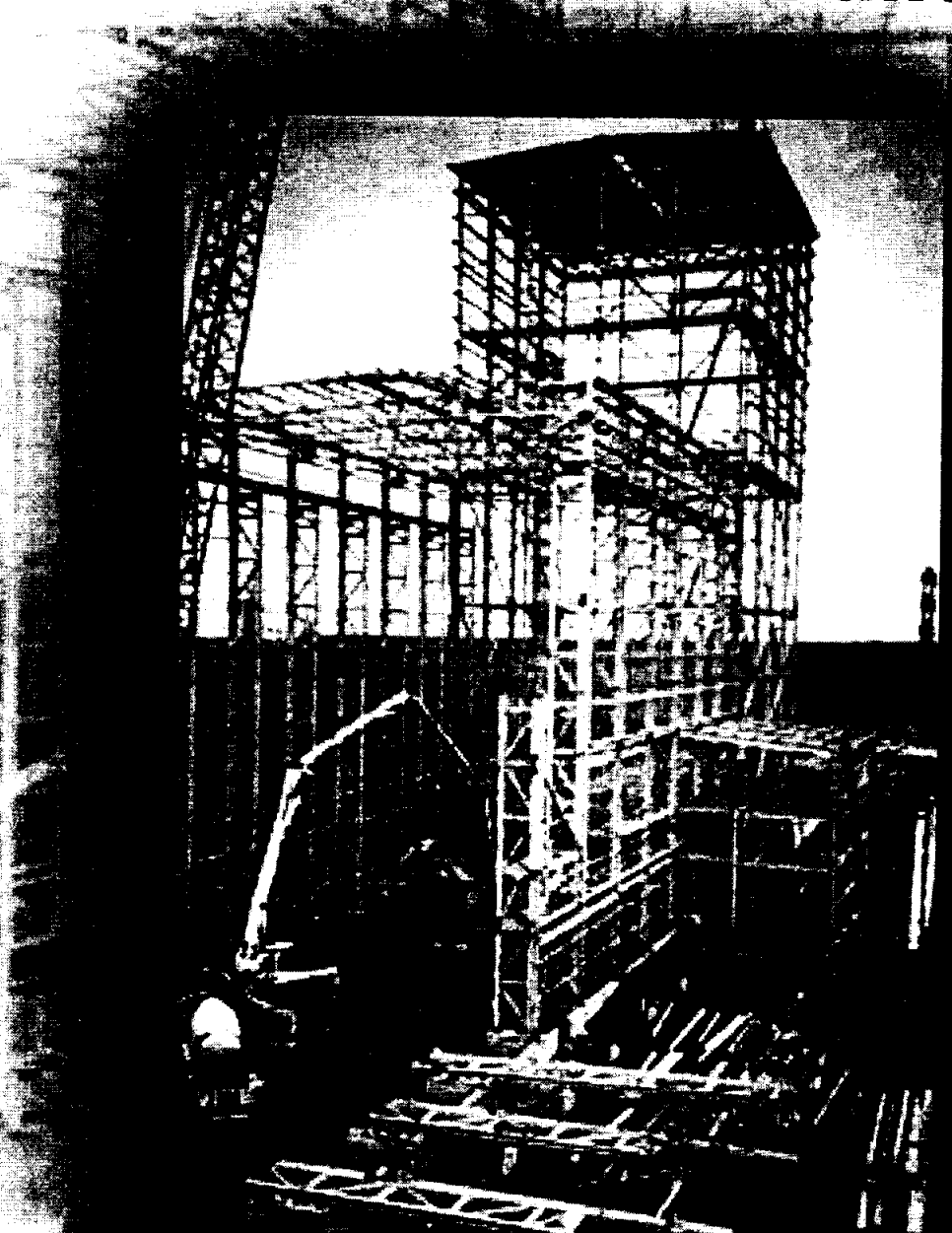
Guide for the Design and Construction of Mill Buildings



AISE Technical Report No. 13

2003

DISCLAIMER

This report has been prepared by a committee of steel company representatives, the Association of Iron and Steel Engineers, and others, who considered the technology available at the time of preparation. This report does not represent either minimum acceptable standards or mandatory specifications. In addition, this report is subject to compatibility with all governmental requirements.

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The use of language in this report that might be construed as mandatory is intended only to preserve the integrity of the report as the committee views it. It is not intended to require strict compliance where not necessitated by safety or operational needs.

FOREWORD

In 1969 the Association of Iron and Steel Engineers first published "Specifications for the Design and Construction of Mill Buildings." *AISE* recognized the need to consolidate available information and to guide designers, contractors, owners and suppliers as to the building requirements of the steel and similar industries. It was revised in 1979, 1991, 1997, and here again in 2003. As originally stated in 1969, the purpose then as now is:

> This specification provides owners, engineers and contractors with a comprehensive and rational approach to the design and construction of mill buildings, and other buildings or structures having related or similar usage.

After review and confirmation of the scope of this Technical Report No. 13, the previous contents of Section 6.0 have been deleted.

This updated report guides the owner and designer through the many assumptions and parameters involved in the design of a mill building. It suggests loads and load combinations for the design of crane runways, roof structures, floors, columns, building frames and foundations.

Information is given for investigation, earthwork and excavation requirements as in the 1979 edition, as well as revisions to vibration, foundations, soil bearing foundation, crane rails and crane rail splices.

All of this information has been reviewed and updated to the current state-of-the-art procedures for design. However, latitude has been provided for even more advanced proven techniques.

All information and direction is within the requirements of national codes and specifications. A listing of many references (also revised) is provided.

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Technical Report No. 13
Guide for the Design and Construction of Mill Buildings

AISE Subcommittee No. 13 on Design and Construction of Mill Buildings was established in 1962.

The Technical Report No. 13 represents an ongoing process of utilizing traditional information and incorporating new techniques, standards and products as they become available to provide guidelines for the design, fabrication, construction and maintenance of mill buildings.

The guide is organized into six sections and three appendices covering general requirements, geotechnical investigation, loads and forces, foundations, floors and walls, and structural steel.

Many thanks to the following members of Subcommittee No. 13 on Design and Construction of Mill Buildings who dedicated their time and knowledge to the revision of the 2003 edition:

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Table of Conte

1.0 General

1.1 Purpose 1
1.2 Scope 1
1.3 Building Codes, Standards and References 1
1.4 Classifications of Structures 1
1.4.1 Mill Buildings, Class A 1
1.4.2 Mill Buildings, Class B 1
1.4.3 Mill Buildings, Class C 1
1.4.4 Mill Buildings, Class D 1
1.5 Engineering Drawings and Details 2
1.5.1 Design Drawings 2
1.5.2 Design Analyses 2
1.5.3 Sealed Drawings 2
1.5.4 Project Record Drawings 2
1.5.5 Detail Drawings 2
1.5.5.1 Structural Steel 2
1.5.5.2 Concrete Reinforcing Steel 2
1.5.6 Equipment Installation, Safety, Maintenance and Repair 2
1.5.7 Clearances 4
1.5.7.1 Crane Clearance, Related Dimensional and Load Information 4
1.5.7.2 Miscellaneous Clearances 4

2.0 Investigation, Earthwork and Excavation

2.1 Purpose 6
2.2 Earthwork 6
2.2.1 Project Specification 6
2.2.2 Excavations—Foundations 6
2.2.2.1 Safety 6
2.2.2.2 Support 6
2.2.2.3 Braced and Open Cut Excavations 6
2.2.3 Protection of Foundation Stratum During Construction (Unless Special Studies Are Made) 6
2.2.4 Dewatering 6
2.2.5 Backfilling Foundations 7
2.2.5.1 Steelmaking Slags 7
2.2.5.2 Resistant Rock Materials 7

3.0 Loads and Forces

3.1 Dead Load 8
3.2 Roof Live Loads 8
3.3 Floor Live Loads 8
3.3.1 Recommended Minimum Live Loads 8
3.3.2 Live Load Reduction Factors 8
3.4 Crane Runway Loads 8
3.4.1 General 8
3.4.2 Vertical Impact, Side Thrust and Traction 9
3.4.3 Runway Crane Stops 9

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3.5 Moving Loads 9
3.5.1 Limited Access Vehicles 10
3.5.1.1 Loads and Impacts Due to Railway Equipment 10
3.5.1.2 Nonstandard Gage Equipment 10
3.5.2 Unlimited Access Vehicles 10
3.6 Contingency Loads 10
3.7 Special Loads 10
3.7.1 Guidelines for Vibratory Loading 10
3.7.2 Conveyor Unbalanced Forces 11
3.7.3 Utility Support Loads 11
3.7.4 Special Roof-Supported Structures 11
3.7.5 Loads from Mains, Ducts and Pipes 11
3.8 Wind Loads 11
3.9 Seismic Loads and Displacements 12
3.10 Load Combinations for Design of Crane Runways and Supporting Structures 12
3.10.1 Symbols and Notations 12
3.10.2 Basis of Design 12
3.10.2.1 Case 1 13
3.10.2.2 Case 2 13
3.10.2.3 Case 3 13
3.10.2.4 Other Load Combinations 13
3.11 Loads on Retaining Walls, Grade Walls and Grade Beams 13
3.11.1 Earth Pressure 13
3.11.2 Vertical Loads 13
3.11.3 Supplemental Loads 13
3.12 Loads on Building Foundations 13
3.12.1 Loads Combinations 14
3.12.1.1 Condition 1 14
3.12.1.2 Condition 2 14
3.12.1.3 Condition 3 14
3.12.1.4 Condition 4 14
3.12.1.5 Condition 5 14

4.0 Foundations, Floors and Walls

4.1 General 15
4.2 Concrete Construction 15
4.2.1 Design and Construction 15
4.2.2 Concrete Strength 15
4.2.3 Setting Anchor Rods 15
4.2.4 Grouting of Base Plates 15
4.3 Soil Bearing Foundations 15
4.3.1 General 15
4.3.2 Ground Water Conditions 15
4.3.3 Effect on Other Structures 15
4.4 Pile and Caisson Supported Foundations 16
4.4.1 General 16
4.4.2 Allowable Pile and Caisson Stresses 16
4.4.3 Splices 17

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4.4.4 Special Provisions for Caisson and Pile Caps 17
4.4.5 Field Control of Pile Driving 17
4.4.5.1 Driving 17
4.4.5.2 Plumbness 18
4.4.5.3 Records 18
4.4.5.4 Load Tests 18
4.5 Retaining and Basement Walls 18
4.5.1 General 18
4.5.2 Stability Criteria 18
4.5.3 Provision for Drainage and Hydrostatic Pressure 18
4.6 Floor Slabs on Grade 18
4.6.1 Design Procedure 18
4.6.2 Subgrade Modulus 18
4.6.3 Subgrade Preparation 18
4.6.4 Vapor Retarder 18
4.6.5 Construction and Control Joints 18
4.6.6 Temperature and Shrinkage Reinforcement 19
4.6.7 Expansion Joints 19
4.6.8 Steelmaking Slag Subgrade Material 19
4.6.9 Resistant Rock Subgrade Material 19

5.0 Structural Steel
5.1 General 20
5.2 Mill Building Framing 20
5.3 Framing Analysis and Drift 20
5.4 Roof Trusses 20
5.5 Bracing System 20
5.6 Expansion Joints 21
5.7 Allowable Stress Range Under Repeated Loads 21
5.8 Crane Runway Girders 21
5.8.1 General 21
5.8.2 Stress Calculations 22
5.8.2.1 Rolled Shapes and Built-up Single Web Plate Girders Having an Axis of Symmetry in the Plane of their Web 22
5.8.2.2 Girders With Back-Up Bracing Systems 22
5.8.2.3 Box Girders With Transverse Diaphragms 23
5.8.3 Web Thickness 23
5.8.4 Bottom Flange Bracing 23
5.8.5 Stiffeners 24
5.8.6 Local Wheel Support 25
5.8.7 Deflection 25
5.8.8 Girder Camber 25
5.8.9 Attachments 25
5.9 Columns 25
5.9.1 General 25
5.9.2 Brackets 25
5.9.3 Column Bases 26
5.10 Floor Framing 26

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5.11 Side Wall and Roof Framing 26
5.12 Depth Ratio 26
5.13 Minimum Thickness of Material 26
5.14 Connections 26
5.15 Spacing of Bolts and Welds 26
5.16 Crane Rails and Joints 27
5.16.1 Bolted Rail Joints 27
5.16.2 Welded Rail Joints 27
5.16.3 Rail Clips, Clamps or Attachments 27
5.16.4 Elastomeric Crane Rail Pads 27
5.17 Inspection and Quality of Welds 27
5.17.1 General 27
5.17.2 Welds on Crane Runway Girders 27
5.17.3 Other Inspections 27
5.17.4 Nondestructive Testing of Other Welds 27
5.18 Tolerances 28
5.18.1 Column Base Lines 28
5.18.2 Anchor Rods 28
5.18.3 Base Plates 28
5.18.4 Column Fabrication Tolerances 28
5.18.5 Crane Runway Girder Fabrication Tolerances 28
5.18.5.1 Crane Girders 28
5.18.5.2 Girder Ends 29
5.18.5.3 Girder Depths 29
5.18.6 Crane Girder and Rail Alignment 29
5.18.7 Tolerances 29

6.0 Miscellaneous (Deleted) 30

7.0 Commentary
7.1 Purpose 30
7.2 Classification of Structures (1.4) 30
7.3 Clearances (1.5.7) 30
7.4 Roof Live Loads (3.2) 30
7.5 Crane Runway Loads (3.4) 30
7.5.1 General (3.4.1) 30
7.5.2 Vertical Impact, Side Thrust and Traction (3.4.2) 30
7.5.3 Crane Runway Stops (3.4.3) 30
7.6 Vibration (3.7.1) 31
7.7 Wind Loads (3.8) 32
7.8 Seismic Forces (3.9) 32
7.9 Load Combinations for Design of Crane Runways and Supporting Structures (3.10) 33
7.9.1 Case 1 Load Combinations (3.10.2.1) 33
7.9.2 Case 2 Load Combinations (3.10.2.2) 33
7.9.3 Case 3 Load Combinations (3.10.2.3) 33
7.10 Soil Bearing Foundations (4.3) 33
7.11 Expansion Joints in Floor Slabs on Grade (4.6.7) 33
7.12 Column and Truss Bents (5.9.1) 33

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7.13 Building Expansion Joints (5.6) 34
7.14 Allowable Stress Ranges Under Repeated Loads (5.7) 34
7.15 Crane Runway Girders (5.8) 34
7.15.1 Rolled Shapes and Built-up Single Web Plate Girders Having an Axis of Symmetry in the Plane of Their Web (5.8.2.1) 34
7.15.2 Unsymmetrical Built-up Members and Closed Section Girders Without Diaphragms along the Length (5.8.2.2 and 5.8.2.3) 34
7.15.3 Bottom Flanges Bracing (5.8.4) 36
7.15.4 Stiffeners (5.8.5) 36
7.15.5 Load Wheel Support (5.8.6) 36
7.16 Columns (5.9) 36
7.16.1 Columns with a Continuous Web Plate Between Building and Crane Column Elements 38
7.16.2 Laced or Battened Columns 40
7.17 Crane Rails and Joints (5.16) 40
7.18 Mains, Ducts and Pipes (3.7.5) 41

8.0 Symbols 58

9.0 References 61

Appendix A
Geotechnical Investigation and Foundation (GIF) Manual 63

Appendix B
Guidelines for the Preparation of Specification for Subsurface Boring and Soil Sampling 73

Appendix C
Recommended Practice for Inspecting and Upgrading of Existing Structures 86

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1.0 General

1.1 Purpose

This report provides owners, engineers and contractors with a rational approach to the design and construction of mill buildings and other buildings or structures having related or similar usage. The report is intended to be a guide for the purchase, design and construction of such units, with the objective that they will be functional, serviceable, economical and safe. Before adapting this report to a particular project, each section should be reviewed for applicability and compatibility with other requirements and regulations (see disclaimer).

1.2 Scope

Design information in this report covers Class A, B, C and D mill buildings as defined in Section 1.4. Reference is made to other design guides, including codes, specifications and manuals, wherever it is deemed appropriate. Information regarding proper site investigations and economical substructure design is included.

1.3 Building Codes, Standards and References

All design and construction shall comply with applicable municipal, state and federal regulations and codes. It is recommended that all building permits be obtained by the owner unless otherwise specified.

1.4 Classification of Structures (7.2)

Classification of structures shall be based primarily on the number of cycles of crane loadings or repetition of a specific loading case anticipated for portions of the structure. The owner must analyze the service and determine which loading condition applies. On the basis of expected service life and rate of load repetitions, the owner shall specify the classification for all or any portion of a building. A service life of 50 years is generally recommended. See Table 5.1 for loading conditions and number of load cycles to establish a 50-year life.

1.4.1 Mill Buildings, Class A. Buildings in this category are those in which members might experience either 500,000 to 2,000,000 repetitions (Loading Condition 3) or over 2,000,000 repetitions (Loading Condition 4) in the expected service life of the structure. It is recommended that the following building types be considered as Class A:

- Batch annealing buildings.
- Billet yards.
- Continuous casting buildings.
- Foundries.
- Mixer buildings.
- Mold conditioning buildings.
- Scarfing yards.
- Coil handling.
- Scrap yards.
- Skull breakers.
- Slab yards.
- Soaking pit buildings.
- Steelmaking buildings.
- Stripper buildings.
- Other buildings (as based on predicted operational requirements).

1.4.2 Mill Buildings, Class B. Buildings in this category are those buildings in which members experience 100,000 to 500,000 repetitions of a specific loading during the expected service life of the structure.

1.4.3 Mill Buildings, Class C. Buildings in this category are those buildings in which members experience 20,000 to 100,000 repetitions of a specific loading during the expected service life of the structure.

1.4.4 Mill Buildings, Class D. Buildings in this category are those buildings in which no member will experience more than 20,000 repetitions of a specific loading during the expected service life of the structure.

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1.5 Engineering Drawings and Details

1.5.1 Design Drawings. Design drawings shall include complete design criteria, loads, pertinent moments, shears and reactions in girders, beams and columns, forces in trusses and the size and specification (ASTM or equivalent) of all material.

If cambering of trusses, beams and girders is required it shall be indicated. Design drawings for column bases and anchorages shall indicate all information necessary for foundation design, such as direct loads, moments, shears and uplift.

Allowable bearing pressure, pile loads, pile type, depth and load test results assumed in foundation design shall be indicated. The design drawings shall show sufficient typical details so that detail drawings can be executed without difficulty or ambiguity. The typical details shall be sufficient to show the type of connection to be used (i.e., high-strength bolts or welds).

Design drawings, general arrangement drawings, clearance diagrams and erection procedure drawings shall be sent to the owner for approval.

1.5.2 Design Analyses. Design computation sheets shall be furnished so that, together with drawings, the completed engineering analyses of all portions of the work are provided. These computation sheets shall be furnished with the design drawings when submitted to the owner for approval.

1.5.3 Sealed Drawings. Design drawings and design analyses, when engineered by any group other than the owner, shall be sealed by the registered professional/structural engineer of record.

1.5.4 Project Record Drawings. When required, the owner shall be furnished a set of reproducible project record drawings, as determined by a final survey of the alignment and elevation of the crane runway girders and columns.

Except as otherwise specified by the owner, the following shall be included:

(1) The location of the building in relation to adjacent property.
(2) The location of permanent benchmarks.
(3) Plumbness of steel work at elevations specified by the owner.
(4) Center-to-center span between runway girders at supporting columns and at mid-span of girders.
(5) Any changes to design shall also be recorded on project record drawings.

1.5.5 Detail Drawings

1.5.5.1 Structural Steel. Such drawings shall be prepared and approved in accordance with AISC specifications (Ref. 1) and with the AISC Code of Standard Practice (Ref. 2).

1.5.5.2 Concrete Reinforcing Steel. These drawings shall be prepared and approved in accordance with the ACI Building Code Requirements for Reinforced Concrete Structures (Refs. 5 and 6).

1.5.6 Equipment Installation, Safety, Maintenance and Repair. Provision should be made for convenient installation, maintenance and removal of equipment. Care should be taken in the design to not preclude parts of the structure from cleaning and painting. The owner shall supply sufficient information so that provision may be made for mounting equipment, piping, and electrical conduits and trays where located in the building structure.

Walkways, platforms, stairs or ladders should be designated to provide for the maintenance of equipment in inaccessible areas. Stairs rather than ladders are preferred where practicable. Provide fall protection and fall restraint in accordance with OSHA or other local authority.

Repair platforms should be included in building designs to accommodate track wheel changes on EOT cranes.

Escape walkways should be included in building designs to permit emergency exits from crane cabs on hot metal cranes.

Overhead trolley hoists or lifting beams in the roof structure should be provided at locations designated to allow for changing of heavy parts of cranes. Capacities of lift beams and permissible loads at hoisting points for maintenance and repair shall be included in the final design drawings and displayed on the structure.

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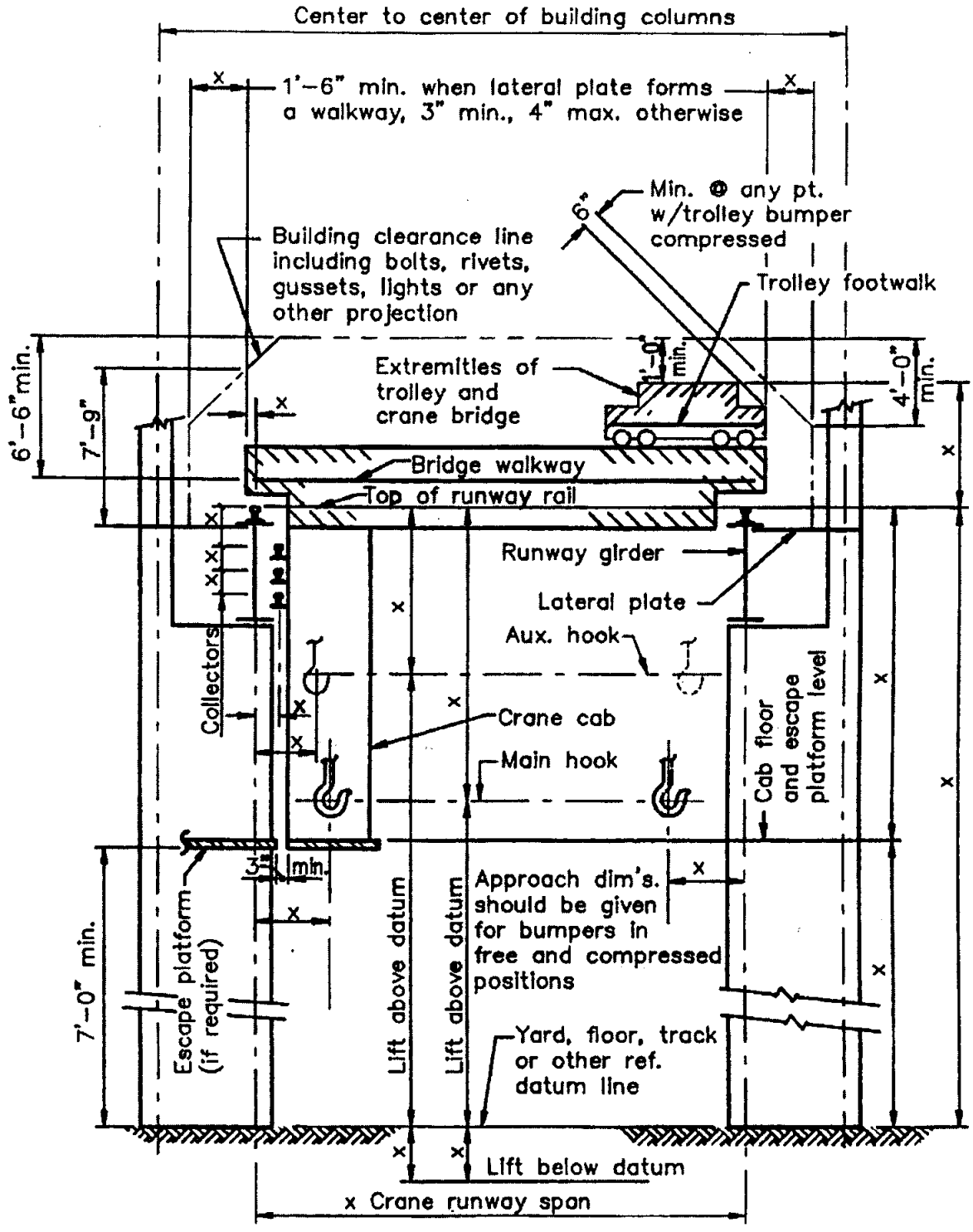

Fig 1.1 — Elevation: Typical crane runway clearance diagram.

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1.5.7 Clearances (7.3)

1.5.7.1 Crane Clearance, Related Dimensional and Load Information. Minimum clearances and required dimensional information are illustrated in Figs. 1.1 and 1.2. A typical crane bridge wheel load and dimension diagram is shown in Fig. 1.3.

It shall be the responsibility of the owner to furnish the following information:

(1) Dimensions marked (x) in Figs. 1.1, 1.2 and 1.3.
(2) Bridge wheel loads.
(3) Weight of trolley.
(4) Total weight of crane.
(5) Bridge and trolley speed.
(6) Cab clearances.
(7) Bridge bumper forces.
(8) Lifted load.
(9) Location of collectors, cable or festoon system.
(10) Lifts, if any, required below floor level.
(11) Desired cab location and elevation of cab floor to suit escape platform (if required), auxiliary access locations, platforms, stairs and ladders.
(12) Size of runway rail, in accordance with *AISE* Technical Report No. 6 (Ref. 11).
(13) Types of cranes.

1.5.7.2 Miscellaneous Clearances. Minimum clearance for medium- or high-voltage cables shall be in accordance with governing codes.

Rail, roadway or snowplow clearances affecting building design shall conform to the standards in railway and highway bridge design specifications (Refs. 14 and 15). Other clearances should be supplied as specified by the owner.

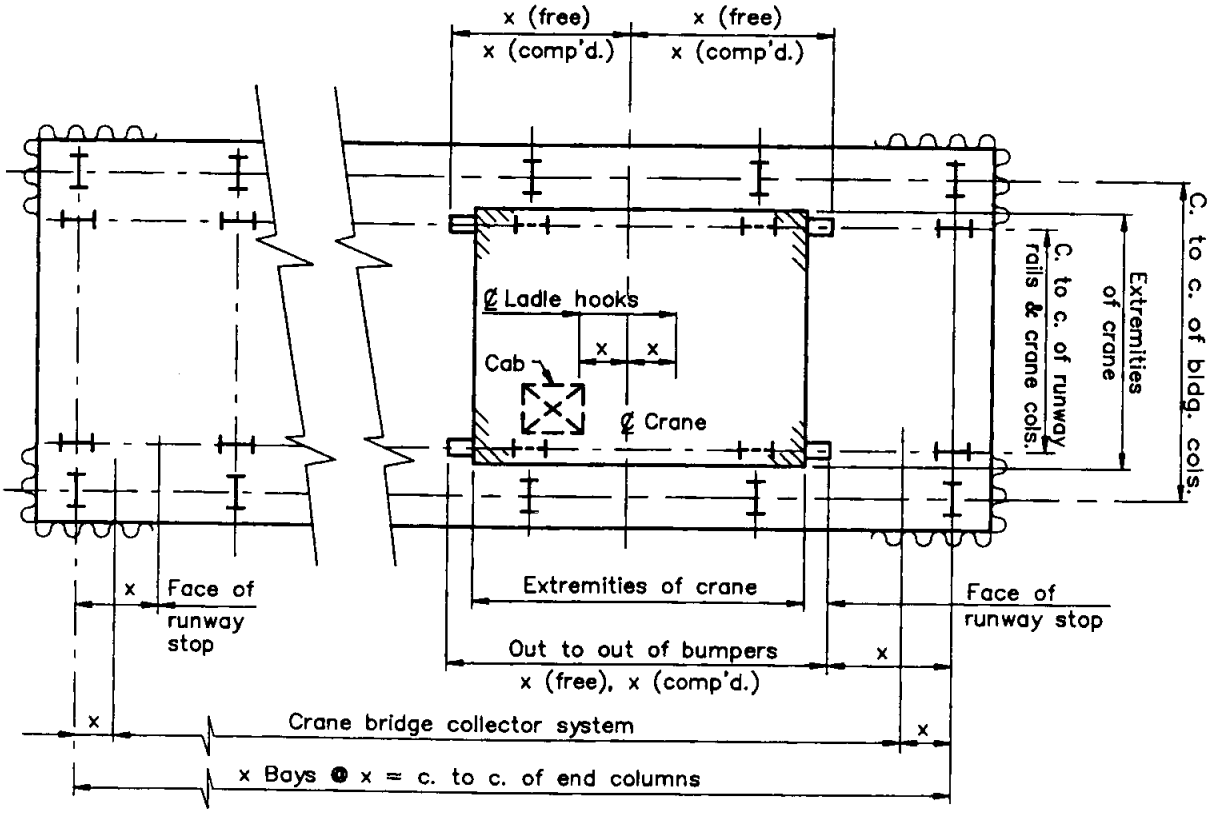

Fig. 1.2 — Plan: Typical crane runway clearance diagram.

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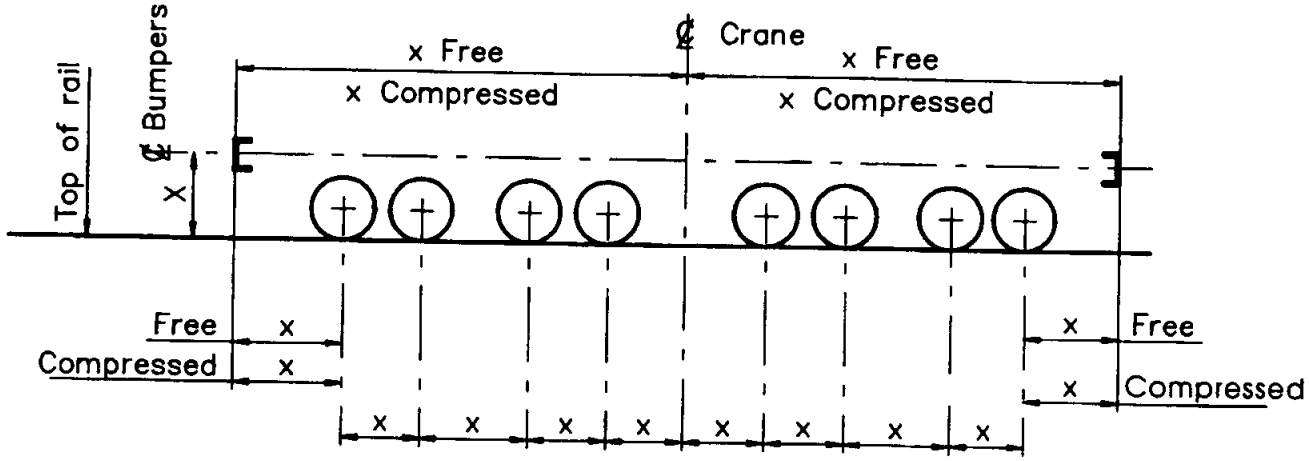

Max. bridge wheel load = x
Min. bridge wheel load* = x
Trolley weight = x
Total crane weight = x

*Bridge wheel load which occurs simultaneously with max. wheel load on opposite side of bridge.

Fig. 1.3 — Typical crane bridge wheel load diagram.

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2.0 Investigation, Earthwork, and Excavation

2.1 Purpose

After a site is considered satisfactory and feasible for use, surface and subsurface exploration, soil drilling and sampling, rock coring and field-testing should be conducted to determine:

(1) Foundation design criteria.
(2) Earthwork design criteria.
(3) Lateral soil pressures for the design of walls.
(4) Subgrade properties for the design of floor slabs on grade.
(5) Recommendations for special and complex soil problems.
(6) Water table.
(7) The electrical and chemical properties of soil to ensure durability issues of in-ground structures. Simple metal scan for selected parameters may be used.
(8) Site classification for seismic design.

The site investigation should be performed and coordinated by a geotechnical engineer in accordance with Appendix A. The requirements and recommendations for the different stages of site investigation are to be applied basically to new sites. Where reliable information complying with Appendix A is available and has been previously secured by the owner, only those additional parts of the investigation needed for the design and construction of the project should be performed. The results of the site investigation along with related design criteria should be published in the Project Geotechnical Report as recommended in Appendix A.

2.2 Earthwork

2.2.1 Project Specification. The owner should furnish specifications in accordance with Appendix A (Section A 2.0) for:

(1) Site clearing.
(2) Embankment construction.
(3) Grading.
(4) Excavations.
(5) Backfilling.

2.2.2 Excavations—Foundations

2.2.2.1 Safety. All excavations shall be conducted and maintained to prevent injuries to the public and to workers, in accordance with all provisions of local, state and federal regulations.

2.2.2.2 Support. All excavations shall be performed in a manner that will prevent movement of earth of adjoining sites and structures thereon, including floor slabs, pavements and foundations, utility lines, etc. Where danger of undermining adjoining foundations of structures exists, lateral support, underpinning for the foundations, or both, shall be provided.

2.2.2.3 Braced and Open Cut Excavations. Unless soil conditions require braced excavations, all open cut excavations shall be performed with adequate safety factors to maintain stable slopes during the construction period and in accordance with design criteria furnished in the Project Geotechnical Report. Soil data developed as described in Section A 2.0 shall be furnished by the owner.

In rock excavations, all loose and overhanging rock shall be removed.

2.2.3 Protection of Foundation Stratum During Construction (Unless Special Studies Are Made). Care shall be taken to prevent disturbance to the bearing stratum due to overexcavation, construction traffic, freezing and water movements.

2.2.4 Dewatering. When the ground water level occurs at an elevation that affects the bearing capacity or the stability of the foundation, a dewatering system shall be installed in accordance with the recommendations in the Project Geotechnical Report. Where dewatering is required, the ground water level may be allowed to rise

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after placement of the foundation, provided that it is kept at a level of at least 3 ft. below the top of the compacted backfill during placement of backfill.

2.2.5 Backfilling Foundations. Backfilling shall be performed after the permanent work has been inspected and approved by the owner. Shoring, when no longer required, shall be removed in a manner that will avoid damage or disturbance to the work. The excavation shall be free of forms, organic matter and trash. Backfill should be clean granular material or cohesive soils and shall be free of trash, roots, organic and frozen materials. Nongranulated steelmaking slag may also be used if conditions set forth in Section 2.2.5.1 are satisfied. Backfill should not be placed on surfaces that are under water, muddy or frozen.

Backfill shall be brought up evenly on all sides of piers and along both sides of walls unless walls are designed for eccentric loading. Care is to be taken to avoid wedging or eccentric action upon or against the structures and to avoid damage to the work. Compaction of backfill at all stages shall be completed in accordance with recommendations as set forth in the Project Geotechnical Report. Where walls are designed as propped cantilevers, backfilling shall not proceed until props are installed.

2.2.5.1 Steelmaking Slags. Because of its potential expansion and chemical properties, the use of steelmaking slag as structural backfill is not recommended. However, nongranulated steelmaking slag, such as open hearth or basic oxygen furnace slag, may be used in structural fills or as backfill if it is first weathered in accordance with the following procedure to reduce or eliminate its tendency to expand.

Steelmaking slag shall be thoroughly soaked with water and placed in controlled stockpiles not exceeding 10 ft. in height. It shall then be kept in a moist condition in the stockpile for a period of not less than six months prior to use. If further crushing and breakdown of steelmaking slag occurs after the stockpile period, it shall then be stockpiled again and kept in a moist condition for an additional six-month period prior to use.

These procedures are not required for processed iron blast furnace slag materials, which are approved as concrete or paving aggregates.

2.2.5.2 Resistant Rock Materials. Because of potential excessive settlements and the difficulty in achieving proper placement, the use of rock materials resistant to compaction as structural backfill is not recommended. Although resistant rock can perform satisfactorily as structural backfill when selected, processed and compacted as recommended in the Appendix, Section A 3.8, indiscriminate use of these materials can result in serious foundation settlement problems.

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3.0 Loads and Forces

3.1 Dead Load

The dead load to be assumed shall consist of the weight of all permanent construction and all material and equipment permanently fastened thereto and supported thereby.

3.2 Roof Live Loads (7.4)

The roof shall be capable of supporting a nonreducible minimum live load of 20 psf assumed to act on all or part of its entire horizontally projected surface, and distributed to produce maximum loading conditions. When geographic location, altitude, local conditions or where local building codes require roof snow loads greater than 20 psf, the greatest value shall be used. Where snow can be trapped, as in valleys or on sheds, provision shall be made for increased snow load.

3.3 Floor Live Loads

Uniform and concentrated floor and platform live loads shall be listed in the project specification for each category of use in accordance with maximum expected process requirements. Movable concentrated loads (as produced by laydown) shall be positioned for maximum design conditions. Concentrated loads shall not be reduced, but uniform live load need not be included in the area covered by the concentrated load.

3.3.1 Recommended Minimum Live Loads. Unless otherwise specified, uniformly distributed live loads shall not be less than the minimum values listed in Table 3.1. Requirements for a specific project shall be reviewed considering anticipated storage or other factors. Loadings listed contain some provision for above floor storage, but adjustments shall be made for special areas such as floor-mounted storage bins or special items on floors such as ladles.

Table 3.1 Recommended Minimum Live Loads, psf

Ironmaking Structures	
Casthouse casting floors	500
Floors adjacent to furnaces	1000
Hoist house—first floor	200
Hoist house balcony	150
Blast furnace top platform	300
Bell level platforms	250
All other platforms	150
Cupola buildings	200
Steelmaking Structures	
Charging floors	500
Service or reline floors	600
Flux or weigh hopper floors	200
Bin floors	200
Teeming platforms	300
Mold preparation platforms	300
Rolling Mill Structures	
Motor room floors, oil cellar roofs, or similar operating floors	1000
Ore Refining and Material Handling Structures, Sintering and Pelletizing Structures	
Operating floors	200
Machine floors	100
Screening floors	75
Conveyor equipment floors	75
Conveyor bridge walks:	
Individual walk members	50
Bridge design	25
Miscellaneous	
Boiler house operating floors	250
Miscellaneous walks, access platforms and stairs	100

3.3.2 Live Load Reduction Factors. Uniform floor loads used in determining column loads shall not be less than the above unless specified by the owner. No reduction shall be applied to the above loads as listed that are 100 psf or less, and no reduction factor reducing the load to less than 0.6 of full uniform live load shall be used.

3.4 Crane Runway Loads (7.5)

3.4.1 General (7.5.1). Crane runway girders and supporting framework shall be designed to carry the cranes with the maximum wheel loads with spacing as provided by the owner. They shall also be designed to support the various load combinations as outlined in Section 3.10.

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3.4.2 Vertical Impact, Side Thrust and Traction (7.5.2). Vertical impact and tractive forces shall be an assumed percentage of the maximum wheel loads as specified in Table 3.2. The total side thrust should be distributed with due regard for the lateral stiffnesses of the structures supporting the rails and shall be the greatest of:

(1) That specified in Table 3.2.
(2) 20% of the combined weight of the lifted load and trolley. For stacker cranes this factor shall be 40% of the combined weight of the lifted load, trolley and rigid arm.
(3) 10% of the combined weight of the lifted load and the crane weight. For stacker cranes this factor shall be 15% of the combined weight of the lifted load and the crane weight.

Table 3.2 Crane Impact, Side Thrust and Traction Forces

Crane	Vertical impact percent of maximum wheel loads	Total side thrust percent of lifted load	Tractive force percent of maximum load on driving wheels
Mill cranes	25	40	20
Ladle cranes	25	40	20
Clamshell bucket and magnet cranes (including slab and billet yard cranes)	25	100	20
Soaking pit cranes	25	100	20
Stripping cranes	25	100*	20
Motor room maintenance cranes, etc.	20	30	20
Stacker cranes (cab-operated)	25	200	20

*Ingot and mold
Note 1: Refer to Appendix C for recommendations for evaluating existing structures.
Note 2: Side thrust should be distributed with due regard for lateral stiffness of the structure supporting the rail.

Lifted load is defined as: a total weight lifted by the hoist mechanism, including working load, all hooks, lifting beams, magnets or other appurtenances required by the service but excluding the weight of column, ram or other material handling device which is rigidly guided in a vertical direction during hoisting action.

For pendant operated cranes, the vertical impact, side thrust and tractive forces shall be as follows:

(1) 10% of maximum wheel load for vertical impact.
(2) 20% of maximum load on the driving wheels for the tractive force.
(3) 10% of the combined weight of the lifted load and crane weight for total side thrust.

Radio-operated cranes shall be considered the same as cab-operated cranes for vertical impact, side thrust and traction.

3.4.3 Runway Crane Stops (7.5.3). The load applied to the runway crane stop shall be included in the design of crane runway girders, their connections and the supporting framework. The maximum design bumper force shall be coordinated with the crane designer and shown on the structural drawings. The design bumper force shall be less than or equal to the maximum allowable force on the crane stop.

3.5 Moving Loads

Moving loads are considered to be:

(1) Limited-access vehicles on tracks, which include locomotives, railroad cars and machinery operated on rails.
(2) Unlimited-access vehicles (all vehicles not limited to travel on rails).

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3.5.1 Limited-Access Vehicles

3.5.1.1 Loads and Impacts due to Railway Equipment. Unless otherwise specified, all floors supporting railroad tracks shall be designed in accordance with Ref. 14.

(1) As a minimum, the following impact factors shall be used in the design:
   (a) Rolling effect (locomotive only): 10% down on one rail and upward on the other.
   (b) Direct vertical effect: 25% of axle load, maximum.
(2) Tractive force—longitudinal tractive force shall be considered in the design of floors supporting limited-access vehicles. This force shall be the greater of:
   (a) 10% of live load without impact
   — or —
   (b) 15% of weight on the driving wheels.

3.5.1.2 Nonstandard Gauge Equipment. Floors supporting nonstandard gauge trackage provided for floor-operated machines shall be designed for maximum wheel loads, impact and lateral forces as designated by the owner.

Vertical impact shall not be less than 25%.

For nonstandard gauge equipment, the height above the rail for application of lateral traction forces shall be designated.

3.5.2 Unlimited-Access Vehicles. Loads caused by vehicles having solid rubber tires, pneumatic tires or tracks shall be considered in floor loading. The critical position of such vehicles shall be determined to produce the maximum force on each structural component. The loading arrangement of the forces produced by these vehicles shall be those producing critical single-wheel or wheel combination loadings to the member under consideration. The magnitude and spacing of wheel reactions shall be designated by the owner or per the AASHTO "Specification for Highway Bridges" (Ref. 15) or both.

In addition to the direct vertical loading, the following impact load shall be applied:

(1) Pneumatic-tired vehicles—30% of the wheel load.
(2) Solid-rubber-tired vehicles—50% of the wheel load. The length and width of the tire contact area to be used and the distribution of the above load shall be as designated by the owner or as specified by Ref. 15. A longitudinal force shall be as designated by Ref. 15 or by the owner, depending on the type of vehicle.
(3) Track type vehicles. Vehicles with air hammer attachments—25% of the track load; vehicles used for lifting, scraping and digging—100% of the track load. Owner must furnish load data, indicate intended area of usage and describe operating procedure.

3.6 Contingency Loads

Buildings and structures should be designed to allow for nominal future changes and additions to the structural loads, unless specified otherwise by the owner. In addition to the known and anticipated loads specified in Section 3, the design of main framing members should allow the application of the following loads:

Floor Beams:	One 5000-pound concentrated vertical load applied midspan.
Roof Beams:	One 3000-pound concentrated vertical load applied midspan.
Roof Trusses:	One 3000-pound concentrated vertical load applied at any single panel point.
Platform Beams:	One 1000-pound concentrated vertical load applied midspan.
Columns:	One 1000-pound concentrated lateral load applied mid-height of any column span, in the weakest flexural direction.

These contingency loads are not cumulative and should be applied to only one member or one panel point at a time.

3.7 Special Loads

3.7.1 Guidelines for Vibratory Loading (7.6). Both the static and dynamic loads generated by the equipment shall be supplied by the equipment supplier. Structures to be designed for problem-free installations of rotating

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and vibrating equipment should be designed so that the lowest approximate natural frequencies of installed equipment (equipment/support/structure and/or soil configurations) as determined by dynamic analysis is 1.5 times the operating frequency of the equipment. Provision for design of supports for vibratory equipment shall include, but not be limited to, the following:

(1) Motors and similarly balanced rotating equipment: Vertical impact—25% of the weight of the equipment.
(2) Vibrating screen supports:
  (a) Live load—weight of the screen plus a reasonable burden on the screen deck.
  (b) Vertical impact—100% of the live load.
  (c) Horizontal impact—50% of the live load.
(3) Pan feeder supports:
  (a) Live load—weight of the pan plus a reasonable burden above pan in hopper.
  (b) Vertical impact—25% of the live load.
  (c) Horizontal impact—25% of the live load.
(4) Gyratory and jaw crushers:
  (a) Live load—weight of the crusher plus burden.
  (b) Vertical impact—100% of the live load.
  (c) Horizontal impact—dependent upon individual installation.
(5) Forced or induced draft fans:
  (a) Vertical impact—25% of fan weight.
(6) Mold oscillators:
  (a) Vertical impact dependent upon installation.
(7) Reciprocating compressors:
  (a) Vertical impact dependent upon installation.

3.7.2 Conveyor Unbalanced Forces. Structures for conveyor supports shall be designed for tight side and slack side belt tension in addition to dead and live loads.

3.7.3 Utility Support Loads. The owner shall designate utility loads and their locations insofar as they affect the design of supporting structures. Examples include electric cable trays, transformers, piping, ducts, etc.

3.7.4 Special Roof-Supported Structures. The owner shall furnish loading information and configuration data pertinent to the roof-supported structure such as transmission towers, racks, tanks, monitors, ventilators, stacks and large ducts. Wind loads on these structures shall also be considered. Dust buildup shall be considered as a part of loads from ducts, ventilators and monitors.

3.7.5 Loads from Mains, Ducts and Pipes (7.18). Supports for loads in buildings from mains, ducts and pipes shall be based on the following:

(1) Process piping shall be assumed full for support design.
(2) Supports for mains and ducts shall be designed for a minimum dust loading of one-fourth of duct depth filled. Consideration for both dry and wet dust density must be investigated.
(3) Support for parallel mains and ducts on the same fan system should be designed for an accidentally full condition of any one duct.
(4) Pipe and duct supports shall be investigated for loadings resulting from temperature changes and differential or unbalanced internal pressure within the system. This shall also apply to water-cooled ducts and pipes conveying gases, steam or liquids.

3.8 Wind Loads (7.7)

As a minimum, all buildings and structures exposed to wind shall be designed to meet the wind load requirements of the local building code. Wind speed and exposure criteria shall be determined in accordance with local building code or ASCE 7, unless higher loads are indicated by the owner's specification or the design engineer's judgement. Building configurations and production operations that may create the internal pressure conditions of partially enclosed structures shall be accounted for in the design. Structures outside the scope of the local building code shall be designed in accordance with appropriate approved national standards.

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3.9 Seismic Loads and Displacements (7.8)

As a minimum, all buildings and structures shall be designed to meet the seismic force, displacement and ductility requirements of the local building code. Site classifications and seismic design categories shall be determined in accordance with the local building code or ASCE 7, unless higher requirements are indicated by the owner's specification or the design engineer's judgement. Where appropriate, a site investigation should be performed in order to determine the site classification for seismic design.

Seismic response interaction between structures and equipment shall be accounted for in the design. The seismic mass of storage equipment such as tanks, bins, silos, hoppers and storage racks shall include the weight of stored material under normal operating conditions. The seismic mass of cranes and trolleys that lift a suspended load need include only the empty weight of the equipment.

For buildings, structures and equipment that must remain serviceable immediately after a design-level earthquake, special consideration should be given to design requirements beyond those specified in the building code.

3.10 Load Combinations for Design of Crane Runways and Supporting Structures (7.9)

3.10.1 Symbols and Notations. For ease of reference, the following symbols and notations correspond closely to those contained in the ASCE (American Society of Civil Engineers) Standard ASCE 7, "Minimum Design Loads for Buildings and Other Structures." These symbols apply only to sections 3.10 and 3.12 and are not included in Section 8 (Symbols).

C_{vs}	vertical loads due to a single crane in one aisle only
C_{ss}	side thrust due to a single crane in one aisle only
C_i	vertical impact due to a single crane in one aisle only
C_{ls}	longitudinal traction due to a single crane in one aisle only
C_{vm}	vertical loads due to multiple cranes
C_{bs}	bumper impact due to a single crane in one aisle only at 100% speed
C_d	dead load of all cranes, parked in each aisle, positioned for maximum seismic effects
D	dead load
E	earthquake load
F	loads due to fluids
L	live loads due to use and occupancy, including roof live loads, with the exception of snow loads and crane runway loads
L_r	roof live loads
S	snow loads
R	rain loads (inadequate drainage)
H	loads due to lateral pressure of soil and water in soil
P	loads due to ponding
T	self-straining forces as from temperature changes, shrinkage, moisture changes, creep, or differential settlement
W	wind load

3.10.2 Basis of Design. Structural design, not including foundations, shall be based on whichever one of the following three cases may govern. Load combinations without cranes, in accordance with ASCE 7 or the local building codes, and other load combinations as shown in Section 3.10.2.4 shall also be considered. Load combinations shown are for allowable stress design.

Axial loads, moments and shears for each type of loading shall be listed separately (i.e., dead load, live load, crane load eccentricities, crane thrust, wind, etc.).

Crane impact loads apply only to runway girders and their connections.

The allowable stress ranges under repeated loads shall be based on procedures covered in Section 5.7 with the estimated number of load cycles in accordance with the building classification covered in Section 1.4. The owner shall designate an increase in the estimated number of load cycles for any portion of the building structure for which the projected work load or possible changes in building usage warrants.

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3.10.2.1 Case 1 (7.9.1)

$$D + C_{vs} + 0.5C_{ss} + C_i$$

This case applies to load combinations for members designed for repeated loads. The number of load repetitions used as a basis for design shall be 500,000 to 2,000,000 (Loading Condition 3) or over 2,000,000 (Loading Condition 4), as determined by the owner, for Class A construction. Class B and Class C constructions shall be designed for 100,000 to 500,000 (Load Condition 2) and 20,000 to 100,000 (Load Condition 1), respectively. This case does not apply to Class D buildings.

The design stress range shall not exceed the allowable stress range determined in accordance with Section 5.7. In lieu of the procedure suggested in Section 5.7, a more sophisticated approach using a variable stress range spectrum may be used.

3.10.2.2 Case 2 (7.9.2)

(1) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C + C_{ss} + C_{ls}$ (Single Crane)
(2) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vm} + C_{ss} + C_{ls}$ (Multiple Cranes)

This case applies to all classes of building construction. Full allowable stresses may be used.

3.10.2.3 Case 3 (7.9.3)

(1) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C + W$
(2) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C + C_{ss} + 0.5W$
(3) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C + 0.67C_{bs}$
(4) $D + L + (L_r \text{ or } R \text{ or } S) + C_d + E$

This case applies to all classes of building construction. The total of the combined load effects may be multiplied by 0.75, with no increase in allowable stresses. No load reduction shall be taken for combinations of dead load and wind only.

3.10.2.4 Other Load Combinations. The structural effects of F, H, P or T shall be considered. For combinations with or without crane loads, including $D + L + (L_r \text{ or } S \text{ or } R) + (W \text{ or } E) + T$, the total of the combined load effects may be multiplied by 0.67, with no increase in allowable stresses.

3.11 Loads on Retaining Walls, Grade Walls and Grade Beams

3.11.1 Earth Pressure. Soil pressures shall be as established by the Project Geotechnical Report. When soil does not strain laterally, the earth pressure is designated as at-rest pressure. To minimize hydrostatic pressure, retaining walls should be constructed with weep holes and drains. Granular backfill should be used wherever possible to reduce the maximum wall loading. Care shall be exercised in compacting the backfill when using heavy vibratory equipment.

3.11.2 Vertical Loads. Vertical loads from a building superstructure, basement floor framing, vehicular or railroad traffic on the walls and beams shall be considered in the design.

3.11.3 Supplemental Loads. Surcharge loads from supplemental loads outside or adjacent to walls and beams shall also be considered in the design.

3.12 Loads on Building Foundations

Column reactions shall be listed in such a way that each individual force or moment can be clearly separated so that they may be combined to cause the most critical loading condition. In listing column reactions, those caused by one or more cranes shall be clearly stated. Wind, seismic and thermal forces shall be presented separately. Foundations shall safely sustain all the loads transmitted to them within the requirements established in the Project Geotechnical Report. In addition to the forces applied on the top of the foundation, concrete foundations

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shall be designed to transmit to the subsoil those floor and surcharge loads imposed in the vicinity of the column that would be directly transmitted to the footings or through grade walls or grade beams.

3.12.1 Load Combinations. Referring to the symbols and notations shown in Section 3.10.1, the following basic conditions and load combinations shall be investigated. Normally permitted increases in allowable soil pressures, pile or caisson capacities for various load combinations should be used unless stated otherwise in the geotechnical report.

The following loading combinations shall be assumed most probable to cause maximum stress, but investigations shall not be limited to these combinations.

3.12.1.1 Condition 1

$$D + L + (L_r \text{ or } R \text{ or } S) + C_{vm}$$

3.12.1.2 Condition 2

$$D + W$$

3.12.1.3 Condition 3

(1) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vm} + C_{ss} + 0.5W$

(2) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C_i + 0.67C_{bs}$

3.12.1.4 Condition 4

$$D + C_d + E$$

3.12.1.5 Condition 5

$$D + C_d + C_{bs}$$

For conditions 1 and 2, the maximum soil bearing pressure, pile or caisson loading shall not exceed the allowable value.

For conditions 3, 4 and 5, the total of the combined load effects may be multiplied by 0.75 with no increase in allowable foundation loads, maximum soil bearing pressure, pile or caisson loads.

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4.0 Foundations, Floors and Walls

4.1 General

This section provides general criteria and procedures for the design of mill building foundation components. These components are: soil bearing column foundations, pile and caisson supported foundations, grade walls, grade beams, retaining walls, basement walls, slabs on grade and other incidental concrete components required for construction of industrial type mill buildings. It is intended as a guide to a uniformly safe design and as an overall concept of design approach.

4.2 Concrete Construction

4.2.1 Design and Construction. All design and construction shall be in accordance with ACI 318 (Ref. 5).

4.2.2 Concrete Strength. Minimum compressive concrete strength shall be 3000 psi in 28 days unless otherwise specified. Durability requirements shall be compatible with the soil conditions (i.e., water/cement ratio, ground water quality, etc).

4.2.3 Setting Anchor Rods. Anchor rods should preferably be set with metal templates and without sleeves. If sleeves are used, they shall be completely filled when the base plate is grouted. Special care shall be taken to exclude water from the sleeves until grouted.

4.2.4 Grouting of Base Plates. Grouting of column base plates should be accomplished after building columns have been plumbed and aligned. If shim packs are used to level base plates and are removed after initial grout has cured, the shim space shall be filled with additional grouting.

4.3 Soil-Bearing Foundations (7.10)

4.3.1 General. The owner shall provide the following design criteria developed by the geotechnical engineer in accordance with the applicable requirements of the Appendix and included in the Project Geotechnical Report:

(1) Allowable soil-bearing pressures.
(2) Earth pressures and safety factors for lateral and rotational stability.
(3) Estimated total and differential settlements for various sizes of foundations at different elevations and coefficients for calculation of lateral movements.
(4) Ground water condition.
(5) Minimum depth of footings for protection from heaving due to frost.
(6) Description and effect on foundation of overlapping soil pressures caused by existing and proposed structures, process and machinery foundations, floor loads, walls, basement surcharges, excavations, vibratory equipment, etc. This will require periodic review until contract plans and construction of all substructures are complete.
(7) When foundations are to be built on nongranulated steelmaking slag, the soils engineer shall test the materials for potential expansion properties. The use of steelmaking slag as a fill material shall comply with the conditions set forth in Section 2.2.5.1.
(8) The use of segregated resistant rock for foundation support is not recommended because of the limitations presented in Section 2.2.5.2. However, if resistant rock material is used for this purpose, it shall be placed in accordance with the Appendix, Section A 3.8.
(9) The durability requirements for the concrete.

4.3.2 Ground Water Conditions. In those geographic locations where fluctuation in ground water level results in swelling and shrinking of soils, foundations shall be located below the depth of ground water influence, or other steps shall be taken to support columns such as on piles, caissons or other deep foundations.

4.3.3 Effect on Other Structures. The effect of all new foundations on adjacent and subsurface structures shall be considered in design.

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4.4 Pile and Caisson Supported Foundations

4.4.1 General. The following design criteria developed by the geotechnical engineer in accordance with the applicable requirements of the Appendix A and included in the Project Geotechnical Report (A 2.3) shall be provided.

(1) Allowable load capacities and uplift with particular consideration of group effect and minimum spacing between piles.
(2) Allowable total and differential settlement and rotation of the base.
(3) Allowable resistance to lateral forces and coefficient for calculation of lateral movement.
(4) Description and effect of existing and proposed structures, walls, floor loads, surcharges, vibratory equipment, the effects of negative skin friction where applicable, etc. Periodic review will be required until contract plans and construction of all substructures are complete.
(5) The depth below ground surface to the point of support for evaluation of pile column strength.
(6) Corrosion protection requirements where aggressive substance or electrolytic action can occur in the pile environment. Steel piling should not be used for electrical grounding where electrolytic action is possible.
(7) When pile caps or grade beams are to be built on nongranulated steelmaking slag, the soils engineer shall test the materials for possible expansion properties. The use of steelmaking slag as fill under grade beams, pile caps or similar structural elements shall comply with the conditions set forth in Section 2.2.5.1.

4.4.2 Allowable Pile and Caisson Stresses. The stresses on any cross-section of a pile shall not exceed the values listed in Table 4.1. Additionally, it shall be assumed that for piles more than 40 ft. in length, installed in material other than fill, peat or organic silt, 75% of the load of an end-bearing pile is carried by the tip. For friction piles, the full load shall be computed at the cross-section located at two-thirds of the embedded length of the pile measured up from the tip. For all types of piles, when bending occurs in the pile, the combined stress shall be proportioned so that:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{(Eq. 4.1)}$$

where:

f_a = Computed average axial stress in column, ksi
f_b = Computed average bending stress in column, ksi
F_a = Axial stress allowed in the absence of bending moment, ksi
F_b = Bending stress allowed in the absence of axial force, ksi

In addition, for prestressed piles:

$$f_a + f_b + f_{pe} \leq 0.45 f'c \qquad \text{(Eq. 4.2)}$$

$$f_a - f_b + f_{pe} \geq 0 \qquad \text{(Eq. 4.3)}$$

where:

f_{pe} = Effective prestress after losses, ksi
f'_c = Ultimate compressive strength of concrete at 28 days, ksi

Thin-shell concrete piles shall not be used in bending unless properly reinforced and designed as a concrete pile. Corrugated shells are not considered as sharing the load.

To be considered as load-bearing, steel shells shall have a minimum thickness of 0.1 in. and a cross-sectional area equal to 3% of the gross area of the pile section. Where a steel shell forms part of the pile or caisson design, appropriate corrosion allowance shall be made for determining the thickness.

If a segment of the pile lacks lateral restraint, or if soil conditions do not provide appreciable restraint, the column strength shall be evaluated between points of support; otherwise, the pile shall be assumed to be continuously supported.

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Pipes, tubes and rolled structural piles shall be designed as columns in accordance with AISC Specification, (Ref. 1) using the limiting stresses listed in Table 4.1.

Concrete filled steel pipe piles, reinforced concrete piles, prestressed concrete piles, precast concrete piles and reinforced concrete caissons shall be designed by either of the following criteria:

(1) The ACI Recommendations for the Design of Piling (Ref. 8) using the limiting stresses listed in Table 4.1.

(2) The ACI Code (Ref. 5). In lieu of more refined information, the following load distribution and strength factors shall be used: dead load—10%, live load—90% and capacity reduction factor of 0.7.

When using either of the above criteria, bar reinforcing in excess of 8% of the average cross-sectional area of the pile shall not be included in the load carrying capacity. The design allowable stresses for nonreinforced concrete piles of any type, nonreinforced concrete filled shells and tubes with wall thicknesses less than 0.1 in. and wood piles, shall be equal to or less than the maximum stresses listed in Table 4.1. These piles shall not be used in tension, and the average stress on the section shall always be in compression.

Table 4.1 Allowable Pile and Caisson Design Stresses

Material		Allowable Stress
Compression:		
Concrete:	Unconfined	$0.33 f'_c$
	Confined	$0.40 f'_c$
Steel:	Pipe, tube or shape	$0.5 F_y$ or 18 ksi*
	Reinforcement	$0.40 F_y$ or 24 ksi*
	Structural core	$0.50 F_y$ or 24 ksi*
Wood:	———	Determined in accordance with ASTM D2899
Tension:		
Concrete:		0
Steel:	Pipe, tube or shape	$0.50 F_y$ or 24 ksi*
	Reinforcement	$0.50 F_y$ or 24 ksi*
	Structural core	$0.50 F_y$ or 24 ksi*
Wood:	———	0
Flexure:		
Concrete:	Compression	$0.45 f'_c$
Steel:	Tension and compression	$0.5 F_y$ or 22 ksi*
Wood:	Tension and compression	Determined in accordance with ASTM D2899

* whichever is lower

4.4.3 Splices. Splices shall be capable of developing the design load of the pile or caisson in compression, tension, shear and bending as required.

4.4.4 Special Provisions for Caisson and Pile Caps. Caisson or piles subjected to tensile force shall be positively anchored to their caps.

4.4.5 Field Control of Pile Driving

4.4.5.1 Driving. The method of driving shall not impair the strength of the pile. Shattered, broomed, crumpled or otherwise damaged pile heads shall be cut back to sound material before continuing the driving. Where a group of piles is to be driven, a survey should be done after driving to detect horizontal and vertical movements. Piles that have suffered vertical movements, in general, shall be redriven to ensure required capacity. Piles that have suffered horizontal movements must be investigated for soundness.

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4.4.5.2 Plumbness. Vertical piles shall not vary more than 2 $^1/_2$% from the plumb position, and no pile shall deviate more than 3 in. in the horizontal dimension from its design location.

4.4.5.3 Records. The contractor shall keep records for each pile driven, giving the designation, tip and cutoff elevations, locations, orientation, resistance to penetration for each foot of penetration and resistance to penetration inch by inch for the last 12 in. of movement. These records shall be submitted to the owner by the contractor in a timely fashion.

4.4.5.4 Load Tests. All load tests, required by the soils investigation, shall be performed as per the Project Geotechnical Report. Unless stated otherwise by the owner, a minimum of two satisfactory load tests should be performed.

4.5 Retaining and Basement Walls

4.5.1 General. Retaining walls and basement walls shall be designed in accordance with ACI Code (Ref. 5) and criteria established by the owner in the Project Geotechnical Report.

4.5.2 Stability Criteria. Retaining wall base shall be sized so that the resultant of all forces acting upon the wall shall lie within the middle third of the base—except when founded on rock or piles or unless permitted otherwise by the foundation criteria in the Project Geotechnical Report. The maximum foundation pressure shall not exceed the safe bearing capacity of the subgrade material. All retaining walls shall have a safety factor of at least 1.5 against overturning and sliding. When safety against sliding is achieved by application of a shear key, the safety factor (omitting the key) shall be greater than 1.0.

4.5.3 Provision for Drainage and Hydrostatic Pressures. To minimize certain load conditions produced by water in the backfill or by frost action, retaining walls should have a positive drainage system. Retaining walls shall be designed for lateral soil pressure plus hydrostatic pressure corresponding to the highest ground water table. For plants subject to flooding, hydrostatic pressure shall be based on flood level. Vertical uplift due to hydrostatic ground water should be a consideration in design of dry pits, etc.

4.6 Floor Slabs on Grade

4.6.1 Design Procedure. The thickness of a slab on grade may be established by use of design charts provided in "Slab Thickness Design for Industrial Concrete Floors on Grade," IS195.OID Portland Cement Association (PCA) (1976), or Corps of Engineers, (COE), U.S. Army, "Engineering Design: Rigid, Pavements for Roads, Streets, Walks and Open Areas," Engineering Manual EM 1110-3-132, Dept. of the Army. (Note that the design charts of the PCA method apply only to loads near center of panel areas and not to loads near corners, edges or joints. The COE method can be used for loads at slab edges.) The Factor of Safety against unreinforced slab cracking shall not be less than 2.0. The modulus of rupture shall be taken as $7.5\sqrt{f_c'}$.

4.6.2 Subgrade Modulus. Subgrade modulus k (pci) shall be established by the geotechnical engineer.

4.6.3 Subgrade Preparation. The Project Geotechnical Report should establish the depth and degree of compaction required for any newly compacted-engineered fill and for any special subgrade preparation for any in-situ soils. Slabs on grade are routinely placed on a base course (drainage fill), the thickness of which should be specified in the project drawings, and the gradation of the permitted soil types should be described in the project earthwork specifications.

4.6.4 Vapor Retarder. Vapor retarders can aggravate cracking and curling during curing. When required, a minimum 6-mil thickness vapor retarder shall be placed under the slab on grade as per the recommendation of the geotechnical report.

4.6.5 Construction and Control Joints. Control joints shall be shown on the drawings. Construction joints shall be located at control joints. For slabs containing no temperature and shrinkage reinforcement (reinforcing bars, welded wire fabric or fibers), spacing of control joints shall not exceed (in ft.) 2 times the slab thickness (in in.).

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4.6.6 Temperature and Shrinkage Reinforcement. For larger spacings of joints (than as outlined in 4.6.5), use of temperature and shrinkage reinforcement is required. Use of welded wire fabric requires special care to ensure that the material is properly located within the slab thickness. Use of the subgrade drag method is not recommended for selecting slab reinforcement.

4.6.7 Expansion Joints (7.11). Expansion joints are to be used only to isolate the floor slab on grade from other structural elements such as column piers, machine bases and at building walls.

4.6.8 Steelmaking Slag Subgrade Material. The use of steelmaking slag as subgrade material for slabs on grade is not recommended because of its potential expansion properties. If steelmaking slag is used for this purpose, it shall conform to the quality standards listed in Section 2.2.5.1.

4.6.9 Resistant Rock Subgrade Material. Using segregated resistant rock as subgrade material for slabs is not recommended because of the limitations presented in Section 2.2.5.2. However, if resistant rock material is used for this purpose, it shall be placed in accordance with the Appendix, Section A 3.8.

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5.0 Structural Steel

5.1 General

Design and workmanship shall comply with the applicable requirements of the AISC Specification (Ref. 1) and the AISC Code of Standard Practice (Ref. 2) except as supplemented herein. If the engineer chooses to use LRFD methods (Load & Resistance Factor Design), many of the recommendations in this report are still valid. The engineer should consult texts, design aids and papers published by AISC, AISE and SSRC for more information and examples of designs.

5.2 Mill Building Framing

Mill buildings are space frame structures. The planar frames consisting of building columns and roof trusses are combined by a bracing system into the space frame. If the building has an above-grade floor connected to this framing, the effect of the floor shall be included in the analysis.

The main function of the bracing system is to stabilize the building space frame, minimize relative horizontal movement between cross-bents, distribute the localized crane loads to adjacent bents and deliver longitudinal framing forces (wind, seismic, crane traction and crane bumper) to the foundations.

5.3 Framing Analyses and Drift

The advantages of the space frame versus a planar frame should be utilized in the mill building frame analyses.

In the framing analyses of those existing mill buildings where the roof truss bottom chord bracing is not developed enough to carry space frame loads, the planar frame model of the building frame could be considered as an alternative solution to the bracing modification.

It is recommended that building columns be designed as fixed or partially fixed at the base. Percent of fixity depends on anchorage details, foundation and soil parameters.

In the calculation of localized crane loads on columns, the total transverse horizontal side thrust from the crane (see Section 3.4.2) shall be distributed between crane runway support columns in proportion to their lateral stiffnesses.

Building frame lateral drift at the top of the crane girders shall be no greater than $^1/_{400}$ of the height from column base or 2 in., whichever is less, for each of the following load conditions:

(1) Crane lateral forces identified in this report
(2) Building wind loads due to a wind speed that has an annual probability of exceedance no greater than 10% (10-year recurrence interval).

These drift limits may be exceeded only when it can be shown that the total drift will not adversely affect the durability of the building and the operation of equipment.

Based on an elastic frame analysis, the variation of crane rail gauge due to gravity loads shall be within +1 in. and $-^1/_2$ in. of the specified gauge. Snow loads of 30 psf or less may be reduced by 50% for this condition only, and snow loads greater than 30 psf may be reduced by 25%.

5.4 Roof Trusses

The roof trusses at the column lines shall be considered a part of the building frame. The frame effect shall be included in the truss member forces.

The truss chord members subjected to local bending shall be analyzed and designed for combined bending and axial stresses.

Primary members, including bracing, wherever possible, shall be connected so that their gravity axes intersect at a point. Where eccentricity exists, the effects of eccentricity shall be considered in the design of the members.

5.5 Bracing System

If load sharing between frames is required, then a continuous bracing system shall be provided. The continuous bracing system shall extend longitudinally between expansion joints and between expansion joints and the ends of the building. This bracing system shall be designed on the basis of calculated building space frame loads. When roof trusses are used, the bracing shall be located in the plane of the bottom chords.

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If it is necessary to interrupt the lower chord bracing system to accommodate crane repair facilities or for any other cause, special analysis of the affected bents and bracing shall be made.

In addition to continuous bracing in the plane of the bottom chords, intermittent bracing may be provided for the top chords of the roof trusses, consisting of sway frames and/or bracing in the plane of the top chords.

Longitudinal bracing, sufficient to transfer wind, seismic and crane traction, or bumper forces to the foundations, should be placed approximately midway between expansion joints or near the midpoint of buildings without expansion joints. The longitudinal force, when bracing to the column bases is impossible, shall be proportionally divided between the total number of effective columns between any two-expansion joints in accordance with their respective stiffnesses.

Knee braces from the crane girder to the crane runway columns are not recommended.

Lateral restraint for columns, compression chords or flanges of trusses and girders shall be considered accomplished when the bracing system is designed to resist a transverse force equal to 2 $^1/_2$% of the resultant compressive axial stress times the compressed flange or chord area.

5.6 Expansion Joints (7.13)

In furnace buildings and similar structures handling hot metal and subject to wide temperature ranges, transverse expansion joints shall be provided at approximately 400-ft. intervals. If buildings are not subject to wide temperature ranges, the distance between transverse expansion joints should be approximately 500 ft.

Multiple-aisle buildings shall be provided with such longitudinal joints as are deemed advisable. If the width of the building exceeds 500 ft. or is composed of more than five aisles, longitudinal expansion joints shall be provided.

Long buildings and runways extending in a direction normal to the axes of other buildings or runways shall not be rigidly attached to each other unless special provision is made for movement or expansion of one structure without causing misalignment in the other.

5.7 Allowable Stress Range under Repeated Loads (7.14)

When the allowable stress range is determined for repeated loads, no differentiation shall be made with respect to different steels and their correspondingly different yield points. Members and fasteners subjected to repeated loads shall be designed so that the maximum design stress range does not exceed the allowable fatigue stress range for repeated loads, as specified in the latest edition of AISC Specification for Structural Steel Buildings. (Use Table 5.1 for loading conditions; see Ref. 1.)

Table 5.1 Loading Conditions

Building Class	Loading Condition	Number of Loading Cycles	
		From	To
D	N/A		
C	1	20,000 (a)	100,000 (b)
B	2	100,000	500,000 (c)
A	3	500,000	2,000,000 (d)
A	4	Over 2,000,000	

(a) About 1 application per day for 50 years
(b) About 5 applications per day for 50 years
(c) About 25 applications per day for 50 years
(d) About 100 applications per day for 50 years

5.8 Crane Runway Girders (7.15)

5.8.1 General. When crane runway girders are designed as simple beams or simply supported box girders, direct interconnection that would restrain relative rotation between adjacent ends of successive girders is not recommended. Independent connections to the column at the end and top of each girder shall be provided. Horizontal diaphragms or trusses at the top of the crane runway girder shall be connected at the column to transfer all horizontal shear to the building frame without the development of appreciable continuity between adjacent spans and to account for girder end rotations.

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The effect of torsional moments and out-of-plane forces at the rail-to-top-flange interface shall be considered. These moments and forces include lateral wheel loads applied to the head of the rail and eccentricity of rail to centerline of girder web. Typically, the eccentricity results from the sum of the construction tolerance for center of web to center of rail, plus horizontal slip of the rail allowed by the clips. An exact analysis and design solution is complex and beyond the scope of this document.

It has been found through experience that, for members designed and constructed in accordance with the recommendations contained in this report, satisfactory performance can be expected without additional strengthening.

Crane girders shall be designed, detailed and fabricated to resist fatigue damage. For all classes of buildings, except as noted herein, a full penetration weld with contoured fillets shall be used between the web and top flange. For building classes C and D, Loading Condition 1, continuous fillet welds may be used, provided that the welds are designed to carry the full applied loading, including local effects of individual wheel loads. The local effects to be considered include the effects of fatigue if there are more than 100,000 cycles of individual wheel loads, and the effects of torsional moments and out-of-plane forces.

Bottom flanges may be welded to web plates with fillet welds, provided they are continuous welds on both sides of the web.

Web plate and flange plate splice welds shall be complete penetration butt welds. Flange plate splice welds shall be ground flush on all sides and edges, with the grinding direction parallel to the span of the girder.

Intermittent fillet welds shall not be used, except for cover plate and stiffener welds on girders for pendant controlled cranes operating within Loading Condition 1 in Building Classes C and D.

There shall be no welded attachments to the bottom flange of the crane girder.

5.8.2 Stress Calculations. Girders shall be proportioned using the gross moment of inertia for compressive stress calculation and the net moment of inertia for tensile stress. To calculate the net moment of inertia, the neutral axis of the gross section shall be used, and the moments of inertia of all holes on each side of the axis shall be deducted.

When trusses are used in lieu of runway girders, local bending stresses between panel points and secondary stresses shall be included in the stress computations.

Stresses due to simultaneous vertical and lateral loadings shall not exceed the requirements as specified herein.

5.8.2.1 Rolled Shapes and Built-up Single Web Plate Girders Having an Axis of Symmetry in the Plane of Their Web (7.15.1). In the design of single web girders, the following interaction formula shall be satisfied:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad \text{(Eq. 5.1)}$$

where:

f_a	=	Computed average axial stress, ksi
f_{bx}	=	Computed stress due to the bending moment about the (X-X) axis, ksi
f_{by}	=	Computed stress due to the bending moment about the (Y-Y) axis, ksi
F_a	=	Axial stress allowed in the absence of bending moments, ksi
F_{bx}	=	Allowable stress for bending about the (X-X) axis, ksi, as specified in Ref. 1
F_{by}	=	Allowable stress for bending about the (Y-Y) axis, ksi, as specified in Ref. 1

Stresses in the tension flange should be based on the net section moment of inertia, considering only the vertical loads.

5.8.2.2 Girders with Backup Bracing Systems. Each web system shall be assumed to take the loads imposed thereon, with no more than a width of $\frac{95t_f}{\sqrt{F_y}}$ of flange plate adjacent to each side of a vertical web included in calculating the section properties.

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NOTE: F_y = Specified minimum yield stress of steel, ksi
t_f = Thickness of beam or girder flange, in.

If a flange has a thinner auxiliary plate continuously attached thereto and approximately in the same plane, the width of the thinner plate assumed as acting with the flange shall be no greater than:

$$\left(\frac{95}{\sqrt{F_y}} - \frac{b_f}{2t_f} \right) t_a = w_e \qquad \text{(Eq. 5.2)}$$

where:

w_e = Effective width of auxiliary plate
t_a = The thickness of the auxiliary plate, in.
b_f = The overall width of the plate, in.

For combined vertical and transverse loads, the interaction Eq. 5.1 shall be satisfied except that F_{bx} shall be the full allowable stress for bending of built-up members in which the compression flange is braced laterally. Resistance to transverse loads shall be assumed to be provided by a horizontal girder composed of the entire top flange of the box system, acting as the web, with no more than half the vertical plate nor more than a depth of vertical web of:

$$\frac{127 t_w}{\sqrt{F_y}} = d_w \qquad \text{(Eq. 5.3)}$$

where:

t_w = thickness of beam or girder web (in.) (see Fig. 5.1)
d_w = effective depth of the vertical web

5.8.2.3 Box Girders with Transverse Diaphragms (7.15.2). In the case of the closed box sections with cross-diaphragms or X-bracing designed to distribute local loads to flanges and webs, the complete cross-section may be assumed to resist the combined vertical and lateral loads. Shear stress due to torsion and bending shall be included. Width-thickness criteria for compression of shear members shall be met.

5.8.3 Web Thickness. The ratio of the clear depth of the web to the web thickness shall not exceed:

$$\frac{h}{t_w} \leq \frac{760}{\sqrt{F_b}} \qquad \text{(Eq. 5.4)}$$

where:

h = clear depth of web between flanges, in.
F_b = Allowable bending stress, ksi, given in Section F1.1 of AISC Specification unless longitudinal stiffeners are used, in which case the design shall be in accordance with AISE Technical Report No. 6 (see Ref. 11)

5.8.4 Bottom Flange Bracing (7.15.3). Crane runway girders with spans 36 ft. and over in Class A, B and C buildings, or 40 ft. and over in Class D buildings, shall have the bottom flanges stiffened by means of a bracing system connected to an adjacent girder or stiffening truss.

Vertical cross-frames shall not be used unless the frame and stiffening truss is designed for the forces imposed, including cyclic considerations. Lacing shall be designed to resist a minimum force equal to 2 $^1/_2$% of the axial force in the bottom flange applied at mid-span. Lacing shall not be welded to the crane girder bottom flange.

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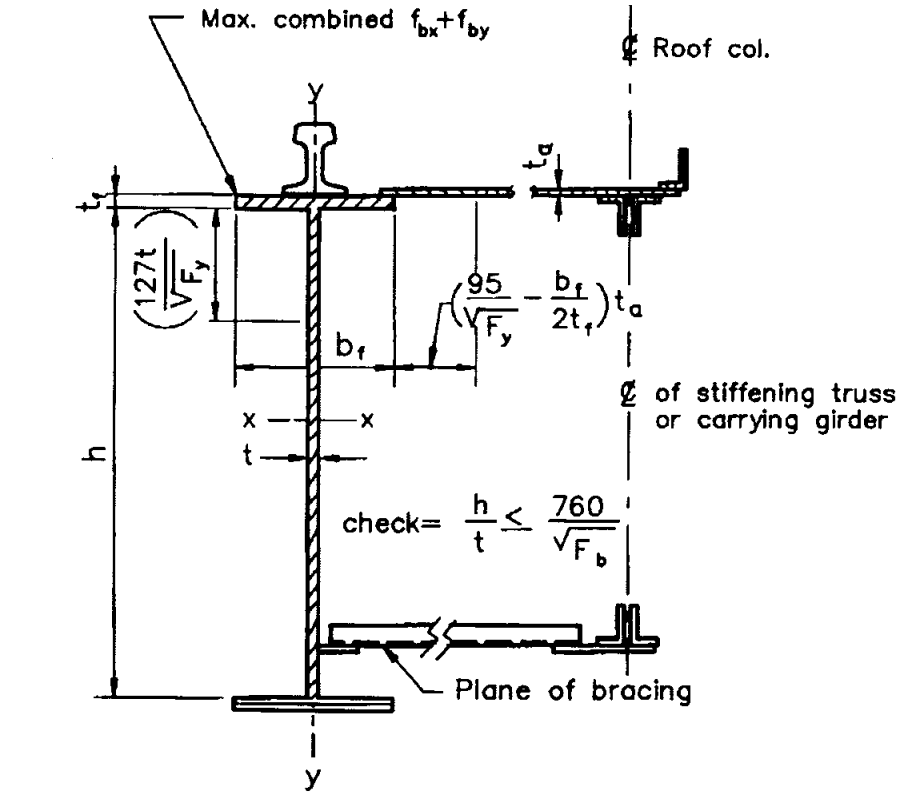

=both f_{bx} & f_{by} ; =f_{bx} only ; =f_{by} only

AREA INCLUDED IN CALCULATED SECTION PROPERTIES

Fig. 5.1—Areas assumed effective as per Sections 5.8.2.2 and 5.8.4.

5.8.5 Stiffeners (7.15.4). Bearing stiffeners shall be used where required to transmit end reactions. Intermediate stiffeners shall be used when required.

Intermediate stiffeners shall be welded to the top (compression) flange with a full penetration (beveled) weld and should be stopped short of the bottom (tension) flange. The end bearing stiffeners shall be welded to the top (compression) flange and bottom (tension) flange with a full penetration (beveled) weld. Alternately, the end bearing stiffeners may be welded to the bottom flange to obtain full bearing.

All welds between stiffeners and web plates or flange plates are to be continuous welds except those for building classes C and D, in which intermittent fillet welds may be used for the intermediate stiffener-to-web connection. The stiffeners shall have clipped corners to provide clearance for the web to flange welds.

If $\frac{h}{t_w}$ is equal to or greater than 70, intermediate stiffeners shall be required at all points where:

$$f_v \geq \frac{64{,}000}{\left(\frac{h}{t_w}\right)^2} \qquad \text{(Eq. 5.5)}$$

where:

f_v = The greatest unit shear stress in the panel under any condition of complete or partial loading, ksi

The allowable shear stress shall be as specified by the latest AISC Specification (Ref. 1).

The clear distance between intermediate stiffeners, when stiffeners are required by the foregoing, shall be such that the smaller panel dimension *a* or *h* shall not exceed:

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$$a \text{ or } h \leq \frac{350 t_w}{\sqrt{f_v}} \qquad \text{(Eq. 5.6)}$$

where:

a = Clear distance between transverse stiffeners, in.
h = Clear distance between flanges, in.

Intermediate stiffeners shall be applied in pairs, one on each side of the web. Intermediate angle stiffeners may be crimped over the flange angles.

Intermediate stiffeners employed to stabilize the web plate against buckling, and not for the transfer of concentrated loads from flange to web, shall be of a section not less than that required by the following formula:

$$I_s = \left(\frac{h}{50}\right)^4 \qquad \text{(Eq. 5.7)}$$

where:

I_s = Moment of inertia of the pair of stiffeners about the centerline of the web, in.^{4}

5.8.6 Local Wheel Support (7.15.5). On riveted or bolted crane runway girders, wheel load concentrations shall be transferred to the web plate of the girder by direct bearing of the top flange. The web plate shall be flush, or project not more than $^1/_{32}$ in. above the back of the flange angles. When full bearing is not feasible, the top flange fastening to the web shall be designed to carry the full wheel load concentration distributed over a distance equal to twice the depth of the crane rail section plus the gauge distance to the top line of fasteners.

On welded plate girders, wheel load concentrations shall be transferred to the web plate of the girder by the web-to-flange weld. In calculating the force per unit length, the wheel load shall be assumed to be distributed over a distance equal to twice the combined depth of the crane rail and girder flange thickness.

5.8.7 Deflection. Maximum deflection of the girders from one crane without vertical impact and any other live loads applied to the girders shall not exceed the following ratios of the span length:

Class A buildings	1/1000
Class B buildings	1/1000
Class C buildings	1/600
Class D buildings	1/600

5.8.8 Girder Camber. Girders of spans greater than 75 ft. shall be cambered for approximately dead load plus half of the live load deflection, without impact.

5.8.9 Attachments. There shall be no attachments or fixtures of any kind, other than those designated on the design drawings, either during or after construction, unless approved by a qualified engineer and added to the drawing as a revision.

5.9 Columns (7.12 and 7.16)

5.9.1 General. Built-up step columns made of two or more segments tied together by solid web plates, lacing or intermittent vertical diaphragms shall have the connecting segments and their connections designed to provide integral behavior of the combined column section.

For columns with intermittent vertical diaphragms or diagonal lacing, the column shafts between panel points and the intermediate web members shall be designed for forces (shear, axial load and bending moments) derived from frame analysis. The effect of out-of-plane bending of columns due to eccentricity of crane girder reactions (longitudinal eccentricity) shall be included.

5.9.2 Brackets. Brackets should not be used to support crane runway girders with total reactions at the column in excess of 50 kips.

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Where crane girders are supported on brackets, impact shall be included in the bracket design and its connection to the column.

5.9.3 Column Bases. Column bases should be above grade and designed to avoid trapping moisture and dirt. Reference should be made to Sections 4.2.3 and 4.2.4 with regard to setting anchor rods and grouting base plates. Wherever columns are subject to damage, consideration shall be given to provide protection by armoring or by other means as specified by the owner.

5.10 Floor Framing

Floors for vibrating machinery of all kinds, together with the supporting framework, shall be rigidly braced in both the horizontal and vertical planes. Special consideration shall be given in the design of floors or structures supporting mechanical equipment to minimize vibration and maintain alignment. Tolerances required by machinery shall be furnished by the owner.

5.11 Side Wall and Roof Framing

The design of light-gauge purlins and girts shall be in accordance with AISI "Specifications and Commentary for the Design of Light Gauge Cold-Formed Steel Structural Members" (Ref. 26). Corrosion should be considered in the design of secondary members where applicable.
Girts shall be connected with a minimum of two bolts at each end.

5.12 Depth Ratio

The following ratios of depth to length shall be used as a guide:

(1) Trusses—1:12
(2) Beams supporting floors for vibrating machinery and track—1:16
(3) Rolled beams and girders for ordinary floors and rafters—1:24
(4) Roof purlins and gable columns—1:32
(5) Girts—1:60
(6) The outstanding legs of tension members having a slope of 45 degrees or less with the horizontal shall be not less than $^1/_{90}$ of the unsupported length.

5.13 Minimum Thickness of Material

The minimum thickness of material exclusive of secondary members such as purlins and girts shall be:

(1) For exterior construction—$^5/_{16}$ in.
(2) For interior construction—$^1/_4$ in.

The controlling thickness of rolled shapes shall be taken as the mean thickness of their flanges, regardless of web thickness. Metal exposed to marked corrosive action shall be suitably protected against corrosion as specified by the owner.

5.14 Connections

Shop and field connections may be riveted, welded or bolted.

Unfinished bolts (A307) may be used for shop and field connections of Class D buildings not containing crane runways or large vibrating equipment, and all structures for connections of secondary members such as purlins, girts, door and window framing and temporary bracing. All other bolted connections shall be made with pretensioned high-strength bolts.

Where connections are bolted, slip critical-type high-strength bolted connections shall be used for members subjected to fatigue cyclic loading or vibrations (Ref. 4). Pretensioned high-strength bolts for bearing-type connections may be used in other connections where specified on drawings.

Appurtenant material shall not be attached to structural members unless added to drawings as a revision and approved by a qualified engineer.

5.15 Spacing of Bolts and Welds

In general, bolted and welded details shall conform to requirements of the AISC Specification except as noted herein.

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In connecting the crane runway girder to the horizontal diaphragm, the bolt spacing shall be no greater than required for full transfer of shear, or no greater than the AISC requirements for intermittent attachment in compression members, or 8 in., whichever is the least. Where a horizontal diaphragm is used and the connection is welded, the weld must be continuous.

5.16 Crane Rails and Joints (7.17)

In the selection of crane rail size, consideration shall be given to the following:

(1) Crane wheel loads (vertical and side thrust)
(2) Crane wheel diameter
(3) Crane wheel hardness
(4) Crane runway activity

For the maximum service life of the rail, it is recommended that Building Classes A and B runways should have head hardened rails, welded rail joints and elastomeric pads.

5.16.1 Bolted Rail Joints. If bolted rail joints are used with nonheat-treated rails; the ends of the rails should be hardened. All bolted rail joints shall be "tight fit," so that the gap between rail ends is no more than $^1/_{16}$ in. The joints shall be staggered across the runway. The amount of stagger shall not equal the wheelbase of the crane. Joints shall not occur at the ends of crane girders. Rail lengths should not be less than 10 ft.

5.16.2 Welded Rail Joints. Rails may be joined by any of several types of procedures. The project specification shall provide complete details as to the approved rail joint welding procedure(s). All finished welds shall have flat bottoms to avoid stress concentrations. Finish profile grinding shall be performed on the rail head.

5.16.3 Rail Clips, Clamps or Attachments. Crane rail clips, clamps or attachments shall be placed in opposing pairs, spaced not over 3 ft. on centers. Spacing is determined by crane wheel side thrust and shall be specified on the project plans. Rail clips, clamps or attachments shall permit the rail to expand and contract longitudinally and limit lateral float to $\pm$ $^1/_8$ in. Hook bolts should not be used on runway systems longer than 500 ft., runway systems in Building Classes A and B, or for cranes with lifting capacities more than 20 tons (Ref. 35). Clearance shall be provided between the ends of the crane rail and obstructions, which would prohibit expansion, such as crane runway stops.

5.16.4 Elastomeric Crane Rail Pads. When elastomeric pads are used, the rail shall be locked laterally by suitable rail clips, which are spaced based on the lateral stiffness of the crane rail and the wheel side thrust.

5.17 Inspection and Quality of Welds

5.17.1 General. All welding shall be performed in accordance with AWS D1.1 (Ref. 3) by qualified welders following qualified welding procedures. As a minimum, all welds shall be visually inspected by an AWS-certified welding inspector, with acceptance criteria as required for statically loaded structures.

5.17.2 Welds on Crane Runway Girders. All crane runway girder welds and acceptance criteria shall meet the requirements of AWS D1.1 (Ref. 3) for cyclically loaded structures. Web plate and flange plate splice welds shall be 100% inspected by radiographic or ultrasonic inspection. Where flange-to-web welds are complete penetration welds, they should be 100% inspected by ultrasonic inspection. Where flange-to-web welds are fillet welds, they should be 100% inspected by liquid penetrant or magnetic particle inspection. Unless otherwise specified in the contract document, all other crane runway girder welds need only comply with Section 5.17.1 of this report.

5.17.3 Other Inspections. Welding inspection that is required but not specified in Section 5.17 of this report shall be specified in the contract documents.

5.17.4 Nondestructive Testing of Other Welds. Additional requirements for nondestructive testing of other welds shall be as required by the specifications and as shown on the design drawings.

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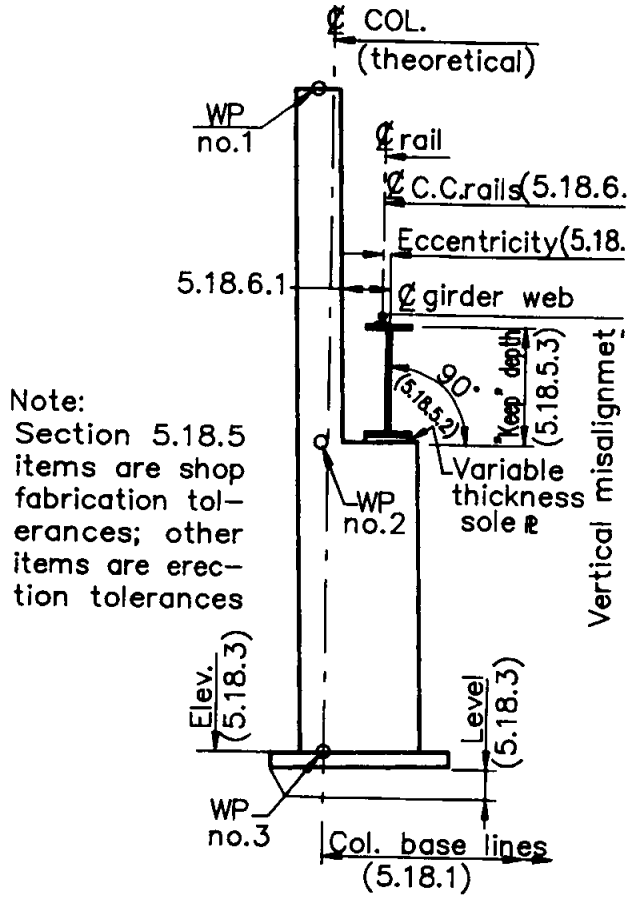

Fig. 5.2—Typical fabrication and erection tolerances.

5.18 Tolerances

5.18.1 Column Base Lines. Column base lines shall be established as parallel lines with gauge maintained to $\pm 1/8$ in.

5.18.2 Anchor Rods. Anchor rods shall be positioned so that no anchor rod deviates from its theoretical position by greater than the factor $0.4H$ where H is equal to the difference between the anchor rod nominal diameter and the diameter of the enlarged hole in the plate, cap plate or column base detail through which the anchor rod passes.

5.18.3 Base Plates. Individual column base plates shall be within $\pm 1/16$ in. of their theoretical elevation and shall be level within 0.01 in. across length or width.

This tolerance shall be maintained at all columns (bay and aisle widths). Two base plates serving as a foundation for a built-up column section shall be at the same level with a total tolerance of $1/16$ in.

5.18.4 Column Fabrication Tolerances. Figs. 5.2 and 5.3 show a typical crane column fabrication. Crane columns shall be shop-fabricated to a work line struck as a straight line between a work point at the bottom of the column, a work point at about the elevation of the crane girder seat and a work point at the top of the column. The work point at about the elevation of the crane girder seat shall not vary more than $\pm 1/8$ in. from the straight line struck between the other two points. AWS straightness tolerances will control between the work points. The girder seat plate is to be located from the column work line with a tolerance of $\pm 1/32$ in.

5.18.5 Crane Runway Girder Fabrication Tolerances

5.18.5.1 Crane Girders. Horizontal sweep in crane runway girders shall not exceed $1/4$ in. per 50-ft. length of girder spans. Camber shall not exceed $\pm 1/4$ in. per 50-ft. girder span over that indicated on the design drawings.

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5.18.5.2 Girder Ends. At the ends of the girder supported by the columns, the bottom flange shall be flat and perpendicular to the web. The flatness tolerance shall be $\pm 1/32$ in. at any point supported by the column cap plate. The perpendicularity of the web to bottom flange shall be less than $\pm 1/64$ in. per foot of flange width.

5.18.5.3 Girder Depths. Depths of crane girders shall be detailed and fabricated to a 'KEEP' dimension at their ends of $\pm 1/32$ in. by use of a variable thickness sole plate.

5.18.6 Crane Girder and Rail Alignment. The centerline of the top of each crane girder at each column shall be aligned horizontally to within $\pm 1/4$ in. of the theoretical base line both sides of the runway.

Center-to-center of crane rails shall not exceed $\pm 1/4$ in. from the theoretical dimensions shown on the drawings adjusted to 68°F. The horizontal misalignment of crane rails shall not exceed $1/4$ in. per 50 ft. of runway with a maximum of $1/2$ in. total deviation from theoretical location.

Vertical misalignment of crane rails shall not exceed $1/4$ in. per 50 ft. of runway with a maximum of $1/2$ in. total deviation from theoretical location.

Crane rails shall be centered on crane girder webs whenever possible. In no case shall the rail eccentricity be greater than three-fourths of the girder web thickness.

5.18.7 Tolerances. Tolerances for fabrication and erection of structural components other than as specified in Section 5.18 of this report shall be in accordance with the applicable requirements of Refs. 1 through 3. It is recommended that an independent alignment/erection survey be conducted prior to the acceptance of the structure in order to ensure that all tolerances specified in Section 5.18.6 are met.

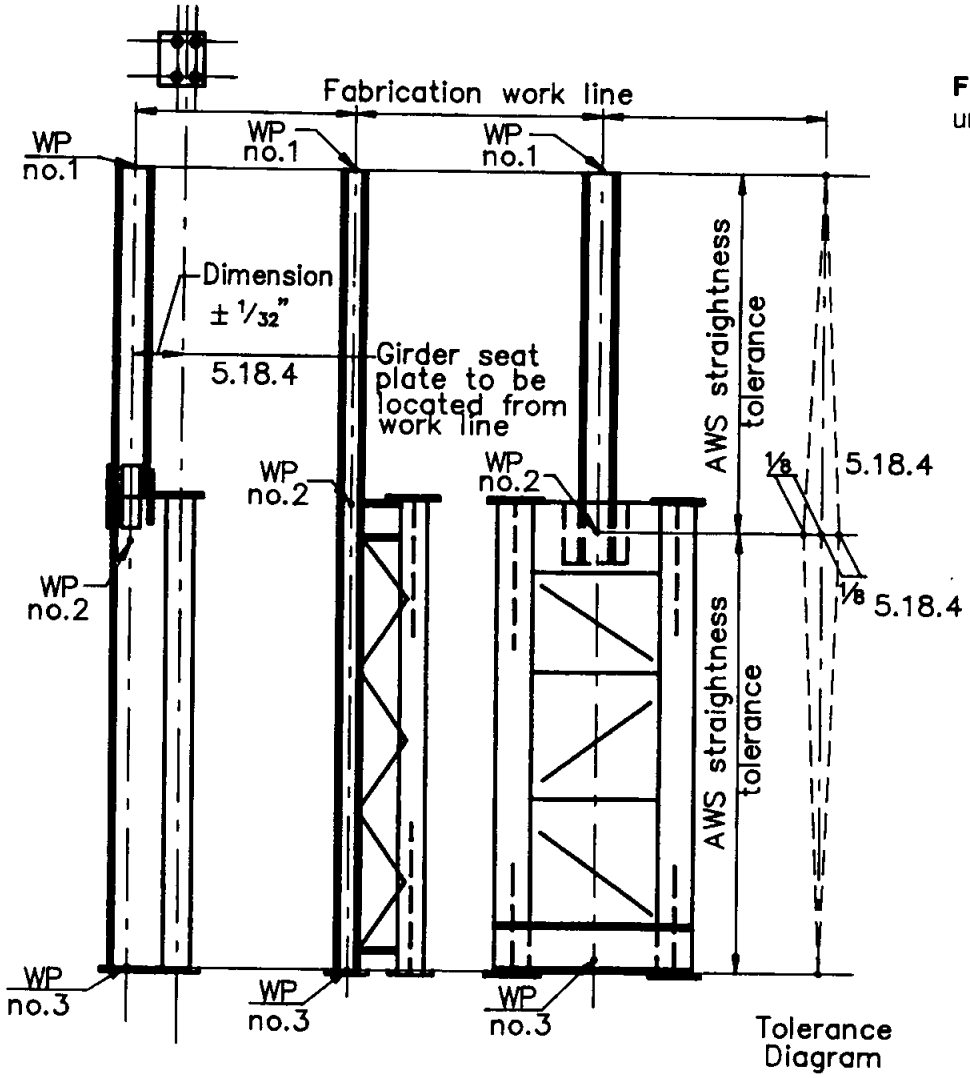

Fig. 5.3—Typical crane column fabrication tolerances.

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6.0 Miscellaneous (Deleted)

7.0 Commentary

7.1 Purpose

It is the purpose of this commentary to amplify, supplement and explain the basis and application of portions of this report not covered elsewhere. Report clauses will be referred to by the appropriate paragraph number.

7.2 Classification of Structures (1.4)

Although a classification A, B, C or D is applied to the entire structure, it should be recognized that only a relatively small portion of the structure usually will be affected.

Crane girders and their supports, which sustain many repetitions of loading, are representative of those portions of the building to which the classifications apply.

7.3 Clearances (1.5.7)

On new buildings a minimum distance of 18 in. is to be provided between the faces of the column and the extremities of the end truck of the crane to provide adequate personnel clearance. Where other means of egress not involving passage between the face of the column and the crane are provided, the 18 in. required in Fig. 1.1 shall be waived and the space limited to preclude personnel clearance.

7.4 Roof Live Loads (3.2)

Recommendation of a minimum live load of 20 psf acting on the entire horizontally projected surface is intended to provide a rigid building even in regions where no snow is to be expected. For those regions where roof snow loads greater than 20 psf live load should be considered because of annual snow pack, and for snow load distribution coefficients, see ASCE 7 (Ref. 9). A snow map in that reference provides information covering the United States. In mountainous regions, local conditions and records should determine specific loading requirements.

7.5 Crane Runway Loads (3.4)

7.5.1 General (3.4.1). During crane operation (bridge and/or trolley travel, hoisting), cranes produce two principal categories of loads on the crane runway support structures: static and dynamic loads.

Static loads are the weight of crane components (bridge, trolley, lifting mechanism, etc.) and a lifted load. These loads shall be provided by the crane manufacturer and compose rated crane vertical wheel loads.

Crane dynamic loads: vertical impact, horizontal side thrust and tractive forces are generated by the crane work (load lifting, trolleying, crane travel) and by the inertia of the crane components and lifted load masses in acceleration and deceleration processes for all functional motions.

Analytical determination of crane dynamic forces shall include consideration of interaction between the mill building and the crane, which develops through the crane runway. In practice, these instantaneous forces have very low probability of simultaneous occurrence.

For existing mill buildings, a field crane test method can be utilized to determine the crane dynamic forces. The most severe crane operation conditions shall be simulated to produce the maximum impact and side thrust forces. A system of strain gauges installed on the crane runway girders and the crane makes it possible to determine the stress fluctuation due to various crane actions producing dynamic forces. The recorded strains then can be transformed into stresses from which, in turn, forces can be calculated.

7.5.2 Vertical Impact, Side Thrust and Traction (3.4.2). In the absence of the appropriate dynamic analyses for crane loads, it is recommended to use the load factors provided in paragraph 3.4.2 to determine the static equivalents to the crane dynamic forces.

For existing mill building upgrade projects, utilization of the load factors recommended in paragraph 3.4.2 for existing mill building upgrade projects could create excessively conservative crane loadings. For recommendations regarding upgrading of existing structures, see Appendix C to this report.

7.5.3 Crane Runway Stops (3.4.3). For the design of the runway and runway stops, the designer must consider the energy-absorbing device used in the crane bumper. The device may be nonlinear (e.g., hydraulic bumpers) or a linear device such as a coil spring.

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The energy absorption device (e.g., hydraulic or spring) shall be designed/selected to satisfy the following criteria:

(1) The deceleration rate for the bridge shall not exceed 16 ft. per second squared at 50% of the full load rated speed or at 50% of the maximum attainable speed, if known.
(2) The device shall be capable of absorbing the energy at 100% of the full load rated speed.
(3) The maximum force generated by the device shall be less than maximum allowable force on the runway stop.

The building, the end stops and their connections shall be designed to withstand the force generated at 100% of the full load rated speed. The recommended increase in the allowable stresses for this condition is 50%.

For computing bridge bumper energy absorption requirements, the trolley must be placed in the end approach producing the maximum end reaction from both bridge and trolley (See Fig. 7.1). This end reaction shall be used as the maximum weight portion of the crane that can act on each bridge bumper. The energy-absorbing capacity of the bumper is based on "power off" and does not normally include the lifted load if it is free to swing.

For new construction, careful attention to reductions in bumper end force can produce cost savings in runway girders, end stop and support structure designs.

The bumper supplier should be required to supply certification of the performance of the proposed bumper. The height of the bumper above the top of rail must be coordinated with or determined by the crane manufacturer.

Fig. 7.2 shows basic calculations for hydraulic and spring bumpers.

7.6 Vibration (3.7.1)

Rotating and vibrating equipment should be designed to minimize excessive vibration levels. Examples are:

- Motors
- Turbines
- Fans/blowers

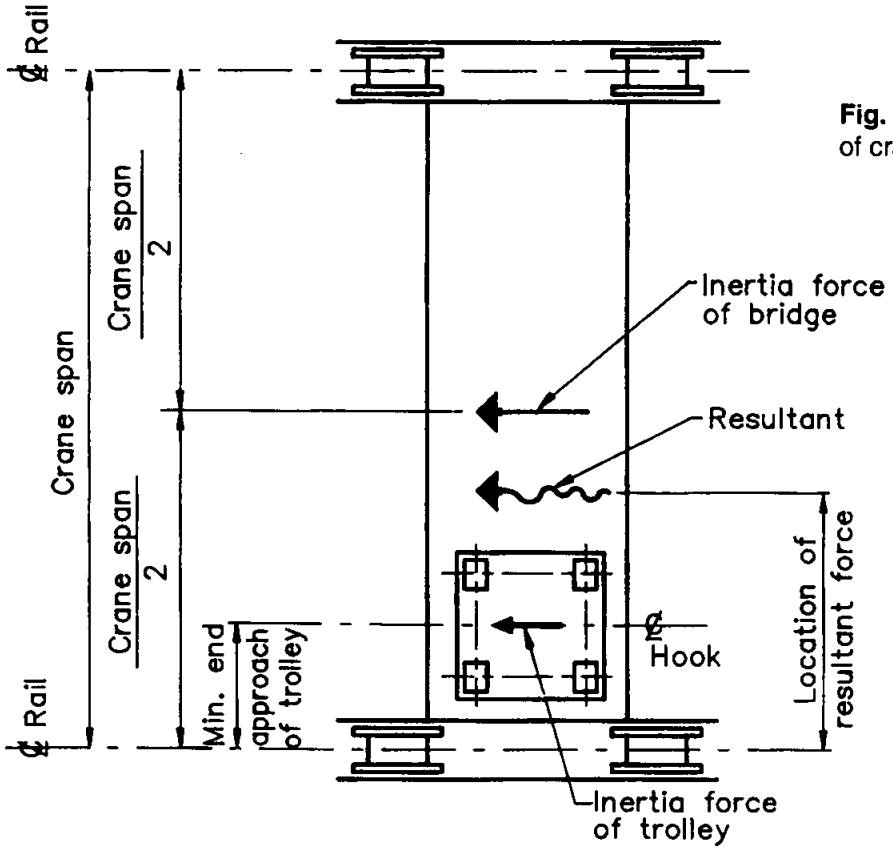

Fig. 7.1—Location of total inertia force of crane.

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- Vibrating screens
- Pan feeders
- Gyratory and jaw crushers
- Mold oscillators
- Compressors

plus other equipment that is subjected to dynamic loadings as a consequence of normal operations. Equipment that is subject to rotating or vibratory loading is susceptible to wear and possible failure if vibration levels are excessive. For some installations, it may be necessary to consider both equipment structure interaction and the effects of soil structure interaction on the supporting structure. This would be the case for equipment installations in very stiff buildings.

Equipment specifications should give the desired relationship between the lowest natural frequency of the equipment and the operating frequency.

7.7 Wind Loads (3.8)

Most jurisdictions in the U.S. base their building code on one of the model building codes. These codes, in turn, base their wind load provisions on ASCE 7 (Ref. 9), subject to the specific limitations and requirements of the local code. For structures located in Canada, compliance should be with the National Building Code of Canada (Ref. 10).

For flexible buildings or structures, a dynamic analysis of wind loads may be appropriate or even required by code.

For structures associated with a high cost of failure or a high cost of overdesign, it may be a good investment to perform model tests in a wind tunnel in order to more accurately estimate the static and dynamic wind behavior of the structure.

7.8 Seismic Forces (3.9)

In determining whether seismic design is required for a structure, one should not assume that significant earthquakes occur only in the western United States. Although earthquakes are less frequent in the east, they may still be quite severe and should be accounted for in the design.

It is important for the design engineer to understand that real earthquakes are a dynamic displacement loading and not just the static force loading assumed by simplified building code calculations. These assumed forces are often greatly reduced from reality and rely on structural ductility well beyond the

Fig. 7.2—Hydraulic Crane Bumper, Runway End Stop Example

Bridge weight, $W_B = 200$ kips

Bridge full load rated speed, $V_B = 6$ ft. per second

Trolley weight, $W_T = 40$ kips

Impact weight per side (load free to swing),

$$W_E = 0.5\ W_B + 0.9^{\dagger}\ W_T = 136 \text{ kips}$$

†*Confirm 0.9 with crane manufacturer*

Allowable deceleration (per *AISE TR 6*):

$$A_{50\%} \le 16 \text{ ft/sec}^2 \text{ @ } 50\% \text{ speed}$$

For deceleration at 50% of full load rated speed:

Stroke must be at least $((V_{50\%})^2(12"))/(2*A_{50\%})$

Stroke must be at least $= \dfrac{3^2 \times 12}{2 \times 6} = 3.38$ in.

Deceleration force must not exceed $(W_e/g) * A_{50\%}$

Deceleration force must not exceed $\dfrac{136}{32.2} \times 16 = 67.6$ kips

Note: The stroke of 3.38 in. is for a 100% efficient bumper and must be adjusted for the actual bumper efficiency. Bumper efficiency varies with the type of bumper and individual bumper design. Typical hydraulic bumper efficiency is 80%, sometimes more. Coil spring bumper efficiency is 50%. For example, if a hydraulic bumper is 80% efficient, then the stroke must be increased to $\dfrac{3.38}{0.8} = 4.23$ in.

For energy absorption at 100% of full load rated speed:

Kinetic energy K_E to be absorbed

$$= \frac{W_E V_B^{\,2}}{2g} = \frac{136 \times 6^2 \times 1000}{2 \times 32.2} = 76{,}025 \text{ lb-ft}$$

The end force F_e is inversely proportional to the stroke, S_B, and to the efficiency, E_B, of the bumper, $F_e = \dfrac{K_E}{S_B E_B}$

For example, a bumper with maximum stroke of 10 in. and 80% efficiency would generate a force of: $\dfrac{76{,}025 \times 12}{10 \times 0.8} = 114{,}038$ lb

If the maximum allowable force on the runway stop is say, 100,000 lb, then the bumper supplier must find a combination of energy absorption, stroke and efficiency that will satisfy this criterion. The bumper must also satisfy the requirement for deceleration at 50% of full load rated speed.

For end force not to exceed 100,000 lb, the bumper stroke would be adjusted as follows: $S_B = \dfrac{76{,}025 \times 12}{100{,}000 \times 0.8} = 11.4$ in.

Therefore, for this example, an acceptable hydraulic bumper at 80% efficiency will have the following characteristics:

minimum stroke length = 11.4 in.
maximum end force at 100% speed = 100 kips

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yield displacement of the structure in order to absorb the energy of the structure's response to an earthquake. It is essential to follow the building code detailing requirements for each construction material in order to provide a safe and ductile structure.

7.9 Load Combinations for Design of Crane Runways and Supporting Structures (3.10)

A major differentiation between building classes arises in the design for repeated loads of crane runway girders and their supports. Reference should be made to Section 1.4 of this report for an indication of the number of cycles expected under the various classifications.

7.9.1 Case 1 Load Combinations (3.10.2.1). These load combinations represent the greatest probable number of load repetitions involving a simple repetition of vertical load from one crane with impact and 50% of the side thrust included.

As an alternate to this method, especially with existing crane runway upgrade projects, the engineer may utilize in fatigue-related analyses the damage accumulation principle for structures subjected to variable amplitude loadings. See Ref. 34 for more information and examples of crane runway fatigue analyses.

7.9.2 Case 2 Load Combinations (3.10.2.2). Case 2 represents the maximum expected load combinations. Fatigue is not a consideration because it is not expected that such critical load positioning will often occur. A load combination of simultaneous side thrust from more than one crane may need to be considered if special cases warrant.

7.9.3 Case 3 Load Combinations (3.10.2.3). Case 3 may occur only a few times during the life of the structure. Examples include the maximum winds that may be expected over a long period of time, the accidental severe impact of a crane hitting the end of a runway, or an earthquake. None of these are to be considered in simultaneous conjunction with each other because of the low probability that such a combination would ever occur.

7.10 Soil-Bearing Foundations (4.3)

Mill buildings are generally tolerant of small differential settlements on the order of 1 or 1 $^1/_2$ in. Larger values may cause difficulties in the operation of cranes, especially with respect to drift during critical operations such as teeming.

In addition, large differential settlements can cause stress problems in the structure depending on its flexibility and connectivity.

In planning the foundation for a mill structure, the engineer should consider but not be limited to the following:

- Variations of the compressible strata in the area of the structure.
- Surcharge effects of equipment or material storage.
- Combinations of different types of foundations as well as spread footings at different elevations.
- Vibration or shock loads.

Each of these might cause differential settlement and might be a reason for changing foundation to piling or other construction.

7.11 Expansion Joints in Floor Slabs on Grade (4.6.7)

Use of expansion joints in floor slabs is not recommended due to the possibility of differential settlement between adjacent slabs.

7.12 Column and Truss Bents (5.9)

This report requires that columns made up of two or more shafts that are connected by intermittent diaphragms (battens or lacing bars) shall be designed so as to act as an integral unit. If there is a continuous longitudinal web plate joining these shafts, the moment of inertia of the complete solid section may be used in the frame analysis.

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7.13 Building Expansion Joints (5.6)

Expansion joints shall preferably be constructed of two (2) independent column shafts.

7.14 Allowable Stress Ranges under Repeated Loads (5.7)

Under cyclic loadings, members, fasteners or weld metal may ultimately develop fatigue cracks, which may lead to structural failure.

The provisions of this section are intended to prevent such failure by imposing stress limitations, which recognizes severity of local stress concentration and number of load cycles. The latest North American fatigue provisions use equations to calculate the design stress range for a chosen design life. Loading conditions are no longer included. The designer should determine the anticipated number of cycles of full load or equivalent by means of load spectrum analysis (mean effective load) or cumulative fatigue damage assessment. The loading conditions have been retained in this report for guidance in the absence of more definite information.

7.15 Crane Runway Girders (5.8)

Simple beam girders are preferred by most mill building engineers because they are readily replaced if damaged, are unaffected by differential settlement of column footings, and may be analyzed readily for the multiplicity of load combinations involving both vertical and lateral forces. To avoid undue end rotation and deflection, they should be as deep and stiff as is practicable within the limits of economy and clearance requirements.

In the design of girder webs, it is to be noted that the tension field design introduced into the AISC Specification in 1961 is not permitted by this report for crane runway girders. The earlier practice of keeping web shear stresses below values that would cause buckling is recommended. The primary reason for not adopting the AISC procedure for girder web design is to avoid lateral bending of the web, which reduces the life of a plate girder under repeated loads. For entirely static load situations, the AISC design procedure should be acceptable. The usual crane runway girder includes a horizontal web plate, which resists shears and participates in resisting bending moments induced by horizontal side thrust loads. The horizontal web usually has the additional function of providing a walkway surface. There also will be one or two vertical web plates, depending on whether or not there is another runway at the same level in an immediately adjacent aisle.

Class A, B or C crane runway girders with spans of 36 ft. or more are required to have their bottom flanges braced by a horizontal truss system. A complete box girder is thus formed. A vertical truss may be substituted for the unloaded vertical web if there is only one runway rail to support. If frequent cross-diaphragms are provided and are designed to transfer a proper share of the loads, the entire box section may be considered to act as a unit in resisting both vertical and horizontal forces. If only a few cross-diaphragms are provided, or none at all, the horizontal side thrust forces are resisted primarily by the upper portion of the girder, and the vertical forces are resisted primarily by the particular vertical side of the girder on which they act. In the latter case, differential deflections will occur when only one side of the girder is loaded. Such deflections introduce cross-bending stresses at the juncture of the web and compression flange and will tend to increase the likelihood of fatigue failure. These deflections should be reduced to a minimum by appropriate bracing systems.

The *AISE* sponsored an engineering study by Cornell University, the objective of which was to investigate welded crane runway girders using 3-D finite element analysis and to develop design recommendations supported by the performance of full-scale tests. The results of the study are covered in a separate *AISE* publication entitled Welded Crane Runway Girder Study (Ref. 37).

7.15.1 Rolled Shapes and Built-up Single Web Plate Girders Having an Axis of Symmetry in the Plane of Their Web (5.8.2.1). This section applies only to open section girders for which the possibility of lateral-torsional buckling must be considered in design. Eq 5.1 provides a transition formula to check the compressive stress in the top flange when simultaneous vertical and lateral loadings are under consideration. In the determination of f_{by} due to lateral load, the stress shall be determined on the basis of the sectional properties of the crosshatched portion of Fig. 7.3a about the (Y-Y) axis, with a modified lateral load $Q_{(eff)}$, as shown in Fig. 7.3b. The allowable bending stress for moment due to the lateral load F_{by} need not be reduced. The use of $Q_{(eff)}$ in place of Q shall not be considered as a requirement in the case of fully boxed members.

The use of Eq 5.1 applies to Case 2 loadings only. In checking for adequacy under repeated loads, for Case 1 loading, lateral buckling need not be a consideration.

7.15.2 Unsymmetrical Built-up Members and Closed Section Girders Without Diaphragms along the Length (5.8.2.2 and 5.8.2.3). The provisions of this section do not exclude application to sections braced on

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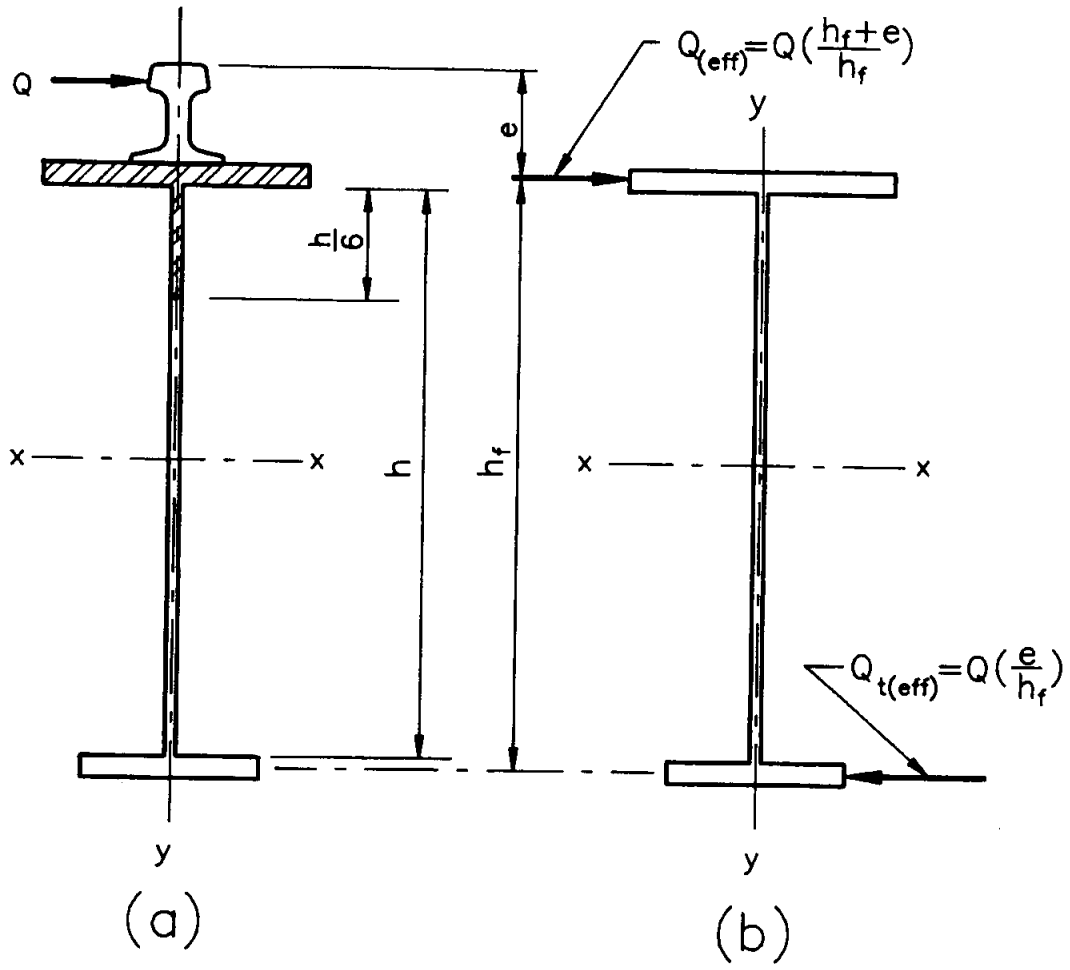

Fig. 7.3—Effective side thrust and effective area resisting side thrust in open section girders.

all sides in which only one or two cross-braces or diaphragms are included, as they would be insufficient to provide continuity of integral box action as covered in Section 5.8.2.3.

The provisions of this section are as shown in Fig. 5.1. Although Eq. 5.1 may be applied, there is no lateral buckling problem, and for Case 2 loadings $F_{bx} = F_{by} = 0.6F_y$, for which the equation becomes simply a check for maximum combined stresses:

$$f_{bx} + f_{by} \leq 0.6F_y \qquad \text{(Eq. 7.1)}$$

The maximum combined stress due to vertical and lateral loading (acting left to right) will be compression at the upper left corner of the cross-section. In calculating f_{bx} due to vertical load, the section properties of the area made up of the diagonal and horizontal crosshatched portions should be considered as effective. The AISC Specification permits an outstanding part in compression to have a width-to-thickness ratio of $\frac{95}{\sqrt{F_y}}$. This applies to half of the flange segment projecting to the left in Fig. 5.1. To the right, it is assumed that the empirical equation, Eq. 5.2, as shown in Section 5.8.2.2, applies. This equation stems from width to thickness ratios allowable when two longitudinal edges of a plate in compression are supported, modified to take care of variation in thickness and to account for the fact that only one side of the girder is effective when frequent regularly spaced intermittent diaphragms are lacking. In calculating f_{by}, the section properties of the area made up of diagonally and vertically crosshatched portions should be considered as effective in resisting horizontal loads (Fig. 5.1). In considering horizontal side thrust, the compression flanges are supported against lateral buckling in the vertical direction by either the main girder or backup truss, depending on the direction of the

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thrust. In considering vertical load, the compression flange at the top is supported against lateral buckling in a horizontal direction by the horizontal side thrust girder.

7.15.3 Bottom Flange Bracing (5.8.4). Although a horizontal truss bracing system is often used to stiffen the tension flange of the crane girder, such a stiffening truss, unless accompanied by cross-bracing at frequent intervals, does not prevent vertical differential deflection of the crane girder with respect to the auxiliary backup truss or adjacent crane girder of interior aisles. For additional information, see Section 7.15.

7.15.4 Stiffeners (5.8.5). Bearing stiffeners designed by AISC specifications are designed essentially as columns. Buckling is prevented in the plane of the web, and the application of the column formula need be applied only for buckling normal to the plane of the web.

Formerly, intermediate stiffeners were not attached to the top flange and were usually specified as tight fit. The resulting gap caused by fabrication tolerance is sufficient to allow the top flange to rotate about the longitudinal axis with each passage of the crane. This transverse bending stress, in combination with the local effects of the crane wheel loads, has caused fatigue cracks in the web after load applications well below 1,000,000 cycles. Laboratory results have shown that welding the stiffener to the underside of the top flange reduces the stresses in the web to a level that provides a significantly longer fatigue life. Other means of restricting such rotation are acceptable. For more detailed discussion and test results, see ASCE *Journal of the Structural Division,* Vol. 102, No. ST5, May 1976.

7.15.5 Local Wheel Support (5.8.6). Previous editions of this report included a formula for computing these stresses. This formula was based on several assumptions that led to results that have been shown to be conservative by a factor of approximately 4. The recommendation for including this stress in design calculation has been deleted from this report because the actual magnitude of these stresses is insignificant for these types of structures.

Considerable research regarding localized wheel load effects has been performed in both Europe and the U.S. during the 1960s through the 1980s. In the U.S., major studies were undertaken by a number of steel producers. Significant results of these studies include recommendations to use a full-penetration weld for the web-to-flange connection, to use deep copes on the vertical stiffeners, and to attach the stiffeners to the bottom of the compression flange using a full-penetration weld.

7.16 Columns (5.9)

Most overhead travelling crane runway girders are supported by stepped columns.

The upper segment of the stepped column, which supports the roof truss, is usually represented by the solid section (wide flange or built-up). The lower segment, which is significantly larger to support crane girders, can be fabricated from different elements as follows:

(1) A single heavy wide flange section.
(2) Two separate elements (building and crane) connected by a continuous web plate, intermittent lacing members, or battens.

Different solutions of built-up columns and mill building stepped columns are shown and discussed in Ref. 25. Spaced columns are not included in these recommendations.

The reader is referred to Ref. 25 for additional information.

The determination of the equivalent length of a stepped column about the strong axis (x-x on Fig. 7.4) is necessary for this column analysis utilizing AISC interaction formulas in applicable cases.

The equivalent length factors for the lower segment (K_L) have been developed for determination of the effective slenderness ratio about the column's strong axis as a function of three parameters:

a_r	=	Ratio of the length of the upper segment to the total length of column
B	=	Ratio of the maximum moment of inertia of the lower segment to the moment of inertia of the upper segment about the strong axis
P_1/P_2	=	Ratio of the axial load in the upper segment (roof and wall load) to the load delivered to the lower segment by crane girders

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Values of the equivalent column length factors for the lower segment in terms of above three parameters are listed in Tables 7.1 through 7.4. These tables are strictly applicable to columns for which the crane column element segment is connected to the building column element by a continuous longitudinal web plate. However, assuming an integral behavior of laced or battened columns, moments of inertia for these lower segments (to use in calculating the "B" ratio) can be determined as a moment of inertia of a combined section.

Equivalent length factors for the upper segment (K_U) can be determined from equation 7.2:

$$K_U = K_L \sqrt{\frac{\left(1 + \frac{P_1}{P_2}\right)}{\left(\frac{P_1}{P_2}\right) B}} \qquad \text{(Eq. 7.2)}$$

The equivalent length factors are applied to the total column length L to determine the equivalent length of each segment.

For buckling about the strong axis (x-x of Fig. 7.4), the stepped column is usually laterally unsupported over its entire length, and for this condition the referenced tables are applicable. For buckling about the y-y axis, lateral support is provided by the crane runway girder at its seat and a backup system (if available), location B in Fig. 7.4.

For buckling about X-X, Table 7.1 assumes a hinge at the base, C, and Tables 7.2 through 7.4 assume the base fully fixed. If less than full fixity is provided by the column anchorage, the equivalent length coefficient should be estimated on the basis of interpolation between appropriate tables.

Exterior wall girts are not assumed to provide longitudinal support in mill buildings for buckling analysis about the Y-Y axis. If the base at C can be considered fully fixed for buckling about the Y-Y axis, then K should be taken

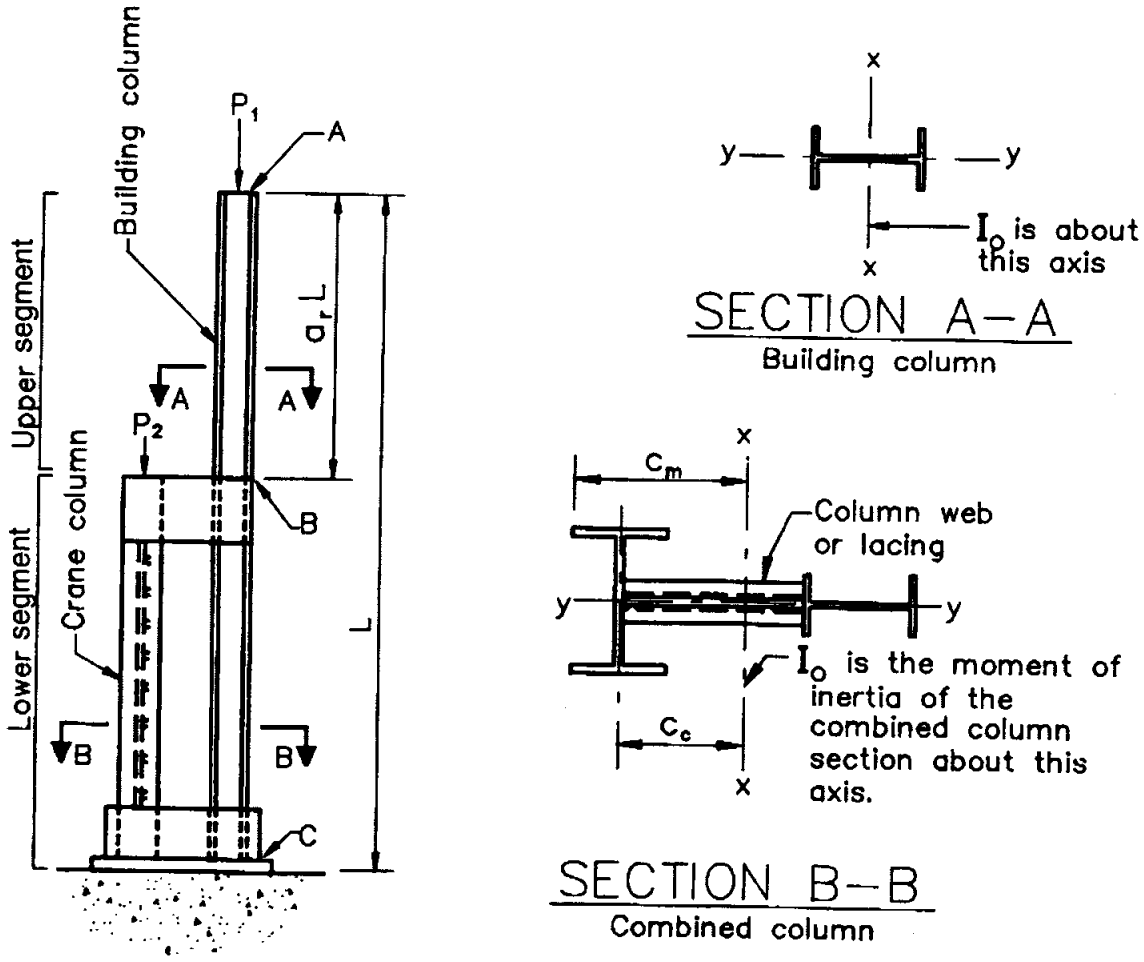

Fig. 7.4—Notation for stepped columns as used in Tables 7.1 through 7.4.

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as 0.8. If less than full rotational restraint is provided by the column anchorage for buckling about Y-Y, K may be assumed to be equal to 1.0.

As an alternative to a tabulated form for definition of stepped column equivalent length ratios about the strong axis (X-X), a computerized solution is available for all column end conditions.

Checking of mill building stepped columns for both axial compression and biaxial bending stresses can be accomplished by utilizing the following equations:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f'_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \qquad \text{(Eq. 7.3)}$$

$$\frac{f_a}{0.6 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad \text{(Eq. 7.4)}$$

Eqs. 7.3 and 7.4 are nearly identical to Eqs. H1-1 and H1-2 of the AISC specification (Ref. 1), except for the introduction of f'_a as differentiated from f_a.

It shall be noted that, if one of the stress terms f_a, f_{bx} and f_{by} in Eqs. 7.3 and 7.4 is taken in its maximum value, the other terms shall be taken in corresponding values determined for a particular load case and at the same section of the column.

The check of the upper segment (A-B) involves axial compression and bending about only one (X-X) axis, and it does not need special explanation.

As applied to the lower segment (B-C) of the mill building column elements shown on Fig. 7.4, the terms in Eqs. 7.3 and 7.4 are discussed and defined as follows:

The lower segments of mill building columns should be divided in two categories:

(a) Columns with a continuous web plate between building and crane column elements.
(b) Columns with intermittent connections between building and crane column elements such as diagonal lacing or battens.

The above two categories of the column lower segments should be modeled in a different manner in the computer framing analyses. The solid web column (category "a") can be modeled as a single member between the column base and the girder seat. The intermittently connected column (category "b") should be modeled as a system of members representing separate building and crane column elements with lacing or battens. This modeling technique will permit one to determine member forces in all components of the built-up columns.

7.16.1 Columns with a Continuous Web Plate Between Building and Crane Column Elements. Terms in Eqs. 7.3 and 7.4 are as follows:

f_a — Maximum or corresponding axial stress $(P_1 + P_2)/A$ where A is area of the entire lower segment cross-section.

f'_a — In checking the lower segment for bending about the y-y axis (see Fig. 7.4), it is conservatively assumed that the crane column element resists all of the bending introduced by eccentricity of crane girder reactions. Thus, the amplification of f_{by} as a result of deflection is dependent on the average axial stress (f'_a) in the crane column element alone. The stress f'_a is determined by adding (or subtracting) the average stress due to moment about the x-x axis, calculated at the centroid of the crane column element, to (or from) the average stress f_a of the entire lower segment.

F_a — This is the allowable axial stress under axial load. It may be for buckling of the entire stepped column about the x-x axis, based on the equivalent length KL/r_x, or it may be determined by buckling about the y-y axis for whatever column length is unsupported. For monosymmetrical columns, where the crane and building column elements are made of

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different sections, consideration of flexural-torsional and torsional buckling of the whole section is required (Ref. 1, paragraph E3 and Ref. 35, Appendix E). The minimum of the F_a values should be used in equations 7.3 and 7.4.

C_{mx} For bending in the plane of the bent, about (X-X), assume a value of 0.85 when all bents are under simultaneous wind load and when side sway is assumed to take place. When one bent is being considered, under maximum crane loading without wind (Case 2 loading), assume a value of 0.95 for C_{mx}. The foregoing values of C_{mx} are applied to the lower segment only.

C_{my} In determining this coefficient, as well as values of all other parameters in the third term of Eq. 7.3, only the crane column element is assumed to be effective. It should be assumed as not rotationally restrained at the top (at B) but is supported against joint translation longitudinally at the same location. Although not restrained, bending due to eccentric loading can be introduced at B. The base (C) usually may be assumed fixed unless footing conditions are poor. Assuming no interaction with the building column element, half of the moment introduced at B as a result of unequal reactions on adjacent girder spans will be carried over to the base, in which case C_{my} = 0.4. If base fixity at C of the crane column element cannot be assumed, take C_{my} = 0.6 (hinged conditions) or interpolate between 0.4 and 0.6.

f_{bx} This is the maximum or corresponding compression stress due to bending about (X-X) at crane or building column elements determined using the entire lower segment section properties.

f_{by} This is the maximum or corresponding compression stress due to bending (Y-Y) in crane column element (see above notes regarding C_{my})

F_{bx} For compression of the crane or building column element side, F_{bx} would be the allowable extreme fiber stress due to bending, reduced below $0.6F_y$ if necessary, because of lack of lateral support. The allowable stress reduction may be based on the allowable axial stress in the crane column element, acting as a column in bending about the (Y-Y) axis as shown in Fig. 7.4. The (Y-Y) axis in this sketch would correspond to the (X-X) axis of the individual crane column element in the steel shapes manual, if a rolled section were used. The allowable column stress, so determined, should be multiplied by the ratio c_m/c_c as defined by Section BB in Fig. 7.4. If this stress is greater than $0.6F_y$, the smaller of the two should be taken as F_{bx}.

F'_{by} Since this component of bending is about the weak axis of the combined crane and building column elements, no reduction in allowable stress need be made for lateral buckling. Also, since the bending resistance is considered to be provided solely by the crane column element of the lower segment the allowable stress for a compact section may be used, if the provisions of Section F1.1 of Ref. 1 are met.

F'_{ex} Since this stress is used as a basis for the determination of the amplification of the column deflection in the plane of bending it should be based on the equivalent length of the complete stepped column, as in the case of F_a, for bending about the x-x axis.

F'_{ey} If the base may be assumed as fixed, let K = 0.8 for the crane column element alone; otherwise, assume that K = 1.0. The length in the determination of KL would be that of the crane column element BC.

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7.16.2 Laced or Battened Columns. For laced or battened columns, terms in Eqs. 7.3 and 7.4 are as follows:

f_a The maximum or corresponding axial compression stress at the crane or building column element is based on axial member forces and the cross-sectional area of the element.

f'_a For the crane column element $f'_a = f_a$, and for the building column element $f'_a = 0$.

F_a This is the allowable stress under axial load. For each column element, F_a shall be determined separately based on slenderness ratios (KLx/r_x and KLy/r_y) about strong and weak axes. In most cases the separate crane and building column elements are doubly symmetrical sections, for which flexural-torsional buckling checking is not required.

C_{mx} and C_{my} For the crane column element, a definition of these bending coefficients about the combined column section strong and weak axes is similar to the definition provided for columns with a solid web plate.

C_{mx} = 0.85 for the building column element.

f_{bx} The maximum or corresponding compression stress in the crane or building column element due to bending about the strong axis (x-x) of the combined column section.

f_{by} The maximum or corresponding compression stress in the crane column element due to bending about the weak axis (y-y) of the combined column section.

F_{bx} The allowable compression stress for separate column elements due to bending about the strong axis of the combined column. In most cases bending in this plane for the lower segments with separate column elements is the local bending between points where the battens or diagonals are connected to the crane or building segment.

F_{by} The allowable compression stress due to bending of the crane column element about the weak axis of the combined column section.

Unbraced lengths for the lateral support about x-x and y-y axes of the lower segment shall govern in determination of the F_{bx} and F_{by} for each column element following recommendations of AISC (Ref. 1).

F'_{ex} and F'_{ey} Same as for the columns with a solid web plate.

Web members (diagonals or battens) should be analyzed for the action of the member forces (axial, shear and bending) following the design criteria recommended by AISC (Ref. 1).

7.17 Crane Rails and Joints (5.16)

The project specification should include reference to acceptable stock lengths, composition, hardness and tolerances for crane rails. By specifying the rail stock length, the quantity of joints can be controlled. Heat-treated rails can help to extend rail life. "Tight fit" bolted splices implement round holes in the rail, round holes in the splice bar and round shank bolts. The rail ends are drawn together upon tightening, thereby eliminating the gap between ends. When bolted splices are not properly maintained, they can become loose, causing high-impact forces and runway and crane damage. On runways in Building Classes A and B, crane rails should be joined to run continuously by puddle arc, manual stick, thermite, flash butt, gas pressure or other acceptable welding method. The project specification shall provide details as to the approved welding procedure(s). Continuously welded rails eliminate the impact forces from crane wheels traversing the rail joints. Special consideration should be given to the rail attachment method. Crane loads, the presence of side guide rollers, floating rails, thermal expansion, runway length and several other factors influence attachment selection and spacing.

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Elastomeric pads installed continuously under the crane rails reduces fatigue, vibration, impact forces and noise. Finished profile grinding should be performed before the crane rail has been placed in its final position.

7.18 Mains, Ducts and Pipes (3.7.5)

In the design of duct systems and their supports, consideration should be given to but not limited to:

- Internal condensation
- Potential explosive limits of the conveyed gases
- Duct collapse due to negative pressures
- Differential expansion between inner and outer shells of water-cooled ducts
- Radiation of heat from air-cooled ducts
- Insulation
- Solar loads
- Thermal fatigue
- Contraction and expansion of steam lines
- Corrosion allowance

Refractory (or otherwise lined) mains, ducts or pipes subjected to water-cooled gas flows should be investigated for possible arching effects due to differential temperatures between upper and lower supports.

Ducts conveying saturated gases or gas with entrained water should be sloped to a suitable drain pocket. The slope should take into consideration the nature of the dust load in the gas.

Consideration should be given to an accidental full dust loading of any duct due to unusual operating procedures or processes.

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Table 7.1 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns

Column ABC Hinged at A and Hinged at C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.00$															
1.00	0.91	0.83	0.80	~0.79	0.78	0.77	0.76	~0.75	0.74	0.74	0.73	~0.73	0.73	0.73	0.73
2.00	0.91	0.84	0.80	~0.79	0.79	0.78	0.78	~0.78	0.78	0.78	0.79	~0.79	0.81	0.91	0.94
3.00	0.91	0.84	0.81	~0.80	0.80	0.80	0.80	~0.81	0.82	0.84	0.85	~0.87	0.89	0.91	0.94
5.00	0.91	0.84	0.82	~0.82	0.83	0.84	0.86	~0.890	0.92	0.95	0.98	~1.02	1.06	1.09	1.13
10.00	0.91	0.85	0.87	0.90	0.94	0.99	1.04	1.10	1.16	1.22	1.28	1.34	1.40	1.45	1.51
20.00	0.91	0.88	1.03	1.11	1.20	1.29	1.38	1.47	1.56	1.65	1.74	1.83	1.92	2.00	2.08
40.00	0.91	1.01	1.36	1.50	1.63	1.76	1.90	2.03	2.16	2.29	2.42	2.55	2.67	2.79	2.90
70.00	0.91	1.26	1.77	1.95	2.12	2.30	2.48	2.65	2.83	3.01	3.18	3.34	3.51	3.67	3.82
100.00	0.91	1.49	2.10	2.31	2.52	2.74	2.95	3.16	3.37	3.58	3.79	3.99	4.19	4.37	4.56
$P_1/P_2 = 0.10$															
1.00	0.92	0.85	0.81	~0.80	0.79	0.79	0.78	~0.77	0.77	0.76	0.76	~0.76	0.76	0.75	0.75
2.00	0.92	0.85	0.82	~0.81	0.81	0.81	0.81	~0.81	0.81	0.82	0.83	~0.84	0.85	0.86	0.87
3.00	0.92	0.85	0.83	~0.83	0.83	0.83	0.84	~0.85	0.87	0.88	0.90	~0.92	0.95	0.97	0.99
5.00	0.92	0.86	0.85	~0.86	0.87	0.90	0.92	~0.95	0.99	1.02	1.06	~1.09	1.13	1.16	1.20
10.00	0.92	0.88	0.94	0.98	1.03	1.09	1.151.	1.21	1.27	1.33	1.39	1.45	1.51	1.57	1.62
20.00	0.92	0.95	1.17	1.26	1.35	1.44	1.54	1.63	1.72	1.81	1.90	1.99	2.08	2.16	2.24
40.00	0.92	1.19	1.58	1.71	1.85	1.99	2.12	2.26	2.39	2.52	2.65	2.77	2.90	3.02	3.13
70.00	0.93	1.52	2.05	2.23	2.42	2.60	2.78	2.96	3.13	3.31	3.48	3.65	3.81	3.97	4.12
100.00	0.95	1.80	2.44	2.66	2.87	3.09	3.31	3.52	3.74	3.94	4.15	4.35	4.54	4.73	4.92
$P_1/P_2 = 0.20$															
1.00	0.92	0.85	0.81	~0.80	0.79	0.79	0.78	~0.77	0.77	0.76	0.76	~0.76	0.76	0.75	0.75
2.00	0.92	0.85	0.82	~0.81	0.81	0.81	0.81	~0.81	0.81	0.82	0.83	~0.84	0.85	0.86	0.87
3.00	0.92	0.85	0.83	~0.83	0.83	0.83	0.84	~0.85	0.87	0.88	0.90	~0.92	0.95	0.97	0.99
5.00	0.92	0.86	0.85	~0.86	0.87	0.90	0.92	~0.95	0.99	1.02	1.06	~1.09	1.13	1.16	1.20
10.00	0.92	0.88	0.94	0.98	1.03	1.09	1.151.	1.21	1.27	1.33	1.39	1.45	1.51	1.57	1.62
20.00	0.92	0.95	1.17	1.26	1.35	1.44	1.54	1.63	1.72	1.81	1.90	1.99	2.08	2.16	2.24
40.00	0.92	1.19	1.58	1.71	1.85	1.99	2.12	2.26	2.39	2.52	2.65	2.77	2.90	3.02	3.13
70.00	0.93	1.52	2.05	2.23	2.42	2.60	2.78	2.96	3.13	3.31	3.48	3.65	3.81	3.97	4.12
100.00	0.95	1.80	2.44	2.66	2.87	3.09	3.31	3.52	3.74	3.94	4.15	4.35	4.54	4.73	4.92

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Table 7.1 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 2

Column ABC Hinged at A and Hinged at C

B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.30$															
1.00	0.93	0.87	0.84	~0.83	0.82	0.82	0.81	~0.81	0.80	0.80	0.73	~0.80	0.79	0.79	0.79
2.00	0.93	0.87	0.85	~0.88	0.85	0.85	0.85	~0.86	0.87	0.88	0.79	~0.90	0.91	0.92	0.94
3.00	0.93	0.88	0.87	~0.87	0.88	0.89	0.90	~0.92	0.94	0.98	0.85	~1.00	1.03	1.05	1.07
5.00	0.93	0.89	0.91	~0.93	0.95	0.98	1.02	~1.05	1.09	1.16	0.98	~1.20	1.24	1.27	1.31
10.00	0.93	0.93	1.05	1.11	1.17	1.23	1.30	1.36	1.42	1.55	1.28	1.61	1.67	1.72	1.78
20.00	0.94	1.09	1.36	1.46	1.56	1.66	1.75	1.85	1.94	2.13	1.74	2.22	2.30	2.39	2.47
40.00	0.95	1.44	1.86	2.01	2.15	2.29	2.43	2.57	2.70	2.97	2.42	3.10	3.22	3.34	3.46
70.00	1.00	1.86	2.43	2.62	2.81	3.00	3.19	3.38	3.55	3.73	3.91	4.07	4.24	4.40	4.56
100.00	1.11	2.20	2.89	3.12	3.35	3.58	3.80	4.02	4.24	4.45	4.66	4.86	5.06	5.25	5.44
$P_1/P_2 = 0.40$															
1.00	0.93	0.88	0.85	~0.84	0.84	0.83	0.83	~0.82	0.82	0.81	0.81	~0.81	0.81	0.81	0.81
2.00	0.93	0.88	0.87	~0.86	0.86	0.87	0.87	~0.88	0.89	0.90	0.91	~0.92	0.93	0.95	0.96
3.00	0.93	0.89	0.89	~0.89	0.90	0.91	0.93	~0.94	0.96	0.99	1.01	~1.03	1.05	1.08	1.10
5.00	0.94	0.90	0.93	~0.96	0.98	1.02	1.05	~1.09	1.13	1.16	1.20	~1.24	1.28	1.31	1.35
10.00	0.94	0.96	1.10	1.16	1.22	1.29	1.35	1.42	1.48	1.54	1.61	1.67	1.73	1.78	1.84
20.00	0.94	1.15	1.43	1.53	1.63	1.73	1.83	1.93	2.03	2.12	2.21	2.30	2.39	2.47	2.55
40.00	0.97	1.52	1.96	2.11	2.26	2.40	2.54	2.68	2.82	2.95	3.09	3.22	3.34	3.46	3.58
70.00	1.05	1.97	2.56	2.76	2.95	3.15	3.33	3.53	3.71	3.89	4.06	4.23	4.40	4.56	4.72
100.00	1.19	2.34	3.05	3.29	3.52	3.75	3.98	4.20	4.42	4.63	4.85	5.05	5.25	5.44	5.63
$P_1/P_2 = 0.50$															
1.00	0.94	0.89	0.86	~0.85	0.85	0.84	0.84	~0.83	0.83	0.83	0.83	~0.82	0.82	0.82	0.82
2.00	0.94	0.89	0.88	~0.88	0.88	0.88	0.89	~0.89	0.90	0.92	0.93	~0.94	0.95	0.97	0.98
3.00	0.94	0.90	0.90	~0.91	0.92	0.93	0.95	~0.97	0.99	1.01	1.03	~1.06	1.08	1.10	1.13
5.00	0.94	0.92	0.96	~0.98	1.01	1.04	1.08	~1.12	1.16	1.20	1.24	~1.28	1.31	1.35	1.39
10.00	0.94	0.99	1.14	1.20	1.27	1.33	1.40	1.46	1.53	1.59	1.66	1.72	1.78	1.83	1.89
20.00	0.95	1.19	1.49	1.59	1.70	1.80	1.90	2.00	2.09	2.19	2.28	2.37	2.46	2.55	2.63
40.00	0.98	1.59	2.05	2.20	2.34	2.49	2.64	2.78	2.92	3.05	3.19	3.32	3.44	3.57	3.68
70.00	1.10	2.07	2.68	2.88	3.07	3.27	3.46	3.65	3.84	4.02	4.20	4.37	4.53	4.69	4.86
100.00	1.25	2.46	3.18	3.42	3.66	3.89	4.12	4.35	4.57	4.79	5.01	5.21	5.41	5.61	5.80

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Table 7.1 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 3

Column ABC Hinged at A and Hinged at C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.60$															
1.00	0.94	0.89	0.87	~0.86	0.86	0.85	0.85	~0.84	0.84	0.84	0.84	~0.84	0.83	0.83	0.83
2.00	0.94	0.90	0.89	~0.89	0.89	0.90	0.90	~0.91	0.92	0.93	0.94	~0.96	0.97	0.98	1.00
3.00	0.94	0.91	0.91	~0.92	0.93	0.95	0.97	~0.99	1.01	1.03	1.05	~1.08	1.10	1.13	1.15
5.00	0.94	0.93	0.98	~1.00	1.04	1.07	1.11	~1.15	1.19	1.23	1.27	~1.30	1.34	1.38	1.42
10.00	0.95	1.01	1.17	1.24	1.30	1.37	1.44	1.50	1.57	1.63	1.70	1.76	1.82	1.88	1.93
20.00	0.96	1.24	1.54	1.65	1.75	1.85	1.96	2.06	2.15	2.25	2.34	2.43	2.52	2.61	2.69
40.00	1.00	1.65	2.12	2.27	2.42	2.57	2.72	2.86	3.00	3.14	3.27	3.40	3.53	3.65	3.77
70.00	1.14	2.15	2.77	2.97	3.17	3.37	3.56	3.76	3.94	4.13	4.31	4.48	4.65	4.82	4.97
100.00	1.31	2.55	3.30	3.54	3.78	4.02	4.25	4.48	4.70	4.92	5.14	5.34	5.55	5.75	5.93
$P_1/P_2 = 0.70$															
1.00	0.95	0.90	0.88	~0.87	0.87	0.86	0.86	~0.85	0.85	0.85	0.85	~0.85	0.85	0.85	0.84
2.00	0.95	0.91	0.90	~0.90	0.90	0.91	0.91	~0.92	0.93	0.95	0.96	~0.97	0.99	1.00	1.01
3.00	0.95	0.92	0.93	~0.94	0.95	0.96	0.98	~1.00	1.03	1.05	1.07	~1.10	1.12	1.15	1.17
5.00	0.95	0.94	0.99	~1.02	1.06	1.09	1.13	~1.17	1.21	1.25	1.29	~1.33	1.37	1.41	1.44
10.00	0.95	1.03	1.20	1.27	1.33	1.40	1.47	1.54	1.60	1.67	1.73	1.80	1.86	1.91	1.97
20.00	0.97	1.27	1.58	1.69	1.80	1.90	2.00	2.10	2.20	2.30	2.39	2.49	2.57	2.66	2.74
40.00	1.02	1.71	2.18	2.33	2.49	2.64	2.78	2.93	3.07	3.21	3.35	3.48	3.61	3.73	3.85
70.00	1.17	2.22	2.85	3.06	3.26	3.46	3.66	3.85	4.04	4.23	4.40	4.58	4.75	4.92	5.07
100.00	1.36	2.64	3.39	3.64	3.88	4.12	4.36	4.59	4.82	5.04	5.25	5.46	5.67	5.87	6.05
$P_1/P_2 = 0.80$															
1.00	0.95	0.90	0.88	~0.88	0.87	0.87	0.86	~0.86	0.86	0.86	0.86	~0.85	0.85	0.85	0.85
2.00	0.95	0.91	0.91	~0.91	0.91	0.92	0.93	~0.94	0.95	0.96	0.97	~0.98	1.00	1.01	1.03
3.00	0.95	0.92	0.94	~0.95	0.96	0.98	1.00	~1.02	1.04	1.07	1.09	~1.11	1.14	1.16	1.19
5.00	0.95	0.95	1.01	~1.04	1.07	1.11	1.15	~1.19	1.23	1.27	1.31	~1.65	1.39	1.43	1.47
10.00	0.96	1.05	1.23	1.29	1.36	1.43	1.50	1.57	1.63	1.70	1.77	1.83	1.89	1.95	2.00
20.00	0.97	1.30	1.62	1.73	1.84	1.94	2.04	2.15	2.25	2.34	2.44	2.53	2.62	2.71	2.79
40.00	1.03	1.75	2.23	2.39	2.54	2.69	2.84	2.99	3.13	3.27	3.41	3.54	3.67	3.79	3.91
70.00	1.21	2.28	2.92	3.13	3.33	3.54	3.74	3.93	4.12	4.31	4.49	4.66	4.83	5.01	5.16
100.00	1.40	2.70	3.47	3.72	3.97	4.21	4.45	4.69	4.91	5.14	5.35	5.57	5.77	5.97	6.16

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Table 7.1 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 4

Column ABC Hinged at A and Hinged at C

a_r B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.90$															
1.00	0.95	0.91	0.89	~0.88	0.88	0.88	0.87	~0.87	0.87	0.86	0.86	~0.86	0.86	0.86	0.86
2.00	0.95	0.92	0.91	~0.92	0.92	0.93	0.94	~0.95	0.96	0.97	0.98	~1.00	1.01	1.02	1.04
3.00	0.95	0.93	0.95	~0.96	0.97	0.99	1.01	~1.03	1.06	1.08	1.10	~1.13	1.15	1.18	1.20
5.00	0.95	0.96	1.02	~1.05	1.09	1.13	1.17	~1.21	1.25	1.29	1.33	~1.37	1.41	1.45	1.48
10.00	0.96	1.06	1.25	1.32	1.39	1.46	1.53	1.60	1.66	1.73	1.79	1.86	1.92	1.98	2.03
20.00	0.98	1.33	1.66	1.76	1.87	1.98	2.08	2.18	2.29	2.38	2.48	2.57	2.66	2.75	2.83
40.00	1.05	1.79	2.28	2.43	2.59	2.75	2.90	3.04	3.19	3.33	3.47	3.60	3.73	3.85	3.97
70.00	1.24	2.33	2.98	3.19	3.40	3.60	3.80	4.00	4.19	4.38	4.56	4.74	4.92	5.08	5.24
100.00	1.43	2.76	3.54	3.80	4.05	4.30	4.53	4.77	5.00	5.22	5.44	5.66	5.87	6.06	6.25
$P_1/P_2 = 1.00$															
1.00	0.95	0.91	0.89	~0.89	0.89	0.88	0.88	~0.88	0.87	0.87	0.87	~0.87	0.87	0.87	0.87
2.00	0.95	0.92	0.92	~0.92	0.93	0.94	0.95	~0.95	0.97	0.98	0.99	~1.01	1.02	1.04	1.05
3.00	0.96	0.94	0.96	~0.97	0.98	1.00	1.02	~1.04	1.07	1.09	1.12	~1.14	1.17	1.19	1.22
5.00	0.96	0.97	1.04	~1.07	1.11	1.14	1.18	~1.23	1.27	1.31	1.35	~1.39	1.43	1.47	1.50
10.00	0.96	1.08	1.27	1.34	1.41	1.48	1.55	1.62	1.69	1.75	1.82	1.88	1.94	2.00	2.06
20.00	0.99	1.35	1.68	1.79	1.90	2.01	2.11	2.22	2.32	2.42	2.51	2.61	2.70	2.78	2.87
40.00	1.06	1.83	2.32	2.48	2.63	2.79	2.94	3.09	3.34	3.38	3.52	3.65	3.78	3.91	4.03
70.00	1.26	2.38	3.03	3.25	3.45	3.66	3.87	4.06	4.26	4.45	4.63	4.81	4.98	5.15	5.31
100.00	1.47	2.82	3.61	3.86	4.11	4.36	4.61	4.84	5.07	5.31	5.52	5.73	5.95	6.15	6.34
$P_1/P_2 = 2.00$															
1.00	0.97	0.94	0.93	~0.93	0.92	0.92	0.92	~0.92	0.92	0.92	0.92	~0.91	0.91	0.91	0.91
2.00	0.97	0.96	0.97	~0.97	0.98	0.99	1.00	~1.01	1.03	1.04	1.06	~1.07	1.09	1.10	1.12
3.00	0.97	0.98	1.01	~1.03	1.05	1.07	1.10	~1.12	1.14	1.17	1.20	~1.22	1.25	1.28	1.30
5.00	0.98	1.02	1.11	~1.15	1.19	1.24	1.28	~1.33	1.37	1.41	1.45	~1.50	1.54	1.58	1.61
10.00	0.99	1.17	1.39	1.46	1.54	1.61	1.69	1.76	1.83	1.90	1.97	2.03	2.10	2.16	2.22
20.00	1.02	1.50	1.86	1.97	2.09	2.20	2.31	2.42	2.53	2.63	2.73	2.82	2.92	3.00	3.09
40.00	1.15	2.03	2.56	2.73	2.89	3.06	3.22	3.37	3.53	3.67	3.82	3.96	4.09	4.22	4.34
70.00	1.42	2.65	3.35	3.58	3.80	4.02	4.23	4.44	4.64	4.84	5.03	5.22	5.40	5.57	5.73
100.00	1.66	3.14	3.99	4.26	4.53	4.79	5.05	5.29	5.54	5.77	6.00	6.23	6.44	6.64	6.84

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Table 7.2 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns

Column ABC Rotation Restrained but Permitted to Sway at Top A and Fixed at Base C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.00$															
1.00	1.00	0.98	0.96	~0.95	0.93	0.92	0.91	~0.90	0.88	0.87	0.85	~0.83	0.81	0.79	0.77
2.00	1.08	1.10	1.08	~1.07	1.06	1.04	1.03	~1.01	0.99	0.97	0.95	~0.93	0.91	0.88	0.86
3.00	1.15	1.18	1.16	~1.14	1.13	1.11	1.09	~1.07	1.05	1.03	1.00	~0.98	0.95	0.92	0.89
5.00	1.26	1.28	1.25	~1.23	1.21	1.19	1.16	~1.14	1.11	1.08	1.05	~1.02	0.99	0.96	0.93
10.00	1.42	1.40	1.34	1.32	1.29	1.26	1.23	1.20	1.17	1.13	1.10	1.07	1.03	1.00	0.96
20.00	1.56	1.49	1.40	1.37	1.34	1.31	1.27	1.24	1.20	1.17	1.13	1.09	1.06	1.02	0.98
40.00	1.66	1.54	1.44	1.40	1.37	1.33	1.30	1.26	1.22	1.18	1.14	1.11	1.07	1.03	0.99
70.00	1.72	1.56	1.46	1.42	1.38	1.34	1.31	1.27	1.23	1.19	1.15	1.11	1.07	1.03	0.99
100.00	1.74	1.57	1.46	1.43	1.39	1.35	1.31	1.27	1.23	1.19	1.15	1.11	1.07	1.04	1.00
$P_1/P_2 = 0.10$															
1.00	1.00	0.98	0.96	~0.95	0.94	0.93	0.92	~0.90	0.89	0.88	0.86	~0.85	0.83	0.81	0.79
2.00	1.08	1.10	1.09	~1.08	1.07	1.05	1.04	~1.02	1.01	0.99	0.97	~0.95	0.92	0.90	0.88
3.00	1.15	1.19	1.16	~1.15	1.14	1.12	1.11	~1.09	1.07	1.04	1.02	~1.00	0.97	0.95	0.92
5.00	1.26	1.29	1.26	~1.24	1.22	1.20	1.18	~1.15	1.13	1.10	1.08	~1.05	1.02	0.99	0.96
10.00	1.42	1.41	1.35	1.33	1.30	1.28	1.25	1.22	1.19	1.16	1.13	1.09	1.06	1.03	1.00
20.00	1.56	1.50	1.42	1.39	1.36	1.33	1.29	1.26	1.23	1.19	1.16	1.12	1.09	1.06	1.02
40.00	1.67	1.55	1.46	1.42	1.39	1.35	1.32	1.28	1.25	1.22	1.18	1.15	1.12	1.10	1.08
70.00	1.72	1.58	1.47	1.44	1.40	1.37	1.33	1.30	1.27	1.24	1.22	1.22	1.23	1.26	1.30
100.00	1.74	1.59	1.48	1.45	1.41	1.38	1.35	1.32	1.30	1.30	1.33	1.37	1.42	1.48	1.53
$P_1/P_2 = 0.20$															
1.00	1.00	0.98	0.96	~0.95	0.94	0.93	0.92	~0.91	0.90	0.89	0.87	~0.86	0.84	0.82	0.81
2.00	1.08	1.11	1.09	~1.08	1.07	1.06	1.05	~1.03	1.02	1.00	0.98	~0.96	0.94	0.92	0.90
3.00	1.15	1.19	1.17	~1.16	1.15	1.13	1.12	~1.10	1.08	1.06	1.04	~1.01	0.99	0.97	0.94
5.00	1.26	1.30	1.27	~1.25	1.23	1.21	1.19	~1.17	1.15	1.12	1.10	~1.07	1.04	1.01	0.99
10.00	1.42	1.42	1.37	1.34	1.32	1.29	1.27	1.24	1.21	1.18	1.15	1.12	1.09	1.06	1.03
20.00	1.57	1.51	1.43	1.40	1.37	1.34	1.31	1.28	1.25	1.22	1.19	1.17	1.14	1.12	1.10
40.00	1.67	1.56	1.47	1.44	1.41	1.38	1.35	1.32	1.30	1.28	1.27	1.27	1.29	1.32	1.35
70.00	1.73	1.59	1.49	1.46	1.44	1.41	1.39	1.39	1.41	1.45	1.50	1.55	1.61	1.68	1.74
100.00	1.75	1.60	1.51	1.48	1.47	1.47	1.49	1.54	1.61	1.68	1.75	1.83	1.91	1.99	2.07

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Table 7.2 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 2

Column ABC Rotation Restrained but Permitted to Sway at Top A and Fixed at Base C

a_r B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.30$															
1.00	1.00	0.98	0.96	~0.96	0.95	0.94	0.93	~0.92	0.91	0.89	0.88	~0.87	0.85	0.83	0.82
2.00	1.08	1.11	1.10	~1.09	1.08	1.07	1.06	~1.04	1.03	1.01	0.99	~0.98	0.96	0.94	0.92
3.00	1.15	1.19	1.18	~1.17	1.16	1.14	1.13	~1.11	1.09	1.07	1.05	~1.03	1.01	0.99	0.96
5.00	1.26	1.30	1.27	~1.26	1.24	1.22	1.20	~1.18	1.16	1.14	1.11	~1.09	1.06	1.04	1.01
10.00	1.43	1.43	1.38	1.35	1.33	1.31	1.28	1.26	1.23	1.20	1.18	1.15	1.12	1.10	1.08
20.00	1.57	1.52	1.44	1.42	1.39	1.36	1.33	1.31	1.28	1.26	1.23	1.22	1.21	1.21	1.21
40.00	1.67	1.57	1.49	1.46	1.43	1.41	1.39	1.37	1.36	1.37	1.40	1.43	1.48	1.53	1.58
70.00	1.73	1.60	1.52	1.50	1.48	1.48	1.50	1.55	1.61	1.67	1.74	1.82	1.89	1.97	2.05
100.00	1.75	1.62	1.55	1.55	1.57	1.63	1.70	1.79	1.87	1.96	2.06	2.15	2.24	2.34	2.43
$P_1/P_2 = 0.40$															
1.00	1.00	0.98	0.97	~0.96	0.95	0.94	0.93	~0.92	0.91	0.9	0.89	~0.87	0.86	0.85	0.83
2.00	1.08	1.11	0.10	~1.09	1.09	1.07	1.06	~1.05	1.04	1.02	1.01	~0.99	0.97	0.95	0.93
3.00	1.16	1.20	1.18	~1.17	1.16	1.15	1.14	~1.12	1.10	1.09	1.07	~1.05	1.03	1.00	0.98
5.00	1.27	1.31	1.28	~1.27	1.25	1.23	1.21	~1.19	1.17	1.15	1.13	~1.11	1.08	1.06	1.04
10.00	1.43	1.43	1.38	1.36	1.34	1.32	1.30	1.27	1.25	1.22	1.20	1.18	1.15	1.14	1.12
20.00	1.57	1.52	1.45	1.43	1.40	1.38	1.35	1.33	1.31	1.29	1.28	1.28	1.28	1.29	1.32
40.00	1.68	1.58	1.50	1.48	1.46	1.44	1.43	1.43	1.45	1.48	1.52	1.57	1.63	1.69	1.75
70.00	1.73	1.61	1.54	1.54	1.54	1.57	1.63	1.69	1.77	1.85	1.93	2.02	2.10	2.19	2.27
100.00	1.76	1.63	1.60	1.64	1.70	1.79	1.88	1.98	2.08	2.18	2.28	2.39	2.49	2.60	2.70
$P_1/P_2 = 0.50$															
1.00	1.00	0.98	0.97	~0.96	0.95	0.95	0.94	~0.93	0.92	0.91	0.89	~0.88	0.87	0.85	0.84
2.00	1.08	1.11	1.10	~1.10	1.09	1.08	1.07	~1.06	1.04	1.03	1.02	~1.00	0.98	0.96	0.95
3.00	1.16	1.20	1.19	~1.18	1.17	1.16	1.14	~1.13	1.11	1.10	1.08	~1.06	1.04	1.02	1.00
5.00	1.27	1.31	1.29	~1.27	1.26	1.24	1.22	~1.21	1.19	1.17	1.14	~1.12	1.10	1.08	1.06
10.00	1.43	1.44	1.39	1.37	1.35	1.33	1.31	1.29	1.26	1.24	1.22	1.20	1.19	1.17	1.16
20.00	1.58	1.53	1.46	1.44	1.42	1.39	1.37	1.35	1.34	1.33	1.33	1.33	1.35	1.38	1.41
40.00	1.68	1.59	1.52	1.50	1.48	1.47	1.48	1.49	1.53	1.58	1.63	1.69	1.75	1.82	1.89
70.00	1.74	1.62	1.57	1.58	1.61	1.67	1.74	1.82	1.90	1.99	2.08	2.18	2.27	2.36	2.46
100.00	1.76	1.64	1.67	1.73	1.82	1.92	2.02	2.13	2.24	2.35	2.47	2.58	2.69	2.81	2.92

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Table 7.2 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 3

Column ABC Rotation Restrained but Permitted to Sway at Top A and Fixed at Base C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.60$															
1.00	1.00	0.99	0.97	~0.96	0.96	0.95	0.94	~0.93	0.92	0.91	0.9	~0.89	0.88	0.86	0.85
2.00	1.08	1.11	1.11	~1.10	1.09	1.09	1.08	~1.06	1.05	1.04	1.02	~1.01	0.99	0.98	0.96
3.00	1.16	1.20	1.19	~1.18	1.18	1.16	1.15	~1.14	1.12	1.11	1.09	~1.07	1.05	1.03	1.02
5.00	1.27	1.31	1.29	~1.28	1.27	1.25	1.23	~1.22	1.20	1.18	1.16	~1.14	1.12	1.10	1.08
10.00	1.43	1.44	1.40	1.38	1.36	1.34	1.32	1.30	1.28	1.26	1.24	1.23	1.21	1.21	1.20
20.00	1.58	1.54	1.47	1.45	1.43	1.41	1.39	1.38	1.37	1.36	1.37	1.39	1.41	1.44	1.48
40.00	1.68	1.60	1.53	1.52	1.51	1.51	1.52	1.56	1.60	1.66	1.72	1.79	1.86	1.93	2.00
70.00	1.74	1.63	1.61	1.63	1.68	1.75	1.83	1.92	2.01	2.11	2.21	2.31	2.40	2.50	2.60
100.00	1.76	1.66	1.73	1.82	1.92	2.03	2.14	2.26	2.38	2.5	2.61	2.73	2.85	2.97	3.10
$P_1/P_2 = 0.70$															
1.00	1.00	0.99	0.97	~0.97	0.96	0.95	0.94	~0.94	0.93	0.92	0.91	~0.89	0.88	0.87	0.86
2.00	1.08	1.12	1.11	~1.11	1.10	1.09	1.08	~1.07	1.06	1.05	1.03	~1.02	1.00	0.99	0.97
3.00	1.16	1.20	1.20	~1.19	1.18	1.17	1.16	~1.14	1.13	1.11	1.10	~1.08	1.06	1.05	1.03
5.00	1.27	1.32	1.30	~1.29	1.27	1.26	1.24	~1.22	1.21	1.19	1.17	~1.15	1.13	1.12	1.10
10.00	1.43	1.45	1.41	1.39	1.37	1.35	1.33	1.31	1.29	1.28	1.26	1.25	1.24	1.24	1.24
20.00	1.58	1.54	1.48	1.46	1.44	1.42	1.41	1.40	1.39	1.40	1.41	1.44	1.47	1.50	1.55
40.00	1.69	1.60	1.54	1.53	1.53	1.54	1.57	1.61	1.67	1.73	1.8	1.87	1.94	2.02	2.09
70.00	1.74	1.64	1.64	1.68	1.74	1.82	1.91	2.01	2.11	2.21	2.31	2.41	2.52	2.62	2.73
100.00	1.77	1.68	1.80	1.89	2.00	2.12	2.24	2.36	2.49	2.61	2.74	2.86	2.99	3.12	3.24
$P_1/P_2 = 0.80$															
1.00	1.00	0.99	0.97	~0.97	0.96	0.95	0.95	~0.94	0.93	0.92	0.91	~0.90	0.89	0.88	0.86
2.00	1.09	1.12	1.11	~1.11	1.10	1.09	1.09	~1.08	1.06	1.05	1.04	~1.03	1.01	1.00	0.98
3.00	1.16	1.21	1.20	~1.19	1.18	1.17	1.16	~1.15	1.14	1.12	1.11	~1.09	1.08	1.06	1.04
5.00	1.27	1.32	1.30	~1.29	1.28	1.26	1.25	~1.23	1.22	1.20	1.18	~1.16	1.15	1.13	1.12
10.00	1.43	1.45	1.41	1.39	1.38	1.36	1.34	1.32	1.31	1.29	1.28	1.27	1.27	1.27	1.27
20.00	1.58	1.55	1.49	1.47	1.45	1.44	1.43	1.42	1.42	1.43	1.45	1.48	1.52	1.56	1.60
40.00	1.69	1.61	1.56	1.55	1.56	1.58	1.61	1.66	1.72	1.79	1.86	1.94	2.01	2.09	2.17
70.00	1.74	1.65	1.67	1.72	1.80	1.89	1.98	2.08	2.19	2.29	2.40	2.51	2.62	2.72	2.83
100.00	1.77	1.69	1.85	1.96	2.08	2.20	2.33	2.46	2.58	2.71	2.84	2.98	3.11	3.24	3.37

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Table 7.2 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 4

Column ABC Rotation Restrained but Permitted to Sway at Top A and Fixed at Base C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.90$															
1.00	1.00	0.99	0.97	~0.97	0.96	0.96	0.95	~0.94	0.93	0.92	0.91	~0.90	0.89	0.88	0.87
2.00	1.09	1.12	1.12	~1.11	1.11	1.10	1.09	~1.08	1.07	1.06	1.05	~1.03	1.02	1.01	0.99
3.00	1.16	1.21	1.20	~1.20	1.19	1.18	1.17	~1.16	1.14	1.13	1.11	~1.10	1.08	1.07	1.06
5.00	1.27	1.32	1.30	~1.29	1.28	1.27	1.25	~1.24	1.22	1.21	1.19	~1.18	1.16	1.15	1.14
10.00	1.44	1.45	1.42	1.4	1.38	1.37	1.35	1.33	1.32	1.3	1.29	1.29	1.29	1.29	1.3
20.00	1.58	1.55	1.50	1.48	1.46	1.45	1.44	1.44	1.44	1.46	1.48	1.52	1.56	1.6	1.65
40.00	1.69	1.61	1.57	1.57	1.58	1.61	1.65	1.71	1.77	1.84	1.92	2.00	2.08	2.16	2.24
70.00	1.75	1.66	1.70	1.76	1.85	1.94	2.04	2.15	2.26	2.37	2.48	2.59	2.7	2.81	2.92
100.00	1.77	1.71	1.90	2.02	2.14	2.27	2.4	2.53	2.67	2.8	2.94	3.07	3.21	3.34	3.48
$P_1/P_2 = 1.00$															
1.00	1.00	0.99	0.98	~0.97	0.97	0.96	0.95	~0.94	0.94	0.93	0.92	~0.91	0.9	0.89	0.88
2.00	1.09	1.12	1.12	~1.11	1.11	1.1	1.09	~1.08	1.07	1.06	1.05	~1.04	1.03	1.01	1.00
3.00	1.16	1.21	1.21	~1.20	1.19	1.18	1.17	~1.16	1.15	1.14	1.12	~1.11	1.09	1.08	1.07
5.00	1.27	1.32	1.31	~1.30	1.29	1.27	1.26	~1.25	1.23	1.22	1.20	~1.19	1.17	1.16	1.15
10.00	1.44	1.46	1.42	1.41	1.39	1.37	1.36	1.34	1.33	1.32	1.31	1.31	1.31	1.32	1.33
20.00	1.58	1.55	1.50	1.49	1.47	1.46	1.46	1.46	1.47	1.49	1.52	1.55	1.6	1.64	1.69
40.00	1.69	1.62	1.58	1.58	1.6	1.64	1.68	1.75	1.82	1.89	1.97	2.05	2.13	2.22	2.3
70.00	1.75	1.67	1.73	1.8	1.89	1.99	2.1	2.21	2.32	2.43	2.54	2.66	2.77	2.89	3.00
100.00	1.77	1.72	1.95	2.07	2.2	2.33	2.47	2.6	2.74	2.88	3.02	3.16	3.29	3.43	3.57
$P_1/P_2 = 2.00$															
1.00	1.00	0.99	0.98	~0.98	0.98	0.97	0.97	~0.96	0.96	0.95	0.94	~0.94	0.93	0.92	0.92
2.00	1.09	1.13	1.13	~1.13	1.13	1.12	1.12	~1.11	1.11	1.1	1.09	~1.08	1.07	1.07	1.06
3.00	1.16	1.22	1.22	~1.22	1.22	1.21	1.2	~1.20	1.19	1.18	1.17	~1.16	1.15	1.14	1.14
5.00	1.27	1.34	1.33	~1.33	1.32	1.31	1.3	~1.29	1.28	1.27	1.26	~1.26	1.25	1.25	1.25
10.00	1.44	1.48	1.45	1.44	1.43	1.42	1.41	1.41	1.4	1.41	1.41	1.42	1.44	1.47	1.49
20.00	1.59	1.58	1.55	1.54	1.54	1.54	1.56	1.58	1.62	1.66	1.71	1.76	1.82	1.88	1.94
40.00	1.70	1.66	1.67	1.71	1.76	1.82	1.9	1.99	2.08	2.17	2.26	2.63	2.46	2.56	2.66
70.00	1.76	1.74	1.93	2.04	2.16	2.28	2.41	2.54	2.67	2.8	2.93	3.07	3.20	3.33	3.47
100.00	1.79	1.85	2.22	2.37	2.53	2.68	2.84	3	3.16	3.32	3.48	3.64	3.80	3.96	4.13

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Table 7.3 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns

Column ABC Hinged at A and Fixed at C

a_r / B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.00$															
1.00	0.61	0.53	0.50	~0.49	0.48	0.47	0.47	~0.46	0.46	0.46	0.46	~0.46	0.46	0.46	0.46
2.00	0.61	0.53	0.50	~0.49	0.49	0.49	0.49	~0.49	0.50	0.51	0.52	~0.52	0.53	0.53	0.54
3.00	0.61	0.53	0.50	~0.50	0.50	0.51	0.52	~0.54	0.55	0.56	0.57	~0.58	0.59	0.59	0.60
5.00	0.61	0.53	0.52	~0.53	0.55	0.58	0.60	~0.62	0.64	0.65	0.66	~0.67	0.68	0.68	0.68
10.00	0.61	0.64	0.62	0.66	0.69	0.72	0.74	0.76	0.78	0.79	0.80	0.80	0.80	0.80	0.79
20.00	0.61	0.63	0.78	0.82	0.86	0.89	0.91	0.92	0.93	0.93	0.92	0.92	0.91	0.89	0.87
40.00	0.61	0.81	0.97	1.00	1.03	1.04	1.05	1.05	1.05	1.04	1.00	1.00	0.98	0.96	0.93
70.00	0.61	0.97	1.11	1.13	1.15	1.15	1.14	1.13	1.12	1.10	1.07	1.05	1.02	0.99	0.96
100.00	0.62	1.08	1.19	1.20	1.21	1.20	1.19	1.17	1.15	1.12	1.10	1.07	1.04	1.00	0.97
$P_1/P_2 = 0.10$															
1.00	0.61	0.55	0.52	~0.51	0.50	0.49	0.49	~0.49	0.48	0.48	0.48	~0.48	0.48	0.48	0.48
2.00	0.61	0.55	0.52	~0.52	0.52	0.52	0.52	~0.53	0.54	0.55	0.55	~0.56	0.57	0.57	0.57
3.00	0.61	0.55	0.53	~0.53	0.54	0.56	0.57	~0.58	0.60	0.61	0.62	~0.63	0.63	0.64	0.64
5.00	0.61	0.55	0.57	~0.59	0.62	0.64	0.66	~0.68	0.70	0.71	0.72	~0.73	0.73	0.73	0.73
10.00	0.61	0.58	0.70	0.74	0.77	0.80	0.83	0.84	0.86	0.87	0.87	0.87	0.87	0.86	0.85
20.00	0.61	0.74	0.90	0.93	0.97	0.99	1.01	1.02	1.02	1.02	1.01	1.00	0.99	0.97	0.95
40.00	0.61	0.96	1.11	1.14	1.16	1.17	1.17	1.17	1.16	1.14	1.12	1.10	1.08	1.05	1.02
70.00	0.65	1.16	1.28	1.29	1.29	1.29	1.28	1.26	1.24	1.22	1.19	1.16	1.16	1.10	1.08
100.00	0.75	1.28	1.37	1.37	1.37	1.35	1.33	1.31	1.28	1.25	1.23	1.20	1.18	1.16	1.16
$P_1/P_2 = 0.20$															
1.00	0.62	0.56	0.53	~0.52	0.52	0.51	0.51	~0.50	0.50	0.50	0.50	~0.50	0.50	0.50	0.50
2.00	0.62	0.56	0.54	~0.54	0.54	0.54	0.55	~0.56	0.57	0.58	0.58	~0.59	0.59	0.60	0.60
3.00	0.62	0.56	0.56	~0.56	0.58	0.59	0.61	~0.62	0.63	0.65	0.66	~0.66	0.67	0.67	0.67
5.00	0.62	0.57	0.61	~0.64	0.66	0.69	0.71	~0.73	0.74	0.76	0.77	~0.77	0.78	0.78	0.77
10.00	0.62	0.64	0.77	0.81	0.84	0.87	0.89	0.91	0.92	0.93	0.93	0.93	0.93	0.92	0.91
20.00	0.62	0.82	0.98	1.02	1.05	1.07	1.09	1.10	1.10	1.09	1.09	1.07	1.06	1.04	1.02
40.00	0.63	1.07	1.22	1.25	1.26	1.27	1.27	1.26	1.25	1.23	1.21	1.19	1.17	1.15	1.13
70.00	0.76	1.29	1.40	1.41	1.41	1.41	1.39	1.37	1.35	1.33	1.31	1.29	1.28	1.28	1.30
100.00	0.89	1.43	1.51	1.51	1.50	1.48	1.46	1.44	1.42	1.40	1.39	1.42	1.42	1.45	1.49

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Table 7.3 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 2

Column ABC Hinged at A and Fixed at C

a_r / B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.30$															
1.00	0.62	0.57	0.54	~0.53	0.53	0.53	0.52	~0.52	0.52	0.52	0.52	~0.52	0.52	0.52	0.52
2.00	0.62	0.57	0.56	~0.56	0.56	0.57	0.58	~0.58	0.59	0.60	0.61	~0.61	0.62	0.62	0.62
3.00	0.62	0.58	0.58	~0.59	0.61	0.62	0.64	~0.65	0.66	0.68	0.69	~0.69	0.70	0.70	0.70
5.00	0.63	0.59	0.65	~0.68	0.70	0.73	0.75	~0.77	0.78	0.79	0.80	~0.81	0.81	0.81	0.81
10.00	0.63	0.69	0.82	0.86	0.89	0.92	0.94	0.96	0.97	0.98	0.98	0.98	0.97	0.97	0.95
20.00	0.63	0.89	1.05	1.09	1.12	1.14	1.15	1.16	1.16	1.16	1.15	1.14	1.12	1.10	1.08
40.00	0.68	1.16	1.31	1.33	1.35	1.35	1.35	1.34	1.33	1.31	1.29	1.28	1.26	1.25	1.24
70.00	0.84	1.40	1.51	1.51	1.51	1.50	1.49	1.47	1.45	1.44	1.43	1.43	1.44	1.46	1.49
100.00	0.99	1.55	1.62	1.62	1.60	1.59	1.57	1.55	1.55	1.55	1.56	1.59	1.64	1.69	1.74
$P_1/P_2 = 0.40$															
1.00	0.63	0.58	0.55	~0.55	0.54	0.54	0.54	~0.53	0.53	0.53	0.53	~0.53	0.53	0.53	0.53
2.00	0.63	0.58	0.57	~0.57	0.58	0.59	0.60	~0.60	0.61	0.62	0.63	~0.63	0.64	0.64	0.64
3.00	0.63	0.59	0.60	~0.61	0.63	0.65	0.66	~0.68	0.69	0.70	0.71	~0.72	0.72	0.73	0.73
5.00	0.63	0.61	0.68	~0.71	0.73	0.76	0.78	~0.80	0.81	0.83	0.84	~0.84	0.85	0.85	0.84
10.00	0.63	0.73	0.86	0.90	0.93	0.96	0.98	1.00	1.01	1.02	1.02	1.02	1.01	1.01	1.00
20.00	0.64	0.94	1.11	1.14	1.17	1.19	1.21	1.21	1.21	1.21	1.20	1.19	1.17	1.16	1.14
40.00	0.72	1.23	1.38	1.40	1.41	1.42	1.42	1.41	1.39	1.38	1.36	1.35	1.34	1.34	1.34
70.00	0.91	1.48	1.59	1.59	1.59	1.58	1.57	1.55	1.54	1.53	1.53	1.55	1.57	1.61	1.65
100.00	1.07	1.65	1.71	1.71	1.69	1.68	1.66	1.66	1.66	1.68	1.71	1.75	1.81	1.87	1.93
$P_1/P_2 = 0.50$															
1.00	0.63	0.58	0.56	~0.56	0.55	0.55	0.55	~0.55	0.54	0.54	0.54	~0.54	0.54	0.54	0.54
2.00	0.63	0.59	0.59	~0.59	0.60	0.60	0.61	~0.62	0.63	0.64	0.65	~0.65	0.66	0.66	0.66
3.00	0.63	0.60	0.62	~0.63	0.65	0.67	0.68	~0.70	0.71	0.72	0.73	~0.74	0.75	0.75	0.75
5.00	0.64	0.63	0.71	~0.73	0.76	0.78	0.81	~0.82	0.84	0.85	0.86	~0.87	0.87	0.87	0.87
10.00	0.64	0.76	0.90	0.94	0.97	1.00	1.02	1.03	1.05	1.05	1.06	1.05	1.05	1.04	1.03
20.00	0.65	0.99	1.15	1.19	1.22	1.24	1.25	1.26	1.26	1.25	1.24	1.23	1.22	1.21	1.19
40.00	0.76	1.28	1.44	1.46	1.47	1.48	1.47	1.46	1.45	1.44	1.42	1.41	1.41	1.42	1.43
70.00	0.97	1.55	1.66	1.66	1.66	1.65	1.64	1.62	1.61	1.61	1.62	1.65	1.68	1.72	1.77
100.00	1.13	1.73	1.79	1.79	1.77	1.76	1.75	1.75	1.76	1.79	1.83	1.88	1.95	2.01	2.08

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Table 7.3 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 3

Column ABC Hinged at A and Fixed at C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.60$															
1.00	0.64	0.59	0.57	~0.57	0.56	0.56	0.56	~0.56	0.56	0.55	0.55	~0.55	0.55	0.55	0.55
2.00	0.64	0.60	0.60	~0.60	0.61	0.62	0.63	~0.64	0.65	0.65	0.66	~0.67	0.67	0.68	0.68
3.00	0.64	0.61	0.64	~0.65	0.67	0.68	0.70	~0.72	0.73	0.74	0.75	~0.76	0.76	0.77	0.77
5.00	0.64	0.65	0.73	~0.76	0.78	0.81	0.83	~0.85	0.86	0.88	0.89	~0.89	0.89	0.89	0.89
10.00	0.64	0.79	0.93	0.97	1.00	1.03	1.05	1.06	1.08	1.08	1.09	1.08	1.08	1.07	1.06
20.00	0.67	1.02	1.19	1.23	1.26	1.28	1.29	1.30	1.30	1.29	1.28	1.27	1.26	1.25	1.24
40.00	0.80	1.33	1.48	1.51	1.52	1.52	1.52	1.51	1.50	1.49	1.48	1.47	1.47	1.48	1.50
70.00	1.01	1.61	1.72	1.72	1.72	1.71	1.69	1.68	1.68	1.69	1.70	1.73	1.77	1.82	1.88
100.00	1.18	1.79	1.85	1.85	1.83	1.82	1.82	1.82	1.85	1.88	1.93	1.99	2.06	2.13	2.21
$P_1/P_2 = 0.70$															
1.00	0.64	0.60	0.58	~0.57	0.57	0.57	0.57	~0.56	0.56	0.56	0.56	~0.56	0.56	0.56	0.56
2.00	0.64	0.61	0.61	~0.61	0.62	0.63	0.64	~0.65	0.66	0.67	0.67	~0.68	0.69	0.69	0.69
3.00	0.64	0.62	0.65	~0.67	0.68	0.70	0.72	~0.73	0.75	0.76	0.77	~0.77	0.78	0.78	0.78
5.00	0.64	0.66	0.75	~0.78	0.80	0.83	0.85	~0.87	0.88	0.90	0.90	~0.91	0.91	0.91	0.91
10.00	0.65	0.81	0.95	0.99	1.02	1.05	1.07	1.09	1.10	1.11	1.11	1.11	1.11	1.10	1.09
20.00	0.68	1.06	1.22	1.26	1.29	1.31	1.32	1.33	1.33	1.32	1.32	1.31	1.29	1.28	1.28
40.00	0.83	1.38	1.53	1.55	1.56	1.57	1.56	1.55	1.54	1.53	1.52	1.52	1.53	1.54	1.57
70.00	1.05	1.66	1.77	1.77	1.77	1.76	1.75	1.74	1.74	1.75	1.77	1.81	1.85	1.91	1.96
100.00	1.23	1.85	1.91	1.90	1.89	1.88	1.88	1.89	1.92	1.96	2.02	2.08	2.15	2.23	2.31
$P_1/P_2 = 0.80$															
1.00	0.61	0.60	0.58	~0.58	0.58	0.58	0.57	~0.57	0.57	0.57	0.57	~0.57	0.57	0.57	0.57
2.00	0.64	0.61	0.62	~0.62	0.63	0.64	0.65	~0.66	0.67	0.68	0.69	~0.69	0.70	0.70	0.70
3.00	0.65	0.63	0.66	~.068	0.70	0.71	0.73	~0.75	0.76	0.77	0.78	~0.79	0.79	0.80	0.80
5.00	0.65	0.68	0.76	~0.79	0.82	0.84	0.87	~0.88	0.90	0.91	0.92	~0.93	0.93	0.93	0.93
10.00	0.66	0.83	0.97	1.01	1.05	1.07	1.10	1.11	1.12	1.13	1.13	1.13	1.13	1.12	1.11
20.00	0.69	1.08	1.25	1.29	1.32	1.34	1.35	1.36	1.36	1.35	1.34	1.34	1.33	1.32	1.31
40.00	0.85	1.41	1.56	1.59	1.60	1.60	1.60	1.59	1.58	1.57	1.57	1.57	1.57	1.59	1.62
70.00	1.08	1.71	1.81	1.81	1.81	1.80	1.79	1.79	1.79	1.80	1.83	1.87	1.92	1.98	2.04
100.00	1.27	1.90	1.96	1.95	1.94	1.93	1.93	1.95	1.98	2.03	2.09	2.16	2.24	2.32	2.40

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Table 7.3 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 4

Column ABC Hinged at A and Fixed at C

a_r / B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.90$															
1.00	0.65	0.61	0.59	~0.59	0.58	0.58	0.58	~0.58	0.58	0.58	0.58	~0.58	0.58	0.58	0.58
2.00	0.65	0.62	0.63	~0.63	0.64	0.65	0.66	~0.67	0.68	0.69	0.70	~0.70	0.71	0.71	0.71
3.00	0.65	0.64	0.67	~0.69	0.71	0.73	0.74	~0.76	0.77	0.78	0.79	~0.80	0.81	0.81	0.81
5.00	0.65	0.69	0.78	~0.81	0.83	0.86	0.88	~0.90	0.92	0.93	0.94	~0.94	0.95	0.95	0.95
10.00	0.66	0.85	0.99	1.03	1.07	1.09	1.12	1.13	1.14	1.15	1.15	1.15	1.15	1.14	1.13
20.00	0.70	1.11	1.28	1.32	1.34	1.36	1.38	1.38	1.38	1.38	1.37	1.36	1.35	1.35	1.34
40.00	0.88	1.44	1.60	1.62	1.63	1.63	1.63	1.62	1.61	1.61	1.60	1.60	1.62	1.64	1.67
70.00	1.11	1.75	1.85	1.85	1.85	1.84	1.83	1.83	1.83	1.85	1.88	1.93	1.98	2.04	2.10
100.00	1.30	1.94	2.00	1.99	1.98	1.98	1.98	2.00	2.04	2.09	2.16	2.23	2.31	2.39	2.47
$P_1/P_2 = 1.00$															
1.00	0.65	0.61	0.60	~0.59	0.59	0.59	0.59	~0.59	0.59	0.59	0.59	~0.59	0.59	0.59	0.58
2.00	0.65	0.63	0.63	~0.64	0.65	0.66	0.67	~0.68	0.69	0.70	0.71	~0.71	0.72	0.72	0.72
3.00	0.65	0.64	0.68	~0.70	0.72	0.74	0.75	~0.77	0.78	0.79	0.80	~0.81	0.82	0.82	0.82
5.00	0.65	0.70	0.79	~0.82	0.85	0.87	0.89	~0.91	0.93	0.94	0.95	~0.96	0.96	0.96	0.96
10.00	0.66	0.86	1.01	1.05	1.08	1.11	1.13	1.15	1.16	1.17	1.17	1.17	1.17	1.16	1.15
20.00	0.71	1.16	1.30	1.34	1.37	1.39	1.40	1.40	1.40	1.40	1.39	1.39	1.38	1.37	1.37
40.00	0.90	1.47	1.62	1.65	1.66	1.66	1.66	1.65	1.64	1.64	1.63	1.64	1.65	1.68	1.71
70.00	1.14	1.78	1.88	1.88	1.88	1.87	1.87	1.86	1.87	1.89	1.93	1.97	2.03	2.09	2.16
100.00	1.33	1.98	2.03	2.03	2.02	2.02	2.03	2.05	2.09	2.14	2.21	2.29	2.37	2.45	2.54
$P_1/P_2 = 2.00$															
1.00	0.67	0.64	0.63	~0.63	0.63	0.63	0.63	~0.63	0.63	0.63	0.63	~0.63	0.63	0.62	0.62
2.00	0.67	0.66	0.68	~0.69	0.70	0.71	0.72	~0.73	0.74	0.75	0.76	~0.77	0.77	0.78	0.78
3.00	0.67	0.69	0.74	~0.76	0.78	0.80	0.82	~0.83	0.85	0.86	0.87	~0.88	0.88	0.89	0.89
5.00	0.68	0.77	0.87	~0.90	0.93	0.95	0.98	~1.00	1.01	1.02	1.03	~1.04	1.04	1.05	1.05
10.00	0.70	0.96	1.11	1.15	1.19	1.22	1.24	1.26	1.27	1.28	1.28	1.28	1.28	1.27	1.27
20.00	0.78	1.25	1.44	1.47	1.50	1.52	1.54	1.54	1.54	1.54	1.53	1.53	1.53	1.53	1.53
40.00	1.01	1.64	1.79	1.82	1.83	1.83	1.83	1.82	1.82	1.82	1.83	1.85	1.88	1.91	1.96
70.00	1.29	1.98	2.08	2.09	2.08	2.08	2.08	2.09	2.11	2.15	2.20	2.26	2.33	2.41	2.49
100.00	1.50	2.20	2.26	2.25	2.25	2.26	2.28	2.32	2.38	2.46	2.54	2.63	2.73	2.83	2.93

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Table 7.4 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns

Column ABC Fixed at A and Fixed at C

a_r B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.00$															
1.00	0.49	0.47	0.45	~0.44	0.43	0.42	0.41	~0.40	0.40	0.39	0.38	~0.37	0.37	0.37	0.36
2.00	0.52	0.49	0.47	~0.46	0.45	0.44	0.43	~0.42	0.41	0.40	0.40	~0.40	0.40	0.41	0.42
3.00	0.54	0.51	0.47	~0.46	0.45	0.44	0.43	~0.42	0.42	0.42	0.43	~0.44	0.45	0.46	0.47
5.00	0.56	0.52	0.48	~0.47	0.46	0.45	0.44	~0.44	0.46	0.48	0.49	~0.41	0.52	0.54	0.54
10.00	0.58	0.53	0.49	0.47	0.47	0.50	0.53	0.56	0.58	0.60	0.62	0.64	0.65	0.66	0.67
20.00	0.60	0.53	0.53	0.57	0.61	0.65	0.68	0.71	0.73	0.75	0.77	0.78	0.78	0.78	0.78
40.00	0.61	0.55	0.69	0.74	0.78	0.82	0.85	0.87	0.89	0.90	0.90	0.90	0.89	0.88	0.87
70.00	0.61	0.66	0.84	0.89	0.93	0.96	0.98	0.99	1.00	1.00	0.99	0.98	0.96	0.94	0.92
100.00	0.61	0.75	0.94	0.98	1.01	1.04	1.05	1.06	1.05	1.04	1.03	1.01	0.99	0.97	0.94
$P_1/P_2 = 0.10$															
1.00	0.49	0.47	0.45	~0.44	0.43	0.43	0.42	~0.41	0.40	0.40	0.39	~0.38	0.38	0.38	0.38
2.00	0.52	0.50	0.47	~0.46	0.45	0.44	0.43	~0.43	0.42	0.42	0.42	~0.42	0.42	0.43	0.44
3.00	0.54	0.51	0.48	~0.47	0.46	0.45	0.44	~0.44	0.44	0.44	0.45	~0.46	0.47	0.48	0.49
5.00	0.56	0.52	0.49	~0.48	0.47	0.46	0.46	~0.48	0.49	0.51	0.53	~0.54	0.56	0.57	0.58
10.00	0.59	0.53	0.49	0.49	0.52	0.55	0.58	0.60	0.63	0.65	0.67	0.69	0.70	0.70	0.71
20.00	0.60	0.54	0.59	0.64	0.68	0.71	0.74	0.77	0.80	0.81	0.83	0.84	0.84	0.94	0.83
40.00	0.61	0.61	0.78	0.83	0.87	0.90	0.93	0.95	0.97	0.97	0.97	0.97	0.96	0.94	0.93
70.00	0.61	0.76	0.95	1.00	1.03	1.06	1.07	1.08	1.08	1.08	1.07	1.05	1.03	1.01	0.98
100.00	0.62	0.88	1.07	1.10	1.13	1.15	1.16	1.15	1.15	1.13	1.12	1.09	1.07	1.04	1.01
$P_1/P_2 = 0.20$															
1.00	0.50	0.47	0.45	~0.45	0.44	0.43	0.42	~0.42	0.41	0.40	0.40	~0.39	0.39	0.39	0.39
2.00	0.52	0.50	0.48	~0.47	0.46	0.45	0.44	~0.43	0.43	0.43	0.43	~0.43	0.44	0.45	0.45
3.00	0.54	0.51	0.48	~0.47	0.47	0.46	0.45	~0.45	0.45	0.46	0.47	~0.48	0.49	0.50	0.51
5.00	0.57	0.52	0.49	~0.48	0.48	0.48	0.49	~0.50	0.52	0.54	0.56	~0.57	0.59	0.60	0.60
10.00	0.59	0.53	0.51	0.53	0.56	0.59	0.62	0.64	0.67	0.69	0.71	0.72	0.73	0.74	0.74
20.00	0.60	0.54	0.64	0.69	0.73	0.76	0.80	0.82	0.85	0.86	0.87	0.88	0.88	0.88	0.87
40.00	0.61	0.67	0.85	0.90	0.94	0.97	1.00	1.02	1.03	1.03	1.03	1.03	1.01	1.00	0.98
70.00	0.61	0.85	1.04	1.08	1.11	1.14	1.15	1.16	1.16	1.15	1.14	1.12	1.09	1.07	1.05
100.00	0.62	0.98	1.16	1.20	1.22	1.24	1.25	1.24	1.22	1.21	1.19	1.17	1.14	1.12	1.10

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Table 7.4 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 2

Column ABC Fixed at A and Fixed at C

a_r B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.30$															
1.00	0.50	0.48	0.46	~0.45	0.44	0.44	0.43	~0.42	0.42	0.41	0.41	~0.30	0.40	0.40	0.40
2.00	0.53	0.50	0.48	~0.47	0.46	0.45	0.45	~0.44	0.44	0.44	0.44	~0.45	0.45	0.46	0.47
3.00	0.54	0.52	0.49	~0.48	0.47	0.46	0.46	~0.46	0.47	0.48	0.49	~0.50	0.51	0.52	0.53
5.00	0.57	0.53	0.50	~0.49	0.49	0.49	0.51	~0.53	0.54	0.56	0.58	~0.59	0.61	0.62	0.63
10.00	0.59	0.54	0.53	0.56	0.59	0.62	0.65	0.67	0.70	0.72	0.74	0.75	0.76	0.77	0.77
20.00	0.60	0.55	0.68	0.73	0.77	0.81	0.84	0.86	0.89	0.90	0.91	0.92	0.92	0.92	0.91
40.00	0.61	0.72	0.90	0.95	0.99	1.02	1.05	1.07	1.08	1.08	1.08	1.07	1.06	1.04	1.02
70.00	0.62	0.91	1.10	1.15	1.18	1.20	1.21	1.22	1.22	1.21	1.19	1.17	1.15	1.13	1.12
100.00	0.62	1.05	1.24	1.27	1.29	1.31	1.31	1.30	1.29	1.27	1.25	1.23	1.22	1.22	1.24
$P_1/P_2 = 0.40$															
1.00	0.50	0.48	0.46	~0.45	0.45	0.44	0.43	~0.43	0.42	0.42	0.41	~0.41	0.41	0.41	0.40
2.00	0.53	0.50	0.48	~0.47	0.47	0.46	0.45	~0.45	0.45	0.45	0.45	~0.46	0.47	0.47	0.48
3.00	0.55	0.52	0.49	~0.48	0.48	0.47	0.47	~0.47	0.48	0.49	0.50	~0.52	0.53	0.54	0.55
5.00	0.57	0.53	0.50	~0.50	0.50	0.51	0.53	~0.54	0.56	0.58	0.60	~0.61	0.63	0.64	0.64
10.00	0.59	0.54	0.55	0.58	0.61	0.64	0.67	0.70	0.73	0.75	0.76	0.78	0.79	0.79	0.80
20.00	0.61	0.57	0.72	0.76	0.80	0.84	0.87	0.90	0.92	0.94	0.95	0.95	0.95	0.95	0.94
40.00	0.61	0.76	0.95	1.00	1.04	1.07	1.09	1.11	1.12	1.12	1.12	1.11	1.10	1.08	1.07
70.00	0.62	0.97	1.16	1.20	1.23	1.25	1.27	1.27	1.26	1.25	1.24	1.22	1.21	1.19	1.20
100.00	0.62	1.11	1.30	1.33	1.35	1.37	1.37	1.36	1.35	1.33	1.31	1.30	1.30	1.33	1.37
$P_1/P_2 = 0.50$															
1.00	0.50	0.48	0.46	~0.46	0.45	0.44	0.44	~0.43	0.43	0.42	0.42	~0.41	0.41	0.41	0.41
2.00	0.53	0.51	0.49	~0.48	0.47	0.46	0.46	~0.46	0.46	0.46	0.46	~0.47	0.48	0.48	0.49
3.00	0.55	0.52	0.50	~0.49	0.48	0.48	0.48	~0.49	0.49	0.51	0.52	~0.53	0.54	0.55	0.56
5.00	0.57	0.53	0.51	~0.51	0.51	0.52	0.54	~0.56	0.58	0.60	0.61	~0.63	0.64	0.65	0.66
10.00	0.59	0.55	0.57	0.60	0.63	0.67	0.70	0.72	0.75	0.77	0.78	0.80	0.81	0.81	0.82
20.00	0.61	0.60	0.75	0.79	0.83	0.87	0.90	0.93	0.95	0.96	0.98	0.98	0.98	0.97	0.97
40.00	0.62	0.80	0.98	1.03	1.07	1.11	1.13	1.15	1.16	1.16	1.15	1.15	1.13	1.12	1.10
70.00	0.62	1.01	1.20	1.25	1.28	1.30	1.31	1.31	1.31	1.30	1.28	1.27	1.25	1.25	1.27
100.00	0.63	1.16	1.35	1.38	1.40	1.41	1.41	1.41	1.39	1.38	1.37	1.37	1.39	1.43	1.47

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Table 7.4 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 3

Column ABC Fixed at A and Fixed at C

a_r	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
B															
$P_1/P_2 = 0.60$															
1.00	0.50	0.48	0.46	~0.46	0.45	0.45	0.44	~0.43	0.43	0.43	0.42	~0.42	0.42	0.42	0.42
2.00	0.53	0.51	0.49	~0.48	0.47	0.47	0.47	~0.46	0.46	0.47	0.47	~0.48	0.49	0.49	0.50
3.00	0.55	0.52	0.50	~0.49	0.49	0.49	0.49	~0.49	0.50	0.52	0.53	~0.54	0.55	0.56	0.57
5.00	0.57	0.54	0.51	~0.51	0.52	0.54	0.55	~0.57	0.59	0.61	0.63	~0.64	0.66	0.67	0.67
10.00	0.59	0.55	0.59	0.62	0.65	0.68	0.71	0.74	0.77	0.79	0.80	0.82	0.83	0.83	0.83
20.00	0.61	0.62	0.77	0.81	0.86	0.89	0.93	0.95	0.97	0.99	1.00	1.00	1.00	1.00	0.99
40.00	0.62	0.83	1.02	1.06	1.11	1.14	1.16	1.18	1.19	1.19	1.18	1.18	1.16	1.15	1.14
70.00	0.62	1.05	1.24	1.29	1.32	1.34	1.35	1.35	1.34	1.33	1.32	1.31	1.30	1.31	1.34
100.00	0.65	1.21	1.39	1.43	1.45	1.46	1.45	1.45	1.43	1.42	1.41	1.43	1.46	1.51	1.56
$P_1/P_2 = 0.70$															
1.00	0.50	0.48	0.46	~0.46	0.45	0.45	0.44	~0.44	0.43	0.43	0.43	~0.42	0.42	0.42	0.42
2.00	0.53	0.51	0.49	~0.48	0.48	0.47	0.47	~0.47	0.47	0.47	0.48	~0.49	0.49	0.50	0.51
3.00	0.55	0.52	0.50	~0.50	0.49	0.49	0.50	~0.50	0.51	0.52	0.54	~0.55	0.56	0.57	0.58
5.00	0.57	0.54	0.52	~0.52	0.53	0.55	0.57	~0.59	0.60	0.62	0.64	~0.66	0.67	0.68	0.69
10.00	0.59	0.55	0.60	0.63	0.67	0.70	0.73	0.76	0.78	0.80	0.82	0.83	0.84	0.85	0.85
20.00	0.61	0.40	0.79	0.84	0.88	0.91	0.95	0.97	0.99	1.01	1.02	1.02	1.02	1.02	1.01
40.00	0.62	0.85	1.04	1.09	1.13	1.16	1.19	1.21	1.21	1.21	1.21	1.20	1.19	1.18	1.17
70.00	0.63	1.08	1.28	1.32	1.35	1.37	1.38	1.38	1.37	1.36	1.35	1.34	1.34	1.36	1.39
100.00	0.68	1.24	1.43	1.46	1.48	1.49	1.49	1.48	1.47	1.46	1.46	1.48	1.52	1.58	1.63
$P_1/P_2 = 0.80$															
1.00	0.50	0.48	0.47	~0.46	0.46	0.45	0.45	~0.44	0.44	0.43	0.43	~0.43	0.43	0.43	0.43
2.00	0.53	0.51	0.49	~0.49	0.48	0.48	0.47	~0.47	0.48	0.48	0.49	~0.49	0.50	0.51	0.51
3.00	0.55	0.53	0.51	~0.50	0.50	0.50	0.50	~0.51	0.52	0.53	0.55	~0.56	0.57	0.58	0.59
5.00	0.57	0.54	0.52	~0.53	0.54	0.56	0.58	~0.60	0.62	0.63	0.65	~0.67	0.68	0.69	0.70
10.00	0.60	0.56	0.61	0.65	0.68	0.71	0.74	0.77	0.80	0.82	0.83	0.85	0.86	0.86	0.86
20.00	0.61	0.65	0.81	0.85	0.90	0.93	0.97	0.99	1.01	1.03	1.04	1.04	1.04	1.04	1.03
40.00	0.62	0.87	1.07	1.12	1.16	1.19	1.21	1.23	1.24	1.24	1.23	1.22	1.21	1.20	1.19
70.00	0.63	1.11	1.31	1.35	1.38	1.40	1.41	1.41	1.40	1.39	1.38	1.37	1.38	1.40	1.44
100.00	0.70	1.28	1.46	1.50	1.52	1.52	1.52	1.51	1.50	1.50	1.50	1.53	1.58	1.64	1.70

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Table 7.4 Equivalent Length Factor, K_L for Lower Segment of Stepped Columns, continued, page 4

Column ABC Fixed at A and Fixed at C

a_r / B	0.10	0.20	0.26	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50
$P_1/P_2 = 0.90$															
1.00	0.50	0.48	0.47	~0.46	0.46	0.45	0.45	~0.43	0.44	0.44	0.43	~0.43	0.43	0.43	0.43
2.00	0.53	0.51	0.49	~0.49	0.48	0.48	0.48	~0.48	0.48	0.49	0.49	~0.50	0.51	0.51	0.52
3.00	0.55	0.53	0.51	~0.50	0.50	0.50	0.51	~0.52	0.53	0.54	0.55	~0.56	0.58	0.59	0.59
5.00	0.57	0.54	0.53	~0.53	0.55	0.56	0.58	~0.60	0.63	0.64	0.66	~0.68	0.69	0.70	0.71
10.00	0.60	0.56	0.62	0.66	0.69	0.73	0.76	0.78	0.81	0.83	0.85	0.86	0.87	0.87	0.88
20.00	0.61	0.67	0.82	0.87	0.91	0.95	0.98	1.01	1.03	1.04	1.05	1.06	1.06	1.05	1.04
40.00	0.62	0.89	1.09	1.14	1.18	1.21	1.23	1.25	1.26	1.26	1.25	1.25	1.23	1.22	1.22
70.00	0.64	1.13	1.33	1.37	1.40	1.42	1.43	1.43	1.43	1.42	1.41	1.40	1.41	1.44	1.49
100.00	0.72	1.30	1.49	1.53	1.54	1.55	1.55	1.54	1.53	1.53	1.54	1.57	1.63	1.69	1.75
$P_1/P_2 = 1.00$															
1.00	0.50	0.48	0.47	~0.46	0.46	0.45	0.45	~0.45	0.44	0.44	0.44	~0.44	0.44	0.43	0.43
2.00	0.53	0.51	0.49	~0.49	0.49	0.48	0.48	~0.48	0.49	0.49	0.50	~0.50	0.51	0.52	0.52
3.00	0.55	0.53	0.51	~0.51	0.51	0.51	0.51	~0.52	0.53	0.55	0.56	~0.57	0.58	0.59	0.60
5.00	0.57	0.54	0.53	~0.54	0.55	0.57	0.59	~0.61	0.63	0.65	0.67	~0.68	0.70	0.71	0.71
10.00	0.60	0.57	0.63	0.67	0.70	0.74	0.77	0.79	0.82	0.84	0.86	0.87	0.88	0.88	0.89
20.00	0.61	0.68	0.84	0.88	0.93	0.96	1.00	0.02	1.04	1.06	1.07	1.07	1.07	1.07	1.06
40.00	0.62	0.91	1.11	1.15	1.20	1.23	1.25	1.27	1.28	1.28	1.27	1.26	1.25	1.25	1.24
70.00	0.64	1.15	1.35	1.40	1.43	1.45	1.45	1.46	1.45	1.44	1.43	1.43	1.45	1.48	1.52
100.00	0.73	1.33	1.52	1.55	1.57	1.58	1.57	1.57	1.56	1.56	1.57	1.61	1.67	1.73	1.80
$P_1/P_2 = 2.00$															
1.00	0.50	0.49	0.48	~0.48	0.47	0.47	0.47	~0.46	0.46	0.46	0.46	~0.46	0.46	0.46	0.46
2.00	0.53	0.52	0.51	~0.51	0.41	0.41	0.41	~0.51	0.41	0.52	0.53	~0.54	0.54	0.55	0.56
3.00	0.55	0.54	0.53	~0.53	0.53	0.54	0.55	~0.56	0.57	0.59	0.60	~0.61	0.62	0.63	0.64
5.00	0.58	0.56	0.56	~0.58	0.60	0.62	0.64	~0.66	0.68	0.70	0.72	~0.73	0.75	0.76	0.76
10.00	0.60	0.60	0.69	0.73	0.77	0.80	0.83	0.86	0.89	0.91	0.92	0.94	0.95	0.95	0.95
20.00	0.62	0.75	0.92	0.97	1.01	1.05	1.08	1.11	1.13	1.15	1.15	1.16	1.16	1.15	1.15
40.00	0.64	1.01	1.21	1.27	1.31	1.34	1.36	1.38	1.38	1.39	1.38	1.38	1.37	1.38	1.39
70.00	0.71	1.28	1.49	1.53	1.56	1.58	1.59	1.59	1.58	1.58	1.58	1.60	1.64	1.59	1.75
100.00	.83	1.47	1.67	1.70	1.72	1.73	1.72	1.72	1.72	1.74	1.78	1.85	1.92	1.99	2.07

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8.0 Symbols

Forces, stresses and moments are expressed in kips, kips per square inch (ksi), and kip feet (kip ft), respectively. Floor loadings, wind loadings, etc., are expressed in kips per square foot (ksf).

A	Cross-sectional area, in.^{2}
a	Clear distance between transverse stiffeners in a plate girder, in.
a_r	Ratio of length of the upper segment of a crane column to the total length from the lowest roof connection to footing
B	Ratio of the maximum moment of inertia of the lower (combined) crane column section to the moment of inertia of the upper section about the same axis
b_f	Flange width of rolled beam or plate girder, in.
C_m	Coefficients applied to bending terms in column interaction formulas to account for load distribution and end conditions
c_c	Distance between neutral axis of complete lower cross-section of a crane column and the centroid of the crane shaft component, in.
c_m	Distance between neutral axis of complete lower cross-section of a crane column and extreme fiber on crane side, in.
e	Eccentricity, in.
F_a	Axial stress allowed in the absence of bending moment, ksi
F_e	Bumper end force at 100% speed, kips
F_b	Allowable bending stress, ksi
F_{bx}	Allowable stress for bending about the (X-X) axis, ksi
F_{by}	Allowable stress for bending about the (Y-Y) axis, ksi
F'_{ex}	Equivalent Euler buckling stress of a stepped crane column, divided by factor of safety, ksi
F'_{ey}	Equivalent Euler buckling stress component of lower segment of a crane column, divided by factor of safety, ksi
F_y	Specified minimum yield stress of steel, ksi
f_a	Computed average axial stress, ksi
f'_a	Average axial stress in crane column component of stepped column, ksi
f_b	Computed bending stress, ksi
f_{bx}	Stress due to bending moment about (X-X) axis, ksi

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f_{by}	Stress due to bending moment about (Y-Y) axis, ksi
f'_c	Ultimate compressive strength of concrete at 28 days, unless otherwise specified, ksi
f_{pe}	Effective prestress after losses, ksi
f_v	Shear stress, ksi
g	Acceleration due to gravity—32.2 fps^2
h	The clear depth of web between flanges, in.
h_f	Depth of girder between flange centroids, in.
I_0	Moment of inertia, about X-X axis, in.^{4} (see Fig. 7.4)
I_S	Moment of inertia of a pair of stiffeners about the centerline of the web, in.^{4}
K	Effective length factor
K_E	Kinetic energy, kip-ft.
K_L	Equivalent column length factor for lower shaft
K_U	Equivalent column length factor for upper shaft
k	Modulus of subgrade reaction, pcf
L	Actual overall length of a member, ft.
E_B	Bumper efficiency
P_1	Column load in upper segment of a stepped crane column, kips
P_2	Column load added to lower segment of a stepped crane column including girder reactions, wall, utility loads, etc.
Q	Side thrust on crane runway girder, kips
$Q_{(eff)}$	Modified side thrust on crane runway girder, kips
$Q_{t(eff)}$	Modified side thrust on bottom flange of crane runway girder
S_B	Bumper stroke, in.
t_w	Thickness of beam or girder web, in.
t_a	Thickness of lateral plate in crane runway girder, in.
t_f	Thickness of beam or girder flange, in.
V_B	Bridge load rated speed, fps

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V_T	Trolley speed, fps
W_B	Bridge weight, kips
W_E	Impact weight/side, kips
W_T	Trolley weight, kips
w_e	Effective width of auxiliary plate, in.
x,y	(As subscripts) axes about which bending takes place, coordinate axes

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Appendix A
Geotechnical Investigation and Foundation (GIF) Manual

Scope. The minimum requirements of a geotechnical investigation for foundations and earthwork are described in this manual.

The scope of the work normally considered for preliminary and general explorations is described in Section A 1. Section A 2 offers a suggested guide to obtain a proposal and to conduct a geotechnical investigation. Earthwork requirements are given in Section A 3.

Methods used to formulate a program of subsurface investigation are presented in *AISE* Technical Report No. 13, Appendix B, Suggested Specification for Subsurface Boring and Sampling.

A 1.0 Geotechnical Investigation

A 1.1 Preliminary Exploration

A preliminary exploration is made to establish the site conditions and includes but is not limited to a review of available topographical and geological information, aerial photographs and data from previous investigations in addition to a site examination. The number and location of borings in relation to the site development are determined mainly from this investigation, and the program of the subsurface exploration is carried out generally using techniques and control of work as described in sections A 1.2 and A 1.3, respectively.

A 1.1.1 Sources of Geological Information. The sources of data on the geology of the United States are available in maps and reports published by government agencies and professional societies. Suggested sources are:

- The United States Geological Survey (USGS). The USGS Index to Publications may be consulted and publications ordered from the Superintendent of Documents, Washington, DC 20005.
- Geological index maps. Individual maps of each state show coverage and sources of all published geological maps.
- Folios of the Geological Atlas of the United States. These contain maps of bedrock and surface materials for many important urban and seacoast areas.
- Geological Quadrangle Maps of the United States. This supplements the older geological folios and includes aerial or bedrock geology maps with a brief descriptive text.
- Water supply papers. Papers on ground water resources in specific localities are generally accompanied by a description of subsurface conditions affecting ground water plus observations of ground water levels.
- Topographic maps. USGS contour maps are generally available for all of the United States, which may be applicable for preliminary site investigations.
- State geologists' bulletins, reports and maps. These provide excellent detailed local geological maps and reports covering specific areas or features. Addresses of all state geological organizations may be obtained from Highway Research Board Bulletin No. 180.
- Geological Society of America (GSA). An index to GSA-published material available from the Geological Society of America.

A 1.1.2 Air Photo Interpretation

A 1.1.2.1 Scale. Aerial photographs at scales of 1:20,000 and 1:40,000 are available for almost the entire United States. For areas of special interest, mosaics have been assembled from individual pictures.

A 1.1.2.2 Sources. The primary organizations of the federal government that provide aerial photographs include U.S. Department of Agriculture (USDA), U.S. Geological Survey (USGS), U.S. Bureau of Reclamation (USBR), U.S. Coast and Geodetic Survey (USC&GS), Department of Defense (DOD) and Tennessee Valley Authority (TVA).

A 1.1.2.3 Utilization and Interpretation. Aerial photographs are utilized for unfamiliar sites and for areas where little information is available. Most aerial photographs are taken as flight strips with 60% or more overlap between pictures. When overlap pictures are viewed stereoscopically, an exaggerated ground relief appears.

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From the appearance of landforms, erosional or depositional features, the character of geotechnical features may be interpreted.

A 1.1.2.4 Limitations. Interpretation of aerial photographs requires considerable experience, and results obtained depend on the interpreter's skill. Aerial photographs necessarily deal with surface and near-surface conditions, and accuracy is limited where dense vegetation obscures ground features. For intensive investigation within developed areas, use of aerial photographs is not essential for a preliminary exploration. Although valuable, the technique does not provide adequate quantitative information for foundation design.

A 1.1.3 Previous Investigation. In developed areas, information from previous work on foundation and subsurface conditions should be reviewed. Records of former construction may contain information on borings, field tests, field instrumentation, ground water conditions and potential or actual problems.

A 1.1.4 Field Appraisal. The existing surface topography and local geological features should be investigated. Rock outcrops, joint patterns and weathering characteristics of exposed rock should be noted and described as they may be related to the investigation.

A 1.2 General Exploration

A general exploration includes borings to recover samples for geotechnical identification. Geophysical methods may also be used to establish a knowledge of the underlying strata. When depths are shallow and equipment is available, test pits and long trenches may be used to establish the subsurface conditions.

A 1.2.1 Geophysical Methods. Geophysical surveys are used to explore large areas or projects of great linear extent rapidly and economically. They indicate average conditions in the proximity of a test setup rather than along the vertical line of a boring. This helps detect irregularities of bedrock surface or interfaces between strata.

The seismic refraction and electric resistivity methods are commonly used. Both methods require specialized equipment and experienced personnel to interpret the field data.

A 1.2.2 Seismic Methods. This method is based on the time required for waves to travel from the source of a blast to points on the ground surface as measured by geophones spaced at intervals along a line at the surface. The refraction of seismic waves at an interface between different strata gives a pattern of arrival time versus distance along a line of geophones.

Refraction methods are used to determine the depth to rock or other lower strata having substantially different wave velocities than the overlying material. The methods are generally limited to depths of approximately 100 ft. and are used where the wave velocities in successive layers become greater with depth. Due care should be taken if dynamiting is undertaken to induce the waves.

A 1.2.3 Electrical Methods

A 1.2.3.1 Resistivity. This technique is based on the difference in electrical conductivity or resistivity of strata. The resistivity of the geotechnical materials at various depths is determined by measuring the potential drop in the current flowing between two existing and two potential electrodes. The resistivity is correlated to material types and is used to determine the horizontal extent and depth of subsurface strata up to 100 ft.

A 1.2.3.2 Drop in Potential. This method is based on the determination of the loss in electrical potential (volts) between three electrodes. The method is similar to the resistivity method but gives a sharper indication of vertical or steeply inclined borders and yields accurate depth determinations. It is more susceptible than the resistivity method to surface interferences and minor irregularities in surface materials.

A 1.2.4 Limitations. With geophysical surveys, the borders between strata can be established, but soil properties are determined only approximately. Sources of error are obtained by differences in moisture, presence of mineral salts or similarities of strata, which affect transmission of source waves leading to vague or distorted conclusions. To supplement geophysical surveys, borings are required to check stratification interpretations.

A 1.2.5 Test Borings. The selection of test boring techniques depends upon stratum changes, material types and permissible disturbance of materials to be sampled. The test boring program should be flexible and should

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be modified in accordance with subsurface conditions encountered, taking into consideration how the data are to be used in the design of the foundations.

A 1.2.5.1 Specific Procedures

(1) Borings without sampling. When only the depth to rock or existence of cavities is to be determined, borings may be made without sampling.

(2) Auger borings. Hand or powered augers are used to advance a hole with periodic removal of material. In some cases, augering may be continuous. The soil changes are determined by visual examination of the material secured. Auger borings are used primarily for shallow exploration above the water table. Though the materials removed are disturbed, they furnish a record of the geotechnicals encountered.

(3) Wash borings. Wash borings are used for the recovery of undisturbed soil samples and in-situ standard soil testings. The hole is advanced by the chopping, twisting action of a bit and jetting with circulating drilling fluids used to remove cuttings from the hole. The soil changes are indicated by the rate of penetration, the action of the rods and examination of cuttings in the drilling fluid. Casing may be required to prevent caving of the hole unless driller's mud or some other stabilizing fluid is used. At specific depths, generally at 3-ft. intervals, sampling and standard soil tests are made.

(4) Rotary drilling. These borings are used for deep exploration or for the penetration of hard materials or strata containing boulders and rocks. The hole is cut by the rotation of a drilling bit as the circulating fluids remove the cuttings from the hole. Changes in soil strata are indicated by the rate of penetration, action of the drilling tools and examination of the cuttings in the drilling fluid.

(5) Core drilling. This procedure is used to drill into bedrock to recover continuous rock cores or to pass obstruction in overburden.

A 1.2.5.2 Test Pits and Trenching. Test pits are used to examine or sample geotechnical materials in situ. Large-diameter rotary bucket augers may be used to form holes in which firm geotechnical materials may be examined and sampled. Pits or trenches may be made with backhoe, bulldozers or clamshell buckets and are inexpensive. Test pits should be located so as not to disturb bearing material at intended positions of shallow foundations.

A 1.2.5.3 General Requirements for Boring Layout. The following rules are a general guide to planning and might not include borings necessary for the construction of all types of mill buildings.

(1) On large sites where subsurface conditions are relatively uniform, borings at 500-ft. spacings may be adequate. Spacing may be decreased in a detailed soil investigation by intermediate borings as required to define variations in subsurface conditions. Where factors such as cavities in limestone or fractures and joint zones in bedrock are to be investigated, wash borings or rotary borings without soil sampling may be necessary at close spacings.

(2) A sufficient number of borings with disturbed soil sampling should be drilled to determine the most representative location for undisturbed soil sampling. Where detailed settlement, stability or seepage studies are required, undisturbed samples of each critical stratum should be obtained from at least one boring. It will be necessary in most cases to augment these data with more detailed testing once the scope of the problem has been determined in the design of the foundations. (2)

(3) Inclined borings are required in special cases where surface obstructions prevent use of vertical holes, or where subsurface irregularities such as buried channels, cavities or fault zones are to be investigated.

A 1.2.5.3.1 Examples of Boring Layouts:

- New sites of large extent—Space preliminary borings so that the area between any four borings includes approximately 10% of total area.
- Sites on soft compressible strata—Space borings 100 to 200 ft. at possible building locations. Add intermediate borings where building sites are determined.
- Large structures with separate, closely spaced footings—Space borings approximately 100 ft. in both directions. Include borings at possible exterior foundation walls, and at machinery or elevator pits.
- Low-load warehouses of large area—Minimum of four borings plus intermediate borings at interior foundations sufficient to determine subsoil profile.
- Isolated rigid foundations 2500 to 10,000 ft.^{2} in area—Minimum of three borings around perimeter. Add interior borings depending on geotechnical conditions encountered in peripheral holes.

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- Isolated rigid foundations less than 2500 ft.^{2} in area—Minimum of two borings at opposite corners. Add holes if erratic geotechnical conditions are encountered.
- Long bulkhead wall—Preliminary borings on line of wall at 400-ft. spacing. Intermediate borings to decrease spacing to 100 ft. or less, depending on the design of wall and geotechnical complexities. Borings should be placed inboard and outboard of the wall line to determine materials in the scour zone at the toe and in the active wedge behind the wall.
- Deep cuts, high embankments—Provide three to five borings on line in a critical direction of established geologic section. The number of geological sections depends on the complexity of the site.
- Caissons and piles. Where these extend to bedrock additional boreholes in the rock to determine its soundness, may be required.

A 1.2.5.4 Depth of Test Borings. The depth of test borings depends on the size and type of proposed structures and is controlled by the character and sequence of the subsurface strata as well as the type of foundation being considered.

A 1.2.5.4.1 Depth Considerations. The following are general guides for planning and do not cover all specific cases.

(1) In general, the depth of the boring should be such suitable stratum to bear applied loads can be determined.

(2) The depth of a boring should be flexible and should be modified as the information is obtained from completed borings.

(3) All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, loose granular materials and potentially expansive fill materials such as steelmaking slag (see Section 2.2.5.1).

(4) Borings in compressible fine-grained strata of great thickness should extend to a depth, where the applied geotechnical stress from superimposed loads is sufficiently small so that consolidation will not significantly influence surface settlement.

(5) Where stiff compact materials are encountered at shallow depths, several borings should extend to depths sufficient to disclose that underlying weaker materials are not present to affect stability or settlement (see Table A1.1).

Table A1.1 Depth of Test Borings

Investigation for:	Borings extended to:
Large structures with separate closely spaced footings	A depth where increase in vertical stress from foundation is less than 10% of the average applied pressure. Generally all borings should extend not less than 30 ft. below the lowest part of foundation unless rock is encountered at a shallower depth.
Isolated rigid foundation	A depth where vertical stress is less than 10% of the average applied pressure. Generally, all borings should extend not less than 30 ft. below the lowest part of foundation unless rock is encountered at a shallower depth.
Long bulkhead wall	A depth below dredge line at least twice the unsupported height of wall. Where geotechnical stratification indicates stability problems, certain borings should reach top of hard strata.
Slope stability	An elevation well below active or potential failure surface or to hard strata.
Deep cuts	A depth below base about equal to the base width of narrow cut. Where cut is above water tables, determine extent of pervious strata below base.
High embankments	A depth about equal to one-half the horizontal length of a side slope. Where soft strata are encountered, borings should reach hard material.

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(6) Where the character of the bedrock is required, or where boulders or irregular weathered material overlies bedrock, borings should penetrate 10 ft. into rock, and at least one boring should extend 40 ft. into rock. In limestone regions, borings should extend through strata suspected of containing solution channels.

(7) Examples of boring depths for representative foundation types and earthwork construction are as shown previously.

A 1.3 Detailed Geotechnical Investigation

A detailed geotechnical investigation should be conducted to supplement the general site information gathered during the preliminary and general exploration as described in Sections A 1.1 and A 1.2. The purpose of the detailed investigation is to establish specific physical geotechnical properties such as, but not limited to, density, moisture content, gradation, permeability, compressibility and shear strength. If bedrock is encountered at the site, core samples should be taken to determine the rock quality designation (RQD) index, classification and compressive strength. The results of the detailed geotechnical investigation will enable the geotechnical engineer to recommend the most suitable foundation system or systems for the type of building construction, anticipated loads, and service factors. The detailed investigation will also define the parameters required for the design of the foundations.

The total quantity of borings, their locations and the frequency of sampling and testing used for the detailed geotechnical investigation should be determined by the geotechnical engineer based on the site information gathered during the preliminary and general exploration.

A preconstruction survey is recommended to determine the precautions necessary during the impending construction.

A 1.3.1 Field Tests and Measurements. Laboratory test results of geotechnical samples are greatly influenced by the disturbance that the samples are subjected to during their extraction, handling and testing. Therefore, upon the direction of the geotechnical engineer, field tests should be conducted to determine in-situ geotechnical properties for correlation with laboratory test results. The following is a list of field tests that may be required:

- Cone penetration (Penetrometer) test.
- Vane shear test.
- Field load test.
- Geotechnical pressure test.
- Permeability test.

In addition to the field testing described above, field measurements and monitoring of site conditions may also be required. The field measurements serve several purposes, such as:

- Determine existing site conditions and the condition of existing adjacent structures prior to construction.
- Establish construction procedures.
- Monitor the effects of construction and the effect of the progressively increased loading of the geotechnical mass as construction proceeds.
- Predict the performance of the completed structures.

Field measurements should include but not be limited to the following items:

- Vertical ground movements.
- Horizontal ground movements.
- Ground water levels and level fluctuations.
- Ground water pressures (pore pressures).
- Excess pore pressure.
- Vibration monitoring during pile driving or blasting.

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A 2.0 Suggested Guide to Obtain a Proposal and to Conduct a Geotechnical Investigation

A 2.1 Minimum Data to Be Provided by the Owner in His Request for a Proposal

- Project title.
- Project location.
- Description of project.
- Purpose of geotechnical investigation (site selection, general exploration).
- Preliminary geotechnical investigation—conceptual—for the formulation of preliminary cost estimates.
- Detailed geotechnicals investigation—to develop geotechnical data for final foundation design.
- Site description, including historical use, if any, of the site; i.e., storage of bulk materials, disposal area, etc.
- Available subsurface information, such as previous test boring data (include with request, or reference location of data); data on foundations of existing facilities in the area, including available settlement records; previous load test data and similar foundation data available for the area.
- General description of proposed facilities, including details of proposed facilities, to provide information on equipment and foundation loads, column spacings, type and size of structures, deep pits and excavations, vibratory and cyclic loadings, and estimates of tolerable total and differential settlement, including details on settlement-sensitive structures and equipment.
- Site data pertaining to drilling borings, such as site access for drilling equipment; availability of water for drilling equipment; requirement, if any, to use union or nonunion labor; working restrictions on the area, if any, as they pertain to safety, personnel, equipment and other; limitations, if any, on working hours for work schedule; and location or existing foundations and utility lines and other restrictions of obstructions, if any, for borings and test pits.

A 2.2 Laboratory and Field Testing of Geotechnical and Rock

Appropriate field and laboratory testing should be conducted with sufficient numbers and types of tests to provide the parameters required in the engineering analyses to satisfy the purpose of the investigation. Special field testing, such as geophysical or vibration measurements, should be conducted when appropriate to provide the parameters necessary for the required engineering analyses.

Field and laboratory testing should be conducted in accordance with applicable ASTM standards. Where ASTM standards have not been established, sufficient documentation should be provided for owner's evaluation of the proposed testing method.

A 2.3 Recommendations to Be Included in the Project Geotechnical Report by the Geotechnical Engineer

(1) Evaluation of foundation alternatives and a specific foundation recommendation for each of the proposed facilities, with due consideration given to project costs, adjacent structures, owner's operations and clearances.

(2) Effects of proposed vibrating equipment foundations:
- Spread footing and mat-type foundations: allowable bearing pressures, foundation sizes and foundation base elevations; allowable maximum toe pressures; estimated settlements and time rate of settlements for various size foundations.
- Deep foundations: conceptual recommendations presenting type, dimensions, appropriate depths, and preliminary estimate of load carrying capacity; load test recommendations and specifications for load tests (if applicable); final recommendations based on load test data (if applicable).
- Preload recommendations.
- Soil densification recommendations.

(3) Deep excavations and retaining structures.
- Lateral soil and hydrostatic pressures for pits and retaining walls.
- Appropriate design parameters for various types of retaining structures, including recommended pressure diagrams.

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- Drainage considerations.
- Appropriate concrete mix.

(4) Unusual design or construction techniques or problems to be anticipated relative to the subsurface conditions encountered in the investigations, including ground water effects.

(5) Recommended protection systems to guard against deterioration of ground facilities caused by soil chemistry.

(6) Ground water effects:
- Possible fluctuations.
- Recommendations for hydrostatic uplift parameters for design of structural piping, pits, etc.
- Appropriate concrete mix for concrete structures.

(7) Dewatering:
- Effects of dewatering on adjacent facilities including but not limited to estimates of settlements of adjacent structures.
- Temporary (construction) and permanent dewatering system recommendations detailing effects on adjacent structures, well water supply systems, etc., if applicable.
- Recommendations for pump tests, if appropriate.

(8) Effects of proposed construction on adjacent existing facilities:
- Procedures to reduce, control or limit movements and settlement of existing foundations.
- Effects on ground water levels.

(9) Specific recommendations for heavily loaded foundations or foundations for other special or unique structures.

(10) Recommendations for foundations subjected to vibratory loadings:
- Design recommendations for foundations to support vibratory loads (specific recommendations applicable to specific vibratory equipment loads, if any, listed in the request for proposal).
- Effects of proposed vibrating foundations on adjacent existing facilities.
- Effects of vibrations from existing facilities on the proposed facilities.

(11) Recommendations for design of roadways and track systems.

(12) Recommendations of borrow materials:
- Location of proposed borrow area.
- Appropriate testing to establish quality.
- Appropriate field explorations to establish quantities.
- Laboratory testing for compaction criteria.
- Limitations on use of proposed borrow material.

(13) Earthwork and site development recommendations:
- General construction comments and recommendations.
- Compaction specifications.

(14) Recommendations for erosion and sedimentation control measures.

A 3.0 Earthwork

A 3.1 Grubbing and Stripping

Grubbing and stripping should be under the direction of a geotechnical engineer. This should consist of the removal of stumps, logs, brush, vegetation, rubbish and other perishable and objectionable matter, and removal and storage of all sod and topsoil and other organic material. All borrow areas, fill areas and cut areas (when the cut material will be used in the fill area) should be grubbed and stripped before commencement of the earthwork.

A 3.2 Site Preparation

Where the site is covered by or includes unsuitable material, the geotechnical engineer should determine its nature and extent and recommend removal or other treatment. Unsuitable material is that which has potential expansion properties or will not satisfactorily accept in-situ compaction because of its composition or water content, or which will cause significant settlement of the overlying structures due to the long-term consolidation. Unsuitable organic materials should be wasted to spoil banks or stored for use as surface dressing on future seeded areas of the completed site. Materials removed because of high water content may be spread and dried or mixed with other dry materials and subsequently used in the fill areas.

Unsuitable materials removed because of steelmaking slag composition and potential expansion properties may be thoroughly soaked with water, placed in stockpiles not exceeding 10 ft. in height, and maintained in a

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moist condition in the stockpile for at least six months in accordance with Section 2.2.5.1, then subsequently used in the fill areas.

A 3.3 Proof-Rolling

After removal of unsuitable materials and before placement of fill, the site should be proof-rolled to provide a stable foundation for the placement of fill and future structures thereon. A minimum 50-ton rubber-tired roller should be used for proof-rolling. A vibratory roller may be used on clean, granular foundation materials (e.g., sand or gravel). The number of passes required will depend on the depth to which compaction or densification is desired. Preliminary testing of various compaction methods will often be necessary to establish the method best suited. The adequacy and method of the proof-rolling should be determined by the geotechnical engineer.

A 3.4 Preloading

If time and economy permit, the bearing capacity and settlement characteristics of unsuitable foundation materials can be improved to acceptable standards by preloading the foundation area. The area should be loaded to the desired maximum allowable bearing pressure under dead and live loads with an adequate safety factor. The preload should remain long enough to allow 90% of predicted primary consolidation to occur. The time required will depend on the soil types and the consolidation characteristics of the various strata.

A 3.5 Placement and Compaction of Earth Fill

The method used for placement of the fill should provide for adjusting the water content of the materials to permit spreading into uniformly mixed layers of a thickness consistent with the type of geotechnical material and compaction equipment to produce the required compaction. After the materials have been dumped and spread, additional water may be added as required. If the water content of the material is too high, the material should not be placed in the fill. Before new layers are placed, the surface of the fill should be inspected by the engineer; additional water, if required, should be added at that time. If the material has dried sufficiently to cause cracks in the surface, it should be scarified to a depth specified by the geotechnical engineer and dampened before new material is deposited thereon. The entire surface shall be maintained in a free draining condition at all times.

Several pieces of equipment should not be permitted to track each other. After the area has been prepared for rolling, no traffic other than water trucks should be permitted to pass over it.

The surface of the fill should have the proper water content required for compaction before additional material is placed. The geotechnical engineer should specify the proper degree of compaction required. Compaction of cohesive materials should be effected using a two-drum articulated type sheepsfoot roller. The frame of the roller should have permanent cleaners attached for keeping the spaces between the feet free of geotechnical material and rock. The feet should be no less than 7 in. long with contact areas of approximately 6 in.^{2} The pressure developed by the rollers should be not less than 750 psi.

When the fill material consists of clean granular materials, lifts should be placed not to exceed 12 in. in thickness. However, in the place of sheepsfoot rollers, vibratory rollers should be used for compaction. The number of passes of this equipment required for adequate compaction should be determined by preliminary tests that correlate type and weight of equipment and number of passes to the degree of compaction required.

Site testing is recommended to ensure the required compaction is achieved.

A 3.6 Placement and Compaction of Weathered Rock Fill

Indurated clay, claystones and shales should be placed in lifts not exceeding 9 in. in thickness. These materials should be crushed and broken using the articulated two-drum type sheepsfoot roller, the 50-ton rubber-tired roller or its equivalent and a D-9 bulldozer or its equivalent. Each lift should be subjected to as many passes of the sheepsfoot roller as required, each pass being followed by the grading and crushing action of the D-9 bulldozer. As maximum disintegration of the weathered rock is the objective in the placing and crushing operation, water in amounts sufficient to saturate the material should be provided. Compaction of each lift should be concluded with a minimum 50-ton capacity rubber-tired roller or its equivalent.

Compaction of well graded granular soil (Granular A and B) should be done to 100% modified Proctor dry density.

To avoid excessive settlements over the years in cohesive soil, the following recommendations are made:

- All particles over 6 in. in size should be removed.
- Moisture content is maintained at 3% above the optimum value during placing.
- As long a rest period as possible is provided to allow the cohesive soil to harden thixotropically.
- Compaction is done to 98% modified Proctor dry density.

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If the specified equipment is not capable of satisfactorily crushing and disintegrating the weathered rock, a 30-ton disk should be utilized. Upon placement and completion of each lift, the surface of the fill should be scarified prior to placement of the subsequent lifts.

A 3.7 Special Compaction Requirements

When nongranulated steelmaking slag is used as granular fill material, it shall first comply with conditions set forth in Section 2.2.5.1.

Successive trips of compaction equipment should overlap. Where new material abuts old material, the old material should be cut or broken by disking or bulldozing until it shows the characteristic colors of undried materials. The rollers should work on both materials, bonding them together.

Where clearances are such that large rolling equipment cannot be properly used, the fill should be compacted by means of mechanical tampers approved by the geotechnical engineer. The degree of compaction should equal that obtained by rolling equipment.

Loose fill shall preferably be placed in 6- to 8-in.-thick layers before compaction. In no case shall fill be placed in loose lifts exceeding 12 in. in thickness.

Laboratory tests should be performed on representative samples of on-site and off-site borrow materials to determine the optimum soil density—moisture content criteria for subsequent site compaction. For fill placed beneath foundations and floor slabs, the field compaction density shall be at least 95% of the maximum laboratory density as determined by the modified proctor test (ASTM D1557). For roads, parking areas and outdoor material storage areas, the field compaction density of the fill should not be less than 95% of the maximum laboratory density. Field density tests of the compacted fill should be conducted to verify that the in-place density meets or exceeds the compaction specifications.

Field density tests of the compacted fill should be initially performed at a minimum rate of one test per 1000 cubic yards of fill placed. If satisfactory control of the compaction is established during the initial filling operations, the rate of testing could be reduced.

Structural fill to support building and/or equipment foundation should extend beyond the footprint of the foundation such that load distribution at 1.5 horizontal to 1.0 vertical can be achieved through the fill.

A 3.8 Placement and Compaction of Resistant Rock Fill

Resistant rock fill is defined as cobble-size rocks (6 to 12 in. in diameter) of sandstone, limestone and igneous-type rock that are resistant to weathering and conventional placement and compaction techniques associated with geotechnical material fills.

The general use of resistant rock fills for structural fill to be used for foundation support is not recommended. The forces at the points of contact of the particles in a fill are roughly proportional to the square of the particle diameters; therefore, the edge-to-edge contact pressures increase tremendously in rockfills as compared, for example, to geotechnical fills. In rockfill, the contact points become crushed under intergranular forces, and the contact areas increase until the contact pressures are no more than the strength of the parent material. Since it has been proved that rock compressive strengths decrease significantly with saturation, wetting of a dry rock fill causes particle crushing and rearranging. As a result of these phenomena, if the grain sizes of the fill matrix material are not properly graded to maximize interparticle contacts, the edge-to-edge contact points will crush with time, increased loading and saturation, causing particle disintegration and possible excessive settlements. Therefore, if resistant rock materials are to be used to support foundations or floor slabs, they should be placed as recommended by the geotechnical engineer.

A 3.9 Final Trimming

The fill area should be constructed to such height above finished grade as will allow for final trimming to the desired lines and grades.

A 3.10 Maintenance of Fill Areas

The fill areas should be maintained in a satisfactory condition throughout the construction period. Construction should cease when satisfactory placing cannot be done because the fill materials are frozen or too wet. In the event that a portion of the fill is placed during freezing weather, frozen borrow materials should not be used in the fill. During periods of alternate freezing and thawing, stockpiled borrow material for the fill should not be used. The upper several in. of fill area may freeze overnight or during other periods of inactivity. If freezing occurs, the frozen material should be removed and spoiled and the surface of the fill area properly scarified prior to the placement of additional fill.

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A 3.11 Excavations

All excavations should be conducted and maintained to guard against and prevent injury to the public and to workmen and should comply with all applicable local, state and federal regulations.

A 3.11.1 Support. All excavations should be performed so as to prevent movement of the earth of adjoining sites and the objects thereon, including floor slabs, pavements, structures, foundations, utility lines, etc. Where danger of undermining adjacent foundations of structures exists, measures should be taken to provide lateral support for the foundations, or the foundations should be underpinned.

A 3.11.2 Open Cut Excavations. All open cut excavations should have stable slopes designed using the geotechnical properties and strength parameter developed by the subsurface investigation and geotechnical testing program.

A 3.11.3 Protection of Foundation Stratum During Construction. All excavations should be performed in the dry. Care should be taken to prevent disturbance of the bearing stratum due to overexcavation, construction traffic, exposure to weather (freezing) and water movements. All clay foundation-bearing strata should be protected by covering them with a 8-in.-thick working mat of blast furnace slag, crushed rock or 4-in.-thick lean concrete. Shale or chalk foundation-bearing stratum should be protected from freezing and slaking by a temporary cover of unexcavated material.

Measures should be taken to prevent upward flow of water into the excavation through the granular-bearing strata. Clay and silt-bearing stratum should be protected against boils or quick conditions caused by excess pore water pressures or trapped pressures in substrata. Installation of sand drains or similar devices to relieve the excess pore water pressures, a dewatering system, or both, will eliminate the trapped pressure.

A 3.11.4 Artificial Improvement of Bearing Stratum. Foundation subgrades for spread footing type foundations should be compacted for at least a depth of 1 ft. below the foundation base to at least 95% of the maximum dry density obtained by the Modified Proctor Test (ASTM D1557).

A 3.11.5 Braced Excavations. All excavations requiring bracing for safety considerations or due to depth and size considerations should be designed using the subsurface information including geotechnical properties and strength parameters developed at the subsurface investigation and geotechnical program.

A 3.11.6 Dewatering. When the ground water level occurs at an elevation above the plan elevation of the foundation subbase, a dewatering system should be installed and the ground water level lowered and maintained at a level of at least 3 ft. below the bottom of the excavation as the excavation progresses. When the excavation is complete, the ground water level should be maintained at a minimum of 3 ft. below the elevation of the foundation subbases until after the foundations are constructed and the excavation has been backfilled. The ground water level should be allowed to rise within the backfill, provided that it is maintained at a level at least 3 ft. below the top of the compacted backfill as the backfill is being placed.

A 3.11.7 Backfilling of Excavations. Backfilling should be performed after the permanent work in the excavation has been inspected and approved. Bracing, when required, should be removed in a manner so as to avoid damage or disturbance to the work, and the excavations should be free of forms and cleaned of trash. Backfill should be clean granular or cohesive soil and shall be free of trash, roots, rocks, boulders, and organic or frozen materials. When nongranulated steelmaking slag is used as granular fill material, it should first comply with conditions set forth in Section 2.2.5.1. Backfill should not be placed on surfaces that are under water, muddy or frozen or contain frost. Backfill should be brought up to final grade, unless otherwise shown or specified, and should be brought up evenly on each side of walls or pipes. Care should be exercised to avoid any wedging action or eccentric action upon or against the structures and to avoid any disturbance or damage to the work. The backfill should be compacted in 6- to 8-in. layers of loose material. Each layer should be compacted as outlined herein. Each layer of fill material should be spread uniformly and compacted to at least 95% of the maximum dry density obtained by the Modified Proctor Test (ASTM D1557). If the area is to be paved, the last 3 ft. of the fill should be compacted to 100% of modified Proctor dry density. Upon placement and compaction of a lift of material, the surface should be scarified to a depth of 2 in. prior to the placement of a subsequent lift.

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Appendix B
Guidelines for the Preparation of Specification for Subsurface Boring and Sampling

B 1.0 General Conditions

This specification is offered as a general guide to the owner for preparation of specifications.

B 1.1 General

The following stipulations, requirements and descriptions of work are hereby defined and described as the General Conditions of the Suggested Specification for Subsurface Boring and Sampling, and all shall apply to the contract unless specifically waived by direction in the Instruction to Bidders, if supplied, and the Contract Agreement.

B 1.2 Points Not Covered by General Conditions and This Specification

Any aspects of the work that are not clearly defined by this specification shall be governed by the rules of the best prevailing practice in the area of the work for that class of work as determined by the engineer.

B 1.3 Definitions

Engineer:	The engineer shall be defined as an authorized representative of the owner.
Contractor:	The contractor shall mean a person, persons or corporation who has submitted a proposal that has been accepted in writing by the owner or his representative. This definition shall also apply to the contractor's authorized representative at the site of the work.
Work:	Work shall consist of furnishing the following: all tools, equipment, materials, supplies, transportation, labor, supervision, logs, records, drawings and all things necessary or incidental to compliance with the requirements of the Contract Documents.
Contract Documents:	Contract Documents shall consist of the Instructions to Bidders, the Suggested Specification (including General Conditions and Technical Conditions), the Plan and Location of the Borings, and Accepted Proposal and Contract Agreement.

B 1.4 Qualification of Bidder

The bidder shall submit to the engineer, upon request, a tabulation of similar work performed by him within the previous two years as evidence of qualification. The tabulation shall include a description of the work, the approximate quantities involved and the name of the party or parties with whom the contract was made.

B 1.5 Inspection of Site by Bidder

It is expected that the bidder shall visit the site and thoroughly acquaint himself with the local conditions relative to the prosecution of the work required by the contract, such as handling and storage of materials and equipment, working conditions, availability of water and other supplies, transportation, access to individual boring locations, etc. Failure to make this inspection will not relieve the successful bidder of his responsibility for properly estimating the cost of satisfactorily performing the complete work as required by the Contract Documents within the time set forth in the Proposal.

B 1.6 Borings

B 1.6.1 Location of Borings and Survey. The approximate locations of the required borings are indicated on the Plan and Location of Borings. The exact location of the individual borings will be determined and staked or otherwise marked in the field by the engineer. It shall be understood that the final locations of some borings may be modified in the field by the engineer, depending upon topographic features and subsurface conditions encountered during progress of the work. Borings on land may be offset from the designated location by the contractor to avoid surface obstructions or impractical working conditions provided that approval is first obtained from the engineer. Test borings in water shall fall within a radius of 5 ft. from the designated locations.

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B 1.6.2 Number and Depth of Borings. The number of borings required is indicated on the Plan and Location of Borings. It shall be understood that the final number of borings may be increased or decreased at the discretion of the engineer.

All borings shall be extended to the depths, elevations or conditions specified in the Instructions to Bidders. The final depths of borings shall satisfy these requirements in all cases unless the engineer specifically directs otherwise in the field.

B 1.6.3 Sequence of Borings. The engineer reserves the right to designate the sequence in which borings will be made.

B 1.6.4 Abandoned Borings. No payment will be made for any boring that has been abandoned by the contractor before reaching the depth, elevation or condition specified on the Plan and Location of Borings, unless the engineer approves and accepts the boring as being completed. The engineer may accept a boring that fails to reach the required depth due to an unusual obstruction that, in his opinion, could not reasonably have been anticipated.

The contractor shall afford the engineer the opportunity to measure the depth of any boring and to inspect samples of materials recovered before abandonment and removal of casing and drilling equipment.

B 1.6.5 Boring Logs. Within five calendar days after completion of the work, five copies of the boring logs giving the information required under Section B 2.13 shall be submitted to the engineer.

B 1.7 Laws, Ordinances, Regulations and Permits

The contractor shall comply with all the laws, ordinances, rules and regulations of the federal and state governments, or of any political subdivisions thereof, that are applicable to the work to be performed under the contract.

All permits and licenses, of whatever nature necessary to the prosecution of the work, shall be obtained by the contractor at his expense.

B 1.8 Pipes, Cables and Underground Structures

It shall be the contractor's responsibility to ascertain the location of all pipes, cables and underground structures in the area of his operations, and to use necessary precautions to avoid them in making his borings. If it is established that the location of a boring will cause interference with an underground facility or structure, the contractor shall so advise the engineer. At his discretion, the engineer may designate a new location for the boring or authorize its omission.

B 1.9 Work on Private Property

The contractor shall make his own arrangements with the owners of property on which borings are located, or over which access is required, with respect to any work thereon and any damages which he may cause. Any expense that he may incur therefrom shall be reflected in the unit prices bid in the accepted proposal.

B 1.10 Work on Public Property

The provisions of Section B 1.9 shall also apply to any public properties on which borings are located, or over which access is required. The contractor shall obtain permission of the appropriate governmental agency before entering such property.

B 1.11 Protection of Work, Persons and Property

The contractor shall provide and maintain any barricades, lights or other safety devices necessitated by hazardous conditions or required by local authority.

B 1.11.1 Injury to Persons and Damage to Property

The contractor shall be responsible for all injury to persons and damage to property resulting either directly or indirectly from his operations. All physical damage shall be repaired promptly.

Upon completion of the work, the contractor shall furnish satisfactory evidence that all claims arising from injury to person or damage to property resulting from his operations have been resolved.

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B 1.11.2 Restoration of Disturbed Areas. Ground areas disturbed by the contractor's personnel and equipment shall be restored as nearly as possible to their original condition, or to the satisfaction of the property owner. This applies in particular to areas planted in crops, grass or shrubbery.

Holes drilled in areas used for pedestrian traffic shall be temporarily plugged or capped immediately upon completion of the boring, and permanently plugged flush with the surface immediately after the final water level has been obtained.

B 1.11.3 Site Cleanup. After completing field operations, the contractor shall promptly remove all equipment and material brought by him to the site and shall restore the site to its original condition as nearly as possible.

B 1.11.4 Safe Practices in Drilling. In accordance with generally accepted drilling practices, the contractor shall be responsible for all matters dealing with safety in performing the work, including safety of all persons and property during performance of the work, his own employees, and any and all employees of subcontractors who may perform on his behalf. This requirement will apply continuously regardless of time or place and will in no way be altered because the engineer gives general directions as to the location where samples should be taken.

B 1.11.5 Insurance. The contractor shall obtain and pay for such insurance as will protect him, the owner and the engineer from claims under the Workmen's Compensation Act and from any other claims for damages for personal injury including death, or for damages to property, both real and personal, that may arise from operations under the contract, whether such operations be by himself or by anyone directly or indirectly employed by him. Insurance coverage shall be in types and amounts as shall be determined by the owner.

B 1.12 Supervision, Personnel and Manner of Prosecution of Work

The contractor shall be represented at the site of the work at all times by a competent boring supervisor or foreman. Directions given him by the engineer shall be binding on the contractor, and such directions will be confirmed in writing when so requested.

Once work commences, a driller shall continue to work on the project until its completion unless the engineer requests his transfer. The contractor shall not transfer a driller without the written approval of the engineer.

B 1.13 Inspection of Work

The contractor shall at all times provide full opportunity for inspection of the work by the engineer. Any work that is unsatisfactory to the engineer shall be remedied immediately to the satisfaction of the Contract Documents and the engineer at the expense of the contractor.

B 1.14 Right to Suspend Work

The engineer reserves the right to suspend the work, wholly or in part, for a period of time as may be necessary due to unsuitable weather or such other conditions that are considered unfavorable for the satisfactory prosecution of the work; or such time as necessary by reason of failure on the part of the contractor to carry out orders given or to perform any or all provisions of the Contract Agreement. No additional compensation shall be paid to the contractor because of such suspension.

B 1.15 Termination of Contract

If the contractor fails to begin the work under contract within the time specified on the Contract Agreement; if he should refuse or fail to prosecute the work with sufficient and proper materials, workmen and equipment; if he should fail to make prompt payment for the material or labor, or disregard laws, ordinances or the instructions of the engineer, or otherwise be guilty of a substantial violation of any provisions of the contract; if he shall be adjudged a bankrupt or he should make a general assignment for the benefit of his creditors; or if a receiver should be appointed on account of his insolvency, then the engineer may, without prejudice to any other right or remedy, and after giving the contractor ten days' written notice, terminate the employment of the contractor. The contractor will at the time be paid for the work accomplished at the unit prices agreed to in the Contract Agreement. No payment in addition to the amount due the contractor for work accomplished will be made.

B 1.16 Patents and Permits

The contractor shall pay all royalties and shall indemnify and save harmless the engineer from any claims for infringement by the reason of the use of any patented designs, device, material or process to be performed or used under the contract.

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B 1.17 Indemnity Provisions

The contractor agrees to hold harmless, indemnify and defend the engineer from and against any and all liability arising out of the performing of the drilling activity described in these documents. This shall not include sole negligence of the engineer, his agents or employees.

B 1.18 Payments

B 1.18.1 Payment for Contracted Work. The prices quoted by the contractor in his proposal and agreed to on the Contract Agreement shall include costs of all work for which the contractor expects to be reimbursed.

B 1.18.2 Payment for Additional Work. If, when preparing his bid, the contractor expects payment for work other than, or in addition to, work outlined in his proposal or Contract Agreement, the contractor shall add to his proposal or Contract Agreement, in ink, the added work for which he expects payment and the unit or lump sum prices for proper execution of said work. All payments to the contractor will be made on the basis of unit prices quoted and made part of the Contract Agreement. No claims for extra work of any kind will be allowed except as specifically ordered in writing by the engineer.

B 1.18.3 Payments Withheld. Payments for the work may be withheld by the engineer for any of the following reasons:

(1) Claims filed, or reasonable evidence indicating a probable filing of claims
(2) Failure of the contractor to make payments for material or labor
(3) Damage to persons or property, or the probability of damage claims

B 2.0 Technical Conditions

B 2.1 Soil Boring

Soil borings are made to determine the true nature, arrangement, thickness and texture of the various soil strata as they exist in the ground. Every effort should be made to locate and record the datum elevation at which any change in stratification occurs. Truly representative samples of the geotechnical material comprising each stratum as it exists in the ground, and including its natural moisture content, should be obtained. Each sample, as it is removed from the ground, should be packed so that it will reach the laboratory in as near as possible the condition in which it was removed from the ground without loss of water or damage by freezing, heating, breakage of containers or other disturbances in transit.

The following procedure shall be used to advance the boring to ensure satisfactory field testing and sampling:

(1) Steel casing of not less than 4 in. ID shall be driven as required to maintain an open hole for field testing and sampling operations. In no case shall the casing be advanced to a depth greater than the depth at which field testing or sampling is to be undertaken.
(2) In advancing the boring, the casing shall be driven down without washing to depths as directed by the engineer, after which the material shall be cleaned out of the bottom of the casing by using a cutting or chopping bit. Drill water may be forced down through the drill pipe and out through ports in the chopping bit to carry the cuttings up and out of the boring. It is imperative that water ports in the cutting bit be arranged so that there is no jetting action of the drill water ahead of the chopping bit. In no case shall the cleaning operation proceed beyond the lower limit of the casing unless specified by the engineer.
(3) The minimum amount of water necessary to carry away the cuttings shall be used.
(4) In borings where rock coring is not anticipated, the casing may be omitted only if it can be shown to the satisfaction of the engineer that sampling operations without the casing will not entrain soils from an elevation higher than the depth at which field testing or sampling is to be made.

The contractor is permitted to use an alternate method of advancing the boring, provided that he can show that a clean hole will be maintained for field testing and sampling operations and that the samples obtained are truly representative of the soil in place. Before proceeding with an alternate method of advancing the boring, the contractor must obtain the written permission of the engineer.

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As the boring is advanced, special care shall be taken to note depths below the ground surface at which there is a loss or gain of the water in the casing. All drilling equipment, including the drill rigs, are to be in good working order at all times throughout the duration of the project. If, in the opinion of the engineer, the equipment supplied is inadequate for proper determination of field strength values or for obtaining the desired samples, it shall be replaced immediately with suitable equipment at the contractor's expense.

B 2.2 Field Testing and Soil Sampling

Standard penetration tests shall be conducted in accordance with ASTM 1586 and conducted at every change of strata and within a continuous stratum at intervals not exceeding 3 ft. between the bottom of one sample and the top of the next sample.

The sampler shall be driven with a guided hammer or ram into undisturbed material below the bottom of the boring after the boring has been cleaned to remove all loose and foreign material. The sampling spoon shall be a 2-in. OD split barrel sampler with an ID of 1 $^3/_8$ in. The inside of the split barrel shall be flush with the inside of the drive shoe. The use of other split barrels is permitted provided the engineer has inspected and approved the sampler.

The bottom of the sampler shall be sharpened to form a cutting edge at its inside circumference. The beveled edge of the drive shoe shall be maintained in good condition and, if excessively worn, shall be reshaped to the satisfaction of the engineer. The drive shoe of the sampler shall be replaced if damage to it causes projections within the interior surface of the shoe. Each drill rig shall be equipped with a minimum of two drive shoes in good condition. Drive shoes shall conform to ASTM Standard D 1586. The hammer or ram used to drive the 2-in. OD sampler shall weigh 140 lbs. and shall fall freely through a height of 30 in. Where a drum and rope device is employed, manila rope shall be used. The number of blows required to drive the sampler each 6 in. for a total depth of 18 in. shall be observed and recorded. The record shall clearly show the number of blows for each 6 in. of penetration. Cumulative blows will not be accepted. In soils requiring 25 blows or more per 6 in. of penetration, the sampler shall be driven 12 in. and the number of blows for each successive 6 in. of penetration shall be observed and recorded. In hard materials requiring more than 50 blows for 6 in. of penetration, the blows for smaller amounts of penetration may be observed and recorded with special note of the amount of penetration actually obtained. In the absence of the engineer, a resistance of more than 50 blows for 6 in. or less penetration shall be considered as refusal. If the boring is extended to depths beyond the point of refusal, rock coring shall begin with a run of 3 ft. or less as directed by the engineer.

When the water table has been reached, particular care must be exercised to maintain the hole full of water at a level higher than the ground water level preceding and during the standard penetration test. During the removal of the washpipe, chopping bit and assembly and insertion of the sampling barrel, a positive inflow of water at the top of the casing shall be maintained. Flap-type trap doors protruding at any point into the inside diameter of the sampler may not be used without prior approval of the engineer. If requested, the contractor shall furnish the engineer with a complete description of the sampler, giving inside and outside diameters, length of barrel and check valve used.

The sampler shall be fastened to its drive pipe by a connection embodying a check valve arranged so as to permit the ready escape of water trapped above the soil sample as the spoon is driven down into the soil, but which will close as the soil sample and sampler are withdrawn, thus preventing the development of hydraulic pressure on top of the soil sample.

Immediately upon removal from the hole, the sampler shall be carefully disassembled and the material classified. The most representative and least disturbed portion of the sample, measuring 3 in. in length, shall be placed immediately into an airtight jar. Where a change in strata occurs within the spoon sampler, a sample of each material shall be placed in separate jars. The depth of the change shall be recorded. The lid of each jar shall be securely fastened. Once sealed, the jar shall not be opened by the contractor. The jar shall be properly labeled as to boring number, depths at both top and bottom of sample, number of sample and number of blows for each 6 in. of penetration, or as otherwise stipulated above. The project identification and date of sampling shall be clearly shown on the label. If the length of sample recovered is insufficient to provide a sample 3 in. long, the most representative and least disturbed part of the soil sample shall be placed in a glass jar. The length of the sample, if less than 3 in., shall be noted on the jar.

The glass jars shall be approximately 5 in. high and 1 $^3/_4$ in. ID at the mouth, with an inside diameter of the jar not more than $^1/_4$ in. larger than that at the mouth. The jar shall be provided with metal screw caps containing a rubber or waxed paper gasket. The glass jars shall be packaged in cardboard boxes that contain individual

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cardboard partitions for each jar. The outside of the box shall be permanently marked with the project number, boring number, sample number and sample depth. Packaging samples from more than one boring in one box is acceptable only if all of the samples from each individual boring can be placed in the box.

If a soil sample is lost or is found unsatisfactory as to size or condition, a second attempt shall be made to obtain a satisfactory soil sample before advancing the casing to a lower elevation. Washed samples will not be accepted unless, in the opinion of the engineer, a spoon sample cannot be reasonably obtained.

If, in the opinion of the engineer, a recovered sample is wash material resulting from the cleaning operation, the contractor shall remove all such material from the boring with a standard clean-out auger, or a clean-out auger with sludge barrel if necessary, to the lower limit of the previous sample, and a second attempt shall be made to obtain a satisfactory sample. A spring-type sample retainer installed in the tip of the sampler shall be used when necessary to prevent loss of the sample.

When very soft, cohesive or water-bearing granular materials are encountered, the hole must be maintained full with water or at a level higher than the ground water level before initiating sampling operations to reduce the possibility of material flowing upward into the casing. Where necessary, the density of the drill water in the casing may be increased by adding bentonite or driller's mud to the drill water.

Where extremely compact material or boulder obstructions prevent further advance of the boring by driving casing or by the wash method, fishtails or boulder busters may be used with the approval of the engineer. Blasting with small explosive charges to facilitate advancing the boring through boulders and other small obstructions will be permitted only after written approval by the engineer. If casing is used, it must be pulled up to an elevation of at least 5 ft. above the elevation of the charge before detonation to avoid damage to the casing. Notation of the size of charge and time of detonation shall be made in the boring records.

The soil samples shall be turned over to the engineer at the site or shall be shipped to the laboratory, as directed by the engineer.

B 2.3 Undisturbed Soil Boring and Sampling

For obtaining 3-in.-diameter undisturbed soil samples, 30-in.-long borings shall be made, as specified under this section and Section B 2.2. At locations in the soil strata selected by the engineer, undisturbed material samples shall be recovered by means of special piston-type samplers. When ready to take such samples, all loose and disturbed materials shall be removed to the bottom of the casing or of the open boring. Cleaning out of the last 6 in. above the intended top of the sample must be accomplished with a standard clean-out auger, or a clean-out auger with a sludge barrel if necessary. Cleaning out shall be done so that the soil immediately below the bottom of the casing shall be as nearly undisturbed as possible. The sampling device connected to the drilling rod shall then be lowered slowly to the bottom of the hole and the sampler forced into the soil for a distance of not less than 24 in. nor more than 27 in. If obstructions such as gravel particles prevent the insertion of the sampling tube, lengths of undisturbed soil samples less than 24 in. will be permitted with the approval of the engineer.

Undisturbed soil samples are to be recovered by means of a thin-wall piston-type sampling device, either a stationary-type sampler in which piston rods extend to the ground surface or a self-contained, hydraulically operated piston sampler that has the approval of the engineer.

When samplers using piston rods extending to the ground surface are used, positive locking of the piston rods with respect to the surface of the ground must be provided to prevent upward or downward motion of the piston during the advance of the sampling tube, and the piston rods must be positively locked to the drill pipe at the surface during removal of the sampler for the depth to which it penetrated undisturbed material. If the piston rods are locked to the mast of a truck-mounted drill rig, the rig shall be blocked and anchored to the ground in such a manner as to prevent motion of the rig during the sampling operation. If approved in advance by the engineer, samples may be recovered in hard materials by an open-type, thin-wall sampling device.

Tubes for undisturbed samples shall be provided by the contractor, and shall be of 16-gauge seamless brass, hard aluminum or steel. Steel tubes shall be seamless steel, properly cleaned and polished on the inside and fully coated with lacquer on the outside. Sample tubes shall have a machine-sharp cutting edge with a flat bevel to the outside wall of the tube. The cutting edge shall be drawn in to provide an inside clearance beyond the cutting edge of 0.015 in., ± 0.005 in.

In the operation of securing the undisturbed samples, the sampler shall be forced into the geotechnical material at a rate approved by the engineer. The sampler shall be pushed or jacked downward and not be driven unless the character of the material is such that driving with the hammer is absolutely necessary and is approved by the engineer.

The sampler, with its contained geotechnical material sample, shall remain in place for from 5 to 30 minutes, depending on the nature of the material being sampled, at which time the contractor shall rotate the drill rod

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through two complete revolutions or until the soil immediately below the sample has sheared. The tube containing the sample shall then be carefully removed from the boring and detached from the driving head, and the sample shall not be extruded from the tube. The sample in the tube shall be carefully squared at each end, not less than $^1/_2$ in. back of the ends of the tube, and both end spaces shall be completely filled with hot, approved sealing wax or material compound. The ends of the tube shall be closed with snug fitting metal or plastic caps, which shall be secured in place with adhesive or friction tape.

In very soft materials, a weighted drilling mud may be required by the engineer, whether or not casing is used, in order to maintain a pressure on the material as nearly equal as possible to that existing before the drilling operations.

Undisturbed samples shall be clearly, accurately and permanently marked to show the number of the hole, the number of the sample, the depth from which the sample was taken, the measured recovery, top and bottom of the sample, and any other information that may be helpful in determining subsurface conditions. Whenever possible, a measurement of the force required to push the undisturbed sample tube into the geotechnical material shall be obtained and recorded, both on the sample tube and on the boring records.

B 2.4 Classification of Geotechnical Material

Geotechnical material samples taken during the site investigation shall be used to classify the underlying strata. The soils shall be described according to the Unified Soil Classification System. Additional terms as to the texture, state, moisture and color (see Sections B 2.4.2 through B 2.4.4) shall be included to provide a complete description of the geotechncial material encountered.

B 2.4.1 Texture. A granular material shall be considered basically either a gravel or a sand. Geotechnical material in either category shall be described as coarse, medium or fine. The supplementary texture of the granular material shall be given through the use of one adjective only. A cohesive soil shall be considered basically either a silt or a clay. The supplementary texture of the cohesive material shall be given through use of one adjective only.

The texture of either granular or cohesive geotechnical material may be modified to disclose the presence of organic material, using such measures as *trace* or *some,* or to disclose the presence of foreign particles in cohesive materials, such as pebbles, using such words as *few* or *many* to indicate amount.

When nongranulated slag materials are encountered, an attempt shall be made to determine the type of slag, i.e., blast furnace slag or steelmaking slag. This can be done by a qualified geotechnical engineer through expansion tests. The importance of this determination is discussed in Section 2.2.5.1 of *AISE* Technical Report No. 13.

B 2.4.2 State. Granular materials shall be defined in terms of density such as very loose, loose, medium dense, dense or very dense. Cohesive soils shall be defined in terms of consistency such as very soft, soft, medium stiff, stiff, very stiff or hard.

B 2.4.3 Moisture. The amount of moisture present in a soil sample shall be defined in terms of wet, moist or dry.

B 2.4.4 Color. The basic color of a geotechnical material, such as yellow, brown, red, gray, blue or black, shall be given and shall be modified if necessary by adjectives such as light, dark, mottled or mixed.

B 2.5 Rock Drilling and Coring

This type of drilling and sampling shall consist of taking a core of rock where the soil boring has refused further penetration. It is for the purpose of determining accurately the nature, strength and character of the rock formation.

The core borings shall be made through the 4-in. casing used for the soil borings. The casing shall be driven and sealed into the rock formations to prevent seepage from the overburden into the hole to be cored.

A series N double tube core barrel with a diamond bit and reaming shell or N-series wire line shall be used to recover rock cores not less than 2 $^1/_8$ in. in diameter. The contractor shall drill the minimum distance into firm bedrock as called for in the Instructions to Bidders or the Plan and Location of Borings or both, or to depths as directed by the engineer. Soft or decomposed rock shall be sampled with a driven sampler whenever possible. The core drill mechanisms shall be of the hydraulic feed type.

The core barrel shall be in efficient operating condition. The drill rods shall be series N only or approved equivalent. No drilling will be permitted with drill rods that are not straight.

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In coring rock including shale, claystone and coal, the contractor shall control the speed of the drill and the drilling pressure, amount and pressure of water and the length of run to give the maximum recovery of the rock being drilled.

The first length of drill run shall not exceed 3 ft. No grinding of core will be permitted. The contractor shall be expert in detecting the blocking of core in barrels. At any suspicion that such is occurring, the barrel shall be removed from the hole, the core removed, and coring shall not be continued until care has been taken to see that the core barrel, bit and other equipment are in satisfactory operating condition.

If poor recovery is experienced due to failure of the contractor to consider the above factors after having been given advance warning by the engineer, the hole shall be redrilled at the expense of the contractor.

If soft or broken rock formations are encountered that cause broken pieces of rock to fall into the hole and cause unsatisfactory coring or if voids of any type, including mined-out coals seams, are encountered that endanger the continued downward progress of the boring, the hole shall be reamed with flush joint casing to a point below the broken formation. The size of the flush joint casing shall permit securing of the specified core size. This procedure shall be repeated as many times as may be necessary to keep the hole clean. The use of standard wire line tools of the specified size is an acceptable alternate procedure.

Where soft or broken rocks are anticipated by the engineer, the contractor shall, upon the engineer's instructions, reduce the length of runs to less than 5 ft. to reduce the core loss and core disturbance to a minimum. Failure to follow the foregoing procedures when ample warning of unusual subsurface conditions has been given in advance shall constitute justification for the engineer to require redrilling, at the contractor's expense, of any boring from which core recovery is unsatisfactory.

When, in the opinion of the engineer, the rock is in either a soft or broken condition, precautions must be taken to keep the core intact as much as possible. The core barrel shall be dismantled horizontally and the core pushed into a trough or removed with equal care by a means approved by the engineer.

The individual drill run in the coring operation should not be in excess of 10 ft. and shall be of such amount, depending on the nature of the rock encountered, so as to achieve maximum core recovery. Every effort shall be made by the contractor to obtain as full recovery of rock as possible, and all significant actions of the bit and reasons for loss of core shall be recorded in the boring log.

Inasmuch as the function of rock borings includes determination of width, direction, extent and spacing of rock fractures or voids that may have occurred due to subsidence or otherwise, the contractor shall exercise particular care in recording water losses, artesian pressures, rod jerks or any other unusual coring experience that, supplementing the core record, will provide information on the nature and extent of fracturing or voids. Fractures and their estimated widths shall be marked in the core boxes and the location of voids shall be clearly indicated.

Immediately upon recovery of the core barrel from the hole, the rock core shall be carefully removed from the barrel, classified and measured for percentage of recovery. Rock cores shall be placed in the sequence of recovery in well-constructed wooden boxes provided by the contractor. Wood partitions shall be placed at the end of each core run and between rows. The depth from the surface of the boring to the top and bottom of the drill run shall be marked on the wood partitions. A wood partition showing the length of core lost shall be placed at the end of each run immediately above the partition showing the depth of the bottom of the run. The order of placing cores shall be the same in all core boxes. The top of each core obtained and its true elevation shall be clearly and permanently marked in each box.

The core boxes shall be marked with the project number, boring number, box number and depths from which the cores were recovered. The core boxes from each boring shall be numbered consecutively from top of boring to bottom of boring and with the total number of boxes in the boring marked in each box, i.e., 1 of 3, 2 of 3, 3 of 3. Both ends and the top of the core box shall be permanently marked with the project number, boring number, box number and core run information, which shall include run numbers, top and bottom depths of run and recovery length. The depth of change of rock strata shall be clearly marked within each box. Special care shall be taken to locate and note the elevation and thickness of all claystone layers, soft decomposed rock, cavities or rock fractures. These shall be clearly shown in each box and on the drill log. The total length of core obtained and the corresponding distance drilled shall be clearly shown on the log of each boring.

Rock cores from two different borings shall not be placed in the same core box.

When each boring is complete, the box containing the cores shall be provided with a tight lid. The number of the boring shall be clearly and permanently marked on the top and on both ends of each box in paint. The core boxes and partitions shall be constructed to accommodate 16 lineal ft. of core in four rows of approximately 4 ft. each, and shall restrain the cores against shifting during transport. The boxes shall be constructed with hinged tops and secured with several screws.

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The cores shall not be transferred from the original field boxes without written approval of the engineer. If approval is obtained, the engineer must be present during the transfer of the cores.

The contractor shall provide suitable dry storage for all rock cores until the completion of the work, at which time they shall be delivered to the destination specified in the Instructions to Bidders, Contract Agreement, or as directed by the engineer.

B 2.6 Classification of Rock Cores

Rock shall be described in accordance with the following items of classification:

Type:	Use geological nomenclature such as limestone, shale, claystone, sandstone, granite gneiss, schist, marble, slate, basalt, quartz. Where necessary, one adjective shall be used to modify the type of rock.
Structures:	Laminated (give approximate angle dip from horizontal), massive.
Condition:	Solid, broken, fractures with stains, fragmented, weathered (rotten), seamy.
Hardness:	Soft, medium soft, medium hard, hard.
Basic Color:	Yellow, brown, red, gray, blue or black; modify the color if necessary using adjectives as light, dark, mottled or mixed.

B 2.7 Ground Water Observations

Observations shall be made of ground water levels in all completed borings. Any and all unusual water conditions and elevations at which there is a gain or loss of water in boring operations, or elevations at which water under excess pressure was found, shall be recorded completely in the boring logs. When water under excess pressure is observed, the drilling operation shall be stopped and the casing extended above the ground surface so as to contain the flow of water. After allowing the water level to come to equilibrium, the height of water above the ground surface shall be recorded. Ground water levels shall be measured before and after pulling the casing, where used, and again 24 hours later.

If more than one day is required to complete a boring, water readings shall be taken each morning prior to the commencement of drilling operations. Whenever required by the engineer, bore holes shall be bailed for observations of ground water conditions. When the open boring process uses natural or commercial drilling mud to stabilize the hole, the hole shall be flushed thoroughly with clean water at the completion of the boring for the purpose of observing ground water levels.

B 2.8 Auger Boring and Sounding

Where it is necessary to clearly establish the depth of firm bedrock for piles or drilled-in-place caissons, auger borings shall be conducted in accordance with ASTM D 1452 using an auger with a minimum diameter of 3 in. The auger teeth shall be high-grade steel or carborundum cutting teeth or equivalent. The augering shall be carried out continuously from the ground surface to refusal of firm hard bedrock. From the feel of the cutting bit and the chippings that come to the surface, the contractor shall give the description of the soils encountered and their approximate depth below the ground surface.

The contractor shall clearly note in his records where the auger passes from soil to decomposed rock.

If required in the Instructions to Bidders or Contract Agreement, soil samples of the chippings that come to the surface shall be secured at regular intervals of 3 ft. and each change in strata. The samples shall be placed in glass jars for classification. It is sufficient to give approximate depths from which the chippings were secured. The jar samples shall be tightly sealed and clearly labeled as to boring number, depth below ground surface, type of material and date of sampling.

Auger borings may also be used for shallow earth explorations.

Sounding devices, such as a cone penetrometer or steel rod driven into the soil, may be used where soil sample recovery is not required. This method may also be used to estimate pile driving depth.

B 2.9 Vane Shear Tests

Vane shear tests shall be conducted in borings at the discretion of the engineer.

The vane shear test apparatus shall be supplied by the contractor. The apparatus shall be vane shear testing equipment with a precision torque head subject to the approval of the engineer. The apparatus shall have a ratio of 720:1 between the crank handle and the vane. The equipment shall be complete and in good working order. Immediately prior to commencement of drilling, the contractor shall have the force gauge calibrated by a testing laboratory approved by the engineer and shall submit the results to the engineer. After calibration, the testing laboratory shall package the gauge and ship it directly to the site.

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When vane shear tests are to be performed, the casing shall be driven to the depth selected by the engineer. The hole shall then be carefully and completely cleaned out to within 6 in. of the proposed test level using chopping bits and cleaning tools that have outlet ports that cause a positive upward flow of the wash water. Tools that may, in the opinion of the engineer, cause jetting action, shall not be used in the cleaning operation. Cleaning out of the last 6 in. above the intended top of the test interval must be accomplished with a standard clean-out auger, or a clean-out auger with sludge barrel if necessary.

When the hole has been cleaned to the satisfaction of the engineer, the proper vane shall be attached to standard drill rods, which shall be properly and securely tightened. Ball bearing-type couplings shall be inserted at the joint between the vane and drill rod, at the first joint below the top of the casing, and at approximately 15-ft. intervals. The vane and drill rods shall be lowered slowly and carefully into the hole until the vane touches the bottom.

An adapter coupling shall then be placed over the drill rods and screwed down on the casing, which must be fixed so as to prevent any rotation. The vane shall then be pressed into the geotechnical material to a length determined by the engineer. The vane shall be forced into the soil in a smooth continuous stroke with no rotation and as little disturbance as possible. The torque head shall then be positioned on the adapter coupling and secured to the drill rods.

Under the direction of the engineer, the vane shear test shall then be conducted. Remolded vane shear tests shall also be conducted at the discretion of the engineer.

Two vane sizes, 2 $^{1}/_{2}$- and 3 $^{5}/_{8}$-in. OD, shall be supplied by the contractor. The contractor shall also supply to the engineer copies of the calibration curves for the torque heads and shall allow the engineer to make tests at the expense of the contractor to determine the accuracy of the calibrations of the torque heads and vane shear equipment.

B 2.10 Pressure Testing (Hydraulic)

Hydraulic pressure testing shall be interpreted to mean the operation of forcing water under pressure into subsurface rock formations through predrilled N-series test holes for the purpose of determining the drainage conditions and grouting requirements. The contractor shall perform all the work and furnish all equipment and supplies required to complete these operations.

Pressure testing equipment to be furnished by the contractor shall include the following: water pumps with minimum capacities of 50 gpm when operating at discharge (gauge) pressures of 150 psi; double expander packers for N-series test holes with rubber expansion elements 6 in. in length set 5 ft. apart; water pipes arranged so that water may be admitted either below the bottom expander or between the two expanders, and connected to the pressure pump through two swing check valves, water meter and pressure gauge. Supplies shall include all accessory valves, gauges, stopcocks, plugs, two sets of expanders, water for testing, standby pumps, fuel, pipes, pressure hose and tools necessary for maintaining uninterrupted tests for each boring to be tested.

Prior to testing each boring, the contractor shall test the apparatus on the ground surface by inserting and sealing it into N-series flush joint casing. A gauge pressure of 100 psi should then be maintained for 5 minutes with no indication of leakage. The contractor should exercise caution when lowering the apparatus into position so that the rubber packers are not damaged.

All pressure tests shall be made in the order and manner specified in this paragraph. The contractor shall pressure test each hole in 5-ft. sections, commencing at the bottom of the boring and progressing upward to the top of rock. For each lift, the maximum water pressure employed should be 1 psi per ft. of rock present above the top expander, but in no case shall the gauge pressure exceed 100 psi. The contractor shall develop the maximum pressure specified by the engineer in accordance with the above statements and, maintaining this pressure constant for a minimum period of 5 minutes, record the total volume of flow in gallons or cubic feet over this time interval. After completion of the above flow test, the pressure pump and flow into the boring shall be simultaneously cut off, and the time noted for each drop of 10 psi in pressure. These tests shall be repeated until the results are satisfactory to the engineer. These procedures shall apply to each 5 ft. lift tested. If the expanders are not adequately sealed against the rock or are in an area of broken rock, the leakage may be observed at the surface by the return of water, in which case the pressure test apparatus should be lowered 1 ft., and the test repeated.

Because of the significance of such tests on estimating surface leakage and on the ultimate grout treatment requirements of foundations, the contractor shall take every precaution to ensure that continuous and reliable pressure tests are completed as specified. If, in the opinion of the engineer, either the condition of the testing equipment or its assembly and arrangement are faulty, the contractor may be required to make a series of check tests at his own expense.

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For each hole that is pressure tested, the contractor shall prepare and submit to the engineer a pressure testing log in addition to the normally prepared boring log. Separate log sheets shall be submitted for each boring. These logs shall indicate the type of pump used, boring number, top and bottom depths below the ground surface of each interval tested, pressure employed in each interval, rate of water injection, time interval over which different pressure ranges were obtained, height of the water swivel above the ground surface and any other observations pertinent to subsequent grouting requirement of foundation treatments specified in the preceding paragraphs.

B 2.11 Special Installations

Work of this nature includes such operations as installing standpipe piezometers, slope indicators, settlement observation points, etc. In addition, the contractor will be required to have available all the equipment, tools, pumps, etc., that are normally required to execute a subsurface investigation as described in the Appendix, Section A 1, Soil Investigation, of *AISE* Technical Report No. 13. When the Instructions to Bidders, Contract Agreement, or both contain requirements for special installations, the contractor shall be responsible for supplying special tools and materials necessary to properly make the installation.

B 2.12 Piezometers

The following procedures shall be followed in installing a piezometer:

(1) The boring shall be drilled to the depth directed by the engineer in the manner outlined in Sections B 2.1 and B 2.5.
(2) Upon completion of the boring, clean water shall be circulated until the overflow is clear and free of particles.
(3) Where the bottom of the piezometer is higher than the bottom of the boring, the lower portion of the boring shall be sealed by pumping or tremieing a cement/sand mixture through a pipe placed at the bottom of the boring. The cement/sand grout shall be allowed to set a minimum of 18 hours.
(4) If a piezometer is to be placed at the bottom of the completed boring, Item (3) may be omitted.
(5) Standard 1 $^1/_4$-in. wellpoint, 30 in. in length, attached to $^3/_4$-in. ID galvanized pipe shall be inserted to the depth selected by the engineer.
(6) As the casing is being withdrawn, the annular space between the wall of the boring and the wellpoint shall then be filled with shot gravel as directed by the engineer, to a point at least 4 ft. above the bottom of the wellpoint. The contractor shall exercise caution in the extraction of the casing to maintain gravel within the casing at all times. However, an excessive height of gravel within the casing will bind against the pipe and wellpoint, lifting it with the casing.
(7) A 6-in. layer of clean sand shall be placed on top of the gravel zone. The boring above this point shall then be filled with a 3-ft. plug of bentonite ball tamped in 6-in. layers and shall be filled to the height directed by the engineer with a bentonite slurry as the casing is withdrawn.
(8) All piezometers shall extend not less than 2 ft. above the ground surface and in all cases shall be of sufficient length to prevent overflow of ground water.
(9) The engineer shall inspect and approve each section of pipe before its installation. Pipe joint sealer shall be used on all joints.
(10) The top of each piezometer shall be provided with a threaded galvanized cap in which an air hole has been drilled.

B 2.13 Records and Reports

The contractor shall keep a continuous field record of the operation of each boring. The record shall consist of an accurate log and description of the materials encountered, a record of samples and rock cores obtained, and a record of the samplers, driving weights and casing used. One copy of the field record shall be made available to the engineer at the completion of each day's work. The following data shall be included in these records:

(1) Dates and times of beginning and completion of work.
(2) Identifying number and location of test boring.
(3) Ground surface elevation at the boring.
(4) Diameter and description of casing.
(5) Total length of each size of casing.

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(6) Length of casing extending below ground surface at the completion of the boring.
(7) Weight, number of blows and drop of hammer used to drive the casing each successive foot.
(8) Depth of ground water table and other water levels as required in Section B 2.7.
(9) Depth of the bottom of sampler at start of driving or pressing for each sample.
(10) Depth to which sampler was driven or pressed.
(11) Weight and drop of hammer used to drive the sampler and number of blows required to drive it each 6 in. for a total depth of 18 in. or otherwise as described in Section B 2.2.
(12) Methods or forces used to press sampler tube when not driven.
(13) Depth at top of undisturbed sample.
(14) Length of sample obtained.
(15) Distance from the bottom of the sampler to the lower end of the sample when the sampler is not filled to the bottom and any other circumstances of obtaining the sample.
(16) Stratum represented by the sample.
(17) Depth of vane, applied torque and angle of rotation at shear failure.
(18) Any sudden dropping of drill rods or other abnormal behavior.
(19) Depth of top and bottom of individual core drill runs.
(20) Percentage of rock core recovery.
(21) Description of rock recovered.
(22) Thickness of each rock stratum.
(23) Depth of rock fractures and cavities.
(24) Loss or gain of drill water or sudden artesian pressure.
(25) Name of drilling rig operator.

The contractor shall submit daily time and material records to the engineer, showing the hours worked by each drill rig on a rental basis. These records shall indicate the driller's name for each rig, regular time and overtime, if any, and all unit price materials used. The records shall be signed daily by the contractor's representative and the engineer except where continuous presence of the engineer is not required. One copy shall be made available to the engineer at the completion of each day's work.

B 2.14 Packing, Protecting and Shipping of Soil or Rock Samples

All samples shall be properly labeled and packed in suitable containers to protect against damage from shifting of samples in boxes or breakage of glass jars or otherwise while in transit. All undisturbed samples shall be protected in every possible way to avoid disturbance of the samples during shipment and shall be stored and shipped in an upright position. All samples shall be protected from excessive heat or freezing. All samples shall be carefully packed to prevent freezing or damage during storage or shipment. Samples shall be properly marked as 'Fragile' and 'Keep Away from Heat or Freezing.' All samples shall be shipped to a laboratory as indicated in the Instructions to Bidders or as directed by the engineer.

B 2.15 Definition of Pay Quantities

The amount of work to be paid for shall be as agreed in the contract. No payment shall be made for frozen or damaged samples, regardless of the cause. Payment shall be made as follows, unless otherwise stated in the Instruction to Bidders or Contract Agreement.

B 2.15.1. For moving equipment, tools and supplies to and from the job, and between borings, for any required rentals and anticipated expendable materials, payment will be made in lump sum as stated in the contract, unless otherwise stated in the Instructions to Bidders or Contract Agreement.

B 2.15.2. For 4-in. minimum diameter soil borings as described in Sections B 2.1. and B 2.2, including record keeping and the recovery of split barrel soil samples but excluding the recovery of undisturbed soil samples, payment will be made at the unit price per foot as stated in the contract for the actual lineal feet of boring made and accepted by the engineer. Measurement shall be made from the surface of the ground to the bottom of the soil boring or to the depth at which rock was encountered as determined by the engineer.

B 2.15.3. Test borings situated in bodies of water of such depth and area as to require the use of ramps or floating platforms shall be paid for as Soil Boring (Water) and Rock Drilling and Coring (Water), and such items will

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be tabulated with their unit prices in the contract. No payment shall be made for the lineal feet of water penetrated.

B 2.15.4. For 3-in.-diameter undisturbed samples as described in Section B 2.3, payment will be made, in addition to payment for the 4-in.-diameter soil boring, for each sample successfully recovered, at the unit price per sample stated in the contract. Such price shall include the cost of the tube, the sealing, protection and shipment to the required destination.

B 2.15.5. For core drilling in rock, as described in Section B 2.5, including the recovery of cores as specified, payment will be made at the unit price per foot stated in the contract for the actual lineal feet of hole cored and accepted by the engineer, measured from the depth at which rock runs encounter the bottom of the boring, as determined by the engineer.

Fragments of rock, boulders and extremely compact formations which are less than 1 ft. in thickness shall not be considered rock, and payment for such footage will be made at the contract unit price for soil boring, irrespective of the method of penetration, unless the amount of core drilling required exceed 10% of total depth of soil boring.

B 2.15.6. For auger boring and driving of sounding devices as described in Section B 2.8, payment will be made at the unit price per foot stated in the contract for the actual lineal feet of boring made and accepted by the engineer. If samples are required, it will be indicated in the Instructions to Bidders or Contract Agreement and shall be reflected on the unit price stated in the contract.

B 2.15.7. For vane shear tests as described in Section B 2.9, payment will be made at the unit price per hour stated in the contract for those tests accepted by the engineer. Payment will be made for the total time elapsed while the vane is within the boring, excluding any time lapse due to equipment failure or other conditions causing an interruption of the test.

B 2.15.8. For pressure testing, as described in Section B 2.10, payment will be made at the unit price per hour stated in the contract for those tests accepted by the engineer. Payment will be made for the total time elapsed from the beginning of the first test at the bottom of the boring until the completion of the final test at the rock surface, excluding any time lapse due to equipment failure or other conditions causing an interruption of continuous testing.

B 2.15.9. Payment under Section B 2.11 shall be in accordance with the unit price schedule agreed upon prior to the execution of the work.

B 2.15.10. No payment will be made for lost tools, drill rods, bits, etc. No payment will be made for casing left in place unless it has been left at the specific request of the engineer.

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Appendix C
Recommended Practice for Inspecting and Upgrading of Existing Structures

C 1.0 Purpose

The purpose of this appendix is to outline reasons why an inspection and/or upgrading of existing structures may be desired and to define methods and procedures to use in accomplishing these objectives.

C 2.0 Reasons for Performing an Inspection or for Upgrading an Existing Structure

- The desire to continue using the structure for the original design purpose for an extended service life.
- An increase in production, which will result in the increased usage of material handling systems such as the increased utilization of existing cranes, the installation of additional cranes, the addition of jib cranes or other equipment, etc.
- The modification of an existing process or the installation of a completely new production facility that would require the upgrading of the capacity of existing cranes and/or equipment and/or the installation of additional or greater capacity cranes and/or equipment.
- The introduction of heavier loading to existing floors or the installation of additional floors.
- Combinations of all the above.

Preliminary layouts may be required to establish the feasibility of the above within an existing facility to ensure that space, access and material handling requirements are satisfied.

C 3.0 Inspection

The owner of the structure should have a defined inspection program for the mill building. The frequency of inspection and the items to be inspected shall be defined within the program. The frequency and inspection items depend upon the age, history and duty cycle for the given structure. The results of these inspections should be documented.

C 3.1 Inspection Plan

An inspection plan should be developed based on the following:

- Site visitation.
- A review of existing drawings and design calculations, if available.
- Interviews with plant personnel in the operations, maintenance and engineering departments.
- A review of the applicable local, state or federal codes.

C 3.2 Reference Specifications and Codes

The following specifications, codes and publications are listed for reference. Utilize the latest version unless otherwise noted.

AASHTO	Standard Specification for Highway Bridges
OSHA	Occupational Safety and Health Administration
AISC	Manual of Steel Construction, 9th Edition, ASD
AISC	Load and Resistance Factor Design Specifications for Structural Steel Buildings
AISC	Iron and Steel Beams—1873–1952
AISI	Specification for Design of Cold-Formed Steel Structural Members
ACI	Building Code Requirements for Structural Concrete
PCA	Portland Cement Association Publications
AISI	Designing Structures to Resist Earthquakes
ASME	Pressure Vessel Code

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ASCE	Minimum Design Loads for Buildings and Other Structures
AWS	Structural Welding Code D1.1
AREMA	Manual for Railway Engineering Maintenance-of-way Association (Fixed Properties)
MBMA	Recommended Design Practices Manual
NAAMM	Grating
SSPC	Steel Structures Painting Manual, Good Practices

C 3.3 Field Inspection

The field inspection should include, but not be limited to, any of the following:

- A visual inspection of the overall condition of the various components of the structure, noting such items as cracking, missing connectors, reduction of area, corrosion, wear and physical damage due to collision, abuse, fire, etc.
- A visual inspection of connections between various components of the structure. This would include items such as welds, bolts, rivets, anchor rods and base plates.
- A notation of any field alterations not recorded on design drawings, such as removal or addition of material, reinforcing, repairs, welded attachments and items removed for clearance.
- Results of the above items may indicate the need for a survey to determine the existence of any alignment, deflection or settlement conditions.

C 3.4 Analysis

The results of the field inspection will furnish the basis for an analysis of the structure to determine what action should be taken to:

- Restore the structure to its original condition, considering the present condition of components that have been subjected to corrosion, impact, fatigue, thermal loads, deterioration, vibration or other damage.
- Evaluate the effects of present and/or proposed loads on the structure. This may require soils testing and/or foundation review.
- Ensure an adequate projected fatigue life.
- Determine actual and allowable stresses, which should be based on engineering judgment.

C 3.5 Reports

The results of the field inspection and the analysis should summarize the following:

- Scope.
- Field conditions.
- History of performance and maintenance of the structure.
- Analysis and limits of the structure.
- Appraisal of alterations, repairs, additions and costs.
- Estimate of engineering effort, number of drawings and specifications, including a schedule indicating duration of the various phases of the project.
- Cost comparison of upgrading versus a new structure.
- Recommendations on future inspections.

C 4.0 Upgrading

C 4.1 Design Parameters

C 4.1.1 General Considerations

- Grade of steel (A7, A36, etc.) used in the original or previously reinforced or modified construction. Samples of steel may be required for analysis to aid in this determination.
- The current crane loads and crane operations as compared to the current code requirements.

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- The dead and live loads used in the original design compared with the present or proposed dead and live loads.
- The allowable stresses permitted at the time of the original design as compared to present-day allowable stresses in steel, concrete, soil and piling.

For information concerning discontinued rolled materials such as I beams, angles, channels and wide flange beams, reference should be made to earlier editions of AISC publications "Manuals of Steel Construction" or "Iron and Steel Beams—1873–1952".

Painting an upgraded structure is a matter of preference and economics. Procedures and methods are well documented in Volumes I and II by the Steel Structures Painting Council.

C 4.1.2 Design Calculations. Complete design calculations shall be prepared either as a supplement to existing calculations or new ones in their entirety if none presently exist.

C 4.1.3 Design Drawings. In the absence of existing reference drawings, all field measurements shall be taken so that new drawings can be produced depicting the existing construction and conditions. The exposure of foundations may be required to determine their type, dimensions and condition, and soils sampling may also be required, especially where drawings and test hole data are unavailable.

C 4.1.4 Loading Recommendation. It is recognized that some of the recommended crane runway loadings in Section 3.4 of Technical Report No. 13 may be conservative. This is appropriate for new mill building design to ensure maximum serviceability consistent with economic considerations. However, engineering judgment should be applied when setting the analysis criteria of an existing structure relative to current requirements, loadings and design methods, without sacrificing present standards of safety.

C 4.2 Reinforcement and Replacement

Consideration shall be given to the replacement or reinforcement of worn, corroded, damaged or deformed material.

C 4.2.1 Removal. When structures or parts of structures are removed, the effect on remaining structures should be investigated.

C 4.2.2 Reinforcing Structural Members. When structural members require reinforcing, consideration shall be given to the following:

- The amount of dead load stress in the original material. This stress will not be shared by the reinforcing material unless external support is provided during the reinforcing process to remove the dead load stress from the member being reinforced.
- The stress in the reinforced member shall be determined for the original and reinforcing material (where they are of different grades of material), with due regard for the actual and allowable stress in both types of material.

C 4.2.3 Welding. When existing members are reinforced by additional material through welding, consideration shall be given to:

- The weldability of the existing material.
- The effect of welding on the fatigue life of the member.
- The effect of the heat from welding on the integrity of adjacent rivets.
- Transferring the entire stress by welding where there is doubt about the integrity of rivets.

C 4.2.4 Connections. The connection of a reinforced member to adjacent member(s) shall be investigated to ensure that there are adequate connections to properly transfer the stresses in the original and reinforcing material to the connecting member.

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