Fig. 12.6, although this becomes expensive if the deck is at a high level. With full-height scaffolding it is possible to work on several spans at the same time to speed up the rate of construction.

For high-level decks, or where access is difficult, the formwork is supported off trestles or by a truss or gantry spanning between the piers, as shown in Fig. 12.7. Trestle and steel beam arrangements are able to span over small obstructions such as rivers, roads and rail lines, and reduce the amount of scaffolding needed. They are usually quicker to erect and dismantle than full-height scaffolding.

Gantry systems are normally self-launching, moving forward across the tops of the piers after each deck section is completed. This results in more rapid construction than either full-height scaffolding or trestle arrangements, and gantries are popular when long lengths of viaducts are being constructed.

The deck section is usually cast in two stages with the bottom slab, webs and diaphragms cast first and the top slab cast soon after. After setting up the outer formwork, the reinforcement and prestressing tendon ducts are fixed, as seen in Fig. 12.8. The internal formwork for the webs and diaphragms is then placed. A short width of shutter is placed along the top of the bottom slab adjacent to the webs to prevent the web concrete from slumping. However, it is not necessary to provide shutters over the rest of the bottom slab.

@Seismicisolation



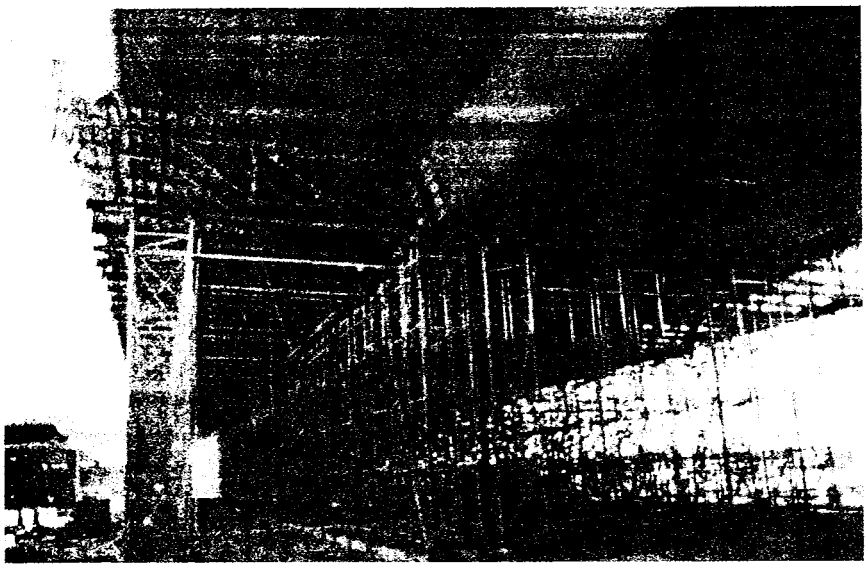


Figure 12.6. Full-height scaffolding from the ground.

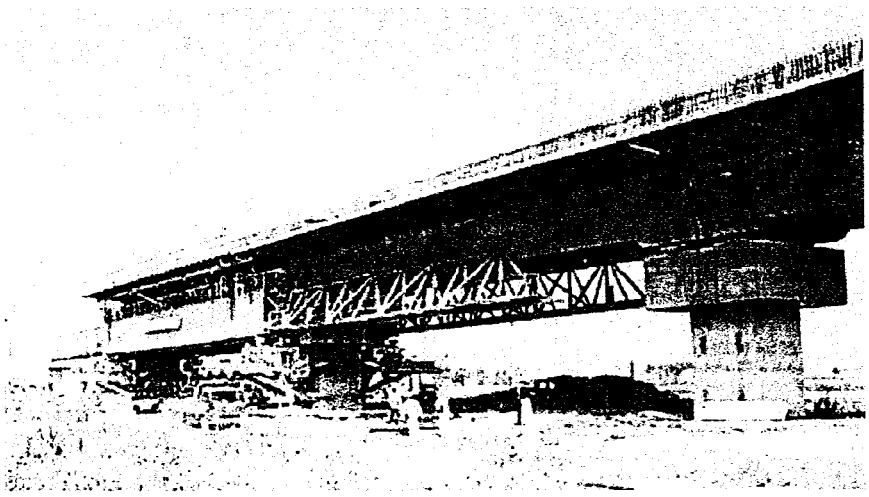


Figure 12.7. Gantry support for formwork. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

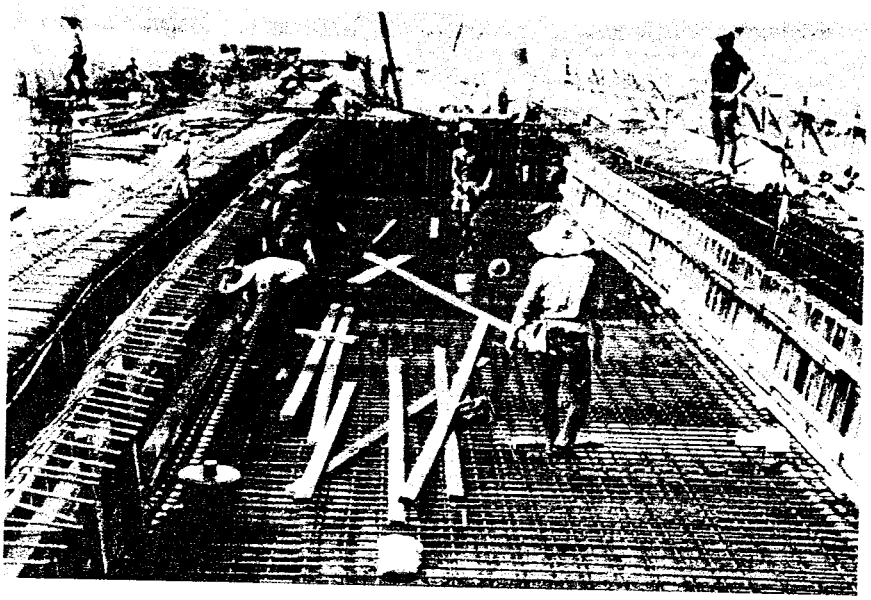


Figure 12.8. Reinforcement, ducts and web shutters being placed.

A separate internal form is used for the top slab between the webs. With the bottom slab and webs already cast, the top slab shutter is usually arranged as a 'table' form supported on the hardened concrete, as seen in Fig. 12.9. After the top slab concrete has gained strength the table form is lowered and moved forward for the next section of deck.

The time lapse between casting the webs and the top slab should be kept to a minimum to reduce any potential early thermal and differential shrinkage effects between the two concrete pours.

A typical construction cycle takes approximately 4 to 6 weeks per span, depending on the span length, the rate of strength gain in the concrete and the time to fix the reinforcement and prepare for concreting.

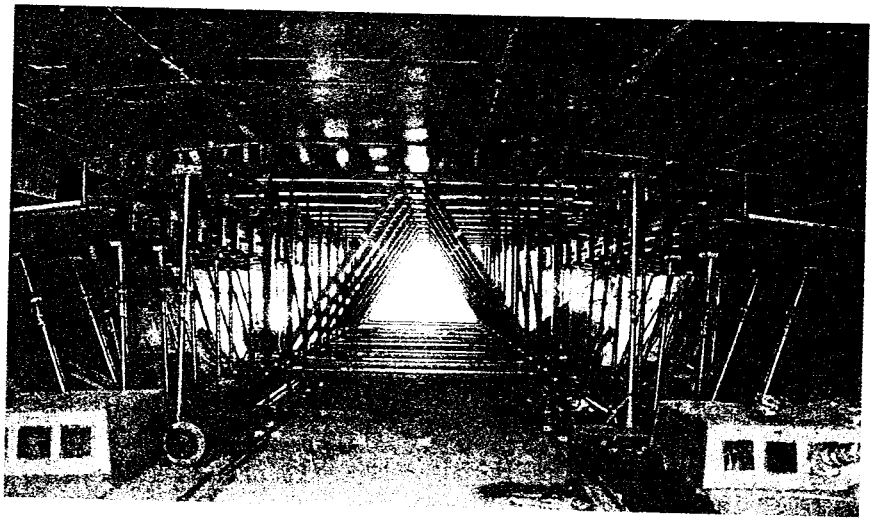


Figure 12.9. Shutter for top slab between webs.

@Seismicisolation

The advantages of span-by-span construction are the ability to arrange the prestress in an efficient manner and the relatively fast construction that is achieved when compared to the balanced cantilever method. The disadvantages are that it requires a lot of falsework, formwork and labour resources to achieve a good construction rate.

Construction by balanced cantilever

With this technique the deck is cast in short lengths within a pair of form travellers as a balanced cantilever about a pier, as shown in Fig. 12.10. This form of construction is particularly suited to long spans and where access beneath the deck is difficult.

The sequence of construction is illustrated in Fig. 12.11. The deck is built out from each side of each pier in a balanced sequence to minimize the out-of-balance forces on the piers. When the cantilevers from adjacent piers have been completed and face each other at mid-span the remaining gap is concreted and the prestress installed to make the deck continuous. The end span length is usually between 60 per cent and 65 per cent of the adjacent span length and the 'balanced' cantilever from the last pier is extended out until it reaches the abutment. Alternatively, this out-of-balance length of deck is supported by temporary props in front of the abutment, or the end section of deck is cast on falsework to support the concrete until the prestress is installed and the span completed.

A section of deck is initially constructed above each pier to enable the form traveller to be installed. This pierhead section is cast on falsework or temporary supports fixed to the pier. Figure 12.12 shows the pierhead section in the background being constructed on falsework with the one in the foreground completed and the frame for the form traveller being assembled. In this example the deck is built integral with the main piers which resist the out-of-balance moments from the cantilever construction without the need for any further temporary works. The twin-leaf pier arrangement provides the necessary moment restraint during construction without being too stiff along the line of the deck, as this would set up large moments and shears in the permanent structure due to the deck movement.

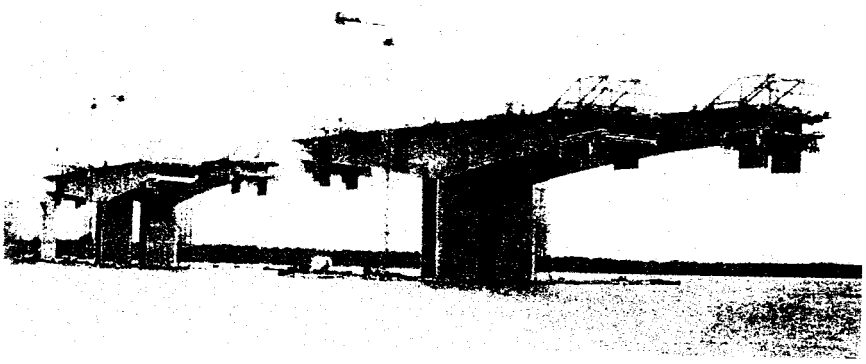


Figure 12.10. Balanced cantilever construction. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

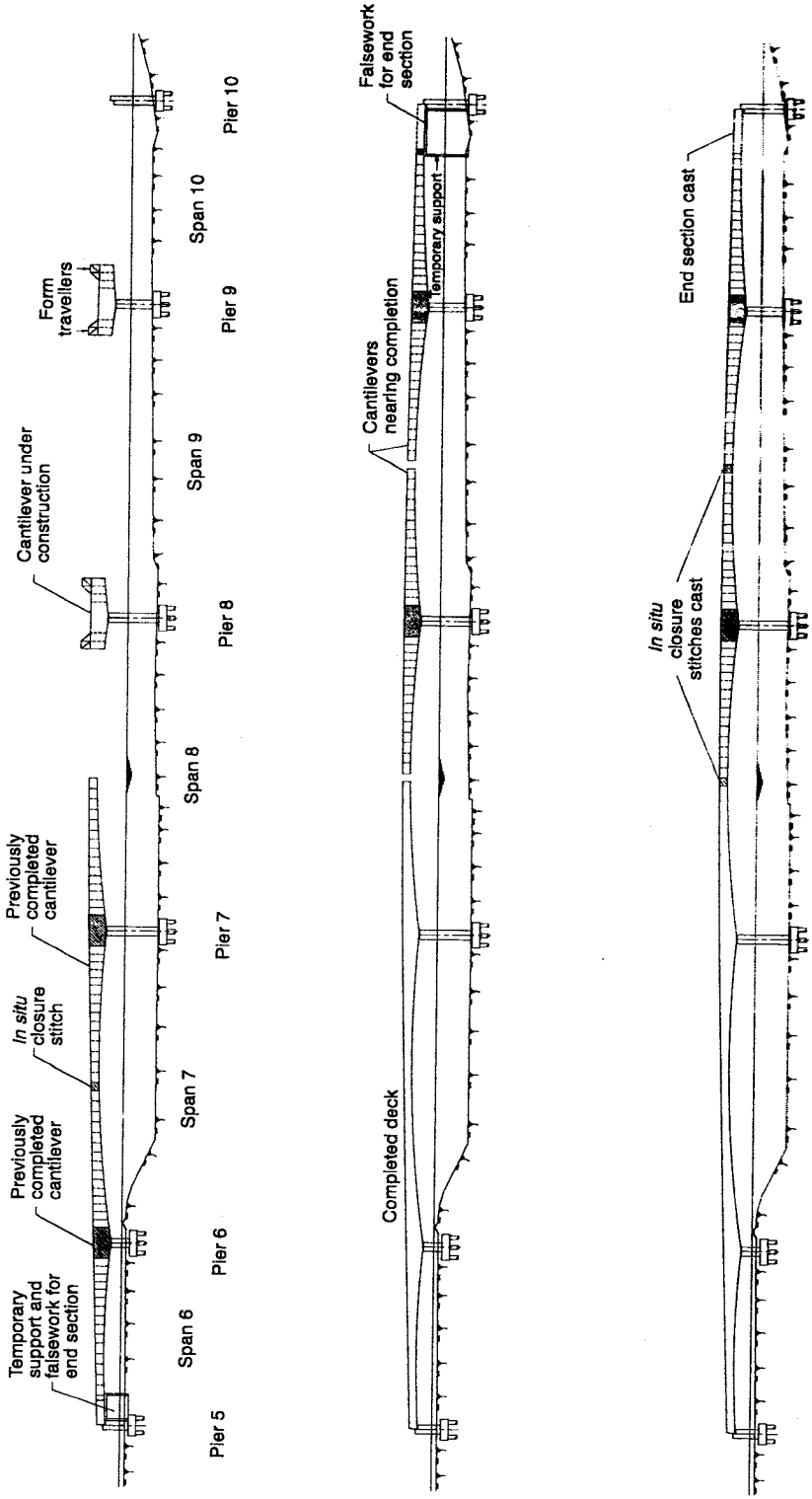


Figure 12.11. Balanced cantilever construction of deck. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

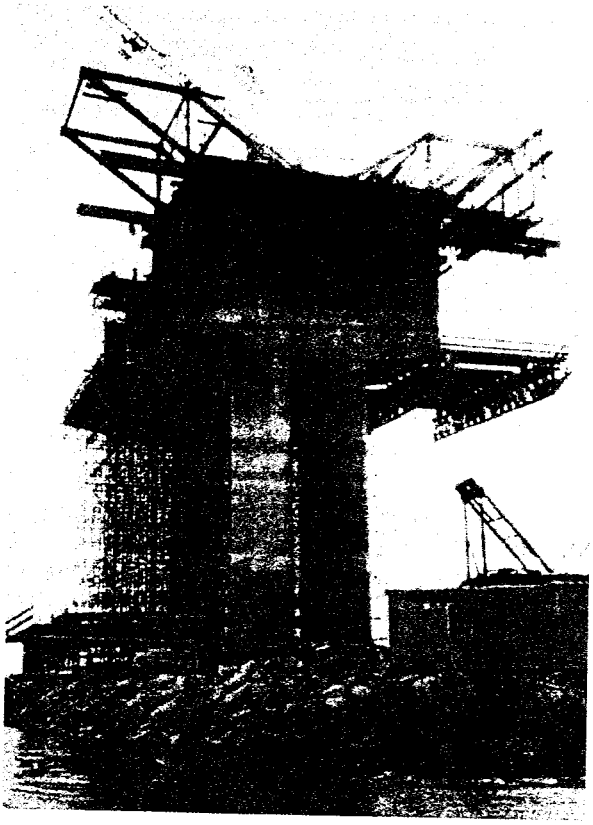


Figure 12.12. Pierhead and setting up of the traveller.

The form traveller consists of a series of shutters that hang from the support frame, as illustrated in Fig. 12.13. The soffit shutter and inner web shutters are adjustable to cater for a varying depth section. The webs are shown vertical which simplifies the shutter arrangement. Sloping webs are

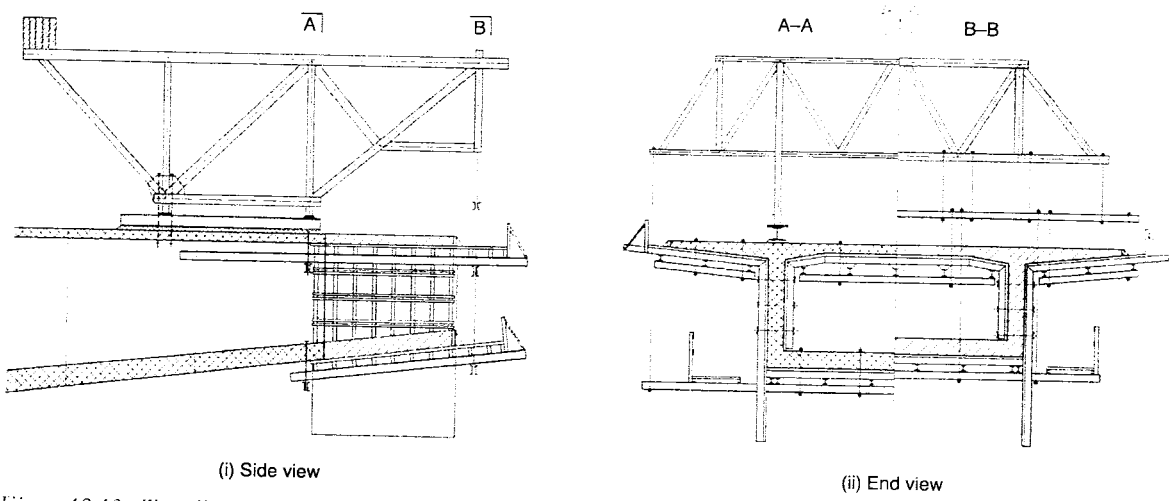


Figure 12.13. Travelling form arrangement. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

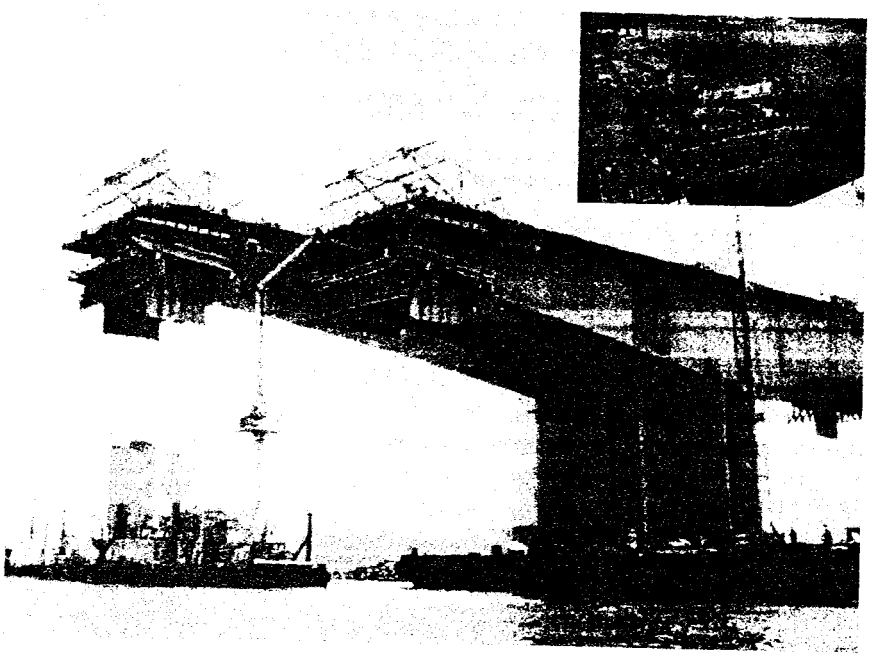


Figure 12.14. Travelling form on deck.

also used, although this complicates the formwork arrangement with the soffit shutter varying in width as the segment height changes. Temporary access is provided between the top of the deck and the void, and working platforms incorporated around the traveller.

The frame of the traveller is bolted down to the previously completed deck. Shown in Fig. 12.14, the support frame carries the weight of the shutters and the wet concrete until the concrete has hardened and the prestress 'cantilever' tendons are installed. The shutters are then stripped from the concrete face and the frame released from the deck to allow it to be slid or rolled forward to construct the next segment. With the form traveller set up, the reinforcement and tendon ducts are placed and the concrete cast.

Cantilever segments are typically between 3 m and 5 m long with the complete cross-section cast at one time. When the concrete has reached sufficient strength, prestressing tendons are installed in the top slab to support the cantilevering deck. Concrete with an early high strength is needed to be able to install the prestress as soon as possible and to achieve a typical cycle time of between 7 or 8 days per segment.

Construction times are reduced by prefabricating sections of the reinforcement cages and lifting them into the travelling form, or by precasting the prestress anchor region so that the prestress may be installed sooner.

As an alternative to using a form traveller, several projects have utilized a temporary overhead truss to support the formwork. Spanning between the piers and the previously completed deck, it moves the formwork forward in stages. When the cantilevers are completed the truss launches forward to the next pier. This is usually suitable only for spans up to 100 m and is generally less economic than using form travellers.

@Seismicisolation

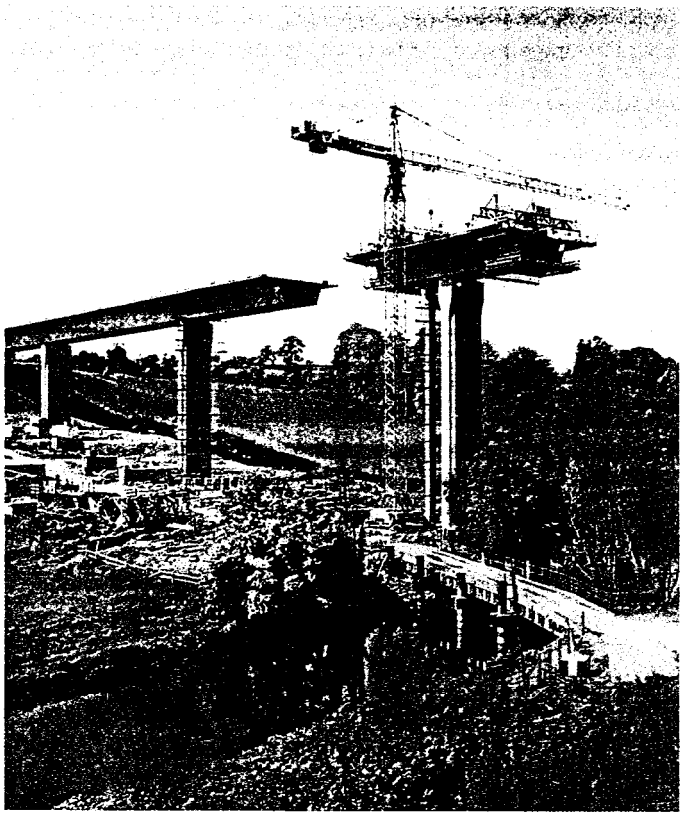


Figure 12.15. Balanced cantilever construction with bearings. Reproduced courtesy of DYWIDAG-Systems International GmbH. Copyright reserved.

At the pier head, the deck is either built into the pier or is placed on bearings to allow rotational and horizontal movements. If the deck is built in at the pier, as shown in Fig. 12.12, this provides the support during the construction on the cantilevers and greatly simplifies the temporary works required. When placed on bearings the deck is held by temporary supports or a tie-down arrangement between the deck and the pier to maintain stability while the deck is being cantilevered. Figure 12.15 shows a balanced cantilever under construction with a prop supporting the deck adjacent to the pier. When the cantilever is completed and joined to the adjacent deck the temporary supports are dismantled to allow the deck to move relative to the pier as it expands, contracts or rotates.

The advantages of the balanced cantilever method are the minimal formwork is used with a high degree of repetition, while propping and other temporary works are also reduced or eliminated. The disadvantage is its slow rate of construction.

Design of *in situ* single-cell box girders

For a single-cell box girder the longitudinal and transverse designs have traditionally been considered separately although it is now more usual to establish three-dimensional finite element models that combine the effects.

@Seismicisolation

The general design, including the creep effects of the dead load and prestress, and the stage-by-stage construction is described in Chapters 5 and 8. The design of the details associated with box girders is given in Chapter 6.

Box behaviour

Thin-walled box girders distort or warp when subjected to torsion and applied loading. This causes a redistribution of both the longitudinal and transverse forces and stresses in the box. These effects are fully described in the C&CA Technical Report on concrete box beams by Maisel and Roll (1974) [2], and they are most accurately determined by using a full-length three-dimensional finite element model for the design.

Shear lag is not usually a problem for normally proportioned boxes, but it may become significant for wider or slimmer decks. In these cases, the shear lag will modify the stress distribution across the section and the peak stresses are used in the serviceability longitudinal stress check.

Prestress layout

The prestress arrangement is governed by the construction method and the stresses generated in the deck during both the construction stages and in the permanent structure.

With span-by-span construction the prestress is also arranged on a span-by-span basis, with a layout similar to that for multi-cell boxes, as illustrated in Fig. 11.12. The tendons are anchored at each construction joint and either coupled or overlapped for subsequent stages. The prestress in each span is either fully installed as each stage is completed, or only the minimum number of tendons needed for construction are stressed with the remainder installed at the end.

With balanced cantilever construction there are three distinct sets of prestressing tendons, as illustrated in Fig. 12.16. First are the 'cantilever tendons' that are placed in the top slab and installed as the deck is being cantilevered out. The second are the 'bottom tendons' placed in the bottom slab to take the sagging moments over the mid-span region, and the third are the 'continuity tendons' which usually follow a draped profile and extend over several spans. In Fig. 12.16 the continuity tendons are shown as external tendons, but they could be internal tendons if preferred by the designer.

With both span-by-span and cantilever construction the prestress tendons are often anchored on the end face of a section. Anchored on either the slab or webs, the design follows the procedure described in 'Post-tensioned tendons', Chapter 6, with bursting, spalling and equilibrium reinforcement being provided. When the next section of deck is cast the anchors become fully encased in the concrete.

Transverse prestressing of top slab

Prestressing the slab transversely as well as longitudinally improves the durability of the structure by reducing crack widths. However, it is usually more costly than the reinforced alternative for all but the wider decks. A

@Seismicisolation

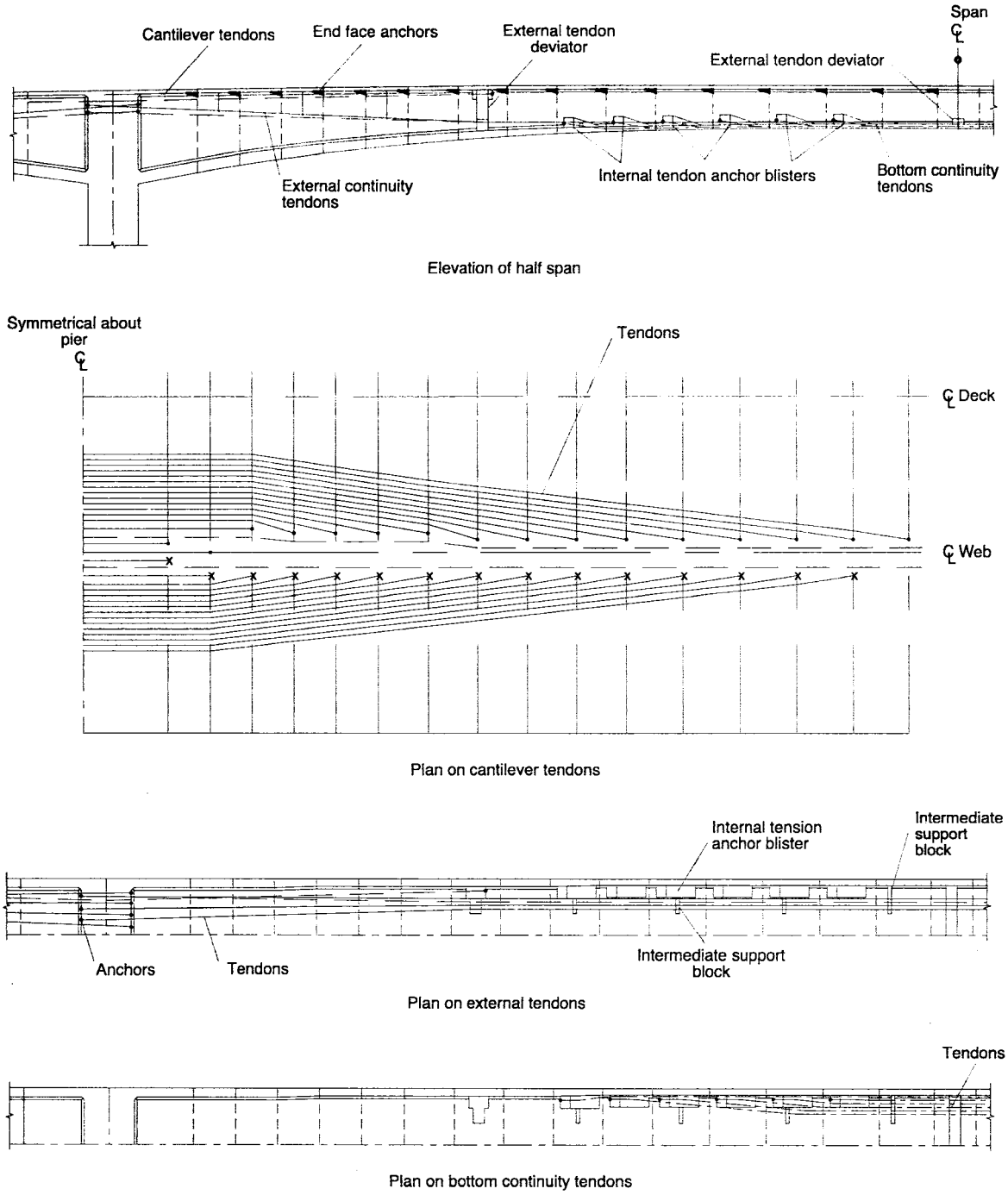


Figure 12.16. Tendon layout for balanced cantilever construction.

@Seismicisolation

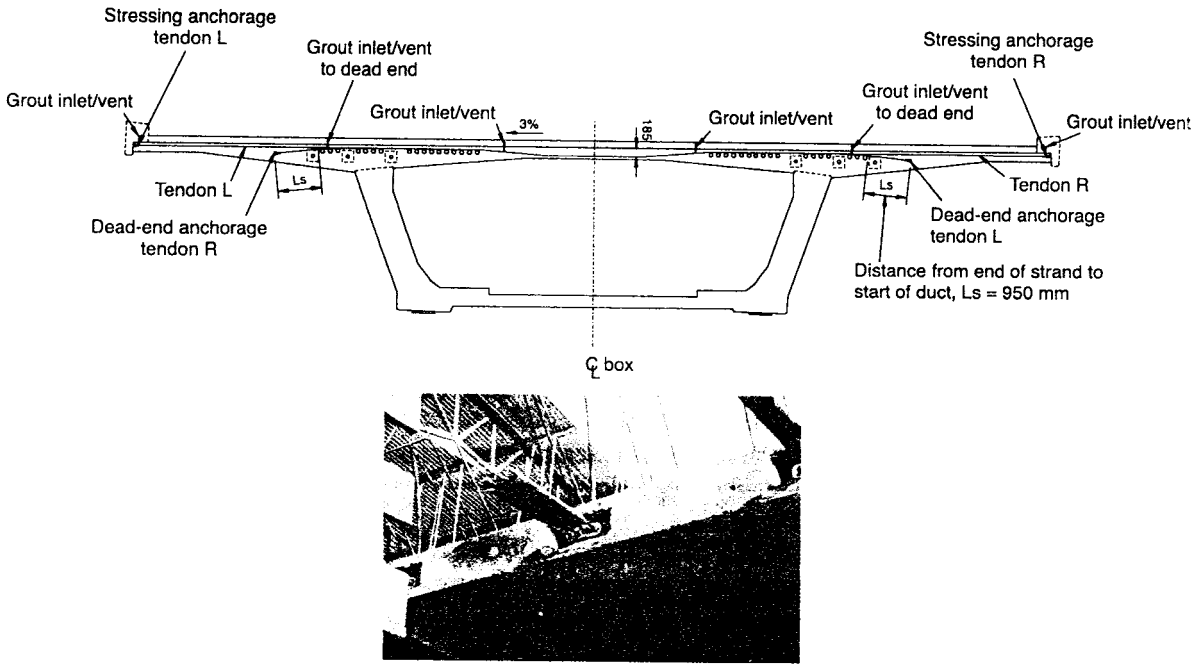


Figure 12.17. Transverse tendons profile and anchorage. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

typical transverse tendon profile is illustrated in Fig. 12.17, with the tendons anchored at the end of the box side cantilevers or with dead-end anchors within the slab. The transverse tendons pass over the top of the longitudinal tendons and flat slab tendon systems are used to take up the minimum space in the top slab.

The top slab is designed as a normal prestressed element with the primary and secondary effects taken into account. Additional tendons are needed over the diaphragms to counter the truss action from the bearing loads and diaphragm behaviour.

Deck articulation

The choice of supporting the deck on bearings or building it integral with the substructure is a function of the design and construction method used. Placing the deck on bearings simplifies the design of the permanent structure, while building the deck into the pier provides stability during balanced cantilever construction, eliminates the bearings and reduces the future maintenance of the bridge.

An integral connection is usually feasible only for piers where the deck movements from shrinkage, creep and temperature are small. Where large deck movements occur the moments and shears in the piers become excessive; although making the pier more flexible reduces the effect. A twin-wall pier arrangement with a gap in between, as seen in Fig. 12.14, provides the necessary moment restraint to the deck while minimizing the stiffness in the longitudinal direction. This type of arrangement is often used on long-span bridges where the scale of the structure suits the appearance of the twin-wall pier.

@Seismicisolation

A similar effect is achieved using two rows of bearings on each pier. The bearings provide rotational restraint while allowing horizontal movement. Either pot or rubber bearings are used with the stiffness chosen to optimize the design effects.

Deck construction

The stresses in the deck and substructure are checked at each stage of construction and with the most critical loading arrangement applied. The stability and strength of the deck and substructure are also checked to ensure that adequate factors of safety are achieved. Temporary props or tie-down arrangement at the piers have to cater for all the out-of-balance forces likely to occur. This is particularly relevant for balanced cantilever construction, but span-by-span construction must also be checked.

For decks constructed *in situ* by the balanced cantilever technique, the temporary loading on the deck includes the following:

- form traveller at the end of the cantilever (typically between 400–1200 kN for spans between 50 m and 200 m);
- construction equipment (typically taken as 10–20 kN on the cantilever tip);
- construction live load (typically taken as 0·5 kN/m^2) applied to the deck to give most adverse effect;
- wind loading on the deck including 'upward' wind under one of the cantilevers.

In addition, it is usual to consider an out-of-balance dead load, with the concrete 2·5 per cent heavier on one side of the balanced cantilever.

References

1. Podolny, W. and Muller, J.M. (1982) *Construction and Design of Prestressed Concrete Segmental Bridges*. New York: John Wiley & Sons.
2. Maisel, B.I. and Roll, F. (1974) '*Methods of analysis and design of concrete box beams with side cantilevers*', London: C&CA Technical Report.

@Seismicisolation

13

Precast segmental box girders

Introduction

The technique of precasting concrete box girders in short segments and transporting them to site to be lifted into position is well established, and it offers many benefits on suitable projects. On early bridges, constructed using this technique, the precast segments were connected with a narrow *in situ* concrete or mortar joint while more recently the segments are usually match-cast in a factory and joined on site with only a thin layer of epoxy between them. Several projects in the USA and Asia have used match-cast segments erected without any epoxy in the joints, a technique known as 'dry-jointed'.

The UK's first use of precast match-cast segments with epoxy joints was the Byker Viaduct, shown in Fig. 13.1. Completed in 1978 the viaduct carries the Newcastle metro over the Byker valley, and was built by a combination of balanced cantilevering and progressive placing. The segments were placed using both a crane and a lifting frame.

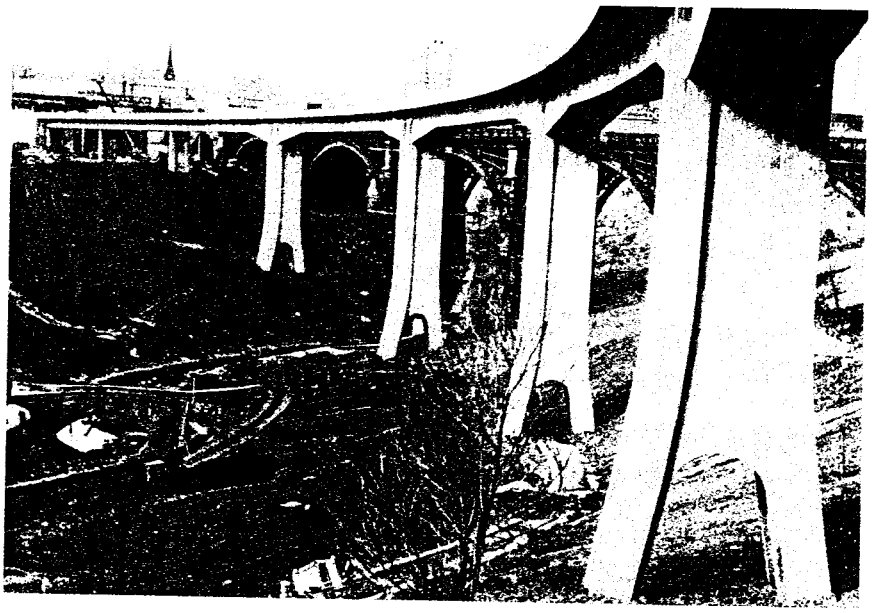


Figure 13.1. Byker Viaduct, England.

@Seismicisolation

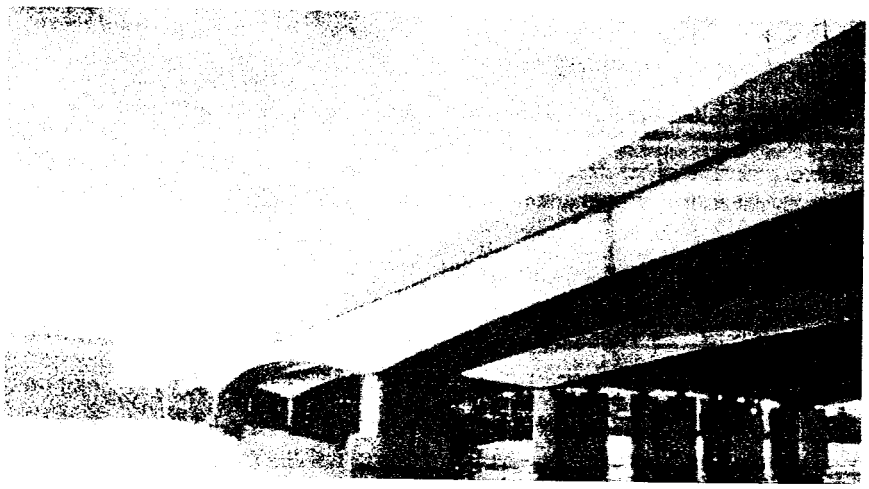


Figure 13.2. Belfast Cross-Harbour Links, Northern Ireland. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

The Belfast Cross-Harbour Links, shown in Fig. 13.2, carries both road and rail traffic through the heart of Belfast, and it was built with precast segments using the balanced cantilever technique. The design was developed during a competitive design-and-build tender and provided an aesthetically pleasing as well as economic solution. The box girder segments were lifted into place by crane before being glued together.

Dry joints were pioneered in the USA, and their first use in Asia was on the Bangkok Second Expressway System (BSES), built in 1990 and shown in Fig. 13.3. Since then dry-jointed decks have been used on many projects in the region. The first section of viaduct constructed on the BSES included over 60 kms of precast segmental deck designed and built in a two-and-a-half-years' period. Large predicted differential settlements between the piers led to a simply supported deck arrangement with the segments erected on a

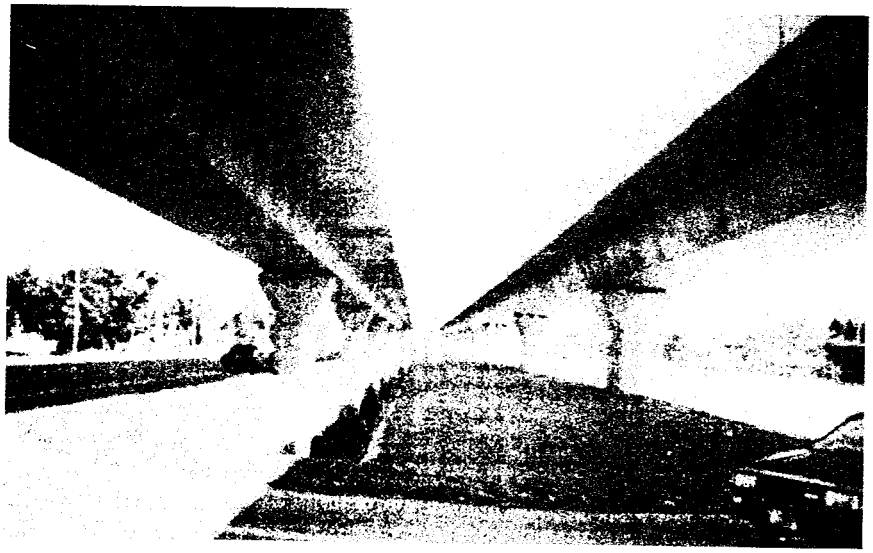


Figure 13.3. Bangkok Second Expressway System, Thailand. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

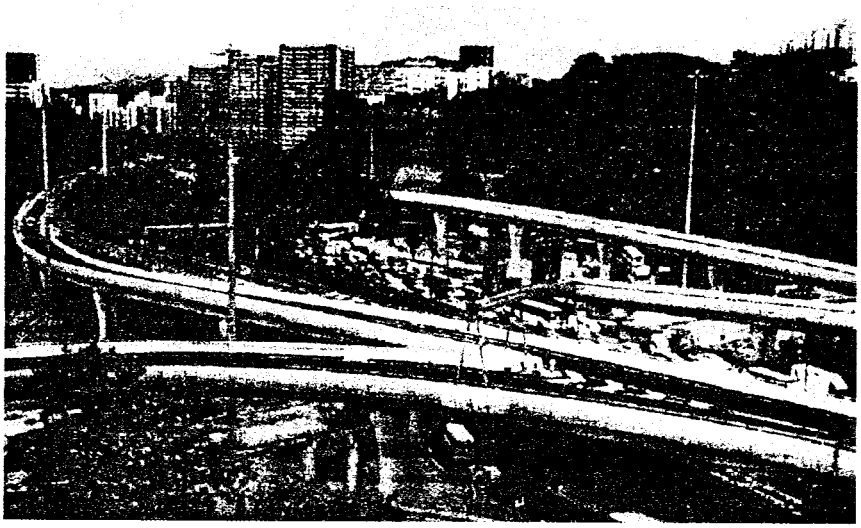


Figure 13.4. Hung Hom Bypass and Princess Margaret Road Links, Hong Kong. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

span-by-span basis using either overhead or underslung self-launching trusses. External prestressing tendons and dry joints were used to achieve a very rapid rate of erection.

Hung Hom Bypass and Princess Margaret Road Link in Hong Kong includes over 6 kms of viaduct built with complex geometry over some of the most congested roads in the city. The viaduct passes over part of the harbour and a busy mainline railway as well as the existing highway network. Using precast segments minimized disruption to the existing infrastructure and allowed the construction to be standardized throughout with all the spans built as balanced cantilevers using a combination of an overhead erection gantry and cranes.

There have been many other notable bridges built around the world using precast segments. They have become a popular form of construction for long urban viaducts as well as major river and estuary crossings. Several cable-stayed bridges have utilized precast segmental concrete decks, such as the Sunshine Skyway Bridge in Florida with a 366 m main span.

The advantages of precast segmental construction include the economies from repetition and the high quality achieved through the factory production; as well as the rapid construction and the minimal disruption to site. The disadvantages include the high initial cost of the erection equipment and casting yard, and the high level of technology needed for both the design and construction.

General arrangement

Precast segmental box girder decks usually have single-cell arrangements with side cantilevers and are similar to the *in situ* box girder decks described in Chapter 12, with a typical section shown in Fig. 12.3. Top slab widths range typically from 6 m to 16 m with box widths between 3·5 m and 8 m.

@Seismicisolation

Wider decks are possible using a twin-cell box arrangement; although with three webs this complicates the casting cell design and operation.

Wider decks are also achieved by supporting the top slab transversely, either by struts, as seen in Fig. 12.5, or by transverse beams extending out from the webs. The Jahra-Ghazali Viaduct in Kuwait, shown in Fig. 13.5, used a single 8 m-wide box spine with transverse beams supporting the deck slab as it widened out to 40 m to carry the diverging carriageway at the access ramps.

Precast segments are used on box girder decks with span lengths from 40 m up to 150 m. Below 40 m other forms of construction are likely to be more economic, while for spans above 150 m the segment weights become excessive and using *in situ* balanced cantilever construction is usually preferred. For spans up to 70 m a constant depth section is used with a span-to-depth ratio of approximately 20 : 1; although between 50 m and 70 m the deck is sometimes haunched in profile. Spans above 70 m are usually haunched with span-to-depth ratios of 16 : 1 at the pier, increasing to 35 : 1 at mid-span.

Web and slab thickness are similar to the *in situ* single-cell box girders described in 'General arrangement', Chapter 12. The factory conditions for casting the segments make it practical to construct thinner webs and bottom slabs where the design allows.

Segment lengths are governed by the handling and transportation constraints. Lengths of up to 3·6 m are usually transportable on public roads without excessive restrictions being imposed by the highway authorities. Where no restraints exist, the segments are made as long as practical to reduce the overall number of segments to a minimum.

Segments weighing up to 65 tonnes are usually within the capacity of cranes available locally, while heavier segments may require special cranes and lifting equipment. The heaviest segment is usually the diaphragm segment and this is often reduced in length to match the weight of the other segments.

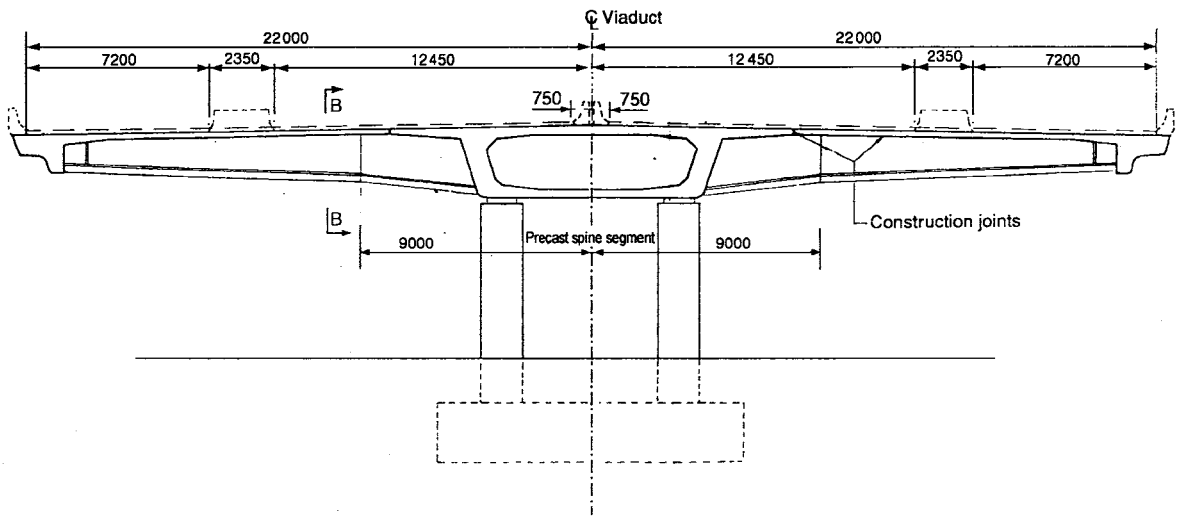


Figure 13.5. Jahra Ghazali Viaduct, Kuwait.

@Seismicisolation

Shear keys are formed on the concrete faces of the segment joints. They are detailed as either large or small keys as discussed later in this chapter ('Shear keys at joint'). The shears keys assist in lining up adjacent segments when they are erected to achieve the required horizontal and vertical alignment. The keys also transfer the shear across the joint during the erection process.

Diaphragms are used in the deck only at the pier and abutment positions to stiffen the box and transfer the deck load on to the bearings and substructure. A typical diaphragm arrangement for a box is shown in Fig. 6.16.

The publication by Podolny and Muller (1982) [1] contains many early examples of precast segmental box girder projects around the world, illustrating the deck arrangements and concrete details used.

Casting of segments

The segments are made in a purpose-built casting yard with special casting cells designed to allow a daily production of segments. These yards are often covered to protect the casting cells against the weather. A typical casting yard, as seen in Fig. 13.6, is divided into a number of areas for the different activities undertaken to construct the segments. A separate area is established for the assembly of the reinforcement in jigs, seen on the left side of the picture. The casting cell is shown in the right foreground, while the segment storage is usually to one side of the casting area. Concrete is often batched on site to ensure good quality control and an uninterrupted supply.

Segments are cast in specially designed steel formwork, using either the short-line or the long-line methods, as illustrated in Fig. 13.7. In the short-line method the segment is cast against its neighbour, and when hardened it is moved into the counter-cast, or conjugate, position to be cast against,

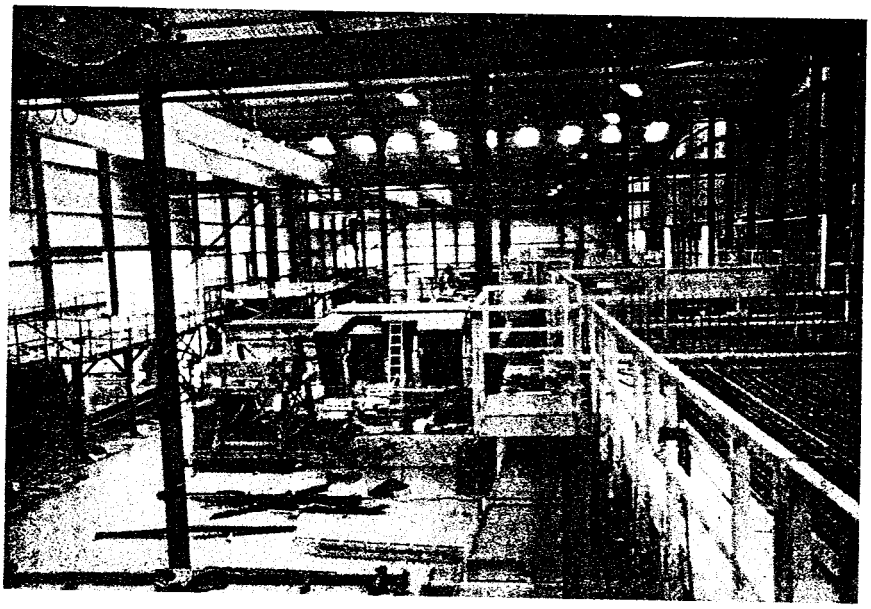


Figure 13.6. General view inside casting yard.

@Seismicisolation

after which it is moved to a storage area. With the long-line method the segments are cast one at a time in a long bed until all the segments in a section of span are finished.

For the short-line method the casting cell consists of a bulkhead, side and cantilever shutters, a soffit form and an internal shutter, as seen in Fig. 13.8. The bulkhead is rigidly fixed at one end of the cell, while the side shutters are able to drop away from the segment to assist in stripping. The soffit shutter supports the reinforcement cage during the concreting of the new segment and needs to be adjustable to set up the counter-cast segment to its required vertical and horizontal alignment.

The inner shutter, shown in Fig. 13.9, forms the central void in the box and has to cater for the formation of the prestress anchor blisters, diaphragms and deviators. The operation of the casting cell requires the inner shutter to be moved back out of the way to allow the segment just cast to be moved into the counter-cast position and the reinforcement for the next segment to be lifted into the casting cell. The inner shutter is designed to allow it to be removed, adjusted and repositioned on a daily cycle. It often incorporates

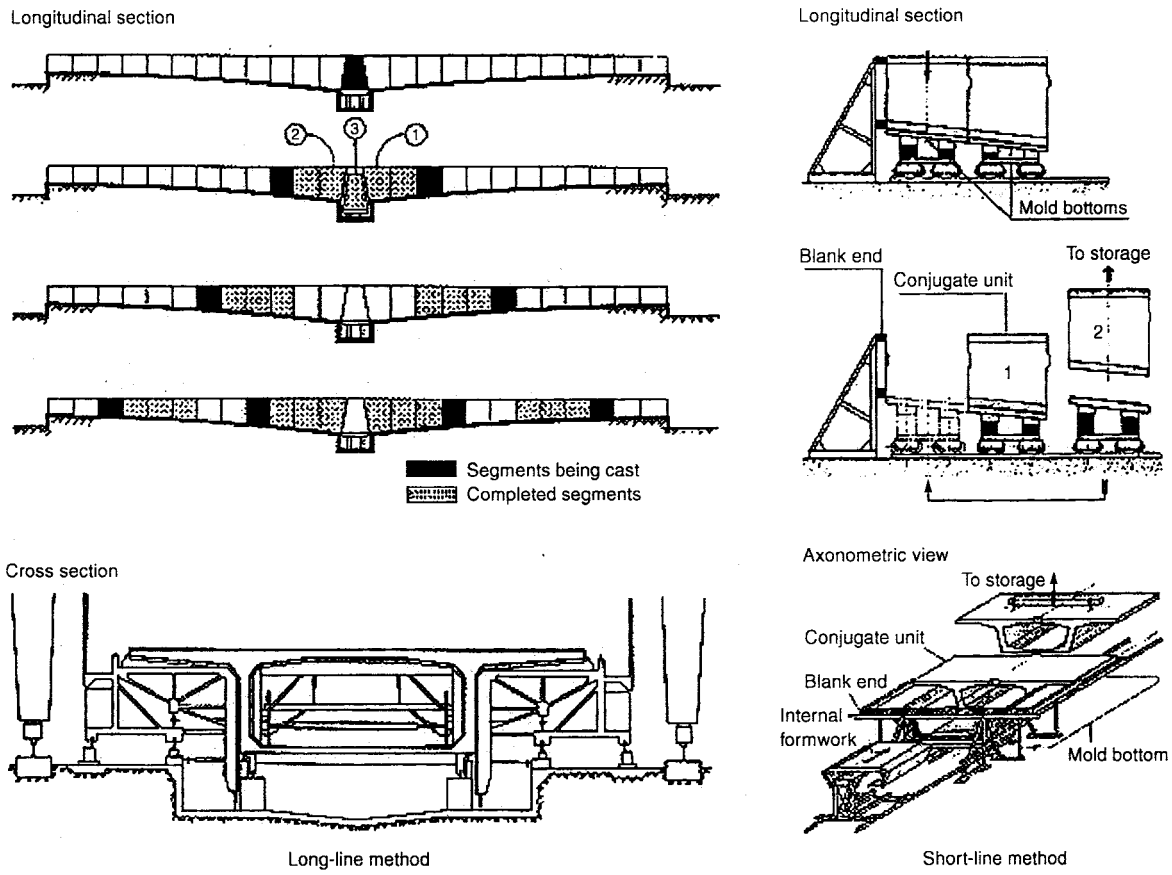


Figure 13.7. Casting cell schematic layout. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

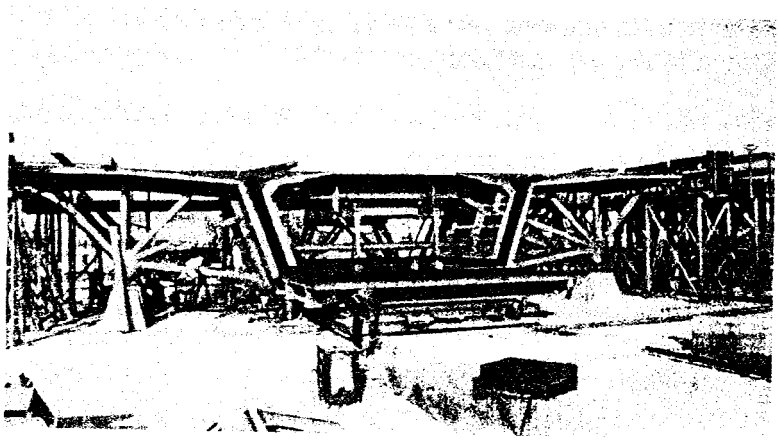
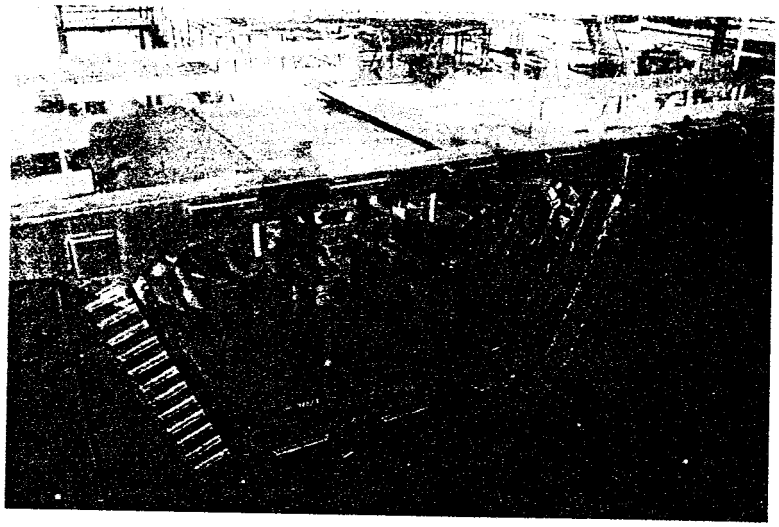


Figure 13.8. Short-line casting bed.



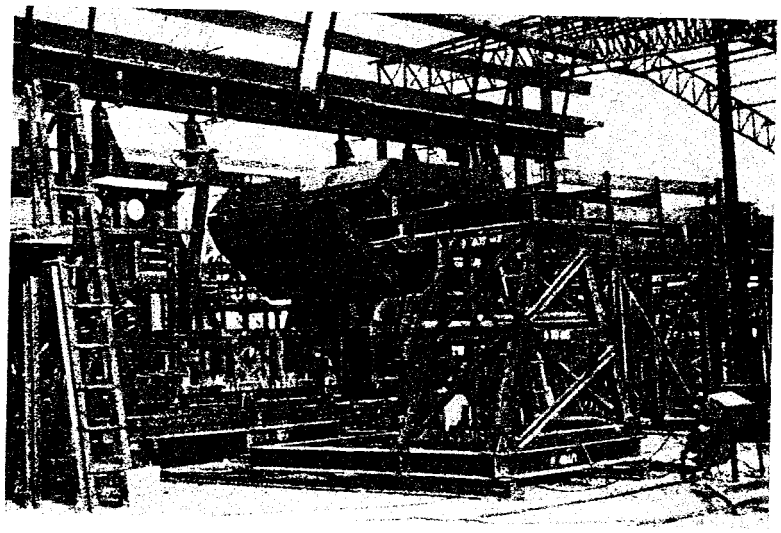


Figure 13.9. Inner shutter ready to slide into position.

@Seismicisolation

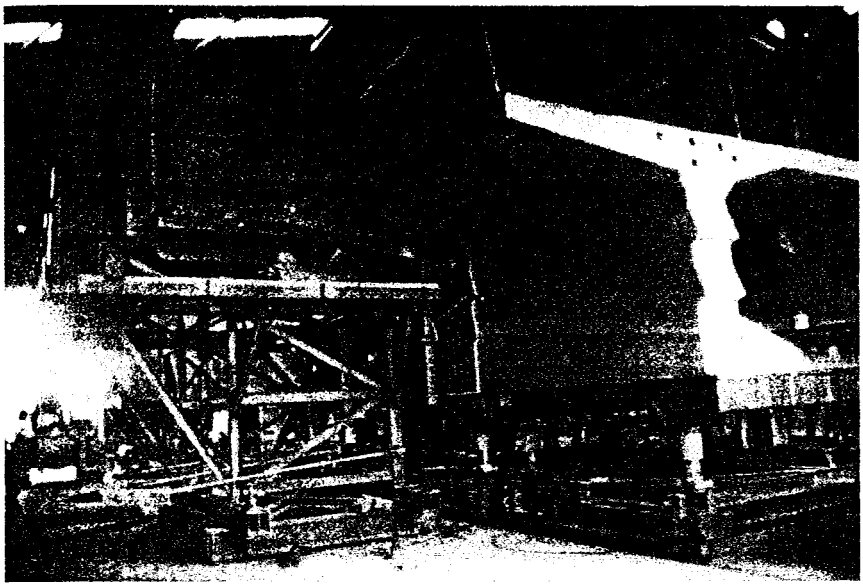


Figure 13.10. Counter-cast segment positioned.

hydraulic supports and adjusting systems to speed up its operating and setting up.

At the bulkhead shutter, seen in the top view of Fig. 13.9, a series of protrusions form shear keys across the end face of the concrete segment. When the segment is moved back and cast against, the concrete face of the new segment follows the shape of the shear keys which are mirrored across the joint.

Figure 13.10 shows the segment moved back into the counter-cast position while still on the soffit shutter. The counter-cast segment is set to the required vertical alignment by adjusting a series of jacks under each leg which move the corners of the segment to the required level. The horizontal alignment is set with a slide mechanism under the shutter which allows the front or back of the counter-cast segment to be moved sideways. With the counter-cast segment in position the reinforcement and tendon ducts are placed and the shutters set up ready to cast the new segment.

In the alternative long-line method the segments are cast against each other on a long casting bed, as shown in Fig. 13.11. The advantages of the long-line method are the simpler shutter design, operation, and easier geometry control systems, while the disadvantages are the extra cost of the longer soffit shutters and the extra space needed for the casting area.

The full length of soffit shutter is set up in position, while the side shutters and internal formwork, as seen in Fig. 13.12, move down the line for each segment casting. After each series of segments have been cast they are individually removed from the bed to a storage area, freeing up the bed for the next set of segments.

With either the short-line or the long-line method, a casting cycle of one segment per day per casting cell is usually achieved for the typical segments in a span. Diaphragm segments and other more complex segments may take

@Seismicisolation

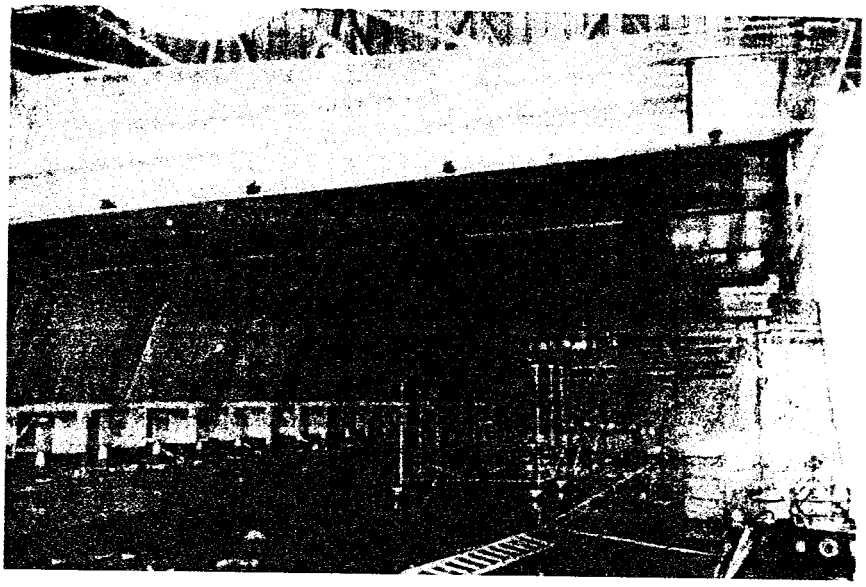


Figure 13.11. Segments on long-line casting bed.

two or three days each. To achieve these cycle times early high-strength concrete reaching 12 to 14 N/mm^2 after 12 hours is needed to allow the internal, side and cantilever shutters to be stripped in preparation for producing the next segment.

The reinforcement cage for each segment, including the prestress anchorages and ducts, is assembled in a jig which is exactly the same shape as the segment to be made. A typical jig is shown in Fig. 13.13. The reinforcement and prestress components are accurately fixed to ensure that the cage will fit into the segment shutters with the required cover and so that the

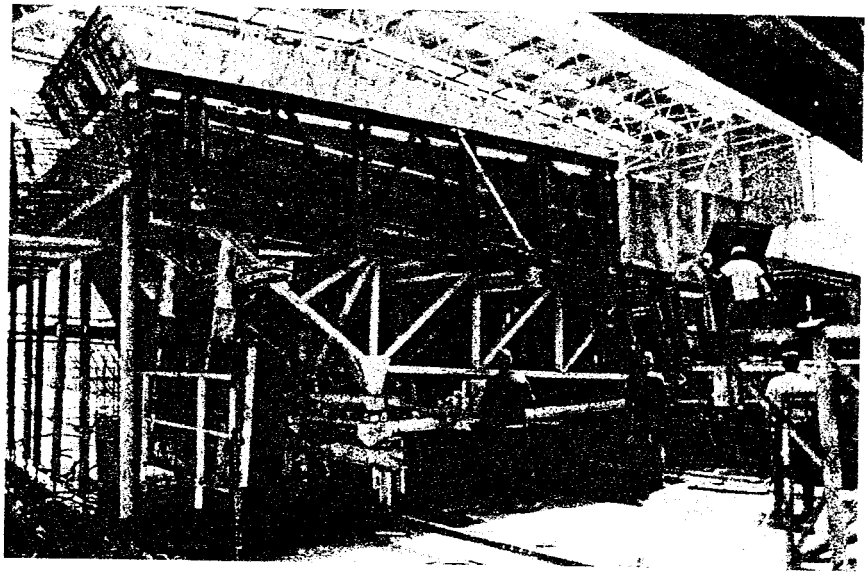


Figure 13.12. Long-line formwork.

@Seismicisolation

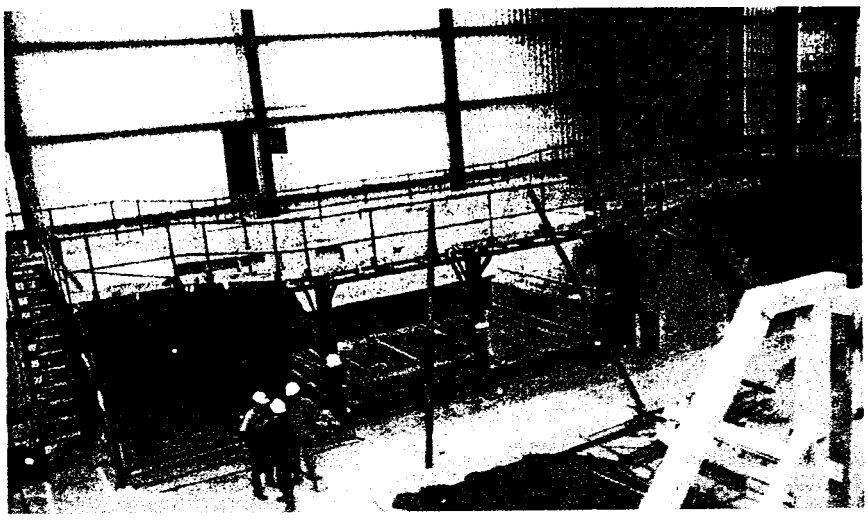


Figure 13.13. Reinforcement assembly jig.

prestress ducts match up with the ducts cast into the adjacent segments. There are usually two or three jigs per casting cell allowing the reinforcement to be assembled over a two or three day period.

After the assembly of the reinforcement and prestressing ducts the cage is lifted and transported to the casting cell using a lifting frame, as seen in Fig. 13.14. The lifting frame supports the different parts of the cage and prevents it from bending or distorting as it is moved. An overhead gantry or special lifting equipment is used to pick up the frame and cage and move them around the casting yard.

One problem that occurs in segmental construction with internal tendons is angular misalignment between the ducts across the joints. When the ducts are installed and set up over the short length of a single segment it is difficult to

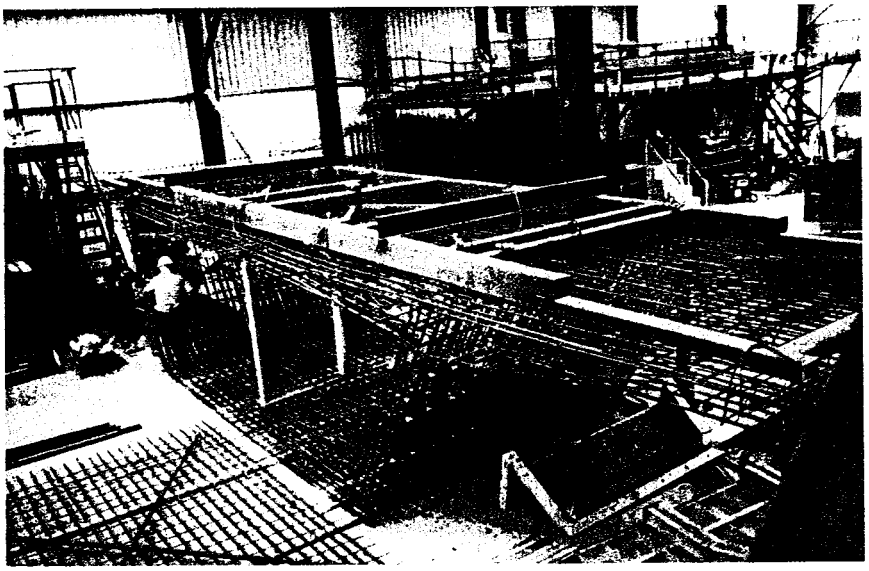


Figure 13.14. Reinforcement cage stored ready to be moved to casting cell.

@Seismicisolation

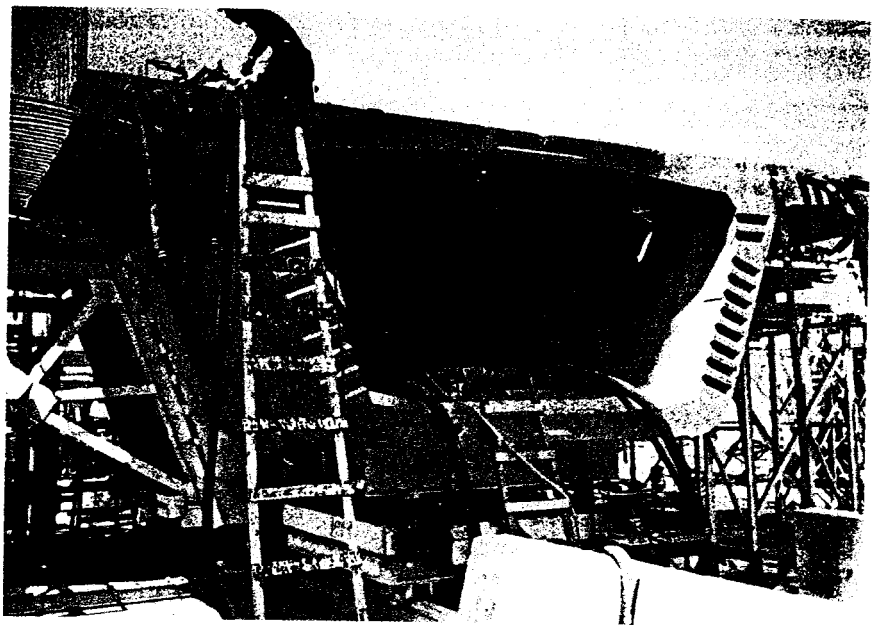


Figure 13.15. Inflated tubes in ducts during segment casting. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

get them aligned exactly with the corresponding duct in the adjacent segment. During concreting the ducts are sometimes dislodged or tend to float up in the concrete if they are not rigidly supported. Misaligned ducts lead to an increase in friction during the stressing of the tendons, resulting in a lower achieved prestress force than required. To hold the ducts in position during the concreting and to prevent any concrete getting into them they are filled with a pneumatic tube, as seen in Fig. 13.15. The tube is inflated to seal the duct and cover any gaps across the segment joints.

By casting a segment directly against its neighbour a perfect fit is obtained when they are erected. Complex alignments are achieved by carefully controlling the relative position of adjacent segments when they are being cast. For the long-line method, the soffit shutter is set up to the required alignment before casting begins while with the short-line method the alignment is controlled by adjusting the position of the segment in the counter-cast position. Decks with a 6 per cent gradient, 10 per cent crossfall and horizontal radii down to 45 m have been successfully built with the short-line casting system.

Figure 13.16 illustrates the alignment control points on a segment. The horizontal alignment is controlled by a series of centreline pins placed at either end of the segments on the top slab mid-way between webs, as indicated in Fig. 13.16(i). A permanent horizontal control line is established along the centreline of the casting cell, with a theodolite left in place at one end and a target permanently placed at the other. These are often supported on steel piles and protected by an enclosure from the effects of the weather to minimize any disturbance to the established setting out line.

The vertical alignment is controlled by four levelling bolts per segment placed above each web and again at each end of the segments, as indicated in Fig. 13.16(i) and (ii). Precise levelling equipment is used to obtain an

@Seismicisolation

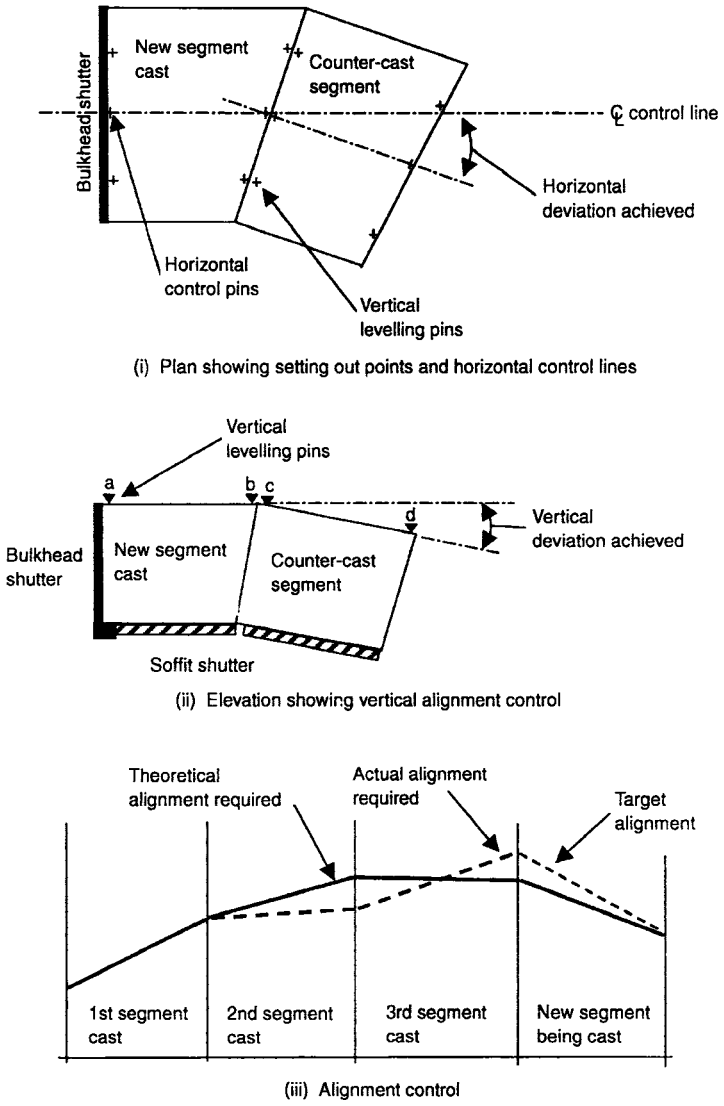


Figure 13.16. Segment setting out.

accurate measurement of the level of each bolt. The vertical alignment of each web is monitored and used to control the deck crossfall as well as the longitudinal profile.

By measuring the relative horizontal and vertical deviations between adjacent segments while they are against each other in the casting cell, it is possible to predict the alignment that will be achieved when all the segments are erected. Figure 13.16(iii) illustrates a typical alignment control curve. The achieved deviation between adjacent segments is plotted against the theoretical alignment curve from which the target alignment for the next segment is derived.

Construction tolerances and movements in the shutters during casting mean that the exact target alignment is seldom achieved, but by precise surveying,

@Seismicisolation

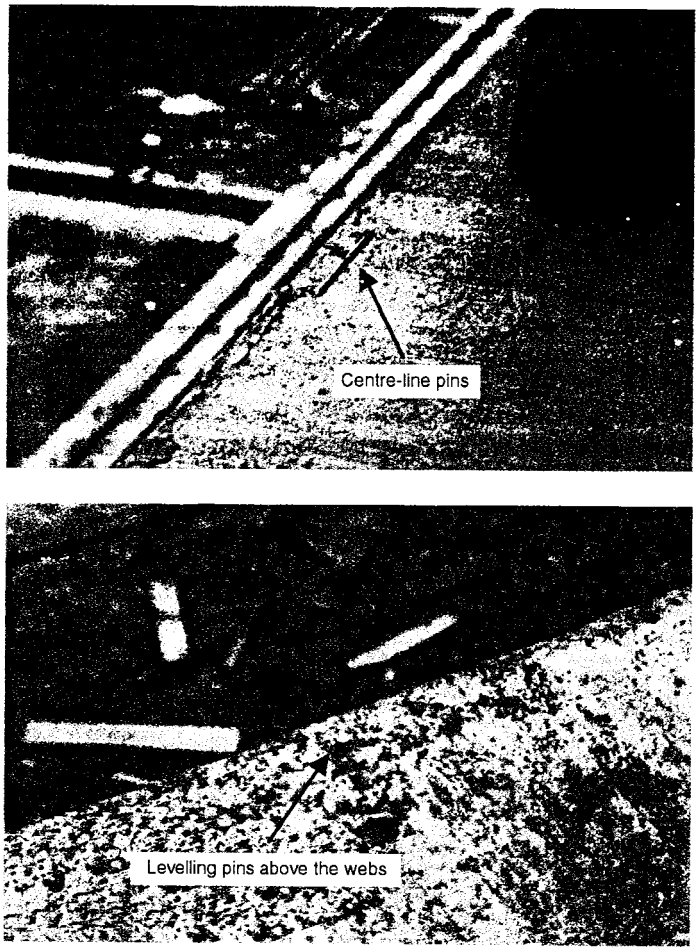


Figure 13.17. Survey pins.

accurately setting up the segments and careful construction the errors are kept to acceptable values and within normal construction tolerances. It is possible to compensate for small misalignments in the casting of each segment by adjusting the target alignment for the next segments to be cast. In this way the theoretical alignment points are continually 'aimed' at and any misalignments compensated for.

The centreline pins are short lengths of bar inserted into the wet concrete, after casting the segment, with the centreline inscribed into the top surface to line up with the horizontal control line. The levelling pins are small round-headed bolts pushed into the fresh concrete. Typical centreline and levelling pins are shown in Fig. 13.17.

Storage and transportation of segments

After leaving the casting area the segments are stored, as seen in Fig. 13.18, until they have gained their full strength and the deck erection reaches the stage when they are required. In order not to warp the segments during

@Seismicisolation

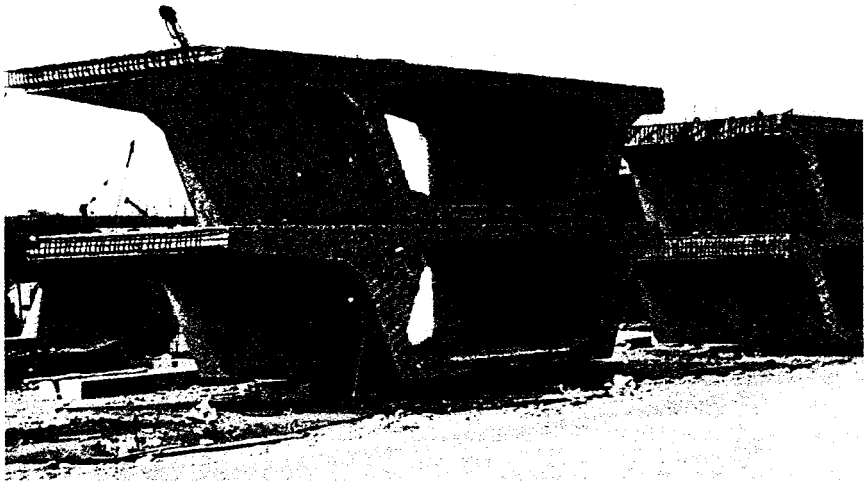


Figure 13.18. Segments in storage yard.

their storage, a simple three-point support arrangement is used, with two support points under each end of one web and the third point under the mid-point of the other web. If any one of the supports settle it should not cause twisting or warping of the segment. Where there is limited space to store the segments it is possible to stack them on top of each other; although the strength of the segments and stability of the arrangement need to be checked.

A transporter, such as the type shown in Fig. 13.19, is used to move the segment from the casting area to the storage yard and from the storage yard to the erection site. Segments transported over a public highway are usually limited in length to 3·6 m to minimize disruption to the traffic.

The segments are lifted on and off the transporters by a crane, as shown in Fig. 13.20, or by special lifting frames. Each segment is likely to be lifted and

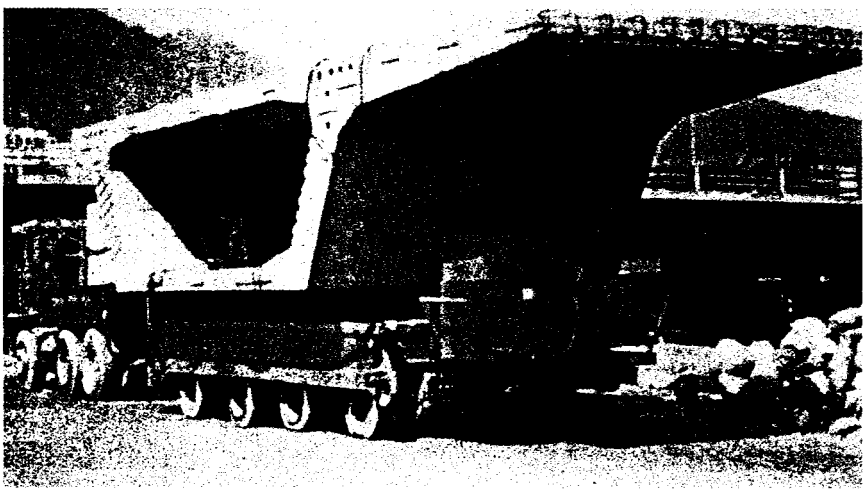


Figure 13.19. Segment on special carrier.

@Seismicisolation

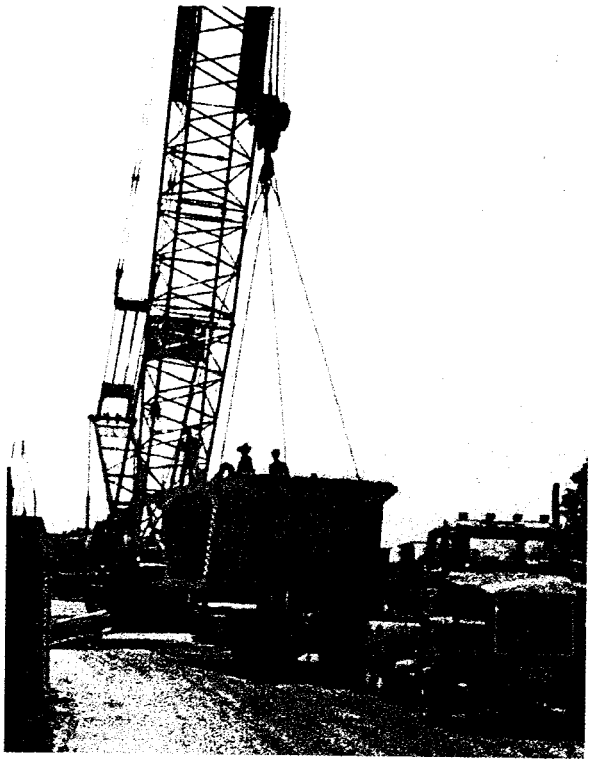


Figure 13.20. Segments lifted by crane at storage yard.

stored several times between the casting bed and its final erection position, and it may be stored for several months before being used.

Although it is sometimes possible to lift the segments with ropes slung under the segment cantilevers, as seen in Fig. 13.20, lifting devices are usually built into the top slab of the segments. Figure 13.21 shows an

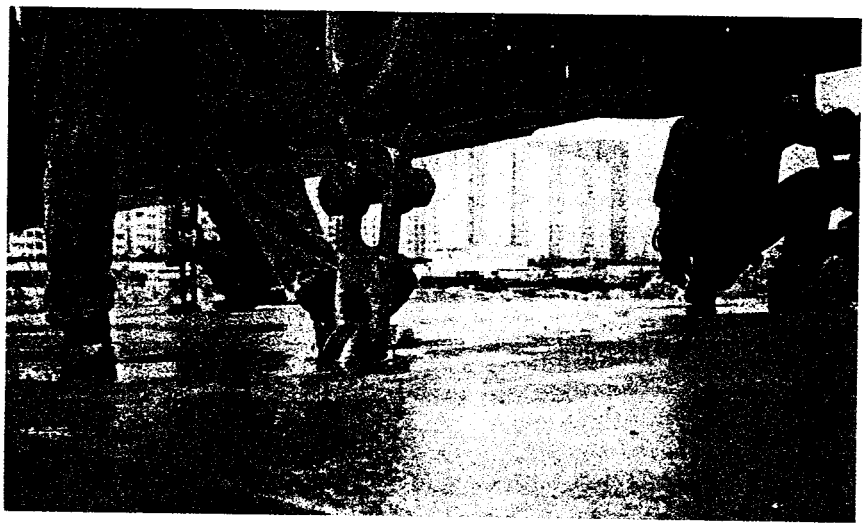


Figure 13.21. Cast-in lifting hooks.

@Seismicisolation

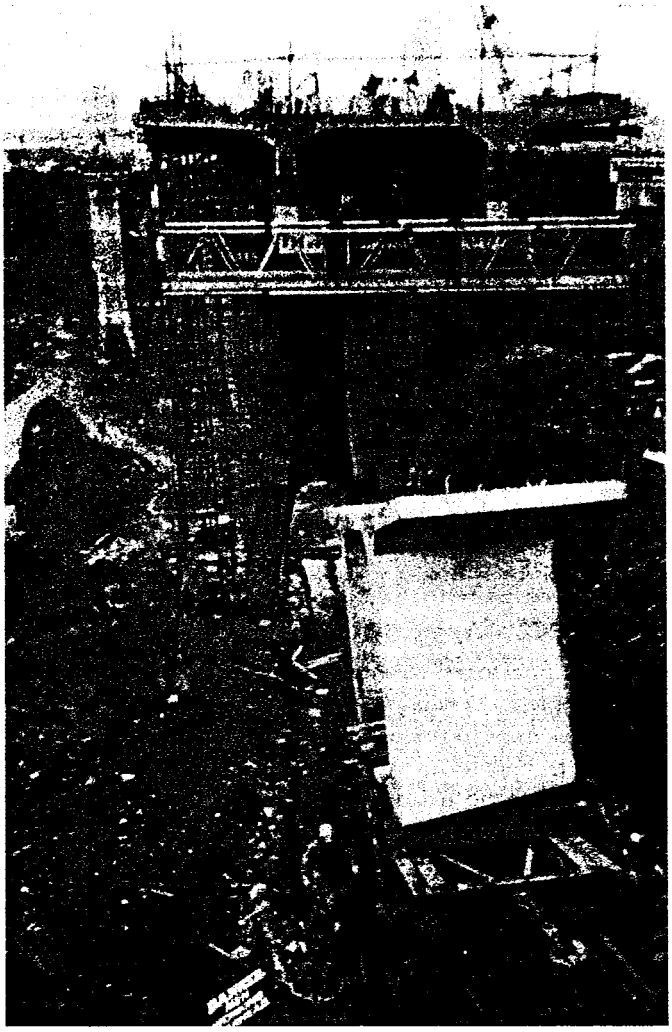


Figure 13.22. Segment positioned on rail-mounted bogey.

arrangement with a steel anchor cast into the web so that a lifting hook can be fitted when required. An alternative arrangement is to leave small holes through the top slab and to use these to place stressbars and clamp on a removable lifting assembly.

For decks built with an erection gantry, the segments are usually delivered along the completed deck with the transporter driving up under the back of the gantry. With erection by crane or lifting frame the segments are first delivered and positioned at ground level.

Where access at the erection site is not possible for wheel-based transporters, rail-mounted bogeys, as seen in Fig. 13.22, may be used to move the segment under the deck ready to be lifted into place. This is a slow process for the erection and is generally limited to projects where access is difficult and lifting frames on the deck are used to raise the segments into position.

Barges, as shown in Fig. 13.23, are used to deliver the segments when crossing a navigable river. Used in conjunction with barge-mounted cranes

@Seismicisolation

Figure 13.23. Segment transported by barge. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

or lifting frames on the deck the barges are usually able to transport several segments at a time.

Segment erection, general

The segments are positioned in the deck either by a self-launching gantry supported on the piers, a crane or ground level gantry if access is suitable, or by lifting frames fixed to the previously erected deck.

Three different techniques are used for erecting the segments, as illustrated in Fig. 13.24. These are the 'balanced cantilever' approach, the 'span-by-span'

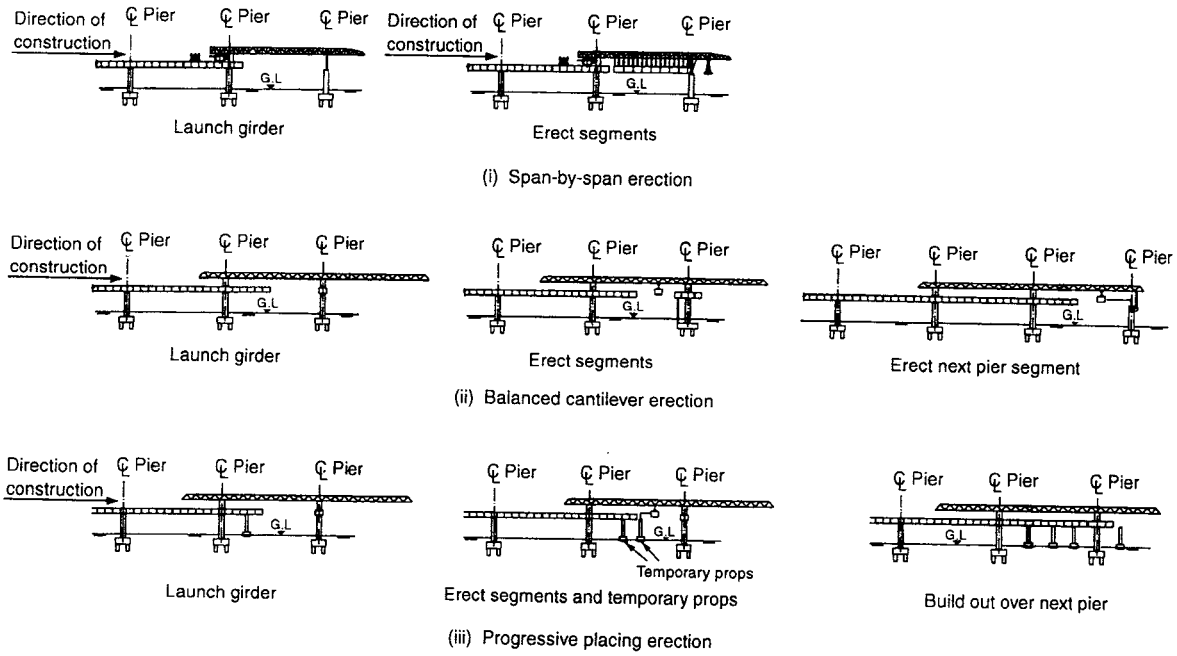


Figure 13.24. Precast segmental deck erection techniques.

@Seismicisolation

approach and the 'progressive placing' of the segments. The balanced cantilever method involves erecting the segments as a pair of cantilevers about each pier. For the span-by-span method all the segments for a span are positioned before the prestress is installed. With progressive placing the segments are built out in one direction from a starting point passing over the piers in the process. These three different techniques are described in more detail in the next three sections.

The first segment to be erected is usually a diaphragm segment sitting over a pier on temporary supports while it is bedded onto the bearings. If the bearing has fixing bolts protruding up from the top plate, box-outs are formed in the underside of the segment to match the bolt location. With the segment held above the bearing the gap between the soffit and top plate, including the box-outs, is filled with cement mortar. If the top plate has no fixing bolts, the bearing top plate is grooved and, with the segment held above the bearing, the gap is filled with epoxy mortar.

Segments to be erected with epoxy in the joints are lightly sandblasted on their end faces before being erected. This removes any deleterious material and provides a good bond with the epoxy glue. Spread evenly over the surface, the epoxy helps to lubricate the joint during erection allowing the segment to slide into position. When hardened the epoxy seals the joint and provides structural continuity between the segments. Application of the epoxy on to one face of each joint between the segments is usually 'by hand', as shown in Fig. 13.25. A uniform thickness is achieved by combing across the joint with a serrated edge that leaves a 1–3 mm thick coating.

When the segments are brought together and the temporary prestress applied the epoxy is squeezed across the joint to fill in any gaps and provide a watertight seal. After stressing the deck the thickness of the epoxy in the joint is between 1 mm and 1·5 mm. The epoxy usually has a minimum compressive strength of 70 N/mm^2, which gives a connection stronger than the concrete either side. Compressible rubber or foam 'O'

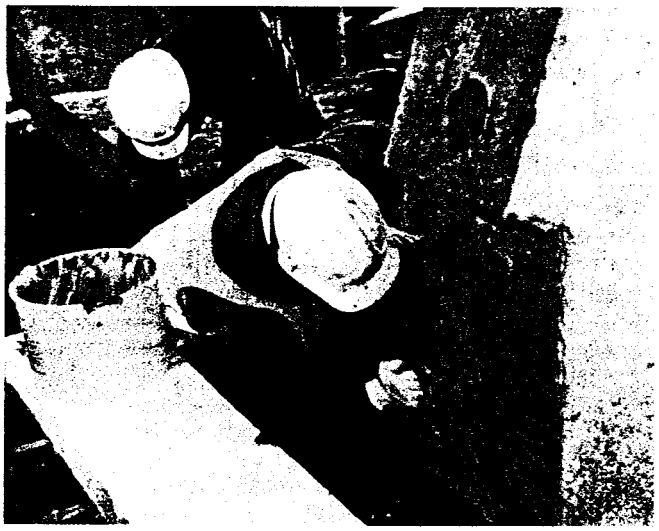


Figure 13.25. Epoxy being spread over joint. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

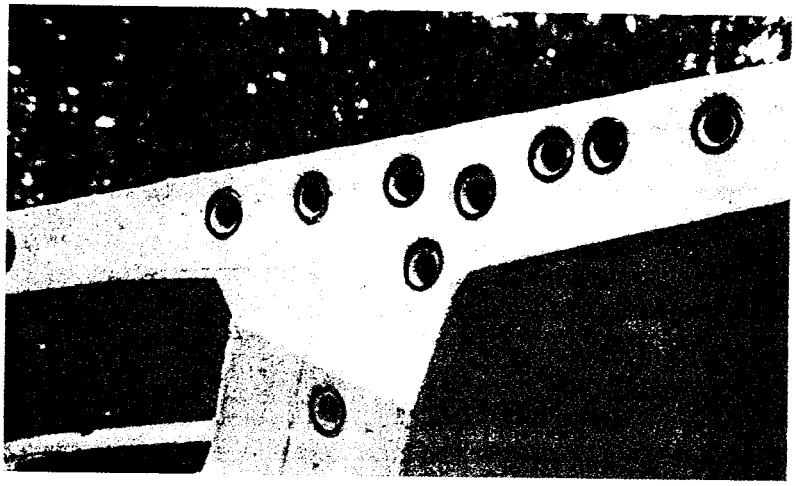
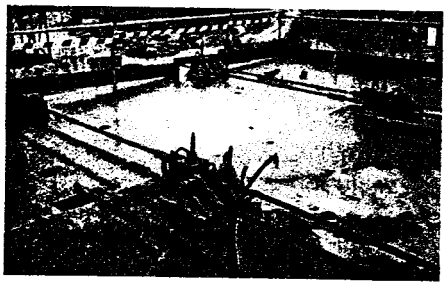
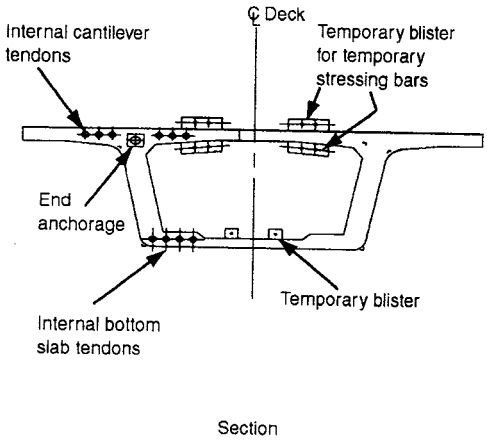


Figure 13.26. 'O' rings around ducts at segment joints.

rings, as shown in Fig. 13.26, are glued to the concrete around each hole to prevent the epoxy from being squeezed into the prestressing ducts.

The epoxy used in the joints only has a short 'open' life and after being placed on the concrete surface the joint has to be closed with the temporary prestress applied within approximately one hour. The temporary prestress, as illustrated in Fig. 13.27, normally consists of bars. These are quick to install, and hold the





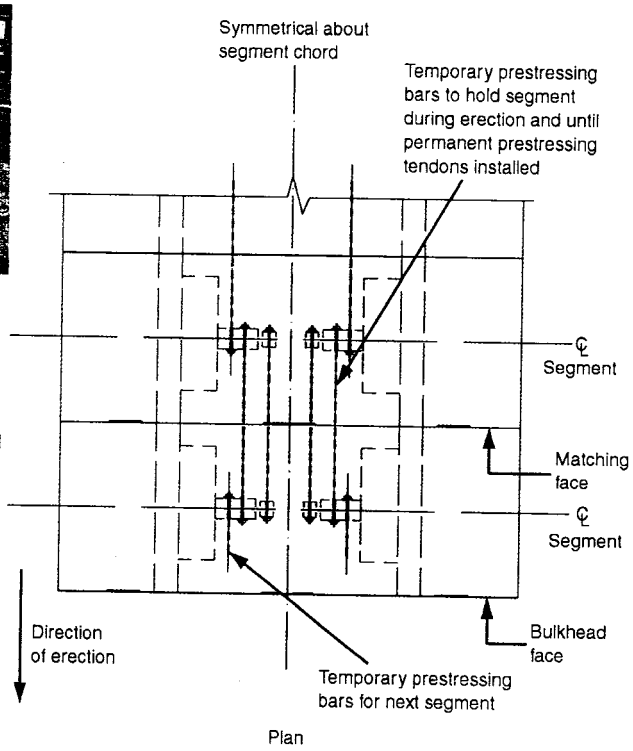


Figure 13.27. Temporary prestress for segments.

@Seismicisolation

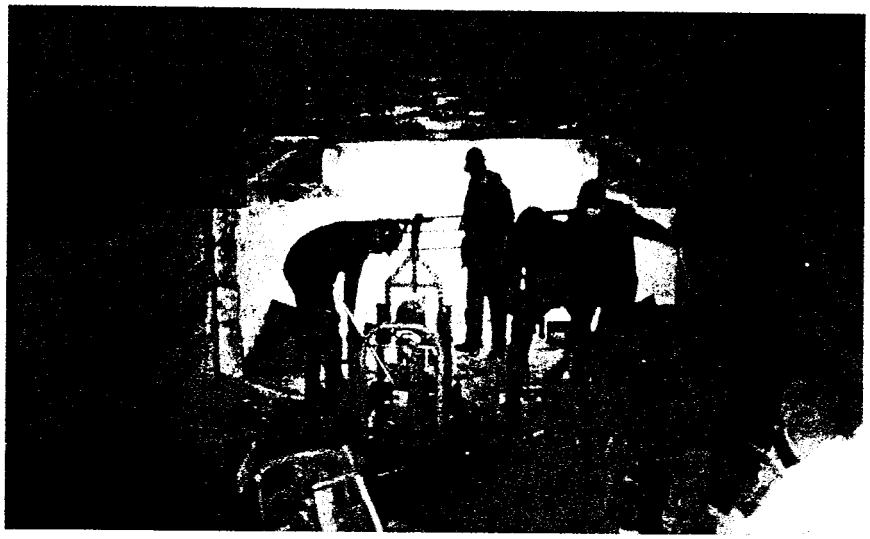


Figure 13.28. View inside box girder as segment is being positioned.

segments in place until the permanent tendons are stressed. To keep the epoxy thickness uniform over the joint, the temporary prestress is applied with an average compressive stress of between 0·2 N/mm^2 and 0·3 N/mm^2. Until the epoxy has hardened the local stress in the segment joint should not be less than 0·15 N/mm^2 and the stress difference due to flexure not more than 0·5 N/mm^2. When the epoxy is put under pressure by the prestress force it tends to squeeze out of the sides and bottom of the joint. A sheet or other cover is placed under the joint to catch the epoxy as it drops.

The temporary prestressing bars are either placed through the segment's top and bottom slabs or arranged outside the concrete, as shown in Fig. 13.27. Bars inside the slabs are usually left in place and grouted to become part of the permanent prestress for the deck. Bars located outside the concrete section are positioned above the slabs and anchored on temporary steel or concrete blocks stressed down to the segment, or placed inside the segment and anchored to concrete nibs protruding from the inner surface of the box. The advantage of using bars outside the concrete is that they are easy to de-stress and re-use.

When segments are being erected they are carefully guided into position to ensure that no damage occurs to the concrete. Figure 13.28 shows the inside of the deck as the segment is being positioned. The segment is initially offered up to the end of the previously erected segment on a 'dry run' with the temporary prestressing in place to ensure that everything fits together and matches. It is then moved back by at least 300 mm and the epoxy applied to the joint surface after which the segment is pulled into position and the temporary prestress fully installed.

After the epoxy has hardened it becomes very brittle and any material protruding from the joint is chipped off to leave a level surface. Figure 13.29 shows a close-up view of the outer surface of the joints after completion of the deck erection. It is not necessary to do any further work on the joints which are noticeable only when viewed from near the deck.

@Seismicisolation

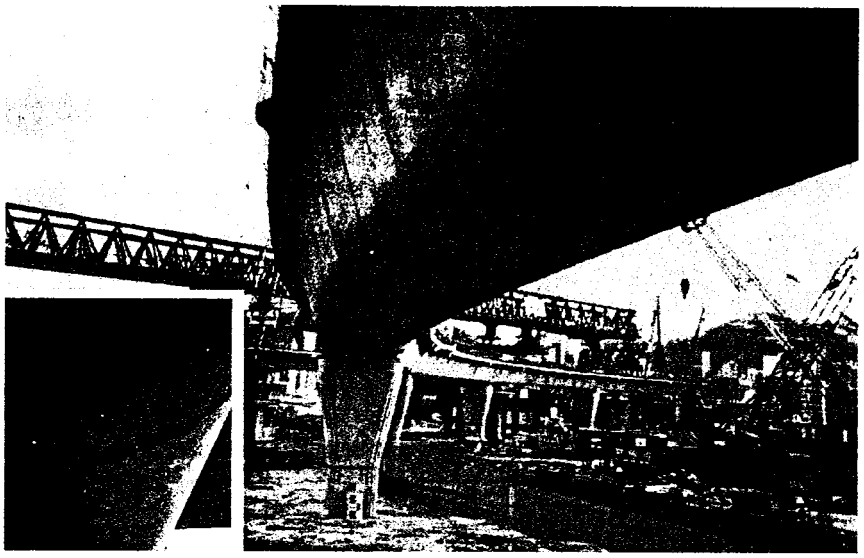


Figure 13.29. Close-up of joint after segment erected.

Several segmental bridge decks, such as the BSES, in Fig. 13.30, have been built with dry joints where no epoxy is used between the segments. The design relies on shear friction to transfer the loads across the joint in both the permanent and temporary conditions. Omitting the epoxy speeds up the erection times, removes hazards associated with epoxy and saves cost. Dry-jointed decks are usually used only in tropical climates where freeze-thaw cycles do not occur, while their effectiveness in more aggressive climates has yet to be proved. The construction of dry-jointed decks is similar to segmental bridge decks with epoxy joints. The casting techniques and erection procedures are the same except that the epoxy is omitted and external tendons are always used.

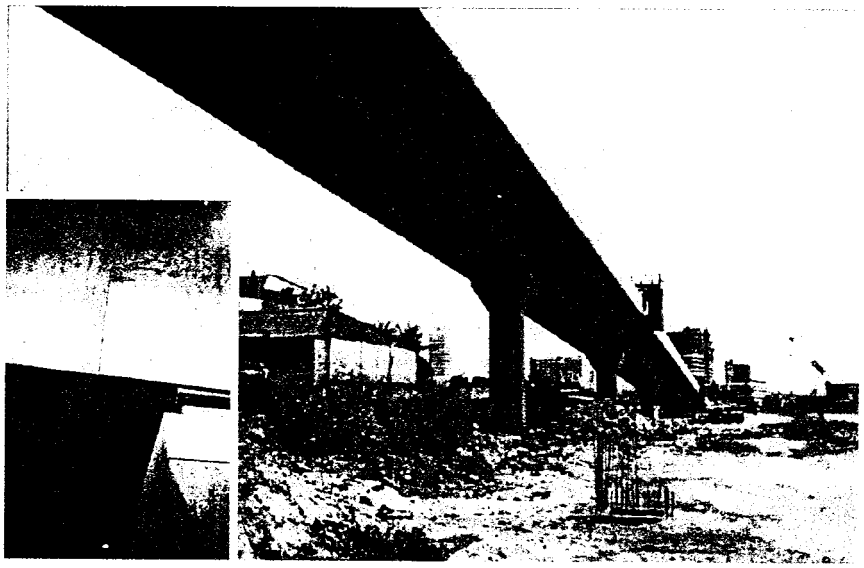


Figure 13.30. Dry-jointed segmental deck.

@Seismicisolation

Before erecting segments for dry-jointed construction the joint faces are cleaned by wire brushing but are not sandblasted, as this may damage the mating surfaces. Without epoxy in the joints temporary prestress is not needed, although bars are sometimes used to pull the segments together and hold them in place until the permanent prestress is installed. When the deck is prestressed the joints fully close up, presenting a neat appearance.

Dry joints do not achieve a full watertight seal between the segments, and this makes the grouting and protection of internal tendons difficult. For this reason external tendons are always used with dry joints. To prevent water seepage through the top slab the top of the joint must be sealed in some way. The early techniques for this included inserting a rubber tube into a preformed groove running across the full width of the top slab. This did not prove to be 100 per cent successful and water was able to seep through the joint and stain the underside of the segment or create 'stalactites'. On the BSES viaducts the dry joints were made watertight with an epoxy seal poured into a narrow recess along the top surface, as illustrated in Fig. 13.31.

The advantages of using dry joints with precast segmental construction are as follows:

- elimination of epoxy;
- elimination of the temporary prestress required with epoxy joints;
- rapid construction times.

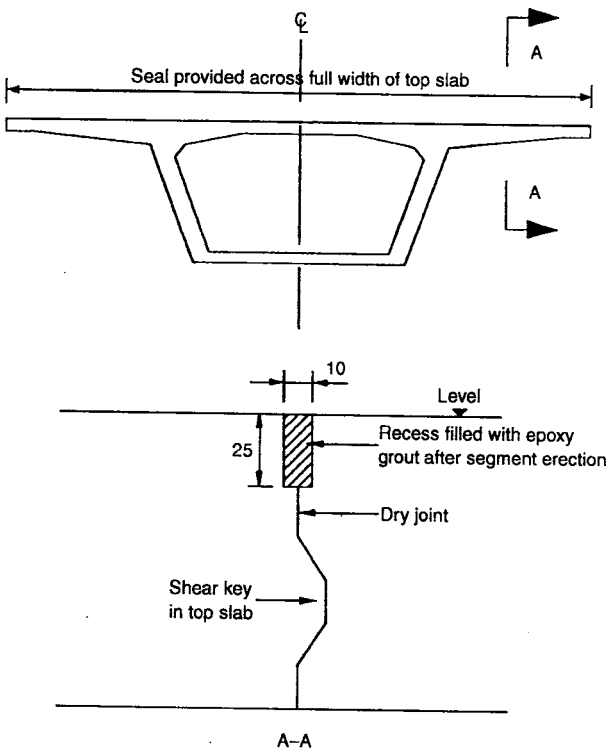


Figure 13.31. Details of seal along top of dry joint.

@Seismicisolation

The disadvantages of dry joints are:

- more prestress required to maintain the ultimate shear and moment capacities;
- external tendons must be used;
- top surface must be sealed to prevent moisture ingress.

When transporting and erecting the segments they must be protected from damage, especially the match-cast faces. They should not be stored where they could be knocked by passing vehicles and must be carefully handled when moved. Spalling of the concrete edges, as seen in Fig. 13.32, may also occur when the segments are being stressed together due to segment misalignment or the presence of deleterious material in the joint. Any damage to the joint surfaces should not be repaired until after the segment has been erected to ensure the match-cast faces maintain a perfect fit.

When the segments are erected they join up to the adjacent segments with the same horizontal and vertical angle deviation as existed when they were cast against each other. This is facilitated by the presence of the shear keys on the webs and slabs which guide the segment into position. The erected segments follow an 'erection alignment' that matches the 'cast alignment' modified by the self-weight and prestress deflections as the deck is built up.

Construction and surveying tolerances may require some correction to the alignment of the segments during the erection process. Small inaccuracies in individual segments or misalignment across joints accumulate when long

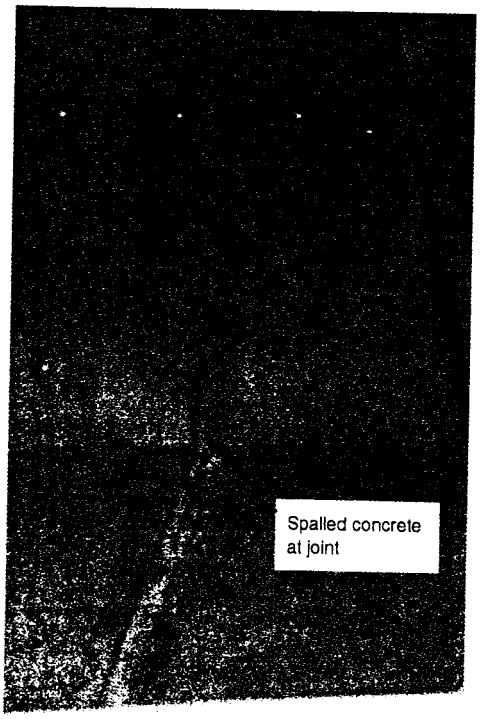


Figure 13.32. Spalling of concrete edges at joint between segments.

@Seismicisolation

lengths of deck are constructed. A small deviation at the first segment results in an increasing misalignment as more segments are erected.

In decks erected by balanced cantilever any small misalignments are corrected within the *in situ* stitch at mid-span. Similarly for the span-by-span and progressive placing methods, misalignments are taken out by short *in situ* stitches introduced at suitable locations along the deck.

If it is necessary to adjust the alignment of the individual segments during their erection this is achieved by using shims or fibre sheets to increase the thickness of the epoxy joint along one edge. Although this technique achieves only a small adjustment to the deviation at any one segment joint the effect is magnified as subsequent segments are built out.

Segment erection by the balanced cantilever method

The most common method of erecting the segments is by the balanced cantilever technique, either with an overhead or ground-level gantry, a crane or by a special lifting frame fixed to the deck.

The first segment over the pier is lifted into position and placed on temporary supports while it is set to the correct alignment. The matchcast segments are then transported into position and lifted either side of the pier segment, being fixed to it with epoxy in the joints and temporary prestress. Further segments are positioned with erection progressing out from both sides of the pier in a balanced manner, as illustrated in Fig. 13.24(ii). When the cantilever reaches mid-span it is connected to the opposing cantilever from the adjacent pier by an *in situ* concrete stitch. Prestressing tendons are then installed along the length of the span and across the stitch to complete the connection.

The cantilevers on either side of the pier are kept as balanced as possible to minimize the temporary works needed to prevent the deck from overturning as it is being built out. The deck and temporary works are usually designed for a maximum of one segment out-of-balance although on some projects two segments are erected simultaneously, one on either side of the pier to reduce the out-of-balance forces.

Using the balanced cantilever technique it is possible to erect a typical 40 m span over a 2·5- to 3-day cycle, while spans of up to 100 m typically take between 7 and 12 days to erect.

Overhead gantries are supported off the piers or the already completed deck, as seen in Fig. 13.33. The segments are transported along the completed deck and lifted up at the back of the gantry by a bogey arrangement which runs along the top chord of the truss and moves the segment into its required position.

After the balanced cantilever at a pier is completed, the gantry moves forward to the next pier ready to erect the next pair of cantilevers. Most gantries are self-launching with the back section supported on rear legs running along the completed deck and with front legs moving on to the next pier. A typical sequence for erection by gantry is:

@Seismicisolation

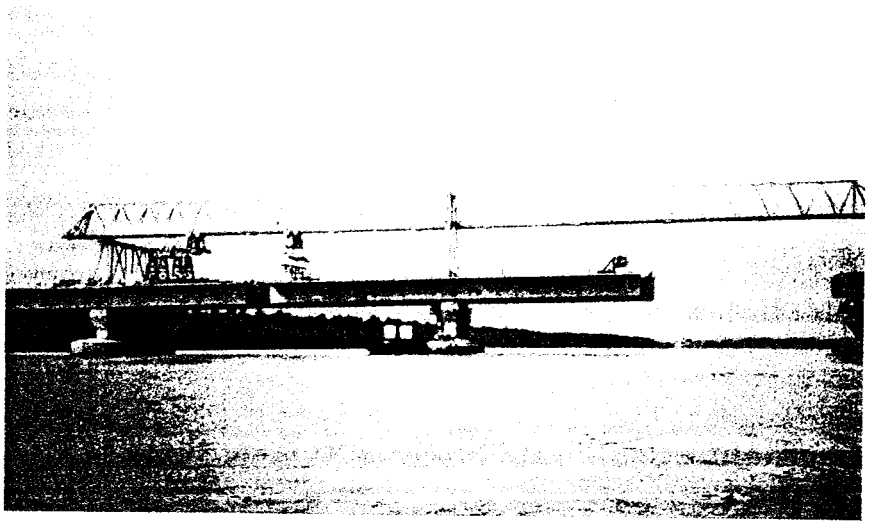


Figure 13.33. Balanced cantilever erection using a gantry. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

(a) Gantry used to position pier segment on to next pier. At this stage the gantry's front leg is supported by a bracket fixed to the next pier and the middle and back legs are on the previously completed deck.
(b) Middle leg moved on to pier segment.
(c) Gantry launched forward to sit 'symmetrically' above the pier where the balanced cantilever is to be erected. The gantry is supported on its middle leg on the pier segment and its back leg on the previously completed deck.
(d) Segments delivered along completed deck, lifted up by the gantry, and moved into position, as seen in Fig. 13.34.

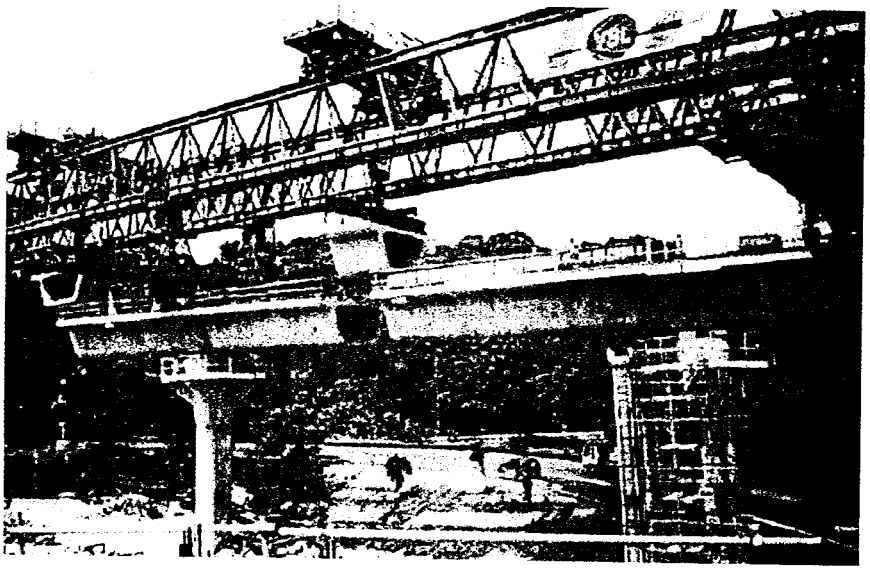


Figure 13.34. Segment being positioned. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

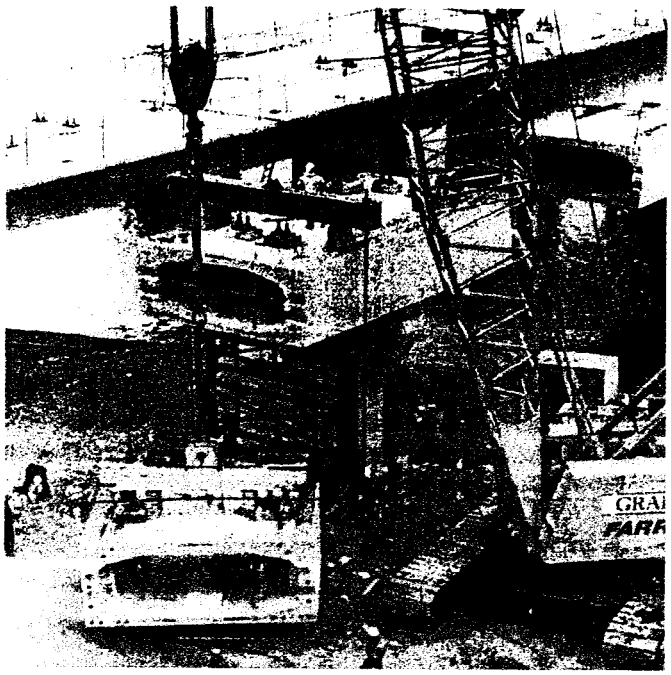


Figure 13.35. Precast segments erected by crane. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

(e) Segments erected and prestress installed until cantilevers completed.
(f) *In situ* stitch at mid-span cast and continuity prestress installed.
(g) Next pier segment positioned and cycle repeated.

Where a crane is used, as in Fig. 13.35, good access is needed to the deck and surrounding area to allow the crane to be positioned and the segments to be transported nearby for lifting into place. Cranes usually give the simplest and most rapid erection procedures with the minimum of temporary works; however, where there are obstructions on the ground crane erection may not be possible.

Ground level gantries running on tyres or rails are used to position segments where a level access is available. The segments are moved into position under the gantry, which lifts them up to be stressed on to the already completed section of deck. Figure 13.36 shows a ground-level gantry being used to erect the deck as a balanced cantilever. Less versatile than a crane, ground-level gantries are usually used only on long viaducts and when an existing gantry is readily available.

Lifting frames fixed to the end of the cantilever are used to pick up and position the next segment, as seen in Fig. 13.37. At the beginning of the erection of each balanced cantilever the pier segment is positioned by crane or cast *in situ* to establish a platform on which the lifting frame is assembled and secured. After picking up and placing the new segment the frame is moved forward and re-secured to the deck, ready to pick up the next segment. After a balanced cantilever is completed the lifting frame is removed from the completed section of deck and lifted onto the pierhead segment at the next pier to repeat the process.

@Seismicisolation

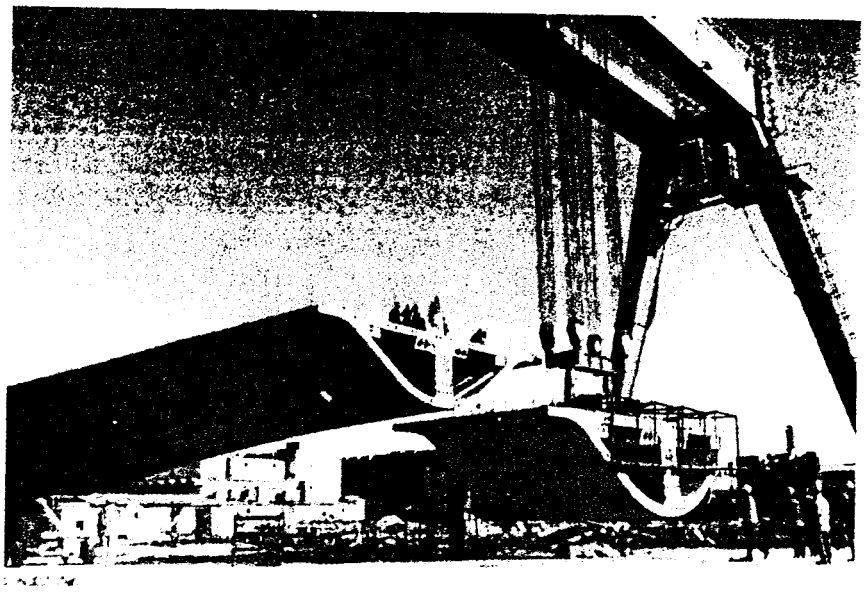


Figure 13.36. Precast segments erected by ground-level gantry. Reproduced courtesy of Freyssinet International. Copyright reserved.

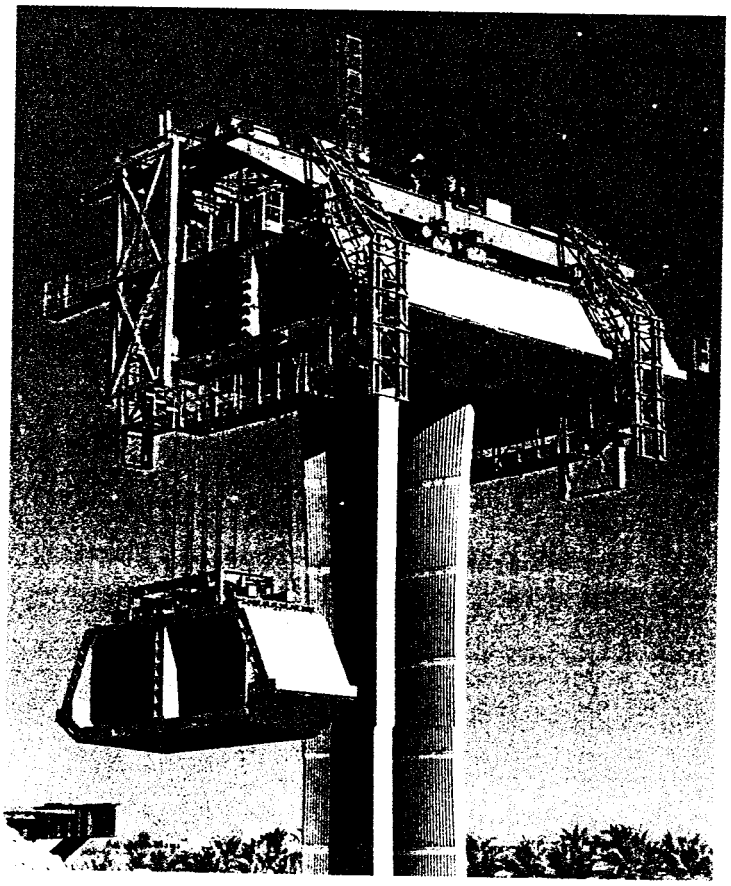


Figure 13.37. Precast segments erected by lifting frame. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

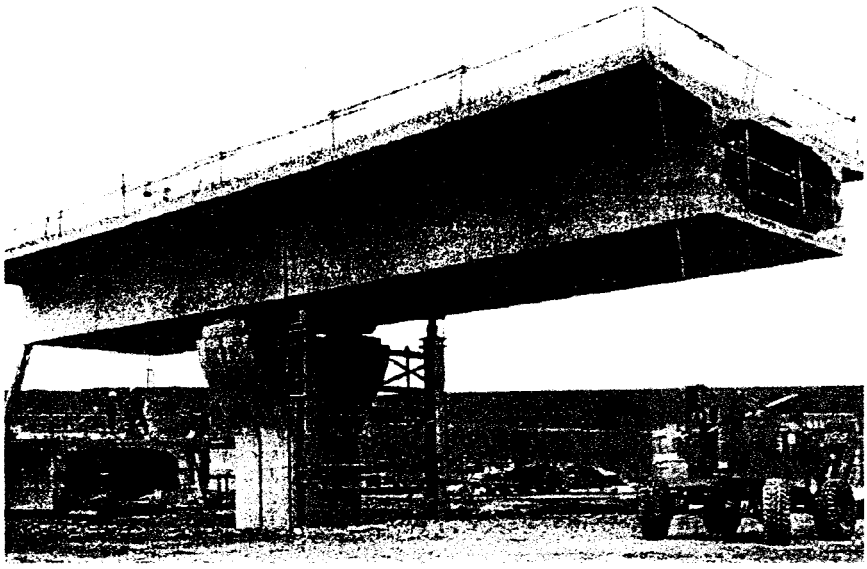


Figure 13.38. Prop and tie to stabilize cantilever. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

Using a pair of lifting frames, one on each cantilever, simplifies the erection process, although using a single lifting frame is possible. When using a single lifting frame it is moved from one side of the pier to the other to lift the segments in turn.

The advantages of placing segments with a crane are that very rapid erection is achieved and the deck may be constructed in several places at once, but good access is required along the complete bridge length. Ground-level gantries also allow rapid erection of the segments but require good access. Overhead gantries achieve fast construction rates and minimize the disruption to the ground beneath the deck. They are well suited to building over rivers or other obstructions although they are usually limited to erecting the deck in a sequential manner and are delayed if problems occur at any one pier or in any one span. Lifting frames are the least used due to their relative slowness and the disruption when moving to the next pier. They also need good access underneath the deck although they can offer economic advantages if only a short length of deck is being constructed.

Where the deck is supported on bearings at the pier the stability of the deck during the cantilevering sequence is achieved by providing a temporary support. When the deck is at a low level a prop and tie-down arrangement is often used, as in Fig. 13.38. It is convenient with this arrangement to support the prop and tie from the pier foundation.

For taller piers, or where there are obstructions beneath the deck, stability is achieved during cantilevering by a bracket clamped to the top of the pier, as seen in Fig. 13.39, which is used to provide a support to prop or tie-down the deck. Alternatively, the deck may be temporarily fixed to the top of the pier with a tie-down arrangement. With this method the deck is supported on the pier top by a combination of the permanent bearings and temporary jacks, and prestress tendons are installed from the top of the deck, through the diaphragm and down into the pier to provide a clamping force.

@Seismicisolation

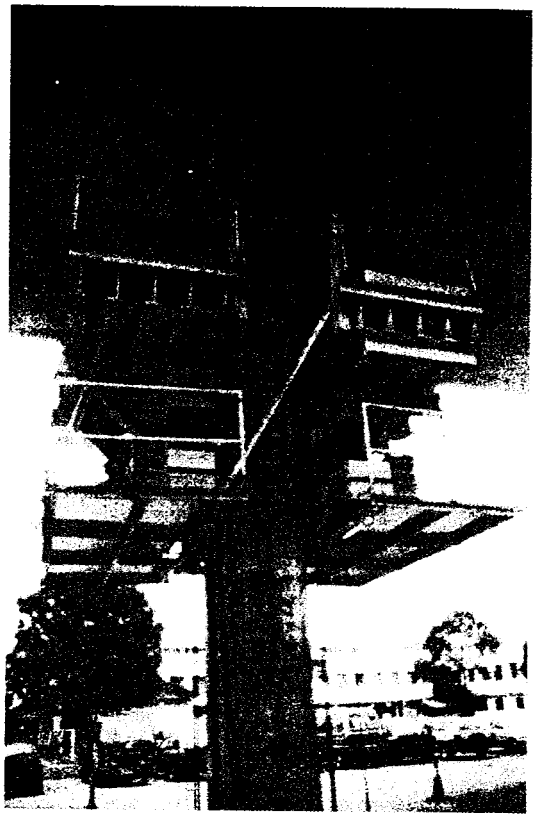


Figure 13.39. Stabilizing bracket around pier.

The temporary prestress to hold the segments together is arranged, as shown in Fig. 13.27. As the cantilever extends out additional temporary prestress bars are added to support the extra dead load, or alternatively some of the permanent prestress tendons, commonly referred to as 'cantilever' tendons, are installed through the top slab.

At mid-span the *in situ* stitch connecting the adjacent cantilevers is concreted using a short length of formwork, as seen in Fig. 13.40. The width of the stitch is typically between 200 mm and 1200 mm depending on the designer's preference. The advantage of a narrow stitch is that no reinforcement is needed within the short length, while a wide stitch gives more room to manoeuvre the segments into position and to stress cantilever tendons on the end face if required.

In addition to the stitch formwork, the ends of the cantilevers are held in place by a frame that spans across the stitch and is fixed to the deck on either side, as shown in Fig. 13.41. If the cantilevers are slightly misaligned the frame is used to pull the ends of the deck to the correct position. It then holds the deck in place to prevent any adverse movements while the stitch is being concreted and until some of the permanent prestress is installed along the span.

With balanced cantilever erection the span lengths are arranged to suit the cantilevering out from each pier, keeping the two sides reasonably balanced.

@Seismicisolation

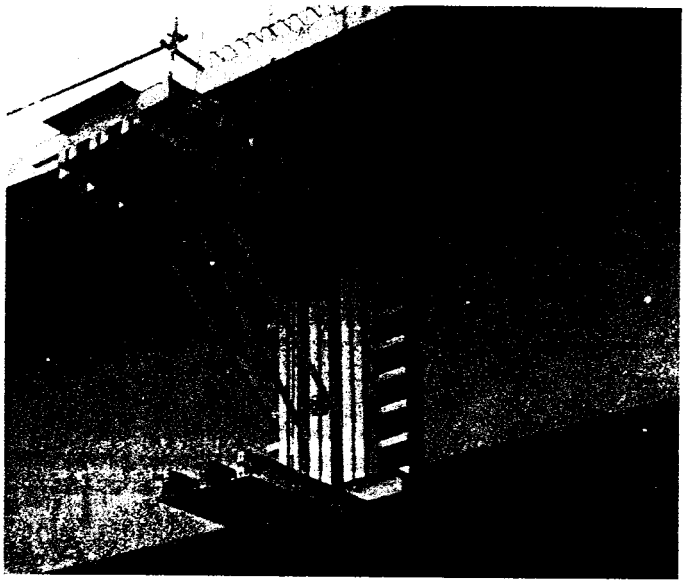


Figure 13.40. Shutter for in situ *stitch at midspan.*

For end spans, which are typically 60 per cent of the length of the adjacent span, the last few 'out-of-balance' segments are supported on temporary props or falsework until the span is completed and the deck made continuous through the installation of the prestress.

The deck is not continuous at movement joints over internal piers and the spans on either side are 'end spans'. In this situation the deck is constructed either with the segments placed on falsework until the adjacent balanced cantilevers are completed, or the first segments either side of the movement

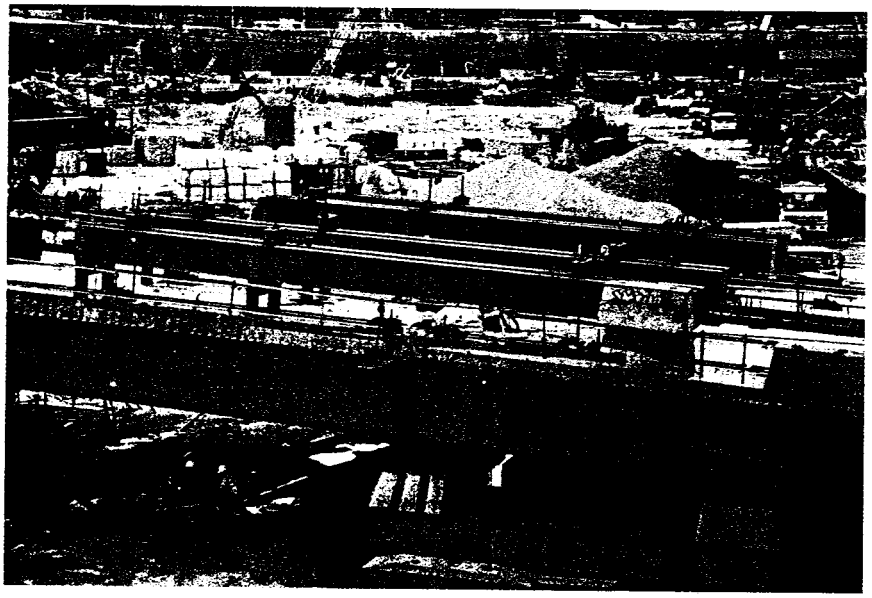


Figure 13.41. Frame to align cantilevers.

@Seismicisolation

joint are clamped together with temporary prestress and a balanced cantilever constructed in the normal way. After the adjacent cantilevers have been constructed, and the deck connected, the temporary prestress across the movement joint is removed and the movement joint formed.

Segment erection by the span-by-span method

For span lengths of less than 50 m, a common erection method is to place the segments span-by-span, as illustrated in Fig. 13.24(i). With this technique a gantry, either overhead or underslung, is used to support a complete span of segments which are pulled together by installing either temporary prestress or the permanent tendons. The gantry then releases the span of segments on to the bearings and launches itself forward ready to erect the next span. For spans above 50 m erection gantries become very heavy and tend to be uneconomic when compared to the balanced cantilever erection method.

Overhead gantries, as shown in Fig. 13.42, are supported off the piers or the completed deck. They move forward by 'stepping' onto the next pier in front of the erection and by running along the top of the already completed section of deck.

Segments are transported along the completed deck up to the back of the gantry where they are lifted up and moved out along the span. The individual segments are placed on hangers, as shown in Fig. 13.43, that are used to align the segment in position. When the complete span of segments is in place, and the permanent prestress installed, the gantry lowers the deck on to the bearings. Overhead gantries are able to deal with variations in span lengths and deck geometry and being above the deck they are not affected by ground-level constraints.

Underslung gantries, as in Fig. 13.44, are positioned beneath the deck normally supporting the segments under their side cantilevers or soffit. The gantries are supported at the piers by brackets fixed to the column top or by props from the foundations. They move forward by launching themselves and stepping between the piers. Underslung gantries are not suitable for decks

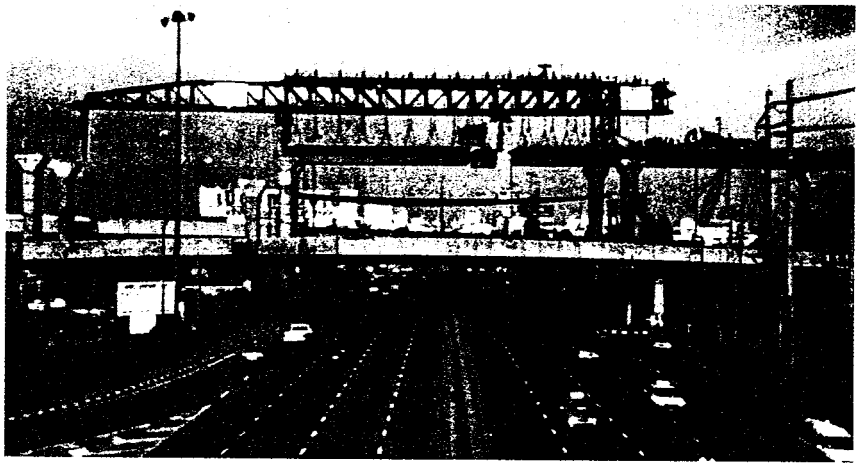


Figure 13.42. Overhead gantry for span-by-span erection. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

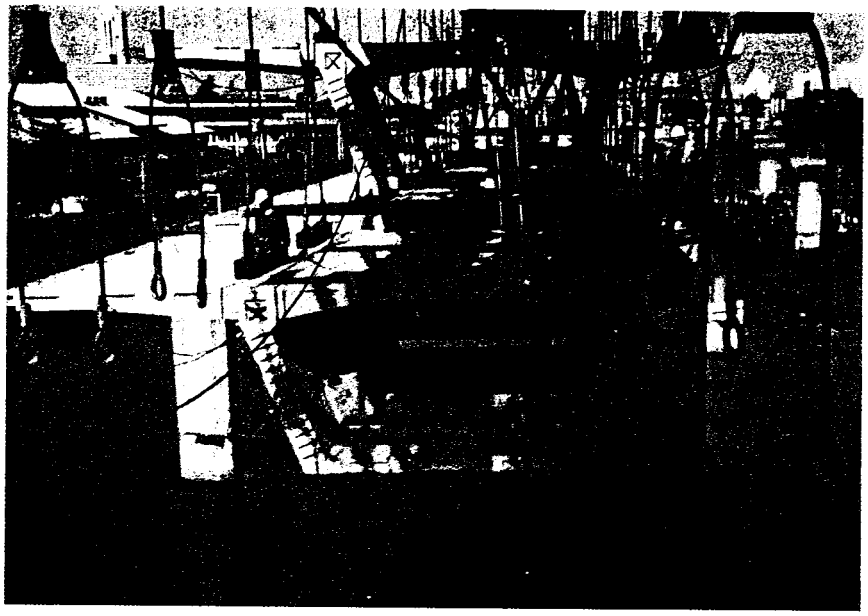


Figure 13.43. Segments hanging from gantry.

on a tight horizontal curve where the piers are significantly out of line and the segments are offsets from the straight chord between the piers. They also project beneath the deck which may cause temporary headroom problems when passing over existing roads or railways.

The segments are lifted on to the gantry with a crane or lifting frame, as seen on the deck behind the gantry in Fig. 13.44. They are then slid along the top of the gantry into position, as shown in the inset. When all the segments for the span are in place the longitudinal prestress is installed and the span released from the gantry on to the bearings, allowing the gantry to be launched forward on to the next pier to repeat the erection cycle.

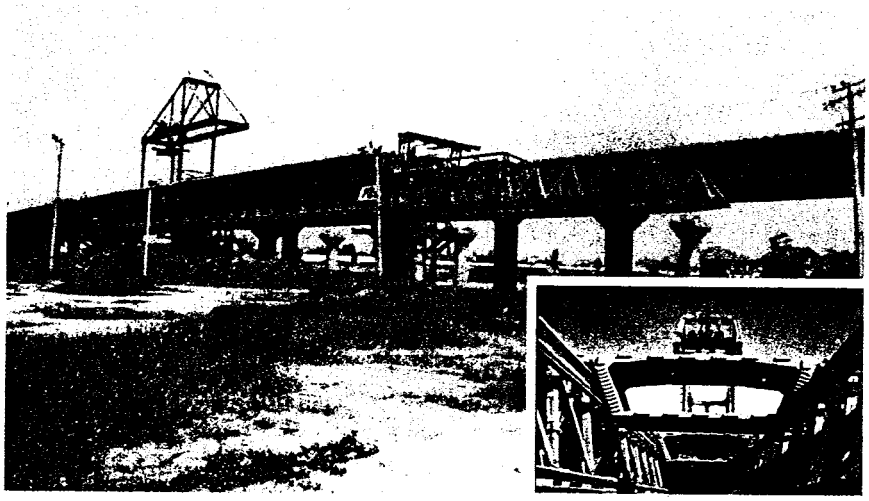


Figure 13.44. Underslung truss for span-by-span erection.

@Seismicisolation

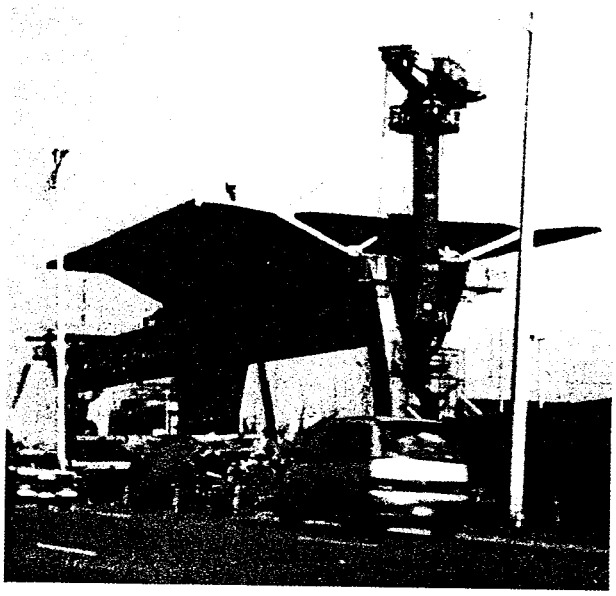


Figure 13.45. Lifting arm on underslung gantry. Reproduced courtesy of Jonathan Hiscock. Copyright reserved.

The deck in Fig. 13.45 was erected using an underslung gantry with a lifting arm at the front to position the segments. The gantry spanned between the piers and supported the segments under the box soffit.

Most dry-jointed segmental decks are constructed using the span-by-span erection method. With the segments supported by the gantry, temporary prestress is not needed other than to help position the segments.

With epoxy in the segment joints temporary prestressing bars are used to hold the segments together while the epoxy is setting. The erection process is simplified by applying the epoxy and stressing several segments together at one time.

Span-by-span erection is used on both continuous decks and simply supported spans; the process of erecting the segments is the same for either arrangement. With a continuous deck adjacent 'spans' are joined together with a narrow *in situ* concrete stitch typically 100 mm or 150 mm wide. This stitch is positioned adjacent to the pier segment or further into the span to suit the design and construction requirements. After the stitch concrete has hardened the continuity prestress tendons are installed to make the deck continuous.

The advantage of overhead gantries is that they are able to cope with a wide range of deck geometries and tight horizontal curves but they are generally more complex to design and assemble, while they are slower in erecting the segments when compared to an underslung gantry.

The advantages of underslung gantries are that they are generally simpler to design, assemble and operate as well as being easier to erect the segments, but they are used only with decks on a large horizontal radius and when headroom is not critical.

With the span-by-span placement method and epoxy in the segment joints, a typical 40 m span is usually erected every 2 or 3 days. With an underslung

@Seismicisolation

gantry and a dry-jointed deck an erection rate of up to one span a day is achievable.

Segment erection by the progressive placement method

With 'progressive' placing the erection starts at one end of the bridge and continues out progressively until the other end is reached, as illustrated in Fig. 13.24(iii). The deck is usually unable to support itself fully as it cantilevers out towards the next pier and temporary stays or props are required along the span, as shown in Fig. 13.46.

As the deck extends away from the pier it deflects under its self-weight and the props or stays are introduced to support the deck and reduce the bending moments and shears. The loads in the props or stays change every time another segment is erected or the lifting frame is moved forward. If the props are susceptible to settlement this is compensated for by incorporating a suitable jack into the arrangement.

When the deck reaches the next pier sufficient prestress is installed to allow removal of all the intermediate props, just leaving in the last prop nearest to the end of the deck. This last prop is used to jack the deck up to the correct level to allow the diaphragm segment to be placed over the bearings at the pier. Additional prestress is then installed to enable the deck to be lowered on to the bearings and the cycle repeated for the next span.

The segments are placed using either a crane, a lifting frame or a gantry similar to the balanced cantilever method described above, and they are erected either with epoxy in the joints or with dry joints if appropriate. *In situ* concrete stitches are often introduced along the deck to assist in controlling the alignment.

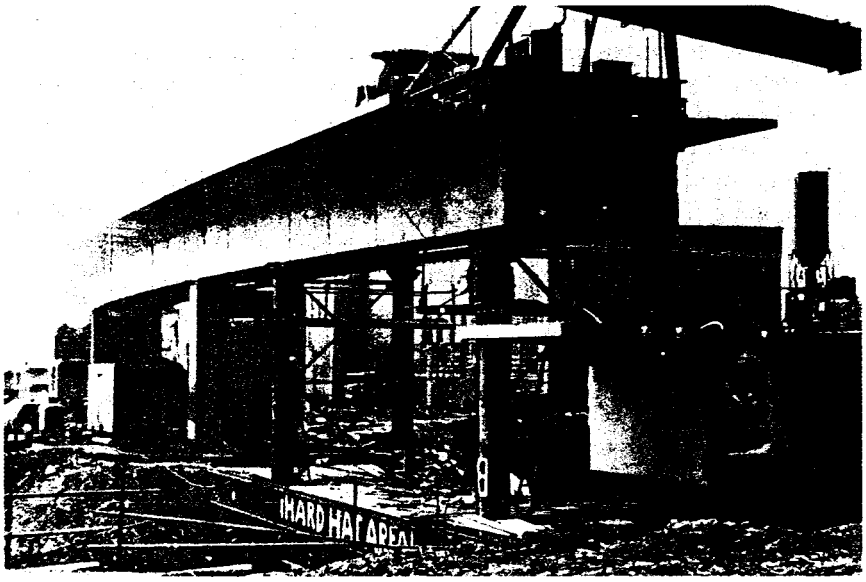


Figure 13.46. Progressive placing of segments with props.

@Seismicisolation

The disadvantages of progressively placing the segments are the slow erection rate when compared to the other techniques, and that it is not well suited to high decks or where they pass over obstructions such as rivers and highways. The need for regular propping or staying of the deck and slow erection tends to make this method more expensive compared to the balanced cantilever and span-by-span methods.

Design aspects associated with precast segmental decks

The design of precast segmental decks is similar to that of *in situ* box girders as described in Chapter 12, with the main difference being due to the presence of the joints. The joints create a discontinuity in the concrete and longitudinal reinforcement, resulting in a reduction in performance under both serviceability and ultimate limit states.

Prestress tendon layout

For precast segmental box girders the prestressing tendon layout is arranged so that the ducts pass through standard positions at each segment joint. This is to simplify the construction of the bulkhead shutter used to cast the segments. The tendon anchorage positions and arrangements are also standardized to simplify the shutter arrangement. Anchors located on the segment end face are commonly used with balanced cantilever construction, while top and bottom blisters located in the corners between the webs and the slabs are fixed at standard positions within the segment.

The prestress layout and tendon profile is dependent on the method of construction. A typical arrangement for a balanced cantilever deck is illustrated in Fig. 13.47. The cantilever tendons are used to support the deck during construction, and the continuity tendons are installed after the cantilevers are joined together. Cantilever tendons are anchored on the end face of the segments or on top blisters inside the box. Continuity tendons are anchored on blisters to give access for installing and stressing. Where external tendons are used they are held in position by deviators and anchored on the diaphragms or on anchor blocks inside the box.

Simply supported decks require the tendons to be in the bottom of the section and a simple draped profile, as illustrated in Fig. 13.48, is used. In this example external tendons are anchored on the diaphragms and at deviators along the box, with the deviators also holding the tendons in a draped profile.

Shear keys at joint

Shear keys are provided on the surface of the joint to help align the segments during erection and to transfer the shear forces between the segments. With epoxy joints the shear transfer through the keys is needed only during the erection and until the epoxy has set. Where dry joints are used the shear keys provide the shear transfer in both the erection and permanent conditions.

@Seismicisolation

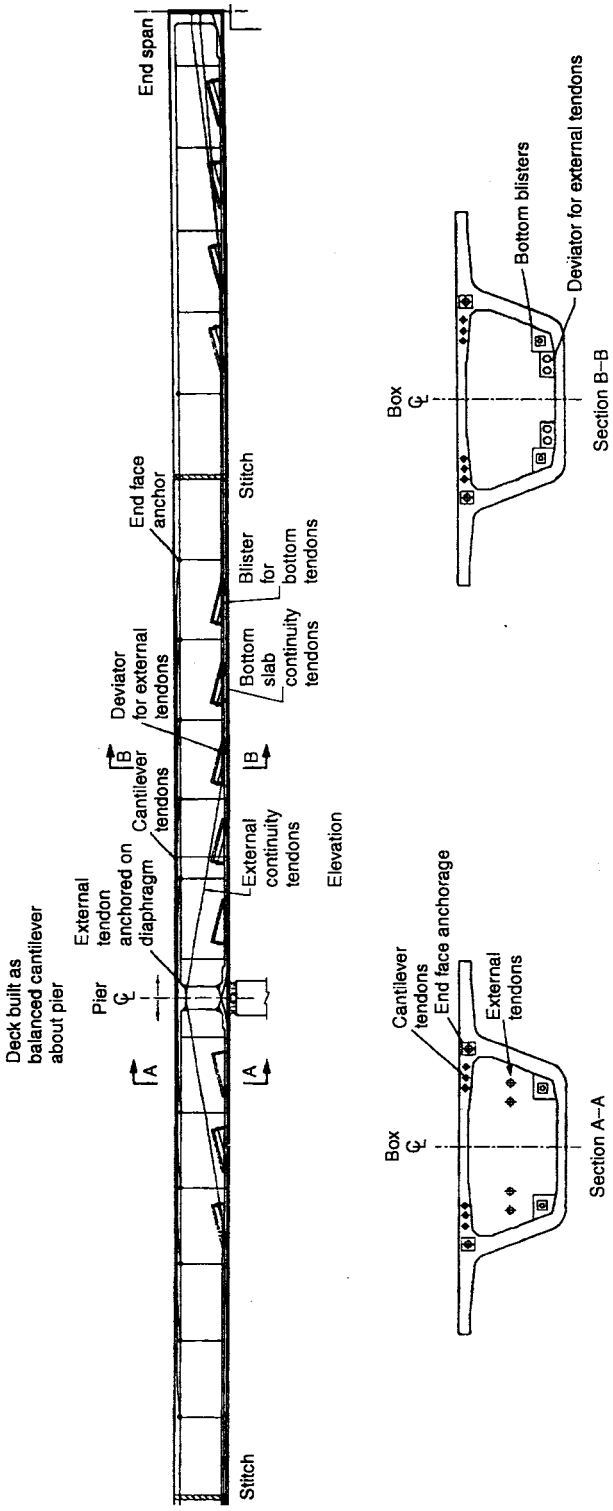


Figure 13.47. Prestress layout for balanced cantilever deck. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

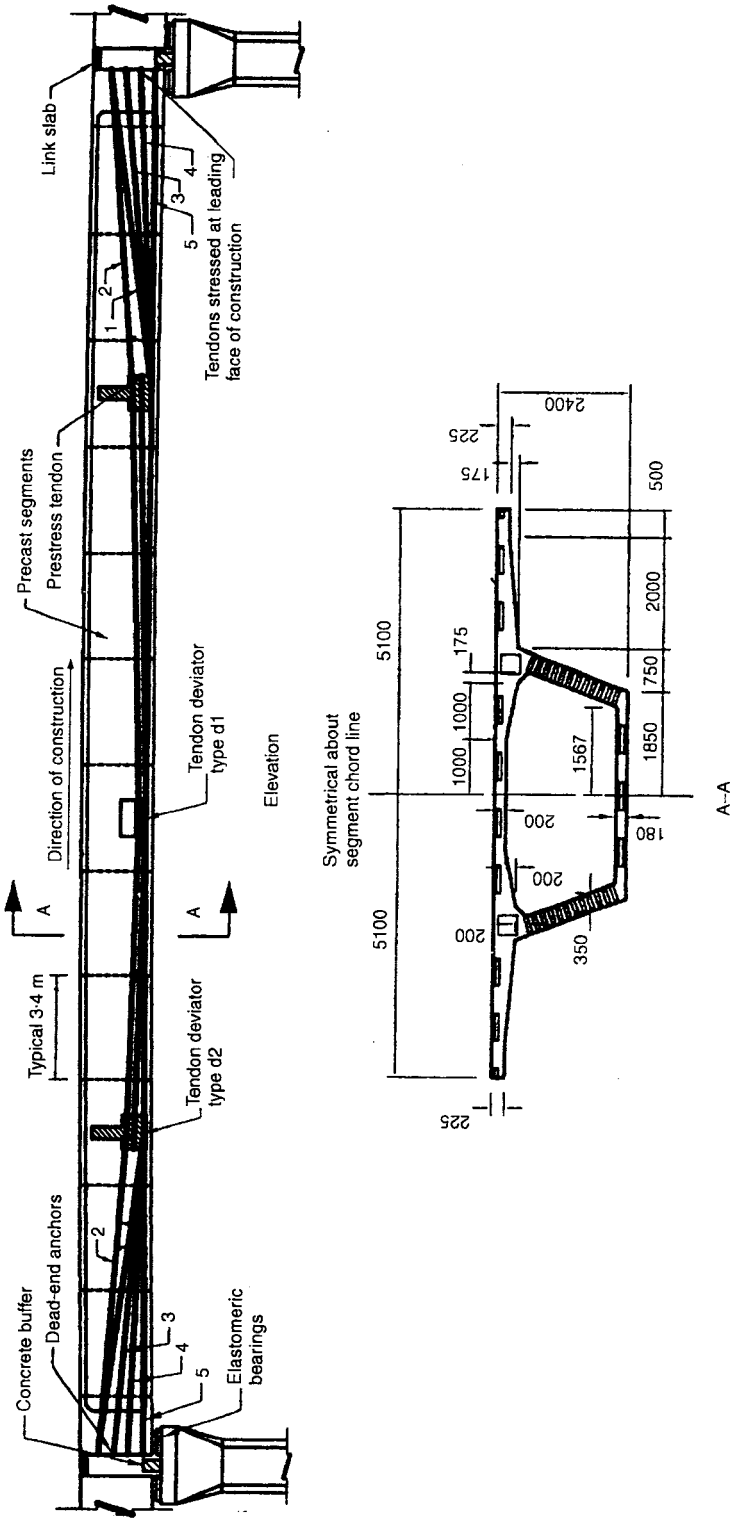


Figure 13.48. Prestress layout for simply supported deck. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

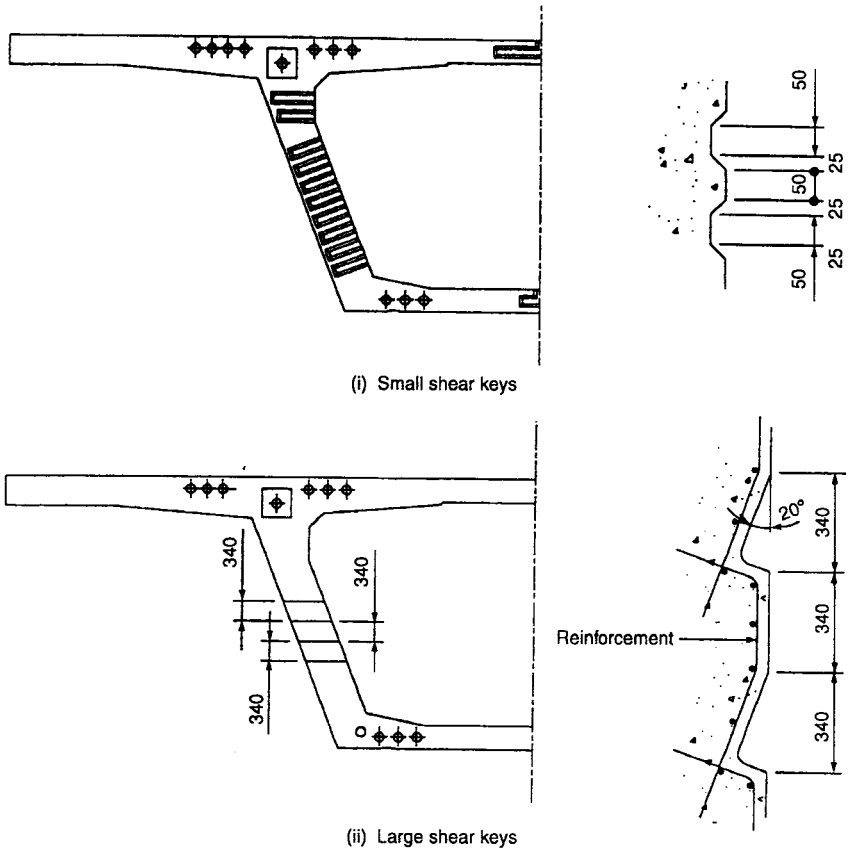


Figure 13.49. Shear key arrangements.

Either a few large or multiple small shear keys are used, as illustrated in Fig. 13.49. Both are designed to prevent a shear failure occurring through the joint. It is common practice to provide additional shear keys above the minimum number required by the design to allow for possible damage to some of the keys during storage or handling.

Considering the shear friction concept as given in AASHTO [2], the capacity of the shear keys is estimated using a coefficient of friction of 1·4, giving:

$$V_k = 1{\cdot}4[A_{sk} \cdot f_{pk} + A_r \cdot f_y]$$

with the values adjusted by the material factors or strength reduction factors as appropriate.

Large shear keys are designed as a concrete corbel with appropriate reinforcement provided, as shown in Fig. 13.50. The multiple small shear keys do not require reinforcement.

Design with epoxy or concrete joints

Segmental decks with epoxy joints are designed similar to *in situ* box girders. The main difference is that the longitudinal bending stresses across the joint

@Seismicisolation

Figure 13.50. Reinforcement for large shear keys.

are always kept in compression under all loading conditions to prevent the unreinforced concrete adjacent to the joints cracking.

The epoxy in the joint creates a bond of greater strength than the concrete itself, and the ultimate shear design is carried out as if the deck is monolithic, although when designing to AASHTO [3] the strength reduction factors for shear are reduced for segmental construction with epoxy joints.

Where wide, cast *in situ* concrete joints are incorporated into the structure, BS 5400 Part 4 (1990) [4] requires the shear force under ultimate loads to be not greater than $0{\cdot}7(\tan\alpha)(0{\cdot}87P)$ to ensure against a frictional failure across the joint. Tan α depends on the interface and for roughened faces is taken as 0·7 during erection and 1·4 after completion of the bridge. P is the horizontal component of the prestress force after all losses.

Design with dry joints

Tests carried out by Macgregor *et al.* (1989) [5] and Rabbat and Sowlat (1987) [6] demonstrated that a dry-jointed segmental deck behaves in a similar manner to an *in situ* or epoxy-jointed structure up to the point when longitudinal bending stresses give rise to decompression of the joints or shear stresses cause the joint to slip. With segmental construction the joints are always kept in compression under service conditions and the longitudinal stress check and other serviceability checks are carried out as if the deck is monolithic.

@Seismicisolation

The shear keys at dry joints provide the mechanism for transferring shear forces across the joints at all stages of the structure's life. Tests carried out by Koseki and Breen (1983) [7], and Buyukozturk *et al.* (1989) [8] have shown that dry joints with multiple small shear keys spread out down the webs are more efficient than large individual keys. Joint slippage occurs after the initial frictional resistance of the mating concrete surfaces is exceeded. The tests indicated that slipping starts after mobilizing a friction of at least 0·5.

The webs are checked to ensure that the unfactored shear force at the joint is less than 40 per cent of the compressive force on the web to provide an adequate factor of safety against joint slippage. The shear force includes the vertical and torsional shears, less the vertical component of prestress. The compressive force includes the prestress modified by the bending stresses present. Suitable reduction factors should be applied to the prestress force estimated from the design. For this 'serviceability' check the shear key effects are not considered as slippage starts before the shear keys strength is mobilized.

The transverse design is carried out as for other box arrangements with an additional check done to ensure that there is sufficient frictional resistance on the joint surface of the top slab to distribute local wheel loads' effects longitudinally across the joint and to prevent joint slippage. To assist in the load transfer across the joint, small shear keys are placed across the top slab, as seen on the segment in Fig. 13.43. Where an *in situ* concrete barrier or edge beam is built on to the top slab its joints should not coincide with the segment joints, so that they assist in providing additional continuity along the deck.

For the ultimate moment design, when the deck is subjected to increased loading beyond the serviceability state the joints decompress and begin to open up, as illustrated in Fig. 13.51. As the load increases more joints along the span open up and the openings extend over a greater part of the segment depth. As the ultimate moment capacity is approached the deck deflections rapidly increase and final failure occurs on the compression side by crushing of the concrete due to excessive strain. With external tendons the strain in the tendons is averaged out over the tendon length and the tendons do not reach their breaking load.

When dry-jointed decks approach their ultimate bending capacity they undergo high deflection with the curvature generated by rotations concentrated at individual segment joints. This is in contrast to monolithic or epoxy-jointed decks, where the rotations and tensile cracks are more evenly spread out. These concentrations of rotation cause a dry-jointed deck to fail at a lower moment than the equivalent monolithic or epoxy-jointed arrangement. This is recognised in AASHTO [3] by the lower strength capacity reduction factor, noted in the section 'Flexural strength' in Chapter 5.

Ultimate shear behaviour of a dry-jointed deck relies on the friction capacity of the joint and shear keys. If the friction between the mating surfaces is exceeded slippage will occur until the shear keys are fully engaged. Increased shear is then resisted until either the bearing resistance capacity or the frictional resistance capacity of the keys is reached, at which stage the keys will either fail by crushing or by shearing off. The shear capacity of a dry joint is significantly less than for an epoxy joint or monolithic structure, with up to 40 per cent reduction being observed in tests.

@Seismicisolation

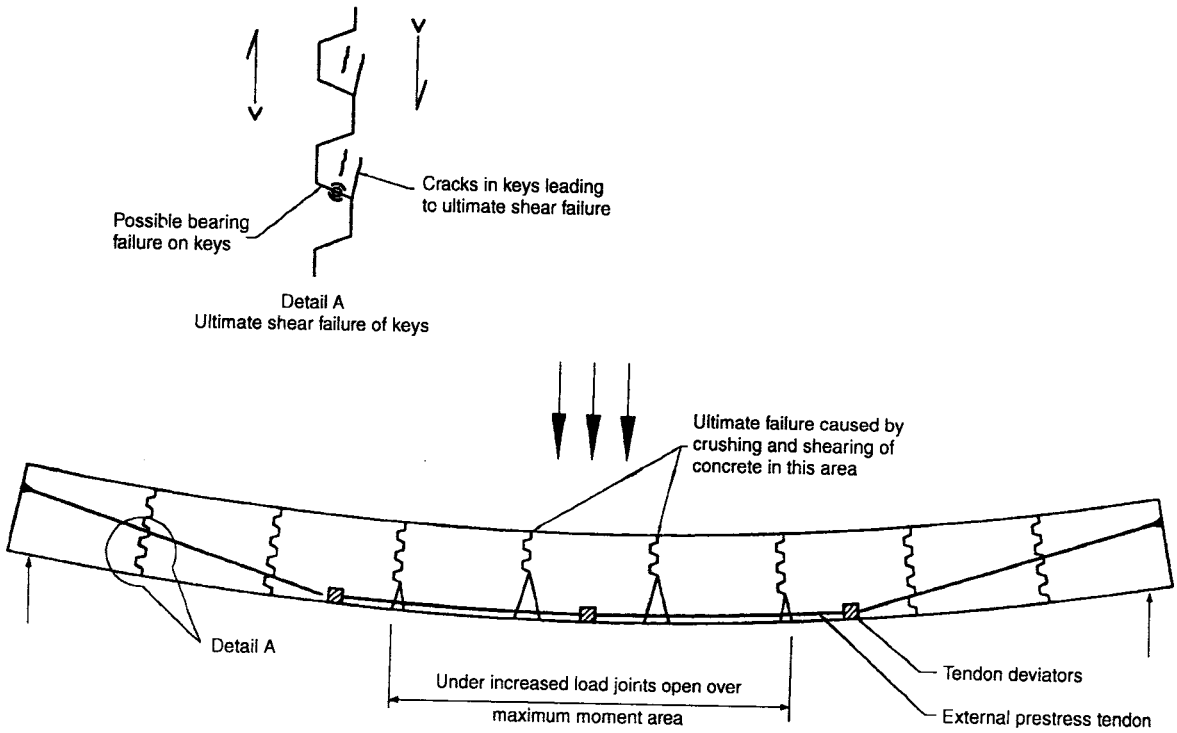


Figure 13.51. Ultimate behaviour of beam with dry joints and external tendons.

The ultimate shear capacity of a web is made up of two components, the resistance of the keys plus the frictional resistance on the remaining web area. Based on the AASHTO [2] shear friction concept the two components are estimate as follows:

$$\text{Shear resistance of multiple keys} = 1{\cdot}4 f_c \times \text{shear key area}$$

$$\text{Frictional resistance of rest of web} = 0{\cdot}6 \times f_c \times (\text{web area} - \text{shear key area})$$

$1{\cdot}4$ = ultimate friction co-efficient for monolithic concrete

$0{\cdot}6$ = ultimate friction co-efficient for abutting concreting

f_c = co-existent compressive stress on the web

Appropriate factors of safety should be applied to the capacity to suit the design codes being used.

With the concrete faces in contact the shear keys are restrained from spalling and bursting. To prevent a bearing failure of the shear keys the total applied ultimate shear force in the webs should not exceed $0{\cdot}6 f_{cu} \times$ shear key bearing area.

Torsional shear stresses in the top and bottom slabs should not exceed the frictional resistance available in a similar manner to that in the webs.

@Seismicisolation

Typical segment details

The features and details of precast concrete segments are discussed above and depend on the particular requirements of the project where they are being used. Figure 13.52 illustrates a typical segment section for an urban viaduct with spans of 45 m.

The webs and bottom slab are thicker near the piers than at mid-span to cater for the higher shear forces and bending compression respectively. The transition in the web and bottom slab thickness is either taken out uniformly over one segment length or is stepped up or down at one end of the segment, as illustrated in Fig. 13.53.

Reinforcement in the segments is standardized as much as possible with repetitive arrangements in the bottom slab, webs and top slab to facilitate the factory assembly of the units.

Deck erection

Similar to *in situ* box girders, the stresses, strength and stability of the deck and substructure and the temporary works designs are checked at each stage of erection.

In addition to the loading noted in Chapter 12, precast segmental decks are checked during erection for a load of up to 50 kN applied at the end of the deck for the segment lifting beam.

Where a lifting frame is fixed on the deck to lift the segments into place, the weight of this is typically between 200 kN and 500 kN depending on the size and weight of the segment being lifted.

Gantries used to erect the segments are usually supported off the substructure or completed deck and apply significant loads which may govern the design of some parts of the structure. For a deck erected by the balanced cantilever method the gantry supports only one segment at a time and it typically weighs between 200 tonnes and 400 tonnes for spans between 50 m and 120 m. For a deck erected by the span-by-span method the gantry supports a

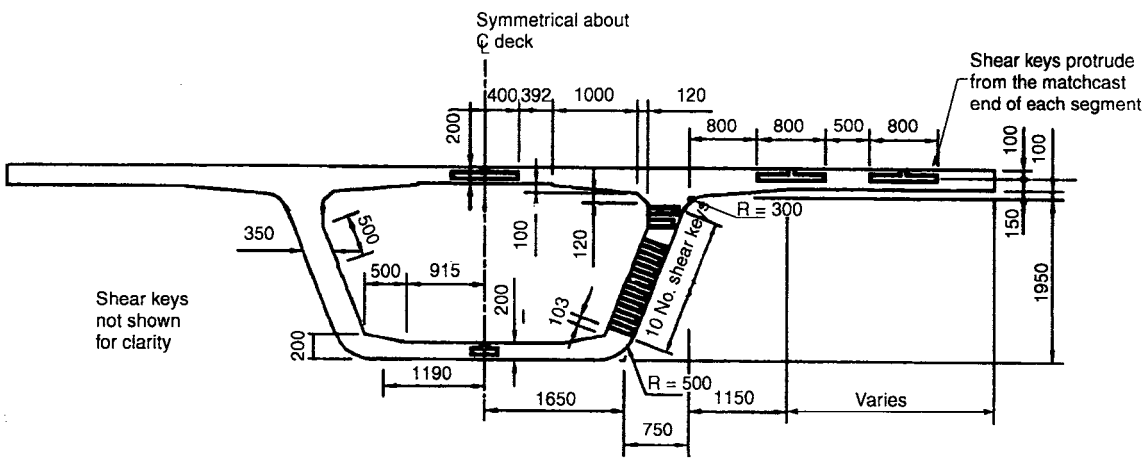


Figure 13.52. Typical segment arrangement. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

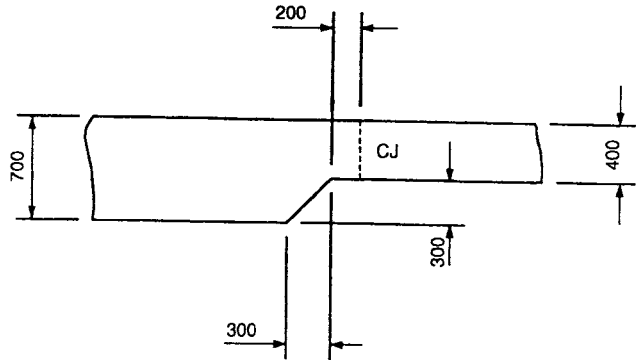


Figure 13.53. Web and bottom slab transition detail. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.
Note: CJ = construction joint.

complete span of segments and typically weighs between 150 tonnes and 350 tonnes for spans between 35 m and 50 m.

The design and stability of a deck erected as a balanced cantilever is usually checked in the temporary condition with one segment out-of-balance, to match the construction sequence adopted. If there is a risk of a segment being dropped, resulting in a situation with two segments out-of-balance, then this is also checked in the design. Several projects have developed erection schemes where segments are erected in pairs to keep the cantilevers balanced during the construction and to minimize the risk of any out-of-balance forces from occurring.

The design of a deck erected by the span-by-span method is generally less sensitive to temporary loading during construction as all the segments are supported from a truss or gantry until the permanent prestress is applied; however, each stage of their construction is checked to ensure that the design and stability of the permanent and temporary works is adequate.

Decks erected using the progressive placing of segments supported by props are affected by settlement of the props, which generate additional stresses in the deck. A jacking system is incorporated into the prop arrangement and used to compensate for any expected settlements.

Decks built span-by-span on gantries or trusses are checked during application of the prestressing and the transfer of load from gantry to the deck structure to ensure the stresses are within acceptable limits. As the prestress is applied the deck lifts up from the gantry; however, as the weight is relieved the gantry deflects upwards and continues to apply a force on to the deck. The gantries are usually more flexible than the deck structure and continue to apply significant load on to the deck even after the deck is fully prestressed. This upward force on the deck could overstress the concrete if not controlled by releasing the deck from the gantry in stages as the prestress is installed.

References

1. Podolny, W. and Muller, J.M. (1982) *Construction and Design of Prestressed Concrete Segmental Bridges*. New York: John Wiley & Sons.
2. AASHTO (1996–2002) *Standard Specification for Highway Bridges, 16th edition*. Washington, DC: American Association of State Highway and Transportation Officials.

@Seismicisolation

3. AASHTO (1999) *Guide Specification for Design and Construction of Segmental Concrete Bridges, 2nd edition*. Washington, DC: American Association of State Highway and Transportation Officials.
4. BS 5400 (1990) *Steel, Concrete and Composite Bridges, Part 4, Code of Practice for the Design of Concrete Bridges*. London: British Standards Institution.
5. MacGregor, R.J.G., Kreger, M.E. and Breen, J.E. (1989) *Strength and Ductility of a Three Span Externally Post-tensioned Segmental Box Girder Bridge Model*. Centre for Transportation Research, The University of Texas at Austin. Research report 365-3F, January.
6. Rabbat, B.G. and Sowlat, K. (1987) 'Testing of Segmental Concrete Girders with External Tendons', *PCI Journal*, March–April, 86–107.
7. Koseki, K. and Breen, J.E. (1983) *Exploratory Study of Shear Strength of Joints for Precast Segmental Bridges*. Centre for Transportation Research, The University of Texas at Austin, Research report 248-1, September.
8. Buyukozturk, O., Bakhoum, M. and Beattie, S.M. (1989) 'Shear behaviour and strength of joints in precast concrete segmental bridges'. Paper submitted to the *Journal of Structural Engineering of the American Society of Civil Engineers*, January.

@Seismicisolation

14

Precast full-length box girders

Introduction

Several major bridge crossings and long urban viaducts have been constructed using concrete box girder decks precast in long sections and lifted into place. The investment needed to set up large casting facilities and to provide special heavy lifting equipment limits this form of construction to projects where long lengths of deck are being built which require a large number of units.

There are several variations to the technique, but they all involve precasting a complete span or part span in a casting yard, transporting the units to site and placing them into position. The units may be over 100 m long and weigh thousands of tonnes, and usually they require special equipment to move and erect them.

The technique was adopted for the viaducts on the 25 km long Saudi Arabia–Bahrain Causeway shown under construction in Fig. 14.1. The 50 m spans were formed using a combination of 66 m and 34 m long precast units. The construction was carried out over a four-year period and was completed in 1985.

The Central Viaduct in Fig. 14.2, part of the Vasco da Gama crossing in Portugal, used 77·6 m long precast units weighing 2200 tonnes each to form the deck. The sections were cast on-shore and transported on barges before being lifted into place. Initially positioned as simply supported on the piers, the units were connected to make the deck continuous.

In many modern cities, new road and rail systems are being constructed to relieve the congestion in the existing infrastructure with the requirement for rapid construction and minimum disruption. These systems often include urban viaducts extending over many kilometres and they provide the opportunity to use full-length precast units to construct the deck. Figure 14.3 shows the Singapore MRT, built in the 1980s and 90s, which adopted a standard full-length precast deck design for the long lengths of viaduct. The units were cast adjacent to the viaduct under construction and transported along the completed deck before being lifted into place by a gantry.

The deck for the 13 km long Confederation Bridge, in Fig. 14.4, was precast using two types of units. The main units were precast in lengths symmetrical about the pier and then lifted in as a balanced cantilever.

@Seismicisolation

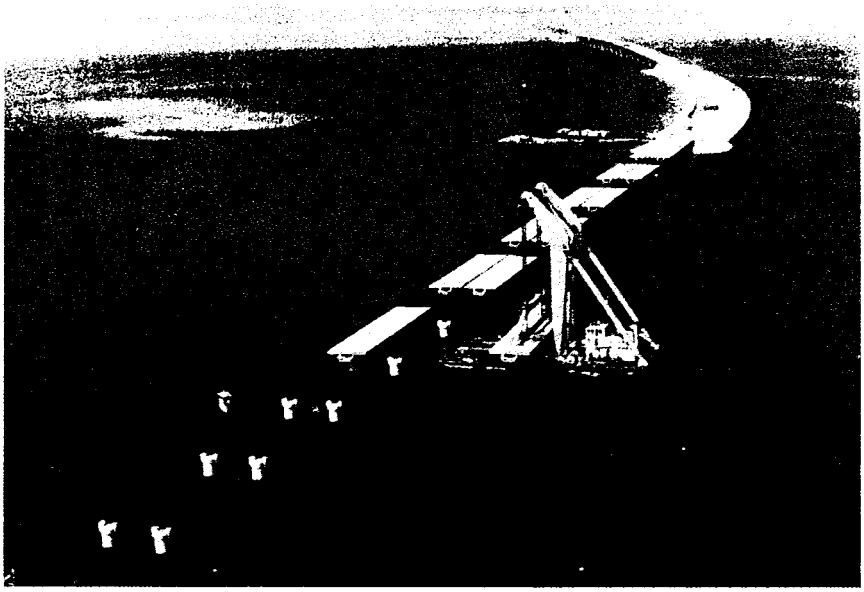


Figure 14.1. Saudi Arabia–Bahrain Causeway. Reproduced courtesy of Ballast Nedam BV. Copyright reserved.

Shorter precast units were placed over the mid-span, between the ends of the two adjacent cantilevers, to complete the arrangement.

Full-length precast box girders have the advantages of rapid deck erection and repetition in their construction and factory conditions for casting the units. They are economic for very long viaducts with many units, where

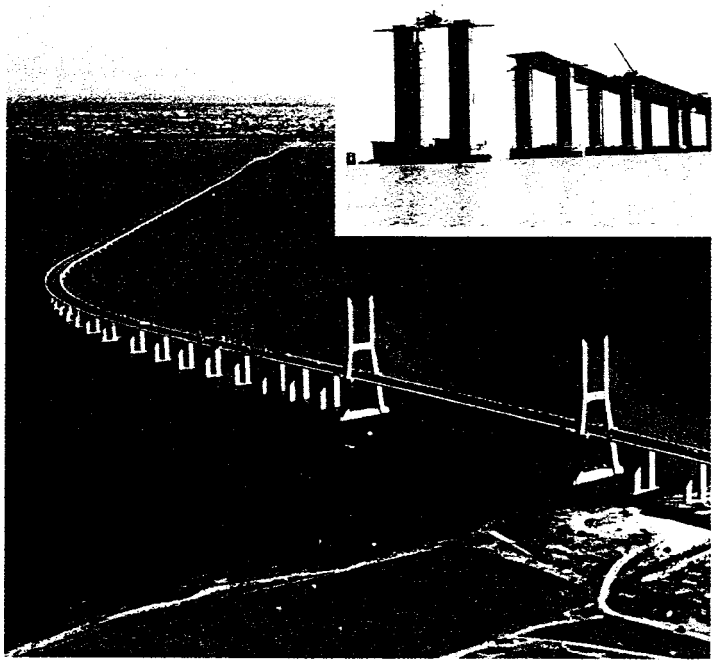


Figure 14.2. Vasco da Gama crossing, Portugal. Reproduced courtesy of Yee Associates and Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

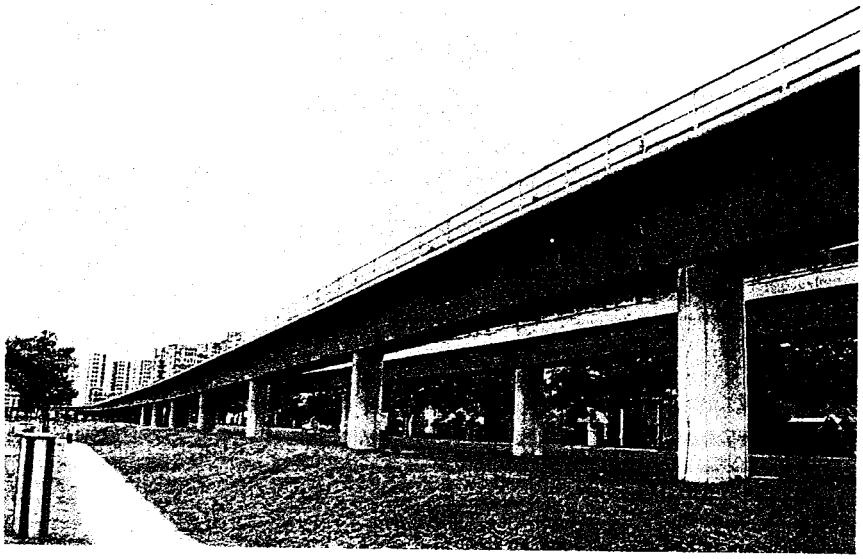


Figure 14.3. Singapore MRT.

the cost of the casting facilities and special equipment is spread out over many uses. Their disadvantages include the high cost of the large lifting and transporting equipment and the extensive areas needed to cast and store the units. They are not suited for highly curved decks as these complicate the transporting and erecting of the units.

Smaller structures, such as footbridges, often have precast decks spanning the full length between piers and these could also be classified as precast

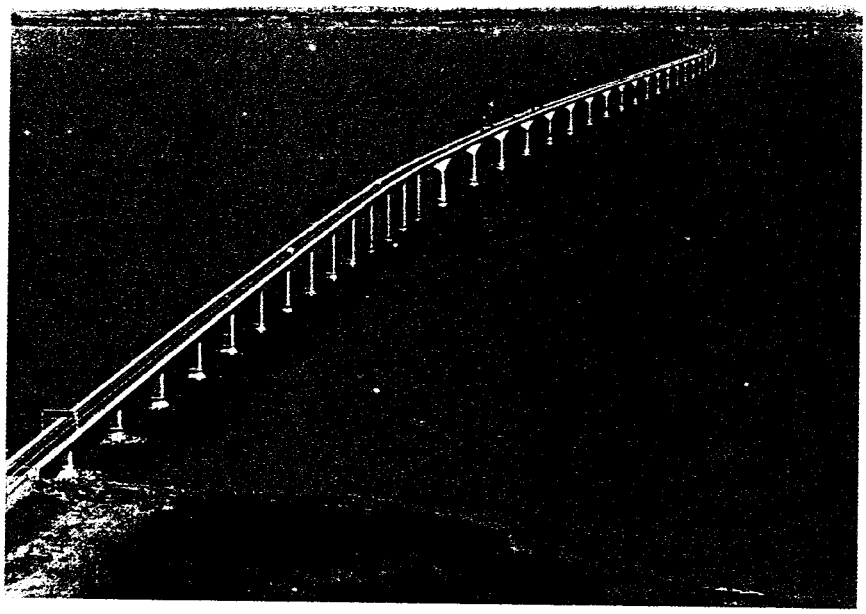


Figure 14.4. Confederation Bridge, Canada. Reproduced courtesy of Ballast Nedam BV. Copyright reserved.

@Seismicisolation

full-length girders, but they are nearer to the precast beam arrangement described in Chapter 10. This chapter covers the application of the technique to the larger structures and longer spans.

General arrangement

Cross-sections for full-length precast decks are usually similar to the single-cell box girder arrangement used for *in situ* or precast segmental construction, as described in Chapters 12 and 13. A box section is efficient from the design aspects and provides robust units for moving and erecting.

The transverse cross-section arrangement depends on the traffic requirements which dictate the top slab width. For light-rail systems, as shown in Fig. 14.3, the top slab is usually only 5 m wide if carrying a single track, or up to 8 m wide with a double track. This results in a narrow box arrangement with short side cantilevers. For highway bridges, with a single box carrying several lanes of traffic, the top slab width is usually greater than 12 m allowing a wider box arrangement to be used, similar to Fig. 12.3.

The precast deck units have diaphragms at the pier positions, similar to the other forms of box construction. The units are usually placed on bearings to simplify the construction process, but they may be made integral with the piers with an *in situ* concrete connection.

The units are often placed directly on the piers and left as simply supported spans, or they are joined together with an *in situ* concrete stitch to form a continuous structure.

The main precast units on the Confederation Bridge were 192 m long and weighed 8200 tonnes. With spans of 250 m the deck units were haunched, 14 m deep at the piers and 4·5 m deep over the mid-span. The maximum size of the precast units depends on the capacity of the lifting and transporting equipment available; however, on the large-scale projects where this type of construction is used it is common to design and build new equipment specifically for the length and weight of the units to be handled.

Either internal or external prestressing tendons, or a combination of both, are used to provide the prestress with the normal anchorage arrangements and blister and deviator details adopted. Tendon layouts depend on the deck arrangements and the erection techniques adopted, with layouts similar to the *in situ* box girder construction described in Chapter 12.

Casting and storage of the units

The casting yard is usually located near the bridge site to facilitate the transfer of the units. A large area is needed to provide sufficient space to build and store the precast units. The casting yard is arranged with areas for bending and assembling the reinforcement and for casting the segments. Separate areas are provided to store the units while they gain strength and wait their turn to be erected.

Good ground conditions are required at the casting yard to support the heavy units or piling must be used under the casting and storage areas to

@Seismicisolation

prevent differential settlements, which may cause the units to distort and become overstressed.

Where adequate concreting facilities are available, the units are often cast in one pour with full-length outer and inner shutters used, as shown for the Taiwan High-Speed Rail project in Fig. 14.5. Casting the units in one pour reduces the construction time although the inner shutter arrangement is usually more complex, and placing the large volume of concrete requires careful planning and control.

The large units may be cast in several stages to simplify the formwork and concreting operation, using techniques similar to *in situ* concrete box girders, as described in 'Construction, span-by-span', Chapter 12. Long units are sometimes divided into a number of shorter segments, cast in stages similar to the long-line technique described in 'Casting of segments', Chapter 13.

The factory conditions for manufacturing the units allow maximum repetition within the different construction activities. To reduce the casting cycle-times the reinforcement is usually pre-assembled and the complete cage moved into the casting cell. Figure 14.6 shows the reinforcement for a full-length span unit being assembled in a jig before being moved into the shutters.

In Fig. 14.7 the reinforcement and shutters have been assembled and the concrete is being placed for the complete unit. Concreting usually starts down each web, until it flows into the bottom slab, at which time the bottom slab is poured. The webs are then filled up and finally the top slab is cast. The large quantities of concrete and long lengths of pour can cause problems with keeping the surface of the concrete fresh without cold joints forming, and retarders may be needed in the concrete mix in some circumstances.

After casting each unit the concrete is cured until it reaches sufficient strength for the prestress tendons to be installed and stressed, and for the

(i) Outer shutters (ii) Inner shutters

Figure 14.5. Casting cell shutters. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

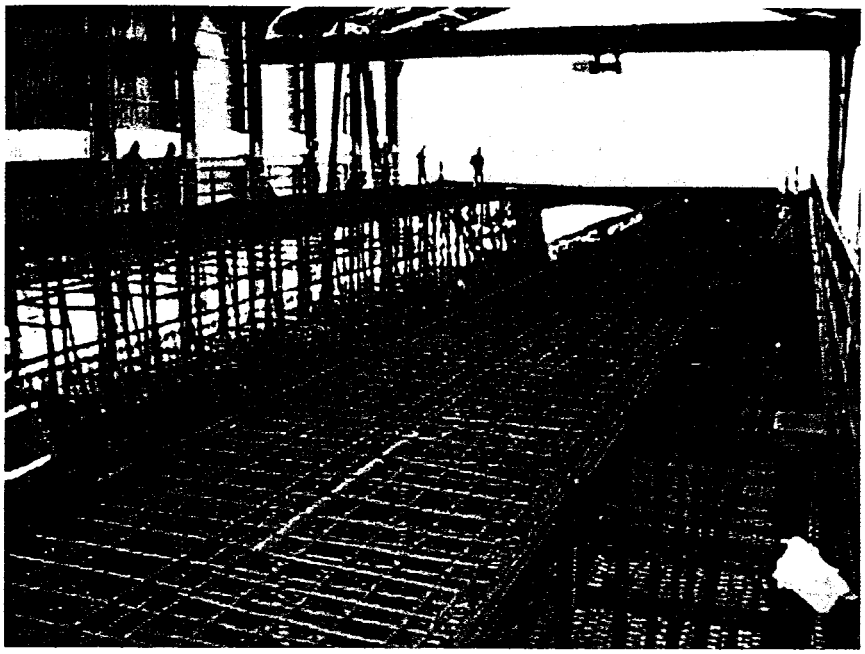


Figure 14.6. Pre-assembled reinforcement. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

unit to be moved. The unit is then taken to a storage area to complete its curing, and until it is needed for erection. A completed unit supported on temporary plinths in the storage area is shown in Fig. 14.8. The temporary bearings must be carefully set to provide an even support to the corners of

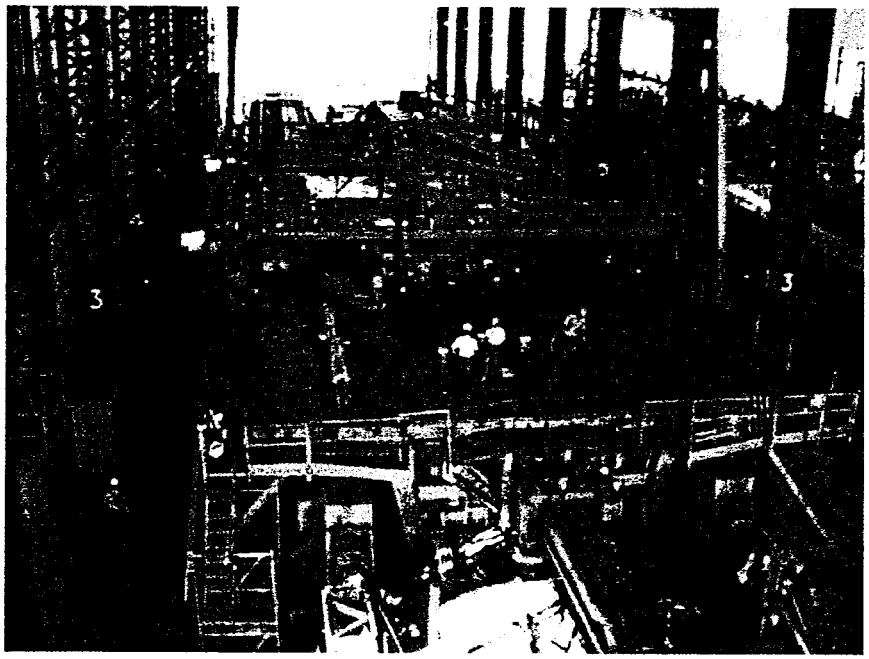


Figure 14.7. Unit being concreted. Reproduced courtesy of VSL Intrafor Asia. Copyright reserved.

@Seismicisolation

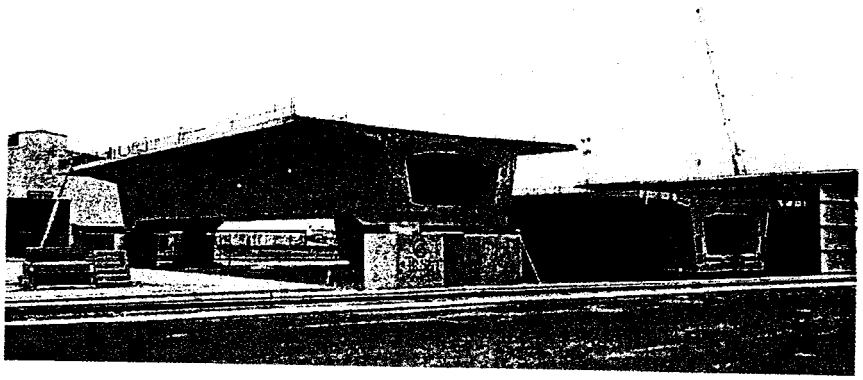


Figure 14.8. Completed precast unit stored. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

the unit and to prevent the units from twisting or warping while the concrete is still maturing.

Transportation of the units

Moving the large units requires special equipment to carry the heavy loads. The units are moved about the casting and storage areas using gantries or lifting frames, as seen in Fig. 14.9. The gantries or frames are either rail mounted or run on wheels depending on the casting and storage yard set up. Rail mounted arrangements are often simpler to assemble and operate, while wheels provide more flexibility when moving the units about the yard.

Special transporters are used for moving the units longer distances over land, similar to the one shown in Fig. 14.10. The multi-axial arrangement

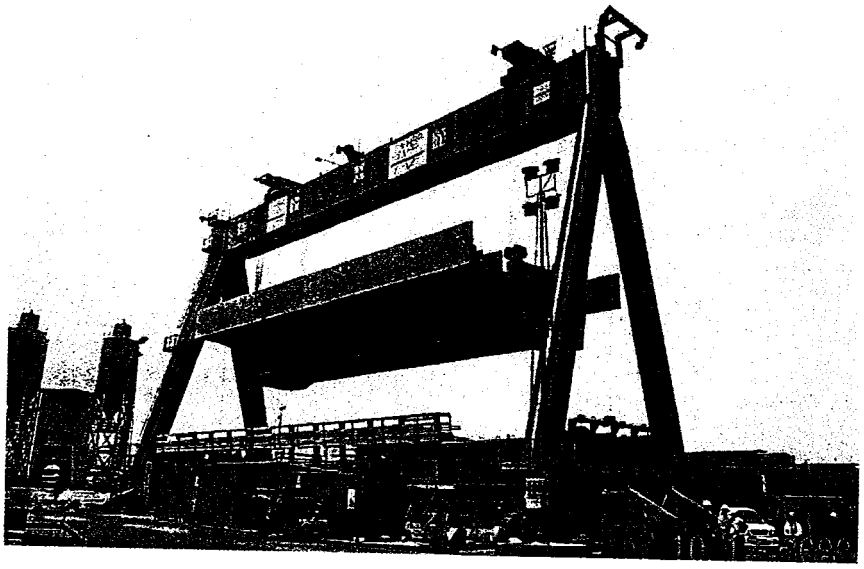


Figure 14.9. Beam with lifting gantry. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

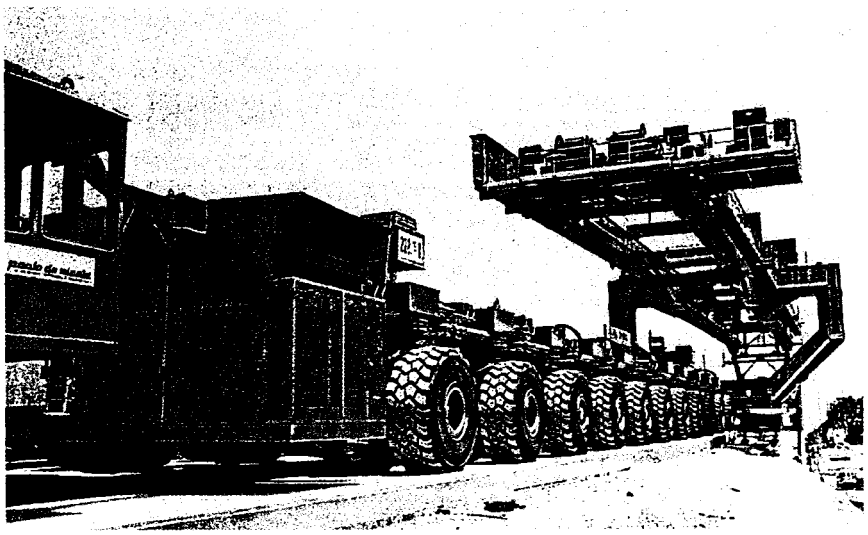


Figure 14.10. Beam transporter. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

spreads out the large loads on the highway pavement. Problems often occur when units are moved over existing bridges or highways with utilities underneath. The existing bridges and highways may need strengthening or protection to allow the transporters to pass over.

The casting and storage areas for the Singapore MRT were next to the line of the viaduct and the units lifted directly on the completed deck, as seen in Fig. 14.11. The 165 tonne and 23 m long units were moved along the deck on a twin bogey arrangement to an erection gantry that lifted them into position on the piers.

Barges are a convenient way to transport the units where a bridge is being constructed over water. Often a combined barge and lifting facility is used to

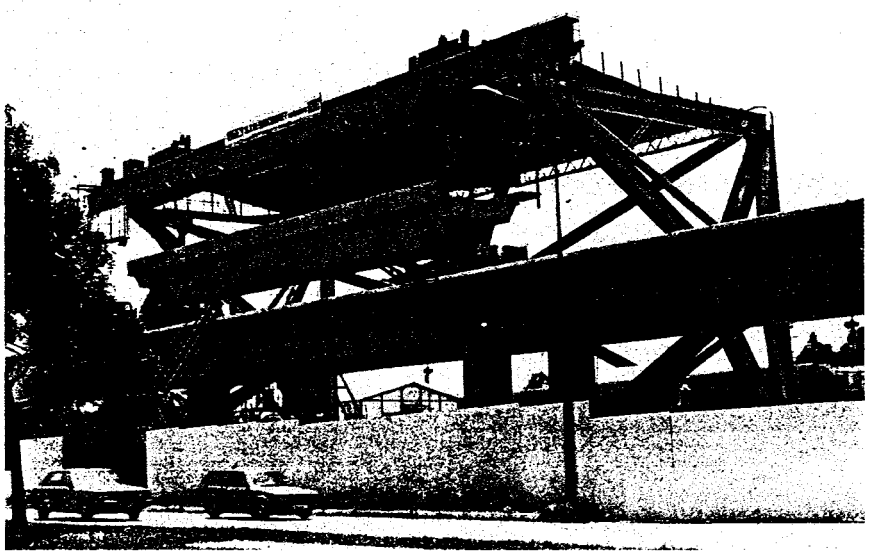


Figure 14.11. Beam being moved on to deck. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

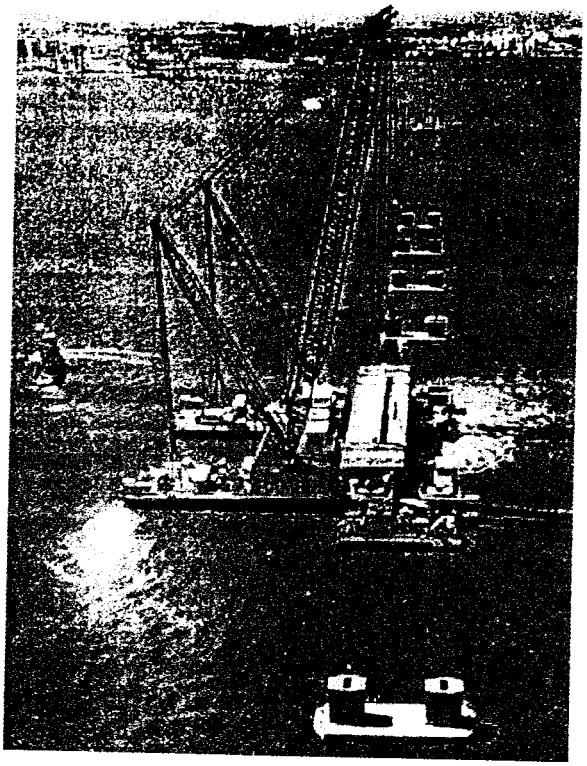


Figure 14.12. Beam being transported by barge. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

pick up the units from a jetty and carry them out to the erection site. The casting and storage yards are located next to the river or sea being crossed to provide easy access to the barges. The units are moved around the yard by gantries or lifting frames and are lifted or slid on to the jetty, where the barges collect them. Figure 14.12 shows a 2200 tonne unit for the Central Viaduct, part of the Vasco da Gama Crossing, being positioned by the floating crane and barge arrangement.

Erection of the units

Erection of the units requires special equipment to lift the heavy loads and to carefully position the beam. The large number of units being erected often justifies the cost of manufacturing special erection equipment for a particular project, with the arrangement chosen to suit the individual project requirements. However, over the years there has built up around the world a number of existing gantries, barges and other heavy-lifting equipment specially developed for bridges constructed with large precast units.

For viaducts over land, the units are usually transported along the completed deck and moved into position using a gantry. After placing the unit on its bearings, the gantry launches itself forward over the next span to repeat the process. Figure 14.13 shows a precast unit on the Taiwan

@Seismicisolation

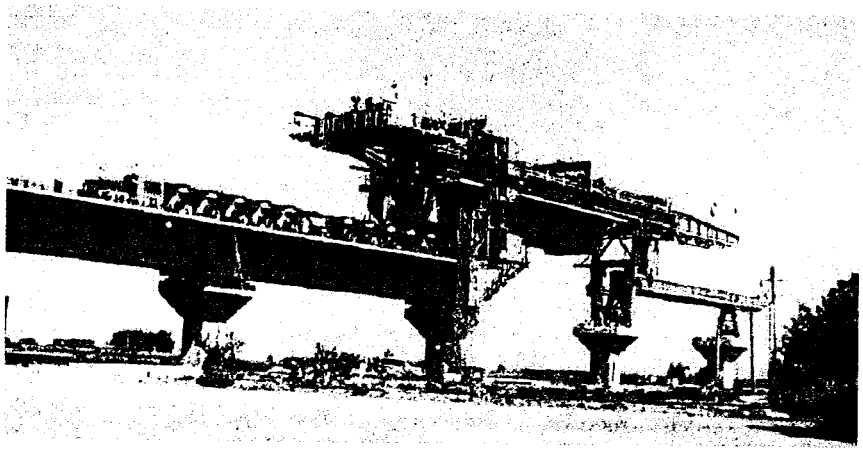


Figure 14.13. Placing beams by gantry. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

High-Speed Rail project being lifted into place by an overhead gantry spanning between the completed deck and the next pier.

Where the bridge is over water and the units are transported by barge they are placed directly on to bearings on the piers. For the Central Viaduct, in Fig. 14.12, the 77·6 m long units spanning between the piers were erected as simply-supported beams and subsequently connected with an *in situ* concrete stitch to make the deck continuous.

On the Saudi Arabia–Bahrain Causeway, in Fig. 14.1, the deck for the 50 m spans was assembled from two different lengths of precast unit. The longer 66 m units were first placed in every other span. The units rested on the bearings at the piers with short cantilevers extending into the adjacent spans. Shorter 34 m long units were positioned to fill the gap, with a half-joint arrangement between the two.

The 250 m long spans on the Confederation Bridge in Fig. 14.14 were constructed using 192 m long precast units placed as a 'balanced cantilever' about each pier with the remaining 58 m gap filled with a shorter precast

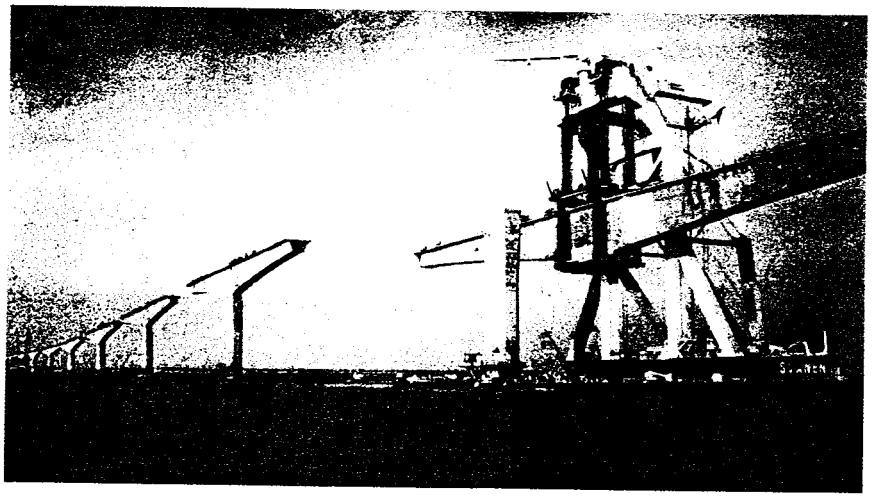


Figure 14.14. Cantilevered deck being lifted into place. Reproduced courtesy of Ballast Nedam BV. Copyright reserved.

@Seismicisolation

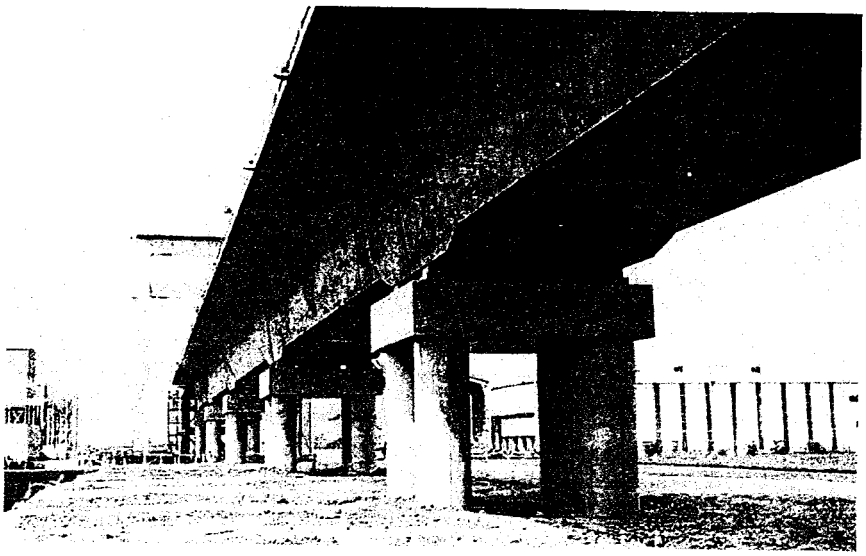


Figure 14.15. Deck on bearings. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

unit dropped into place. The units placed as a balanced cantilever were supported at the piers to provide adequate stability in the temporary condition.

Where the units span between piers and are arranged as simply supported beams they are usually supported by two bearings at each end, with a typical arrangement shown in Fig. 14.15. For spans less than 50 m the small deck movements and loads generated allow either mechanical or rubber bearings to be used. For spans longer than 50 m the higher loads and greater deck movements usually require mechanical bearings. With rubber bearings, the precast units are placed directly on to them without any bedding mortar

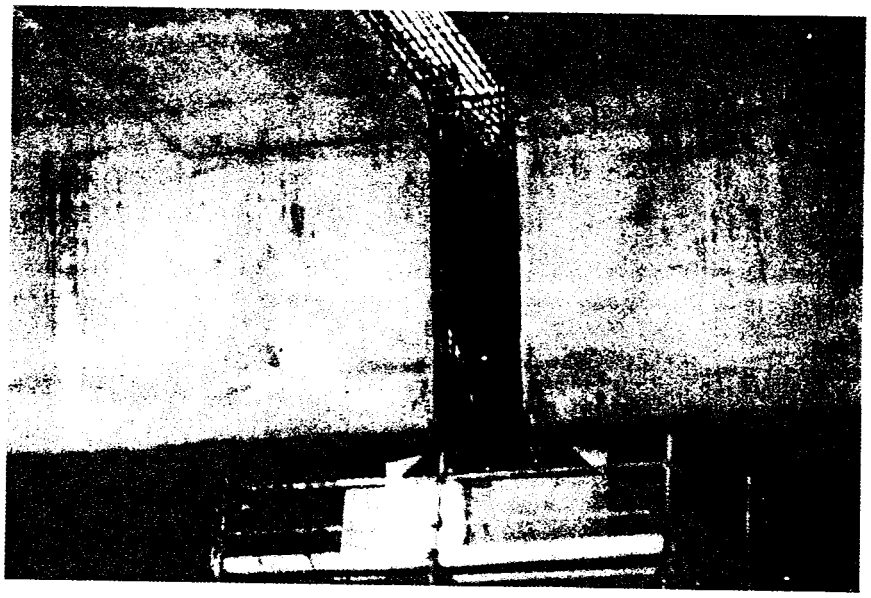


Figure 14.16. Stitch for deck continuity. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

needed. If mechanical bearings are used the units are held above the top plate on temporary supports while the gap is filled with grout.

For decks made continuous at the piers the units are initially placed on temporary supports. After erecting the units in adjacent spans the gap between their ends is filled with *in situ* concrete with the permanent bearing installed beneath. Figure 14.16 shows two units in place at a pier with the reinforcement being fixed before placing the formwork and concreting the stitch. After the concrete has reached sufficient strength, permanent prestressing tendons are installed across the connection to complete the deck.

A rapid erection rate is achieved when using full-length precast units because of the minimal work needed on site. For smaller units and simpler arrangements, an erection rate of one unit per day is often achieved, although the very large units may take longer than this.

Design of full-length precast box girder decks

The design of full-length precast box girders is similar to the *in situ* box girders, covered in Chapter 12. The main difference being the need to check the forces and stresses during the transportation and lifting of the units.

The design of the units in the temporary condition is checked to ensure that sufficient reinforcement and prestress are present at each stage to cater for the forces and stresses set up. The weight from any construction equipment and other temporary loads is applied in the temporary condition as with the other types of construction. The stability of the units is checked during storage, transportation and erection, and temporary supports are provided as necessary.

As the units are moved, lifted and placed into position they are subjected to dynamic loading from the handling process. The magnitude of the dynamic loading depends on the equipment and procedures adopted, and every effort should be made to minimize any potential impact loading on the precast concrete. In normal circumstances it is sufficient to check the deck in the temporary condition when being moved for a 20 per cent increase in the dead load to cater for the dynamic effects.

@Seismicisolation

15

Incrementally launched box girder bridges

Introduction

Where the bridge alignment is straight or on a constant radius curve incrementally launching a concrete deck overcomes access problems or avoids obstructions at ground level. Incrementally launched decks usually have a box girder section, although beam-and-slab decks have also been successfully launched using the same techniques.

Many major bridges have utilized this technique for their construction, such as the viaduct for the Dornoch Firth Viaduct in Fig. 15.1 where the deck was launched into position with spans of 43·5 m and an overall length of 890 m.

The Hasdel Viaduct, in Fig. 15.2, was a contractor's alternative design developed to simplify the construction over the deep valley. The 325 m

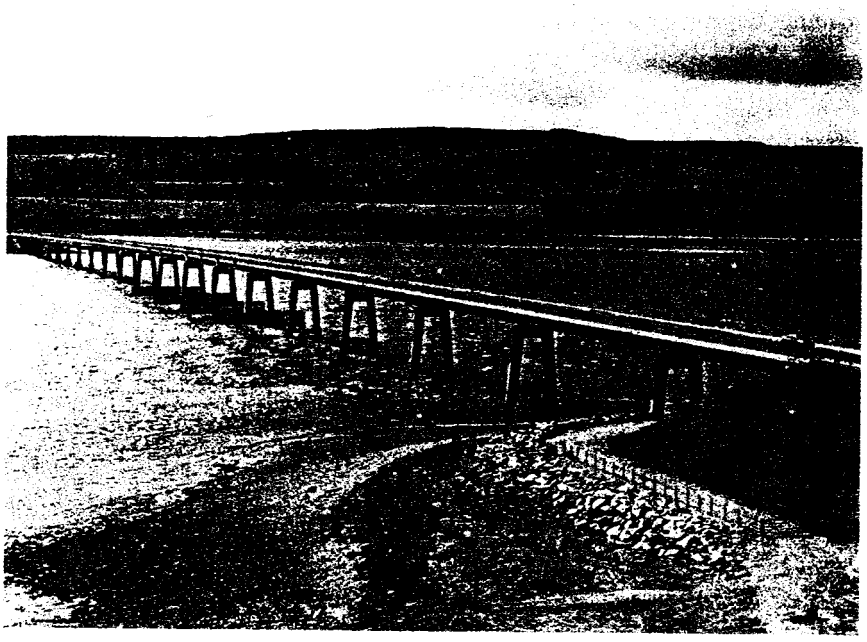


Figure 15.1. Dornoch Firth Viaduct, Scotland. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

Figure 15.2. Hasdel Viaduct, Turkey.

long deck with spans up to 58 m was pushed out from behind the abutment in stages after each 29 m long section was cast. Using the incremental launch technique reduced the temporary works needed for the deck construction and avoided any associated works in the valley.

The technique involves casting the deck in sections behind one of the abutments. After each section is cast the deck is pushed or pulled out over the piers, as seen in Fig. 15.3. The first segment is cast and moved forward

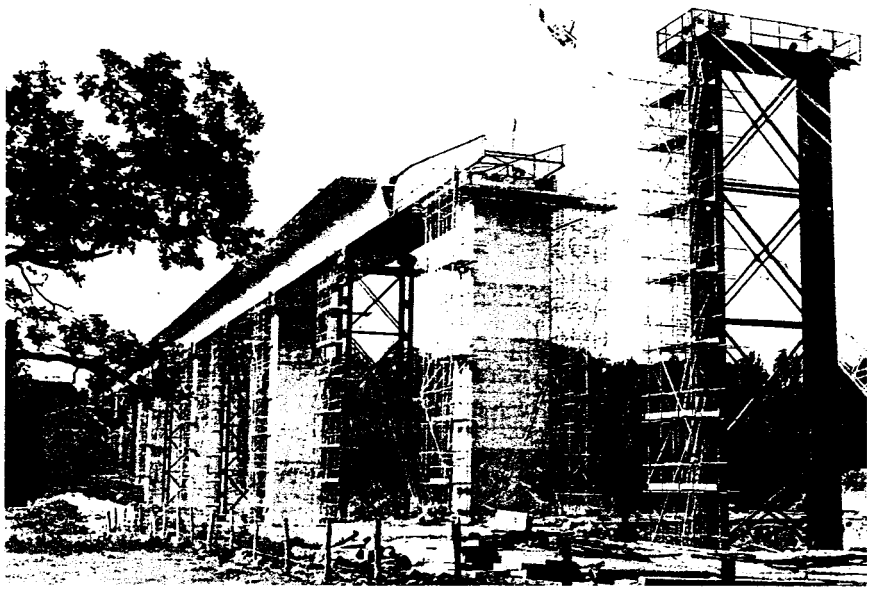


Figure 15.3. Deck during launch over piers. Reproduced courtesy of Freyssinet International. Copyright reserved.

@Seismicisolation

on temporary bearings. The second segment is then cast against the first and both are moved forward by a further increment with subsequent segments cast and the deck moved until it reaches the opposite abutment and its final position.

Generally used for spans less than 60 m, the technique often utilizes temporary supports which are placed to reduce the effective span length during launching. It is possible to launch longer spans, but the construction becomes less economic as the spans increase.

The advantages of incrementally launching a deck include the need for only limited temporary works and the minimal disruption to the ground below. The disadvantages include higher concrete and prestress quantities in the deck and the relatively slow construction rate.

General arrangement

The deck arrangement is similar to the *in situ* concrete box girders described in Chapter 12. The main difference is the requirement to provide sufficient strength and bearing capacity at the bottom of each web for the temporary bearings supporting the deck during the launch. This results in the area at the junction between the web and bottom slab being thickened up, as illustrated in Fig. 15.4.

The outer face of the bottom of the webs is often made vertical to simplify the guidance arrangement during the launch; however, it is possible to guide the deck with the webs sloping.

Referring to the deck in its final position, the width of the web over the mid-span region is governed by the shear design requirements during the deck launching. As each section passes over the temporary bearings it is subjected to significant shear forces. The width of the webs near the supports is governed by the shear requirements in the permanent condition. The webs

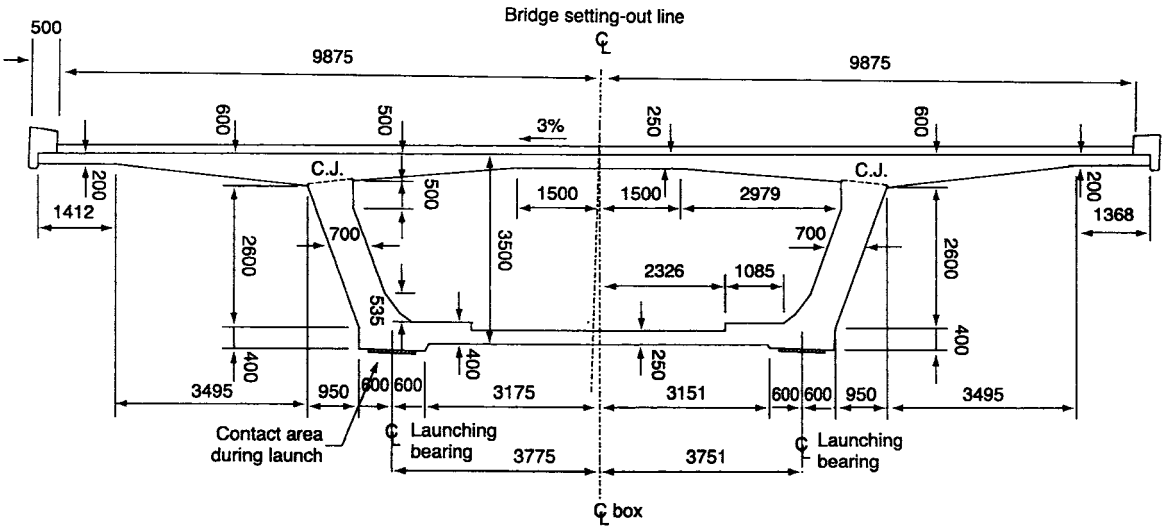


Figure 15.4. Typical launched deck section. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

are often kept a constant thickness throughout the length of the deck to simplify the construction and cater for the shear stresses during the launch.

Box girder decks usually have a single-cell arrangement with the box supported by a temporary bearing under each web during the launch.

Using a twin- or multi-cell box with several webs causes difficulties with arranging the temporary bearings at each support. If two bearings are used, one under each outer web, the box structure may not be strong enough to transfer the load in the other webs out to the bearings during the launch stages. Using more than two bearings at each support makes it difficult to control the load distribution between them due to the construction tolerances. This may cause the bearings to be overloaded or the deck to be overstressed.

A temporary nosing arrangement, as described in 'Launching the deck' later in this chapter, is connected to the front of the deck during the launch. The launching nose is fixed against a diaphragm or wall across the end of the deck. It is not necessary to provide other diaphragms in the box during the launch. The permanent diaphragms may be cast after the deck launch is completed.

The launching requirements for a beam-and-slab deck dictate an arrangement for two main longitudinal girders, with the deck slab spanning between or supported on secondary transverse beams. The main longitudinal girders are used for the launch with the temporary bearings positioned beneath.

To allow a deck to be launched its soffit surface must have a uniform profile to slide on. Decks may be launched with significant gradients and crossfalls, and with tight horizontal curvature as long as these provide a constant launching geometry. When the alignment has both horizontal and vertical curvatures, these are arranged to give a constant 'arc' for the deck soffit to launch along.

The range of moments to be taken by each section requires a deeper deck than that used with other forms of structure for the same span length. The ratio of launched span to deck depth is usually 16 : 1 or less.

Segment lengths are standardized for an individual bridge deck and are usually between 15 m and 30 m long depending on the span arrangement and overall deck length. Longer segments reduce the total number required, giving a shorter construction period. Shorter segments require a smaller casting area with less formwork and labour needed.

The deck is usually continuous between abutments; however, it is possible to include expansion joints along the length if required. Temporary prestress is used to connect the deck across the expansion joint and make it continuous for launching. After the launch is completed the temporary prestress is removed, and the deck is separated to form the expansion joint.

The longitudinal prestress in the deck is divided into two groups: the launch prestress, installed as the deck is being built, and the supplementary prestress installed after the launch is completed. The launch prestress usually consists of straight tendons placed in the top and bottom slabs. These are anchored on the construction joint between segments and coupled or lapped to extend through each new length of deck cast. The supplementary prestress consists of additional tendons in the top and

@Seismicisolation

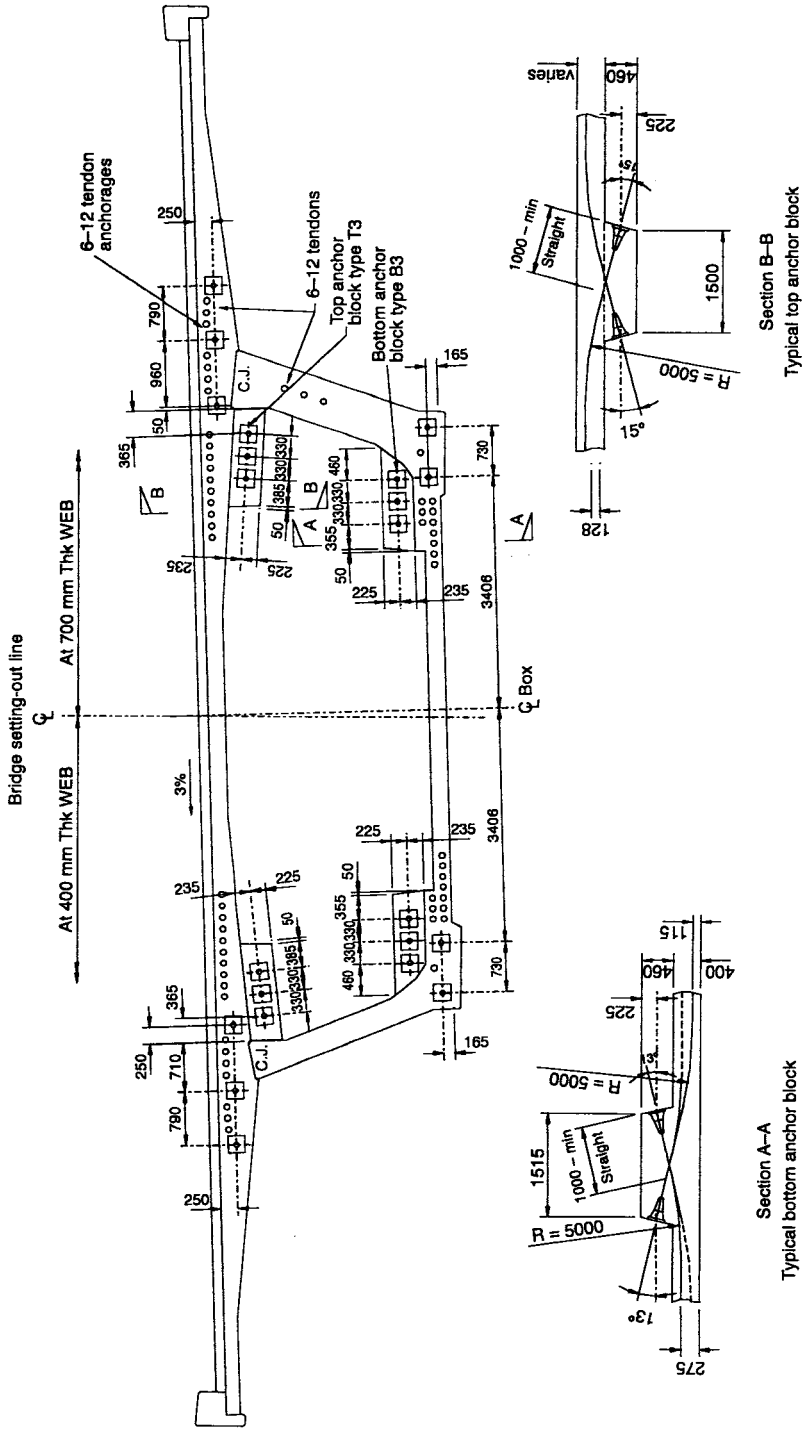


Figure 15.5. Tendon anchorage details. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

bottom slabs and draped tendons running through several spans anchored at blisters within the box. A typical arrangement for the end face anchors on the construction joint and the blisters inside the box is illustrated in Fig. 15.5.

Casting the deck

A specially prepared casting yard is first located behind the abutment, as seen in Fig. 15.6. The yard is arranged with separate areas to assemble the reinforcement, to concrete, and to launch the deck. The formwork is placed along the line of the deck and set back behind the abutment a distance sufficient to provide stability to the deck in the initial stages of the launch.

The launching nose is assembled in position at the front of the casting cell at the beginning of the deck construction process. The nose is placed on temporary supports, as illustrated in Fig. 15.7, with the first deck section cast up against it. The number and location of the temporary supports depends on the position of the casting cell behind the abutment, the length and design of the segments and the stability of the deck.

There are several different sequences for assembling the reinforcement, casting the deck and launching. The reinforcement is either pre-assembled and lifted into position or fixed behind the cast deck and pulled into position when the deck is launched forward. The top slab to the deck is often cast as a separate operation after the bottom slab and webs to simplify the internal formwork. Four of the most common sequences are outlined below:

(1) Casting bottom slab first:
   (a) Bottom slab reinforcement and tendon ducts assembled.
   (b) Bottom slab cast.
   (c) Install first-stage launch prestress.

Figure 15.6. Casting area behind abutment.

@Seismicisolation

(d) Deck launched forward with bottom slab moved into next section of casting cell.
(e) Web and top slab reinforcement and tendon ducts fixed.
(f) Web and top slab concrete cast at the same time as the next section of bottom slab in the bay behind.
(g) Install second-stage launch prestress.
(h) Deck launched forward and next sections of webs, top slab and bottom slab constructed.

(2) Casting bottom slab and webs first:
(a) Bottom slab and web reinforcement, and tendon ducts assembled.
(b) Bottom slab and webs cast.
(c) Install first-stage launch prestress, if needed.

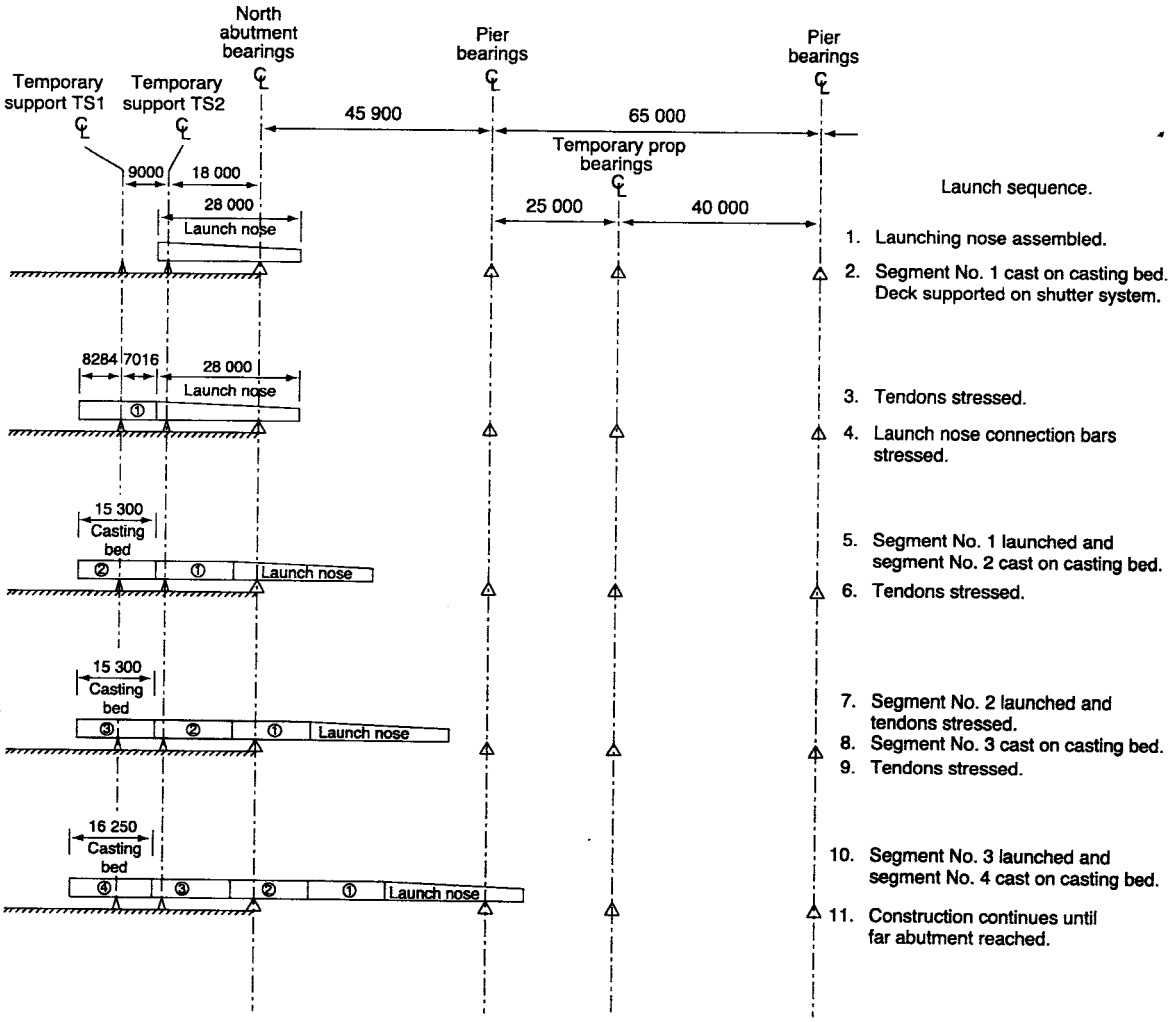


Figure 15.7. Typical casting and launching sequence for deck. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

(d) Deck launched forward with bottom slab and webs moved into next section of casting cell.
(e) Top slab reinforcement and tendon ducts fixed.
(f) Top slab concrete cast at the same time as the next section of bottom slab and webs in the bay behind.
(g) Install second-stage launch prestress.
(h) Deck launched forward and next sections of bottom slab, webs and top slab constructed.

(3) Casting complete section in stages:
(a) Bottom slab and web reinforcement, and tendon ducts assembled in casting bay.
(b) Bottom slab and webs cast over full segment length.
(c) Shutters positioned for top slab.
(d) Top slab reinforcement and tendon ducts fixed.
(e) Top slab concrete cast over full segment length.
(f) Install launch prestress.
(g) Deck launched forward and next section constructed.

(4) Casting complete section in single pour:
(a) Reinforcement and tendon ducts for complete section assembled in casting bay.
(b) Shutters positioned, including internal forms to inner web and the soffit of the top slab.
(c) Complete deck section cast over full segment length.
(d) Install launch prestress.
(e) Deck launched forward and next section constructed.

A typical shutter arrangement for sequence (1) above is illustrated in Fig. 15.8. After the bottom slab concrete has hardened it is able to support the internal form for the webs and top slab which greatly simplifies the shutter design. The outer formwork system is designed to be lowered, leaving the deck on temporary supports ready for launching.

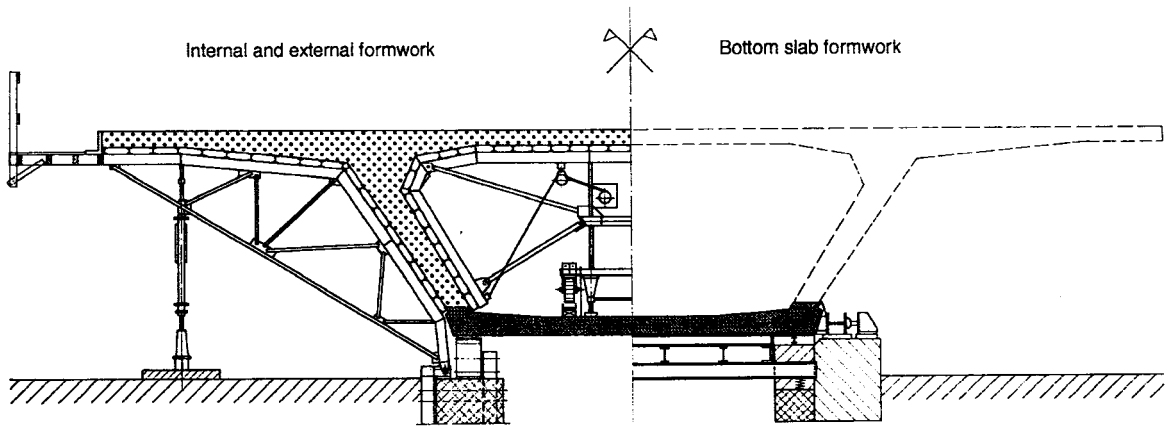


Figure 15.8. Formwork arrangement. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

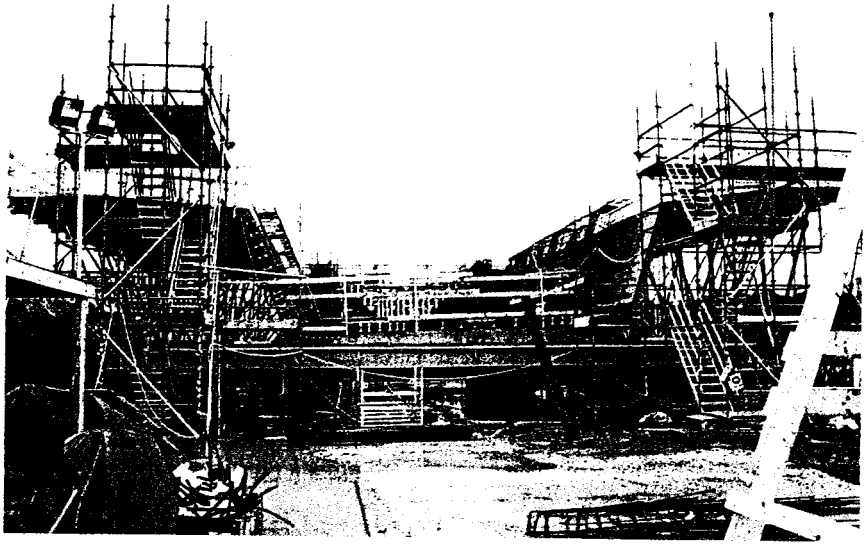


Figure 15.9. Outer formwork assembled.

A typical casting area for sequence (3) above is shown in Fig. 15.9, with the outer formwork in place and the reinforcement being fixed before casting the bottom slab and webs. In this example the formwork is supported on steel beams with jacks underneath to allow the entire outer shutter to be lowered in one piece. An alternative arrangement, often used, is to provide temporary concrete walls under each of the webs. These support the soffit shutter and provide a runway for the deck to slide along during the launch.

After setting up the outer formwork and fixing the reinforcement the inner form for the webs is installed, as seen in Fig. 15.10. Where form ties are permitted through the web the shutters consist of standard panels placed

Figure 15.10. Casting cell looking towards deck.

@Seismicisolation

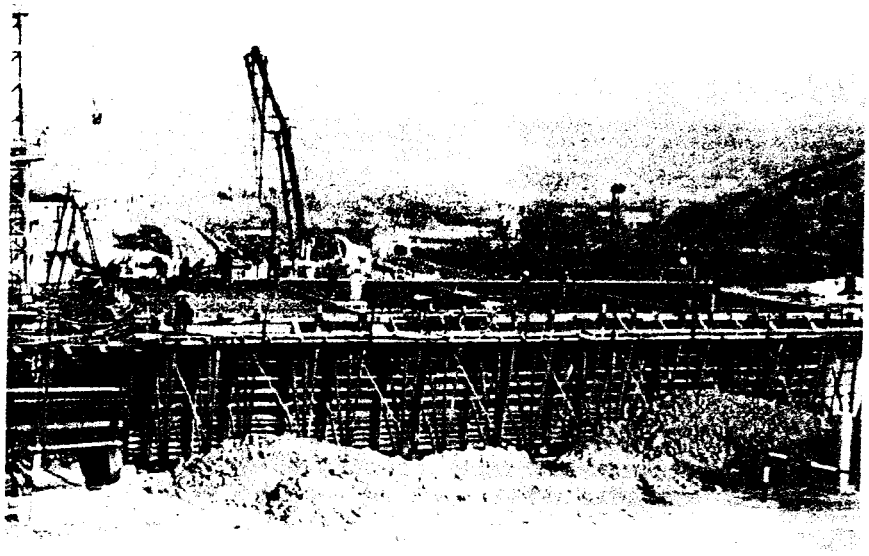


Figure 15.11. Concreting the deck slab.

and supported in the traditional way. Where form ties are not permitted the inner form is supported using a frame to hold it in place and to resist the concrete pressures.

With the top slab cast after the bottom slabs and webs, a table-form is used to support the fresh concrete between the webs. The table-form is placed after the bottom slab concrete has hardened. A typical table-form arrangement, still inside the previously cast section of deck, is seen in the centre of Fig. 15.10. After casting the top slab the table-form is usually left in the deck during launching. It is then moved into position when the next section of top slab is ready for construction.

Casting of the deck is done using conventional techniques with the concrete skipped or pumped into position, as seen in Fig. 15.11. Early high-strength concrete is used to allow the shutters to be stripped and the prestress to be installed as soon as possible after concreting. The deck is then launched forward freeing up the casting bed for the construction of the next segment to start.

By separating the reinforcement fixing and concreting into different areas of the casting cell, as for sequences (1) and (2) above, these operations are progressed at the same time to minimize the construction cycle time. Similarly, prefabricating the reinforcement and lifting the cages into the shutters reduces the cycle time. A casting and launching cycle typically takes between 7 and 10 days.

Launching the deck

As described above, the deck is cast in sections and incrementally launched into position with a typical sequence, illustrated in Fig. 15.7.

As the deck slides out it is supported at the piers, abutments and other temporary support positions by temporary bearings, as seen in the foreground

@Seismicisolation

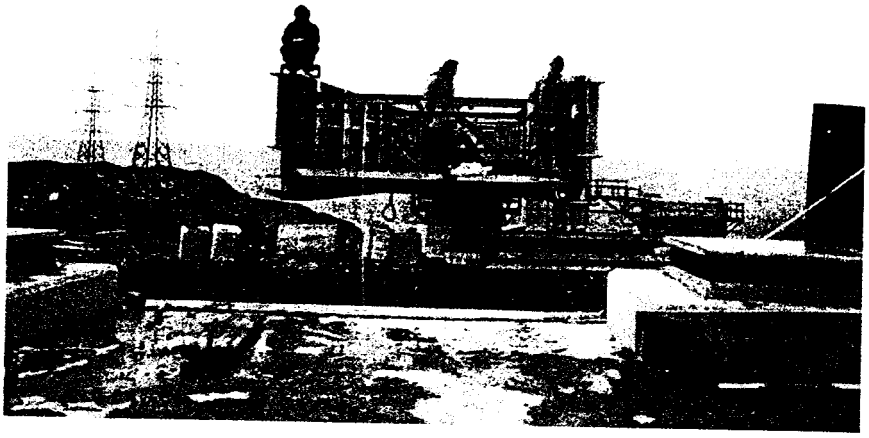


Figure 15.12. Temporary bearings for launch.

of Fig. 15.12. These temporary bearings support the deck under the soffit as it passes above. They consist of a steel plate, surfaced with a stainless steel sheet, supported on a mechanical or rubber bearing. A teflon pad is fed in between the bearing's stainless steel sheet and the concrete soffit as the deck slides over to provide a low-friction interface. When the launch is complete the deck is raised on temporary jacks to allow the temporary bearings to be removed and the permanent bearings installed.

A combined permanent and launch bearing is shown in Fig. 15.13. These bearings are set in their permanent position with the bottom plate placed horizontally and the normal slide mechanism locked into position. The bearing's top plate is tapered to match the longitudinal gradient of the deck and a stainless steel sheet fixed over the top. During the launch a teflon pad is fed in between the stainless steel sheet and the concrete deck in the same way as with temporary bearings. After completing the launch the deck is jacked up and the stainless steel sheet and teflon pads removed. The gap between the deck and bearing is then filled with grout.

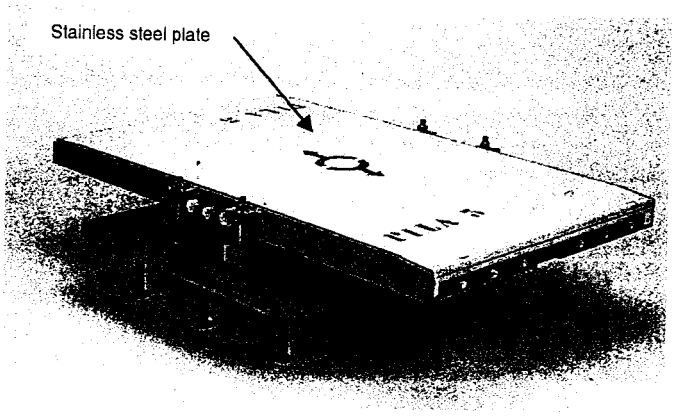


Figure 15.13. Combined permanent and launch bearings. Reproduced courtesy of VSL/CTT-Stronghold. Copyright reserved.

@Seismicisolation

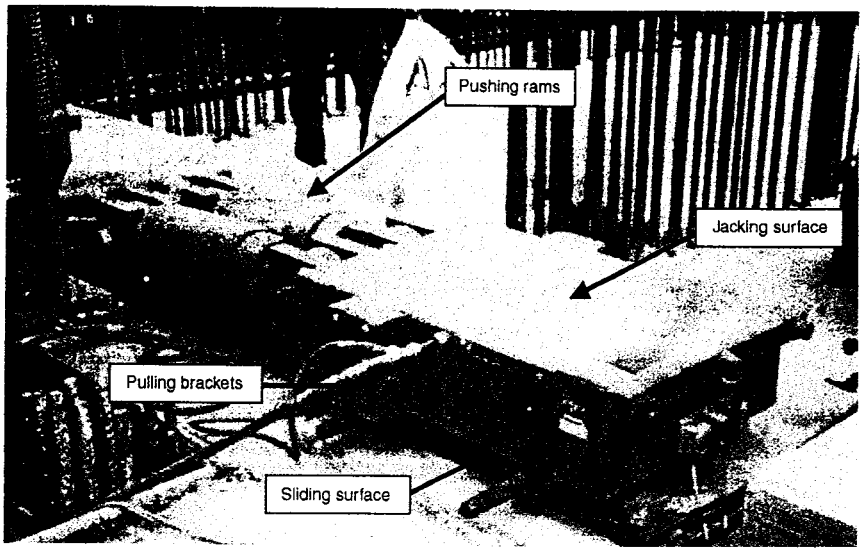


Figure 15.14. Push launching jack.

The deck is launched either by pushing or pulling out over the spans. The launching device is usually fixed to the abutment which provides the resistance against the horizontal thrust needed to move the deck. A typical launch rate is between 4 m and 6 m an hour.

Push launching jacks, as seen in Fig. 15.14, are positioned on the abutment in front of the bearing positions and under each web. The launching jacks raise the deck a small amount to generate sufficient friction to grip the structure. Then they push the deck forward a short distance before lowering it on to the temporary bearings. After releasing the structure the launching jacks move back ready to start another stroke.

The interface between the launching jack and the concrete deck is usually able to generate a frictional force of at least 40 per cent to 60 per cent of the vertical load carried. Launching jacks rely on being able to generate sufficient friction with the deck to generate the launching force needed. This is not always possible at the beginning of a launch when the vertical reaction on the jack is small. The launching nose may be sitting on the launching jack with very little load present. To overcome this, most launching jacks include a pulling bracket through which a bar is attached to the front of the deck and the jack used to pull the deck out until there is sufficient weight to generate the required friction.

Similarly, insufficient friction may be available when launching long lengths of deck. This becomes critical towards the end of the launch when the back of the deck approaches the launching jack reducing the vertical load. If this occurs, it may be more appropriate to use a pulling device throughout the launch.

An arrangement for pulling the deck into position by the use of strand jacks is illustrated in Fig. 15.15. The jacks are located at the abutment which provides the anchor point and resistance against the force generated. The strands are fixed to a frame located at the back of the deck section just cast

@Seismicisolation

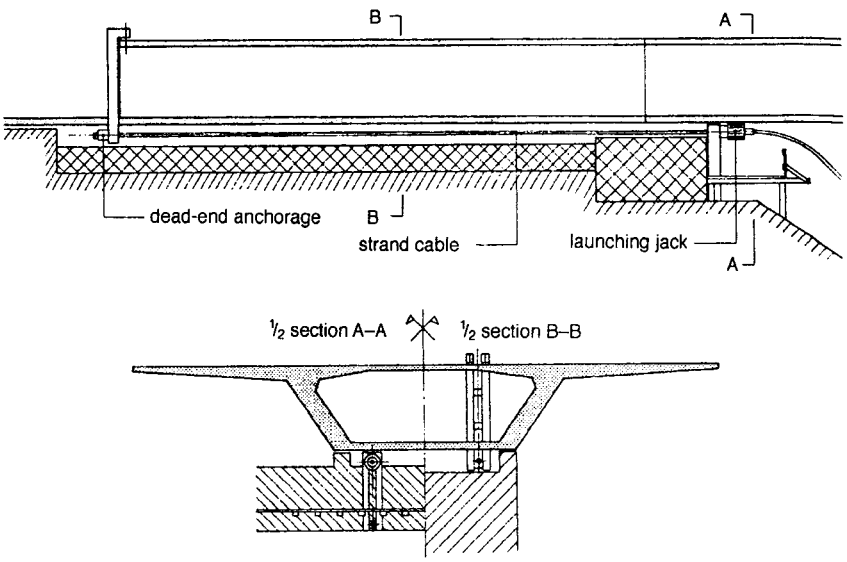


Figure 15.15. Pulling arrangement. Reproduced courtesy of VSL International. Copyright reserved.

and are pulled forward by the jacks. After each section is launched the frame is removed and fixed to the back of the next section.

Lateral guides, as seen in Fig. 15.16, are fixed to the abutment and piers to keep the deck correctly aligned during the launching sequence. The guides prevent the deck from moving sideways and resist any transverse loads that may be imposed on the deck. It is not necessary to provide lateral guides at every pier or temporary support, although there must always be a sufficient

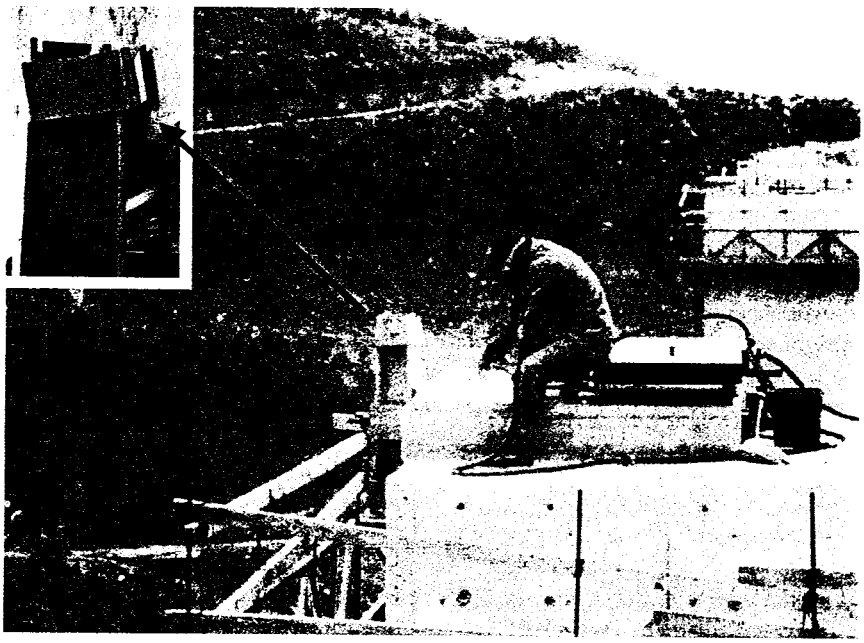


Figure 15.16. Guides fixed to piers and bearing plinths being prepared.

@Seismicisolation

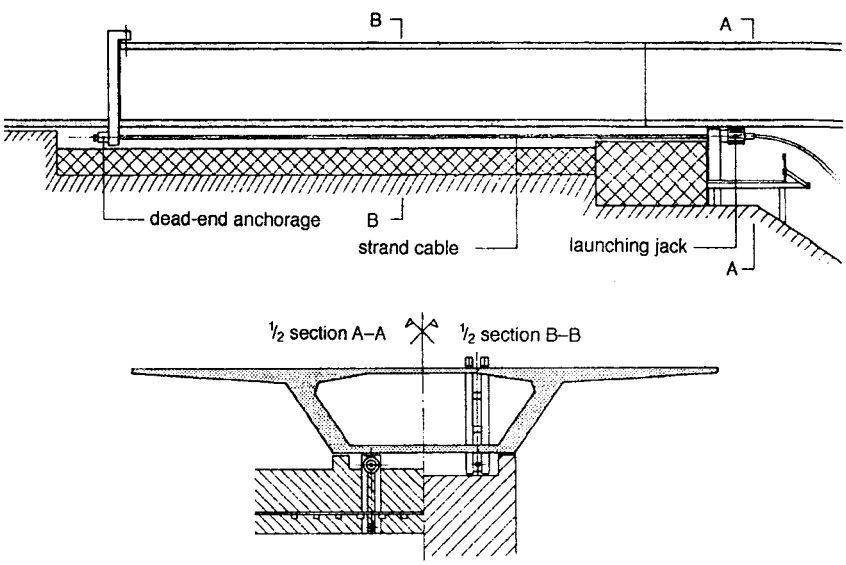


Figure 15.15. Pulling arrangement. Reproduced courtesy of VSL International. Copyright reserved.

and are pulled forward by the jacks. After each section is launched the frame is removed and fixed to the back of the next section.

Lateral guides, as seen in Fig. 15.16, are fixed to the abutment and piers to keep the deck correctly aligned during the launching sequence. The guides prevent the deck from moving sideways and resist any transverse loads that may be imposed on the deck. It is not necessary to provide lateral guides at every pier or temporary support, although there must always be a sufficient

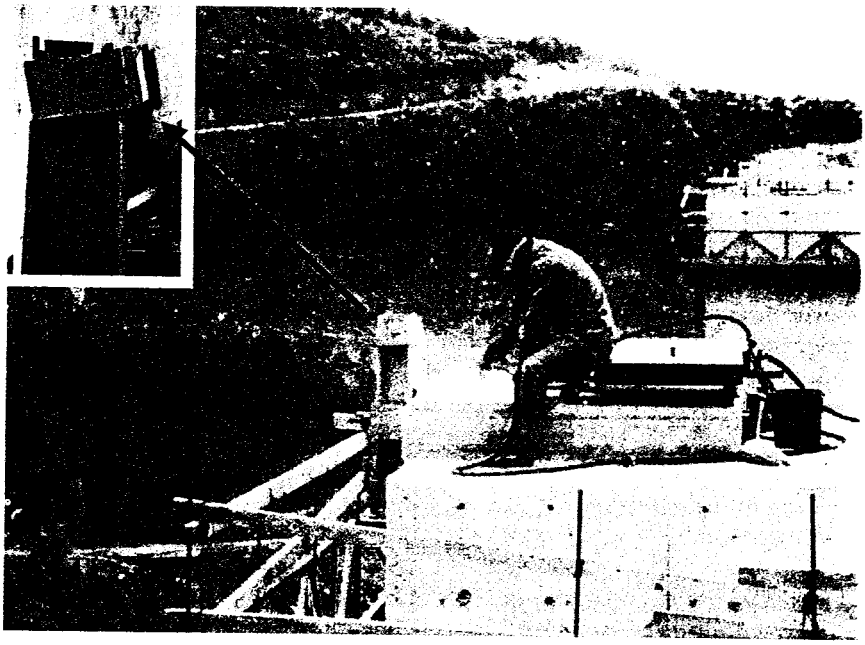


Figure 15.16. Guides fixed to piers and bearing plinths being prepared.

@Seismicisolation

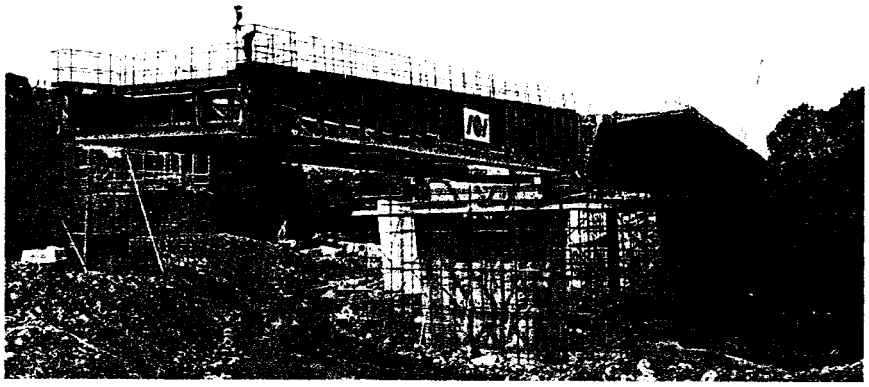


Figure 15.17. Launching nose on deck.

number to adequately restrain the deck. The lateral guides are removed after the launch of the deck is completed and the permanent bearings installed.

As the deck is pushed out and extends beyond the leading pier, large cantilever moments occur until the next pier is reached. To reduce these moments a temporary steel launching nose is fixed to the front of the deck, as shown in Fig. 15.17. The launching nose allows the deck to be supported by the next pier at an earlier stage in its launching cycle and reduces the bending moments and shears that occur over the front part of the deck.

The effectiveness of the launching nose is a function of its length and stiffness, and the optimum arrangement needs to be chosen to balance the cost of the nose against the cost of catering for higher moments and forces in the deck. The launching nose length is typically 60 per cent of the launch span length, and usually has a stiffness (EI) of between 10 per cent and 15 per cent of the concrete deck.

Steel girders or trusses are used for the launching nose. A typical arrangement using twin girders separated with cross bracing is seen in Fig. 15.12. The nose may be of constant cross-section throughout to simplify its fabrication, but is usually tapered, reducing in depth at the front end to minimize weight and save cost.

An alternative to using a launching nose is to utilize stay cables to reduce the bending moments and shears. The stays are supported on a temporary tower located back from the front of the deck. The force in the stays is adjusted to control the moments and forces imposed on the deck as it moves forward. This technique has been used on a number of projects in the past, but it is generally not favoured due to the complexity involved in controlling the temporary stay forces during the launch.

The forces in the temporary supports and the line and level of the deck are regularly monitored during the launching of the deck to ensure that the deck is behaving as expected. The pushing force and verticality of the piers and temporary supports are also monitored with any change from the expected situation being an indication of problems with the launch.

@Seismicisolation

Design aspects associated with launched box girder decks

The design of launched decks is similar to *in situ* decks when they are in their final position, but during the launching process they are subjected to a different force and stress regime that has to be catered for. The design of the deck during the launch is described in the following section. The paper by Rowley (1993) [1] and the book by Rosignoli (2002) [2] discuss the different aspects of launching in more detail.

Longitudinal design during launch

The deck is checked at each stage during its launch, and prestress and reinforcement provided to suit. The longitudinal analysis is carried out using a line beam model which includes the launch nose structure. The model changes as each section of deck is cast and as the deck is launched out over the supports.

A typical moment distribution along the deck, just before the deck reaches a pier and just after, is illustrated in Fig. 15.18. As the deck moves over the piers the moments and shears at each section change.

At some stage during the launch each section will be over a pier or temporary support and subjected to hogging moments or in a mid-span position and subjected to sagging moments. A typical envelope of moments at each deck section during the launch phase of a three-span bridge is given in Fig. 15.19. The launch prestress is designed to cater for

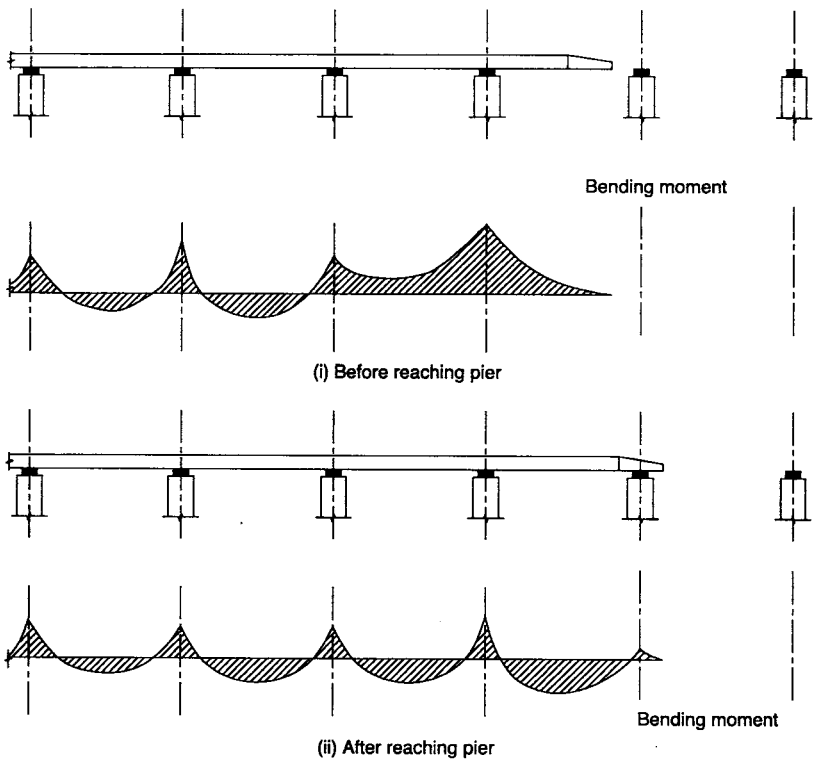


Figure 15.18. Bending moments in deck during launching.

@Seismicisolation

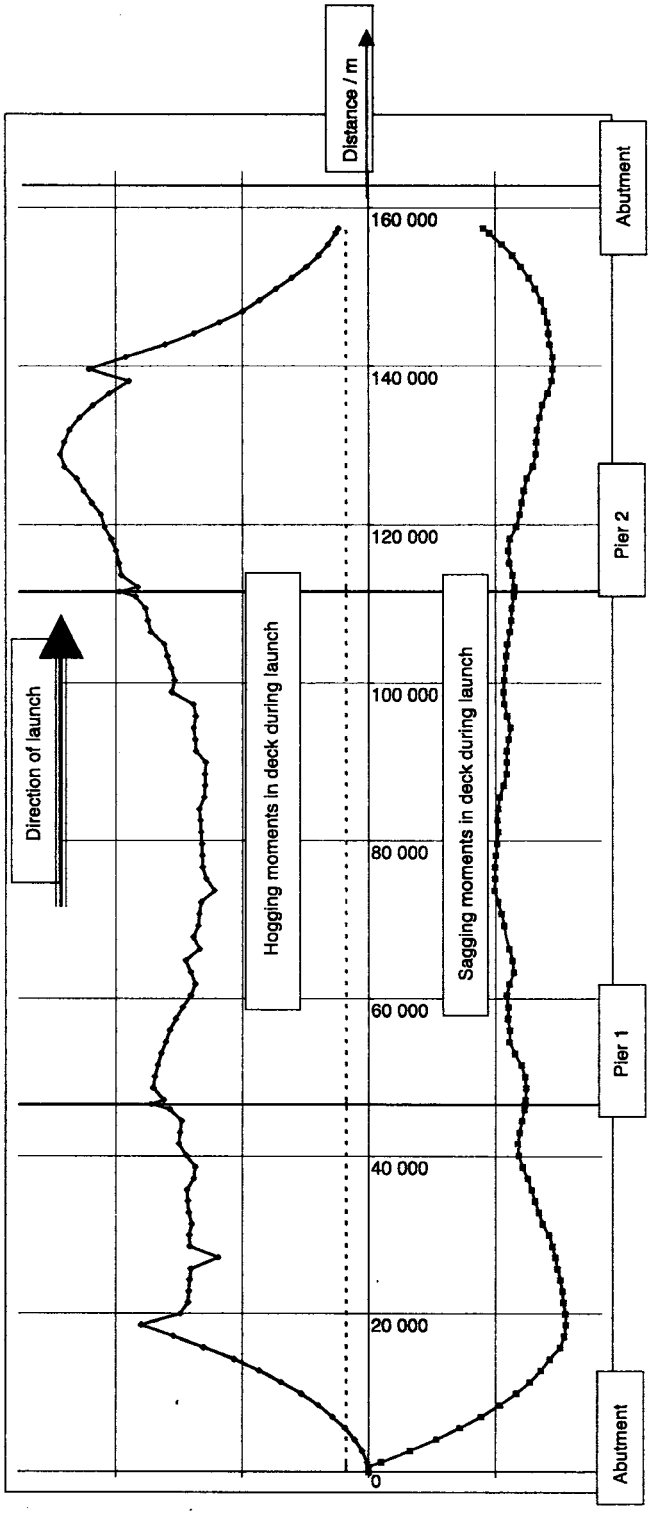


Figure 15.19. Bending moment range in deck during launching. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

the range of bending stresses at each section. The largest hogging moments occur near the front of the deck, just before the launching nose reaches the next pier. The largest sagging moments also occur near the front after the launching nose has reached the pier but just before the front of the deck reaches it. Elsewhere the range of moments is more uniform, reducing at the back for the last section to be cast.

The launch prestress is usually arranged to give a uniform compressive stress across the section, typically about $5\,N/mm^2$. Additional prestress is provided at the front of the deck to cater for the higher moments generated. After completion of the launch the supplementary prestress is installed to balance the bending moment profile generated by the deck in its final position.

Transverse and local design during launch

Each section of deck has to be strong enough to resist the loads from the temporary bearings as it passes over them.

It is preferable for the temporary bearings during the launch to be located directly under the centre of the webs. If this is not possible, bending moments and shear forces are induced in the bottom slab and the bottom of the web in both the longitudinal and transverse directions. The structural behaviour in this region is analysed locally using a three-dimensional finite element model, as illustrated in Fig. 15.20. The stresses derived from the analysis are used to determine the tensile and shear forces to be catered for by the reinforcement and concrete.

The analysis and design of the deck above the temporary bearings must allow for the increase in load and adjustment in support position due to the construction tolerances discussed below.

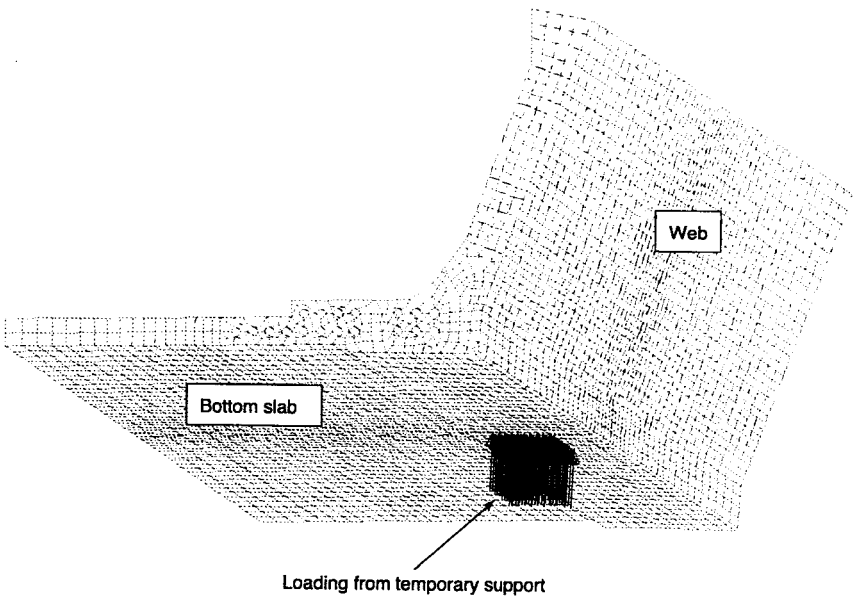


Figure 15.20. Analysis of bottom slab and web during launch. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

Construction tolerances

Unevenness of the concrete surface, misalignments of the bearings and differential settlement of the piers and temporary supports generate additional moments, shears and torsion in the deck during the launching operation.

Typical values assumed in the design are:

Soffit concrete level = ±3 mm from design level
Level of temporary bearings = ±2 mm from design level
Differential settlement between adjacent supports = ±5 mm

The differential settlement depends on the soil properties and the type of foundation used, while tight construction controls are needed to achieve the tolerances for the concrete and bearing level.

The above tolerances may combine to give a total displacement of ±10 mm on the deck soffit, causing differential effects longitudinally between adjacent piers and transversely between the bearings. The change in levels redistributes the loads in the temporary bearings causing bending, shear and torsional effects in the deck.

Construction tolerances for the lateral position of the deck and temporary bearings produce a misalignment between the two compared to their theoretical positions. This increases if there is any lateral movement of either the deck or the piers during the launch. To allow for this in the design the position of the temporary bearings under the deck is assumed to vary by up to 75 mm transversely. This allowance may be reduced if suitable measures are taken to limit the misalignment that can occur.

When setting the bearings after completing the launching the deck is jacked up and the bearings installed with a specified load built in. In this way the construction tolerances are compensated for in the final structure.

Construction loading

As the deck is being launched there is usually very little temporary load imposed on the deck, while the launching nose weight is already included in the structural analysis model.

If storage of material or heavy equipment is not permitted on the deck the construction loading allowed for in the design is typically taken as:

(a) In front of abutment bearing:

Point load = 10 kN anywhere on the deck for miscellaneous equipment
General working load = 2 kN/m of deck

(b) Behind abutment bearing and in casting area:

Point load = 20 kN anywhere on the deck slab for miscellaneous equipment
General working load = 4 kN/m run
Weight of formwork for deck slab between webs = 1 kN/m^2 of deck

Loads on supports during launching

During the deck launch the friction in the temporary bearings creates a load at the top of the piers and temporary supports, as illustrated in Fig. 15.21(i).

@Seismicisolation

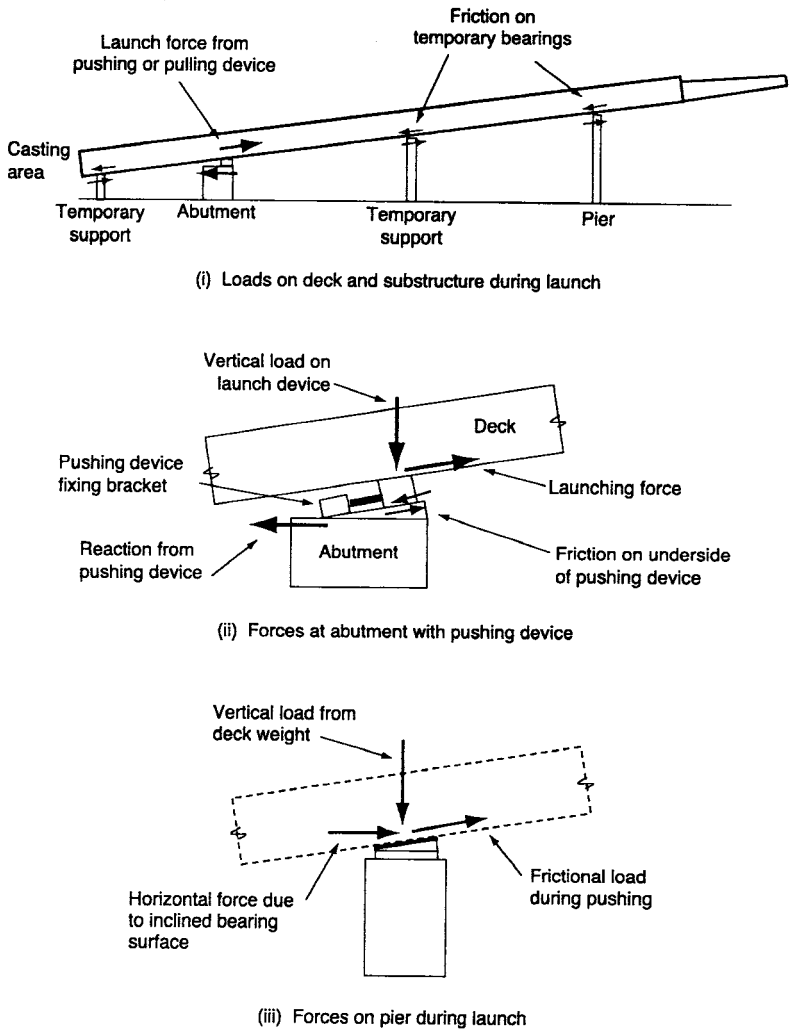


Figure 15.21. Forces on substructure during launching.

When greased slide pads are used the friction is typically between 2 per cent and 6 per cent of the vertical load.

The top surface of the temporary bearings is aligned parallel to the deck rather than horizontal and this introduces an additional horizontal load on to the support, as illustrated in Fig. 15.21(iii). This force is equal to the vertical load times the % gradient.

The piers and temporary supports are designed to resist these horizontal loads in combination with the co-existent vertical loads. Providing stays or guys from the top of the support to an adjacent pier stiffens up the support and reduces the effects of the loads.

As the deck moves out and gets longer the launching force increases to overcome the increased frictional force on the temporary bearing. Greater pushing forces are required when a deck is being launched up a slope, while a braking device is needed when going down a slope.

@Seismicisolation

For the pushing arrangement, shown in Fig. 15.21(ii), the upper surface in contact with the deck is subjected to the launching force generated from the friction resistance at all the other supports. The lower surface of the pushing device runs over a stainless steel sheet and generates a frictional resistance as the device moves forward, proportional to the load on the pushing arrangement. The force needed to launch the deck is the sum of the frictional resistance at all the temporary supports and the underside of the pushing device.

The abutment is designed to resist the horizontal forces generated during the launch and to prevent sliding or overturning. Additional resistance is mobilized by providing the casting area with a ground slab as a working platform and by connecting this slab to the abutment.

References

1. Rowley, F. (1993) 'Incremental launch bridges: UK practice and some foreign comparisons', *The Structural Engineer*, **71/7**, April, 111–16.
2. Rosignoli, M. (2002) *Bridge Launching*. London: Thomas Telford Publishing.

@Seismicisolation

16

Cable-stayed bridges

Introduction

Many recent major cabled-stayed bridges have utilized post-tensioned concrete in the main deck elements. The use of stay cables to support the deck results in a slim section and makes long spans possible. Concrete is a versatile material able to combine efficient deck arrangements with an aesthetically pleasing appearance. The construction of concrete cable-stayed bridges uses many of the traditional techniques associated with the other types of concrete bridges.

As concrete is good in compression it is an ideal material to resist the high compression forces generated in the deck by the stays, especially adjacent to the pylons. Elsewhere, where the compression from the stays is less, the deck is prestressed to overcome the tension and bending stresses generated by the deck behaviour. The stiff concrete section assists in distributing the load along the deck and between the stays, while its high mass and damping characteristics reduce its susceptibility to vibrations or aerodynamic movements.

Sunshine Skyway Bridge, Florida, in Fig. 16.1, uses a concrete box girder deck to give an elegant cable-stayed arrangement. It was built in 1987 after the

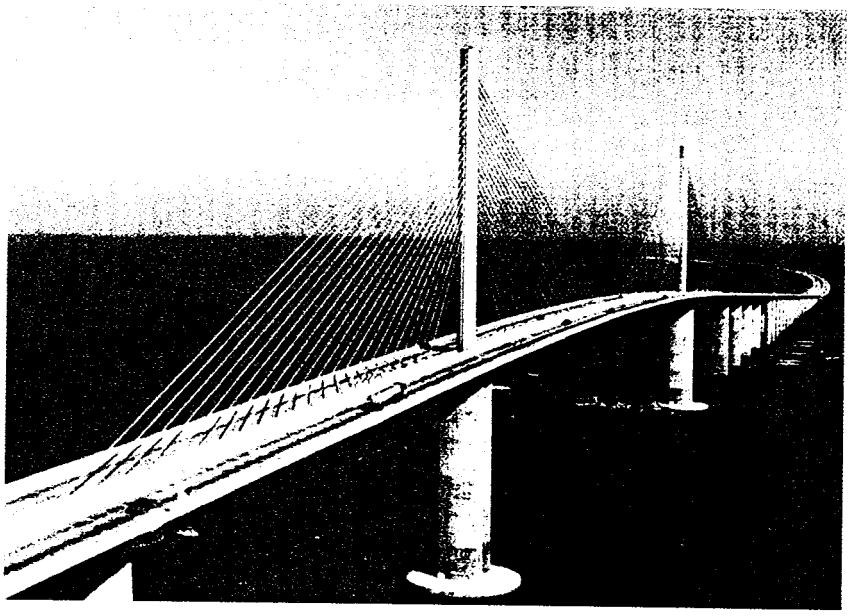


Figure 16.1. Sunshine Skyway Bridge, USA. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

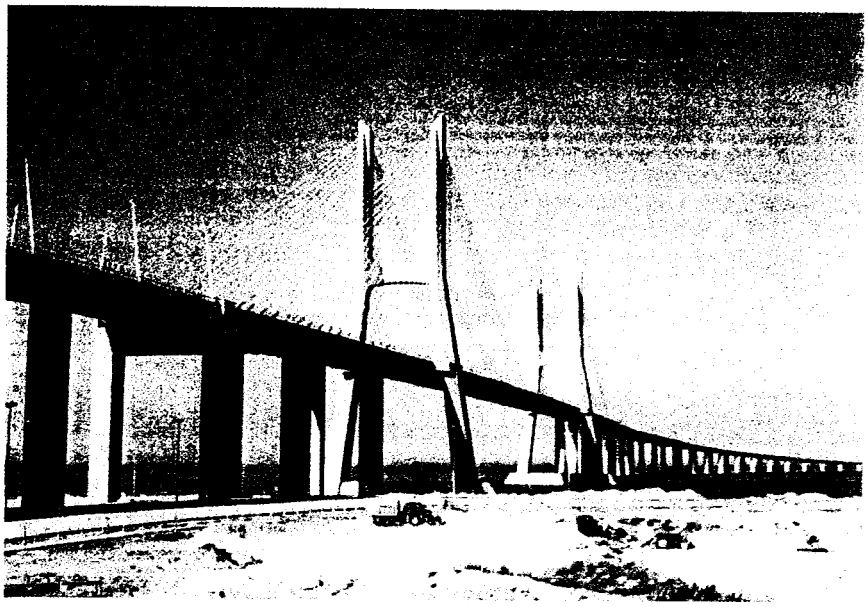


Figure 16.2. Vasco da Gama crossing, Portugal. Reproduced courtesy of Yee Associates. Copyright reserved.

previous bridge was damaged by a ship collision. The 366 m long main span, back spans and approach viaducts are all constructed from precast concrete box girder segments. A 29 m wide deck slab is part of a 4·5 m deep single-box deck arrangement, supported by a single plane of stays anchored down the centre of the box.

The Main Bridge of the Vasco da Gama crossing, Portugal, shown in Fig. 16.2, is part of a much longer estuarine crossing. The 420 m long main span of the cable-stayed bridge provides the required clearance to the shipping lanes below. The slim beam-and-slab deck is only 2·6 m deep and is supported by two planes of stays, one down either side of the deck. Development of the bridge design was part of a design-and-build tender with the concrete arrangement giving the most cost effective solution.

The 'A' shaped pylon and asymmetric arrangement of the bridge in Fig. 16.3 provides a distinctive landmark crossing over the River Dee in Wales. The main span with a length of 194 m has a beam-and-slab concrete deck arrangement and an edge beam depth of 1·7 m. The back span was cast on falsework built up from the ground, while the main span over the river used a form traveller to cast the deck *in situ* as a cantilever.

For long cable-stayed spans the deck is often a combination of steel and concrete elements. Normandie Bridge in France, with a main span of 856 m, uses a steel box over the centre part of the main span and a concrete box adjacent to the pylons in the main span and over the back spans. The light steel box reduces the dead load over the long main span. The concrete box section is better able to resist the high compressive forces in the deck near the pylons, and it provides a heavier weight in the shorter back spans to counterbalance the longer main span.

The design and construction of cable-stayed bridges is complex and requires many different aspects to be considered. This chapter looks at the

@Seismicisolation

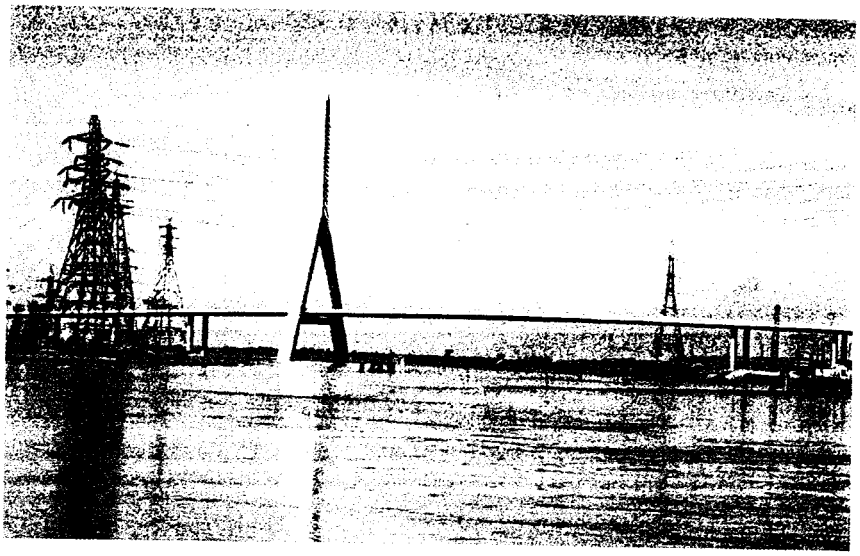


Figure 16.3. River Dee crossing, Wales. Reproduced courtesy of Gifford. Copyright reserved.

general aspects involved with prestressed concrete decks when used on cable-stayed bridges. For a more detailed description of cable-stayed bridges reference should be made to specialist books and other literature, noted at the end of this chapter and Appendix C.

Cable-stayed bridge arrangements

Of all the types of prestressed concrete bridges, the cable-stayed form gives the most diverse range of structural arrangement with each bridge usually being unique. Concrete cable-stayed bridges are built with one, two or more pylons; span arrangements are symmetrical or asymmetrical; and piers may be used in the back spans. The choice of the final arrangement is often due only to the designer's preference.

Concrete cable-stayed decks are either a box or a beam-and-slab arrangement, with typical layouts illustrated in Fig. 16.4. The beam-and-slab arrangement produces the lightest decks, but it needs two planes of stays for support, while the box girder arrangement provides a stiff deck and is used with either one or two planes of stays. Box cross-sections are often precast to give a fast and simple erection process.

The box girder arrangement illustrated in Fig. 16.4(i) has inherent torsional strength, which makes it suitable for use with a single plane of stays. This deck shape is used on the Brotonne Bridge in France, shown in Fig. 1.17, which has a main span of 320 m and box depth of 3·8 m. It is also used on the Sunshine Skyway Bridge, shown in Fig. 16.1.

Twin planes of stays provide some torsional stiffness to the overall structural behaviour and less-stiff deck arrangements such as a 'semi-box', shown in Fig. 16.4(ii), or a beam-and-slab, as shown in Fig. 16.4(iii), are usually adopted.

A semi-box arrangement is used on the Pasco-Kennewick Bridge, USA, with a main span of 299 m and deck depth of 2·13 m. Typical examples of

@Seismicisolation

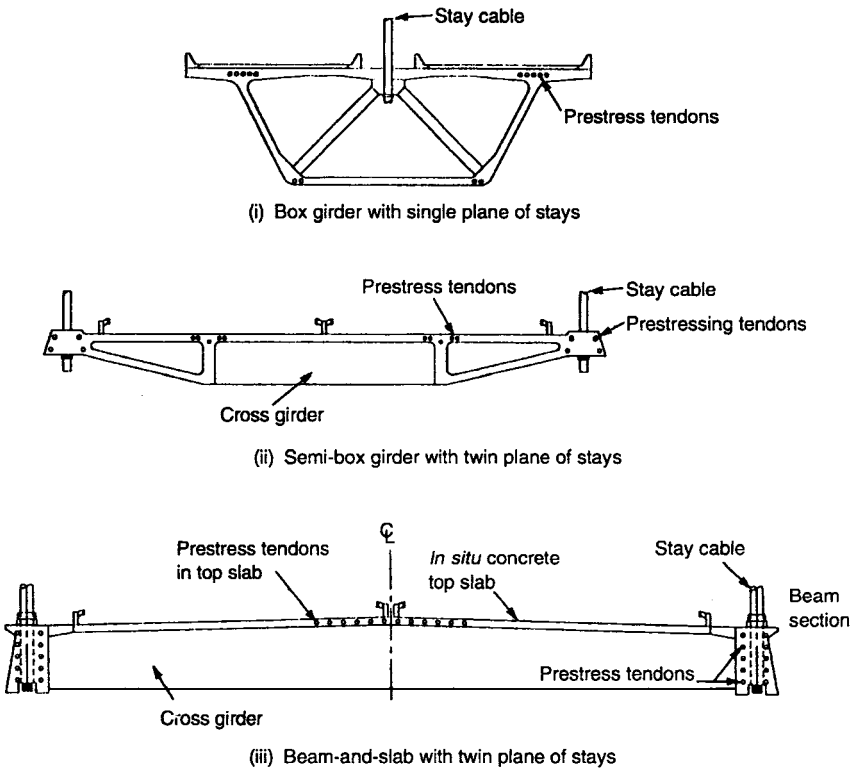


Figure 16.4. Concrete deck arrangements for cable-stayed bridges.

beam-and-slab decks are the Vasco da Gama Bridge, in Fig. 16.2, and the River Dee crossing, in Fig. 16.3.

The length of the main span and height of the deck above the ground is dictated by the obstruction being crossed or the minimum clearance requirements of any traffic passing underneath.

The width of the deck is chosen to suit the users' requirements. Footbridges may be only a few metres wide, yet a road bridge, carrying a dual carriageway, may be over 30 m wide.

Decks are typically between 1·4 m and 3 m deep for a beam-and-slab arrangement and between 2·5 m and 4·5 m deep for a box arrangement, depending on the spacing of stays, the deck width and the live load being carried.

The shape of the pylons is usually based on 'H', 'A' or 'I' layouts although there are many variations of these. The pylon shape has the biggest influence on the appearance of a cable-stayed bridge and the final arrangement is often developed based on aesthetic considerations. To obtain an efficient design for the cable-supported structure the height of the pylon above the deck is normally between 20 per cent and 25 per cent of the main span length where two pylons are used, or between 40 per cent and 50 per cent of the main span length where a single pylon is used.

At the pylon the deck is either built-in to the legs, supported on bearings or left hanging freely from the stays. Building the deck into the pylon provides a

@Seismicisolation

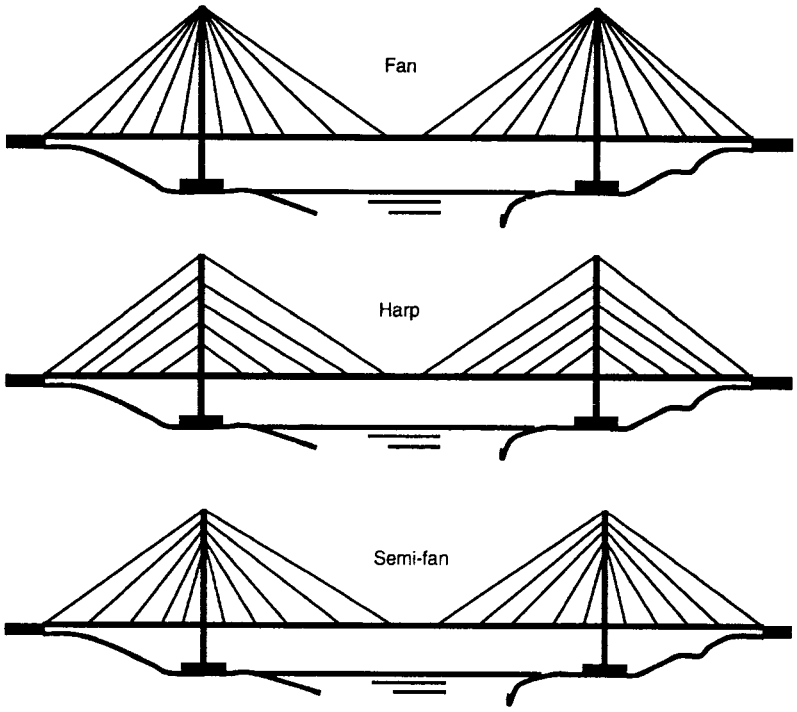


Figure 16.5. Multi-stay arrangements.

fixed support, but it causes a 'hard spot' along the deck which attracts load and may overstress the concrete. Placing the deck on bearings on a crossbeam softens the support, while hanging the deck freely on the stays results in a more uniform distribution of bending moment along the deck.

Early cable-stayed bridges used only a few stays widely spaced along the deck, while, more recently, most projects use multi-stay systems with closely spaced stays. Fan, harp or semi-fan layouts of stays are illustrated in Fig. 16.5.

A fan arrangement is the most efficient in terms of generating the support to the deck, resulting in least weight of stay material, but the anchorage on the pylon is congested. A harp arrangement is often considered aesthetically better, but the stays work less efficiently. The semi-fan arrangement is a compromise between the two with the stay anchors on the pylon spread out to ease congestion and the stay angle maximized to give reasonable efficiency. The semi-fan arrangement is the most commonly adopted arrangement.

With multi-stay arrangements the stays' spacing along the deck is typically between 8 m and 12 m. This provides a good support to the deck and allows thin, flexible decks to be used.

Several different types of stay are used with concrete cable-stayed bridges: the most common being based on 7-wire prestressing strand. Multi-strand stays, with up to 127 strands, capable of carrying forces of 1600 tonnes are available, and special anchorage and sheathing systems have been developed to ensure the integrity and durability of the stay. The strands are usually

@Seismicisolation

galvanized and individually sheathed inside a grease-filled high-density polyethylene duct to provide a multiple protection system.

Lock-coil, stressbars and parallel wire stays are also used where preferred by the designer. All stays adopt a multiple protection system, with galvanized wires or bars surrounded by additional protection such as ducting or painting.

Construction of concrete cable-stayed bridges

Concrete deck sections are usually constructed *in situ* as segments cast inside a form traveller or with precast elements transferred to site and lifted into position. Both methods involve cantilevering the deck out from the pylons in a balanced manner and installing the stay cables as the cantilever extends, as illustrated in Fig. 16.6.

As the deck is cantilevering during construction, high bending moments are generated over the end section due to the weight of the construction equipment, form traveller and wet concrete of the new segment. Temporary stay systems, such as the arrangement indicated in Fig. 16.6, are used to distribute the high loads on the end of the deck back along the structure and into the permanent stays.

Where there is good access under the deck the concrete may be cast *in situ* on full-height falsework, as shown in Fig. 16.7, before the stays and prestressing tendons are installed. The advantage of casting the deck on full-height falsework is the simpler concreting procedure and stay installation, which results in faster construction. However, with difficult access, high structures or long decks, the cost of extensive falsework is prohibitive.

Several projects have combined the use of full-height falsework for the back spans, when the access is good, with a form traveller to cantilever out

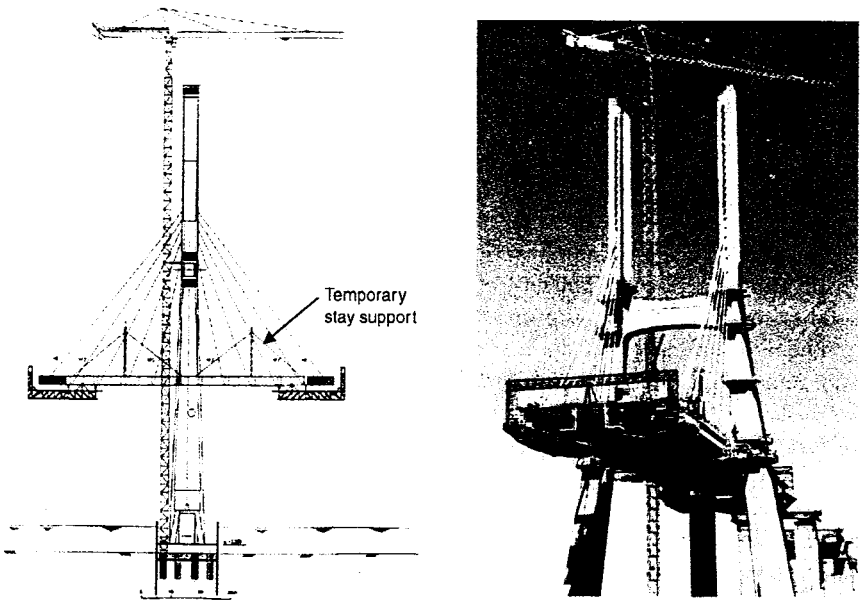


Figure 16.6. Balanced cantilever construction. Reproduced courtesy of VINCI Construction Grands Projets and Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

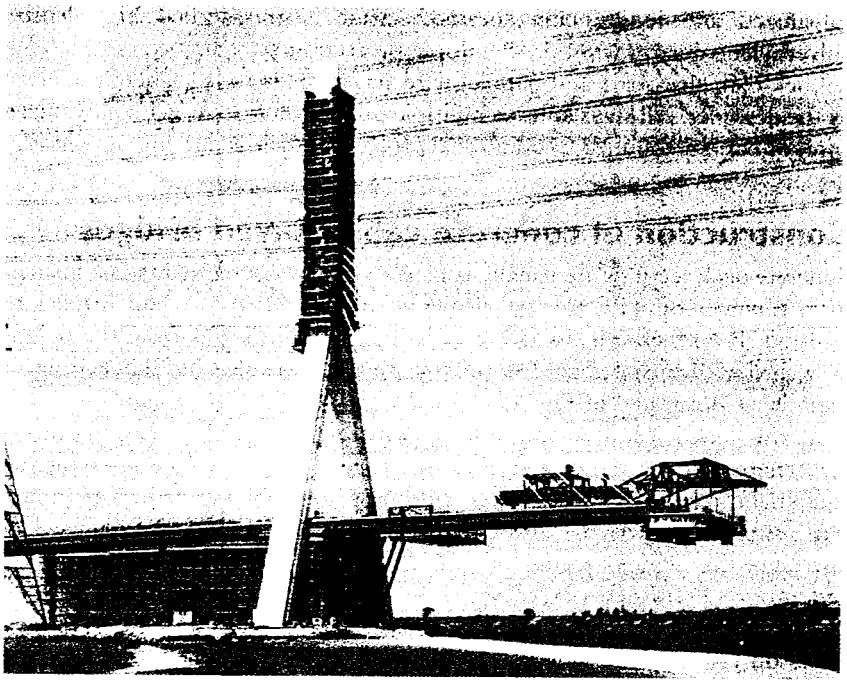


Figure 16.7. Deck cast partly on falsework and partly with traveller. Reproduced courtesy of Gifford. Copyright reserved.

the main span over the obstruction being crossed. This gives the advantages from both systems.

Stressing of the stays is normally done from the deck anchorage with the jacks located on the form traveller or access platform. Some projects have stressed the stays from the pylon anchorage end, although this is usually where the pylons are large enough to provide sufficient room to arrange the lifting equipment and access platforms to position the jacks.

The deflections, forces and stresses in a cable-stayed bridge continually change as the deck is being built, and comprehensive monitoring is carried out to ensure that the structure is behaving as expected. As well as geometry surveys, the stresses, temperatures and stay forces are monitored, as described in the paper by Al-Qarra (1999) [1].

Setting out and surveying the position of the deck requires careful control due to the many factors that influence the structural behaviour. As the temporary loads and construction equipment move along the deck the pylon and deck deflect. The natural variations in the properties of the concrete used affect the achieved alignment at each stage of the construction.

The temperature of the deck, stays and pylon affects the vertical, longitudinal and transverse alignment of the deck. Temperature differences between the concrete and stays cause bending in the deck and a change to the deck levels. Differential temperature gradients through the concrete also change the deck profile. It is difficult to determine accurately the temperature differentials within the structure at any particular time of day and hence to predict the theoretical position of the structure at that time. It is usual to do all of the setting out and surveying activities during the early hours of the morning, when any temperature differentials are at a minimum.

@Seismicisolation

Box girder decks

Concrete box girders are either cast *in situ,* precast, or in a combination of the two. The techniques used in their construction are similar to that for other types of box girder decks, described in Chapters 12 and 13, with stay cables installed to support the deck during the construction and in the final arrangement.

In situ box girders are cast in short lengths in a form traveller, as described in Chapter 12. The length of the segments is usually between 3 m and 5 m, to suit the stay spacing, with stays being anchored at every second or third segment. During the construction the deck is subjected to high bending moments at the end of the cantilever and either reinforcement or prestress is used to cater for this.

Precast box girders use match-cast segments, formed and transported as described in Chapter 13. Their erection follows a similar sequence as for a balanced cantilever deck except that they have stays attached as the cantilever is extended out. Temporary prestressing is used to join the segments together and support the deck before the stays and permanent prestressing tendons are installed.

Precast segments are usually positioned by a lifting frame or crane located on the completed deck. The 250 tonne precast segments on the Wadi Leban Bridge were raised into place using a lifting frame, as seen in Fig. 16.8. Using precast, match-cast segments and lifting them into place minimizes the work done on site and greatly reduces the construction time.

Several projects have combined precasting and *in situ* construction. The box of the Brotonne Bridge in France, seen in Fig. 1.17, used thin precast web elements lifted into the form traveller with the slabs cast *in situ* to achieve a construction cycle of three days for a 3 m long segment.

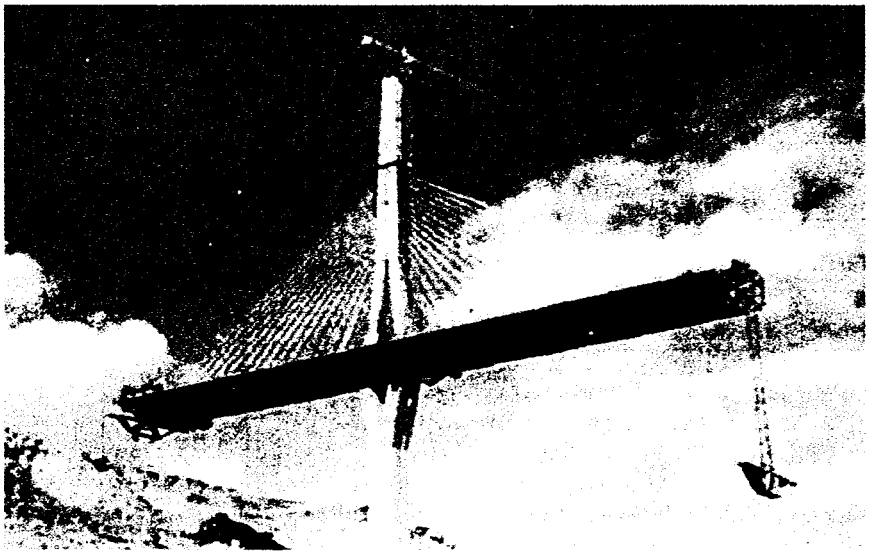


Figure 16.8. Lifting deck segments into place. Reproduced courtesy of VSL International. Copyright reserved.

@Seismicisolation

Beam-and-slab decks

Decks consisting of prestressed concrete edge beams, transverse girders and concrete deck slabs have become popular in recent years because they produce light and slender decks, thus combining both economy and aesthetics. Two planes of stays are used, one down either side of the deck, to provide transverse and torsional stiffness to the structure.

Cable-stayed decks utilizing concrete beam-and-slab arrangements have been built with main spans of up to 500 m. The 830 m long Vasco da Gama crossing's cable-stayed bridge, in Fig. 16.9, illustrates a typical layout for this type of deck arrangement. The additional piers in the back spans provided stability during the construction and stiffened up the cable structure in the permanent arrangement.

Another example, the Yamuna cable-stayed bridge in India, is illustrated in Fig. 16.10. The cable-stayed deck is continuous with the side spans which

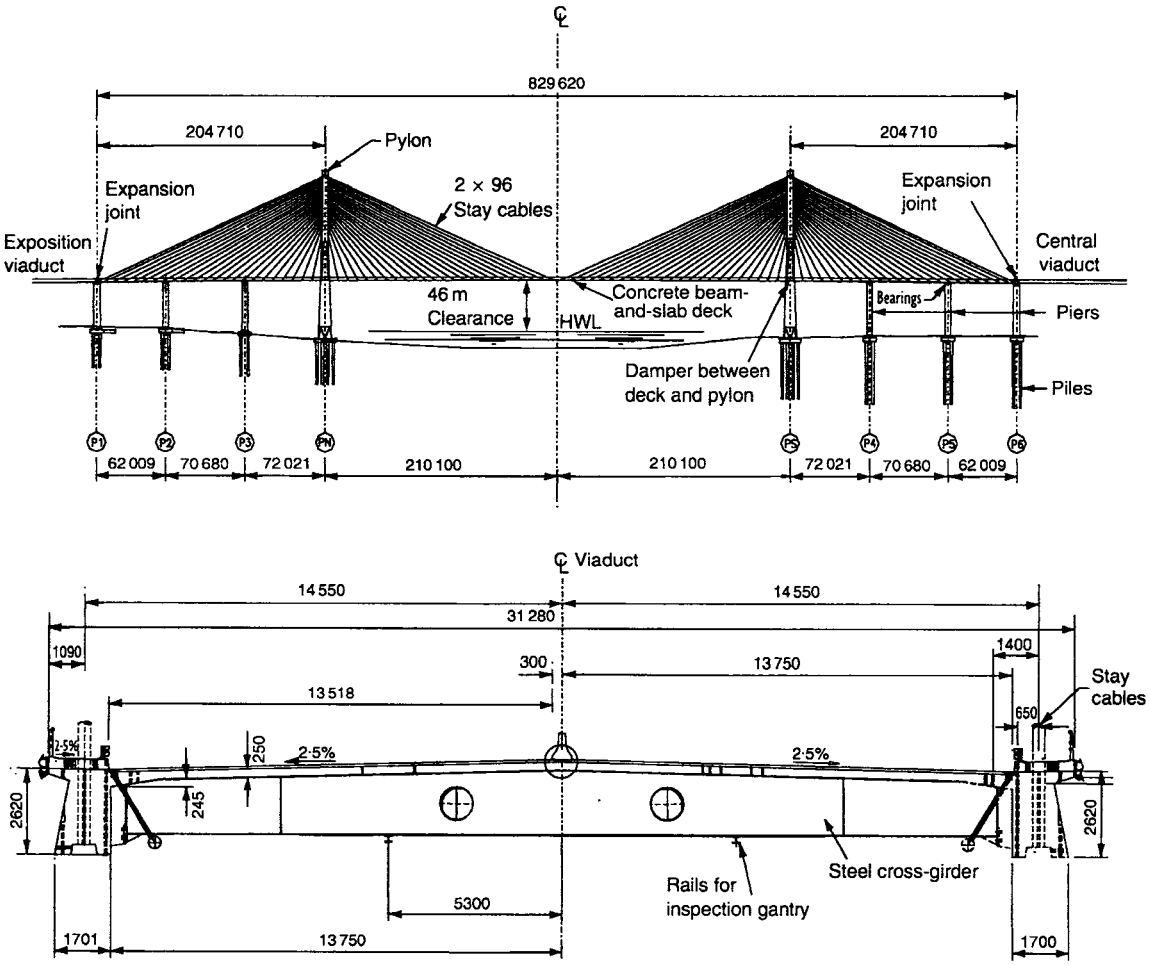


Figure 16.9. Vasco da Gama crossing cable-stayed bridge layout. Reproduced courtesy of VINCI Construction Grands Projets. Copyright reserved.

@Seismicisolation

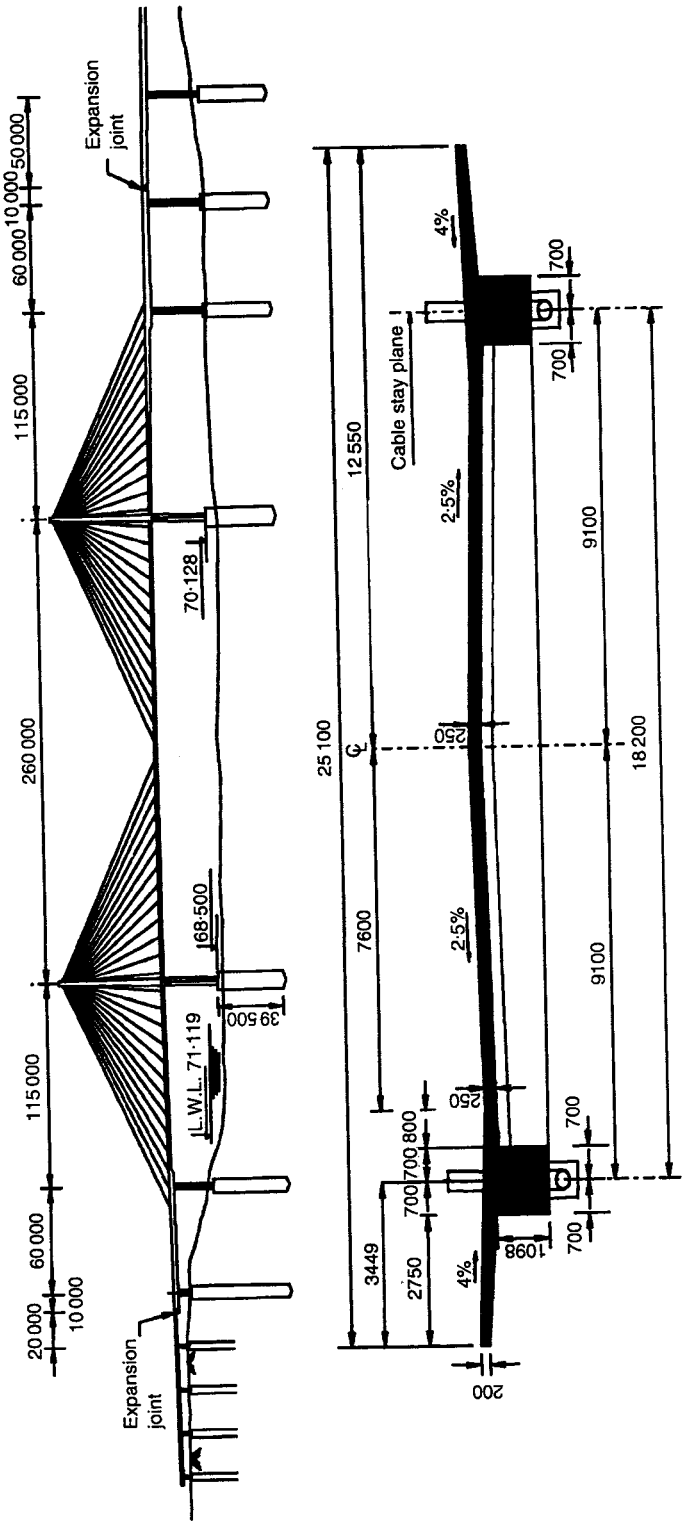


Figure 16.10. Yamuna cable-stayed bridge layout. Reproduced courtesy of Cowi. Copyright reserved.

@Seismicisolation

stiffen up the cable supported structure and provide a stiff support for the outer stays to be anchored on. By placing the footpaths outside the plane of stays the distance between the edge beams was reduced and the deck slab arrangement more balanced.

The concrete edge beams provide the anchorage for the stays. They stiffen the deck longitudinally and support cross-girders spanning transversely. The position of the cross-girders usually matches the stay locations, with additional cross-girders between to give a typical spacing of between 4 m and 5 m. The deck slab thickness is usually between 250 mm and 300 mm.

Using steel beams for the cross-girders minimizes the deck weight and simplifies the form traveller construction and operation. The Vasco da Gama crossing's cable-stayed bridge, with a distance of 27·5 m between the edge beams, uses steel cross-girders spaced at 4·5 m centres supporting a 250 mm thick deck slab.

Concrete cross-girders usually have a simple 'I' or 'T' arrangement and are either precast or cast *in situ*. The concrete cross-girders on the Yamuna cable-stayed bridge in India are spaced at 5 m centres with a 250 mm deck slab, spanning 16·8 m between the edge beams. They were cast *in situ* and post-tensioned with multi-strand tendons. Concrete cross-girders may be simply reinforced instead of prestressed where the deck width and design allow.

Edge beams and deck slabs are usually cast *in situ* inside a form traveller, although precast sections have been used on several projects. Figure 16.11 shows the form traveller of the Vasco da Gama crossing set up ready to cast the next 9 m long segment of deck. Segment lengths usually match the stay spacing. The construction time for each segment is typically between 7 and 12 days.

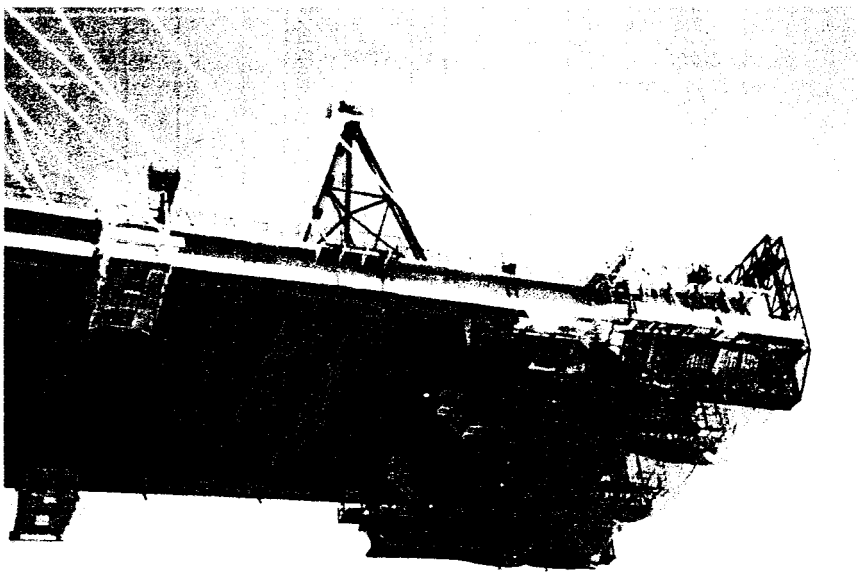


Figure 16.11. Form traveller for casting deck. Reproduced courtesy of Hyder Consulting Ltd. Copyright reserved.

@Seismicisolation

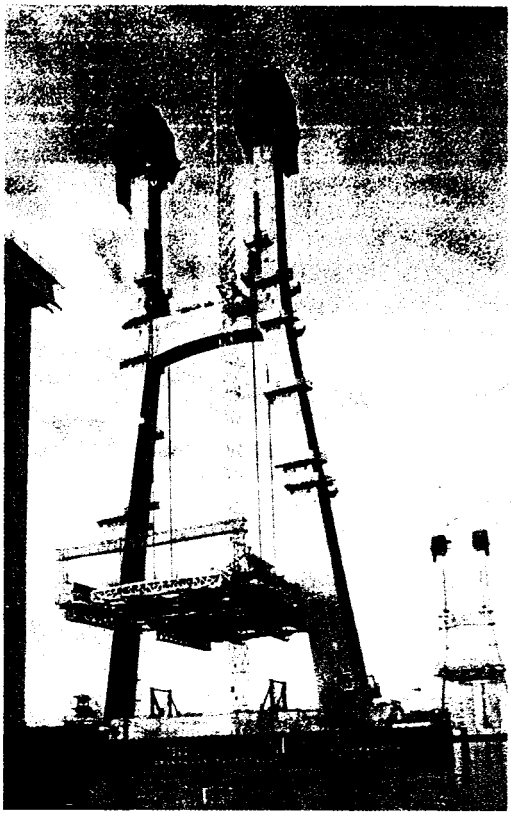


Figure 16.12. Form travellers being lifted into position. Reproduced courtesy of VINCI Construction Grands Projets. Copyright reserved.

A special formwork arrangement is required for the first section of deck at the pylon. The pylon table is usually cast using full-height scaffolding from the ground or with the standard form travellers adapted and lifted into place, as seen in Fig. 16.12. After positioning the deck formwork the reinforcement and tendon ducts are installed and the concrete cast.

When the deck concrete has reached sufficient strength the first stays are installed to support the deck, enabling the falsework to be removed or the form travellers released and moved forward. The form travellers are then set up for the construction of the next section of cantilevered deck, as shown in Fig. 16.13.

Prefabrication of parts of the reinforcement cage, as seen in Fig. 16.14, simplifies the overall construction process and reduces the cycle time for the construction of each deck section. Prefabricating is of particular benefit in the area where the stay is fixed to the deck. The stay anchor, adjacent cross-girder and edge beam requirements result in a complex reinforcement and tendon arrangement which is easier to pre-assemble in a 'factory' environment before being lifted into place.

On the Vasco da Gama crossing the multi-strand stays were installed before concreting the deck section, as seen in Fig. 16.15. The stays were initially anchored on to the form traveller with only a small load applied, and they

@Seismicisolation

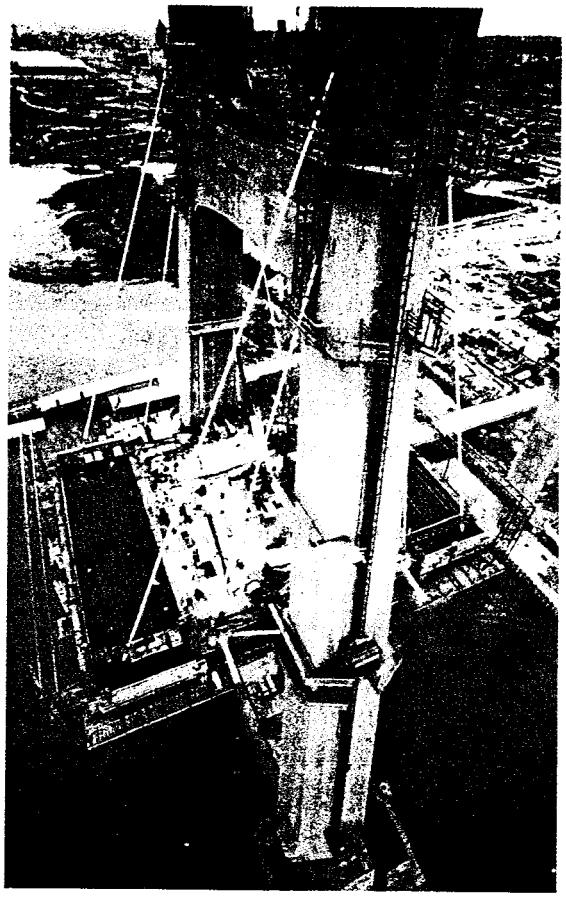


Figure 16.13. Beam-and-slab deck under construction. Reproduced courtesy of VINCI Construction Grands Projets. Copyright reserved.

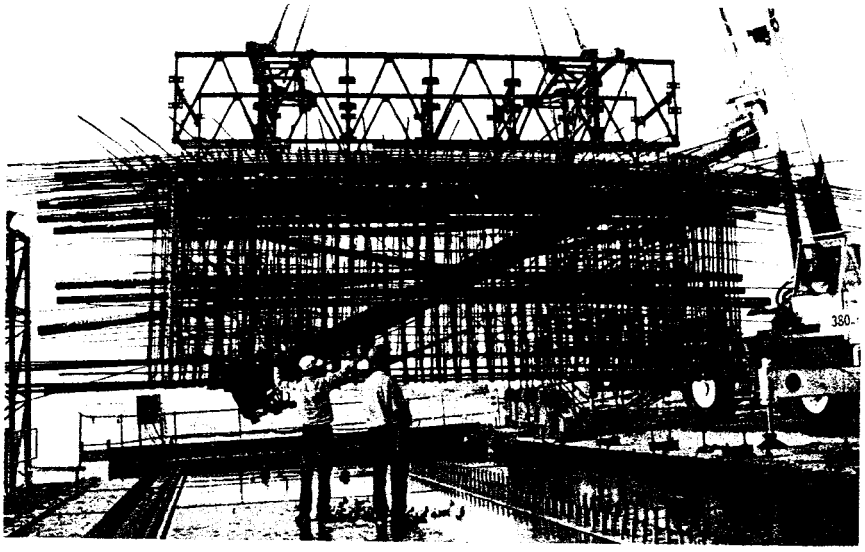


Figure 16.14. Prefabricated reinforcement cage. Reproduced courtesy of VINCI Construction Grands Projets. Copyright reserved.

@Seismicisolation

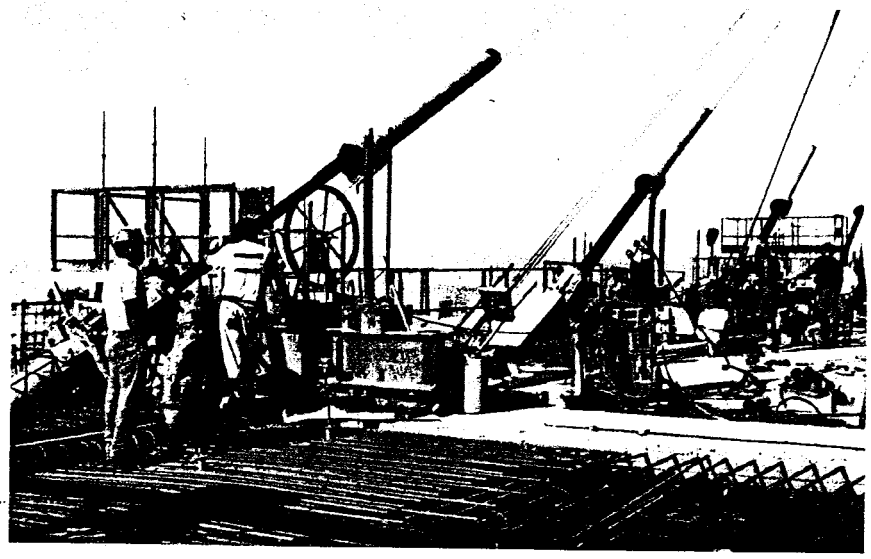


Figure 16.15. Stays being installed. Reproduced courtesy of VINCI Construction Grands Projets. Copyright reserved.

supported the traveller under the weight of the fresh concrete. Temporary stays, as seen in Figs 16.6 and 16.11, were used to help distribute the load at the end of the cantilever back into the deck and other stays. After the edge beam concrete had gained sufficient strength the stay anchors were transferred to their permanent position and fully stressed.

With *in situ* beam-and-slab construction the deck is usually designed as a reinforced concrete section as it cantilevers out. Sufficient reinforcement is provided to cater for the moments and shears generated. The longitudinal prestress tendons are installed after the closure stitches in the main span and back spans are completed.

Design aspects associated with concrete cable-stayed bridges

The design of cable-stayed bridges is complex and must take into account many different factors to build up the overall structural behaviour. The influence of the deck, pylon and stay arrangements and the behaviour of the stayed structure under dead, superimposed and live loads are described in the books by Troitsky (1988) [2] and Walther *et al.* (1988) [3]. The effects of wind loading and the associated dynamic behaviour are described in the *TRL State-of-the-art Review 5* by Hay (1992) [4].

Typical sections for concrete cable-stayed decks are illustrated in Fig. 16.4, although many variations of these basic arrangements can be used. The optimum arrangement for any particular project will depend on specific requirements such as deck width, traffic arrangements and minimum span length. The choice of stay arrangement and pylon shape also influences the deck layout, while aesthetics and architectural requirements play a large part in the final arrangement chosen.

@Seismicisolation

The main difference between the design of concrete and other cable-stayed bridges is that the creep and shrinkage in the concrete changes the forces in the deck and stays, and the deflections, over the life of the structure.

Analysis of cable-stayed bridges

In simplistic terms, the deck behaves like a beam-on-elastic foundation with each stay providing a spring support to the deck. Initial sizing of the deck and stays during the conceptual stage of the design may be carried out using a line-beam model, as illustrated in Fig. 16.16(i). The equivalent stiffness of the support at each stay location is given by:

$$K_s = (E_i A_{stay} \sin^2 \alpha)/L_{stay}$$

The modulus of elasticity of the stay is modified due to the sag in the cable which is dependent on the force in the stay as follows:

$$E_i = E_s/[1 + \gamma_s^2 H_{stay}^2 E_s/12(T_{stay}/A_{stay})^3]$$

As an initial estimate the stress in the stays, T_{stay}/A_{stay}, may be taken as between 35 per cent and 40 per cent of the UTS, to represent the typical situation with all the dead load and some of the live load present.

A simplistic model is adequate to develop the structural arrangement and element sizing during the concept design, but a more sophisticated model is needed to take into account all the aspects involved with the detailed analysis of cable-stayed bridges.

The deck behaviour interacts with the stay and pylon behaviour and the design of the deck itself is an integral part of the overall design of the cable structure. To carry out the detailed design of the structure a full model of

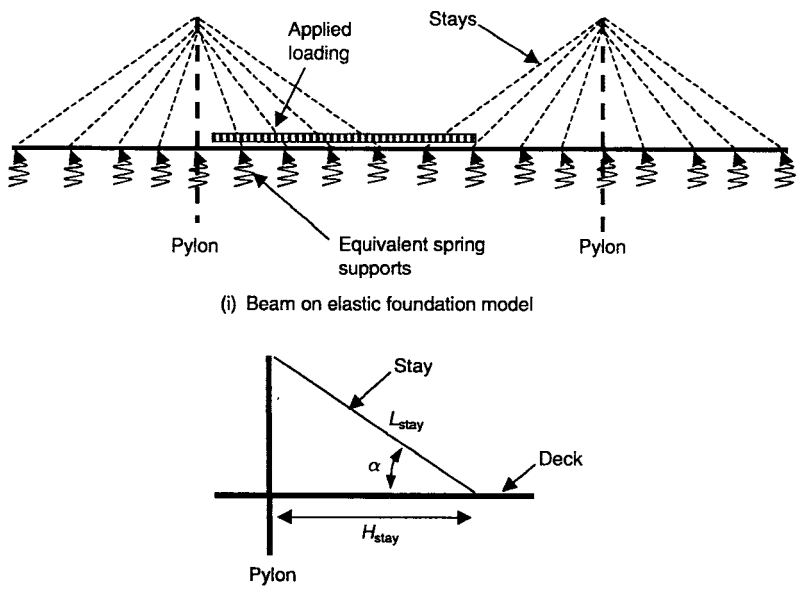


Figure 16.16. Simplistic analysis model.

@Seismicisolation

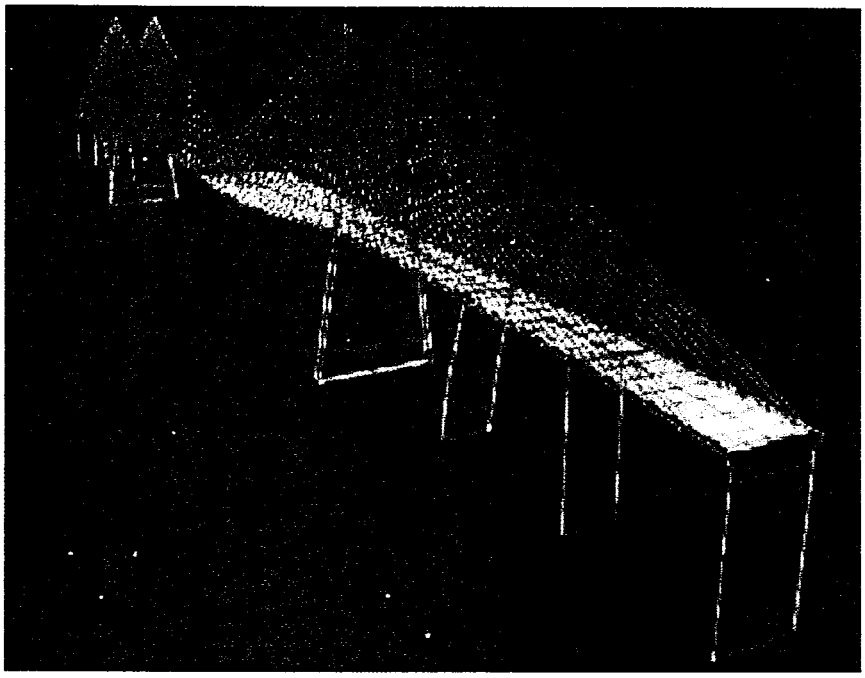


Figure 16.17. Three-dimensional analysis model.

the superstructure and substructure is established to derive the moments, shears and axial forces in the deck under the different loading conditions. Figure 16.17 is an example of a three-dimensional finite element model established using the LUSAS software to derive the overall behaviour and dynamic parameters of a cable-stayed bridge, and shows the deflected shape under asymmetrical loading in the main span.

Cable-stayed bridges often have wide slender decks and shear lag effects are significant close to the supports. An effective width approach, as described in Chapter 5, may be used when designing the structure with a two-dimensional analysis model. With the three-dimensional model illustrated in Fig. 16.17 the shear lag is automatically taken into account in the overall structural analysis.

When applying the effective width approach the stays are usually considered as soft supports and do not give the shear lag effect associated with a rigid support. The effective width of the deck slab is the full width of the slab over most of the cable supported section of deck. A reduced width is taken adjacent to the piers and pylons if the deck is rigidly supported at these points.

Specialist software, such as ADAPT described in Chapter 8, is required to combine the long-term effects, such as creep and shrinkage of the concrete and relaxation of the prestress and stays with the overall behaviour of the structure.

The analysis of cable-stayed bridges must incorporate the stage-by-stage construction of the deck and include for any built-in stresses that occur, as described in Chapters 5 and 8.

The analysis output shown in Fig. 16.18 illustrates the deflected shape of a concrete deck for a cable-stayed bridge after all the long-term effects have

@Seismicisolation

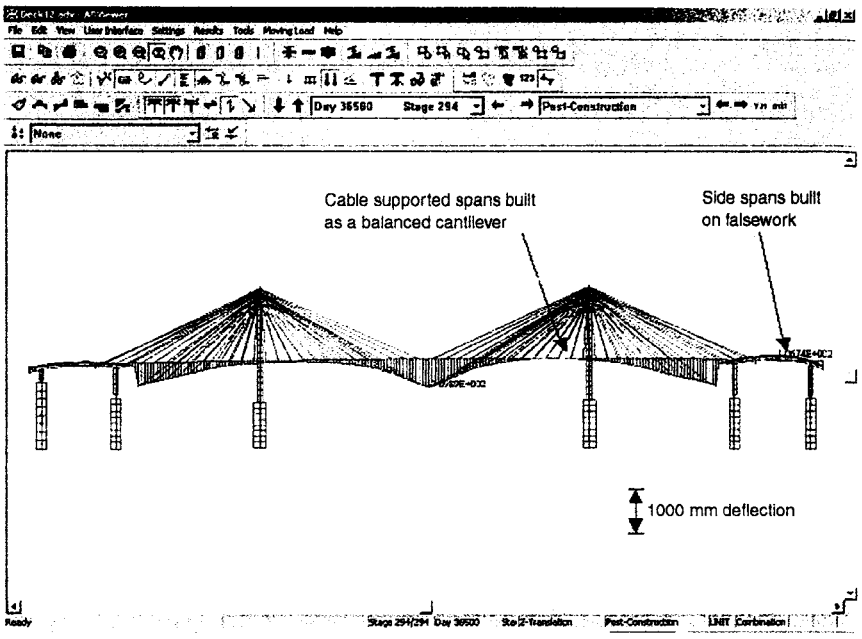


Figure 16.18. Deflected shape of cable-stayed deck. Reproduced with permission.

occurred. The deflections shown assume the deck was built without any precamber imposed on the alignment. In practice the deck is precambered during construction to give the correct deck profile on completion.

During the construction the forces in the stays are set to optimize the deck design in the temporary stages. At the end of the deck construction the stays are re-stressed to give the required deck profile and to produce a uniform distribution of bending moments and shears along the length of the deck. Figure 16.19 illustrates a typical dead-load bending moment distribution in the deck elements after the stays have been adjusted.

The final dead-load bending moments in the deck are set by adjusting the force in the stays during the final re-stressing. The analysis model must incorporate the adjustments to the stay forces during the construction stages and interact with the design process to derive the optimum installation and final force for the stays.

Deck design and behaviour

The concrete deck resists the moments and forces from the imposed loading and from the deck and stay interaction. Adjacent to the pylons the longitudinal compression generated by the stays is usually sufficient to counter the tensile stresses from the bending and shear in the permanent condition. At mid-span and near the ends of the deck there is little compression from the stays, and prestressing tendons are installed to keep the stresses within the acceptable limits. Figure 16.20 illustrates a typical axial force profile along the length of a concrete cable-stayed deck, assuming prestressing tendons are installed over the mid-span and ends of the deck as indicated.

The prestress tendon profiles are usually 'straight' with the number and extent of the tendons determined by the stresses generated in the concrete

@Seismicisolation

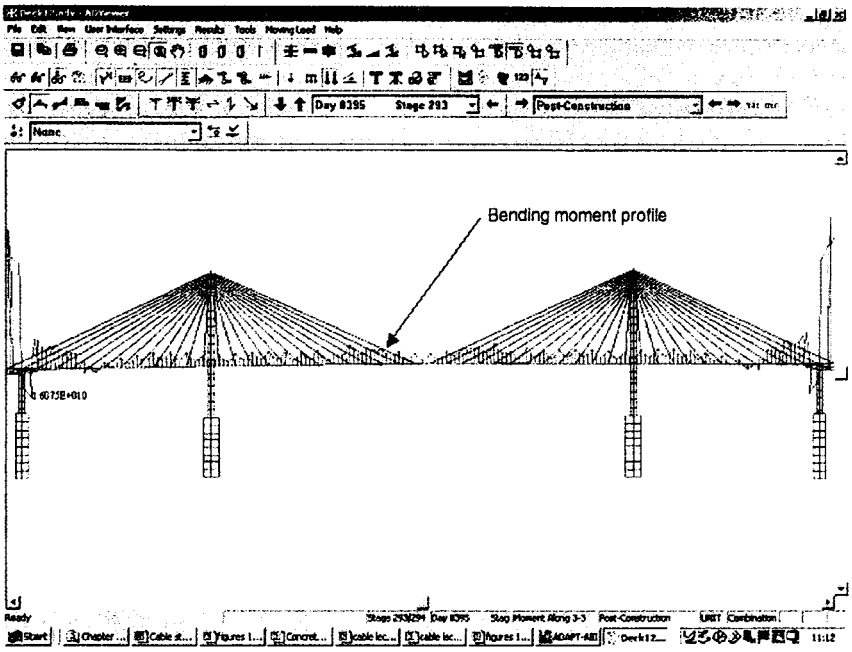


Figure 16.19. Dead-load bending moment profile. Reproduced with permission.

from the loading and overall structural behaviour. With a box girder deck the tendons run through the top and bottom slabs and are anchored on blisters inside the box, as illustrated in Fig. 6.6. With a beam-and-slab deck the tendons are positioned in the edge beams and deck slab as illustrated in

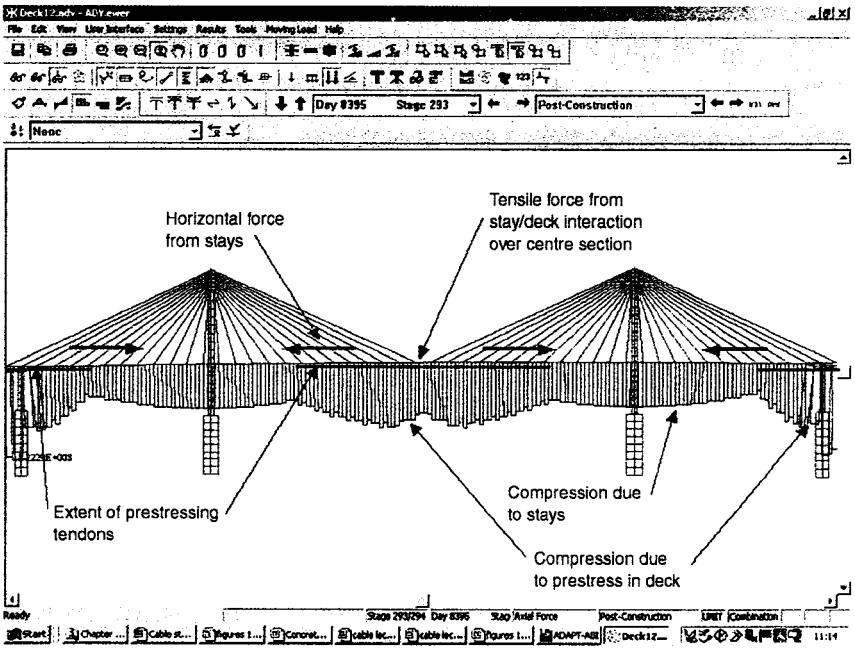


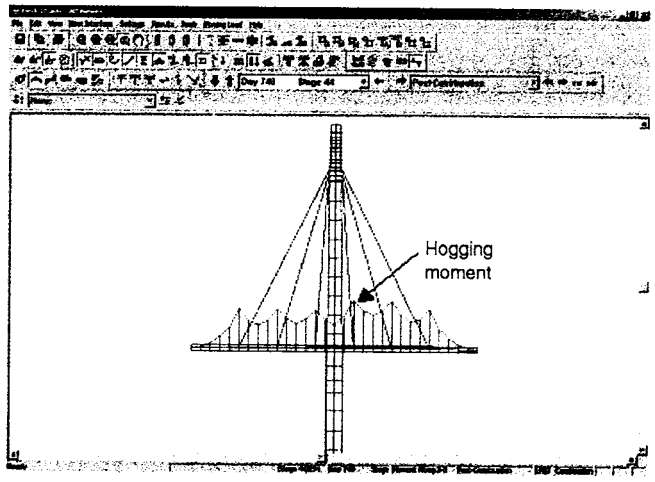
Figure 16.20. Axial force in deck. Reproduced with permission.

@Seismicisolation

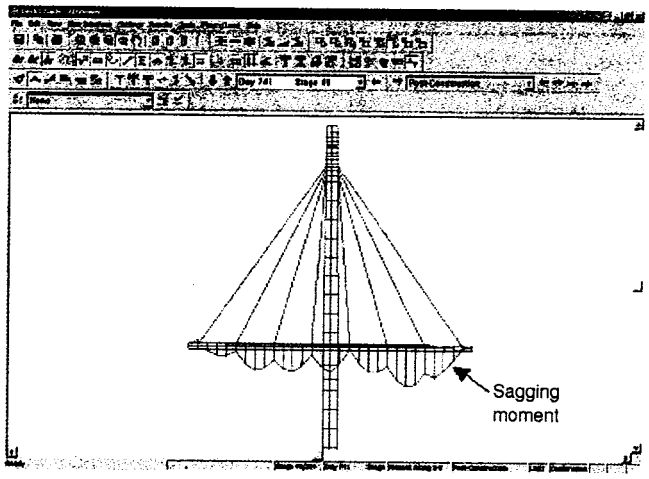
Fig. 16.4(iii). Tendons in the slab are usually anchored in recesses on the concrete face, while the edge beam tendons are anchored on blisters formed on the beam.

The deck design is often governed by the forces and stresses generated during the construction. Each stage of the erection sequence is checked for both the strength and serviceability requirements and for overall stability.

To check the design at each erection stage, the partly built structure is modelled with all the construction loads considered to derive the moments, forces, stresses and deflections. Figure 16.21 shows the range of bending moments from a typical analysis during deck cantilevering. These moments are combined with the co-existent axial and shear forces to check the section capacity and serviceability stresses or crack widths.



(i) Before installing stays



(ii) After installing stays

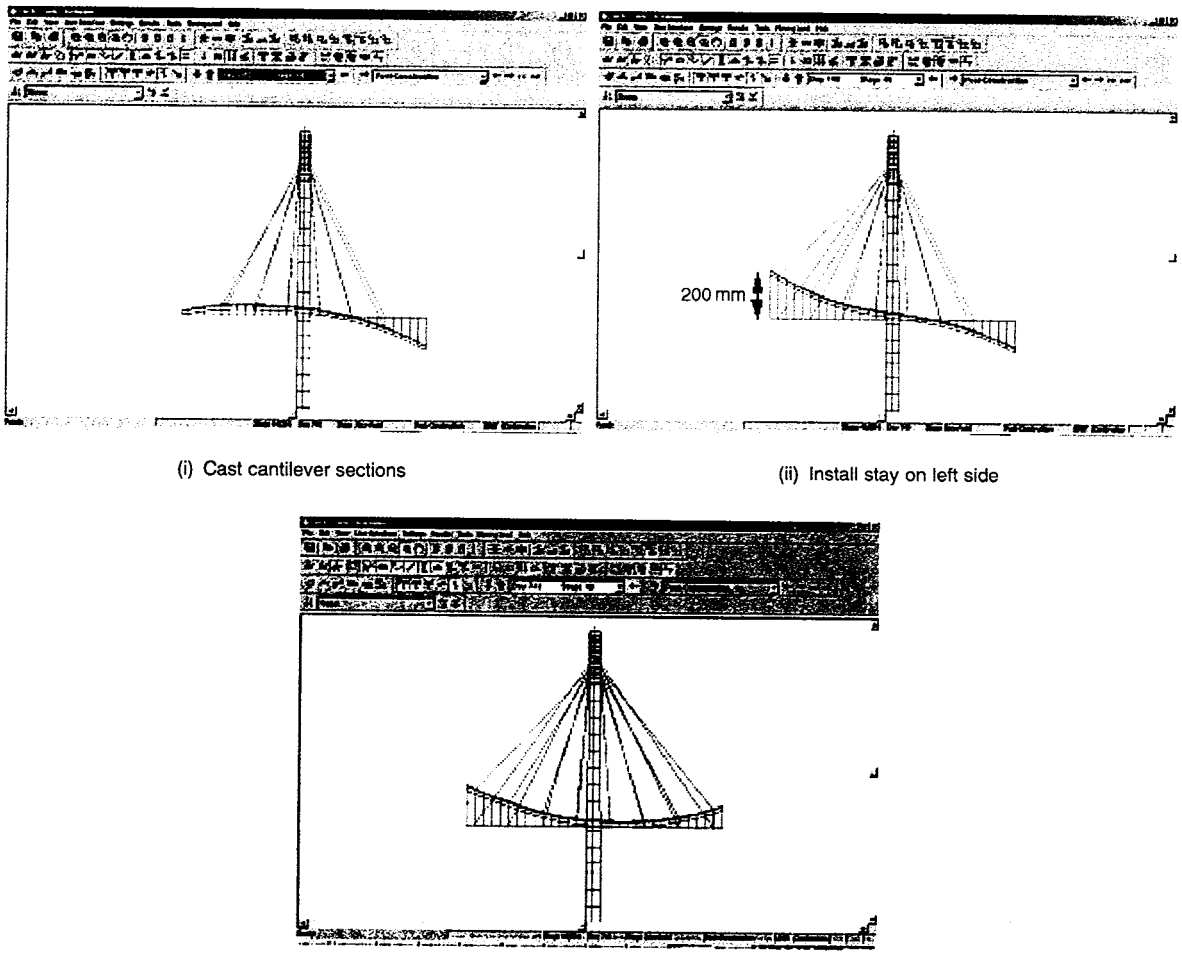
Figure 16.21. Bending moments during deck cantilevering. Reproduced with permission.

@Seismicisolation

During casting of each deck section the additional weight at the end of the cantilever causes hogging in the previously completed deck, as illustrated in Fig. 16.21(i). The hogging is increased as the construction equipment and form traveller are moved forward. When the next stays are installed they pull the deck up and create a sagging moment along the completed deck, as illustrated in Fig. 16.21(ii). The cycle of hogging and sagging moments continues as the deck is built out.

The compression in the concrete deck generated by the stays during the construction stages is normally insufficient to keep the concrete stresses within acceptable limits. Reinforcement is provided in *in situ* decks to give the required strength and control crack widths in the concrete. With precast decks additional prestressing is installed to keep the joints in compression.

The deflection of the deck at each construction stage, as illustrated in Fig. 16.22, is extracted from the analysis for use in monitoring the deck



(i) Cast cantilever sections

(ii) Install stay on left side

(iii) Install stay on right side

Figure 16.22. Deck deflections during cantilevering. Reproduced with permission.

@Seismicisolation

behaviour and to estimate the precamber required. When a section of deck is cast the additional concrete weight causes the deck to deflect downward, as indicated in Fig. 16.22(i). On installing the stay on one side of the pylon, the deck is pulled up on that side while the profile on the other side is also affected, as illustrated in Fig. 16.22(ii). Installing the next stay on the other side of the pylon balances the forces and deflections, as indicated in Fig. 16.22(iii).

The changes in forces and stresses in the deck and the deflections that occur during the construction are much greater in cable-stayed bridges than other bridge types. Although this does not cause any particular problem in their design, they are more sensitive to variations in the properties of the concrete and to secondary effects in the structural behaviour.

Large deflections in the pylon and deck of a cable-stayed bridge cause the p–δ effect to be significant, requiring the use of structural analysis software that takes into account geometric non-linear behaviour.

Concrete modulus of elasticity, E_c, values stated in design codes and books are typical values that can vary by up to ±20 percent compared to a particular mix. The creep and shrinkage of concrete depends on many factors and can vary greatly from the typical values stated in the design codes. Whereas this may not be significant for other bridge types, for cable-stayed bridges it may be necessary to carry out laboratory testing on the actual concrete mix being used to determine accurately the concrete parameters.

The deck often acts as a reinforced concrete section during construction and the concrete is designed to crack. Where the concrete is heavily reinforced or significant cracks occur this changes the stiffness of the section which affects the expected deflections and again must be taken into account.

Deck dynamic behaviour

It is necessary to check a cable-stayed bridge for its dynamic behaviour under both live and wind loading. The natural frequencies of the cable-supported structure and stays are determined using one of the proprietary three-dimensional finite-element software programs available, such as *LUCAS* or *STAAD-Pro*. The natural frequency must be different from the vibration frequency of the traffic using the bridge to prevent potential problems. The natural frequencies of the structure are also used to develop the models for the wind engineering studies.

The theory and analysis of dynamic behaviour are explained in the guide by Maguire and Wyatt (1999) [5]. The different aspects of wind engineering are described in the book by Hay (1992) [4]. Dynamic behaviour of the complete structure under wind is usually determined from wind-tunnel tests and involve confirming that wind effects are not critical over the design wind speeds expected. If wind-tunnel tests indicate that vibrations or instability might occur the usual solution with concrete decks is to make small adjustments to the deck arrangement. Changing the shape of the edge of the deck or altering the flow path of the wind above or below the deck is usually sufficient to overcome the problem.

Wind effects are checked at critical stages during the construction as well as on the final structure. The construction stages with the deck cantilevering out

@Seismicisolation

are often more susceptible to dynamic behaviour than the completed structure. In the temporary situation the deck is stabilized by fixing it to the pylons or providing tie-down stays between the deck and foundations. Alternatively, temporary piers or damping devices are used to provide stability.

The heavier weight and higher damping of concrete decks compared to steel decks makes them less susceptible to vibrations and dynamic effects. The damping, expressed as the logarithmic decrement, may be 3 per cent for a prestressed concrete deck and 5 per cent for a reinforced concrete deck.

Stays

The most common type of stay is made up of 7-wire prestressing strands. Lock-coil cables, multi-bar and parallel wire systems are also used, but they are less popular. Stays are designed to have a maximum stress of less than 45 per cent UTS under normal loading to minimize the risk of fatigue failures. This is usually increased to 55 per cent UTS for exceptional loading conditions.

The stays impart a large force locally on to the deck, and the transfer of this force into the concrete section influences the deck layout and requires careful consideration. The possible stay locations are restricted by the traffic layout on the deck slab. The stays are set back out of the way of the traffic and fully protected from any possible accidental damage.

With beam-and-slab arrangements the stiff edge beams provide the ideal location to place the stay anchors. On box girders the webs do not always coincide with the possible locations for the stays and the anchors are located under the deck slab with local stiffening or additional transverse structural members to distribute the load across the section, as illustrated in Fig. 16.4(i).

Where the stays are fixed to the concrete deck the arrangement is similar to an external prestressing anchorage, although with extra protective systems incorporated and always detailed to be re-stressable. The concrete around the stay anchor is designed in the same way as for a prestressing anchor, described in Chapter 6. A typical edge beam anchor arrangement is illustrated in Fig. 16.23.

Stays are vulnerable to vibrations from wind effects and traffic loading, and damping devices are usually incorporated into the stay arrangement. The anchorage detail often includes a circular rubber bearing, positioned at the end of the stay pipe, that acts to reduce any local bending in the ends of the stay and to dampen out any vibrations. Other damping methods adopted include hydraulic pistons fixed between the stay and the deck, and damping ropes fixed across several stays to connect them together. In recent years a phenomena known as wind–rain induced vibration has been observed under a combination of rain and low wind speeds between 7 m/s and 20 m/s. To prevent this a ribbed or roughened sheathing is used to encase the stay and to control the water and wind interaction.

Temporary loading

For *in situ* construction the temporary loading on the deck usually includes the following:

@Seismicisolation

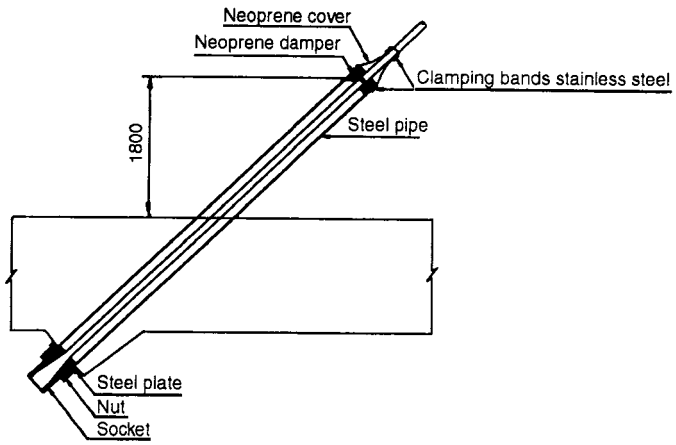


Figure 16.23. Stay anchorage arrangement. Reproduced courtesy of COWI and Hyder Consulting Ltd. Copyright reserved.

- form traveller at the end of the deck cantilever, typically between 1000 kN and 2500 kN depending on deck size and construction sequence;
- stressing jacks for stays, typically 30 kN located on the form traveller or on the pylon as appropriate;
- winch for installing stay, typically 30 kN placed on the deck either positioned at the pylon or near the stay anchorage;
- deviator to guide stay into anchorage, typically 10 kN placed near the deck anchorage;
- uncoiler for stay, typically 30 kN, placed near the deck anchorage or at the pylon;
- construction equipment, typically taken between 30 kN and 40 kN, placed on the tip of the deck cantilever;
- construction live load, typically taken as 0·5 kN/m^2 applied to the deck and form traveller to give the most adverse effect;
- wind loading on the deck including 'upward' wind under one of the cantilevers and dynamic wind on the partially completed structure.

@Seismicisolation

For decks with precast elements, the form traveller is replaced by lifting equipment, which is often of a similar weight.

In addition, it is usual to consider an out-of-balance dead load, with the concrete 2·5 per cent heavier on one side of the balanced cantilever.

References

1. Al-Qarra, H. (1999) 'The Dee Estuary Bridge – control of geometry during construction', *The Proceedings of the Institution of Civil Engineers, Civil Engineering*, **132**, February, 31–39.
2. Troitsky, M.S. (1988) *Cable Stayed Bridges, 2nd edition*. Oxford: BSP Professional Books, Blackwell Scientific Publications Ltd.
3. Walther, R., Houriet, B., Isler, W. and Moia, P. (1988) *Cable Stayed Bridges*, London: Thomas Telford Ltd.
4. Hay, J. (1992) 'Response of bridges to wind', *TRL State-of-the-art Review 5*. Norwich: HMSO.
5. Maguire, J.R. and Wyatt, T.A. (1999) *Dynamics, an introduction for civil and structural engineers*, ICE Design and Practice Guides. London: Thomas Telford, the Wind Engineering Society and SECED.

@Seismicisolation

17

Other prestressed concrete bridge types

Introduction

As well as the more common types of construction, presented in Chapters 9 to 16, prestressed concrete is used with several other forms of bridge deck arrangement as discussed in the following sections.

Extra-dosed bridges

The extra-dosed bridge is a variation on the cable-stayed arrangement, and it combines a stiff concrete deck with shallow cable stays anchored to a reduced height pylon. A typical arrangement is illustrated in Fig. 17.1(i). The pylon

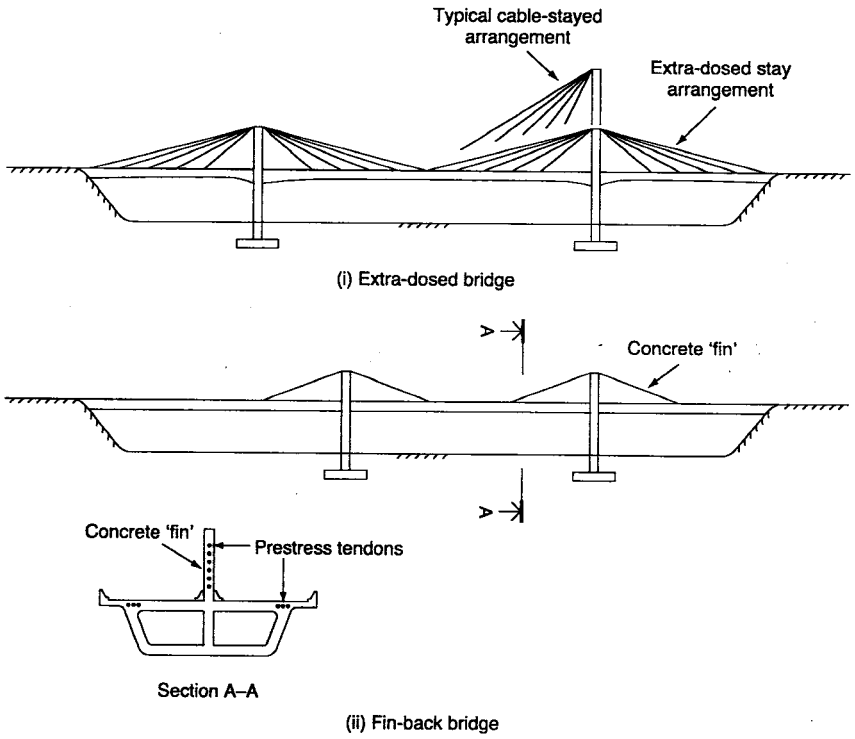


Figure 17.1. Extra-dosed and fin-back bridge arrangements.

@Seismicisolation

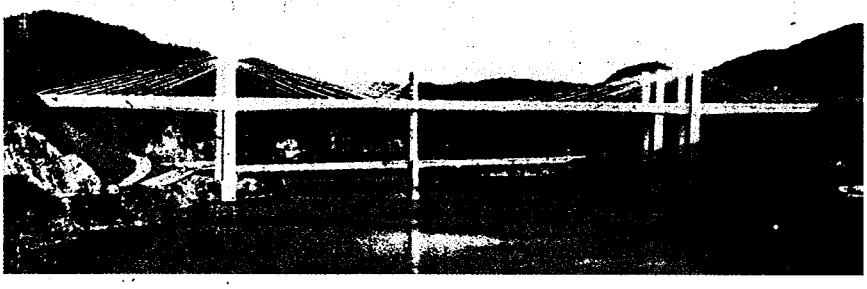


Figure 17.2. Tsukuhara Bridge, Japan. Reproduced courtesy of Freyssinet International. Copyright reserved.

height above the deck is approximately 10 per cent of the span length, compared to between 20 per cent and 25 per cent for a similar cable-stayed bridge. Extra-dosed bridges are constructed with either a concrete box or a beam-and-slab deck, and the result is an elegant and aesthetic structure. They are considered economic in the 100 m to 200 m span range.

The relative stiffness of the deck and shallow angle of the stays gives a structural behaviour more like an externally prestressed deck rather than a cable-stayed structure, and the design is often carried out on that basis.

A typical example of an extra-dosed bridge is the Tsukuhara Bridge in Japan, shown in Fig. 17.2. The twin-deck structure has a single-cell box girder on each side with the stays anchored along the edge of each box. The 323 m long three-span bridge has a main span length of 180 m and short pylons rising to 16 m above the deck level.

The Sunniberg Bridge, Switzerland, shown under construction in Fig. 17.3, utilized a slender beam-and-slab deck with spans of 140 m and a pylon height

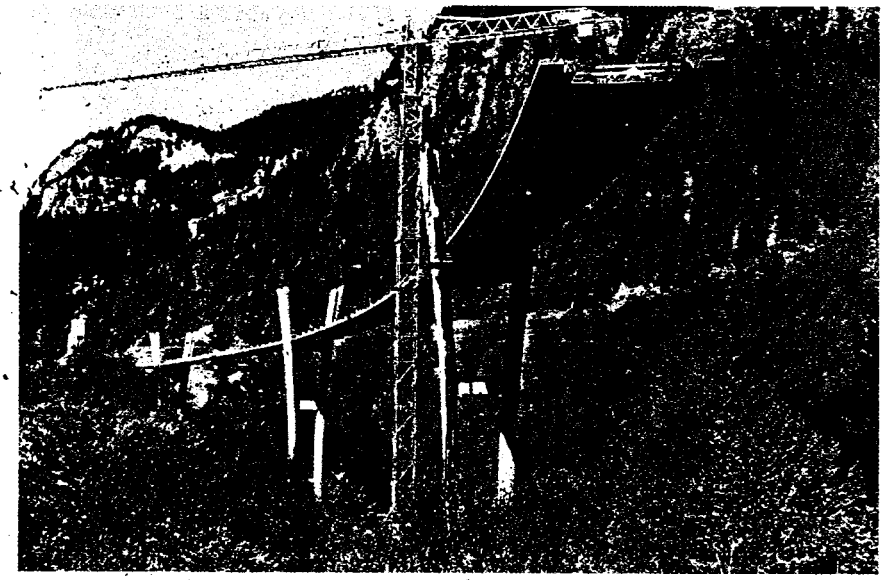


Figure 17.3. Sunniberg Bridge, Switzerland. Reproduced courtesy of BBR Systems Ltd. Copyright reserved.

@Seismicisolation

of 15 m above the deck. The concrete deck was constructed as a balanced cantilever built out from each of the pylons using form travellers, with the stays installed as the construction progressed.

The design and construction of extra-dosed bridges combine many of the features described in Chapters 12, 13 and 16.

Fin-back bridges

These are similar to the extra-dosed bridge, but with the stays replaced by tendons encased in a concrete wall or 'fin', as illustrated in Fig. 17.1(ii). The fin is an extension of the deck section and stiffens the deck over the pier while also providing protection to the tendons.

There are only a few examples where this type of structure has been used and they usually have a concrete box girder deck with the fin extending up above a central web. The Barton Creek Bridge, in Fig. 17.4, has a main span of 104 m and an overall length of 210 m. It was built using the *in situ* balanced cantilever technique with a constant deck cross-section throughout. A detailed description of the Barton Creek Bridge is given in the paper by Gee (1991) [1].

The design and construction of fin-back bridges are similar to the *in situ* or precast concrete box girders described in Chapters 12 and 13.

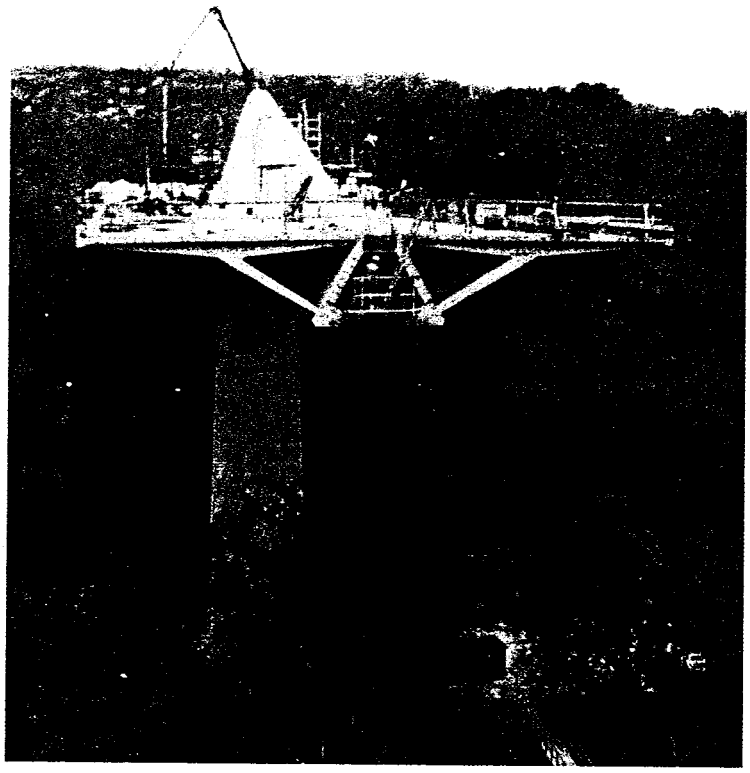


Figure 17.4. Barton Creek Bridge, USA. Reproduced courtesy of Tony Gee and Partners. Copyright reserved.

@Seismicisolation

Truss bridges

Trusses are often used with steel bridges, and they produce light and efficient structures. However, they are less common on concrete structures because of the complex design and construction of the joint between the diagonals of the truss and the deck slabs.

Several long-span precast segmental concrete viaducts and cable-stayed decks have utilized trusses for the webs to reduce dead weight, resulting in savings in quantities of prestress, stays and concrete.

Bubiyan Bridge, Kuwait, shown in Fig. 17.5, was constructed using precast segments with the top and bottom slabs and the diagonal members all in concrete. The 2·5 km long crossing has typical spans of 40 m with the deck continuous in 200 m or 240 m sections. Segments were match-cast using the long-line method described in Chapter 13. The diagonal members were cast first and lifted into the bed for the top and bottom slabs to be cast. Segment erection was span-by-span using an overhead gantry to position the segments. After placing a complete span of segments the deck was prestressed with external post-tensioned tendons.

In France, the Boulonnais Bridge, shown in Fig. 17.6, uses steel sections for the diagonal truss members. The deck was constructed using precast match-cast segments lifted into place by an overhead gantry and erected by the balanced cantilever technique. External tendons provide the longitudinal prestress in the deck.

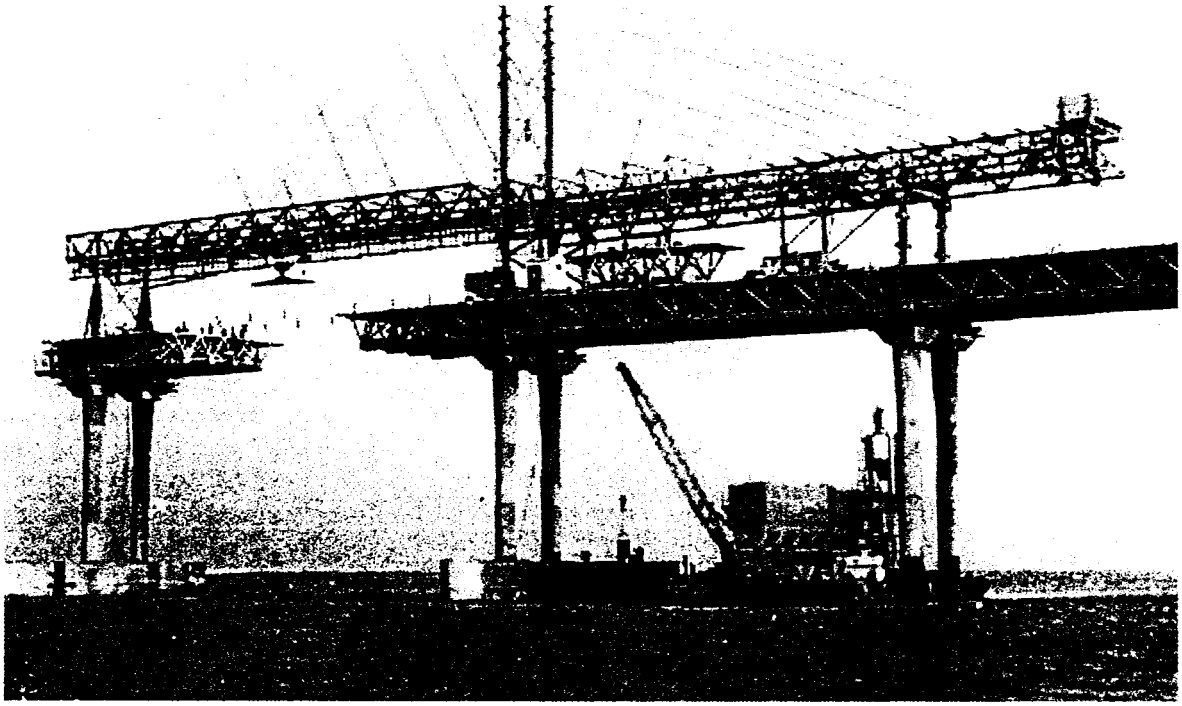


Figure 17.5. Bubiyan Bridge, Kuwait. Reproduced courtesy of Bouygues. Copyright reserved.

@Seismicisolation

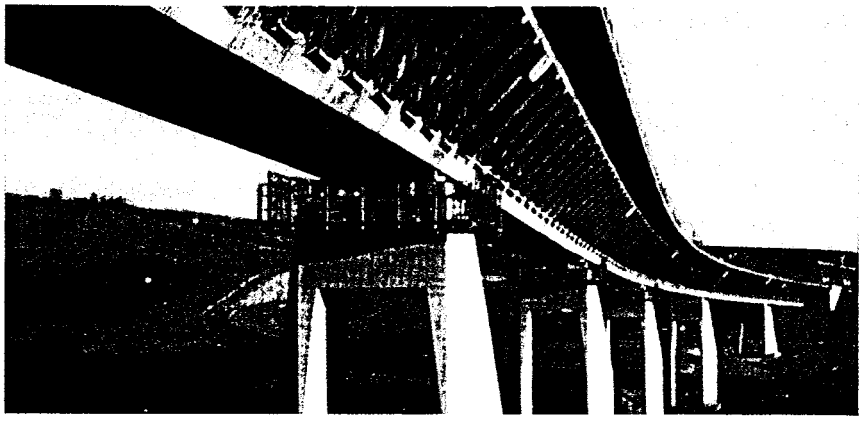


Figure 17.6. Boulonnais Bridge, France. Reproduced courtesy of VSL International. Copyright reserved.

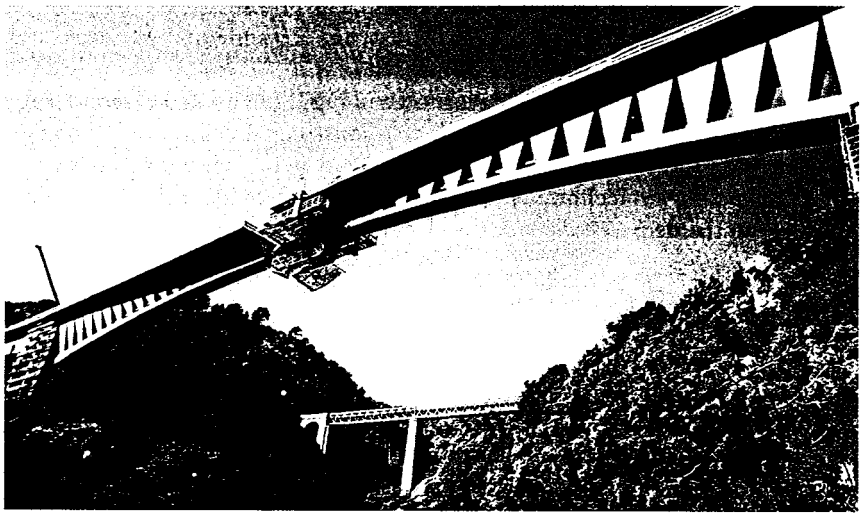


Figure 17.7. Vecchio Bridge, Corsica.

On early projects, where the diagonal truss members were plain reinforced concrete sections, cracking occurred in the concrete. The diagonals are subjected to high tensile forces and applying prestressing to the concrete helps to prevent this cracking from occurring.

The Vecchio Bridge in Corsica, shown in Fig. 17.7, is an innovative structure combining balanced cantilevering construction with an unusual truss deck arrangement. The triangular sections of web are efficient in transferring the shear forces and provide a striking appearance. It was designed by Berds Mikaelion, Lanslas Paulik, and Rázel Technic & Methods and constructed by Razel Technic & Methods using DYWIDAG prestressing systems.

Arch bridges

Arches are used to create long spans, with the arch either above or below the deck level. Arches extending above the deck provide support through hangers, and lighter steel or composite decks are usually preferred over

@Seismicisolation

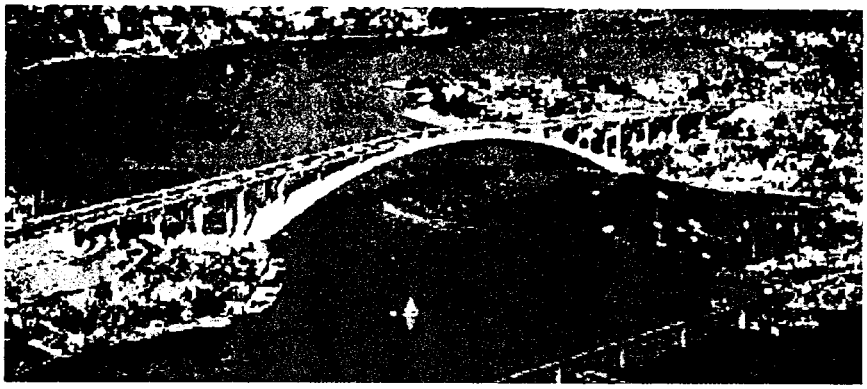


Figure 17.8. Gladesville Bridge, Australia. Reproduced courtesy of Maunsell Australia Pty Ltd. Copyright reserved.

heavier concrete arrangements. When the arch is below the deck level concrete decks provide a suitable solution.

The arch section is naturally in compression and seldom needs to be prestressed, while the deck often utilizes prestressed concrete elements. The deck is usually supported on columns resting on the arch and is similar in design and construction to a standard bridge deck with the same span lengths. *In situ* concrete slabs, *in situ* boxes, precast prestressed beams and launched concrete box girders are all used with an arch supporting arrangement.

The Gladesville Bridge, Australia, described by Baxter *et al.* (1965) [2], was built in 1964 and is shown in Fig. 17.8. It has a 305 m span reinforced concrete arch that supports a concrete deck formed using prestressed, precast concrete 'T' beams with spans of 30 m. The arch was formed from precast hollow box sections erected on falsework. The columns were precast and lifted into place, while an overhead gantry placed the precast beams.

Figure 17.9 shows the Barelang Bridge under construction in Indonesia. The reinforced concrete arch has a span of 245 m and is a hollow box section.

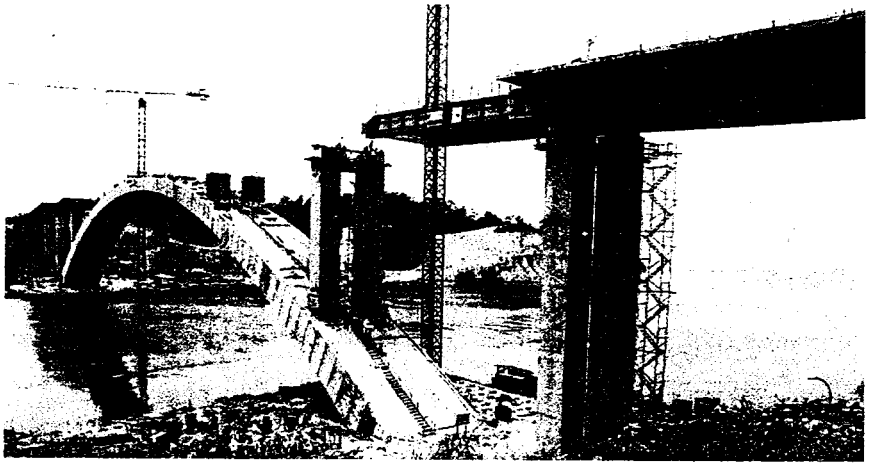


Figure 17.9. Barelang Bridge, Indonesia. Reproduced courtesy of Freyssinet International. Copyright reserved.

@Seismicisolation

Vertical columns extend up from the arch at 35 m centres to support the prestressed concrete box girder deck. The 385 m long deck was incrementally launched over the columns and into position using the techniques described in Chapter 15.

Footbridges

Footbridges are usually considered under a different category from the longer and heavier road and rail bridges, although the techniques used for their design and construction are similar. The lighter loading on footbridges plus the narrow widths result in lighter and more slender decks. The aesthetic and performance requirements are more to the forefront in footbridge design and result in some innovative and unusual arrangements.

A standard footbridge arrangement used throughout Singapore is shown in Fig. 17.10. The deck is either precast or cast *in situ* with the span length and arrangement varied to suit the location.

The footbridge in Fig. 17.11 is typical of the arrangement found in Hong Kong, with the walkway and canopy cast *in situ*. With spans up to 30 m, the 'U' shaped lower section of walkway is prestressed with post-tensioned tendons running the full length in each edge beam.

The Kilmacanoque stressed-ribbon footbridge shown under construction and when completed in Fig. 17.12 uses a very thin concrete deck supported by cables anchored on the abutments at either end. The shallow deck section follows the draped profile of the cables. Precast concrete sections were first hung from the cables, which were encased by an *in situ* concrete topping.

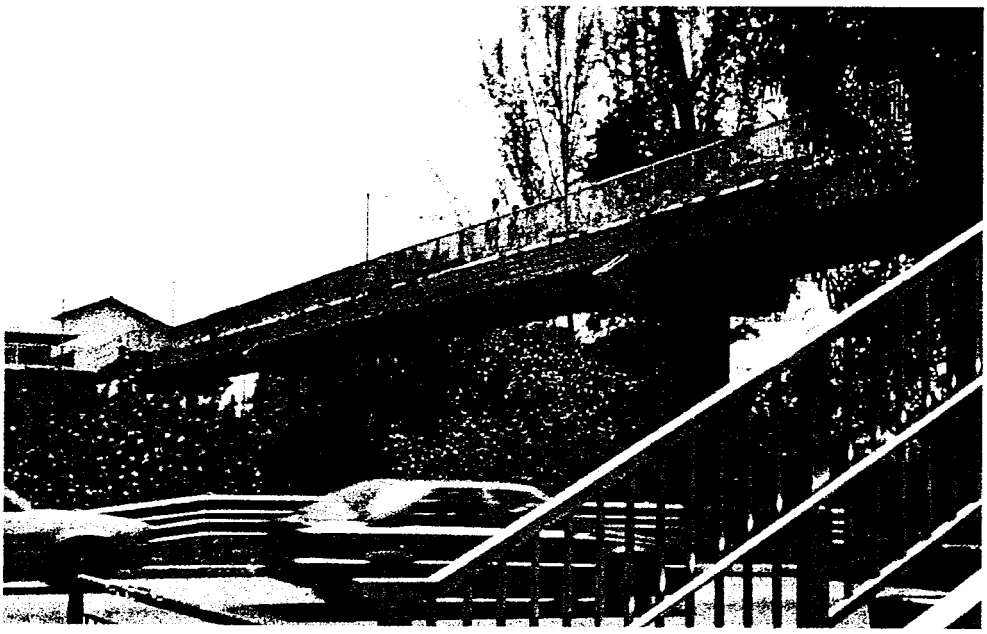


Figure 17.10. Footbridge, Singapore.

@Seismicisolation

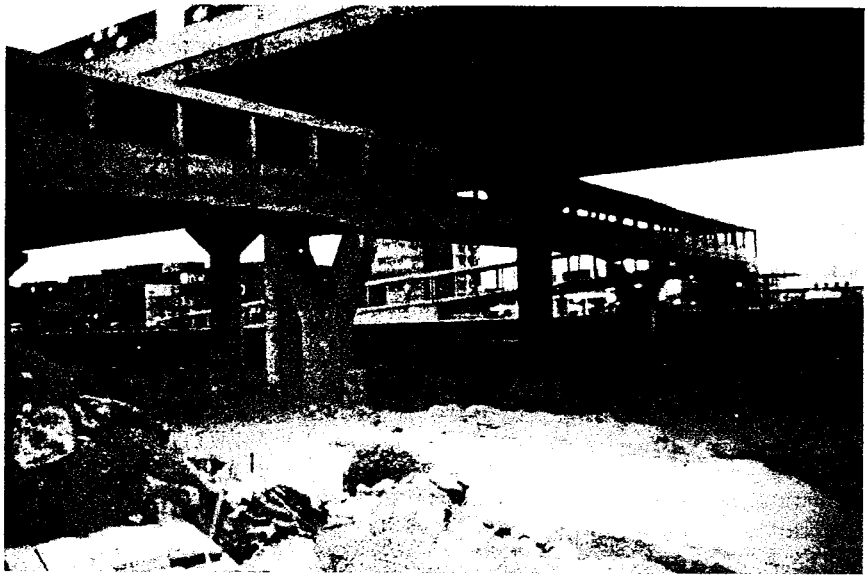


Figure 17.11. Footbridge, Hong Kong.

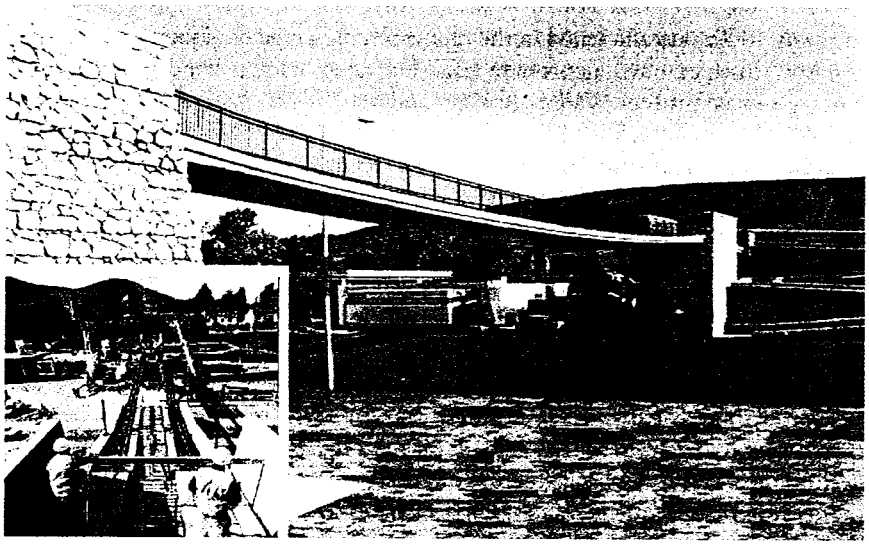


Figure 17.12. Kilmacanogue stressed-ribbon footbridge, Ireland. Reproduced courtesy of Roughan and O'Donovan, Dublin. Copyright reserved.

The lighter decks of footbridges make them well suited to precasting, either in full length or in sections, ready to be rapidly assembled on site.

References

1. Gee, A. (1991) 'Concrete fin-back bridge in USA', *The Proceedings of the Institution of Civil Engineers*, Part 1, **90**, February, 91–122.
2. Baxter, J.W., Gee, A.F. and James, H.B. (1965) *Gladesville Bridge*. The Proceedings of the Institution of Civil Engineers, **30**, 489–530.

@Seismicisolation

18

Problems and failures

Introduction

The design and construction of bridges present some of the greatest challenges for structural engineers and occasionally this results in problems occurring, ranging from minor cracking in the concrete to the collapse of a structure.

The more spectacular bridge failures, such as the Tay Bridge collapse in 1897, the Tacoma Narrows suspension bridge twisting itself apart in 1940, or the steel box collapses at Milford Haven, Yarra and Cologne in the early 1970s, are all etched in the memories of bridge engineers. Prestressed concrete bridges have also made headline news with collapses such as the Ynys-y-Gwas Bridge, Wales, in 1985 and the Injaka Bridge, South Africa, in 1998.

Other less dramatic problems occur more regularly and prestressed concrete bridges have suffered their own particular troubles in the past, some of which are discussed in the following sections. From these problems and failures has come a better understanding of the design and the construction requirements of concrete bridges.

Prestressing components

A critical stage for prestressing tendons and their components is during the stressing process when they are subjected to the largest forces in their design life. During lock-off the force in the tendon at the stressing anchorage reduces with subsequent reductions occurring over time due to the creep and shrinkage in the concrete, and relaxation in the tendon.

Wire, strand and tendon failures

Wires in a multi-strand tendon occasionally break during the stressing operation. This is normally identified by a sharp 'bang' heard during tensioning of the tendon. Individual wires break for a number of reasons such as damage occurring during installation or kinks in the duct causing stress concentrations. When the tendons are anchored the jaws of the wedges bite into the wires causing indents and, if the tendon is re-stressed, failures sometimes occur where the wires have been weakened.

Breakage of a single wire in a 7-wire strand, which is itself one of a number of strands making up a larger tendon, may not be significant, in which case no

@Seismicisolation

remedial action is required. By stressing the tendon to its specified jacking force the correct prestress is still applied to the structure with the other wires sharing the full load. If several wires in a tendon break then the remaining wires may not be capable of carrying the full load. In this case the tendon is removed and the damaged strands replaced.

Complete strands have occasionally broken during stressing. As well as a loud 'bang' being heard, the strand may 'jump' back out of the stressing jack when the breakage is close to the end of the tendon. This is often accompanied by a drop in pressure in the stressing jack as the overall force in the tendon reduces. If there is insufficient spare capacity in the remaining strands, or in the overall design, the tendon is de-stressed and the broken strand replaced.

Strands have slipped through the wedges, either at the dead-end anchorage during stressing or at the stressing end during lock-off of the tendon. This may be caused by incorrect seating of the wedges around the strand or by dirt or grease getting between the jaws and the wires.

Couplers with prestressing bars have failed when the threaded connection has not been fully engaged. This is avoided by carefully measuring the length of thread engaged and by ensuring that the connections are complete.

There have been cases of a prestressing anchor breaking or moving into the concrete during the jacking operation. This is often caused by honeycombing or deficient concrete behind the cast-in bearing plate or trumpet.

Tendon extensions

With multi-strand post-tensioned tendons, the extensions of the strands during stressing inevitably varies from that expected, with a range of ±5 per cent usually being considered acceptable. If the variation is outside this range it may indicate a problem along the tendon. Higher extensions may be due to lower friction than expected or it may be more serious if a strand has broken or slipped through the wedges at the dead-end.

Lower extensions usually indicate that the friction between the tendon and duct is greater than that allowed for in the design, or wobbles and kinks in the duct have increased the resistance as the tendon is pulled. The first option is usually to increase the stressing force to compensate for the extra losses, although this is normally limited to a maximum force in the tendon of 80 per cent UTS. Alternatively, the design is re-assessed for the lower prestress forces associated with the reduced extensions to determine if it is acceptable. If the design is not acceptable the tendon is usually de-stressed and re-stressed again, if necessary with soluble oil used to reduce the friction.

There is also a possibility that the tendon is snagged along its length, or concrete has got into the duct and prevented the complete length of tendon from being stressed, which would adversely affect the design of the deck. An indication of this is that the wedges at the dead-end anchorage do not fully pull in during the stressing. In this case the tendon is de-stressed, removed and re-installed to see if the blockage clears itself. If that fails to

@Seismicisolation

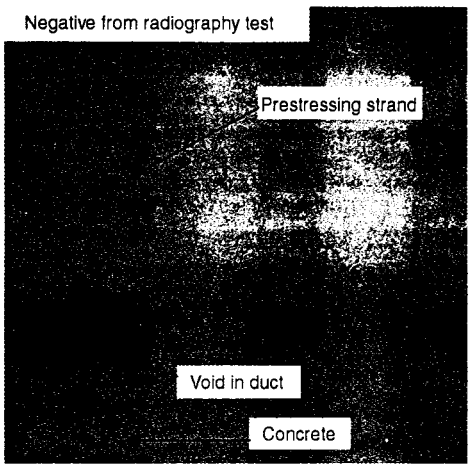


Figure 18.1. Incomplete grouting.

solve the problem, the blockage is located and cleared before the tendon is re-stressed.

Grouting and ducts

Grouting is another activity that has caused problems in the past, with voids being left in the ducts. Many existing post-tensioned concrete bridges have been inspected with a significant percentage found to have voids present, although only a small proportion of these showed signs of tendon corrosion. Figure 18.1 is a negative from a radiograph survey clearly showing up a void in a duct.

Voids are caused due to a number of reasons, including blockages in the duct, entrapped air or leaking of the grout. Guidelines on grouting are given in TR 47 (2002) [1] which should go some way to help eliminate this problem in the future.

On precast segmental decks there have been occasions when the pre-stressing ducts have 'crossed over' within a segment. When the tendon is threaded in at the anchorage of one duct it emerges at the anchorage of the other one. This does not normally cause too much of a problem as the number of tendons at each section is still maintained; however, it can be of concern if any one tendon becomes too long resulting in excessive friction losses along its length.

Corrosion

A number of bridges around the world have suffered from corrosion to the prestressing tendons or anchors well before the end of their design life. A survey carried out in the UK by the Highways Agency during the late 1990s revealed that 4 per cent of the bridges inspected had heavy to severe corrosion of the tendons with another 8 per cent having moderate corrosion present. The reasons for the corrosion range from poor detailing and specifications to deficiencies in construction and workmanship.

@Seismicisolation

The need for robust protection to the tendons is now established and, as the industry becomes more aware of past problems, incidents of corrosion should become less frequent.

Concrete and reinforcement

Problems relating to the concrete and reinforcement are commonplace although most are no more than relatively minor inconveniences that are simply dealt with on site.

Concrete cracks

Concrete cracks for many reasons, both design- and construction-related, and it is common for cracks to appear at some time or other in most concrete bridges. Reinforced concrete design assumes that cracks occur in order for the reinforcement to fully work, with crack widths of up to 0·3 mm accepted in most design codes.

Prestressed concrete bridges are usually designed with tensile stresses limited to a level where cracks should not occur. However, this applies only to the concrete that is prestressed, and other areas designed as reinforced concrete sections, such as the transverse members, diaphragms and anchorage areas, are liable to exhibit cracks. Despite this, it always seems to surprise engineers when cracks appear and steps are usually taken to seal them, such as injecting low viscosity resin into the cracks.

Cracks can occur due to early thermal effects within the fresh concrete and differential shrinkage between concrete cast at different times. These cracks are controlled by adopting a suitable concrete-mix design, good curing of the concrete and providing sufficient distribution reinforcement based on the approach given in BD 28/87 (1987) [2] and BA 24/87 (1989) [3].

Plastic settlement after placing of the concrete causes reflective cracking above the reinforcement which is prevented by using a suitable concrete mix design and by good compaction of the concrete during placing.

With post-tensioning, partially stressing some of the tendons early, when the concrete is still young, helps to prevent cracks from opening up. Most prestressing systems have anchorages designed to be loaded when the concrete has reached 25 N/mm^2 or 30 N/mm^2; however, this is based on the full load being applied. If the tendons are initially stressed to only 50 per cent or less of their design force it should be possible to stress them when the concrete is at a lower strength.

High stresses are generated around the tendon anchor block as the force spreads out into the concrete section. The local tensile stresses generated can cause cracking in the concrete. These cracks are controlled by reinforcement arranged to counter the tensile stresses. It is difficult to determine the precise stress pattern for arrangements with complex concrete or anchorage layouts and large cracks can occur.

Spalling of the concrete on the end-face of an anchorage looks unsightly, but it does not adversely affect the strength of the anchor block and needs repairing only to reinstate the protection to the reinforcement and anchor arrangement.

@Seismicisolation

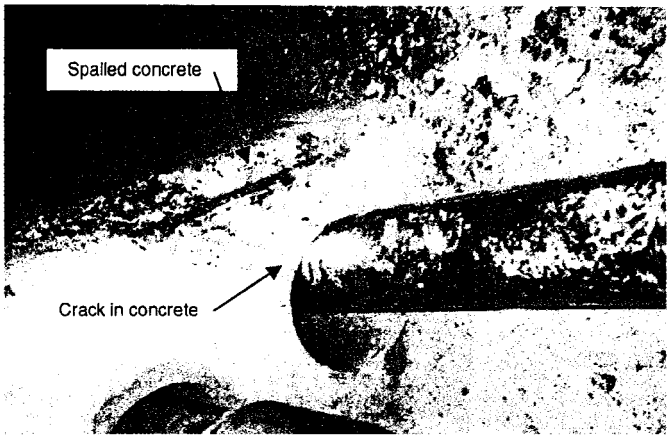


Figure 18.2. Spalling at deviator.

Small cracks in the bursting region of an end block are acceptable provided their width is within the acceptable design limit, while excessive crack widths may indicate that remedial measures are needed. Cracks that occur in the adjacent concrete, to the side and behind the anchorage area are assessed in a similar way.

External tendons impose a large concentrated force on to their anchor blocks and deviators. Cracking of the concrete is controlled by closely spaced reinforcement placed to counter the tensile stresses generated. Where the tendons or holes are misaligned this changes the distribution of the force along the deviator and, in the worst cases, causes kinks in the tendons as they emerge at the end of the deviator. This can give rise to cracks in the concrete or spalling of the edges, as seen in Fig. 18.2.

Several concrete box girders with external tendons have experienced failures at their anchor blocks and deviators. Built in the 1970s, anchor blocks on the A3/A31 bridge in the UK exhibited severe cracking during the construction and were strengthened to enable the bridge to be completed. Elsewhere there have been deviator failures with the tendon pulling out of the concrete, as seen in Fig. 18.3, requiring the deviators to be repaired and strengthened. This emphasizes the importance of ensuring that the anchor blocks and deviators fully secure the external tendons with the applied force evenly spread out.

Curved tendons have burst out of the concrete when running near the concrete surface. Adequate cover must always be provided to the prestressing ducts. Extra reinforcement must be placed, where the tendons are highly curved, to tie the tendon in to the main body of the concrete.

Honeycombing

Prestressed concrete bridges are usually highly stressed, and require high strength, dense concrete. Achieving well-compacted concrete is not always easy, as construction sites present many practical problems to be dealt with. Additionally, the tendons and reinforcement in post-tensioned concrete decks are often closely spaced, hindering the concreting operation. These factors can lead to honeycombing in the concrete, as seen in Fig. 18.4.

@Seismicisolation

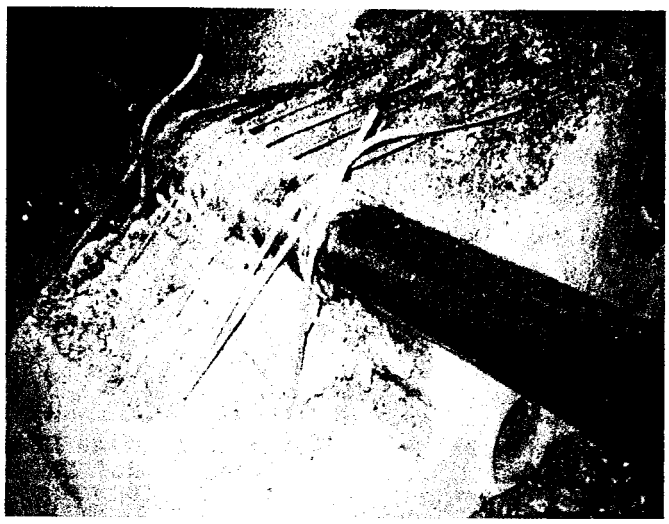


Figure 18.3. Deviator failure.

The anchor-block reinforcement, shown in Fig. 18.5, was made more congested by the designer doubling up on the bursting reinforcement, with both spirals and links detailed. When the rest of the web, diaphragm and top slab reinforcement were placed it left little room for the concrete.

Figure 18.4. Honeycombing around anchors.

@Seismicisolation

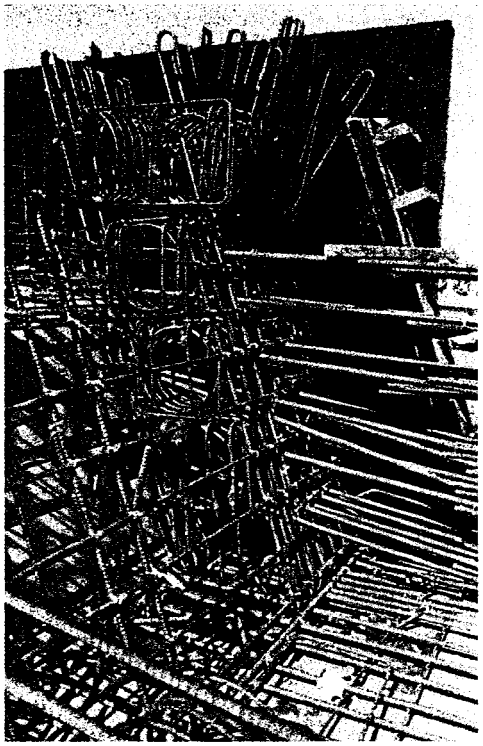


Figure 18.5. Congested reinforcement.

Although these regions are always heavily reinforced, careful selection of the concrete shape and the reinforcement detailing can ease construction and improve the concreting.

In some areas it is necessary to use smaller aggregate in the concrete, with super-plasticizers added to the mix to help the concrete flow around the reinforcement, ducts and anchors. Temporary windows should always be left in the shutters so that the concrete is seen to fill up the space and to allow vibrators to be inserted where necessary. It also helps to adjust the reinforcement locally to leave gaps for the concrete and vibrators to be inserted.

With proper planning and procedures it should always be possible to achieve a good dense concrete, although site conditions sometimes work against this and honeycombing and other concreting problems do occasionally occur. However, it was difficult to see why the honeycombing and missing concrete occurred in the web of the deck in Fig. 18.6.

In most cases the honeycombing is simply cleaned out and the voids filled up with fresh concrete; although there have been several bridge decks demolished and rebuilt because of severe defects.

Concrete cover

A common failing in the past has been inadequate concrete cover to the reinforcement and prestressing ducts. Construction tolerances in the formwork and in the bending and fixing of the reinforcement can result in the design cover not being achieved. This reduces the protection to the reinforcement

@Seismicisolation

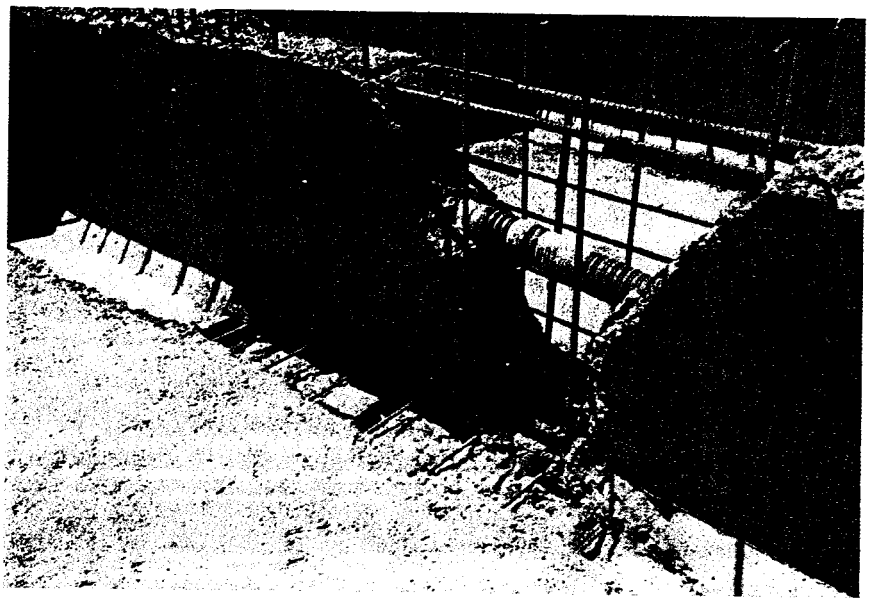


Figure 18.6. Honeycombing and voids in concrete.

and tendons and adversely affects the durability of the structure. The need for remedial works must be considered on a case-by-case basis. In the past these have included breaking out and recasting the concrete, spraying on additional concrete to increase the cover, applying a protective coating to the concrete or doing nothing if this is acceptable.

Problems during construction

Bridges are often at their most vulnerable during their construction, and prestressed concrete decks have experienced their share of failures and mishaps. Most failures are due to mistakes during the construction process or deficiencies in the design of the temporary or permanent works under the temporary loading. Accidents can happen at any time.

Failures due to design

A few concrete box girder bridges have experienced problems during the launching process. In 1988 the Mainbrücke Stockstadt viaduct in Germany, collapsed while being pushed out over the piers. Temporary stay cables were used to support the deck during the launch, a technique that had been used successfully before. As the deck moves forward it is difficult to control the forces in the stays and the deck, and the use of stays in this manner is seldom seen today.

The deck of the Injaka Bridge, South Africa, collapsed during its launch in 1998. Failure of the bottom slab above the temporary bearings and subsequent overstress of the launching nose and deck is thought to have resulted in the collapse.

Full-height falsework used with *in situ* concrete construction suffered a number of notable failures in the UK in the 1960s resulting in the publishing

@Seismicisolation

of the Bragg Report (1975) [4]. Deficiencies in design, lack of understanding of some imposed loading conditions, poor construction practices and inadequate founding of the falsework all contributed to the problems encountered. The investigations carried out during the drafting of the report led to more stringent design codes and construction guidance being developed.

These failures demonstrate the need to clearly understand the behaviour of concrete bridges and their interaction with the temporary works. The importance of the detailing, especially in highly stressed regions and of the design during each stage of the construction as well as in the permanent condition, must be recognized by the designer.

Construction procedures

Concrete bridges often require sophisticated construction techniques with the design and construction closely linked. Good communication between the design and construction teams is essential to minimize the risk of problems. The designer must understand the construction requirements and the contractor must appreciate the design requirements and limitations.

The construction must also be carefully controlled to prevent problems. In the early 1990s an *in situ* post-tensioned concrete multi-cell box girder deck came near to collapse when the falsework supporting it was released by the scaffolders before the prestress had been installed. Fortunately, there was sufficient reinforcement in the deck to hold it up and the cracks closed up again when the prestress was subsequently applied.

Less fortunate were the workmen and engineers on the Vasco da Gama Bridge, in Portugal, when the form traveller fell from the deck during construction of the cable-stayed bridge. The traveller was being moved forward and adjusted to pass the side pier when the accident happened.

Precast concrete elements require lifting, transporting and positioning, during which they run the risk of being dropped or damaged. Figure 18.7 shows an incident when a precast beam undergoing a two-crane lift was

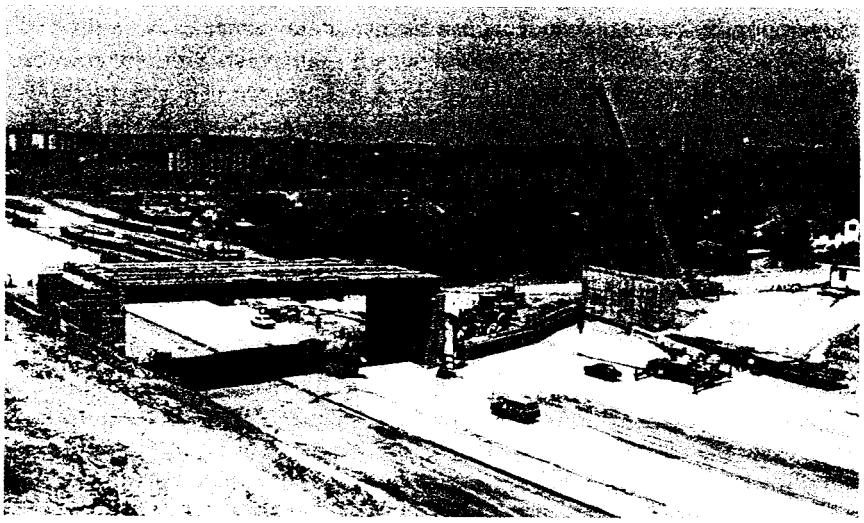


Figure 18.7. Precast beam dropped.

@Seismicisolation

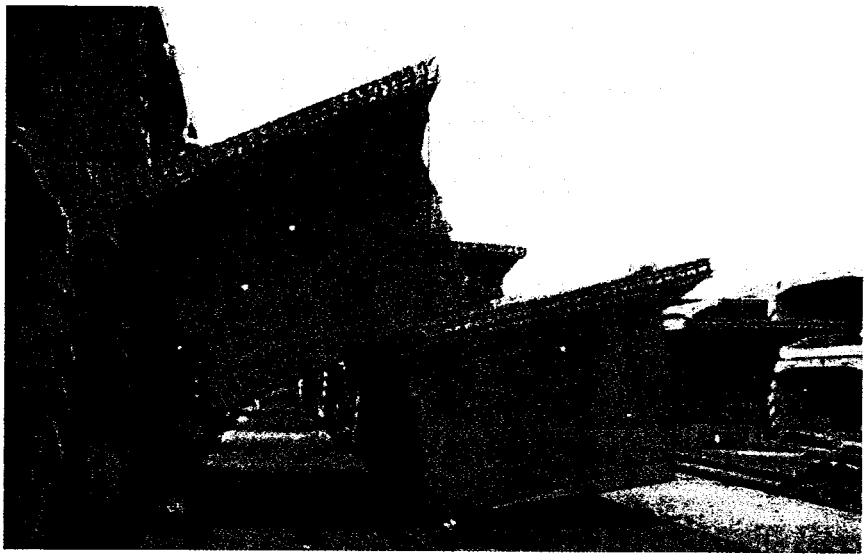


Figure 18.8. Segments toppling in storage yard.

dropped, due to one of the crane jibs failing. Two-crane lifts are always difficult and require careful control to ensure each crane properly carries the load.

Precast beams need supporting when placed on their bearings and before the deck slab is constructed. Although the bearings are usually horizontal it has been known for beams to slide off or topple over.

Precast box girder segments have been dropped during handling and when being transferred to the deck. Although not a common occurrence, several projects have suffered from the dropping of a segment, such as the A13 viaduct in Dagenham and the Second Severn Crossing approach viaducts.

Gantries used for erecting precast box girder segments or precast beams have encountered problems although collapses are rare. On the Second Severn Crossing approach viaducts a segment was dropped when it moved on to the wrong section of the gantry and caused the girder to fail. Gantries are complex structures with numerous mechanical and moving parts that need careful operation and regular inspections and maintenance.

Storing precast segments can give rise to problems, as seen from Fig. 18.8 where the segments toppled over in the storage area. As well as instability of the segments, they can also be damaged in their handling and by being hit from passing traffic.

Structural behaviour problems

Bridge decks do not always behave as expected and this is often first noticed during the construction phase. Deflections and movements of the deck are often different from the estimated design value, due to the many different factors that affect the construction and concrete parameters. Usually the differences are relatively small and do not adversely affect the design, although several bridges have suffered from significantly higher movements than expected which have caused problems and needed remedial works.

@Seismicisolation

The Long Keys Bridge in Florida was reported to have experienced higher longitudinal movements than expected which may have contributed to problems with the expansion joints and substructure. In the 1970s and 1980s several long-span balanced cantilever decks developed significant vertical sags over their spans, due to higher concrete creep than expected.

Under-estimating the prestress effects on curved bridges has resulted in several box girder decks lifting off one or more of their bearings. Post-tensioned tendons placed symmetrically in a continuous box girder deck, with a tight horizontal curvature, generate significant torsional effects that redistribute the bearing loads.

Problems after opening

Most pretressed concrete bridges perform well after opening, provided inspections and maintenance are carried out regularly.

Durability

The problem of corrosion and other durability issues are described in Chapter 3, with the collapse of the Ynys-y-Gwas Bridge, Wales, due to corrosion of the tendons, prompting improvements in design and construction procedures.

Another reported incidence of corrosion was on the Mid-Bay Bridge in Florida, where two of the external tendons were replaced in 2000. The 6 km long multi-span viaduct was constructed in 1993 with draped external tendons used to prestress the 41 m long box girder spans. The external tendons were encased in cement grout inside HDPE ducts. One tendon was completely de-stressed and hanging slack while the other tendon had ruptured strands protruding from the duct. It is thought that the existence of voids and grout bleed water had established an environment where corrosion occurred.

Rehabilitation and modifications

The Koror-Babelthaup Bridge, on the Pacific Islands, collapsed soon after strengthening in 1996. Originally built in the late 1970s, the *in situ* concrete box girder deck had a 241 m long main span with a hinge joint at mid span. Designed as a pair of cantilevers, the main span experienced excessive deflections and in 1996 the deck was made continuous with additional external tendons installed under the top slab along the complete length of deck. The modifications were successfully completed, but after several months the deck collapsed at a time when very little live load was present. The reasons for the collapse have not been published, but it would appear that the change to the structural system, plus the additional tendons caused an overstress in the deck adjacent to the piers.

References

1. Concrete Society (2002) *Technical Report No. 47. Durable Bonded Post-Tensioned Concrete Bridges, 2nd edition*. Slough: The Concrete Society.

@Seismicisolation

2. Highways Agency (1987) *(MDRB) Departmental Standard, BD 28/87, Early Thermal Cracking of Concrete*. Norwich: HMSO.
3. Highways Agency (1989) *(MDRB) Departmental Standard, BA 24/87, Early Thermal Cracking of Concrete (including amendment No. 1 (1989))*. Norwich: HMSO.
4. Bragg, S.L., Ahm, P., Bowen, F.M., Champion, S., Kemp, L.C., Mott, J.C.S., Wilshere, C.J., Greville, W.E.J. and Hume, I.J. (1975) *Final Report of the Advisory Committee on Falsework*. London: HMSO.

@Seismicisolation

Appendix A: Definitions

Anchor	A mechanical element placed at the end of post-tensioned tendons to hold the tendon and transfer the load from the tendon to the concrete.
Bar	Round drawn-out steel element, typically 15 mm to 75 mm diameter, for prestressing.
Blister	A concrete block cast on the side of the deck to anchor the tendons against.
Bonded tendon	Where the tendon is bonded to the concrete member by being cast-in, as for pre-tensioning, or through concrete grout for post-tensioning.
Box girder	A deck arrangement with a box cross-section, comprising a top slab, webs and a bottom slab.
Cable	A number of strands placed together to form a stressed element.
Cable-stayed	An arrangement where the deck is supported along its length by stays fixed to a pylon.
Concordant profile	The profile and layout of the prestressing tendons giving no secondary moments.
Coupler	A mechanical device for connecting tendons together to make a longer element.
Creep	The change in strain within the concrete over time under a constant applied stress.
Deviator	A concrete or steel support, for external tendons, which is fixed to the deck.
Diaphragm	A concrete wall placed across the deck beneath the top slab to provide rigidity to the structure and/or transfer the loads in the webs to the bearings.
Duct	A steel, plastic or high-density polyethylene (HDPE) tube into which tendons are placed.
External tendon	Where the tendon is placed outside the concrete section.
Extra-dosed	Similar to a cable-stayed bridge with a shallow angle of stays and a short pylon.
Fin-back	Similar to an extra-dosed arrangement, but the stays are encased in a concrete wall extending up from the deck.

@Seismicisolation

Grout	A cement–water mixture or grease compound used to fill the void around the tendon within a duct.
Incrementally launched	A technique of casting the deck in short sections at one end of the structure and launching the deck into position in stages.
In situ concrete	Concrete cast directly into its final position.
Internal tendon	Where the tendon is placed within the concrete section.
Post-tensioning	Where the tendon is stressed after the concrete has hardened, and with load transfer during the stressing operation.
Precast concrete	Concrete cast away from its permanent position, cured and moved into place.
Prestressing	Applying a stress to the concrete by pre-tensioning or post-tensioning.
Pre-tensioning	Where the tendon is stressed before placing the concrete, and with load transfer occurring after the concrete has set.
Primary moment	The direct moment applied at a section due to the prestress tendon, equal to the prestress force multiplied by the offset from the N–A.
Psuedo-box	A deck arrangement achieved by placing precast 'I' or 'M' beams side-by-side and connecting their top and bottom flanges with an *in situ* slab to form a multi-box structure.
Relaxation	The reduction in stress in the tendon over time under a constant strain.
Secondary moment	The moment set up by the applied prestress in a statically indeterminate structure in addition to the primary moments.
Segmental	A technique of precasting the deck in short segments and assembling on site.
Shrinkage	The change in length of the concrete over time as the concrete matures.
Strand	Normally 7-wire strand, typically 13 mm or 15 mm diameter, for prestressing; made up of six wires twisted around a central straight wire.
Tendon	A wire, strand, bar or bundle of strands used to prestress the concrete.
Thixotropic grout	Special grouting material which is fluid when shaken but becomes gelled when standing.
Unbonded tendon	Where the tendon is not connected to the concrete other than at the anchor positions, allowing the tendons and concrete to act independently along the tendon length.
Wire	Round drawn-out elements of steel, typically 5 mm to 7 mm diameter, for prestressing.

@Seismicisolation

Appendix B: Symbols and notations used

AASHTO	American Association of State and Highway Transportation Officials
A or A^1	Area of reinforcement to be provided
A_c	Area of concrete section
A_L	Area of concrete considered for longitudinal shear check
A_o	Area enclosed by median wall lines around the box
A_p	Area of non-prestressed reinforcement
A_r	Area of reinforcement across failure plane
A_s	Area of slab
A_{sk}	Area of shear key
A_{sl}	Area of longitudinal reinforcement
A_{st}	Area of leg of link around section
A_{stay}	Area of stay cable
A_{sv}	Area of shear reinforcement
A_t	Cross sectional area of tendon
b	Breadth of member or web
b_e	Minimum effective width of element resisting applied torsion
b_s	Width of slab
b_v	Width of section subjected to horizontal shear
C	Compressive force generated in concrete at ultimate moment capacity
c	Coefficient used in equation for relaxation of tendon steel
d	Distance from tendons to compression face
d_c	Depth of compression in concrete at ultimate moment capacity
d_t	Distance from reinforcement to compression face
d_x	Increment of length of tendon
d_1	Larger dimension from line of action of anchor force to the boundary on non-symmetrical prism
d_2	Smaller dimension from line of action of anchor force to the boundary on non-symmetrical prism
D_t	Nominal diameter of tendon
E	Modulus of elasticity for the material considered
E_c	28-day secant modulus of elasticity of concrete
E_i	Equivalent modulus of elasticity of stay cable
E_s	Modulus of elasticity of stay cable
E_t	Modulus of elasticity of tendon
f_c	Stress in the concrete at point considered
f'_c	28-day cylinder strength of concrete
f_{cb}	Stress in concrete at bottom of section
f'_{ci}	Cylinder strength of concrete at age being considered

@Seismicisolation

f_{ci}	Cube strength of concrete at age being considered
f_{cp}	Compressive stress at centroid of section due to prestress
f_{ct}	Stress in concrete at top of section
f_{cu}	28-day cube strength of concrete
f_d	Stress due to unfactored dead load at tensile face of section subject to M_{cr}
f_e	Initial stress in non-prestressed reinforcement
f_p	Initial stress in tendon
f_{pt}	Stress in tendon after time, t (hours)
f_{pc}	Stress due to prestress only at the centroid of the tendons
f_{pe}	Effective prestress after all losses
f_{pi}	Stress increase in tendon
f_{pk}	Average compressive stress over shear keys
f_{pt}	Stress due to prestress only at the tensile face
f_{pu} or f_s'	Characteristic strength of tendon
f_s	Initial stress in non-prestressed reinforcement
f_{si}	Increase in stress in non-prestressed reinforcement
f_{su}^*	Average stress in tendon at ultimate load
f_t	Tensile strength of concrete taken as $0{\cdot}24\sqrt{f_{cu}}$
f_y	Characteristic strength of reinforcement
f_y^*	Yield strength of tendon taken as: $0{\cdot}9f_s'$ for low-relaxation strand $0{\cdot}85f_s'$ for stress-relieved strand $0{\cdot}85f_s'$ for type I (smooth) bar $0{\cdot}80f_s'$ for type II (deformed) bar
f_{yl}	Characteristic strength of longitudinal reinforcement
f_{yv}	Characteristic strength of link reinforcement
F	Force in tendon at point being considered
F_0	Force applied to tendon by jack
F_{bst}	Anchor bursting force
F_o	Force applied by the jack at the anchor
F_s	Force generated in top slab due to differential shrinkage
h	Overall depth of member
h_{max}	Larger dimension of the section
h_{min}	Smaller dimension of the section
h_{wo}	Web or slab thickness
HDPE	High density polyethylene
H_{stay}	Horizontal projected length of stay cable
I	Second moment of area of section
k	Wobble coefficient
k_1	Longitudinal shear coefficient
k_i	Concrete bond coefficient
k_f	Coefficient for deck frequency
k_t	Coefficient dependent on type of tendon
K	Factor used in AASHTO for torsion capacity, based on concrete strength and stress present
K_s	Equivalent spring stiffness for stay support
l	Lever arm at ultimate moment
l_c	Losses of stress in tendon due to creep of concrete
l_E	Losses of stress in tendon due to elastic shortening of concrete
l_f	Stress in tendon at point being considered expressed as amount below the 70 percent UTS level
l_r	Losses of stress in tendon due to relaxation
l_s	Losses of stress in tendon due to shrinkage of concrete

@Seismicisolation

L_{stay}	Inclined length of stay
l_t	Transmission length for anchorage of pre-tensioned strand
L	Length of deck
L_c	Effective flange width
L_s	Width of longitudinal shear failure plane
L_{sp}	Length of span of deck
L_T	Free length of tendon
M	Moment at section due to ultimate loads
M_a	Moment generated by the change applied 'instantaneously'
$M_{as\text{-}built}$	Moment as constructed
M_{cr}	Cracking moment of section
M_{final}	Final moment after creep effects
M_{inst}	Moment if the structure is built instantaneously
M_o	Moment necessary to produce zero stress in the concrete at the tensile face
M_p	Primary moment from prestress on section
M_r	Ultimate moment of resistance at a section
M_s	Prestress secondary moment
m_t	Mass per metre run of tendon
m_d	Mass per metre run of deck
m_1	Moment due to unit restraint moment applied at pier 1
m_2	Moment due to unit restraint moment applied at pier 2
	N–A
	Neutral axis of section
P	Total unfactored prestress force acting on section
P_h	Horizontal force from prestress tendon
P_o	Force applied to each strand by jack
P_R	Radial force from tendon on concrete
P_v	Vertical force from prestress tendon
p	Load applied to a structural member
p	Ratio of reinforcement, A_r/bh
R	Radius of curved tendon
S_v	Spacing of link reinforcement
S_L	Spacing of longitudinal reinforcement
S_t	Spacing of transverse reinforcement
T	Tensile force generated at ultimate moment
T_c	Torsional cracking moment
T_s	Torsional moment due to serviceability loads
T_{stay}	Force in stay
T_u	Torsional moment due to ultimate loads
t	Time since prestress applied or age of concrete as appropriate
UTS	Ultimate tensile strength
v	Shear stress in the concrete due to ultimate loads
v_c	Ultimate shear stress allowed in concrete
v_L	Ultimate longitudinal shear stress in the concrete
v_t	Torsional shear stress
V	Shear force due to ultimate loads
V_c	Ultimate shear resistance of concrete at section
V_{cr} or V_{ci}	Ultimate shear resistance of concrete cracked in flexure
V_{co} or V_{cw}	Ultimate shear resistance of concrete uncracked in flexure
V_k	Ultimate shear resistance of shear key
V_{nh}	Nominal horizontal shear strength
V_s	Ultimate shear resistance provided by the reinforcement

@Seismicisolation

V_d	Shear force at section due to unfactored dead load
V_L	Longitudinal shear force per unit length
V_p	Vertical component of prestress
v_{tmin}	Minimum torsional shear stress, above which reinforcement should be provided
V_u	Factored shear force at section
X_1	Secondary moment at support 1
X_2	Secondary moment at support 2
x	Distance of point being considered from the tendon anchor
x_i	Smaller centreline dimension of torsion link
y or y'	Distance in section from N–A to point being considered
y_i	Larger centreline dimension of torsion link
y_o	Half length of side of anchor block
y_{po}	Half length of side of loaded area
y_t	Distance in section from N–A to tensile face
z_t	Elastic sectional modulus referred to top face (I/y)
z_b	Elastic sectional modulus referred to bottom face (I/y)
δ	Deflection of a structural member under load
γ_{fL}	Partial load factor
γ_L	Load factor
γ_m	Partial safety factor for strength
γ_{f3}	Analysis factor
γ_s	Density of stay (kg/m^3) including sheathing
θ	Total angle change in the tendon over distance x in radians
μ	Friction co-efficient
ρ	Ratio of the reinforcement crossing the plane. Equal to $A_s/(bh)$
σ	Stress in concrete
σ_b	Stress in concrete at bottom of section
σ_{bb}	Stress in concrete at bottom of beam
σ_{bt}	Stress in concrete at top of beam
σ_c	Stress in concrete adjacent to prestress tendon
σ_{ci}	Stress in concrete adjacent to prestress tendon at time of transfer
σ_{cd}	Stress in concrete adjacent to prestress tendon due to change in dead load since tendon installed
σ_{final}	Final stresses in section
$\sigma_{as\text{-}built}$	Stress in section due to construction sequence
σ_{inst}	Stress in section if built instantaneously
σ_{st}	Stress in concrete at top of slab
σ_t	Stress in concrete at top of section
ϕ	Creep factor or AASHTO strength reduction factor
Δ_L	Change in deck length
Δ_t	Change in effective temperature
Δ_{cs}	Shrinkage strain deformation of the concrete
Δ_s	Differential shrinkage stress between *in situ* slab and precast beam
ξ_s	Depth factor for shear, given in Table 9 of BS 5400 part 4
α	Coefficient of thermal expansion of the concrete per °C
γ_m	Partial safety factor for strength
ε_{ct}	Strain in concrete at top fibre
ε_{cb}	Strain in concrete at bottom fibre
ε_p	Strain in prestress tendon
ε_s	Strain in non-prestressed reinforcement

@Seismicisolation

Appendix C: Further reading and useful references

Introduction

The following references are intended as a guide for further reading as well as references for the preceding chapters. Also included is a list of website addresses which can provide further information and details of the prestressing systems and other equipment used on prestressed concrete bridges.

Books

1. Abeles, P.W. (1949) *The Principles and Practice of Prestressed Concrete*. London: Crosby Lockwood.
2. Clark, L.A. (1983) *The Concrete Bridge Design to BS 5400*. Harlow: Construction Press.
3. England, G.L., Tsang, N.C.M. and Bush, D.I. (2000) *Integral Bridges*. London: Thomas Telford.
4. Guyun, Y. (1951) *Prestress Concrete*. Paris: Edition Eyrolles.
5. Hambly, E.C. (1991) *Bridge Deck Analysis, 2nd edition*. London: Chapman & Hall.
6. Highways Agency (1996) *The Appearance of Bridges and Other Highway Structures*. London: HMSO.
7. Kong, F.K. and Evans, R.H. (1987) *Reinforced and Prestressed Concrete, 3rd edition*. Wokingham: Van Nostrand.
8. Lee, D.J. (1994) *Bridge Bearings and Expansion Joints, 2nd edition*. London: E&FN Spon.
9. Magnel, G. (1950) *Prestressed Concrete*. London: Concrete Publications.
10. Neville, A.M. (1995) *Properties of Concrete, 4th edition*. London: Longman.
11. Pennells, E. (1978) *Concrete Bridge Designers Manual, 1st edition*. London: Cement and Concrete Association.
12. Podolny, W. and Muller, J.M. (1982) *Construction and Design of Prestressed Concrete Segmental Bridges*. New York: John Wiley.
13. Prichard, B. (1992) *Bridge Design for Economy and Durability*. London: Thomas Telford.
14. Prichard, B. (ed.) (1994) *Continuous and Integral Bridges, 1st edition*. London: E&FN Spon.
15. Pucher, A. (1976) *Influence Surfaces of Elastic Plates, 5th edition*. Wien, Austria: Springer-Verlag.
16. Ramberger, G. (2002) *Structural Bearings and Expansion Joints for Bridges*. Zurich: International Association for Bridge and Structural Engineering.
17. Rosignoli, M. (2002) *Bridge Launching*. London: Thomas Telford.

@Seismicisolation

18. Ryall, M.J., Parke, G.A.R. and Harding, J.E. (eds) (2000) *Manual of Bridge Engineering*. London: Thomas Telford.
19. Schlaich, J. and Scheef, H. (1982) *Concrete Box-girder Bridges*. Zurich: International Association for Bridge and Structural Engineering.
20. Sutherland, J., Humm, D. and Chrimes, M. (2001) *Historic Concrete – The Background to Appraisal*. London: Thomas Telford.
21. Troitsky, M.S. (1988) *Cable Stayed Bridges, 2nd edition*. Oxford: BSP Professional Books, Blackwell Scientific.
22. Walther, R., Houriet, B., Isler, W. and Moia, P. (1988) *Cable Stayed Bridges*. London: Thomas Telford.

Design guides and technical reports

1. Bragg, S.L., Ahm, P., Bowen, F.M., Champion, S., Kemp, L.C., Mott, J.C.S., Wilshere, C.J., Greville, W.E.J. and Hume, I.J. (1975) *Final Report of the Advisory Committee on Falsework*. London: HMSO.
2. Comité Euro-International du Beton (1992) *Durable Concrete Structures, 2nd edition*. London: Thomas Telford.
3. Concrete Society (2002) *Technical Report No. 47, Durable Bonded Post-tensioned Concrete Bridges, 2nd edition*. Slough: Concrete Society.
4. Concrete Society and IStructE (1971) *Technical Report TRCS 4, Falsework*. London: Concrete Society.
5. CIRIA (1976) *Guide No. 1, A Guide to the Design of Anchor Blocks for Post-tensioned Prestressed Concrete Members*. London: CIRIA.
6. CIRIA (1977) *Guide No. 2, The Design of Deep Beams in Reinforced Concrete*. London: CIRIA.
7. CIRIA (2001) *Bridge Detailing Guide*. London: CIRIA.
8. CIRIA (1985) *Report No. 106, Post-tensioning Systems for Concrete in the UK: 1940–1985*. London: CIRIA.
9. CIRIA (1996) *Report No. 155, Bridges – Design for Improved Buildability*. London: CIRIA.
10. Fédération International de la Préconstraint (FIP) (1990) *Guide to Good Practice, Grouting of Tendons in Prestressed Concrete*. London: Thomas Telford.
11. Hay, J. (1992) *Response of Bridges to Wind*. TRL State-of-the-Art Review, 5. Norwich: HMSO.
12. Highways Agency and TRL (1999) *Post-tensioned Concrete Bridges Anglo-French Liaison Report*. London: Thomas Telford.
13. Koseki, K. and Breen, J.E. (1983) *Exploratory Study of Shear Strength of Joints for Precast Segmental Bridges*. Centre for Transportation Research, The University of Texas at Austin, Research Report 248-1, September.
14. MacGregor, R.J.G., Kreger, M.E. and Breen, J.E. (1989) *Strength and Ductility of a Three Span Externally Post-tensioned Segmental Box Girder Bridge Model*. Centre for Transportation Research, The University of Texas at Austin. Research Report 365-3F, January.
15. Maguire, J.R. and Wyatt, T.A. (1999) *Dynamics, an Introduction for Civil and Structural Engineers*. ICE Design and Practice Guides. London: Thomas Telford, Wind Engineering Society and SECED.
16. Maisel, B.I. and Roll, F. (1974) *Methods of Analysis and Design of Concrete Box Beams with Side Cantilevers*. C&CA Technical Report, London.
17. Nicholson, B.A. (1997) *Simple Bridge Design Using Prestressed Beams*. Leicester: Prestressed Concrete Association.

@Seismicisolation

18. Powell, L.C., Breen, J.E. and Kreger, M.E. (1988) *State of the Art Externally Post-tensioned Bridges with Deviators*. Centre for Transportation Research, The University of Texas at Austin. Research Report 365-1, June.
19. TRRL (1977) *Laboratory Report 765, Temperature Difference in Bridges: Basis of Design Requirements*. Crowthorne, Transport and Road Research Laboratory.
20. West, R. (1973) *Recommendations on the Use of Grillage Analysis for Slab and pseudo-slab Bridge Decks*. London: C&CA/CIRIA.

Articles

1. Aalami, B.O. (1990) 'Load balancing: a comprehensive solution to post-tensioning', *ACI Structural Journal*, November–December, 662–70.
2. Al-Qarra, H. (1999) 'The Dee Estuary Bridge – control of geometry during construction', *The Proceedings of the Institution of Civil Engineers, Civil Engineering*, **132**, February, 31–9.
3. Baxter, J.W., Gee, A.F. and James, H.B. (1965) 'Gladesville Bridges', *The Proceedings of the Institution of Civil Engineers*, **30**, 489–530.
4. Beaney, N.J. and Martin, J.M. (1993) 'Design and construction of the Dornock Firth Bridge: construction', *The Proceedings of the Institution of Civil Engineers, Transp.*, **100**, August, 145–56.
5. Brockman, C. and Rogenhofer, H. (2000) 'Bang Na Expressway, Bangkok, Thailand – world's longest bridge and largest precasting operation', *PCI Journal*, January–February, 26–38.
6. Burgoyne, C.J. and Stratford, T.J. (2001) 'Lateral instability of long-span prestressed concrete beams on flexible bearings', *The Structural Engineer*, **79:6**, March, 23–6.
7. Buyukozturk, O., Bakhoum, M. and Beattie, S.M. (1989) 'Shear behaviour and strength of joints in precast concrete segmental bridges'. Paper submitted to the *Journal of Structural Engineering of the American Society of Civil Engineers*, January.
8. Catchick, B.K. (1978) 'Prestress analysis for continuous beams: some developments in the equivalent load method', *The Structural Engineer*, **2:56B**, June, 29–36.
9. Clark, L.A. (1984) 'Longitudinal shear reinforcement on beams', *Concrete*, **18:2**, February, 22–3.
10. Garrett, R.J. and Cochrane, R.A. (1970) 'The analysis of prestressed concrete beams curved in plan with torsional restraints at the supports', *The Structural Engineer*, **3**, March, 128–32.
11. Gallaway, T.M. (1980) 'Design Features and prestressing aspects of Long Key Bridge', *PCI Journal*, **25:6**, November–December, 84–96.
12. Gee, A. (1991) 'Concrete fin-back bridge in USA', *The Proceedings of the Institution of Civil Engineers*, Part 1, **90**, February, 91–122.
13. Hewson, N.R. (1992) 'The use of dry joints between precast segments for bridge decks', *Proceedings of the Institution of Civil Engineering*, **92**, November, 177–84.
14. Hewson, N.R. (1993) 'The use of external tendons for the Bangkok Second Stage Expressway', *The Structural Engineer*, **71**, December, 412–15.
15. Inversen, N., Faulds, J.R. and Rowley, F. (1993) 'Design and Construction of the Dornock Firth Bridge: design', *The Proceedings of the Institution of Civil Engineers, Transp.*, **100**, August, 133–44.
16. Lewis, C.D., Robertson, A.I. and Fletcher, M.S. (1983) 'Orwell Bridge-design', *The Proceedings of the Institution of Civil Engineers, Part 1*, **74**, November, 765–78.
17. Lopes, S.M.R. and Do Carmo, R.N.F. (2002) 'Bond of prestressed strands to concrete: transfer rate and relationship between transmission length and tendon draw-in', *Structural Concrete*, Vol. 3, **3**, 117–26.

@Seismicisolation

18. Magura, D.D., Sozen, M.A. and Siess, C.P. (1964) 'A study of stress relaxation in prestressing reinforcement', *PCI Journal*, **9:2**, April.
19. Moreton, A.J. (1990) 'Segmental bridge construction in Florida: a review and perspective', *Proceedings of the Institution of Civil Engineers, Part 1*, **88**, June, 381–419.
20. Muller, J. (1980) 'Construction of Long Key Bridge', *PCI Journal*, **25:6**, November–December, 97–111.
21. Podolny, W. and Mireles, A.A. (1983) 'Kuwait Bubiyan Bridge – a 3-D precast segmental space frame', *PCI Journal*, January–February, 68–107.
22. Rabbat, B.G. and Sowlat, K. (1987) 'Testing of segmental concrete girders with external tendons', *PCI Journal*, March–April, 86–107.
23. Rawlinson, J. and Stott, P.F. (1962) 'The Hammersmith Flyover', *The Proceedings of the Institution of Civil Engineers*, **23**, December, 565–624.
24. Rowley, F. (1993) 'Incremental launch bridges: UK practice and some foreign comparisons', *The Structural Engineer*, **71:7**, April, 111–16.
25. Smith, L.J. and Wood, R. (2001) 'Grouting of external tendons – a practical perspective', *Proceedings of the Institution of Civil Engineers*, Issue 1, **146**, February, 93–100.
26. Smith, W.J.R., Benaim, R. and Hancock, C.J. (1980) 'Tyne and Wear metro: Byker Viaduct', *Proceedings of the Institution of Civil Engineers*, Part 1, **68**, November, 701–18.
27. Sriskandan, K. (1989) 'Prestressed concrete road bridges in Great Britain: a historical survey', *Proceedings of the Institution of Civil Engineers*, Part 1, **86**, April, 269–303.
28. Taylor, H.P.J. (1998) 'The precast concrete bridge beam: the first 50 years', *The Structural Engineer*, **76:21**, November, 407–14.
29. Van Leonen, J. and Telford, S. (1983) 'Orwell Bridge-construction', *The Proceedings of the Institution of Civil Engineers*, Part 1, **74**, November, 779–804.
30. Witecki, A.A. (1969) Simplified method for the analysis of torsional moment as an effect of a horizontally curved multispan continuous bridge', *American Concrete Institute*, First International Symposium on Concrete Bridge Design, pp. 193–204.
31. Wood, R.H. (1968) 'The reinforcement of slabs in accordance with a pre-determined field of moments', *Concrete*, February, 69–76.
32. Woodward, R.J. and Williams, F.W. (1998) 'Collapse of Ynys-y-Gwas Bridge, West Glamorgan', *Proceedings of the Institution of Civil Engineers*, Part 1, **85**, 635–69.

Standards, codes of practice and specifications

1. Highways Agency (1998) *Manual of Contract Documents for Highway Works, Volume 1, Specification for Highway Works*. Norwich: HMSO.
2. Highways Agency, Departmental Standards (MDRB). Norwich: HMSO.
   - BD 20/92 Bridge Bearings. Use of BD 5400: Part 9: 1983.
   - BD 24/92 Design of Concrete Bridges. Use of BS 5400: Part 4: 1990.
   - BD 28/87 Early Thermal Cracking of Concrete.
   - BD 33/94 Expansion Joints for Use in Highway Bridge Decks.
   - BD 37/01 Loads for Highway Bridges.
   - BD 47/99 Waterproofing and Surfacing of Concrete Bridge Decks.
   - BD 52/93 The Design of Highway Bridge Parapets.
   - BD 57/01 Design for Durability.
   - BD 58/94 The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing.
3. Highways Agency, Advice Notes. Norwich: HMSO.
   - BA 24/87 Early Thermal Cracking of Concrete (including amendment No. 1 (1989)).
   - BA 26/94 Expansion Joints for Use in Highway Bridge Decks.
   - BA 36/90 The Use of Permanent Formwork.

@Seismicisolation

BA 41/98	The Design and Appearance of Bridges.
BA 42/96	The Design of Integral Bridges.
BA 48/99	Waterproofing and Surfacing of Concrete Bridge Decks.
BA 57/01	Design for Durability.
BA 58/94	Design of Bridges and Concrete Structures with External Unbonded Prestressing.

4. British Standard

CP 115	*The structural use of prestressed concrete in building, 1959.*
BS 12:1991	*Specification for Portland cement.*
BS 4447	Specification for the performance of prestressing anchorages for post-tensioned construction.
BS 4486	Hot rolled and processed high tensile alloy steel bars for the prestressing of concrete.
BS 5400	Steel, concrete and composite bridges. Part 1 (1988) General Statement. Part 2 (1990) Specification for loads. Part 4 (1990) Code of practice for the design of concrete bridges. Part 9 (1983) Bridge Bearings.
BS 5896	High tensile steel wire and strand for the prestressing of concrete.
BS EN 197-1	*Part 1, Composition, Specification and Conformity Criteria for Common Cements.*
BS EN 447	*Grout for Prestressing Tendons – Specification for Common Grout.*
BS EN 523	*Steel Sheaths for Tendons – Terminology, Requirements, Quality Control.*

5. AASHTO (1996–2002) *Standard Specification for Highway Bridges, 16th edition.* Washington, DC: American Association of State Highway and Transportation Officials.
6. AASHTO (1998–2002) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 2nd edition.* Washington, DC: American Association of State Highway and Transportation Officials.
7. AASHTO (1999) *Guide Specification for Design and Construction of Segmental Concrete Bridges, 2nd edition.* Washington, DC: American Association of State Highway and Transportation Officials.
8. ASTM A416M-99 (1999) *Standard Specification for Uncoated 7-wire Steel Strand for Prestressing Concrete.* Pennsylvania: ASTM International.
9. ASTM A722M-98 (1998) *Standard Specification for Uncoated Steel Bar for Prestressing Concrete.* Pennsylvania: ASTM International.
10. CEB-FIP (1993) *Model Code 1990.* London: Comité Euro-International du Beton, Thomas Telford.
11. EUROCODE (2002) No. 2. *Design of Concrete Structures – Part 1: General rules and rules for buildings.* Pr EN 1992-1-1.
12. EUROCODE (1996) No. 2. *Design of Concrete Structures, Part 2: Concrete Bridges.* ENV 1992–2.

@Seismicisolation

Websites

Prestressing systems		
BBR	www.bbrsystems.ch	Details of prestressing systems. General descriptions of bridge construction and project data.
DYWIDAG	www.dywidag-systems.com	General website with comprehensive details of the DYWIDAG prestressing system.
Freyssinet	www.freyssinet.com www.freyssinet.co.uk	General website with description of products as well as project data.
Macalloy Bar Systems	www.macalloy.com	General website with details of Macalloy bars.
MK4	www.mekano4.com	Description of different forms of bridge construction. Brochures for prestressing system and bearings can be downloaded.
VSL	www.vsl-intl.com	General website with details of different construction activities. Details of projects and services.

Material and products		
Prestressing wire and strand		
Carrington Wire	www.carringtonwire.com	Descriptions of wires available.
Trefileurope	www.ispat.com	Details of wires and strands.
Tycsa	www.tycsa.com	Comprehensive data on wire and strands available for bridgeworks.
Bridon Wire	www.bridonltd.com	Details of wires for bridges.
Precast beams		
Tarmac Precast Concrete Ltd	www.tarmacprecast.com	General website with reference to precast beams.
Redlands	www.redlandprecast.com.hk	General website with bridge beams and precast parapets.
Bearings, expansion joints and drainage		
(see also companies and websites under prestressing systems above)		
Bridge Joint Association	www.bridgejoints.org.uk	Details and drawings of different expansion joints used on bridges.
Britflex	www.usluk.com	Details of bridge expansion joints and waterproofing systems.
Bowman	www.bowmanconstructionsupply.com	Information sheets on a range of bearings and expansion joints.
D.S. Brown	www.dsbrown.com	Details of expansion joints and bearings, which can be downloads.
Ekspan	www.ekspan.co.uk	Details of bearings, expansion joints and drainage.
Mageba SA	www.mageba.ch	General website with examples of projects.
Maurer Sohne	www.maurer-soehne.de	General description of services provided.
Maclellan Rubber	www.macrubber.com	Details of rubber bearings.
Prismo	www.prismo.co.uk/bridges	Description of Thorma joints.
Proceq SA	www.proceq.com	Details of bridge bearings and expansion joints.
RJ Watson	www.rjwatson.com	Details of Disktron bearings.
Parapets		
Baco Contracts	www.baco-contracts.com	General website with reference to aluminium parapets.
Deck waterproofing		
ASL Contracts	www.aslcontracts.co.uk	Details of Servideck and Bridgeguard systems.
Dew Pitchmastic	www.dewpitchmastic.co.uk	Description of PmB waterproofing membranes.
Grace	www.grace-construction.com	Details of Bituthen® waterproofing membrane.

@Seismicisolation

Material and products		
Deck waterproofing – Continued		
Stirling Lloyd	www.stirlinglloyd.com	General website with details of Eliminator(K) waterproofing system.
Universal Sealants	www.usluk.com	Details of Britflex waterproofing system.
Formwork and falsework		
Peri Ltd	www.peri.ltd.uk	Descriptions of formwork and falsework systems with examples of projects.
SGB Formwork	www.sgb.co.uk	General website of services offered.
Thyssen Hünnebeck	www.thyssen-huennebeck.com	Description of formwork and falsework systems.
RMD	www.rmdformwork.com	General website of services offered.
Doka	www.doka.com	General website with descriptions of formwork and falsework systems.
Erection gantries and construction equipment		
Deal	www.deal.it	Description of erection equipment, moulds and formwork, with examples of projects.
NRS	www.nrsas.com	Description of gantries, form travellers and movable scaffolding with comprehensive library of pictures.
Paolo de Nicola S.p.A	www.paolodenicola.com	Examples of erection equipment.
Tony Gee and Partners	www.tgp.co.uk	Examples of gantries and temporary works designs.
Bridge access systems		
MOOG	www.moog-online.de	Description and examples of bridge inspection equipment.
Barin S.p.A	www.barin.it	Technical details and sketches of bridge inspection equipment.

Institutions, societies and research bodies		
American Concrete Institute (ACI)	www.aci-int.org	General website of the activities and publications.
American Segmental Bridge Institute	www.asbi-assoc.org	Details of publications and activities. Drawings of standard bridges.
British Cement Association	www.bca.org.uk	General website of activities, publications and services.
Concrete Bridge Development Group	www.cbdg.org.uk	General website of activities.
Concrete Society	www.concrete.org.uk	General website of activities and publications.
Fédération International du Beton	http://fib.epfl.ch	Details of activities, publications and industry news.
IABSE	www.iabse.ethz.ch	Information and references on bridges and associated topics.
Institution of Civil Engineers	www.ice.org.uk	Extensive list of library references.
Institution of Structural Engineers	www.istructe.org.uk	Extensive list of library references.
Post-tensioning Institute	www.post-tensioning.org	General website of the institute's activities.
Precast/Prestressed Concrete Institute	www.pci.org	Details of publications, news and general information on precast and prestressed concrete.
Portland Cement Association	www.portcement.org	www.portcement.org/br/br-tech_refs.asp contains an extensive list of USA references on concrete bridges.

@Seismicisolation

Appendix D: Proprietary systems

Introduction

This appendix gives details of several of the major prestressing systems that are available and can be used as a guide when designing and detailing the prestressing arrangements. All the details published here are with the kind permission of the companies noted.

As well as these systems there are other similar products available in the different local markets around the world.

For any particular project, the details of the systems locally available should be confirmed before commencing the detailed design.

Multi-strand systems

Most of the major prestressing suppliers produce a multi-strand prestressing system. VSL, Freyssinet, DYWIDAG, BBR and MEKANO4 all produce systems that have similar characteristics:

- use of 13 mm (0·5″) or 15 mm (0·6″) diameter strands;
- standardized tendon anchorages and couplers for use with up to 55 strands;
- selection of anchorage types;
- steel or polyethylene ducts;
- grouting with cement mortar or other materials;
- all strands in a tendon stressed simultaneously, but with each strand individually anchored;
- stressing in steps.

Details of the VSL system are given in Figs D1 to D7. The typical tendon units used and the associated steel duct sizes for internal tendons are given in Fig. D1.

For enhanced corrosion protection and improved fatigue resistance of the tendons the VSL PT-PLUS™ system, or similar, with corrugated polyethylene ducts is adopted. Details of the VSL PT-PLUS™ system are given in Fig. D2.

VSL use a variety of anchorage arrangements with a typical detail for their live end anchorage type EC given in Fig. D3 and the dead-end anchorage type

@Seismicisolation

13 mm (0·5″) strand

Tendon unit	Number of strands	Duct diameter internal/external (mm)	Min. breaking load: Strand type Euronorm 138-79 or BS 5896: 1980, Super (kN)	Min. breaking load: Strand type ASTM A416-85 Grade 270 (kN)
5-1	1	25/30	186	184
5-2	2	40/45	372	367
5-3	3	40/45	558	551
5-4	4	45/50	744	735
5-6	6	50/55	1116	1102
5-7	7	55/60	1302	1286
5-12	12	65/72	2232	2204
5-18	18	80/87	3348	3307
5-19	19	80/87	3534	3490
5-22	22	85/92	4092	4041
5-31	31	100/107	5786	5695
5-37	37	120/127	6882	6797
5-43	43	130/137	7998	7899
5-55	55	140/150	10 230	10 104

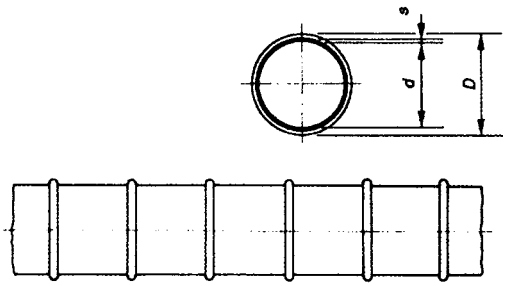
15 mm (0·6″) strand

Tendon unit	Number of strands	Duct diameter internal/external (mm)	Min. breaking load: Strand type Euronorm 138-79 or BS 5896: 1980, Super (kN)	Min. breaking load: Strand type ASTM A416-85 Grade 270 (kN)
6-1	1	30/35	265	261
6-2	2	45/50	530	521
6-3	3	45/50	795	782
6-4	4	50/55	1060	1043
6-6	6	60/67	1590	1564
6-7	7	60/67	1855	1825
6-12	12	80/87	3180	3128
6-18	18	95/102	4770	4693
6-19	19	95/102	5035	4953
6-22	22	110/117	5830	5735
6-31	31	130/137	8215	8082
6-37	37	140/150	9805	9646
6-43	43	150/160	11 395	11 210
6-55	55	170/180	14 575	14 339

Figure D1. VSL typical tendon and steel duct sizes.

H given in Fig. D4. Details of two coupler arrangements are given in Fig. D5. Details of VSL's jacks, anchor pockets and clearances for the multi-strand tendons are given in Figs D6 and D7.

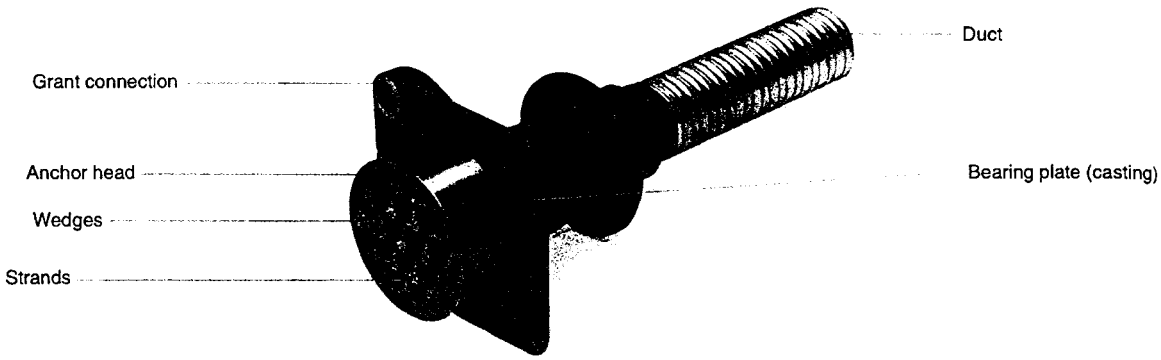
@Seismicisolation

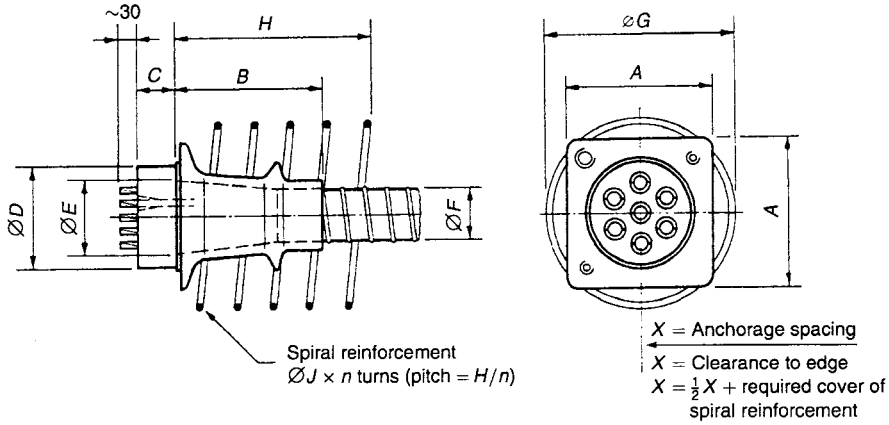


Strand type 13 mm (0·5″)	Strand type 15 mm (0·6″)	Dimensions of duct (mm)		
Tendon unit	Tendon unit	*d*	*D*	*s*
5-12	6-7	59	73	2
5-19	6-12	76	91	2·5
5–31	6-19/6-22	100	116	3
5-55	6-37	130	146	3

Figure D2. VSL PT-PLUS™ duct sizes.

@Seismicisolation

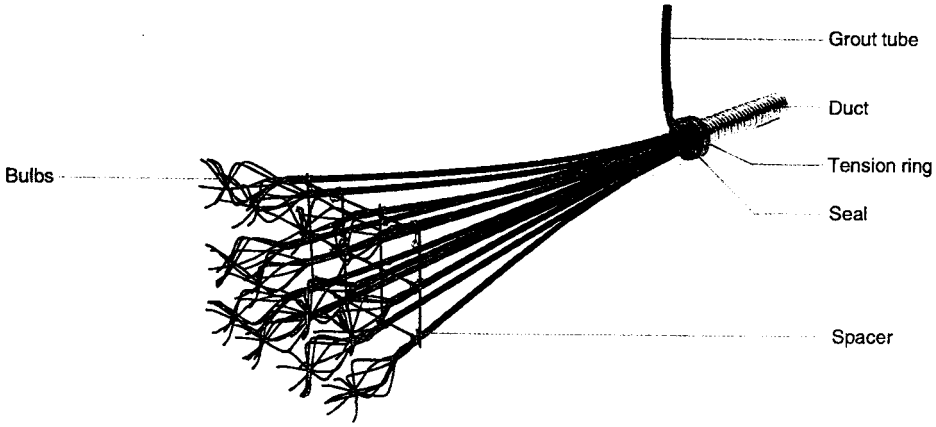


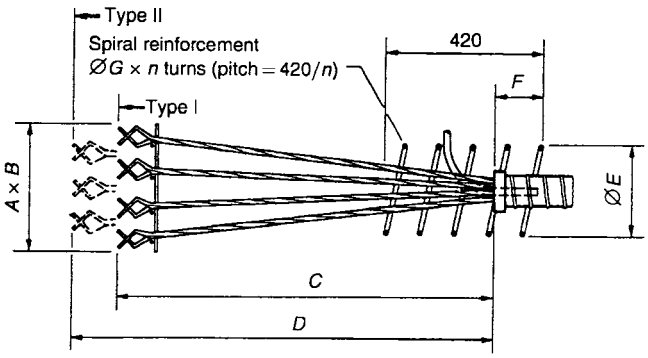


Tendon unit	A	B	C	ØD	ØE	ØF internal/ external	ØG	H	ØJ	n	X
Strand type 13 mm (0·5″)											
5-3	120	130	50	90	50	40/45	130	150	10	3	155
5-4	135	125	50	95	55	45/50	160	150	10	3	180
5-7	165	155	55	110	74	55/60	205	200	12	4	235
5-12	215	215	60	150	104	65/72	285	250	14	5	305
5-19	270	285	75	180	135	80/87	365	300	16	6	385
5-22	290	335	85	190	150	85/92	395	360	18	6	415
5-31	340	365	95	230	172	100/107	470	400	18	8	490
5-37	370	360	105	240	188	120/127	510	420	20	7	535
5-55	430	460	130	290	230	140/150	620	540	22	9	655
Strand type 15 mm (0·6″)											
6-3	135	125	50	95	55	45/50	160	150	10	3	185
6-4	150	155	55	110	65	50/55	190	200	12	4	210
6-7	190	170	60	135	84	60/67	260	250	14	5	280
6-12	250	245	75	170	118	80/87	345	300	16	6	365
6-19	310	305	95	200	150	95/102	440	350	18	7	460
6-22	340	365	100	220	172	110/117	470	400	18	8	495
6-31	390	350	120	260	192	130/137	560	480	20	8	590
6-37	430	450	135	280	215	140/150	610	540	22	9	640
6-55	520	530	160	340	255	170/180	740	630	26	9	780

Figure D3. VSL anchorage type EC.

@Seismicisolation





The force in the tendon is transferred by bond of the strands in the concrete and by the bulbs at the end of the strands

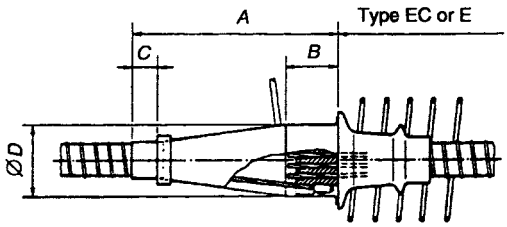
Strand type 13 mm (0·5″)

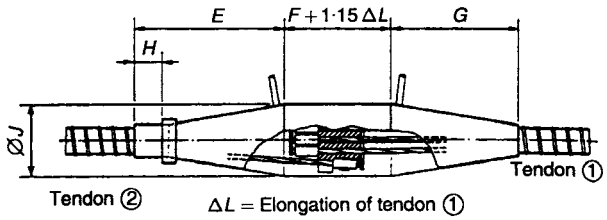
Tendon unit	Alternative	Type	*A*	*B*	*C*	*D*	Ø*E*	*F*	Ø*G*	*n*
5-3	1	I	230	70	930	–	–	–	–	–
5-4	1	I	310	70	930	–	–	–	–	–
	2	I	150	170	930	–	–	–	–	–
5-7	1	II	370	70	1130	1280	180	155	12	7
	2	II	170	190	1130	1280	180	155	12	7
5-12	1	II	350	190	1130	1280	200	155	14	7
	2	I	310	270	1130	–	200	155	14	7
5-19	1	II	470	190	1130	1280	230	155	14	7
	2	II	310	390	1130	1280	230	155	14	7
5-22	1	II	570	190	1130	1280	300	155	16	7
	2	II	390	390	1130	1280	300	155	16	7
5-31	1	II	670	310	1330	1480	350	155	16	7
	2	II	470	430	1330	1480	350	155	16	7
5-37	1	II	770	310	1530	1680	350	165	18	7
	2	II	470	550	1530	1680	350	165	18	7
5-43	1	II	870	350	1530	1680	400	165	18	7
	2	II	670	430	1530	1680	400	165	18	7
	3	II	570	550	1530	1680	400	165	18	7
5-55	1	II	1170	350	1830	1980	400	175	20	7
	2	II	870	430	1830	1980	400	175	20	7
	3	II	570	670	1830	1980	400	175	20	7

Strand type 15 mm (0·6″)

Tendon unit	Alternative	Type	*A*	*B*	*C*	*D*	Ø*E*	*F*	Ø*G*	*n*
6-3	1	I	290	90	950	–	–	–	–	–
6-4	1	I	390	90	950	–	–	–	–	–
	2	I	190	210	950	–	–	–	–	–
6-7	1	II	450	90	1150	1300	200	155	14	7
	2	II	210	230	1150	1300	200	155	14	7
6-12	1	II	430	230	1150	1300	230	155	14	7
	2	I	390	330	1150	–	230	155	14	7
6-19	1	II	570	230	1150	1300	300	155	16	7
	2	II	390	470	1150	1300	300	155	16	7
6-22	1	II	690	230	1150	1300	350	155	16	7
	2	II	470	490	1150	1300	350	155	16	7
6-31	1	II	810	260	1550	1700	400	165	18	7
	2	II	570	510	1550	1700	400	165	18	7
6-37	1	II	1050	370	1850	2000	400	175	20	7
	2	II	690	510	1850	2000	400	175	20	7
6-43	1	II	1050	370	1850	2000	400	175	20	7
	2	II	810	510	1850	2000	400	175	20	7
	3	II	690	650	1850	2000	400	175	20	7
6-55	1	II	1410	370	2150	2300	460	185	22	8
	2	II	1050	510	2150	2300	460	185	22	8
	3	II	690	790	2150	2300	460	185	22	8

@Seismicisolation



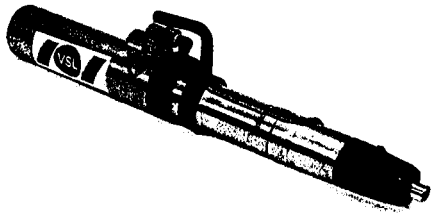


Coupler type K				
Tendon unit	*A*	*B*	*C*	*ØD*
Strand type 13 mm (0·5″)				
5-3	430	140	40	130
5-7	550	140	60	170
5-12	650	140	65	200
5-19	740	140	80	240
5-22	830	140	90	260
5-31	1140	140	95	350
5-37	1320	180	125	390
5-55	1370	200	150	420
Strand type 15 mm (0·6″)				
6-2	380	150	30	130
6-3	490	160	60	150
6-4	520	160	60	160
6-7	630	160	75	190
6-12	730	160	80	240
6-19	860	160	95	280
6-22	930	160	95	310
6-31	1090	180	135	360
6-37	1390	200	135	430

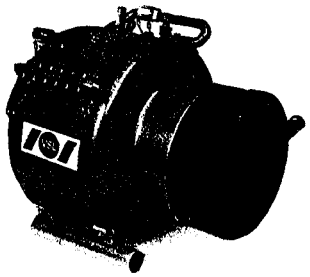
Coupler type V					
Tendon unit	*E*	*F*	*G*	*H*	*ØJ*
Strand type 13 mm (0·5″)					
5-3	290	175	230	40	130
5-7	410	175	330	60	170
5-12	510	175	420	65	200
5-19	600	175	490	80	240
5-22	690	175	570	90	260
5-31	1000	175	870	95	350
Strand type 15 mm (0·6″)					
6-2	230	200	185	30	130
6-3	330	205	250	60	150
6-4	360	205	280	60	160
6-7	470	205	370	75	190
6-12	570	205	460	80	240
6-19	700	205	570	95	280

Figure D5. VSL couplers types K and V.

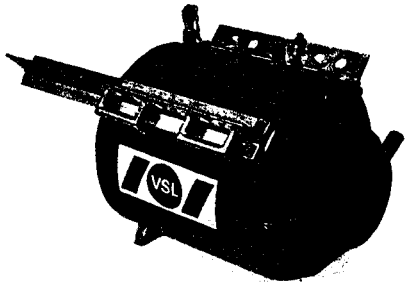
@Seismicisolation



Type I (ZPE-23 FJ)



Type II (ZPE-19)

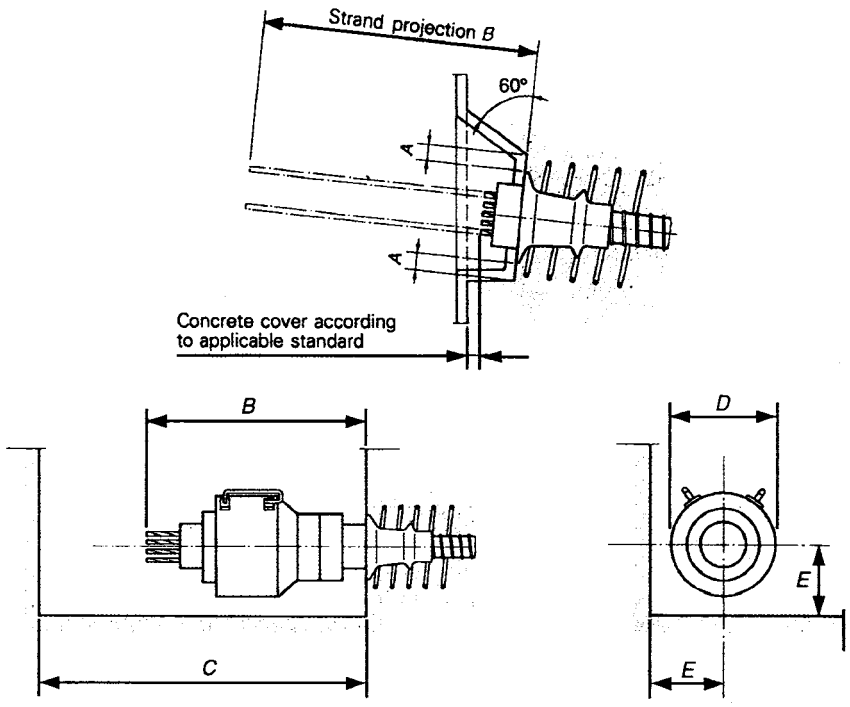


Type III (ZPE-500J)

Designation	ZPE-23FJ	ZPE-30	ZPE-3	ZPE-60	ZPE-7/A	ZPE-12/St2	ZPE-200	ZPE-19	ZPE-460/31	ZPE-500	ZPE-750	ZPE-1000	ZPE-1250
Type	I	III	III	III	III	II	III	II	II	III	II	III	II
Length (mm)	790	720	475	615	690	550	960	750	580	1000	1185	1200	1290
Diameter (mm)	116	140	200	180	280	310	315	390	485	550	520	790	620
Stroke (mm)	200	250	160	250	160	100	300	100	100	200	150	200	150
Piston area (cm^2)	47·10	58·32	103·6	126·4	203·6	309·4	325·7	500·3	804·0	894·6	1247·0	1809·5	2168·0
Capacity (kN)	230	320	500	632	1064	1850	2000	2900	4660	5000	7500	10 000	12 500
(bar)	488	549	483	500	523	598	614	580	580	559	601	553	577
Weight (kg)	23	28	47	74	115	151	305	294	435	1064	1100	2290	1730
Used for 13 mm/0·5″ tendon types	5-1	5-1	5-2, 5-3	5-2 to 5-4	5-6, 5-7	5-12	5-12	5-18, 5-19	5-22, 5-31	5-22, 5-31	5-31, 5-37	5-37 to 5-55	5-37 to 5-55
Used for 15 mm/0·6″ tendon types	6-1	6-1	6-2	6-2, 6-3	6-4	6-6, 6-7	6-6, 6-7	6-12	6-18, 6-19	6-18 to 6-22	6-31	6-31 to 6-43	6-31 to 6-55

Figure D6. VSL jack details.

@Seismicisolation



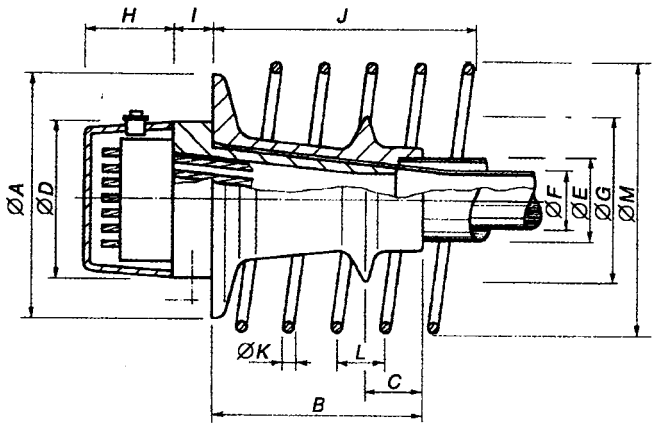
Jack type	A min.	B	C	D	E
ZPE-23FJ	–	300	1200	116	90
ZPE-30	30	600	1100	140	100
ZPE-3	30	550	1000	200	150
ZPE-60	30	650	1100	180	140
ZPE-7/A	30	800	1200	300	200
ZPE-12/St2	50	700	1300	310	200
ZPE-200	50	1100	2100	330	210
ZPE-19	50	850	1500	390	250
ZPE-460/31	60	700	1500	485	300
ZPE-500	80	1150	2000	585	330
ZPE-750	80	1350	2300	570	365
ZPE-1000	80	1300	2200	790	450
ZPE-1250	90	1350	2250	660	375

Figure D7. VSL anchorage block-out and jack clearances.

External tendon systems

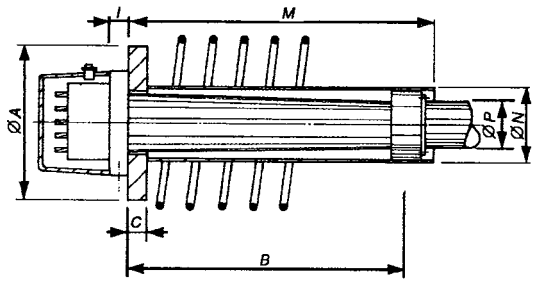
The main differences between external and internal multi-strand tendons are the anchor arrangement and the use of HDPE ducts, as described in Chapter 2. Again, all the major prestress suppliers have developed suitable arrangements for external tendons, with two typical VSL anchorage details given in Figs D8 and D9. The tendon and ducts are effectively debonded from the anchor trumpet and surrounding concrete to allow future replacement if necessary.

@Seismicisolation



Tendon unit	Dimensions													
	A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*
Strand type 0·6″														
6-7	250	245	60	175	95	75	190	100	45	345	16	55	330	6
6-12	310	305	65	225	115	90	235	115	45	235	18	60	400	7
6-19	390	350	70	275	145	110	295	135	45	540	22	65	510	8
6-31	430	450	80	305	155	125	320	160	55	605	22	65	550	9
6-37	520	530	90	320	183	140	420	170	55	745	26	80	640	9

Figure D8. VSL 'external' stressing anchorage type Ed.



Tendon unit	Dimensions						
	A	*B* min	*C*	*I*	*M*	*N*	*P*
Strand type 0·6″							
6-1	80	35	10	35	According to the dimensions of the work at the anchorage	According to the dimensions of the opening at the anchorage	According to the material and dimensions of the duct
6-2	120	155	15	35			
6-3	150	155	25	35			
6-4	170	145	25	45			
6-7	220	145	35	45			
6-12	300	325	45	45			
6-19	370	485	55	45			
6-22	400	605	60	45			
6-31	470	635	75	55			
6-37	520	775	80	55			

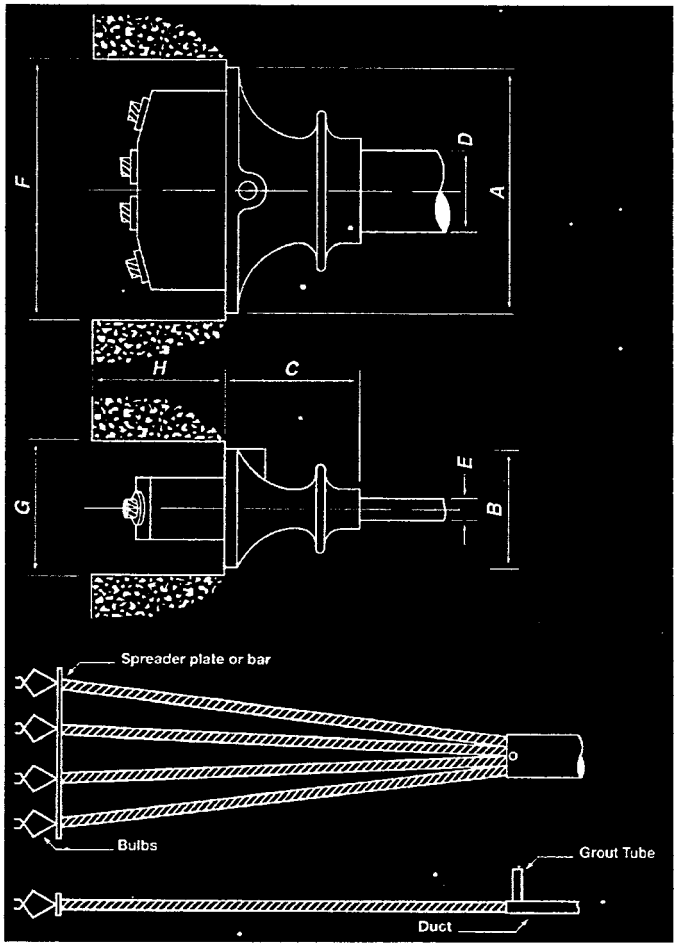
Figure D9. VSL 'external' stressing anchorage type Edm.

@Seismicisolation

Flat-slab systems

Used in thin slabs, this multi-strand tendon arrangement utilizes either 13 mm or 15 mm diameter strands. They are installed inside flat ducts to take up the minimum space and to be able to achieve the maximum eccentricity for the strand within the slab. Details of the Freyssinet slabstress system are shown in Fig. D10. VSL, DYWIDAG, BBR and Mekano4 also provide similar flat-slab systems.

@Seismicisolation



Tendon reference	No. of strands	Strand type	Total ultimate tensile strength kN	80% UTS	60% UTS	Transfer strength of concrete N/mm^2
4S13	4	12·9 Super	744	595	446	25
4S13D	4	12·7 Drawn	836	669	502	25
4S15	4	15·7 Super	1060	848	636	25
4S15E	4	15·7 Euro	1116	893	670	28
4S15D	4	15·2 Drawn	1200	960	720	33
5S13	5	12·9 Super	930	744	558	25
5S15	5	15·7 Super	1325	1060	795	25

	Anchor			Duct		Pocket		
Anchor ref. Dimension	Width *A*	Depth *B*	Length *C*	Width *D*	Depth *E*	Width *F*	Depth *G*	Length *H*
4S13	195	85	95	75	20	215	90	90
4S15	230	100	110	75	20	230	110	120
5S13	220	75	215	70	19	260	100	100
5S15	260	80	270	90	19	360	100	100

Figure D10. Freyssinet slabstress tendons.

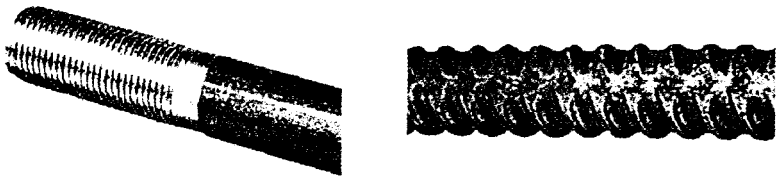
@Seismicisolation

Bar systems

DYWIDAG, VSL and McCalls provide bar prestressing systems, described in Chapter 2. Bars were used in some of the first prestressed concrete structures and most of the systems have been established for many years. Prestressing bars are either fully threaded along their complete length or are smooth with just the ends threaded.

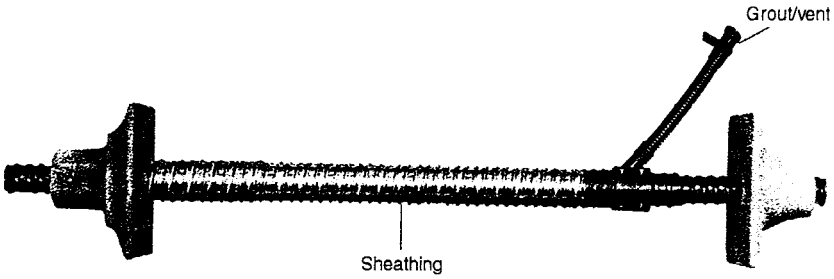
DYWIDAG bar post-tensioning 'smooth bar' and 'threadbar®' arrangements are shown in Fig. D11. The bars are available in lengths of up to 30 m.

The DYWIDAG smooth bar has threaded ends with a rolled-on thread manufactured in the factory to the specific length required. The rolled



(a) Smooth bar (b) Threadbar®

Figure D11. DYWIDAG prestressing bars.



Technical data

Type		Smooth bar			Threadbar®				
Steel grade $f_{p0\cdot1k}/f_{pk}$	N/mm²	835/1030	1080/1230		835/1030			1080/1230	
Diameter/type		32G	36C	26E	32E	36E	26D	32D	36D
Diameter	mm	32	36	26	32	36	26	32	36
Nom. cross section area	mm²	804	1018	551	804	1018	551	804	1018
Weight	kg/m	6·31	7·99	4·48	6·53	8·27	4·48	6·53	8·27
Pitch	mm	3	3	13·0	16·0	18·0	13·0	16·0	18·0
Yield load $f_{p0\cdot1k}A$	kN	671	1099	460	671	850	595	868	1099
Ultimate load $f_{pk}A$	kN	828	1252	568	828	1049	678	989	1252
Fatigue stress range									
Steel (0·55 f_{pk}/0·9$_{p0\cdot1k}$)	N/mm²	290/230	290/230	240/210	240/210	240/210	240/210	240/210	240/210
Nut/coupler 0·6 f_{pk}	N/mm²	98	98	98	78	78	78	78	78
Bending									
Elastic bending min. *R*	m	19·50	12·10	16·20	19·50	21·90	8·90	10·75	12·10
Cold bending min. *R*	m	4·80	5·40	5·30	6·40	7·20	5·30	6·40	7·20

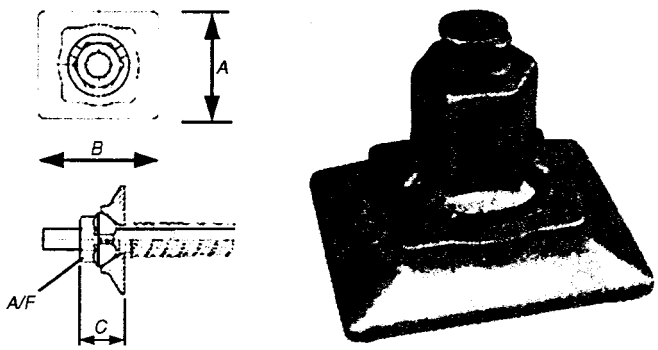
Values based on German approval. Customizations according to different codes and concrete strengths applicable.

Figure D12. DYWIDAG bar post-tensioning details.

@Seismicisolation

thread results in a characteristic low seating loss and high fatigue resistance. The DYWIDAG threadbar® is hot rolled and comes with thread-like deformations over its entire length. The threadbar® is cut to length and is instantly threadable without further preparation.

Figure D12 gives the data for the DYWIDAG bar systems. Details of the anchorage and coupler components are given in Figs D13 and D14 respectively. The steel duct details are given in Fig. D15. The jacks used with the DYWIDAG bar systems are shown in Fig. D16.



Technical data

Bar type		Smooth bar			Threadbar®				
Steel grade $f_{p0\cdot1k}/f_{pk}$	N/mm²	835/1030	1080/1230		835/1030			1080/1230	
Nom. diameter	mm	32·0	36·0	26·5	32·0	36·0	26·5	32·0	36·0
Diameter/type	–	32G	36C	26E	32E	36E	26D	32D	36D
A	mm	–	160	–	–	140	120	140	160
B	mm	–	180	–	–	165	130	165	180
C	mm	–	75	–	–	107	90	107	115
Width across flats *A/F*	mm	–	65	–	–	60	50	60	65

Details of the anchorage zone

Bar type			Smooth bar			Threadbar®				
Steel grade $f_{p0\cdot1k}/f_{pk}$		N/mm²	835/1030	1080/1230		835/1030			1080/1230	
Nom. diameter		mm	32·0	36·0	26·5	32·0	36·0	26·5	32·0	36·0
Diameter/type		–	32G	36C	26E	32E	36E	26D	32D	36D
Concrete strength class B25	Y	Centre distances	–	280	–	–	–	200	250	280
		Edge distances	–	160	–	–	–	120	145	160
	X	Centre distances	–	320	–	–	–	220	280	320
		Edge distances	–	180	–	–	–	130	160	180
Concrete strength class B35	Y	Centre distances	–	260 (380)	–	–	(340)	190	230	260 (380)
		Edge distances	–	150 (190)	–	–	(170)	115	135	150 (190)
	X	Centre distances	–	290 (440)	–	–	(400)	210	260	290 (440)
		Edge distances	–	165 (220)	–	–	(200)	125	150	165 (220)
Concrete strength class B45	Y	Centre distances	–	230	–	–	–	170	210	230
		Edge distances	–	135	–	–	–	105	125	135
	X	Centre distances	–	260	–	–	–	190	230	260
		Edge distances	–	150	–	–	–	115	135	150

Figure D13. DYWIDAG bar anchor details.

@Seismicisolation

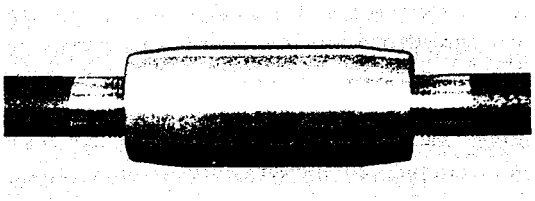
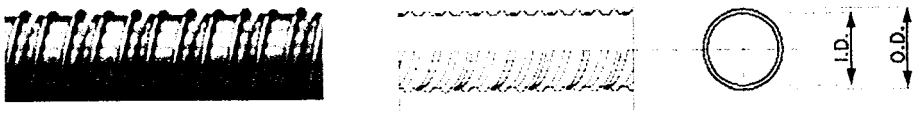


Figure D14. DYWIDAG bar coupler.

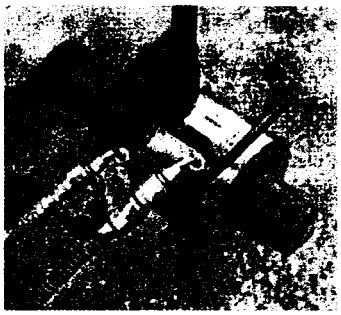


Type		Smooth bar			Threadbar®				
Steel grade $f_{p0\cdot1k}/f_{pk}$	N/mm²	835/1030	1080/1230		835/1030			1080/1230	
Diameter/type		32G	36C	26E	32E	36E	26D	32D	36D
Diameter	mm	32	36	26	32	36	26	32	36
Internal diameter I.D.	mm	44	51	38	44	51	38	44	51
External diameter O.D.	mm	49	57	43	49	57	43	49	57
Min. centre distances	mm	79	92	68	79	92	68	79	92
Support distances	m	0·5–2·5			0·5–2·5				
Wobble angle β	°/mm	0·3			0·3				
Friction coefficient μ	–	0·25			0·5				

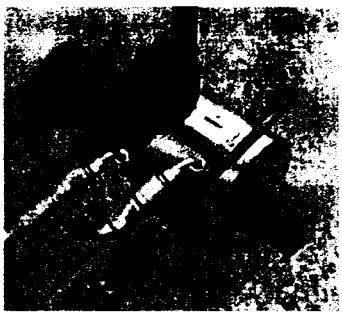
The values are based on the German Code (DIN). Adaptations to other code systems or concrete strengths (e.g. ASTM, BS, etc.) are possible.

Figure D15. DYWIDAG bar duct sizes.

@Seismicisolation



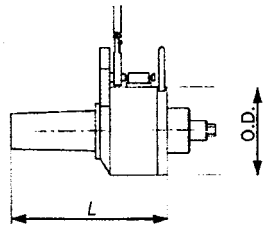
Jack 60 Mp series 04



Jack 110 Mp series 04

Bar type	Smooth bar		Threadbar®					
Steel grade	835/1030	1080/1230	835/1030			1080/1230		
Nom. diameter	32	36	26·5	32·0	36·0	26·5	32·0	36·0
Diameter/type	32G	36C	26E	32E	36E	26D	32D	36D
60 Mp	●		●	●	★	●	★	
110 Mp	○	●		○	●		●	●

★ Use 60 Mp if only stressed 55% to ultimate load.
○ Can be stressed with 110 Mp on certain exceptions.



Technical data

Jack types	Length *L* (mm)	O.D. (mm)	Stroke (mm)	Piston area (cm^2)	Capacity (kN)	Capacity (MPa)	Weight (kg)
60 Mp Series 04	401	190	50	132·5	625	50	36
60 Mp Series 05	456	190	100	132·5	625	50	44
110 Mp Series 01	494	267	50	235·6	1100	50	46
110 Mp Series 03	594	267	150	235·6	1100	50	54

Jack dimensions (for block-out design)

Jack types	*A* (mm)	*B* (mm)	*C* (mm)	*D* (mm)	*E* (mm)
60 Mp Series 04	225	176	106	106	300
60 Mp Series 05	225	231	106	106	300
110 Mp Series 01	275	219	125	120	375
110 Mp Series 03	275	319	125	120	375

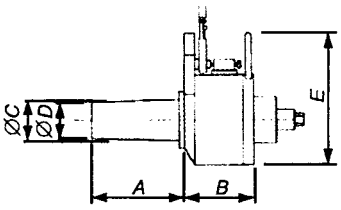


Figure D16. DYWIDAG bar jack data.

@Seismicisolation

Wire prestressing systems

Multi-wire stressing systems were used on some of the very first prestressed bridges and are still available from BBR. The BBRV system uses bundles of individual 7 mm diameter wires fixed into the anchorage with a BBRV button head. Each wire is mechanically fixed in the anchor head and can reach its full rupture load without any slippage. Before the tendons are grouted they are re-stressable and de-stressable.

The BBRV anchoring method allows the production of post-tensioning tendons with any number of single wires and therefore with any given magnitude of prestressing force. Standard tendon arrangements are illustrated in Fig. D17 and tendons details given in Fig. D18. Details of the BBRV anchorages, couplers and jacks are given in Figs D19, D20 and D21 respectively.

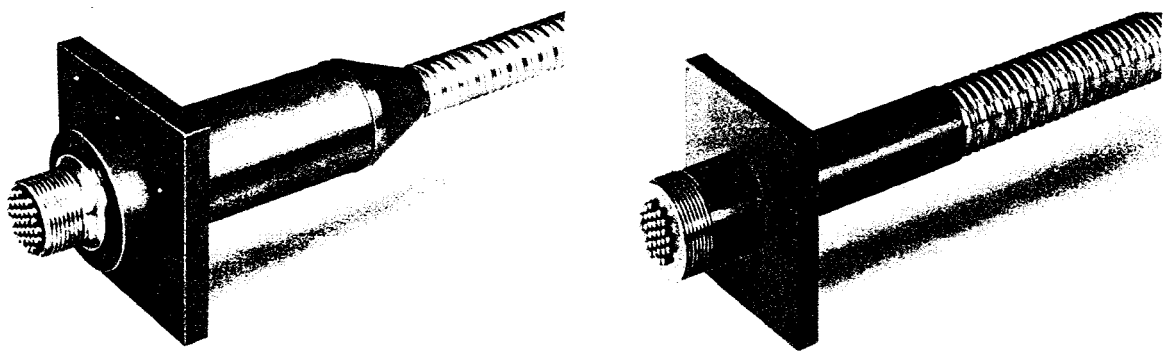
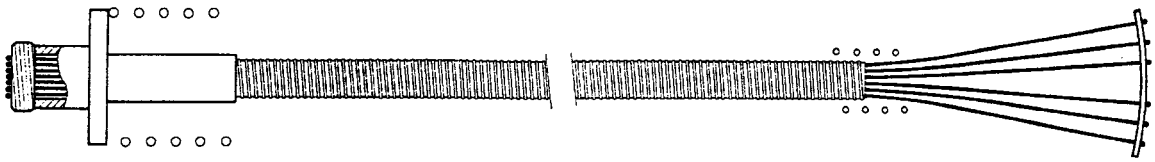


Figure D17. BBRV wire tendons.



Stressing anchorage type A

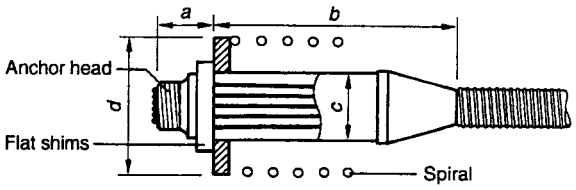
Fixed anchorage type S

Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Ultimate tensile force (u.t.s. 1670 N/mm^2)	kN	449	772	1221	1992	2700	3342	3919	5269	6555
Stressing force at 0·8 u.t.s.	kN	359	618	977	1594	2160	2674	3135	4215	5244
Stressing force at 0·75 u.t.s.	kN	337	579	916	1494	2024	2507	2939	3952	4916
Cross section	mm^2	269	462	731	1193	1616	2001	2347	3155	3925
Weight of wires	kg/m	2·11	3·62	5·74	9·36	12·86	15·70	18·42	24·76	30·80
Conduit I.D.	mm	30	40	50	55	65	75	80	90	100

Figure D18. BBRV standard wire tendons.

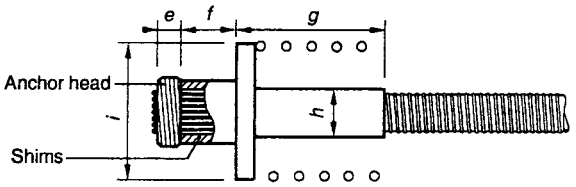
@Seismicisolation

Stressing anchorage type L



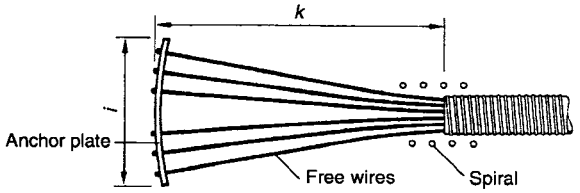
Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Anchor	*a* mm	63	74	91	108	123	135	156	180	205
Trumpet length min.	*b* mm	250	250	250	280	300	300	300	340	360
Trumpet diameter	*c* mm	70	88	102	123	138	153	171	193	219
Bearing plate	*d* mm	140	170	200	245	285	315	345	400	450

Stressing anchorage type A



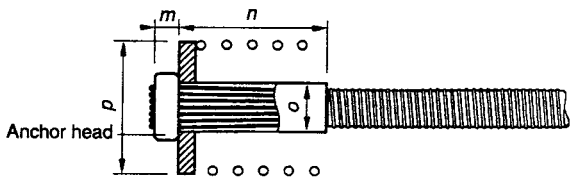
Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Anchor	*e* mm	25	27	36	43	49	56	67	78	85
Elongation max.	*f* mm	200	200	200	200	200	250	250	350	350
Trumpet length	*g* mm	170	185	200	280	310	335	360	390	420
Trumpet diameter	*h* mm	37	49	59	76	87	97	105	120	135
Bearing plate	*i* mm	140	170	200	235	270	300	330	380	430

Fixed anchorage type S



Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Fan length	*k* mm	460	550	660	830	880	960	1010	1060	1180
Anchor plate square	*l* mm	120	160	200	250	280	320	350	400	450
Anchor plate rectangular	*l* mm	70	90	120	140	160	180	200	240	260

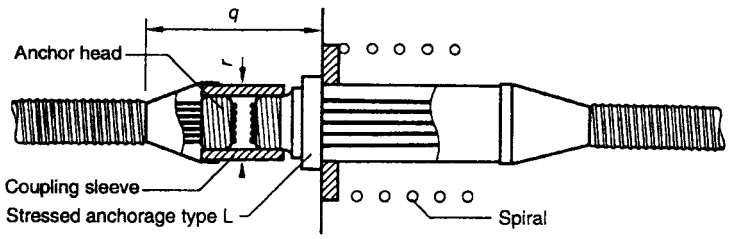
Fixed anchorage type F



Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Anchor	*m* mm	25	27	36	43	49	56	67	78	85
Trumpet length	*n* mm	170	185	200	280	310	335	360	390	420
Trumpet diameter	*o* mm	37	49	59	76	87	97	105	120	135
Bearing plate	*p* mm	140	170	200	235	270	300	330	380	430

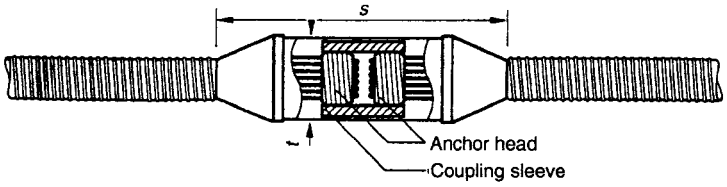
Figure D19. BBRV wire anchorages.

@Seismicisolation



Fixed coupling type LK

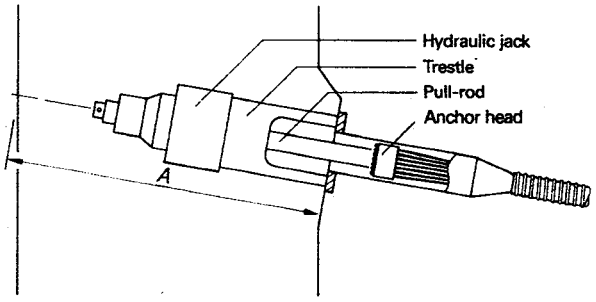
Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Trumpet length	*q* mm	230	260	290	350	410	430	470	570	630
Trumpet diameter	*r* mm	70	88	102	123	138	153	171	193	219



Movable coupling type LK1

Number of wires dia. 7 mm		7	12	19	31	42	52	61	82	102
Trumpet length min.	*s* mm	600	620	670	750	810	880	950	1080	1150
Trumpet diameter	*t* mm	70	88	102	123	138	153	171	193	219

Figure D20. BBRV wire couplers.



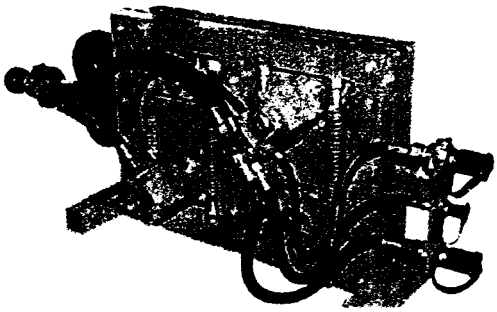
Type of stressing jack		NP 60	NP 100	NP 150	NP 200	NP 250	NP 300	GP 500
Max. jack force	kN	620	1030	1545	2060	2575	3090	5150
Jack diameter	mm	160	205	250	290	315	350	560
Jack stroke	mm	100	100	100	100	100	100	400
Jack weight	kg	28	50	83	117	147	196	1260
Space requirement *A*	mm	1700	1700	1700	2000	2000	2000	2500

Figure D21. BBRV wire jack details.

@Seismicisolation

Auxiliary equipment

The other equipment used for completing the installation of the tendons includes strand-pushing equipment, hydraulic pumps to operate the stressing jacks and the grouting equipment. Details of the DYWIDAG equipment used are given in Figs D22 to D24.

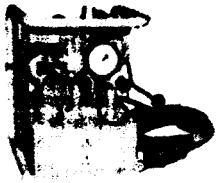


ESG 8-1

Type	Tensile or compression force (kN)	Pushing speed (m/s)	Weight (kg)	Dimensions $L \times W \times H$ (mm)	Hydraulic pumps
ESG 8-1	3·9	6·1	140	1400/350/510	ZP 57/28

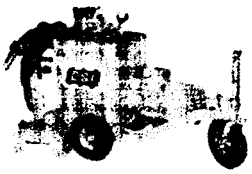
Figure D22. DYWIDAG strand-pushing equipment.

@Seismicisolation



77-159 A

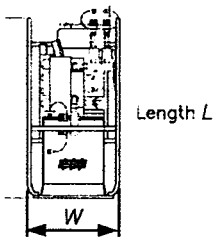
R 6.4

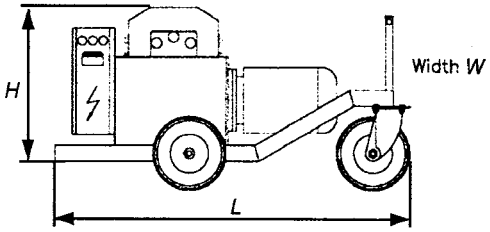


R 11.2–11.2/210

Jacks*	SM 200	HoZ 950	HoZ 1700	HoZ 3000	HoZ 4000/250	6800	9750	15 000
Pumps								
77-159 A	•	•						
77-193 A	•	•	•					
R 3	•	•	•					
R 6.4	•	•	•	•	•			
R 11.2–11.2				•	•	•		
R 11.2–11.2/210					•	•	•	•
ZP 57/28				For all pushing devices				

* For pistons without power seating



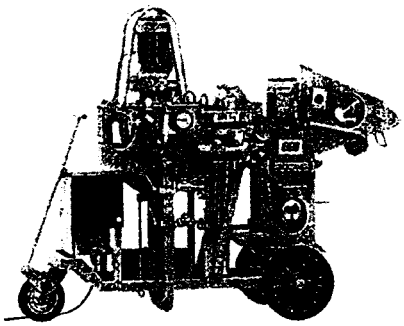


Technical data

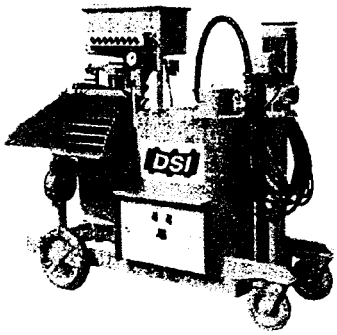
Pumps	Operation pressure (MPa)	Capacity, V min (l/min)	Eff. oil amount (l)	Weight (kg)	Dimensions $L \times W \times H$ (mm)
77-159 A	70	3·0	10·0	60	420/380/480
77-193 A	70	3·0	10·0	63	420/380/480
R 3.0	70	3·0	13·0	98	600/390/750
R 6.4	60	6·4	70·0	310	1400/700/1100
R 11.2–11.2/210	55	11·2/22·4	170·0	720	2000/800/1300
ZP 57/58	16/22	53/80	175·0	610	1260/620/1330

Figure D23. DYWIDAG hydraulic pump data.

@Seismicisolation



MP 2000-5



MP 4000-2

Grouting equipment	Maximum injection pressure (MPa)	Capacity (l/h)	Weight (kg)	Dimensions $L \times W \times H$ (mm)
MP 2000-5	1·5	420	60	2000/950/1600
MP 4000-2	1·5	1500	63	2040/1040/1750
P 13 EMRT	8·0	3000	98	2150/1750/1500

Figure D24. DYWIDAG grouting equipment.

@Seismicisolation

Company and person index

Page numbers in italics refer to diagrams and illustrations.

ADAPT Corporation
  ABI software 141–142, *143*
    ABI-Gen 150
    cable-stayed bridges 297–302, *297*, *298*, *299*, *300*, *301*
    input data 143–145
    output 145–146, *146*
    structural analysis 146–150, *147*, *148*, *149*
    traveler 145, *145*, 148–149, *148*

Baur, Willi 13
BBR
  BBRV, anchors 10
  prestressing systems 20, 21, Appendix D

Dyckerhoff, Eugen 9
DYWIDAG, post-tensioning systems 21, Appendix D

Freyssinet, Eugène 9, *9*, 10, 20–1
Freyssinet, prestressing systems 21, Appendix D

Lee-McCall, anchors 10
Leonhardt, Fritz 10, 13

Macalloy, prestressing bars 21
Magnel-Blaton, anchors 10
Magnel, Gustave 10
Mouchel, LG and Partners 10
Muller, Jean 10

STUP, 21

VSL, post-tensioning systems 20, Appendix D

Widmann, Gottlob 9

@Seismicisolation

Project index

Page numbers in italics refer to diagrams and illustrations.

Algeria
  Oued Fodda Bridge 9
Australia
  Gladesville Bridge 311, *311*
Austria
  Inn Bridge 13

Bahrain
  Saudi Arabia–Bahrain Causeway 250, *251*, 259
Brazil
  Rio de Peixe Bridge 12

Canada
  Confederation Bridge 250–251, *252*, 259–260, *259*
  Sherbrooke Footbridge 17, *18*

England
  A13 Viaduct 323
  A3/A31 Bridge 318
  Byker Viaduct 1, *1*, 206, *206*
  River Dee crossing 1, *2*
  Hammersmith Flyover 12–13
  Medway Bridge 12, *12*
  Northam Bridge 12
  Nunns Bridge 12
  Second Severn Crossing 323
  St James's Park footbridge 11–12, *11*

France
  Annet Bridge 9, *9*
  Boulonnais Bridge 309, *310*
  Brotonne Bridge 13, *14*, 284, 289
  Choisy-le-Roi 10
  Normandie Bridge 283
  Plougastel Bridge 9
  Vecchio Bridge, Corsica 310, *310*

Germany
  Aux Bridge, Saxony 9
  Lahn Bridge 12
  Oella Bridge 9
  Mainbrücke Stockstadt viaduct 321
  Mangfall Bridge 12

Hong Kong
  footbridges 312, *313*
  Hung Hom Bypass 14–15, *14*, 208, *208*
  Kwun Tong Bypass 179, *179*
  MTR Island Line 190, *190*

Ireland
  Analee Bridge 156, *157*
  Balbriggan Bridge 151, *151*
  Killarney Overbridge 2, *3*
  Kilmacanoque Footbridge 312, *313*
  M4 Overbridge 151, *152*

Japan
  Tsukuhara Bridge 307, *307*

India
  Yamuna cable-stayed bridge 290, *291*, 292
Indonesia
  Barelang Bridge 311–312, *311*

Korea
  Peace Footbridge 17, *18*
Kuwait
  Bubiyan Bridge 309, *309*
  Jahra Ghazali Viaduct 209, *209*

Malaysia
  Malaysia–Singapore Second Crossing 191, *191*

Northern Ireland
  Belfast Cross-Harbour Link 207, *207*

Pacific Islands
  Koror-Babelthaup Bridge 324

@Seismicisolation

Portugal
Vasco da Gama Bridge 15, *15*, 250, *251*, *258*, 259, 283, *283*, 285, 290, *290*, 322

Saudi Arabia
Wadi Leban Bridge 289, *289*
Scotland
Dornoch Firth crossing 13, *13*, 262, *262*
Singapore
footbridge 312, *312*
MRT 250, 257, *252*
South Africa
Injaka Bridge 314, 321
Switzerland
Sunniberg Bridge 15, *16*, 307–308, *307*

Taiwan
High Speed Rail Project 254, 258
Thailand
Bangkok SES viaduct 14, *156*, 207–208, *207*, *226*
Turkey
Bosporus Crossing *6*
Hasdel viaduct 262–263, *263*

UAE
Ghantoot Overbridge 157, *157*
USA
Barton Creek Bridge 308, *308*
Lake Pontchartrain Bridge 11
Linn Cove Viaduct 10–11, *11*
Long Keys Bridge 324
Mid-Bay Bridge 324
Pasco-Kennewick Bridge 284
Shelton Bridge 10
Sunshine Skyway Bridge 282–283, *282*, 284

Venezuala
Rafael Urdaneta Bridge 13
Rio Caroni Bridge 13

Wales
Ceiriog Viaduct 1, *2*
River Dee crossing 283, *284*, 285
Ynys-y-Gwas Bridge 17, 34–35, 46, 314, 324

@Seismicisolation

Subject index

Page numbers in italics refer to diagrams and illustrations.

abutments
  beam-and-slab bridges 162–163, *162*
  box girder bridges 42–43, *42*
access
  box girder bridges
    abutments 42–43, *42*
    internal 40–43, *41*, *43*
  decks 34, 40, *40*
aesthetics *126*, 127, 156
allowable stress design checks, concrete 93–94
anchorages
  BBRV 10
  blisters/blocks, reinforcement 104–106, *105*, *106*, *107*
  concrete failures 315
  external tendons 8, *8*
    diaphragms 114
  friction conical 10
  internal *7*, 8
  multi-strand 23–24, *23*
    dead-end 25, *26*
    live-end 25, *25*
  post-tensioned tendons 99–103, *100*
    dead-end 103, *104*
    reinforcement 101–103, *101*
  pre-tensioned strands 98–99, *99*
  recesses/pockets, reinforcement *105*, 106–107
  reinforcement
    blisters/blocks 104–106, *105*, *106*, *107*
    bursting 101, *101*
    dead-end 103, *104*
    equilibrium 102–103, *103*
    spalling 102
  slippage through 315
arch bridges 310–312, *311*

balanced cantilever bridges
  box girder bridges
    multi-cell 186, *186*
    precast full cantilever 259–260, *259*
    precast segmental *222*, 223, 229–231, *230*, *231*, *232*, 233–236, *233*, *234*, *235*, 248
    single-cell 197, *197*, *198*, 199–201, *199*, *200*, *201*
  cable-stayed bridges 287–288, *287*
  erection 128–129
  first 12, *12*
  temporary props 201, *201*
  truss decks 310, *310*
  travellers, form 199–200, *199*, *200*
bars
  couplers 27, *27*, 107–108
  pre-stressed 24, *24*
    precast beams 60–61, *61*
    relaxation losses 71
  pre-stressing jacks *32*, 33
  specifications *22*
beam-and-slab bridges *156*
  see also cable-stayed bridges; precast beam bridges; precast beams
  abutments 162–163, *162*
  beam spacing 160, *160*
  bearings 170
  continuous 176
    abutments 162–163, *162*
  deck slab
    casting 165, 172–173, *172*
    formwork 171–172, *172*
    reinforcement 177–178, *177*
    stresses *174*, 175
  edge beams 160, *160*
  erection gantries 170–171, *171*
  in situ 156–157, *157*
    falsework 163–164, *164*
  integral 176
    abutments 162, *162*
  self-launching trusses 164–165, *164*
  stress distribution
    creep 174, *174*

@Seismicisolation

deck *174*, 175
shrinkage 173, *174*, 175
transverse diaphragms 163, *163*
beams *see* precast beam bridges; precast beams
bearings
launch/permanent 271–272, *272*, 278, *278*, 279–280, *280*
pot 170
rubber 170
box girder bridges *2*
abutments, access 42–43, *42*
bending, shear lag 74–75, *74*
cable-stayed *see* cable stayed bridges
construction depths 121–122
curved 66–67
decks
in situ 122, *122*
launched 122
warping 78
diaphragms 110–112, *111*
end walls 113–114, *113*
intermediate 114, *114*
drainage 38, *39*
effective flange width 75–76, *76*
external tendons 62, *62*
finite element models 136, *137*
incrementally launched *see* incrementally launched box girder bridges
inspection access 40–43, *41*, *43*
joints, match-cast 122
launched
failures 321
stress analysis 146–147, *147*
multi-cell *see* multi-cell box girder bridges
post-tensioned *13*
first 13
precast full-length *see* precast full-length box girder bridges
precast segmental *see* precast segmental box girder bridges
precast segments 122, *123*, 125
single-cell *see* single-cell box girder bridges
tendon location 56–57
grout leakages 57
torsional stresses 81, 111
transverse analysis 135–136, *136*
use of 121

cable-stayed bridges *2*, 15, *15*, 125, *282*, *283*, *284*
beam-and-slab 284, *285*, 290, *291*, 292
cross-girders 292, *292*
box girders 124, *124*, 208, *285*
unit lifting 289, *289*
decks
bending moments 298–300, *298*, *299*
casting 287, *288*, 293, *294*
dynamic behaviour 302–303
reinforcement 293, *294*
design, analysis 296–298, *296*, *297*, *298*
early 13, *14*
erection
balanced cantilever 287, *287*
falsework 287–288, *288*
stay installation 300–302, *300*, *301*
stress analysis 148–150, *148*, *149*
temperature effects 288
extra-dosed 15, 306–308, *306*, *307*, *308*
pylons 284, 285–286
stays
anchorages 292, 293, 295, *295*, 303, *304*
arrangements 286, *286*
elasticity 296
geometry 296, *296*
multi-strand 286–287, 303
single planes *285*
stressing 288
twin planes 284, *285*
vibration 303
temporary loadings 303–305
carbon fibre reinforced tendons 18–19
collapses
post-strengthening 324
Ynys-y-Gwas Bridge 17, 34–35, 46, 314, 323
concrete
admixtures 88, 320
aggregate size 320
allowable stress design checks 93–94
compaction 37
compression
resistance 282
stresses 71–72
cover, inadequate 320–321
cracking 37–38
partial prestressing 82
plastic settlement 317
prevention 317, 318
spalling 317–318, *318*
thermal 317
creep
compressive stress 71–72
decks 76
prestress 66
deviator failures 318, *319*
elastic shortening 69–70
flexural strength, requirements 94
high-strength 17–18, *18*
honeycombing 318–320, *319*, *321*
horizontal shear 96
longitudinal shear 92
modulus of elasticity 88
partial prestressing 82–83, 88, 92
properties 1

@Seismicisolation

Subject index

concrete (*continued*)
quantities 132–133, *132*
shear capacity 95
shrinkage losses 72–73
steam curing 167
strengths 87–88, *87*
tendon anchorages 38
torsional capacity 95–96
ultimate shear resistance 90–91
ultimate torsional shear 91–92
construction, process and design 3–4
corrosion
electrically induced 43–44
external tendons 17, 35
galvanizing 17, 36
protection from 17, 26, 35–37, *36*
tendons 316–317, 324
costs
external tendons, construction 131–132
multi-span viaduct 131
optimized *131*
precast beam bridges 130
precast beams, repetition work 157
single-span bridge 130
couplers
failures 315
tendons 26, *27*, 107–108
cranes
see also gantries; lifting frames
barge 258, *258*
dropped beams 322–323, *322*
precast segments 219–221, 233
road *220*, 231, *231*
two-crane lifts 323
creep
compressive stress 71–72
decks 76
prestress 66
redistribution of moments 83–84, *83*
curved decks
articulation 128
post-tensioned tendons 66–67

de-stressing
external tendons 44
jacks 31, 33
dead load moments, frame analysis 135, *136*
decks
see also bridge types by name
abutments
continuous with 128
prestress 65, *65*
access 34
beam-and-slab bridges
casting 165, 172–173, *172*
formwork 171–172, *172*
reinforcement 177–178, *177*
box girder bridges
multi-cell 180–181, *180*
precast 247–248, *247*
single-cell 194, 196, *196*, 204–205
concrete creep 76
creep, redistribution of moments 83–84, *83*
curved
articulation 128
post-tensioned tendons 66–67
drainage 38, *39*
expansion joints 128
height, span length ratio 129
pre-camber 76
prestressing effects 88
reinforcement
longitudinal 96
transverse 96
segmental, match-cast *1*, 10
service loads, allowable stresses 73, 89
serviceability limit state stress checks 89
shear
capacities 79–81, *80*
construction effects 84
longitudinal 81–82, *82*
spans, ranges 2–3, 15, *16*, 120, *120*, 126, *126*, *127*
stress levels, checking 74–75, *74*
thermal expansion 84–85
stresses 85–86, *86*
torsional stresses 81
ultimate moment of resistance 89–90
vibration frequencies 76–77
design
and construction process 3–4
durability 4
excessive deflections 323–324
maintenance requirements 4
deviators
external tendons
concrete beam 115–116, *115*, *116*
concrete block 115–116, *115*, *116*
failures 318, *319*
steel 115, *115*, 117–118
stress analysis 118–119, *118*
diaphragms
anchorages, external tendons 114
box girder bridges 110–112, *111*
end walls 113–114, *113*
intermediate 114, *114*
external tendons, anchorages 114
transverse, precast beam bridges *111*, 112–113
drainage, decks 38, *39*
ducting
see also grouting
coupling *29*, 30
crossing 316

@Seismicisolation

curved 108–109, *109*
external 109–110, *110*
high-density polyethylene 28–30, *29*, 35, 37
friction, prestressing 67–68, *68*
internal
corrugated plastic 28, *28*, *29*
corrugated steel 55–56
placing 108
precast beams, post-tensioned 167
pressure testing 37
air 54–55
water 55–56
sizes *23*
tendons
curved effects 73, *73*
pull-through placing 31, *31*
push-through placing 30–31, *30*
durability, design for 4, 34

effective flange width
box girder bridges 75–76, *76*
precast beam bridges 75
elastic shortening, concrete 69–70
environmentally sensitive areas, bridging 10
equivalent load method, prestress moments *64*, 66, 137, 139
erection, redistribution of moments 83–84, *83*
expansion joints, bearings 128
external tendons
advantages 129–130
anchors 8, *8*
ban on 17, 35
corrosion protection 35, 36–37, *36*
costs, construction 131–132
de-stressing 44
deflections 79
deviators
concrete beam 115–116, *115*, *116*
concrete block 115–116, *115*, *116*
failures 318, *319*
steel 115, *115*, 117–118
diaphragms, anchorages 114
disadvantages 130
ducting, high-density polyethylene 28–30, *29*
movement joints 39–40
post-tensioned 4
protection 17, 26
replaceable 26
pre-tensioned 4
prestress
primary effects 63–64, *64*
secondary effects 63–66, *64*
removal, deck 74
replacement 44
extra-dosed bridges 15, 306–308, *306*, *307*, *308*

falsework
beam-and-slab bridges 163–164, *164*
box girder bridges 181, *182*, *184*, 185, 194, *195*
failures 321–322
overstressing 88
fin back bridges *306*, 308, *308*
flexural strength requirements, concrete 94
footbridges 312–313, *312*, *313*
stressed-ribbon 312, *313*

gantries 238
see also cranes; lifting frames
balanced cantilever erection 230–231, *230*, *231*
failures 323
overhead 236, *236*
span-by-span erection 247–248
underslung 236, *237*, *238*
grouting
admixtures 47–48
bleed water 50
ducts 37
grease 58
incomplete 316, *316*
leakages 56–57
mixes 47–48
mixing pans 50–51, *50*
pumping 57–58
machinery 50–51, *51*
standards 47
thixotropic 48
trials 53–54
ducting 54–56
fluidity 48, *49*, 50
venting 30, 51–53, *52*, 54–55
closing 57
crests 53
voids 316, *316*
wax 58

highway traffic, vibration frequencies 77

in situ multi-cell box girder bridges 125, 179, *179*
advantages 179
casting 185, *185*
construction joints 184
decks 180–181, *180*
design, grillage models 186–187, *187*, 189
diaphragms, transverse 180, *180*
disadvantages 180
erection
balanced cantilever 186, *186*
truss 186
falsework 181, *182*, *184*, 185

@Seismicisolation

in situ multi-cell box girder bridges (*continued*)
formwork 181–183, *182*, *183*, *184*
prestressing tendons 181, *181*, *184*, 185, 187, *188*
reinforcement 183, *183*, 187, *188*
sections 180–181, *180*, *181*
in situ single-cell box girder bridges 125, 190, *190*, *191*
decks
articulation 204–205
casting 194, 196, *196*, 205
reinforcement *196*
distortion 202
erection 190–191
balanced cantilever 197, *197*, *198*, 199–201, *199*, *200*, *201*
gantry supports 194, *195*
span-by-span *184*, 194
falsework 194, *195*
post-tensionsed tendons 193
prestressing
cantilever tendons 202, *203*
continuity tendons 202, *203*
transverse 202, 204, *204*
sections 192, *192*
shear lag 202
span/depth ratios 191
support struts 192–193, *193*
twin 192, *193*
uses 190
width 192–193, *192*, *193*
incrementally launched box girder bridges *262*, *263*
abutments 281
advantages 264
casting
deck 271, *271*
formwork 269–270, *269*, *270*
reinforcement 270–271, *270*
sequences 267–269, *268*
yards 267, *267*
concept design 122–123, *123*, 125
construction tolerances 279
decks
casting 271, *271*
sections 264–265, *264*
disadvantages 264
launching
alignment 274–275, *274*
bearings 271–272, *272*, 278, *278*, 279–280, *280*
bending moments 276, *276*, *277*, 278
noses 267, *268*, 275, *275*
pull strand jacks 273–274, *274*
push jacks 273, *273*
requirements 265
sequence 263–264, *263*
stay cables 275
stress analysis 278, *278*
support forces 279–281, *280*
prestressing, longitudinal 265, *266*, 267
span/deck depth ratios 265
tendons, anchorages *266*, 267
influence coefficient method, prestress moments 66, 139–40, *140*
internal tendons 4
advantages 130
alignment 215–216, *216*
ban 17, 35
corrosion protection 36, *36*
disadvantages 129–130
ducting 28–30, *28*, *29*
post-tensioned 4
pre-tensioned 4, 7–8, *7*

jacks
lifting *33*
frames 165–166, *166*
stressing
bars 31, *32*
multi-strand 31, *32*
single-strand *31*
joints
precast segments
concrete 74, 243
dry 206, 207, 226–228, *226*, *227*, *228*, 244–246
advantages 227
disadvantages 228, *228*
epoxy 206, 223–225, *223*, 243–244
match-cast 74, 122
shear keys 240–243

lifting, precast beams 169–171, *170*, *171*, 173, 322–323, *322*
lifting frames
see also cranes; gantries
precast segments 231, *232*, 233

maintenance requirements, design 4
match-cast segments *see* precast match-cast segments
materials, quantities 132–133, *132*
modulus of elasticity, concrete 88
moments
dead load, frame analysis 135, *136*
grillage models 135, *135*
motorways, elevated urban 12–13, 14–15, *14*
movement joints, leakage 39–40
multi-span viaducts 126–7, *127*
multiple-beam bridges, curved 67

obstructions, bridging over 157–158

@Seismicisolation

partial prestressing 82–83, 88, 92
post-tensioned tendons 7–8, *8*
anchorages 99–100, *100*
reinforcement 101–103, *101*
corrosion protection 36–37, *36*
precast beams 60, 61–62, *61*, *62*
stressing *8*
post-tensioning
definition 88
precast beams 60, 61–62, *61*, *62*, 167–168, *168*
single-cell box bridges 193
solid slab decks 152
pre-camber, decks 76
pre-tensioned tendons 6–7, *6*
anchorages 98–99, *99*
corrosion protection 36–37, *36*
precast beams 60, *61*
pre-tensioning, definition 88
precast beam bridges *3*, 125
see also beam-and-slab bridges
bending, shear lag 74–75, *74*
costs 130
decks slabs, *in situ* 121
diaphragms
half-depth 112, *113*
transverse 110–112, *111*
early 11–12, *11*
effective flange width 75
lifting, failures 322–323, *322*
use of 121
precast beams *3*
casting beds 165, *165*
casting yards 165, *165*, *166*, *167*
costs, repetition work 157
ladder arrangement 161–162, *161*
lifting 169–171, *170*, *171*, 173
failures 322–323, *322*
local overstressing 168
post-tensioned 60, 61–62, *61*, *62*, 167–168, *168*
ducting 167
tendon arrangement 176
pre-stressed bars 60–61, *61*
pre-tensioning 60, *61*, *159*, 162, 177, *177*
anchorages 98–99, *99*
jacking frames 165–166, *166*
shutters 166–167, *166*, *167*
sections 13, 121, *121*, 159–160, *159*, 160–161, *160*, *161*
spans
depth ratios 161
ranges *161*
stresses 5–6, *5*
transporting 169, *169*, 173
precast full-length box girder bridges *251*, *252*
advantages 251–252
applications 250
casting
formwork 254, *254*
yards 253–254
curing 254–256, *256*
decks
bearings 260–261, *260*
stitches *260*, 261
design 261
erection
balanced cantilever 259–260, *259*
crane 258–259, *259*
gantries 258–259, *259*
lifting gantries 256, *256*
prestressing tendons 253
reinforcement 254, *255*
spans 253
transport
barges 257–258, *258*
road 256–257, *257*
unit dimensions 253
precast match-cast segments
joints, compression 74
urban viaducts 14–15, *14*
precast segmental box girder bridges *206*, *207*, *208*
alignment
sections 216–218, *217*, *218*, 228–229, 235–236, *235*
tendons 215–216, *216*
applications 209
cable stayed 208
casting
cycles 213–214
formwork 210–211, *211*, *212*, 213, *214*
long-line beds 213, *214*
yards 210–211, *210*, *211*, *212*, 213, *213*
decks
bearings 233
erection 247–248
stitches 234, *235*
diaphragms *119*, 210
erection
balanced cantilever *222*, 223, 229–231, *230*, *231*, *232*, 233–236, *233*, *234*, *235*
progressive placing *222*, 223, 239–240, *239*
span-by-span *222*, 223, 236–239, *236*, *237*, *238*
history 206–207
jointing
dry 206, 207, 226–228, *226*, *227*, *228*, 244–246
epoxy 206, 223–225, *223*, 244–245

@Seismicisolation

precast segmental box girder bridges (*continued*)
prestressing
temporary 224–225, *224*
tendon layout 240, *241*, *242*
reinforcement 214–215, *215*
segments
lifting 219–221, *220*, 231, *231*, *232*, 247–248
storage 218–219, *219*, 323, *323*
transport 219, *219*, 221–222, *221*, *222*
shear capacity, ultimate 245–246, *246*
shear keys 210, 228, 240, 243, *243*, *244*
capacity 246, *246*
tendons, internal 215–216, *216*
top slab widths 208–209
web/slab thickness 209, *209*
weight 209
precasting, yards 6–7, *6*
prestress
definition 88
effects
computer analysis 137, *138*
secondary *64*, 137
systems, proprietary 20–21, Appendix D
prestress moments
equivalent load method *64*, 66, 137, 139
influence coefficient method 66, 139–140, *140*
prestressed bars, precast beams 60–61, *61*
prestressed beams, ultimate moment of resistance 77–79, *78*
prestressed bridges
design rules 17
post-war developments 10–12, 17
pre-war 9
prestressing
balanced cantilever construction 202, *203*
concrete, elastic shortening 69–70
definitions 4–5, *5*
extension 68–69, *69*
external tendons
primary effects 63–64, *64*
secondary effects 63–66, *64*
failures during 314–316
friction 67–68, *68*
moments
equivalent load method *64*, 66
Influence Coefficient method 66
partial 82–83, 88, 92
transverse 202, 204, *204*
progressive placing, box girder bridges, precast *222*, 223, 239–240, *239*

railways, bridging over 158, *158*
reactive powder concrete (RPC) 17–18
reinforcement
anchorages
blisters/blocks 104–106, *105*, *106*
bursting 101, *101*
equilibrium 102–103, *103*
spalling 102
box girder bridges
multi-cell 183, *183*, 187, *188*
precast 214–215, *215*
single-cell *196*
deck slab, beam-and-slab bridges 177–178, *177*
honeycombing 319–320, *319*
inadequate cover 320–321
longitudinal, torsional effects 96
quantities 132–133, *132*
transverse, torsional effect 96
relaxation losses, tendons 70–71, *70*

self-launching trusses, beam-and-slab bridges 164–5, *164*
service loads, decks, allowable stresses 73
serviceability limit state stress checks (SLS) 89
shear
capacity
concrete 95
decks 79–81, *80*
construction effects 84
decks, longitudinal 81–82, *82*
grillage models 135, *135*
horizontal, concrete 96
longitudinal 92
ultimate resistance 90–91
ultimate torsional 91–92
shear keys 210, 228, 240, 243, *243*, *244*
capacity 246, *246*
shear lag, decks, box girder bridges 74–75, *74*, 202
slab bridges *see* beam-and-slab bridges; solid slab bridges; voided slab bridges
software
input data 143–145
output 145–146, *146*, 150
structural analysis 146–150, *147*, *148*, *149*
traveller 145, *145*, 148–149, *148*
three-dimensional finite-element 134
solid slab bridges 125, 151, *151*
analysis, grillage models 154–155
decks, post-tensioning 152
in situ 152
span-by-span erection
box girder bridges
precast 236–239, *236*, *237*, *238*, 248
single-cell *184*, 194
spans
length, deck height ratio 129
ranges 2–3, 15, *16*, 120, *120*

@Seismicisolation

stage analysis, cumulative 141
stressed-ribbon footbridges 312, *313*
stressing jacks
  bars *32*, 33
  de-stressing action 31, 33
  multi-strand tendons 31, *32*, *33*
  single-strand tendons 31, *31*, 33

tendons
  see also external tendons; internal tendons; wires
  anchorages, internal *7*, 8
  breaking loads *23*, 24
  carbon fibre reinforced 18–19
  concrete shrinkage effects 72–73
  corrosion 324
  couplers 26, *27*, 107–108, 315
  curved 318
  ducting
    curved effects 73, *73*
    friction 67–68, *68*
    snagging in 315–316
  elastic shortening 69–70
  extension 315–316
  failures 315
  multi-strand
    anchorages 25, *25*, *26*
    sizes 23–24, *23*
    stressing jacks 31, *32*, *33*
  post-tensioned 7–8, *8*
    corrosion protection 36–37, *36*
    precast beams 60, 61–62, *61*, *62*
    pull-through placing 31, *31*
    push-through placing 30–31, *30*
    stressing *8*
  pre-tensioned 6–7, *6*
    corrosion protection 36–37, *36*
    precast beams 60, *61*
  prestress
    constraints 62–63
    primary effects 63–64, *64*
    secondary effects 63–66, *64*
  prestressing, extensions 68–69, *69*, 315–316
  relaxation losses 70–71, *70*
  single-strand, stressing jacks 31, *31*, 33
  wires 6
thermal expansion
  decks 84–5
    stresses 85–86, *86*
torsional capacity, concrete 95–96
torsional stresses
  box girder bridges 81, 111
  decks 81
transport
  precast units 169, *169*, 173
    barges 221–222, *222*, 257–258, *258*
    rail 221, *221*
    road 219, *219*, 256–257, *257*
truss bridges 12, 309–310, *309*, *310*
truss construction, multi-cell box bridges 186

ultimate moment of resistance 77–9, *78*, 89–90
ultimate shear resistance 90–91
ultimate torsional shear 91–92
urban viaducts 12–13, 14–15, *14*, 250, *252*

viaducts, multi-span 126–127, *127*
vibration frequencies
  decks 76–77
  stays 303
voided slab bridges 125, 151–152, *152*
  advantages 154
  analysis, grillage models 154–155
  in situ 152–153, *153*
  prestress tendons 153
  void formers 153–154, *153*, *154*

wires
  see also tendons
  breakages 314–315
  early standards 10
  galvanized 21, 36
  specifications 21, *22*

@Seismicisolation

PRESTRESSED CONCRETE BRIDGES: DESIGN AND CONSTRUCTION

Nigel R. Hewson

Prestressed concrete decks are commonly used for bridges with spans between 25 m and 450 m as they provide economic, durable and aesthetic solutions in most situations where bridges are needed.

Prestressed Concrete Bridges: Design and Construction clearly explains the principles behind the design and construction of prestressed concrete bridges, illustrating the interaction between the two. The book covers all the different types of deck arrangement and construction techniques used, ranging from in situ slabs and precast beams, segmental construction and launched bridges, and cable-stayed structures.

This book describes each of the different types of deck commonly used and looks at the associated analysis and design process. Detailed descriptions of the prestressing components and systems used are also included.

This book is an essential reference for both the experienced engineer and graduate who want to learn about prestressed concrete bridges.

Nigel R. Hewson, with over 25 years of experience, is a recognised authority on the design and construction of prestressed bridged. Nigel has been involved with many major bridge projects and has been responsible for the design of Malaysia-Singapore Second Crossing, Hung Hom Bypass in Hong Kong, Belfast Cross Harbour Links, Bangkok Second Stage Expressway and many other highway and rail bridges. He has recently worked on a number of highway schemes in the UK and led a design team on the Taiwan High Speed Rail project.

Nigel, based in Surrey with Tony Gee and Partners, has written a number of articles and papers on prestressed concrete bridges and is an Associate Lecturer at the University of Surrey on this subject as well as on cable-stayed and suspension bridges. He is currently an active member of the Concrete Bridge Development Group in the UK.

ISBN 07277-3223-4

9 780727 732231

Thomas Telford

@Seismicisolation

Copyrighted material