NCHRP REPORT 458

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Redundancy in Highway Bridge Substructures

TRANSPORTATION RESEARCH BOARD

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SUBJECT AREAS
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study to develop a methodology for considering substructure redundancy in the design and evaluation of highway bridges. The report builds on the work on bridge superstructure redundancy presented in *NCHRP Report 406* and integrates the findings in that report into a single recommended specification for addressing bridge redundancy during design. The material in this report will be of immediate interest to bridge design specification writers and to bridge engineers interested in the quantitative assessment of bridge redundancy.

The AASHTO *LRFD Bridge Design Specifications* require the consideration of redundancy while checking the strength of all bridge members including substructure elements. The requirements, however, in some cases, are subjective, and the specifications recognize the importance of continued research to provide improved quantification of redundancy in bridge structural systems.

NCHRP Project 12-47, "Redundancy in Highway Bridge Substructures," was initiated to extend the methodology developed in NCHRP Project 12-36, "Redundancy in Bridge Superstructures," to bridge substructures. The findings of this earlier study were presented in *NCHRP Report 406.*

The research was performed by Imbsen & Associates, Inc. of Sacramento, California, with consultants Michel Ghosn and Fred Moses, who are the authors of *NCHRP Report 406.* The current report fully documents the methodology used to develop and calibrate a process for quantifying redundancy in bridge substructures and for calculating appropriate adjustment factors to maintain a uniform level of structural reliability. An appendix to the report contains recommended language for introducing the quantification of redundancy for superstructures and substructures into the *AASHTO LRFD Bridge Design Specifications.*

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research project reported herein, sponsored by the Transportation Research Board under the NCHRP Project 12-47, was performed by Imbsen & Associates, Inc., of Sacramento, California.

Dr. Toorak Zokaie initiated the project while employed by Imbsen & Associates, Inc. His earlier efforts are gratefully acknowledged. Mr. Po Lam of Earth Mechanics, Inc., provided consultation on geotechnical engineering issues and performed the foundation stiffness evaluation for various foundation types and soil conditions. Mr. Robert Cassano assisted in reviewing the draft report and the recommended specifications.

The authors gratefully acknowledge the support received from Dr. Roy A. Imbsen as well as the entire staff of Imbsen & Associates, Inc., which enabled the project team to make the transition and complete the project as planned.

The **Transportation Research Board** is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's mission is to promote innovation and progress in transportation by stimulating and conducting research, facilitating the dissemination of information, and encouraging the implementation of research results. The Board's varied activities annually draw on approximately 4,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation.

The National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Bruce M. Alberts is president of the National Academy of Sciences.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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REDUNDANCY IN HIGHWAY BRIDGE SUBSTRUCTURES

SUMMARY

The goal of this project is to define rational measures of bridge substructure redundancy. These measures should provide a framework that may be incorporated into the design process to systematically account for system redundancy and the enhanced safety and reliability levels provided by the system as compared with those of the individual members. This goal is accomplished by introducing a tabulated set of system factors that may be used when designing and evaluating the safety of bridge substructures of common configurations, to determine the required component capacity for these substructures as a function of their degrees of redundancy. In this context, a bridge substructure is considered as safe if it provides a reasonable safety margin against ultimate failure (whether due to excessive component damage or overall system collapse) and it does not exhibit excessive displacements that render the bridge inadequate for use.

Redundancy of a bridge substructure is defined as the capability of the substructure system to continue to carry loads (vertical and lateral) after the failure of any of its components. Various non-natural, environmental, and natural hazards may cause the overloading or the damage of a substructure. These hazards include vessel/vehicle collisions, overweight trucks, winds, earthquakes, and scouring. Depending on the nature of the load and the structural details involved, the failure can be either ductile or brittle. The failure types have drastically different consequences on the bridge system behavior.

A structural system consists of many components that interact with each other and work together to resist externally applied loads. In a nonredundant system, the failure of any critical member (a weakest link) will result in the collapse of the system. In a redundant structural system, two or more components must fail before the structural system collapses. The degree of redundancy for a given substructure system may vary over a wide range.

The limit states considered to ensure adequate substructure redundancy and structural safety are as follows:

- **Ultimate Limit State**—This is defined as the ultimate capacity of the intact bridge substructure system (i.e., load-carrying capacity). Failure of a substructure may be due to excessive local deformation in the plastic hinge zone resulting in local component rupture or crushing, or the formation of a global, plastic system mechanism leading to incipient collapse. For ductile structures (e.g., substructures with confined

concrete columns), the deformation capacity of critical sections is sufficiently large that a plastic limit state mechanism can be formed, that is, global system collapse mechanism. For less ductile structures (e.g., substructures with unconfined concrete columns), the component deformation capacity is limited and local component crushing/fracture will occur often leading to the unloading of the system.

- **Functionality Limit State**—This is defined as a maximum *total* lateral displacement at top of the substructure bent reaching a value of H/50, where H is the average clear column height of the bent. This may be caused by the combination of large column drift and excessive deformation in the foundation/soil system. The maximum lateral displacement is set at a level so that, in addition to structural deformation, it also captures the possible failure in the soil and foundations.
- **Damaged Condition Ultimate Capacity**—This is defined as the ultimate capacity of a substructure subjected to major damage such as the loss of a major component. The loss may be due to a brittle failure of a column because of a collision, the failure of a connection or a joint, or the partial washing away of the foundation due to scour. The system factor tables provided in this study do not consider damage scenarios for bridge substructures as it was found that typical bent cap designs do not have the capacity to transfer the load resulting from the complete failure of one column. However, for specific cases, the direct analysis procedure (described in Section 2.6 and Section 3.9) can be used to evaluate the redundancy and safety of the bridge in damaged condition accounting for the limited capacity of the cap members.

PIERPUSH LATERAL STRENGTH EVALUATION

The first challenge in quantifying substructure redundancy is the ability to predict the ultimate strength of the system as a whole. This prediction is accomplished by performing a nonlinear analysis under incrementally applied loads and by monitoring the resulting structural response closely. As the load level increases, plastic actions develop at critical sections and the structural behavior becomes progressively more nonlinear. The analytical model must also properly account for the effect of foundation/soil conditions that have been shown to have a significant effect on the expected structural response and the system's reserve strength. The analysis should monitor the occurrence of important nonlinear events that include progressive yielding at various critical sections. The *event-to-event response monitoring* would allow the tracing of the entire nonlinear force-displacement relationship from initial yielding to ultimate collapse. In between, various nonlinear events may occur that include the formation of plastic hinges at various locations; accumulation of local deformations (e.g., plastic rotations and maximum strains); shearing failures; and reduction of lateral load (moment) capacity due to P-Δ effects in the columns.

The analytical procedure starts with the evaluation of the section strength and deformation capacity. Based on these results, a nonlinear beam-column model is developed accounting for the interaction of column axial force and bending moment (P-M). This information is implemented in the PIERPUSH program using a lumped-plasticity formulation. Since the column P-M interaction surface defines the yield surface, this formulation automatically accounts for the effect of varying column axial loads as the lateral loads are gradually incremented. An illustrative example is included in Section 2.7 that highlights the features of the analysis procedure.

MEASURES OF REDUNDANCY

Based on the past performance of representative bridge substructures, a target redundancy level is established. This corresponds to the redundancy level observed in average

four-column bents with unconfined concrete columns. These bents are most typical for regions outside earthquake prone areas. The safety associated with this target redundancy level is considered as the minimum required for substructures of all configurations.

Three measures of redundancy are defined in this investigation. In accordance with system reliability theory, the most rigorous definition for redundancy is the *enhanced safety level* provided by the system compared to the safety of the individual components. This enhanced safety may be represented in terms of the *relative reliability index*, $\Delta\beta$, defined as:

$$\Delta\beta = \beta_{system} - \beta_{member}$$

where β_{system} is the reliability index of the structural system and β_{member} is the component reliability index used in conventional LRFD specifications. The most intuitive and deterministic definition for redundancy is the *enhanced system strength* beyond that which leads to first component failure. This is defined in terms of the *system reserve ratios*, R_u, R_f, and R_d, for the ultimate, functionality and damaged condition limit states, respectively:

$$R_u = LF_u/LF_1 \qquad R_f = LF_f/LF_1 \qquad R_d = LF_d/LF_1$$

where LF_u is the load factor on the lateral load effect that causes the collapse of the system and LF_1 is the load factor that causes the failure of the first member of the intact substructure. LF_f is the load factor that causes the functionality limit state of the initially intact substructure to be exceeded. LF_d is the load factor that causes the collapse of a damaged substructure.

For codified design implementation, a *system factor* ϕ_s may be introduced in the design check equation to account for system redundancy during the design of bridge substructure components. By introducing the system factor into the design equation of the current AASHTO Load and Resistance Factor Design (LRFD) Specifications, the system's redundancy and safety levels are accounted for implicitly while the reliability formulation remains transparent to the end user. In addition, if the bridge substructure's configuration is of the common type for which system factor tables are provided, a nonlinear analysis need not be performed during the design process. Otherwise, a nonlinear pushover analysis is required as part of the direct redundancy analysis procedure.

NCHRP Report 458 demonstrates that the three redundancy measures defined in this study are closely related. It is also determined that for a bridge substructure to be adequately redundant it should satisfy a target redundancy level that produces a relative reliability index $\Delta\beta_{target}$ of 0.5. This means that the complete system should have a reliability index equal to 0.50 higher than that of a member. (The current LRFD Specifications were calibrated to produce $\beta_{target\ member} = 3.5$).

The system reserve ratio R_u that corresponds to a $\Delta\beta_{target} = 0.5$ is $R_u = 1.20$. In other words, to satisfy the target redundancy level, the load that causes the system to collapse must be 20 percent higher than the load that causes the first component failure assuming that the component is designed to just satisfy the design equation. Similarly, a system reserve ratio $R_f = 1.20$ is set as the target for the functionality limit state and a system reserve ratio $R_d = 0.50$ is set for the damaged condition limit state. The latter target indicates that an adequately redundant system should be able to carry some load even after one of its members has totally failed. The load that a damaged system should be able to carry must be at least 50 percent of the load that causes the failure of the first member in an intact structure. Because most current bent systems cannot satisfy this damaged condition criterion, it has been decided to limit the check of the damaged condition to only bridges that are classified as critical.

Based on the target system reliability level selected, a set of system factors (ϕ_s) are developed for typical two-column bents and four-column bents (representative of typical multicolumn bents) for both ultimate and functionality limit states. Various foundation/soil conditions and structural parameter variations are included in the tables. These recommended system factors are within a minimum of 0.8 and a maximum of 1.20. The system factor ϕ_s for a system that satisfies the target redundancy level is 1.0. The system factor is less than 1.0 for nonredundant systems and is greater than 1.0 for systems with higher levels of redundancy.

The system factor is in essence a penalty-reward factor. For nonredundant system, the component design strength should be increased proportionally to $1/\phi_s$. It should be noted that this increased component design strength will not make a nonredundant system redundant. *It merely improves the system safety level to be compatible with that of an adequately redundant system, that is, meeting the target system reliability.* For a redundant system, the component design strength can be relaxed accordingly.

Single-column bents and pier walls are considered as nonredundant and a system factor (ϕ_s) of 0.8 should be used unless the direct analysis procedure is applied in the evaluation. Also, if the component shear failure controls or a joint failure occurs, the redistribution of the loads to other members within the system will not occur. For these cases, a system factor $\phi_s = 0.8$ is recommended.

IMPLEMENTATION

The system factors provided can be directly used during the design of substructure components to account for the level of system redundancy. For structural configurations not addressed in the tables, a direct analysis procedure is described to perform the redundancy evaluation of the system. Both approaches can be used for new designs as well as for the safety evaluation of existing structures. For existing structures, it is important to point out that bridges typically may have some overstrength beyond the required design value. On the other hand, existing bridges may suffer from deterioration such that the original *provided* strength is reduced. (It may still be greater than the *required* design strength.) This difference between the as-provided strength and the required strength (whether lower or higher) should be recognized and accounted for during the check of redundancy.

The assumptions made in this investigation are described, in detail, in Section 2.5. One of the most critical assumptions is the behavior of beam-column joints and column-footing joints. It is assumed that the joints are strong enough so that nonlinear deformation will occur in the columns. For new designs particularly in the high-level seismic zones, there is a minimum requirement that the joints must be designed to behave elastically. For regions outside earthquake-prone areas, this may not be true even for new designs as the nominal lateral load could be quite low. In these situations, it is important to perform a thorough strength evaluation of the joints before assigning the system factors to the structural components of the bridge substructure. The evaluation of the joint can be done using the provisions provided in the current LRFD Specifications.

This project has established a solid framework for the redundancy evaluation of bridge substructures. Based on assumptions made and the practical limitations of this project, future research areas are identified to complement the current research. Any new information that may be made available in the future can be readily incorporated into the framework proposed in this report.

CHAPTER **1**

INTRODUCTION AND RESEARCH APPROACH

The objective of this research project is to develop a rational method to include substructure redundancy during the design and the safety evaluation of highway bridge substructures. The proposed method should be applicable for inclusion in the AASHTO Load and Resistance Factor Bridge (LRFD) Design Specifications. The scope of the study and the research approach are presented in this Chapter.

1.1 PROBLEM STATEMENT AND RESEARCH OBJECTIVES

Redundancy is an important structural characteristic recognized in most design applications as desirable and even necessary. In general, redundancy is defined as the ability of a structural system, particularly a bridge system, to sustain damage without collapsing. The AASHTO's LRFD Specifications define collapse as *a major change in the geometry of a bridge rendering it unfit for use*. These descriptive statements illustrate the subjective nature currently associated with the concept of redundancy. For example, questions that remain unanswered include what type of damage and overload levels should the structure be able to sustain without collapsing, and what major geometric changes would render a bridge unfit for use?

According to current engineering practice, redundancy should provide a structure with adequate *alternative load paths* in the case of excessive live loads or major component failures. Three types of redundancy are defined as follows:

- *Internal redundancy*, which means that the failure of one element will not result in the failure of the other elements of the member. For example, cracks that develop in one element do not spread to other elements.
- *Structural redundancy*, which refers to the redundancy that exists as a result of the continuity within the load path. Any statically indeterminate structure such as continuous beams and rigid frames would belong to this type. The Standard Specifications for Highway Bridges (AASHTO, 1996) usually does not assume structural redundancy to be sufficient. For example, even though a continuous two-span two-girder bridge is structurally indeterminate, the standard AASHTO criteria would technically classify it as nonredundant. However, if each critical section of a statically indeterminate system has sufficient ductility capacity against sudden rupture, the system would provide reserve strength allowing it to carry loads beyond the formation of the first plastic hinge.
- *Load path redundancy*, as defined by AASHTO Specifications, refers to the number of supporting elements. A structure is nonredundant if it has only one or two load paths. For example, a bridge superstructure composed of only one or two parallel girders is regarded as nonredundant. Failure of one girder of a system with one or two load paths is assumed to result in the collapse of the span, hence, the bridge is considered to be nonredundant.

The main difficulty in implementing concepts of redundancy in engineering practice is the lack of simple measures that designers can use to verify the adequacy of their designs. The 1996 AASHTO Standard Specifications for Highway Bridges requires the consideration of redundancy when designing steel bridge members. Section 10.3 of the AASHTO manual requires different allowable fatigue stress ranges for "*redundant*" and "*non-redundant load path*" structures. By AASHTO's definition, "*a component may be considered a non-redundant load path member when the failure of a single element could cause collapse.*" In addition, the Standard Specifications consider the redundancy of substructures under the seismic design provisions for lateral loads. The Specifications recommend different "response modification factors" depending on the type of the substructure and the number of columns in a bent.

The AASHTO LRFD Bridge Design Specifications outline a format explaining how redundancy and other parameters related to global response can be included in the design process using a "*load factor modifier,*" η_R, that relates to the redundancy of the structure. The approach and format are valid for the design of both superstructures and substructures. However, the value of η_R, was determined by judgment rather than through a calibration process.

Superstructure redundancy was investigated in NCHRP Project 12-36 as reported in *NCHRP Report 406* (Ghosn and Moses, 1998). The study proposed definitions of redundancy based on current acceptable practices and provided quantitative measures of redundancy based on superstructure geometry and *member ductility*. The limit states considered in the context of bridge redundancy included *collapse* and *loss of function* due to overloads of intact bridges

as well as the consequences of potential damage to major load carrying components. It should be noted that the system redundancy is highly dependent on the ductility capacity of critical components. System factors were proposed to ensure uniform system performance for different bridge configurations, geometrical arrangements, and material and structure types. This is accomplished by calibrating system factors based on structural reliability theory.

The objective of this study is to develop a methodology for considering substructure redundancy during the design and evaluation of highway bridge substructures. The concepts described in *NCHRP Report 406* are expanded to substructures. The focus of this study is on bridge bents including foundation/soil systems and abutments. The final results of the research include a set of specifications to be incorporated into the AASHTO-LRFD Specifications for more rational consideration of substructure system redundancy. The specifications developed in this project include system redundancy factors for typical bridge substructures and also outline a procedure for calculating redundancy factors directly for unusual bridges.

1.2 SCOPE OF THE STUDY

Redundancy is a function of the structural behavior of the total system. In order to consider the redundancy of a bridge, the overall system behavior and the interaction of the superstructure, substructure, and the foundation must be considered. However, because of the intricacy of the problem at hand and the large number of factors that influence the behavior of the complete system, it may be reasonable to divide the system into its subsystem components (superstructure-substructure-soil/foundation) and study each of these separately as a first step toward a comprehensive analysis of the complete system's redundancy.

As substructures and foundations are normally designed for vertical loads with relatively high safety factors, the lateral load is the most important load that affects substructure redundancy. This observation is further supported by reports from field inspections of many substructure failures. In most bridge designs, the superstructures load is transferred to the substructure through bearing supports. In these cases, the superstructure provides little resistance to lateral loads and the behavior of the substructure and superstructure systems, in most cases, may be studied independently, as long as the applied vertical load effects on the substructure, including axial forces and moments, are adequately accounted for. As an example, the behavior of the bridge bents supporting simple spans is effectively independent of the superstructure behavior. Coupling the behavior of the superstructures and substructures may not be valid when the two subsystems are integrally connected. Such bridge designs are widely observed on the West Coast, but are not common in other parts of the United States. For this reason, this research is focused on the behavior of typical bent types as subsystems uncoupled from the complete system.

The scope of the research project is set in accordance with the ultimate objective of the project, that is, developing practical design specifications for substructure redundancy of highway bridges. Redundancy factors are developed for typical bent types supported on typical foundations. These include pier walls, single-column bents, two-column bents, and multicolumn bents. Four-column bents are studied as the representative case for all multicolumn bents. Because redundancy is defined as the system's reserve strength beyond the first component damage, redundancy in single-column bents and pier wall-type substructures is very limited. Even for well-confined single columns and for well-proportioned structural pier walls, the reserve strength (i.e., strength redundancy) can only come from the strain-hardening effect, which is rather limited for typical designs. Therefore, single-column bents and pier walls are considered as nonredundant, and the focus of the report is on the redundancy evaluation of two-column bents and four-column bents. Similarly, since shearing failures and joint failures, such as bar pullout, are brittle and their occurrence results in the complete unloading of the system, bents subjected to such failures are considered to be nonredundant. Therefore, the emphasis in this study is on the failure of the main piers in bending. The ability of the substructure system to carry some load after a brittle failure such as that expected from shearing of a column or a joint failure is considered in this study when discussing "substructure in damaged condition."

Initially, bridge abutments were to be included in this study. However, because of the limited number of abutment configurations used in practice and the fact that most structural failures have occurred in bents, it has been decided to limit this study to the behavior of bents.

Multispan bridges are formed by several bents, and lateral loads may transfer from bent to bent through the superstructure. As mentioned above, the ability of the superstructure to transfer such loads depends on the type of substructure-superstructure connection used. But, as the connections in current design practices are designed mostly for vertical load considerations, their ability to transfer lateral loads from bent to bent is limited. This ability is not true for the behavior of bridges under longitudinal loads. In this case, each bent is essentially acting as a single column configuration except for the cases where the superstructure is monolithic with the substructure. But, as previously discussed, monolithic constructions are still not common and will not be addresses in this study. For most cases involving the application of transverse loads, the effect of superstructures on the redundancy of substructures is small.

Based on these considerations, a more focused scope of work has been followed that results in a practical procedure for considering substructure redundancy in typical bents. This procedure consists of proposing a set of system factors that can be used during the design of typical substructure configurations. In addition, a direct redundancy analysis procedure

is developed so that engineers can directly calculate system redundancy factors for the substructures. This procedure consists of the nonlinear analysis of the bridge substructure and the identification of the limit state loads for each strength, functionality, and damage limit states. Appropriate values of system factors can then be extracted from the results of the nonlinear analysis to satisfy a set of predetermined target system capacity levels.

1.3 RESEARCH APPROACH

In this study, redundancy is defined as a measure of the capability of the bridge substructure to carry loads beyond the elastic capacity of individual members or after the damage of one main load-carrying member. Member capacity in this case is defined as that determined using the AASHTO-LRFD Specifications. The following tasks are carried out to develop a framework for including redundancy in the design and evaluation of bridge substructures:

1. Identification of typical bridge substructure configurations including the foundation types that should be addressed,
2. Determination of the loading conditions that should be used in the safety assessment of bridge substructures,
3. Development of analytical models to study the actual nonlinear behavior of bridges with different substructure configurations and foundation types,
4. Definition of appropriate limit states that quantify the safety and functionality of the bridge substructure systems,
5. Implementation of reliability models to account for the uncertainties associated with determining the response of intact and damaged bridge substructures for a variety of loading conditions and damage scenarios,
6. Calibration of system factors that can be used as measures of bridge substructure redundancy and that also can be used to account for bridge redundancy during the design and load capacity evaluation of typical bridge substructure configurations,
7. Presentation of the factors developed in Step 6 in a specifications format compatible with the AASHTO LRFD Specifications, and
8. Development of a step-by-step procedure for computing substructure redundancy directly.

The research approach is summarized in the following paragraphs.

Identification of Substructure Types—A survey of the AASHTO member agencies was conducted to identify the most common substructure types. This survey indicated that bridge substructures may be classified in various structural and material types. The following types have been identified:

- Four typical bent types: pierwalls, single-column bents, two-column bents, and multicolumn bents.
- Three foundation types: spread footing, pile footing, and pile extension (or shaft).
- Three soil conditions: shallow (rock), deep sand, and deep clay.
- Three superstructure-to-substructure connection types: monolithic, continuous on bearings, and simply supported spans.

Since the behavior of a substructure is affected by its relative flexibility, the foundation types and supporting soil types were grouped into eight categories in accordance with the resulting foundation flexibility. These categories are: (1) spread footings on normal soils, (2) spread footings on stiff soils, (3) drilled shafts in soft soils, (4) drilled shafts in normal soils, (5) drilled shafts in stiff soils, (6) piles in soft soils, (7) piles in normal soils, and (8) piles in stiff soils.

Typical ranges for substructure geometric parameters (such as column height, column width, and thickness) and material properties (such as concrete strength, steel strength, and reinforcement ratios) are identified from the survey results. Parametric analyses are performed to identify the effect of size and geometric variations on bridge behavior. Each bridge model is analyzed and the results are used to calculate appropriate system redundancy factors.

Substructure Safety and Loading Conditions—This step is required to establish an understanding of the inherent safety of substructures and the most likely causes for substructure failures. The necessary structural configurations and associated loading conditions (including the sequence of load applications) are thus identified. As substructures are typically designed with a relatively high safety factor for normal (vertical) loading, experience has shown that most damage and failures are caused by lateral loads associated with extreme events (e.g., earthquake, scour, and vessel collisions). Experience has also shown that, except for the California-style monolithic box-girder superstructure, the effect of superstructure on substructure lateral redundancy is small. The only exception is the longitudinal response of the bridge frame. Based on these considerations, the study focuses on the behavior of intact bridge substructures under lateral load and damaged substructures. The damage scenario considered is the brittle failure of one column potentially caused by scouring or vessel collision.

Each bent model should be analyzed by applying incremental horizontal and vertical loads and studying the post-elastic behavior of the substructure. Because it is unlikely that the vertical load (live load) and horizontal loads will reach their maximum possible values simultaneously and since substructure redundancy is primarily controlled by the lateral load that is associated with the highest degree of uncertainty and is the most likely load to cause failure, the application of vertical loads from that of the horizontal loads is decoupled. In this context, the nominal vertical loads are

applied on the substructure and kept constant while the horizontal loads are incrementally applied. This approach will provide a conservative estimate of the contributions of vertical loads to substructure safety.

Analytical Procedure—The analytical procedure consists of performing nonlinear analysis of substructure frame bents, considering the nonlinearity of the columns and the flexibility of the foundation/soil systems. The loading consists of a base load equal to the design load in both vertical and lateral directions. Then, the horizontal lateral load is incrementally applied until failure. The following conditions are monitored during the analysis process:

- First member failure, and subsequent yielding events;
- Development and accumulation of inelastic deformation (plastic rotation or extreme-fiber strain) at critical sections;
- Ultimate strength; and
- Ultimate functionality limit. (Excessive *total* bent displacement.)

To account for the potential damage to one column due to scour or other extreme events such as vessel collision, a revised bent model for a four-column bent is generated to simulate the damage scenario, and the above incremental lateral loading process is repeated.

As a result of these analyses, and by considering the statistical variation of the loads, material properties, and uncertainties in the analytical models, several sets of system redundancy factors are obtained for typical substructure configurations and representative foundation/soil systems. Design specifications are developed to guide the design engineers in considering the system factors during the design of bridge substructures.

The analytical models account for nonlinear column behaviors and P-delta effect. As done during routine engineering practice, nonlinear soil behavior is represented by equivalent linear springs for each category of foundation and soil support systems. Two options are evaluated for considering foundation properties: (1) to include the piles and soil stiffness properties directly in the model as distributed Winkler-type springs (linear or nonlinear); and (2) to resolve the foundation properties into a set of lumped springs at the base of each footing and simplifying the bent frame analysis. If done properly, both approaches should yield identical results. A number of computer programs have been evaluated for possible use in this study. The final decision has led to the selection of the program NEABS (Nonlinear Earthquake Analysis of Bridge Systems) (Penzien, Imbsen, and Liu, 1981) that has been used extensively in the last 25 years for bridge structures and foundations. The analytical procedure has previously been benchmarked by comparing to a number of analytical formulations including the rigorous finite element procedure as implemented in SASSI program for Seismic Analysis of Soil Structure Interaction effects (Liu et al., 1997a). In this project, a limited comparison of the results of NEABS to those from the Florida-Pier program further confirmed the validity of using NEABS for the purposes of this study.

Failure Criteria and Limit States—Recognizing that conventional bridge design emphasizes component safety, the key to defining system redundancy is the proper definition of substructure failure criteria. Under incrementally applied lateral loads, a series of nonlinear events will occur. Initial yielding of the steel reinforcement and even the partial yielding of the section may not constitute a major nonlinear damage event. Major damage occurs only when significant yielding is observed at a section resulting in a significantly reduced tangent stiffness of the component moment-curvature relationship. Subsequently, if the loading is continued beyond first member failure, the additional load will be redistributed to other less heavily loaded members if they exist (e.g., in multiple-column bents) Three possibilities exist regarding the ultimate limit state:

- If all components are ductile, a statically indeterminate system will eventually form a plastic mechanism—an incipient collapse state;
- If the columns are not well confined, and thus they do not ensure ductile behavior, or if the shear strength is not sufficient, critical sections may experience local material rupture because of either flexure or shear.
- As the lateral displacement increases, the negative stiffness caused by the column axial compression P-Δ effect will result in reduced lateral force capacity.

Additional consideration should be given to the functionality of the substructure under increasing lateral displacement, that is, at what stage does the displacement become too excessive to ensure the safety and the proper function of the system.

Reliability Modeling of Bridge Substructures—Once the typical substructures and the variability of the parameters are identified, the range of uncertainties in material, fabrication, and analysis are quantified based on available published results, and simplified reliability models are developed for the purpose of reliability calibrations.

Calibration—Once the analysis is complete and all load factors corresponding to different failure conditions (limit states) are determined; a reliability-based analysis is performed to calculate the system redundancy factors. This procedure considers the variability of loads, analysis, material properties, and geometric dimensions. Target reliability indices are determined to specify an “acceptable” level of reserve strength beyond the design strength. The target indices are chosen to reflect the level of safety currently experienced by bridge substructures that expert bridge engineers consider adequately safe. System factors are determined so

that all substructures designed by including these factors will provide the same target system reliability index. If a bridge bent does not have enough reserve strength, then higher member capacity will be required. If a bridge bent has more than required reserve strength, then some reduction in member design capacity may be allowed. This was accomplished by specifying a set of system factors (ϕ_s) that can be incorporated into the design equation.

The relationship among the system reserve ratio, relative reliability index, and system redundancy factor is developed. This relationship makes it feasible to perform direct redundancy evaluation of project-specific applications.

Development of the Specifications—The results of the above work is compiled into a set of specifications, which can be incorporated into the LRFD Specifications. These specifications include a set of system redundancy factors for typical bridge substructures of different bent types. The specifications also recommend the use of direct analysis procedure that can be followed to calculate the system redundancy factors for nontypical bridge configurations.

1.4 ORGANIZATION OF THIS REPORT

Chapter 2 of this report presents the overall formulation of the reliability-based redundancy and the codified procedure. Results of agency survey are used to develop the set of typical substructure configurations and foundation/soil systems that are analyzed in this study. Important limit states are identified for both ultimate and functionality considerations and for bridges in both intact and damaged conditions. A nonlinear lateral strength evaluation procedure is described and a guided tour of the analysis model is provided by following a detailed example to illustrate the nonlinear response monitoring process during the incremental analysis procedure and to highlight the interpretation of results. Extensive parametric analysis results obtained using the PIERPUSH program are presented. PIERPUSH contains the pre- and post-processor algorithms used in conjunction with NEABS. These PIERPUSH results are used to derive the system reserve ratios that provide deterministic measures of system redundancy. These results also highlight the important parameters to be considered in the development of specifications.

Chapter 3 presents in detail the reliability formulation for system redundancy considerations. A reliability-based redundancy measure is defined. The response surface approach using a perturbation technique is implemented to compute the system reliability indices for bridge substructures. These calculations used the extensive results presented in Chapter 2. Target values for redundancy and system safety are extracted from these analyses. These target values are expressed in terms of the relative reliability index, $\Delta\beta_u$, and the system reserve ratio, R_u. The relationship among the various redundancy measures (deterministic and probabilistic) is developed for use in calibration of the system factors that are appropriate for typical substructure configurations. A direct redundancy evaluation procedure is presented for project-specific applications. Finally, based on the calibration results, several sets of redundancy factors are recommended for implementation in the AASHTO LRFD Specifications.

Chapter 4 gives the conclusions of this study and presents several areas of future research needs. Appendix A includes the draft specifications. Results of PIERPUSH analyses for all parametric studies are included in Appendix B. Both lateral force and bent total displacement are summarized in tables. The nonlinear lateral force-displacement relationships obtained from PIERPUSH for all structure-foundation systems considered are included. Appendix C summarizes the results of the survey of state DOTs. Appendix D provides tabulated summaries of the nonlinear analysis results, the reliability calculations and system factors.

Note: Appendixes B through D as submitted by the research agency are not published herein. However, they are available for loan on request to NCHRP.

CHAPTER 2

FINDINGS—SUBSTRUCTURE REDUNDANCY

2.1 INTRODUCTION

The final result of the project will be a set of recommendations that can be used to revise and upgrade the AASHTO LRFD Specifications. The draft specifications are presented in Appendix A. The research approach has been summarized in Section 1.4. This chapter describes the methodology, analysis assumptions, and results of nonlinear analyses of various substructures. Detailed results for all nonlinear analysis are included in Appendix B. In particular, the system reserve ratios of the bridge substructure are presented in detail. These system reserve ratios form the basis of the reliability analysis that leads to the system factors specified in Appendix A. These system factors are intended for use during the design of bridge substructures to account for the effect of substructure redundancy. Results of reliability calibration of the system factors are presented in Chapter 3.

2.2 MEASURES OF REDUNDANCY

Redundancy of a bridge structure is defined as the ability of the structure to continue to carry load after the failure of one of its members. A *deterministic* interpretation of this redundancy is the system *reserve ratio* (sometimes known as strength reserve ratio) that represents the ultimate capacity of the structural system as compared to the capacity of the system to resist first component failure.

Let the lateral force corresponding to the first component damage be denoted as F_1 and the ultimate lateral force of the substructure system be denoted as F_u; then the reserved strength ratio of the system R_u is defined as:

$$R_u = F_u/F_1 \tag{2.1}$$

These lateral forces can also be expressed in terms of the nominal lateral load, W_n, and the associated load factors, namely,

$$F_1 = LF_1 * W_n; \qquad F_u = LF_u * W_n \tag{2.2}$$

Therefore,

$$R_u = LF_u/LF_1 \tag{2.3}$$

However, uncertainties exist in loads, substructure, and foundation properties (geometry, material, and fabrication), as well as in the analysis and design methods. In the spirit of the reliability-based Load and Resistance Factor Design (LRFD), these uncertainties must be accounted for systematically.

As the traditional LRFD methodology is based on the component reliability expressed by the member reliability index, Δ_{member}, the enhanced safety and reliability of the bridge system is denoted by the system reliability index, Δ_{system}. Therefore, a reliability-based redundancy measure is the *relative reliability index*, $\Delta\beta$, which represents the increased safety/reliability margin beyond the component failure as implemented in traditional LRFD codes:

$$\Delta\beta = \beta_{system} - \beta_{member} \tag{2.4}$$

Equation 2.4 is valid for complete bridge systems as well as subsystems (such as superstructures and substructures). Redundancy of bridge superstructures was addressed in *NCHRP Report 406* (Ghosn and Moses, 1998). This project focuses on bridge substructures following the same procedure. The Response Surface Method using the perturbation technique obtains the system reliability index required. By performing extensive parameter studies, the limit state failure function of the system can be constructed numerically as the response surface function. This is described in more detail in Chapter 3.

The basic approach followed can be summarized as follows: Knowing that a particular bridge substructure system is more redundant than the average population of bridge structures with adequate system safety levels, the design conservatism implied in the AASHTO design procedures may be relaxed somewhat. On the other hand, if a substructure is known to be less redundant than the target of the design code calibration process, the substructure system should be penalized by imposing more stringent requirements, for example, reduced resistance factors.

2.2.1 Current Code Format for Redundancy Implementation

The AASHTO LRFD Specifications outline a format explaining how redundancy and other parameters related to

global response can be included in the design process using *load factor modifiers*. In the LRFD format, these can be conveniently expressed as additional system factors either on the resistance side, ϕ_s, or as load factor modifiers on the load effect side of the equation, η:

$$\phi_s \, \phi \, R_n = \gamma_D D_n + \gamma_L L_n + \gamma_W W_n; \qquad (2.5.a)$$

or,

$$\phi \, R_n = \eta \, (\gamma_D D_n + \gamma_L L_n + \gamma_W W_n) \qquad (2.5.b)$$

Where ϕ_s is the system factor relating to the redundancy and ductility of the system; ϕ is the member resistance factor; R_n is the nominal resistance of the member; γ_D is the dead load factor; D_n is the nominal dead load; γ_L is the live load factor; L_n is the nominal live load including impact; γ_W is the lateral load factor (e.g., wind factor); W_n is the nominal lateral load (e.g., wind load); and η is the load factor modifier relating to the redundancy and ductility of the system.

The load factor modifier, η, specified in the current AASHTO LRFD Specifications is composed of three terms: The first term relates to the *redundancy* of the system, the second term relates to the *ductility*, and the third term relates to *operational importance*. In this study, it is proposed to use the format of Equation 2.5.a because redundancy relates to the capacity of the system and thus should be applied on the resistance side of the equation as is traditionally done in LRFD methods. Unlike the current load factor modifier, the proposed system factor consists of one term only, tabulated for different substructure configurations. Differences in member ductility levels are considered by using different tables for confined and unconfined column sections. Instead of explicitly using operational importance factors, different limit states are considered leaving the engineers and bridge owners the option of choosing the appropriate limit states depending on the characteristics and the location of the bridge as well as its operational importance. This approach is consistent with current trends to *develop performance-based design methods* in bridge engineering.

Before proceeding with the reliability calculations and the calibration of the system factors ϕ_s, one must identify the population of representative substructures and foundations. Based on the incremental nonlinear analyses performed for these representative substructures, a basic understanding of the nonlinear behavior and redundancy of typical substructure systems are developed. These analyses have been conducted not only for structures with average parameters, but also for a family of structures with a range of parameters. These range of bridge substructure parameters are identified from a survey of the substructure and foundation configurations provided by state DOT agencies.

Based on the understanding of the nonlinear behavior of the substructure systems, reliability calculations using the response surface method can be performed to establish the system reliability index, β_{system}, and the corresponding *relative reliability index*, $\Delta\beta$, for each substructure included in the database. The next step is to establish a target relative reliability index as a minimum level of safety required. This can be simply the average obtained from a population of substructures deemed to be redundant and that represent the performance of current design practices. This target index will serve as the guide in calibrating the required system factors, ϕ_s.

The researchers defined three measures of structural redundancy: system reserve ratio, R_u; relative reliability index, $\Delta\beta$; and the system redundancy factor, ϕ_s. It is important to note that these three redundancy measures are closely related as shown in Section 3.5. These relationships are crucial to the implementation of a codified design procedure or a direct redundancy evaluation.

The proposed *system factors* are presented in Chapter 3. They are developed to ensure uniform system performance for various bridge configurations, geometrical arrangements, materials, section types, and supporting foundation and soil conditions.

2.3 TYPICAL SUBSTRUCTURES

A survey of the AASHTO member agencies was conducted to identify the common substructure types. This survey revealed that typical bridge substructure systems can be classified under several categories:

- Four typical bent types: pier wall, single-column bent, two-column bent, and multicolumn bent.
- Three foundation types: spread footing, pile footing, and pile extension (or drilled shaft).
- Three soil conditions: shallow (rock), deep sand, and deep clay.
- Three superstructures to substructure conditions: monolithic, continuous on bearings, and simply supported spans.

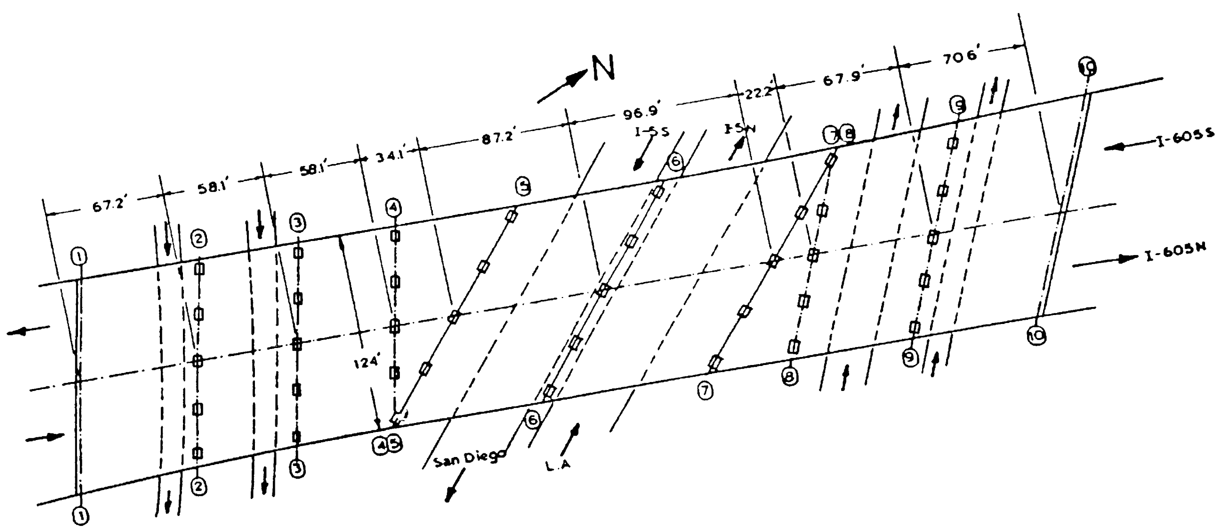
Since the net effects of foundation and soils on the substructure behavior is primarily attributed to the *relative flexibility*, the researchers grouped the foundation type and the supporting soil into eight categories in accordance with the resulting foundation flexibility. This is an important parameter for substructure redundancy as will be explained later in this chapter.

The effect of superstructure-to-substructure connection is considered less important except for the monolithic box girder superstructure. For typical girder-on-bearings, the ability to transfer horizontal loads between adjacent bents through the superstructure is rather limited. This vulnerability of bridge bearings has been vividly demonstrated in bridge damage following almost every earthquake. The damage often occurred

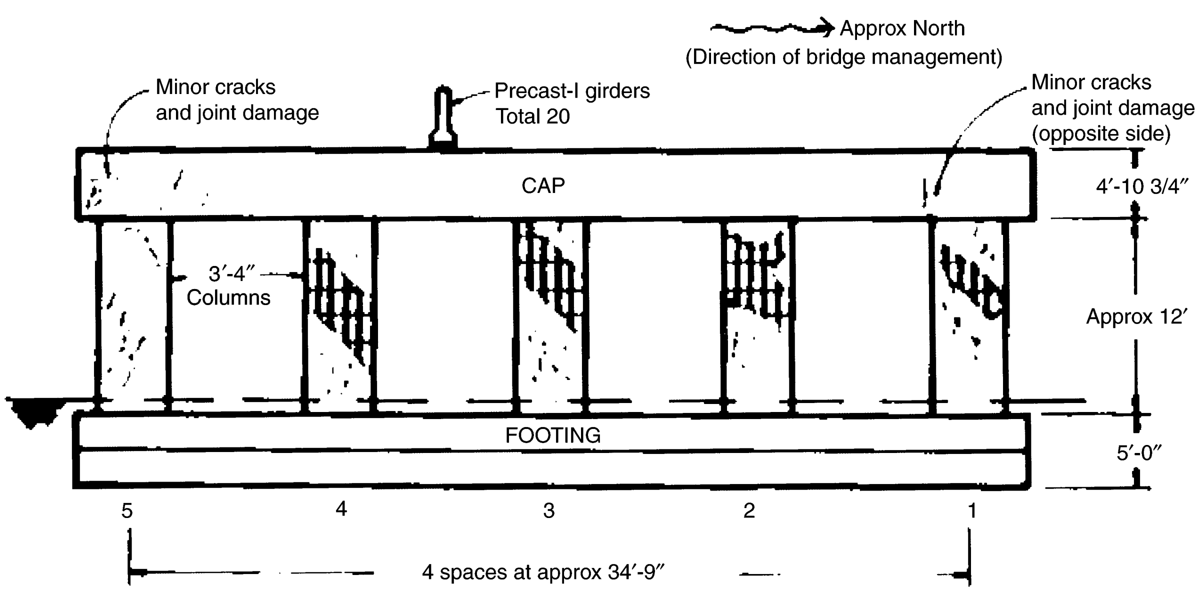
in the bearings because of their inability to transfer horizontal loads.

2.3.1 Example of Substructure Damage Under Lateral Loads

An example is the I5/605 separation structure that suffered substructure failure during the 1987 Whittier, California, earthquake. As shown in Figure 2.1, the main crossing consists of a two-span, simply supported concrete girder bridge. Each span has 20 girders and carries six lanes of traffic. The center pier is a five-column bent on a common pile-supported footing. The earthquake loading in the vicinity of the bridge was not very strong (about 0.2 g). However, because of a deficiency in column shear capacity, all five columns sheared off and the bridge almost collapsed onto Interstate Highway 5 below. The damage to both abutments was limited to bearings only. However, because of the damage done to bearings, the superstructure did not provide any load sharing to the central bent, that is, there was no redundancy. A post-earthquake correlation study using the NEABS program confirmed that the failure initiated at the columns, and the superstructure did not reduce the degree of damage incurred in the central pier substructure (Liu et al. 1990). Although the failure in the columns is due to shearing forces, the fact that the bearing



a) Plan View



b) Elevation of Central Bent and the Damage Pattern

Figure 2-1. I5/605 separation structure damaged during the 1987 Whittier earthquake.

did not allow for the transfer of load to the abutments demonstrates the ineffectiveness of the superstructure to increase the redundancy of substructure systems. Based on these and other similar observations, this study focused on the nonlinear behavior of individual bents without considering the contribution of the superstructure to system redundancy.

A notable exception to this observation is the concrete box girder superstructure that is monolithic with the substructure bents. This is a totally different class of bridges. Even for this situation, a basic understanding of the nonlinear behavior of substructure bents is fundamental to the evaluation of lateral load redundancy. Hence, the focus of this study is on the substructure system.

2.3.2 Typical Substructure Types

Based on the survey results, a total of 59 bridge substructure configurations were collected from 18 agencies. The average parameters of the four substructure types are summarized in Table 2-1. The following parameters are considered in the response sensitivity studies:

- Column height;
- Column width (size);
- Concrete strength, f_c';
- Steel yield strength, f_y;
- Longitudinal reinforcement ratio, ρ_{long}; and
- Transverse reinforcement ratio, $\rho_{transv.}$

For each of these parameters, average values and higher and lower values are established based on the survey results. This results in a total of 13 cases for each substructure type. For the two-column bent and four-column bent cases, these parameters are summarized in Tables 2-2 and 2-3, respectively. The average width of the bent cap is 14 m for single-column bent and two-column bents, and 15 m for four-column bents.

In considering the parametric variations, it is noted that the gross column cross section area is closely *correlated* with the area of the deck supported by the bent. Therefore, the superstructure dead load as well as the foundation below should be correlated with the column width (i.e., axial load capacity). This allows readers to define the total superstructure dead load in terms of the column cross section area:

$$\text{Superstructure Dead Load (kN)} = 1{,}000\ nh^2 + 1{,}500 \qquad (2.6)$$

where n is the number of columns in the substructure pier, and h is the width (cross section size) of the column. The additional weight of the wearing surface is about 27 percent of the superstructure dead load.

2.3.3 Representative Foundations

Based on an analysis of the survey results, three foundations and three soil conditions are found to be most typical. The possible combinations of foundation and supporting soils are grouped into eight categories. (It was considered unlikely to have spread footings founded on soft soil. This foundation/soil condition was excluded in the subsequent study.) The foundation configurations (e.g., number of piles, size of footing, etc.) are affected by the column width (size). Foundation stiffness coefficients for the eight categories are summarized in Table 2-4 for the two-column bents and in Table 2-5 for four-column bents. These foundation stiffness coefficients are obtained using the standard procedure developed in a

TABLE 2-1 Parameters of the four substructure types

a. Single-Column Bents

	Low	Avg	High
Height (m)	4.5	7	8.5
Width (m)	1.4	1.65	1.9
% Steel (r_s)	2.5%	3.0%	3.5%
% Shear Steel (r_s)	0.18%	0.25%	0.32%
f'_c (MPa)	22	27	32
f_y (MPa)	400	450	500
Bridge Width (m)	12	14	18

b. Two-Column Bents

	Low	Avg	High
Height (m)	4	11	18
Width (m)	0.8	1.2	1.6
% Steel (r_s)	1.1%	2.3%	3.5%
% Shear Steel (r_s)	0.20%	0.32%	0.45%
f'_c (MPa)	22	27	32
f_y (MPa)	400	450	500
Bridge Width (m)	8	11	14

c. Multi-Column Bents

	Low	Avg	High
Height (m)	3.5	6.5	9.5
Width (m)	0.5	1.0	1.5
% Steel (r_s)	0.60%	1.85%	3.10%
% Shear Steel (r_s)	0.10%	0.27%	0.47%
f'_c (MPa)	22	27	32
f_y (MPa)	400	450	500
Bridge Width (m)	10	15	20

d. Pier Walls

	Low	Avg	High
Height (m)	4.5	6.5	8.5
Width (m)	0.5	1.0	1.5
% Steel (r_s)	1.50%	1.70%	1.90%
% Shear Steel (r_s)	0.10%	0.20%	0.40%
f'_c (MPa)	22	27	32
f_y (MPa)	400	450	500
Bridge Width (m)	11	13	15

TABLE 2-2 Parameters for two-column bent

Variation #	Variation	Low	Average	High
1	Height [m]	4	11	18
2	Width [m]	0.8	1.2	1.6
3	Concrete Strength [MN/m^2]	22	27	32
4	Steel Strength [MN/m^2]	400	450	500
5	ρ_{long} [%]	1.10	2.30	3.50
6	ρ_{trans} [%]	0.18	0.32	0.45

TABLE 2-3 Parameters for four-column bent

Variation #	Variation	Low	Average	High
1	Height [m]	3.5	6.5	9.5
2	Width [m]	0.5	1.0	1.5
3	Concrete Strength [MN/m^2]	22	27	32
4	Steel Strength [MN/m^2]	400	450	500
5	ρ_{long} [%]	0.60	1.85	3.10
6	ρ_{trans} [%]	0.18	0.32	0.45

FHWA funded study (Lam and Martin, 1986). Although foundation and soil behavior is nonlinear, it is sufficiently accurate to use an equivalent linear approximation for foundation modeling. In most instances, foundations are designed with a higher safety margin against catastrophic failure. Thus, the behavior of a substructure under lateral load is usually controlled by the column members or pile extensions. In most cases, the capacity of the foundation/soil system is such that they can carry some overload. An exception would be encountered in the case of lateral instability or gross settlement due to liquefaction and/or scouring. This latter case is beyond the scope of the current study. However, the excessive foundation displacement in soft soil conditions is accounted for by the functionality criteria where the total displacement of the bent is monitored.

TABLE 2-4 Two-column bent, foundation stiffness

a. Two-Column Bent - Average Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	97200	72900	3650000
2 spread\stiff\	147000	110000	5530000
3 extension\soft\	443077	5226	113726
4 extension\normal\	1107000	17784	220882
5 extension\stiff\	1994000	46628	367348
6 pile\soft\	675400	18870	376700
7 pile\normal\	1689000	85870	941700
8 pile\stiff\	3039000	299000	1695000

b. Two-Column Bent - Low Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	61500	46100	999000
2 spread\stiff\	93100	69800	1510000
3 extension\soft\	295358	2283	34614
4 extension\normal\	738462	7474	65038
5 extension\stiff\	1329231	19067	105915
6 pile\soft\	450300	12580	94170
7 pile\normal\	1126000	57240	235400
8 pile\stiff\	2026000	199300	423800

c. Two-Column Bent - High Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	120000	89900	7120000
2 spread\stiff\	182000	136000	10800000
3 extension\soft\	590769	9259	260623
4 extension\normal\	1476923	32421	519329
5 extension\stiff\	2658462	86849	879287
6 pile\soft\	1351000	37730	1413000
7 pile\normal\	3377000	171700	3531000
8 pile\stiff\	6079000	598000	6357000

TABLE 2-5 Four-column bent, foundation stiffness

a. Four-Column Bent - Average Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	77800	58300	1870000
2 spread\stiff\	118000	88300	2830000
3 extension\soft\	369000	8030	54195
4 extension\normal\	923100	36500	109932
5 extension\stiff\	1661000	127200	188483
6 pile\soft\	450000	12580	94170
7 pile\normal\	1126000	57200	235000
8 pile\stiff\	2026000	199000	424000

b. Four-Column Bent - Low Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	38900	29200	234000
2 spread\stiff\	58900	44200	354000
3 extension\soft\	184615	2647	7339
4 extension\normal\	461500	12050	14000
5 extension\stiff\	830700	42000	23000
6 pile\soft\	450300	12580	94170
7 pile\normal\	1126000	57240	235400
8 pile\stiff\	2026000	199300	423800

c. Four-Column Bent - High Column Width

	$K_{vertical}$ (kN/m)	$K_{transverse}$ (kN/m)	$K_{rotation}$ (kNm)
1 spread\normal\	117000	87500	6310000
2 spread\stiff\	177000	133000	9560000
3 extension\soft\	553900	15350	168600
4 extension\normal\	1380000	69900	356000
5 extension\stiff\	2490000	243300	628600
6 pile\soft\	1013000	28300	565000
7 pile\normal\	2533000	128800	1413000
8 pile\stiff\	4560000	448500	2543000

2.3.4 Analytical Models for Sensitivity Studies

To account for the parametric variability and the different foundation and soil conditions, a total of 104 (= 13 × 8) cases are considered for each substructure type. These would model 13 different variations for structural parameters (column sizes and material properties) and 8 different foundation/soil systems.

2.4 SUBSTRUCTURE SAFETY AND LOAD APPLICATIONS

Redundancy is defined as the additional safety margin beyond that associated with the reliability of the first component failure, which is the basis of current LRFD Specifications. For most cases, substructure redundancy can be assessed by an evaluation of the nonlinear behavior of individual bents. The vertical loads (i.e., the maximum design dead and live loads) on the superstructure should be considered in conjunction with the lateral loads.

It is judged unlikely that the extreme live load on the superstructure and the extreme lateral load acting on the bent will occur simultaneously. Furthermore, past observations have shown that substructure failures have been mostly

caused by lateral loads. Therefore, the application of vertical loads and lateral load are de-coupled. Based on the AASHTO LRFD Specifications, the maximum nominal live load effect associated with the load factors corresponding to Strength V Limit State is applied during the analysis. This is caused by two lanes of live load on the right side of the bent. The live loads include an HS-20 truck (325 kN) and a lane load (260 kN) in each of the two lanes. The vertical load effects (live load and dead load) are applied to the substructure as the base load. Then, a nonlinear analysis is performed by incrementing the lateral load pushing toward the right side of the bent.

2.5 FAILURE CRITERIA AND LIMIT STATES

The performance of a bridge substructure is affected by the following:

- Vertical (dead and live) loads acting on the superstructure and the weight of the substructure;
- Interaction of vertical load and the lateral displacement of the bent (i.e., P-Δ effect);
- Strength and deformation capacity at the critical sections;
- Foundation soil performance; and
- Behavior of beam-column joints and column-footing joints.

The following assumptions are made in the subsequent analytical investigations:

a. The foundation and soil performance are represented by the equivalent linear modeling of the expected nonlinear soil behavior. Because of the higher safety factor applied in foundation design, it is assumed that failure will be controlled by structural components. For soft soil and flexible foundation, this equivalent linear model is still capable of predicting excessive displacement and rotation indicative of foundation failure.
b. Using AASHTO criteria, beam-column joints and column-footing connections are designed sufficiently strong so that the first failure will occur in the columns or the cap beams (i.e., away from the complicated joint regions). Avoiding failure in the joint is a well-accepted design goal for new bridges so that joint capacity always exceeds member capacity. However, for many aging existing structures and for structures located outside earthquake-prone areas, joints may not be designed for lateral loads and may be susceptible to damage. If the joint region is the weak link, then substructure redundancy may not be achievable because of the brittle nature of a joint failure.
c. Superstructures do not improve substructure redundancy, because superstructure-to-substructure connections are typically designed for transferring vertical load effects but are not designed for *significant* lateral load. For this reason, the most frequently observed earthquake damage to bridges is the bearing damage. Initially, bridge bearings provide significant initial stiffness in both longitudinal and lateral directions; however, these stiffness properties will quickly deteriorate when bearings become damaged.

2.5.1 Limit States

Three types of limit states are considered in this study: the ultimate limit state for the intact bridge substructure, functionality limit states for the intact bridge substructure, and the ultimate limit state for the damaged bridge substructure. These limit states are controlled by *events* that occur in the structure system during the overload process. These events are traced during the nonlinear analysis leading to one of the limit states.

In the analysis performed, the *ultimate limit state of the intact bridge substructure* is caused by one of the following events: (1) the formation of a global collapse mechanism resulting in a state of *incipient collapse*; (2) concrete crushing leading to the loss of a component (one column); and (3) instability due the P-Δ effect. When any of these limits are reached, the ability to continue carrying additional loading is limited. Subsequently, in Chapter 3, item 3 is no longer considered as a separate limit state because the P-Δ effects are already included in the bending moments that cause the formation of the mechanism (item 1) and produce the strain leading to concrete crushing (item 2). Thus, item 3 needs not be considered on its own.

The functionality limit state of the intact bridge substructure is considered by monitoring the total lateral displacement at bent cap. If the displacement reaches a certain level, it may no longer be safe for the public to use the bridge, and the bridge would be considered unfit for use. Even if the physical damage is not excessive, the bridge may have lost its intended function. Initially, several possible functionality limit states were considered. These include (1) a relative displacement (top of column relative to bottom of column) equal to 2.5 percent of column height; (2) a total displacement equal to column height/200; (3) a total displacement equal to column height/100; and (4) A total displacement equal to column height/50. All these criteria were investigated in order to decide the most appropriate criterion for functionality. After inspecting the results, it was decided to recommend criterion (4) as the final criterion for the functionality limit state. The decision was based on the following arguments: Criterion 1 ignores important deformations that may occur at the base of the columns due to soil and foundation flexibility. Criteria 2 and 3 are found to often occur before the first failure of a column. Thus, in many situations they occur in the linear elastic range before nonlinear behavior is initiated. Since the goal of this project is to study the behavior of substructures after the failure of one member, criterion 4 is deemed to be most appropriate. Also, for cases of columns that are highly confined founded on relatively stiff foundations,

criterion 4 occurs at lateral load levels that are on the same order of magnitude as those which cause the crushing of the column. For these reasons, a total displacement equal to column height/50 is recommended in Chapter 3 although the other criteria have been used in the development stages of this study.

The third type is the *ultimate limit state of the damaged bridge*. This situation simulates the occurrence of damage associated with foundation scouring or vessel collision. The resistance of one of the columns is taken out. During the development stages, two scenarios are considered for the exterior column of a four-column bent: (1) column was damaged to the extent that flexural resistance was lost, but can still carry axial force (this is simulated by replacing the damaged column by a truss element), and (2) column was severely damaged such that both axial and flexural resistances were completely lost. This is simulated by removing the column from the model. The same damage limit scenarios were also developed for the damage involving an interior column of the four-column bent. Subsequently, in Chapter 3, only the second damage scenario is used corresponding to the complete loss of an exterior column. This damage scenario has been used to remain consistent with the damage scenario proposed in NCHRP Project 12-36, whereby the exterior member of a multigirder superstructure was removed. Also, most collision damage is likely to hit the exterior columns rather than the interior ones; and by taking the most severe condition (total loss of the column), the researchers conservatively cover the partial damage condition.

2.6 ANALYTICAL PROCEDURE FOR LATERAL PUSHOVER ANALYSIS

This section describes the analytical procedure and the various analytical steps performed during the lateral pushover analyses of bridge bents using the program PIERPUSH. The program combines two independent modules: the reinforced concrete section analysis program BIAX (Wallace, 1992) and the nonlinear structural analysis program NEABS (Penzien, Imbsen & Liu, 1981). The objective of the analysis is to monitor the development of nonlinear events occurring in the structure and the associated lateral force levels, namely, the *event-to-event response monitoring*. The important nonlinear events that PIERPUSH monitors are as follows:

1. *Effective* yield of the cross section—The moment-curvature relationship for a reinforced concrete section is curvilinear from initial elastic behavior through cracking and first yielding of steel reinforcement, to progressive yielding of the section. In practice, it is sufficiently accurate to represent this curvilinear relation by a simplified *bi-linear* relationship. The *effective yield point* is selected as the point where a significant reduction in the stiffness occurs and inelastic deformation (rotation) of the section begins to grow at a much faster rate with little increase in moment. This is the event corresponding to *component damage* as defined in current codified design practices.
2. *Local deformation limits*—The ability of the structure system to carry additional loads following the initial damage in one or more components depends on the deformation capacity of components, that is, the ability *to sustain large concentrated deformation*s *in the plastic hinge zones*. The maximum plastic rotation in the plastic hinge zone is controlled by the allowable/tolerable concrete compressive strain in the extreme fibers. For *unconfined concrete, this is 0.004 (0.4 percent); and for confined concrete section, this is 0.015 (1.5 percent).* These are values that have been generally accepted by the profession in recent seismic retrofit design practice nationwide (ATC-32, 1996; Liu et al. 1997b). However, these values may need further examination. If these values are set too low (i.e., too conservatively), premature component damage will control system capacity and the global limit state mechanism will not form. During the development stages of this study, the researchers also considered an extreme case where the local deformation capacity is set as infinite. Using an infinite concrete crushing strain allows for the careful analysis of the behavior of substructures to understand how the ultimate capacity is influenced by the various geometric and material properties. This study also allows an evaluation of the sensitivity due to the maximum strain limits imposed. The final results of Chapter 3 account for the possible crushing of confined and unconfined concrete as defined in this section.
3. *Shear strength*—Although column shear failure is brittle, this event is still monitored during the incremental analysis process. It can be argued that, because of the brittle nature of shear failure, this type of failure must be prevented in new designs by specifying safety margins sufficiently higher than those set for flexural behavior. If this capacity design principle is followed, shear failure event will not occur. It is beyond the scope of this study to verify the member design criteria set by AASHTO during the development of LRFD.
4. *P-Δ Effect*—This event is the interaction of vertical load effect and lateral displacement of the bent. As lateral displacement increases and the critical section reaches progressive yielding, the negative stiffness induced by the column axial compression may become critical. When the section is within the elastic range, this P-Δ effect causes only slight softening of the flexural behavior. Once the effective yield of the section is reached (significant progressive yielding and reduction of stiffness), the P-Δ effect may cause significant deterioration of the flexural capacity of the members and the lateral load capacity of the substructure system. In the development stages, the researchers defined a loss of 25 percent of the moment capacity due to P-Δ effect as a critical event that may result in instability. Subse-

quently, this criterion was not used in Chapter 3 as a separate limit state because the P-Δ effects are already included in the moments that cause the crushing of concrete (local deformation limits), those that affect the formation of the global mechanism (Item 5), as well as those of the functionality limit states mentioned above.

5. *Formation of global mechanism*—Following the progressive development of plastic hinges in the substructure system, eventually a limiting, incipient plastic collapse mechanism is formed. This clearly represents the ultimate global failure state.
6. *Foundation performance*—Since foundation and soil medium were idealized as equivalent linear, there is no nonlinear event to be observed. However, the nature of the equivalent linearization model is to account for the nonlinear behavior by the softened stiffness value used. Therefore, for flexible foundation on soft soil, the magnitude of foundation displacement and rotation are indications of foundation performance under lateral loads. This is accounted for by monitoring the total bent displacement in the functionality limit state.

2.6.1 System Reserve Ratio

The additional load-carrying capacity beyond the first component damage (as the researchers defined redundancy) can be expressed as the system reserve ratio, R_u. This is the ratio of the ultimate lateral strength versus the first component damage. Any one of the nonlinear events just described may govern the ultimate lateral strength. In essence, the system reserve ratio is a deterministic measure of substructure redundancy. In the next section, detailed results for these ratios will be presented to illustrate the prominent factors that affect the system redundancy. These results are derived from extensive parametric studies and form the basis of the subsequent reliability calibration for the system factors presented in Chapter 3.

2.6.2 Structural Model

The bridge bent with *n* columns is modeled as a two-dimensional frame with beam-column elements. Nonlinear frame elements based on a *lumped plasticity formulation* are used for the potential plastic hinge zones in the columns (Tseng and Penzien, 1973; Imbsen and Penzien, 1984). The inelastic behavior is described by the *yield surface* of axial force and biaxial or uniaxial bending moment. (In this study, a 2-D frame model was used where all bending is uniaxial.) The nonlinear foundation behavior is represented by a set of equivalent linearized stiffness coefficients based on the expected foundation displacements. This foundation stiffness set is placed under each column using a zero-length element. If necessary, the foundation can be idealized as nonlinear models as well. However, our previous experiences indicate that the equivalent linear modeling is sufficiently accurate (Liu et al. 1997a, 1998a, 1998b), and the more critical nonlinear behavior lies in the substructure members. An example is shown in Figure 2-2 for a two-column bent.

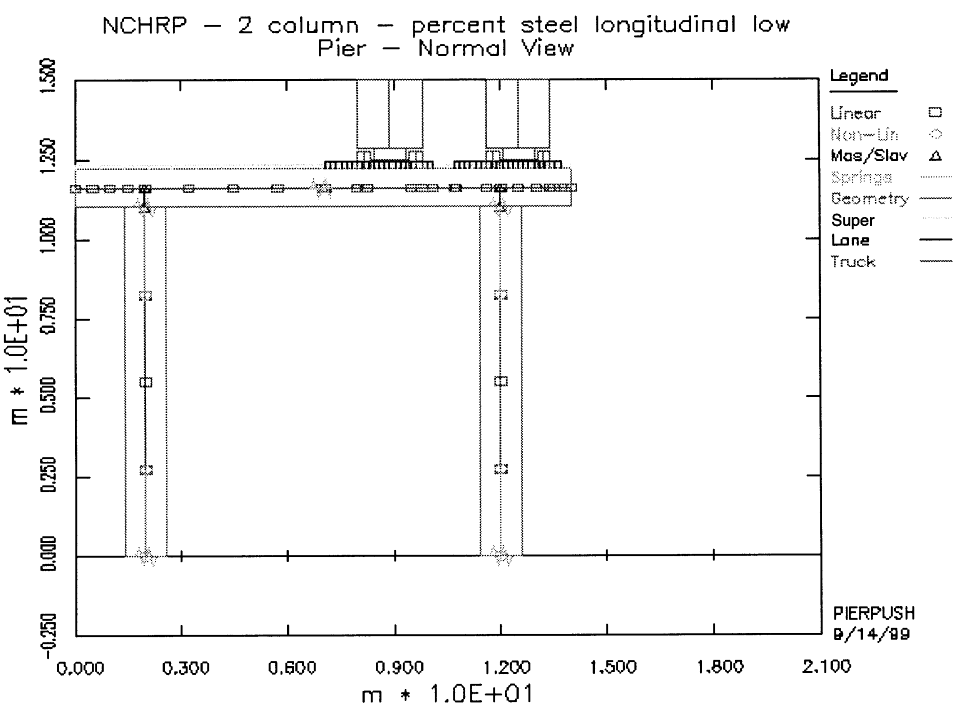


Figure 2-2. Example structure—two column bent.

2.6.3 Section Analysis

Based on the column cross-section geometry, the longitudinal reinforcement layout, the amount of lateral (confinement) reinforcement, and the material properties, an input file is developed for the section analysis program, BIAX. The objectives of the section analysis are (1) to develop the nonlinear moment-curvature (M-ψ) relationship, (2) to establish the plastic deformation capacity corresponding to prescribed limiting strains for concrete, and (3) to determine the axial force-bending moment (P-M) interaction curve that defines the yield surface of the column used in the NEABS program.

BIAX discretizes the cross section into concrete fibers and discrete reinforcing steel bars as shown in Figure 2-3. The effect of transverse (confinement) reinforcement is accounted for by using the appropriate constitutive relation for concrete (confined or unconfined). With the axial strain at a reference axis and the section curvature specified, BIAX determines the individual fiber strains using linear section kinematics according to the conventional Bernoulli-Euler beam theory. By invoking the constitutive relations for steel and concrete and the equilibrium conditions, the distribution of the fiber axial stresses can be determined. The sectional force resultants, that is, axial force and bending moment, are obtained by integrating the fiber axial stresses over the cross section.

An elastic-perfectly-plastic bilinear stress-strain relation is used for reinforcing steel defined by elastic modulus E_s and yield stress f_y. The rupture strain is assumed to be 0.1 (10 percent). For concrete in compression, use the relation:

$$\sigma_c = \begin{cases} f_c\left[2\dfrac{\varepsilon}{\varepsilon_0} - \left(\dfrac{\varepsilon}{\varepsilon_0}\right)^2\right] & \varepsilon \le \varepsilon_0 \\ f_c - \dfrac{f_c - \sigma_r}{\varepsilon_u - \varepsilon_0}(\varepsilon - \varepsilon_0) & \varepsilon_0 < \varepsilon \le \varepsilon_u \\ \sigma_r & \varepsilon > \varepsilon_u \end{cases} \tag{2.7}$$

where $\varepsilon_0 = 0.002$ is the strain at peak stress, and σ_r is a residual stress for high strain levels. Note that the preceding equation requires positive values for ε. Figure 2-4 shows the stress-strain relations for both confined and unconfined concrete assuming a compressive strength of $f_c' = 27$ MPa. The residual stress for confined concrete is $\sigma_r = 0.2\, f_c' = 5.4$ MPa. Zero residual stress is assumed for unconfined concrete. The transition to the residual stress of the stress-strain relation is at a strain of 0.006 and 0.03 for unconfined and confined concrete, respectively.

For concrete in tension, BIAX uses the following relation:

$$\sigma_c = \begin{cases} E\varepsilon & \varepsilon \le \varepsilon_{cr} \\ \dfrac{f_t}{1+\sqrt{200\varepsilon}} & \varepsilon > \varepsilon_{cr} \end{cases} \tag{2.8}$$

where $\varepsilon_{cr} = f_t/E$ and $E = 2f_c'/\varepsilon_0$. In the section capacity analysis, it is reasonable to assume that the tension zone is fully cracked. Hence the researchers neglect any concrete tensile stress and set $f_t = 0$ as input to BIAX.

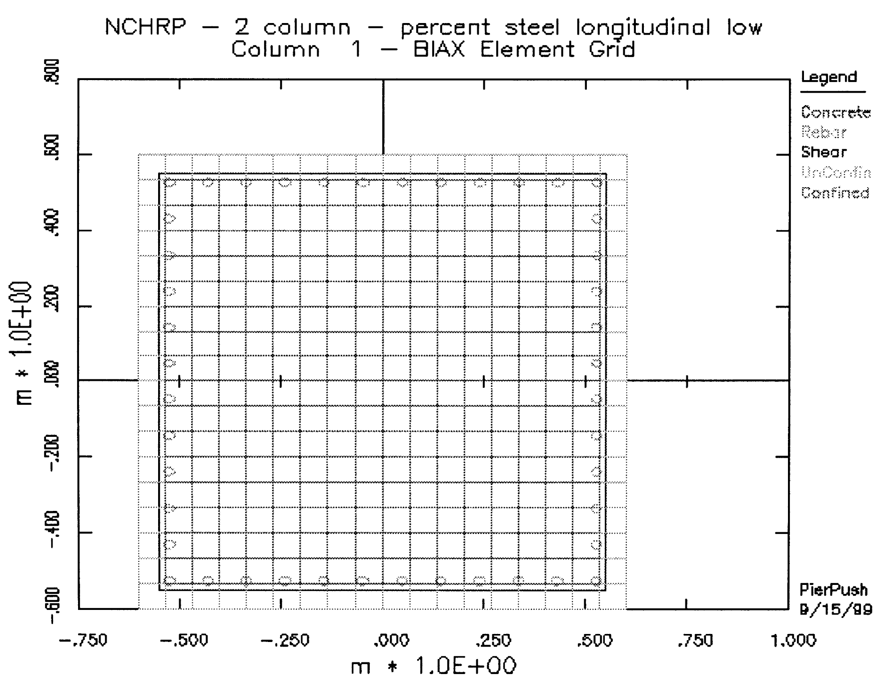


Figure 2.3. Discretized column section.

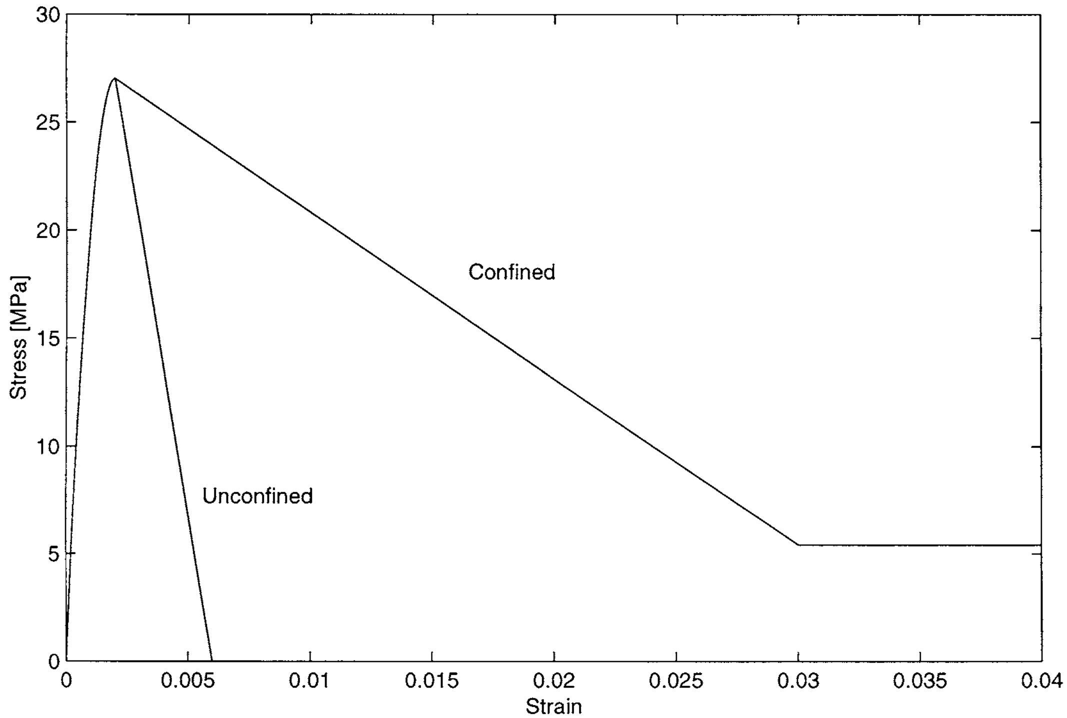


Figure 2.4. Stress-strain relation for concrete in compression.

2.6.4 Nonlinear Beam-Column Model

BIAX generates a series of moment-curvature relations corresponding to different levels of axial force. These are combined to establish the bending moment-axial force interaction surface, which defines the yield surface of the nonlinear column elements. Within the yield surface, the component is linear elastic. Once the yield condition is reached, under combined axial force and bending moments, the continued yielding is governed by the kinematic strain-hardening law and the plastic flow rule. As lateral load increases, the column axial force, as well as the moment, varies. This is automatically accounted for in the lumped plasticity formulation as the yield condition is defined by the combination of axial force and moment. The plastic rotation developed at the critical section provides the necessary information for performance evaluation.

2.6.5 Strain Hardening

The moment-curvature relation is established for the whole range of curvatures up to the crushing curvature defined by the ultimate concrete strain. For reinforced concrete sections considered in this study, zero strain hardening was assumed. Significant strain hardening in reinforced concrete section occurs only at very high strain levels that are beyond the maximum allowable compressive strain for concrete. However, to avoid numerical instability caused by the perfect-plastic relationship, a small hardening stiffness of 1 percent of elastic stiffness is used. This minimal amount of strain hardening is only included for numerical reasons and produces results that do not significantly deviate from those of perfectly plastic relationship.

2.6.6 Plastic Hinge Length

A plastic hinge length is required to interpret the nonlinear analysis results. From the nonlinear pushover analysis, the force-displacement relationship is determined. Furthermore, during the analysis process, the researchers monitor the development of the maximum displacements and the associated plastic rotations at critical sections. In the lumped plasticity model for beam-column elements, all the plastic deformation is concentrated at a single point; however, in real components, inelastic deformations are spread over a distance defined as the plastic hinge zone. A commonly accepted modeling assumption is that the maximum curvature at the critical section can be averaged over a plastic hinge length, L_p, such that:

$$\psi_{P,\,\max} = \theta_{p,\,\max}/L_p \tag{2.9}$$

where $\psi_{p,\,\max}$ is the maximum plastic curvature estimated for the critical section, and $\theta_{p,\max}$ is the maximum plastic rotation over the entire plastic hinge zone. Based on experimental observations, the plastic hinge length is estimated as:

$$L_p = 0.10 L_{\mathrm{eff}} \tag{2.10}$$

where L_{eff} denotes the distance from the plastic hinge to the point of contra-flexure. For columns with double-curvature deformed shape such as those of frames with stiff column caps and reasonably stiff foundations, as encountered in bridge substructures, L_{eff} is half the column height.

2.6.7 Incremental Static Analysis and Event-to-Event Response Monitoring

Based on the bent configuration provided by the user, and the bending moment-axial force interaction surface determined by BIAX, PIERPUSH writes an input file for the program NEABS. NEABS is then executed to perform incremental, displacement controlled, static nonlinear analysis. The goals of this analysis are to monitor the progressive development of nonlinear events and to determine the associated lateral forces acting on the bent. This event-to-event tracing leads finally to the predefined limit states.

2.7 PIERPUSH—A GUIDED TOUR OF THE NONLINEAR PUSHOVER ANALYSIS AND INTERPRETATION OF RESULTS

To illustrate the pushover analysis procedure and interpretation of results in the context of substructure redundancy, a two-column bent is selected as an example. This is an 11-m high, two-column bent with 1.2 m × 1.2-m columns, as shown in Figure 2-2. The column longitudinal reinforcement is double symmetric with 44 bars with A = 360 mm^2. The section is lightly reinforced with a reinforcement ratio of 1.1 percent. The concrete cover is 7.5 cm. Material properties used are

For concrete: $E_c = 25{,}000$ Mpa; $f_c' = 27$ Mpa; and $\gamma_c = 23.5$ kN/m^3

For reinforcing steel: $E_s = 200{,}000$ Mpa; $F_y = 450$ Mpa

The superstructure dead load is 5560 kN and is uniformly distributed along the bent cap. Including the self-weight of the bent, the dead load is about 6800 kN. The live load of 1385 kN consists of lane and truck loads and is placed along the bent cap to cause maximum effects in the right column (Column 2) in the direction of push (see Figure 2-2). The resulting maximum axial compression in Column 2 due to dead and live load is 4400 kN. The axial force in the other column is somewhat lower. The foundation stiffness is 72,900 and 97,200 kN/m in the transverse and vertical directions respectively, and the rotational stiffness is 3,650,000 kNm/rad. With these parameters, the example discussed here is classified as a two-column bent with "average column height and width; average f_c' and f_y; low percent longitudinal reinforcement ratio; spread foundation on normal soil." (This classification is defined in Section 2.8, and this structure is classified as structure case 11 and foundation soil category 1.) Results of this structure are included in Appendix B as Case #73 (not published herein).

The discretized column cross section is shown in Figure 2-3. The stress-strain relations for concrete are shown in Figure 2-4. As a first step, the section moment-curvature relations are computed. These analyses will be repeated for a range of axial forces from tension capacity to ultimate axial compression capacity of the column. Two cases are considered: (1) zero axial force and (2) an axial compression due to dead and live load. Results are shown in Figure 2-5. The range of curvature values corresponds to a compressive concrete strain at the extreme fiber up to 0.015 (1.5 percent). The moment curvature relation of a reinforced concrete section provides important information for interpreting results and assessing the performance of the structure. By specifying the limiting concrete strain levels, the corresponding limiting curvature values can be determined for each desired axial force level. For the two levels of axial force, Figure 2-6 shows both the extreme steel and concrete strain as a function of the section curvature. As expected, the section under high axial compression reaches its ultimate compressive strain of $\varepsilon_c = 0.015$ for confined concrete at a lower curvature than that with zero axial force. Thus, the section under compression has a lower plastic rotation capacity; that is, it is less ductile than the section under pure bending.

Section Deformation Capacity—Based on the M-ψ relations obtained from BIAX, the following critical curvatures are determined:

ψ_y = yield curvature = 0.0030

ψ_u = ultimate curvature
= 0.0483 for confined concrete section, and
= 0.0145 for unconfined concrete section.

For the total column height of 11 m, the plastic hinge length can be estimated as 0.55 m, that is, 0.1 × (0.5 × 11 m) for columns in double curvature. Thus, the plastic hinge rotation capacities are determined as:

$\theta_{P,\max} = 0.55 * (0.0483 - 0.0030)$
$= 0.025$ for confined concrete section;
$= 0.55 * (0.0145 - 0.0030)$
$= 0.006$ for unconfined section.

Based on the moment-curvature analysis results for the entire range of axial forces, the interaction curve for the column is obtained as shown in Figure 2.7. Using the least square technique, a cubic polynomial relation, which is required for input to the NEABS program, is fitted to this curve as shown below:

$$\frac{|M|}{M_u} = 1 - 4.96 \cdot \frac{P}{P_u} - 6.56 \cdot \left(\frac{P}{P_u}\right)^2 - 0.952 \cdot \left(\frac{P}{P_u}\right)^3 \qquad (2.11)$$

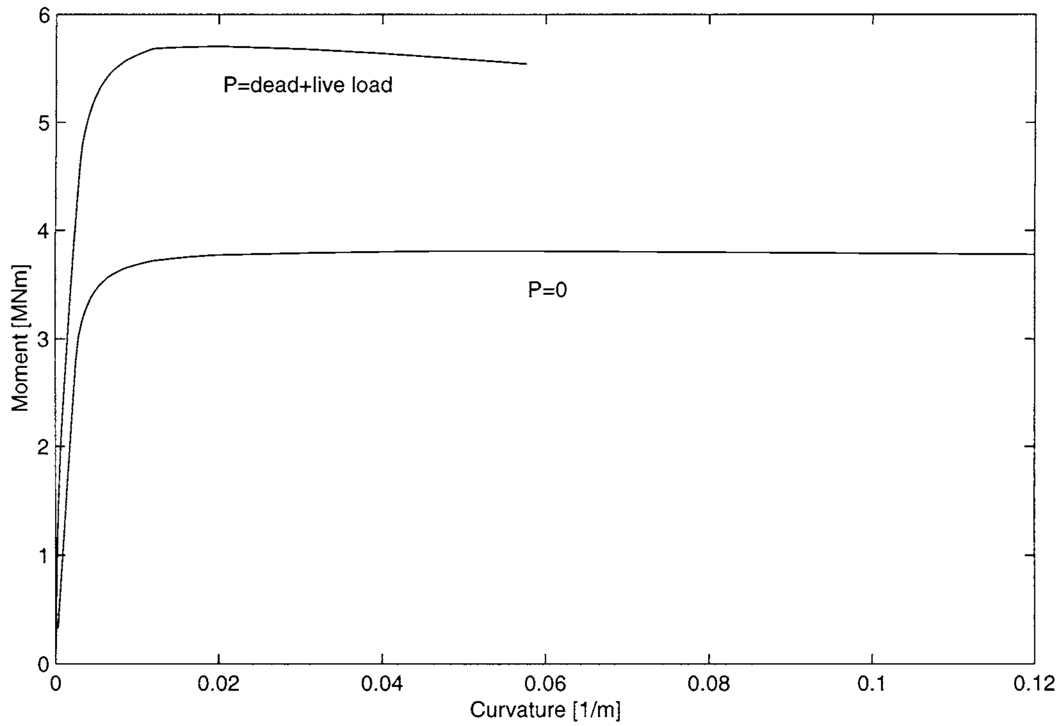


Figure 2-5. Moment-curvature relation for column sections.

where P_u = 39,800 kN and M_u = 3,751 kNm are the ultimate compression force and bending moment, respectively. P and M are the combination of applied axial force and bending moment that would cause the failure of the section. Figure 2-7 presents both the actual and the approximated interaction relations for comparison. With this generalized yield condition defined, the lumped plasticity model will automatically account for the effect of varying axial force during pushover analysis.

With these data available, the nonlinear model is completely defined and the nonlinear analysis under incrementally applied lateral load can be executed. Figure 2-8 shows the development of column end moments at four critical sections as a function of the imposed bent cap displacement. The two bottom plastic hinges form almost simultaneously at a displacement of about 0.063 m, followed by a third plastic hinge at about 0.08 m. The final plastic hinge forms at about 0.13-m cap displacement producing a *system* collapse mechanism such that the force cannot be increased any further. The lateral bent displacements are thus 0.063 m for the first component failure and 0.13 m for the final mechanism limit state.

Since the system mechanism occurs only if the local deformation limits at all hinges and concrete crushing strains are not exceeded, it is important to monitor the accumulation of plastic deformations in each of the critical sections. For this purpose, the plastic rotations in the four plastic hinges are shown in Figure 2-9 as a function of the bent displacement. Knowing the plastic rotation capacity available, this figure allows the comparison of the plastic rotation capacity to the plastic rotation demand obtained from the NEABS nonlinear analysis.

Figure 2-9 reveals that the plastic rotation capacity for a confined plastic hinge of θ_{pc} (= 0.025) is exhausted at a pier displacement of 0.28 m; while a displacement of about 0.13 m causes a plastic rotation exceeding the capacity for an *unconfined* plastic hinge θ_{pu} (= 0.006). Therefore, for substructure with confined concrete, the critical plastic hinge has sufficient deformation capacity to allow the formation of the fourth and final plastic hinge, and the mechanism is the governing event. For substructural components with unconfined concrete, the plastic hinge rotation is exhausted at a displacement of 0.13 m slightly lower than the 0.136 m at which the mechanism forms, which means that excessive local damage at plastic hinges due to the limited rotation capacity will render the substructure unfit for use.

Alternatively, Figure 2-9 can be used to trace the event-to-event development, and to identify important events. For example, the first component damage is detected when the first plastic hinge forms at the bottom of Column 2, that is, when the first non-zero plastic rotation is observed (dash-dot line). The structure reaches the final mechanism when the final plastic rotation becomes non-zero at top of Column 1 (dashed line).

Finally, the global force-displacement relation is shown in Figure 2-10. Based on the PIERPUSH results, the lateral forces and the corresponding bent displacements at the important events are:

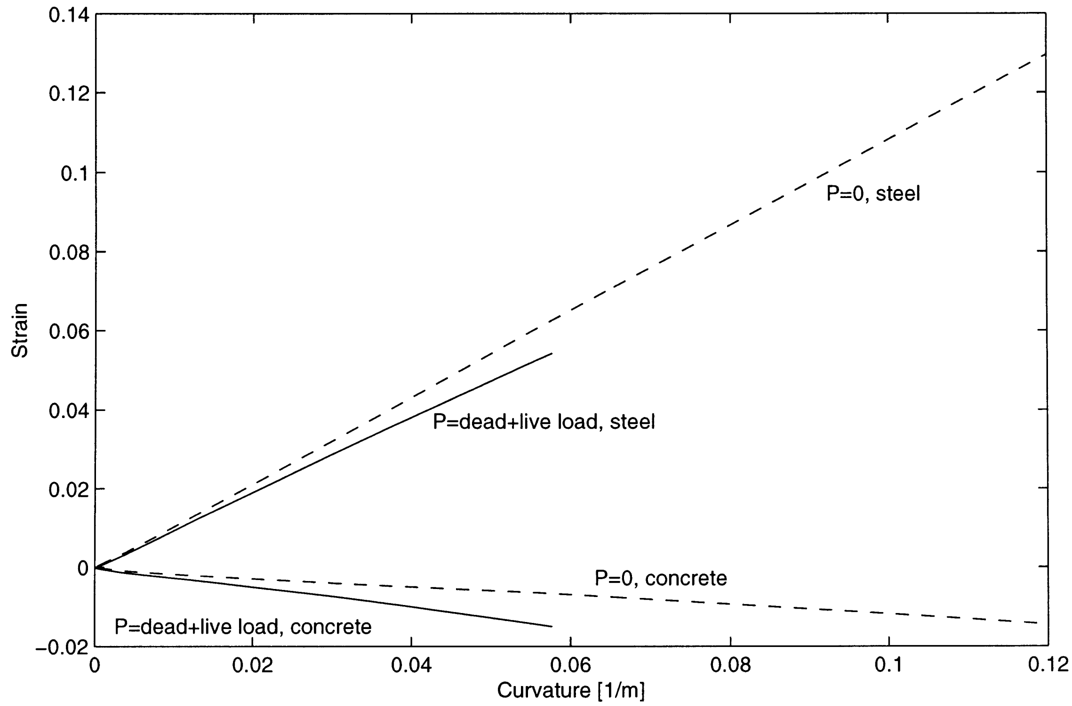


Figure 2-6. Strain-curvature relation.

Event	Displacement (cm)	Force (kN)	R_u
First Component Damage	6.3	1519	
System Mechanism	13.6	1851	1.22
Local Damage—-Unconfined	12.7	1821	1.20
Local Damage—-Confined	27.8	1748	
Excessive Displacement (H/50)	22.0	1789	

The pier force associated with the formation of the first plastic hinge is F_1 = 1,519 kN corresponding to a bent displacement of 0.063m. For the case with *confined* concrete, the governing event is the system mechanism occurring at a bent displacement of 0.136 m and the ultimate lateral force level of F_u = 1,851 kN. The resulting strength reserve ratio is 1.22 (= 1,851kN/1,519kN). For *unconfined* concrete, however, the ultimate limit state is control by the excessive local damage at a displacement of 0.127 m and the ultimate lateral force of 1821 kN. The resulting strength reserve ratio is 1.20 (= 1,821 kN/1,519kN). In this case, the difference is not significant. But in other cases, the unconfined case may be much less.

2.8 SYSTEM RESERVE RATIOS—A DETERMINISTIC MEASURE OF REDUNDANCY

To provide a sufficient database for the reliability calibration, extensive parametric studies were performed. The detailed results of the analyses of all substructure configurations and material variations considered in this study based on the responses provided by the state DOTs are summarized in Appendix C (not published herein). This section examines in detail a limited number of cases to provide an understanding of the nature of these results. In addition to studying the behavior of one-column bents and pier walls, 11 sets of structure variations of two-column and four-column bents are considered in this section. The parametric variations for the two-column and four-column bents are summarized below:

Case	Parameter	Value
1	All Parameters	Average
In the following cases, all parameters are average values except:		
2	Column Height	Low
3	Column Height	High
4	Column Width	Low
5	Column Width	High
6	Concrete Strength	Low
7	Concrete Strength	High
8	Steel Strength	Low
9	Steel Strength	High
10	Longitudinal Reinforcement Ratio	Low
11	Longitudinal Reinforcement Ratio	High

For each structural variation, eight foundation and soil conditions are considered as listed below:

ID	Foundation	Soil Condition
1	Spread Footing	Normal Soil
2	Spread Footing	Stiff Soil
3	Pile Extensions	Soft Soil
4	Pile Extensions	Normal Soil
5	Pile Extensions	Stiff Soil
6	Pile Group Foundation	Soft Soil
7	Pile Group Foundation	Normal Soil
8	Pile Group Foundation	Stiff Soil

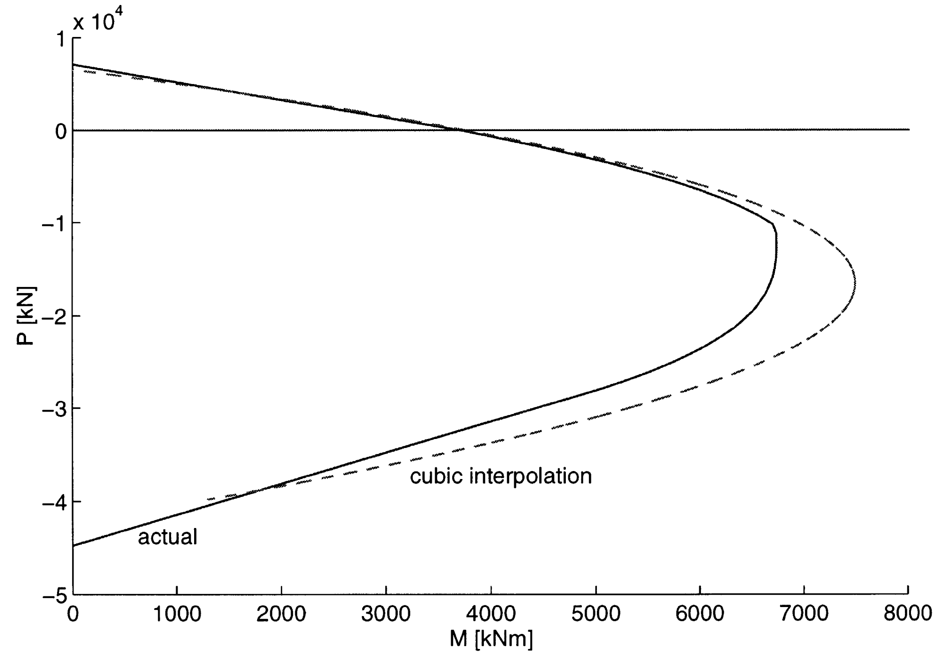


Figure 2-7. Axial force-bending moment interaction.

2.8.1 Single-Column Bents

Single-column bents are nonredundant. As soon as the first damage event occurs, the system reaches the incipient collapse condition. If shear failure is prevented and the controlling event is the plastic hinging and the occurrence of concrete crushing, then redundancy depends on the ductility capacity of the member. This is particularly critical because the strain-hardening phenomenon, which in these calculations is included only to provide numerical stability, would contribute to strength only at a very high ductility level. Therefore, any additional strength reserve in one-column bents is

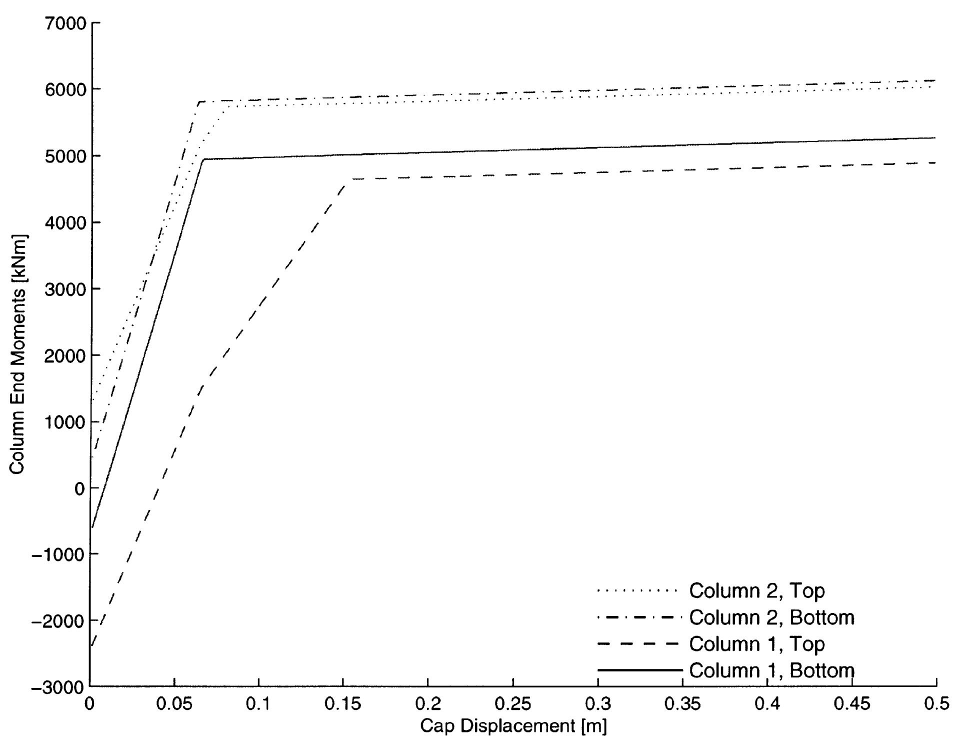


Figure 2-8. Bending moment-displacement relation.

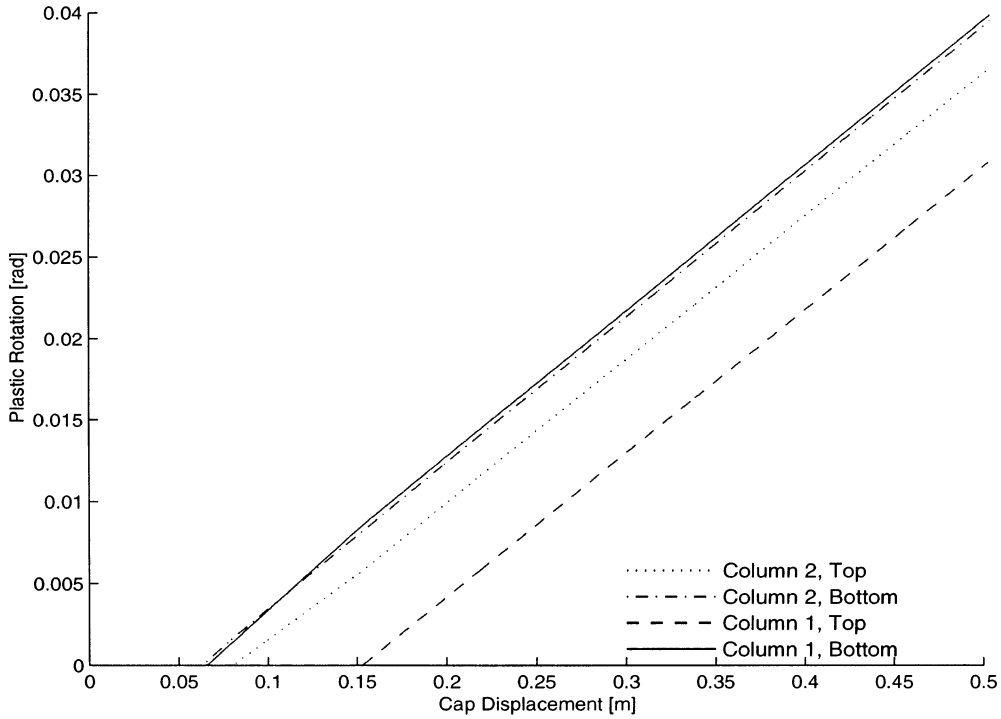


Figure 2-9. Plastic rotation-displacement relation.

very limited. Based on sensitivity studies carried out using PIERPUSH, the system reserve ratios for unconfined and confined one-column concrete bents are calculated respectively as

$$R_u = 1.0078 + 16.775/F_1$$
$$R_c = 1.0327 + 14.971/F_1 \qquad (2.12)$$

where F_1 is the force level (kN) of the first component damage and ranges from 1500 to 4500 kN. Therefore, the reserved strength ratios for single-column bents are limited to 1.02 for unconfined concrete and 1.04 for confined concrete. These flexural reserve strength ratios are very limited and are considered to be negligible. Furthermore, as the flexural yielding progresses at the critical section, its shear capacity may also begin to deteriorate. Therefore, it is prudent to conclude that the single-column bents constitute nonredundant systems. However, this does not mean that they are not safe.

2.8.2 Pier Walls

Pier walls can be classified into two categories in accordance with their lateral load carrying mechanism:

1. Taller, slender walls resist lateral loads in *cantilever* action. The behavior of these is similar to that of the single-column bents. Hence, tall slender pier walls are nonredundant.
2. Squat walls resist applied lateral loads in *strut-and-tie actions*. The ability of these walls to carry additional load (beyond the development of significant yielding) depends on the wall section and design details. In hollow walls, the wall thickness must be sufficient to prevent local buckling. For all squat walls, the transverse reinforcements must be sufficient for the compression strut action to fully develop the required strength. Further, the anchorage at the wall footing corner must be sufficient to prevent unzipping failure. Also, as is the case with single-column bents, the system reserve strength of squat walls depends on the ductility capacity of the member. Because all these requirements are not always simultaneously satisfied, it is prudent to consider that squat pier walls are also nonredundant.

On the basis of these considerations, all pier walls are classified as nonredundant in this investigation.

2.8.3 Two-Column Bents

The complete results for two-column bents under the effects of lateral loads are summarized in the tables provided in Appendix B.1. (Nonlinear force-displacement relations are shown in Figures B-1 to B-12 in Appendix B-2 for all cases

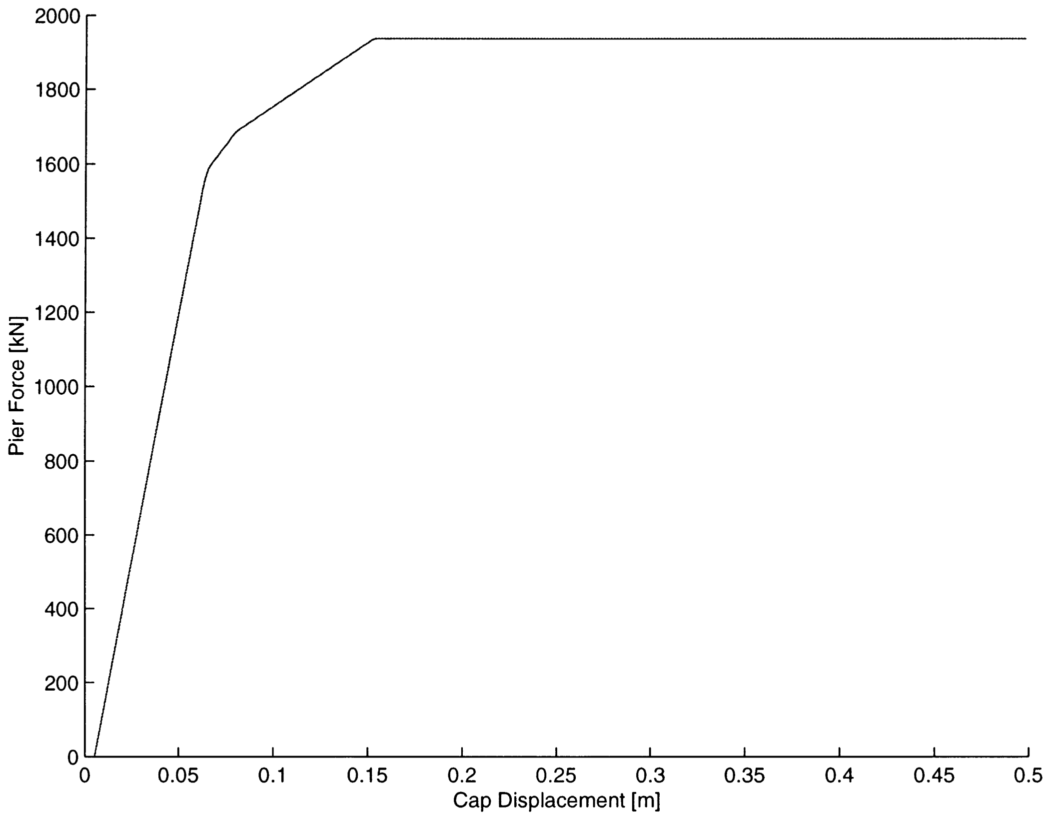


Figure 2-10. Global force-displacement relation.

considered.) This section describes the results for two cases: (1) local deformation capacity is infinite so that the system mechanism can form and (2) local deformation limits are included leading to possible crushing of concrete. Case (2) was found to govern many two-column cases. Results for case (1) are presented below to identify the effect of each structural parameter and the foundation/soil conditions.

2.8.3.1 System Mechanism Limit State

The most direct and easy way to introduce the notion of redundancy is to look at redundancy as a result of static indeterminacy. Since a statically indeterminate structure has more constraints than necessary for stability, the first component failure (e.g., formation of the first plastic hinge) usually does not constitute system failure. System failure occurs when the number of plastic hinges causes the structure to become unstable. The researchers thus define the system mechanism limit state as the *n*th component failure that leads to incipient instability of the structure. Assuming the bent cap to respond elastically, a two-column bent collapses when four plastic hinges have formed in the column top and bottom locations. Similarly, eight-column plastic hinges define collapse in four-column bents. *The reserve strength associated with the mechanism limit state is the lateral force increase between the first and the final plastic hinges.* The system *reserve ratio* is the ratio of these forces. Generally, a higher degree of static indeterminacy of the four-column bent results in higher system reserve ratios when the mechanism limit states govern. For example, the average two-column bent on a spread normal foundation has a reserve strength ratio of 1.20 while the corresponding four-column bent has a reserve of 1.28. For unconfined concrete, the same two-column bent has a system reserve ratio of 1.14 compared to 1.23 for the four-column bent.

For the two-column bents with average structural properties, Figure 2-11 shows the force-displacement relations. Similar force-displacement relations for other parametric variations are included in Appendix B.2 (available on loan). In each of these figures, results are presented for the same structure founded on eight different foundation/soil conditions as shown by different colors. On the force-displacement curves, two "*" symbols indicate *first yield (first plastic hinge)* and *incipient collapse (final plastic hinge)*. Note that the term "first yield" is used to indicate the first component damage (corresponding to the effective yield force level of the first critical section encountered). This corresponds to one section reaching its moment strength capacity. The term "collapse" indicates that the structure cannot support any additional lateral force. In this study, the analysis carried out is displacement-controlled such that the force-displacement relation can be extended

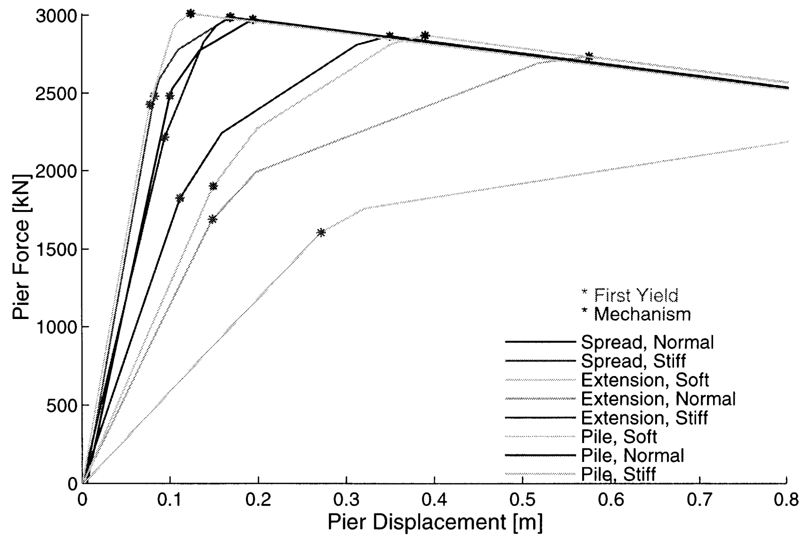


Figure 2-11. Two-column bent, force-displacement relation, average properties.

even beyond the mechanism limit. After the mechanism has formed, the global elastic tangent stiffness becomes zero and the force-displacement relation has a negative slope due to the P-Δ effects. For a flexible structure on a flexible foundation, the force-displacement curve may be flattened out before mechanism forms, also a result of P-Δ effects (see Figure B-4, for example). Note that since each figure represents one structure for different foundation conditions, the lateral forces at collapse are different. The lateral force at collapse would be identical for all foundation types if P-Δ effects were ignored. For flexible structures on flexible foundations, P-Δ effects may significantly reduce the collapse load (see Figure B-4).

In general, the structure on a soft pile extension foundation does not reach its mechanism limit state within the maximum displacement limit imposed on the bent cap. In subsequent portions of this section, the researchers do not consider this foundation type any further. Figures B-1 to B-11 show that *the foundation effect* significantly influences the lateral force at first yield. In the following sections, this phenomenon is examined in more detail.

Shown in Figure 2-12 are the system reserve ratios for the two-column bents (i.e., the lateral force corresponding to the formation of the final plastic hinge normalized with respect to the force at the effective yield of the first critical section) for different parametric variations. The five subplots, as stated below, correspond to the five structural properties that are subject to variation: (a) column height, (b) column width, (c) concrete strength, (d) steel strength, and (e) longitudinal reinforcement ratio.

In each subplot, the results for the average structure are repeated followed by the low and high values. Seven foundation types are considered in each subplot. (As mentioned before, the soft pile extension foundation was excluded.)

Figure 2-12 reveals the following major findings:

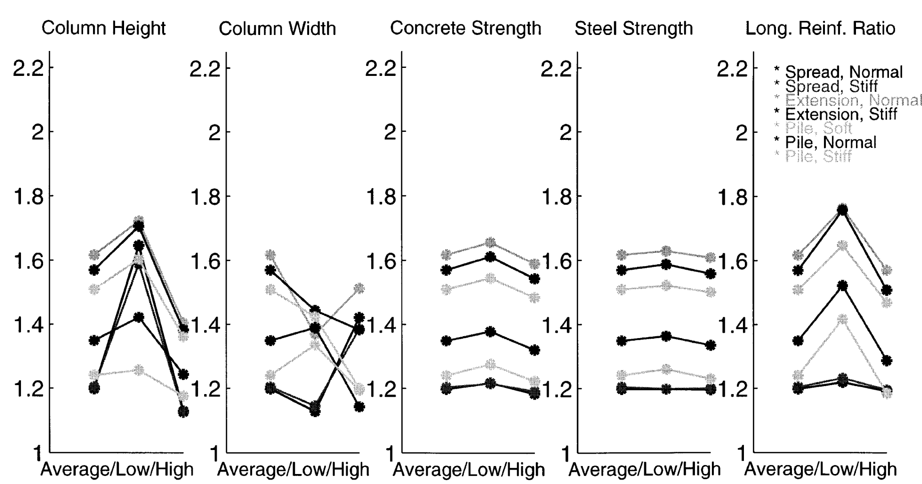


Figure 2-12. Two-column bent, reserve strength ratio.

- The system reserve ratios vary from about 1.1 to 1.8.
- For a given soil and foundation type, the system reserve ratios vary only mildly for varying material strengths (both concrete and steel).
- For varying longitudinal reinforcement ratio, the variation in the reserve strength ratio is somewhat more pronounced.
- For variation in column height and width, significantly different reserve ratios were observed for a given foundation type. For shorter columns, the system reserve ratio is higher than for average structures independent of the foundation conditions and lower for taller columns. With the system reserve ratio increased from 1.2 to 1.6, the variation is the largest for spread foundations. For pile foundations the variation is rather modest.
- The trend between small and large column widths (Case 4 and 5) is more complex. For example, the strength reserve ratio significantly increases from Case 4 (small column width) to Case 5 for the relative stiff foundation "spread/normal"; while it decreases considerably for the more flexible foundation "pile/normal."
- For all variations, the researchers observe significantly *increasing system reserve ratio with decreasing foundation stiffness. In most cases, structures on flexible foundations have a larger reserve ratio than those on stiff foundations.* In fact, the only notable exceptions are the structures with wide columns.

The above observations are for loads corresponding to the formation of a collapse mechanism. The analysis includes the additional moments from vertical loads due to P-delta effects. The rest of this section attempts to study the reasons behind the trends in the system reserve ratio noted above.

Figure 2-12 reveals that certain trends exist regarding the varying strength reserve ratios as a function of foundation and structure type. Possible causes are examined by taking a detailed look at the structural response. Since bending of cap and columns controls the bent response, bending moment diagrams and deflected shape of the bent at characteristic times during the incremental analysis provide valuable information.

Limiting Cases—Effects of the moment distribution on the strength reserve ratio can best be explained by considering the moment diagrams for three representative cases:

- Figure 2-13a shows the moment distribution with a relatively low foundation moment when the first plastic hinge forms at the column tops. This occurs if the foundation/soil condition is extremely flexible compared with that of the bent. This case leads to significant system reserve, because it takes a substantial additional force increment to mobilize the bending capacity at the column base.
- In the second case (Figure 2-13b), the first plastic hinge forms at the base with the top bending moment still relatively small. This corresponds to the case of stiff foundation/soil condition and flexible column (tall and/or smaller section). Again, the system reserve ratio is large.
- In the final case (Figure 2-13c), the moment diagram in the column is such that moments at top and bottom are essentially the same. Plastic hinges form at the column top and bottom locations almost simultaneously. This makes the structure much *less redundant* than the other two cases.

The actual moment distribution depends on the relative stiffness values of cap, column, and foundation. As will be discussed later, the foundation stiffness is a particularly important parameter influencing the system reserve ratio. Against this background, and with all other parameters equal, it is important to point out that:

- The moment distribution in Figure 2-13a approaches that of Figure 2-13c with increasing foundation stiffness; while,
- The moment distribution in Figure 2-13b approaches that of Figure 2-13c with decreasing foundation stiffness.

In other words, if the first plastic hinge forms at the column top, a more flexible foundation has a favorable influence on the strength reserve ratio. If the first plastic hinge forms at the bottom, a stiffer foundation has a positive influence on the strength reserve ratio.

Influence of Foundation Stiffness—In order to explain different reserve ratios for the same structure on different foundations, the researchers plotted the bending moment diagrams and the deflected shapes for the average structure for a stiff and a flexible foundation (spread/normal in Figures 2-14 and 2-15 and extension/normal in Figures 2-16 and 2-17). The foundation stiffness coefficients for these two foundations are summarized below:

Foundation/Soil Condition	Rotational (kN-m/rad)	Transverse (kN/m)	Vertical (kN/m)
Spread/Normal	3,650,000	72,900	97,200
Extension/Normal	220,882	17,784	1,107,000

The moment diagrams and deflection shapes were plotted for three instances: (1) when only vertical (dead and live)

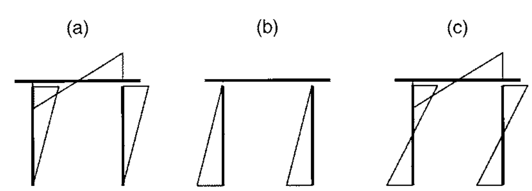


Figure 2-13. Limiting moment distributions.

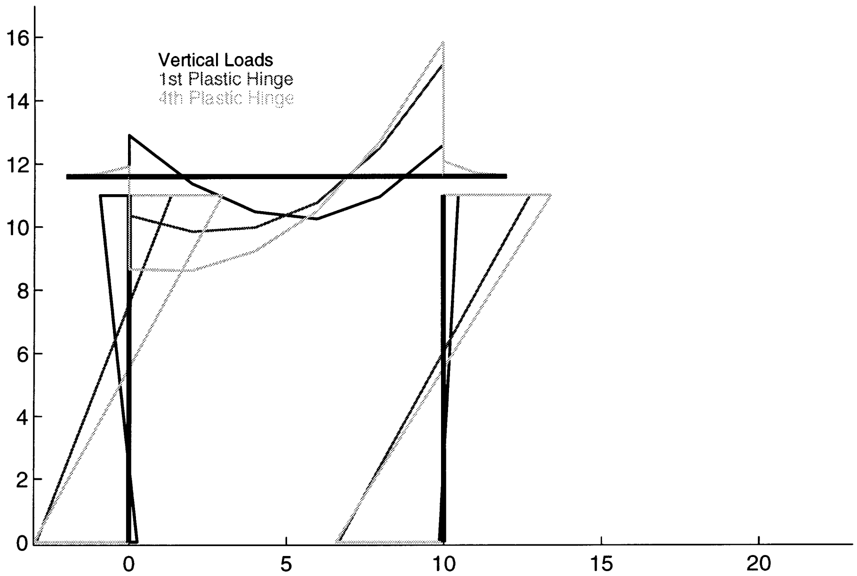


Figure 2-14. Qualitative moment diagram, "average, spread, normal."

loads were applied; (2) when the first plastic hinge formed; and (3) when the system mechanism formed with four plastic hinges.

The strength reserve ratios for these two cases represent the two extremes in Figure 2-13 for the structure with average properties.

- For the (relatively stiff) spread foundation, the first two plastic hinges form almost simultaneously in the two column bottom locations as shown in Figure 2-14. At that time, the top moment at Column 2 is already very close to its capacity, which does not leave much "room" (reserve capacity) for additional lateral force increment. Therefore, less redundancy is obtained. This case is similar to the situation depicted in Figure 2-13c.
- For the pile extension foundation, the first plastic hinge forms at the top of Column 2 as shown in Figure 2-16. At that time, the bottom moments are still relatively

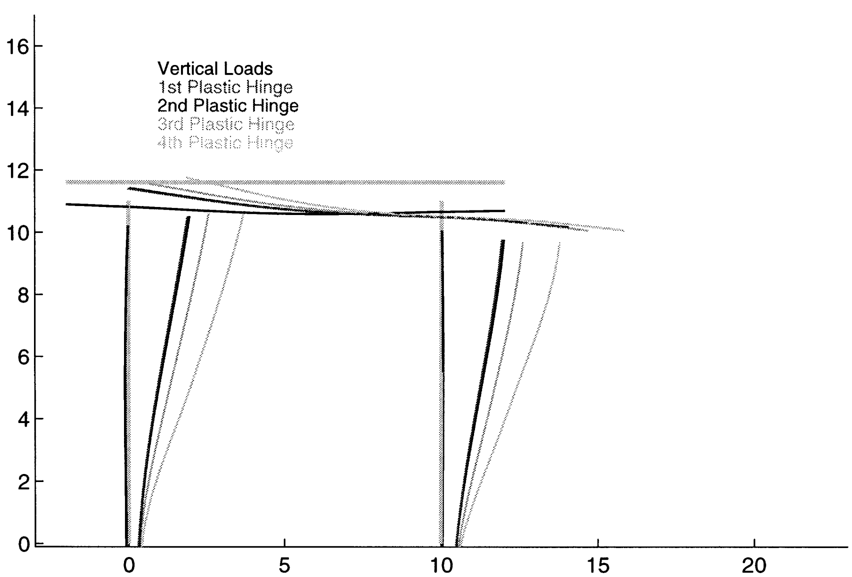


Figure 2-15. Deflected shape (magn.: 20), "average, spread, normal."

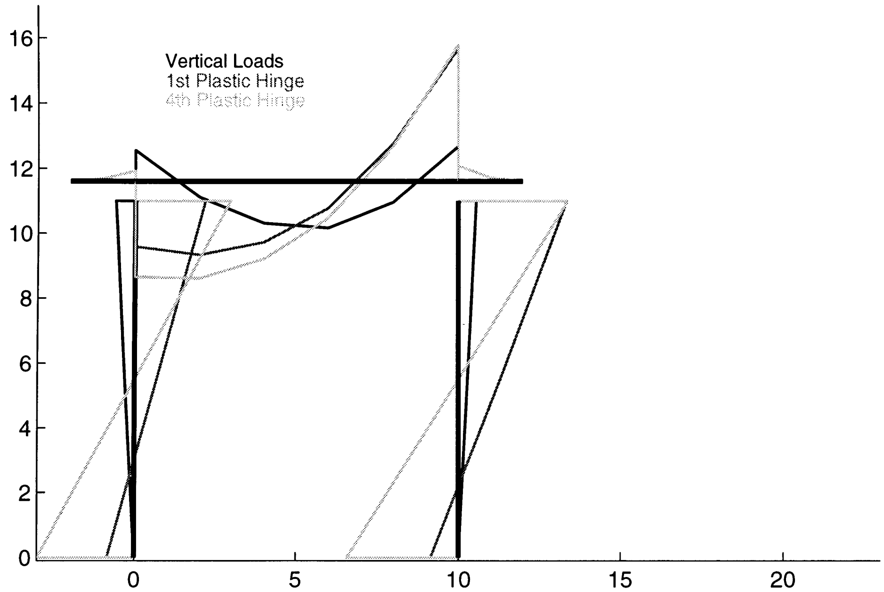


Figure 2-16. *Qualitative moment diagram, "average, extension, normal."*

small—similar to that depicted in Figure 2.13a. This allows the lateral force to increase further resulting in much higher system reserve ratio. The additional system reserve, however, comes at the expense of large displacements due to the foundation rotation as shown in Figure 2-17. At failure, the cap displacement is 50 cm for the pile extension foundation.

Influence of Column Stiffness (Column Height)—The focus is now on the difference between Cases 2 and 3, that is, effect due to short and tall column heights. For this comparison, the structure is founded on the flexible "extension/normal" foundations. Again, both the moment diagram and deformed shape are presented for the same three instances defined above. Based on the bending moment distribution at first yield for both short and tall columns (Figures 2-18 and 2-19), the first two plastic hinges form on top almost simultaneously. At this stage, the magnitude of the bottom moments relative to the corresponding section capacity, however, is different resulting in different reserve ratios. When first yield

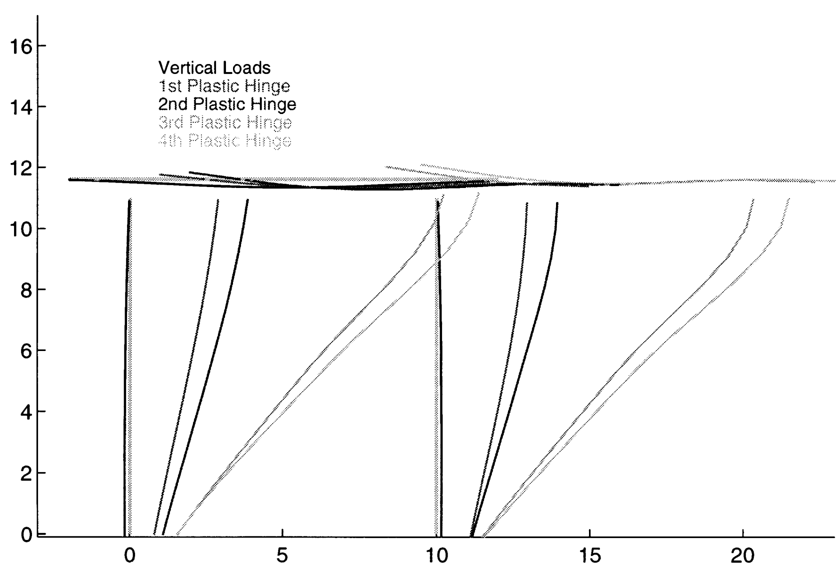


Figure 2-17. *Deflected shape (magn.: 20), "average, extension, normal."*

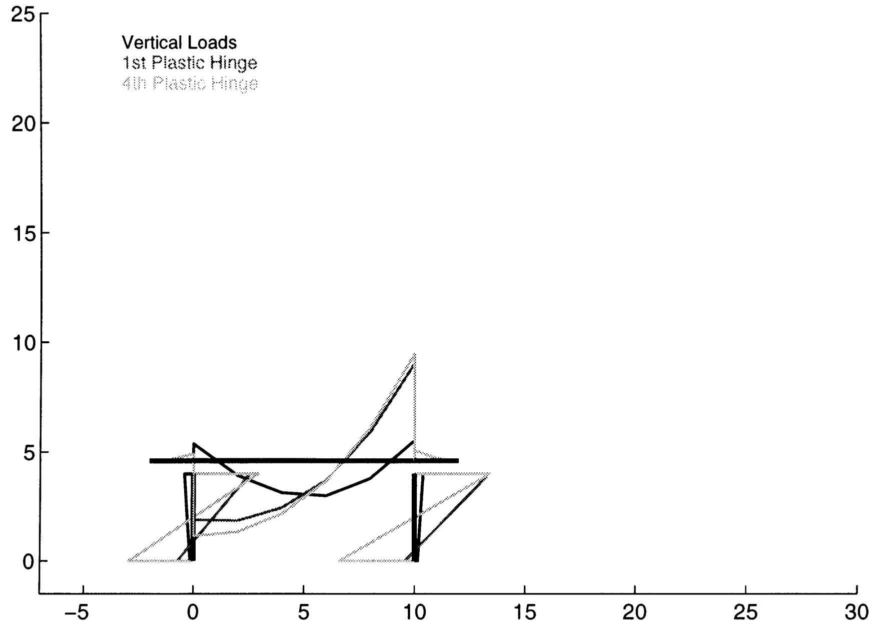


Figure 2-18. Qualitative moment diagram, "height, low, extension, normal."

occurs at column tops, the maximum base moment is about 10 percent of its section capacity for shorter columns (as shown in Figure 2-18), and is about 40 percent of the section capacity for taller columns (as shown in Figure 2-19). As a result, the system reserve ratio for the case with tall columns is less than that with shorter columns. The stiffness of the short column is quite high, and most of the displacement occurs in the foundation as shown in Figure 2-20. The net structural deformation is small resulting in higher strength reserve ratio. However, the much higher displacement for the short column may invoke the functionality limit state. The 4-m-long column displaces about 50 cm. This is far greater than the functionality limit (4 m/50 = 8 cm). For the tall column case, significant deformation occurred in the column (as shown in Figure 2-21) resulting in lower strength reserve.

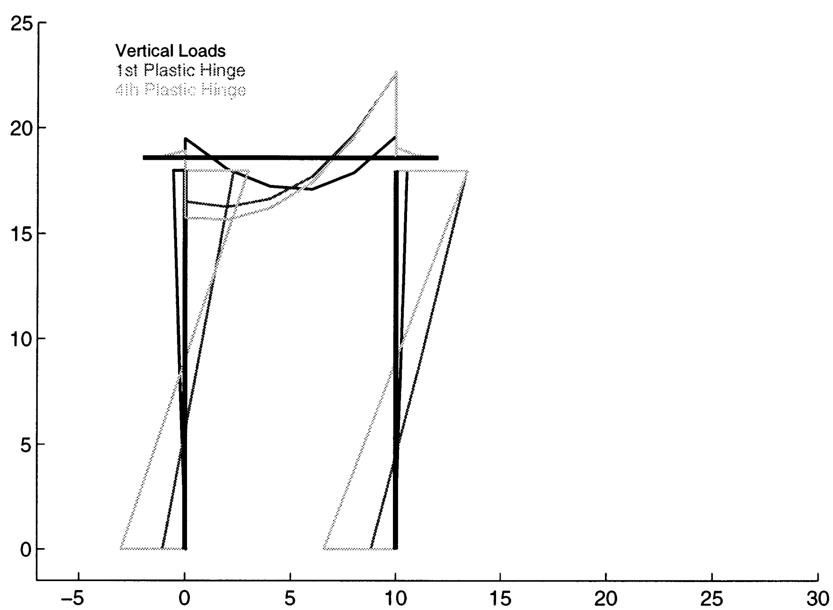


Figure 2-19. Qualitative moment diagram, "height, high, extension, normal."

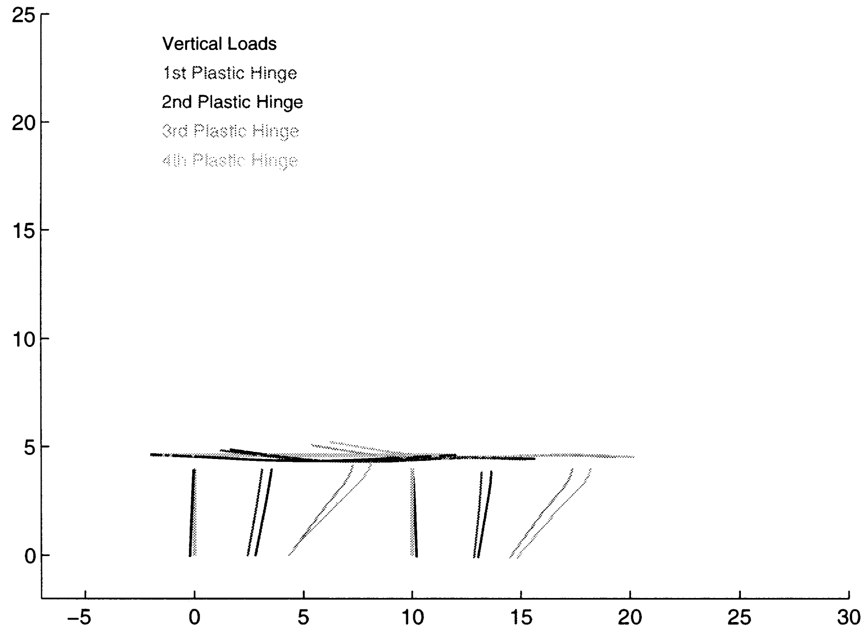


Figure 2-20. Deflected shape (magn.: 20), "height, low, extension, normal."

Influence of Column Stiffness (Column Width)—Interpretation of Case 4 (small column width) and Case 5 (large column width) is more complex. This is because virtually all structure parameters change as the column area changes. These are

- Dead load and hence column axial force and moment,
- Column capacity,
- Column stiffness, and,
- Foundation stiffness.

The last item is particularly noteworthy because the spread foundation stiffness for the structure with small column width is different from the spread foundation stiffness for the structure with large column width. The same results apply to all other foundation and soil types. In an attempt to understand the structural response in Cases 4 and 5, look at the "spread/normal" and "pile/normal" foundations. The foundation stiffnesses for these four structures are summarized below in the order of increasing rotational stiffness:

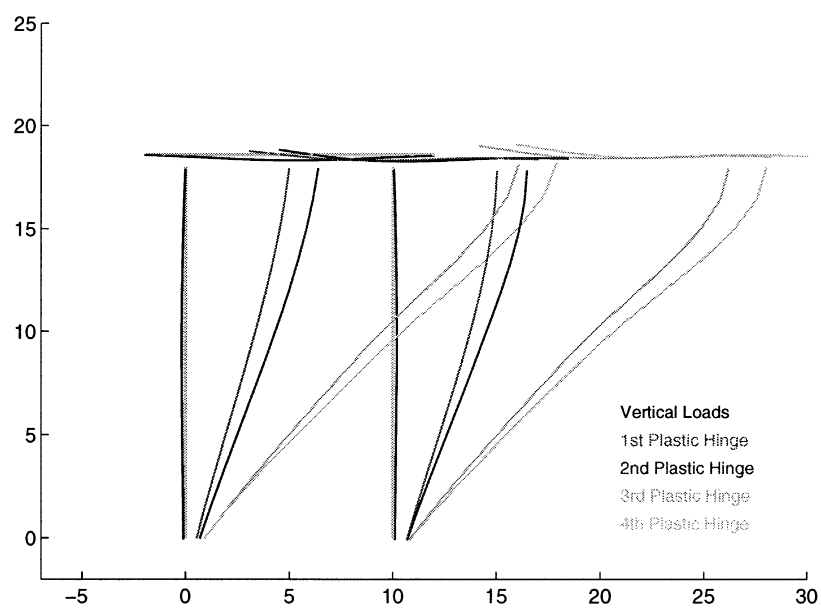


Figure 2-21. Deflected shape (magn.: 20), "height, high, extension, normal."

Column Width	Foundation/Soil Condition	Rotational (kN-m/rad)	Transverse (kN/m)	Vertical (kN/m)
Low	Pile/Normal	235,400	57,240	1,126,000
Low	Spread/Normal	999,000	46,100	61,500
High	Pile/Normal	3,531,000	171,700	3,377,000
High	Spread/Normal	7,120,000	89,900	120,000

From Figure 2-12 the researchers observed that, for increasing column width, the system reserve ratio increases for the "spread/normal," but decreases for the "pile/normal" foundation.

Figures 2-22 to 2-29 plot the bending moment diagrams and the deflected shapes for the four structures. The moment diagrams clearly reflect the differences in the system reserve ratios.

- For the structure with small column width on "spread/normal" foundation, two plastic hinges form virtually simultaneously at top and bottom of Column 2 (see Figures 2-22 and 2-23). At the same time, the other column end moments (in Column 1) are relatively close to their capacity (see Figure 2-22). This situation is similar to that of Figure 2-13c leading to small system reserve.
- For the structure with small column width on "pile/normal" foundation, the moment diagram at the first yield resembles that of Figure 2.13a, and the plastic hinge formations are well spaced as shown in Figures 2-24. This condition leads to large reserve strength as expected.
- On the other extreme is the wide column structure founded on the "spread/normal" foundation. The rotational stiffness is the highest resulting in a moment diagram (Figure 2-26) similar to Figure 2.13b. The plastic hinge formed at the column bottom locations first (Figures 2-26). As explained above, it is now the stiffer foundation/soil system that raises up the system reserve strength by approaching the moment distribution of Figure 2-13b. For the same structure founded on "pile/normal" foundation, the rotational stiffness is lower, and the resulting moment diagram at the first yield (Figure 2-28) approaches Figure 2-13c. The system reserve strength ratio is lower.

As a result, a higher system reserve was observed for the structure on a spread foundation than on a pile foundation.

Sensitivity of System Reserve Ratio with Respect to Vertical Loads—The above discussion focused on the influence of cap, column, and foundation stiffness on the bending moment diagram at first yield, which controls the system reserve ratio. The bending moment distribution, however, is also a function of the vertical loads (dead load and maximum live load effects) acting on the bent and it is of interest to study the sensitivity of the strength reserve ratio with respect to the vertical load. Since vertical loads affect both the lateral force at first yield and when a mechanism forms, the net effect is not immediately clear.

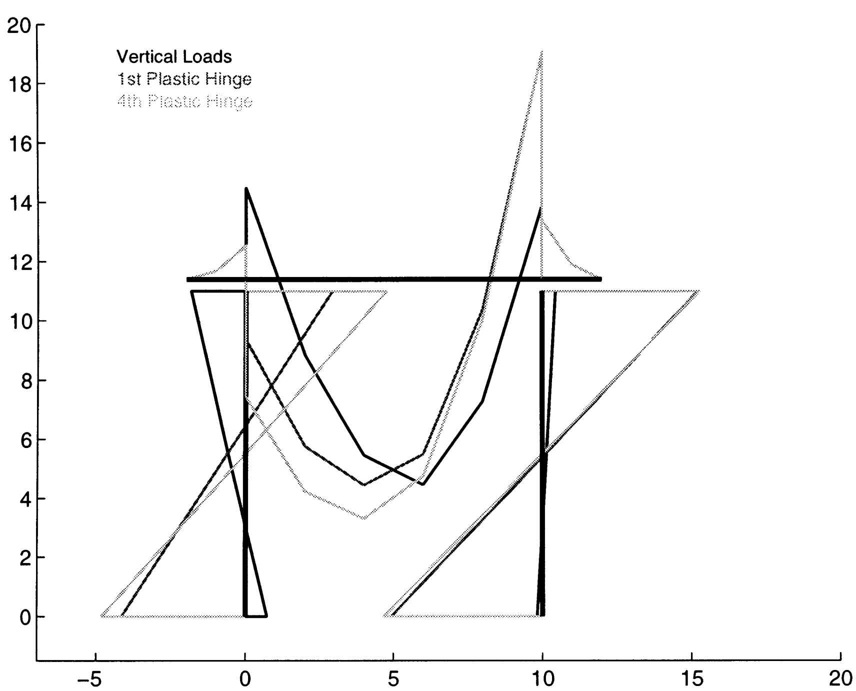


Figure 2-22. Qualitative moment diagram, "width, low, spread, normal."

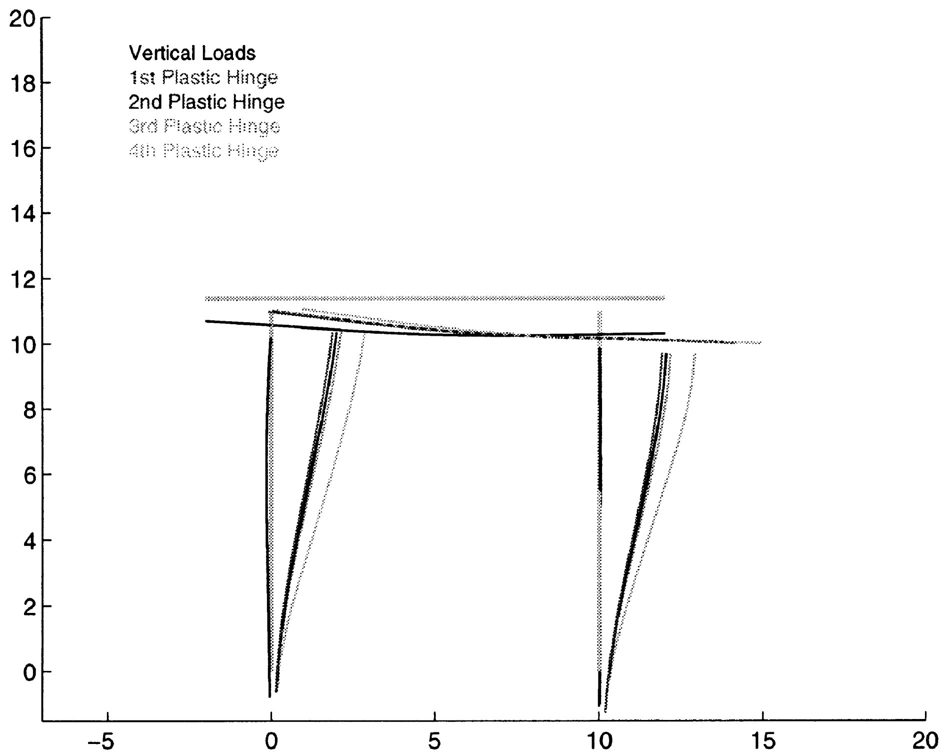


Figure 2.23. Deflected shape (magn.: 20), "width, low, spread, normal."

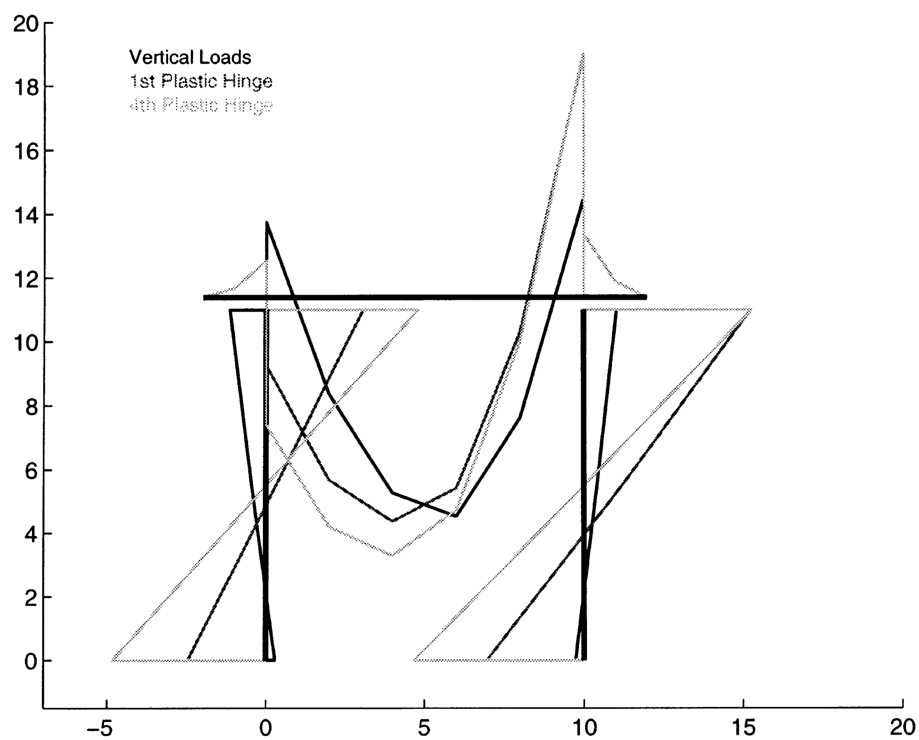


Figure 2-24. Qualitative moment diagram, "width, low, pile, normal."

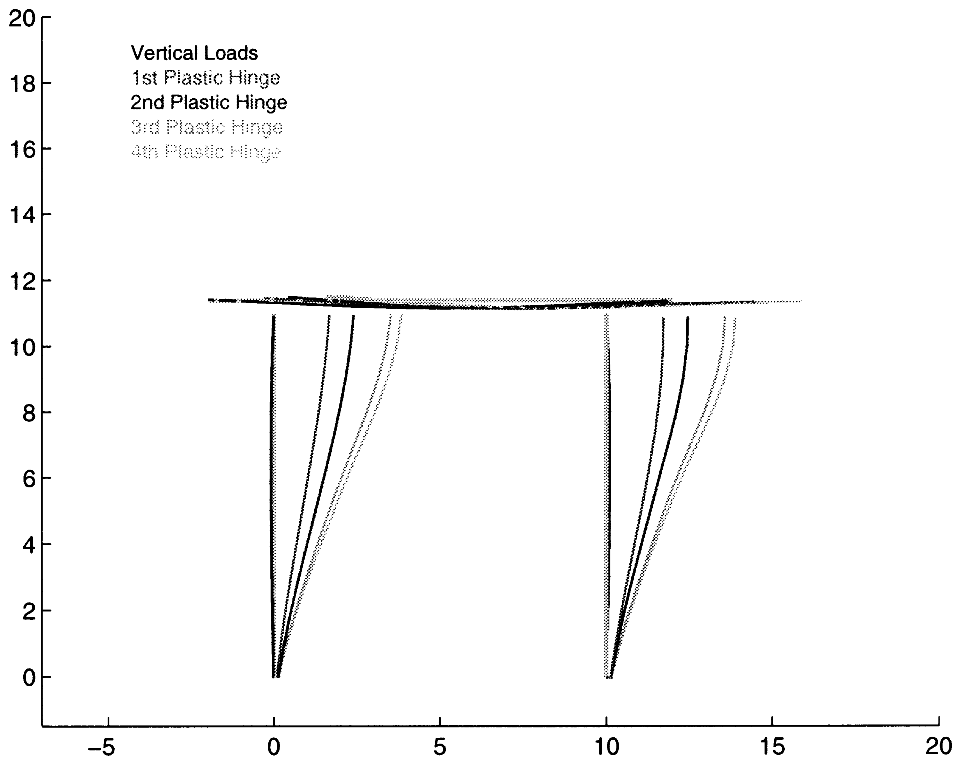


Figure 2-25. Deflected shape (magn.: 20º), "width, low, pile, normal."

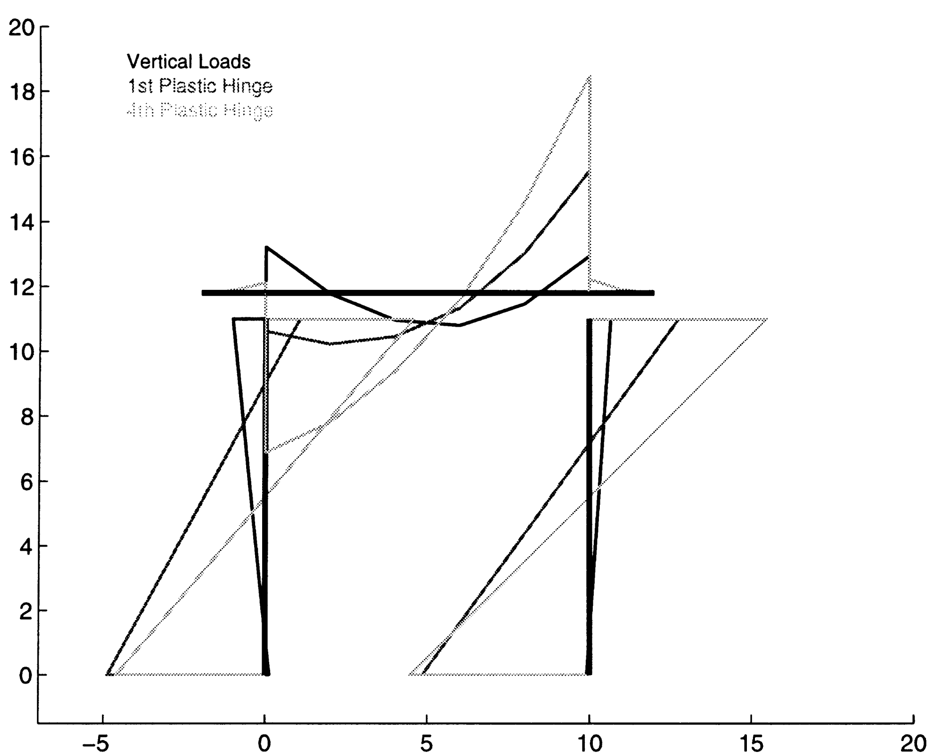


Figure 2-26. Qualitative moment diagram, "width, high, spread, normal."

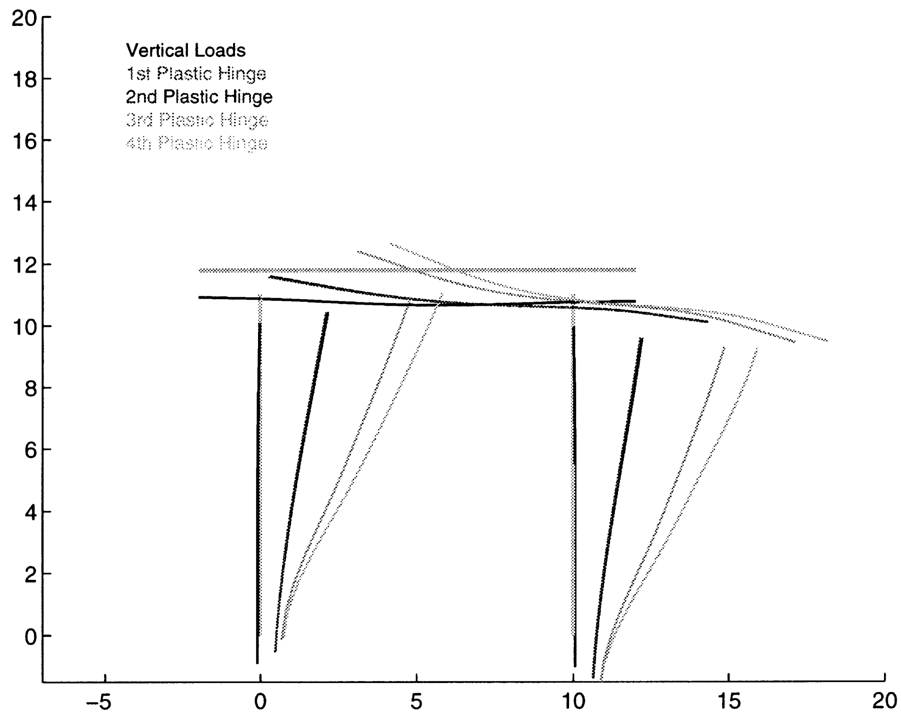


Figure 2-27. Deflected shape (magn.: 20), "width, high, spread, normal."

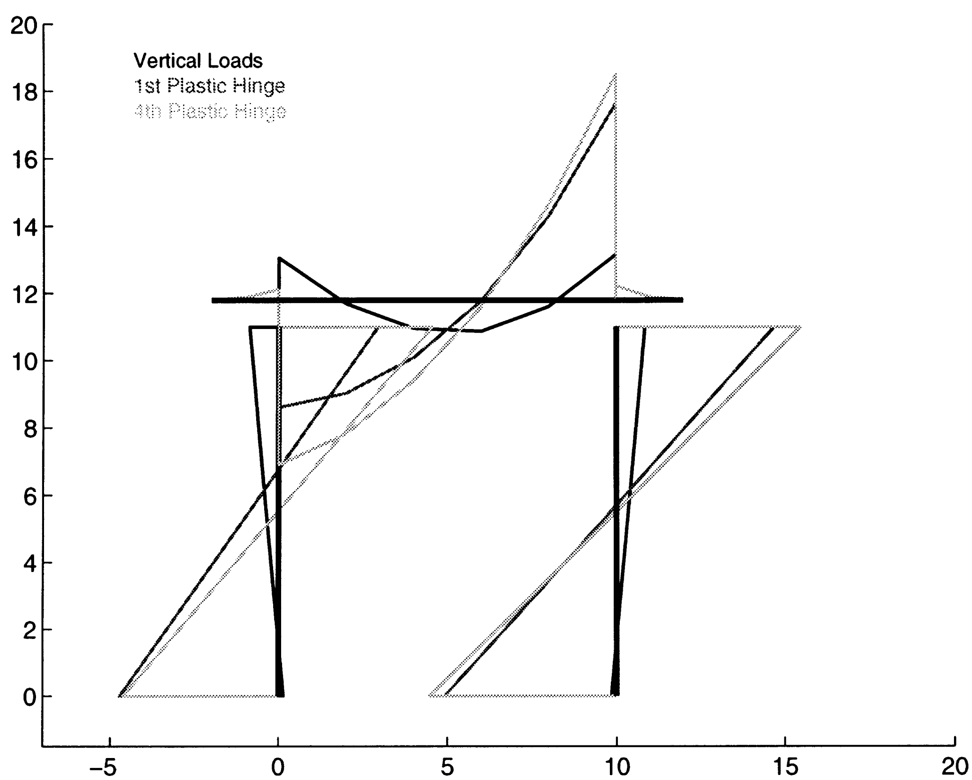


Figure 2-28. Qualitative moment diagram, "width, high, pile, normal."

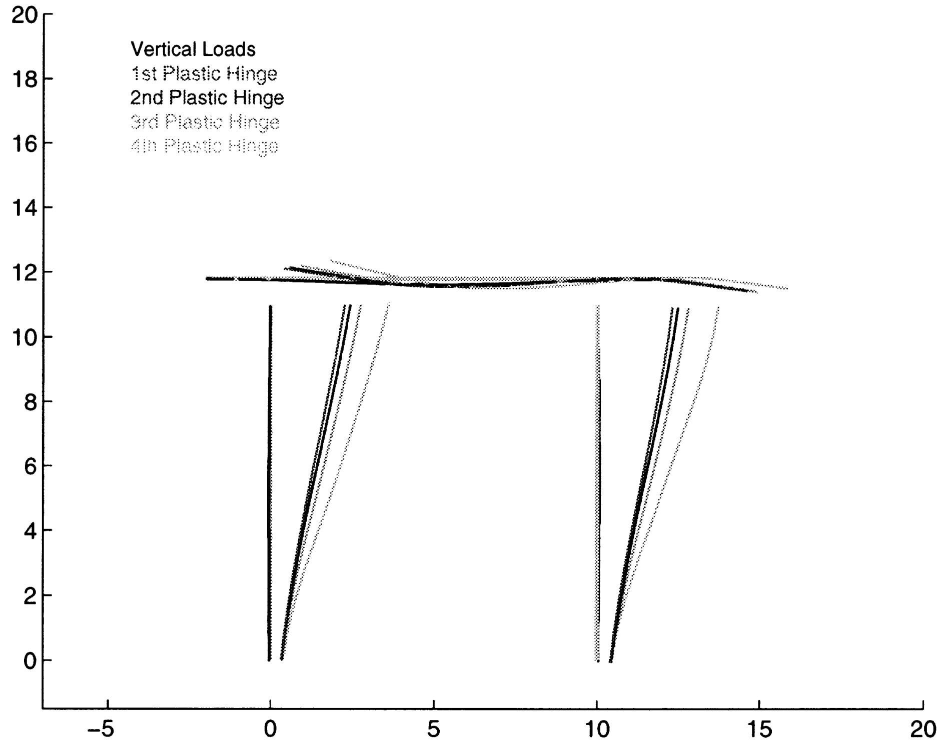


Figure 2-29. Deflected shape (magn.: 20), "width, high, pile, normal."

As an extreme case, the researchers analyzed the two-column bent for the 11 structure variations for lateral forces only (i.e., removing all vertical loads). The results are presented in Figure 2-30 by comparing the system reserve ratios with and without vertical loads. It was observed that in most cases the system reserve ratio decreases slightly when the vertical load is removed.

2.8.3.2 Mechanism and Deformation Limit States

Before the final plastic hinge form, there are significant plastic rotations accumulated in the existing plastic hinges. Plastic hinge rotation demand and capacity depend on a variety of factors such as the amount of longitudinal and transverse steel, and the axial force. Whether or not the structure is able to form a mechanism by developing all possible plastic hinges thus depends on the plastic rotation capacity in the critical plastic hinge, or more precisely on the extreme concrete compressive strain allowed before concrete crushing occurs. In order to judge whether a structure has sufficient deformation capacity to form a mechanism, it is necessary to study the concrete strain demand in the extreme fiber of the critical section.

Uncertainty in assessing the strain demand in a section largely stems from the definition of the plastic hinge length, that is, the zone over which plastic deformation is spread. The longer the plastic hinge length is the smaller the resulting section deformation demand for a given plastic rotation. On the resistance side, limited knowledge regarding how lateral confinement, combined axial force, and bending action, affect the amount of strain that concrete can sustain before crushing, contributes to the uncertainty associated with determining the limiting compressive strain of concrete. Nevertheless, engineering practice has generally used a concrete crushing strain of 0.004 for unconfined columns and 0.015 for confined columns as acceptable values for design purposes.

To understand the demand imposed on the critical section *at the mechanism limit state*, Figure 2-31 plots the extreme concrete compressive strain versus the system reserve ratio. Cases in which the extreme strain is below 5 percent are marked by a "*" symbol followed by two numbers identifying the respective combination of structure (1-11) and foundation (1-8). Cases in which the extreme strain exceeds 5 percent are marked by an upward pointing triangle. As expected, there is a positive correlation between the two response quantities, namely, *with increasing system reserve ratio the strain demand also increases.* For example, a system reserve ratio of more than 1.5 almost always comes at the expense of a maximum concrete strain of more than 1.5 percent. Likewise, system reserve ratios smaller than 1.3 always leads to extreme strains less than 1.2 percent. There is however significant variability.

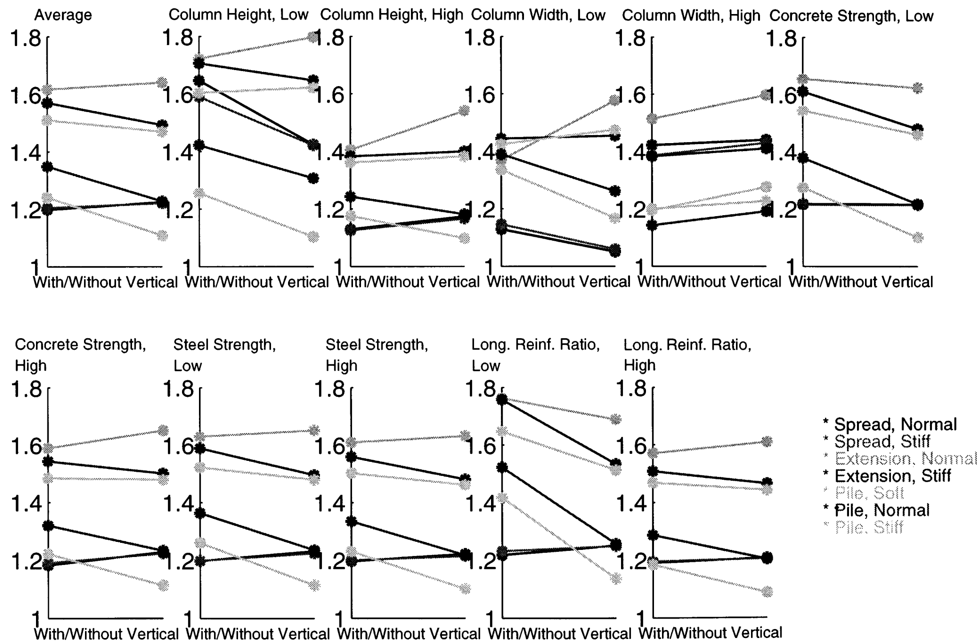


Figure 2-30. Strength reserve ratio with and without vertical loads.

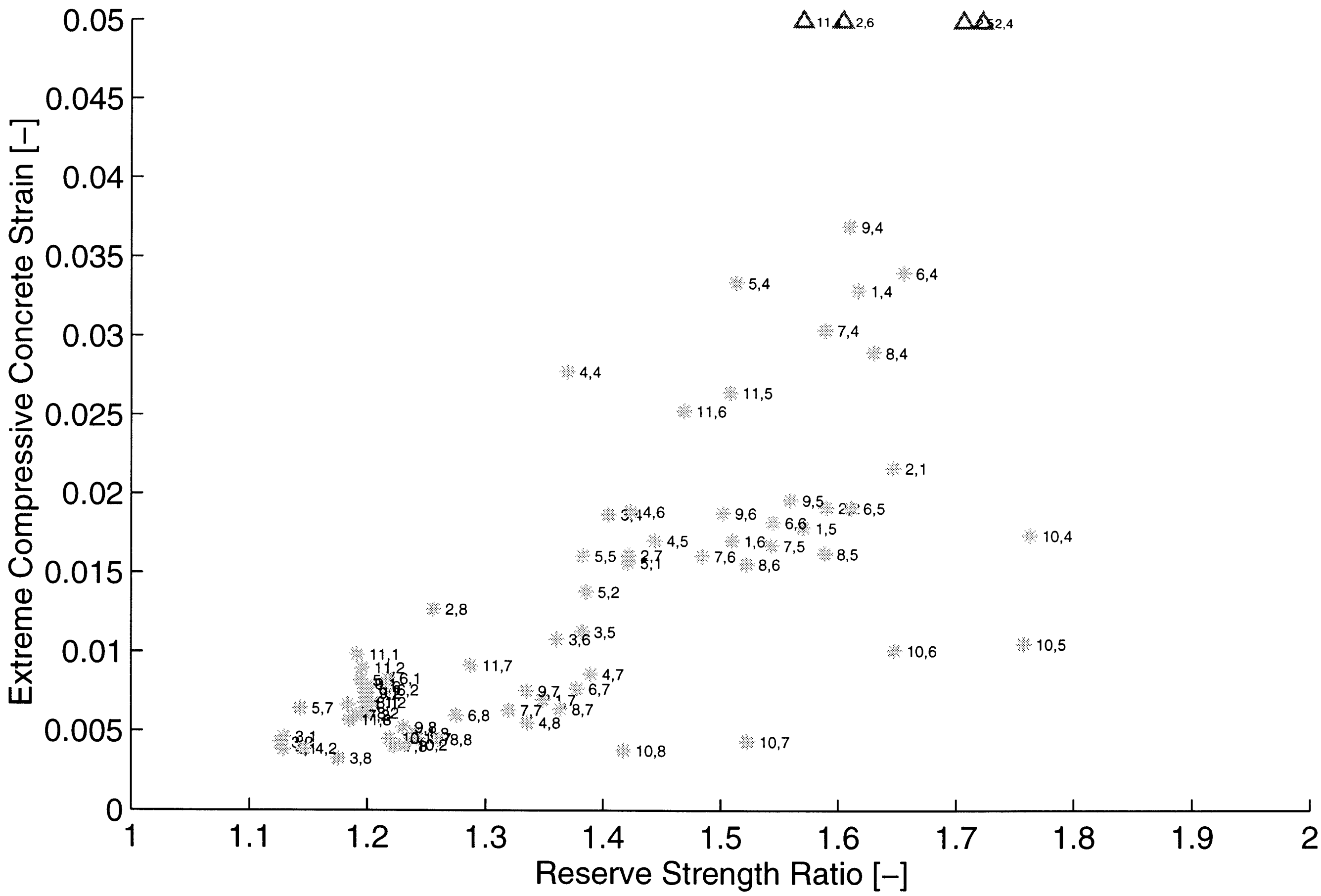


Figure 2-31. Two-column bent, extreme concrete compressive strain versus strength reserve ratio.

As expected, structures with extreme strains above 2 percent at the mechanism limit state are either on the "extension/normal" foundation (foundation type 4) or have properties commonly associated with brittle response, that is, shorter column height or high reinforcement ratio (structure variations 2 and 11). Likewise, structures with large system reserve ratio and modest strains all have a low reinforcement ratio (structure variation 10), a property directly associated with ductile response.

Summary—If the local deformation limit ($\varepsilon_{c\ max}$) is set at 0.015 for confined concrete, the system reserve ratio is limited to about 1.4, except for the case with low longitudinal reinforcement ratio. For unconfined concrete if the local deformation limit ($\varepsilon_{c\ max}$) is set at 0.004, the system reserve ratio is reduced to 1.2 for most cases.

2.8.4 Four-Column Bents

The nonlinear force-displacement relations for all four-column bents are included in Appendix B.3 (Figures B-13 to B-23). Figure 2-32 shows the force displacement relations obtained for the four-column bents with average structural properties. Due to the much smaller column spacing compared to the two-column bent structure, the four-column bents are significantly stiffer. First yield and mechanism events occur at roughly half the displacements observed for the two-column bent.

Figure 2-33 shows the system reserve ratios for the four-column bents. As expected, the higher degree of static indeterminacy and the correspondingly higher number of potential plastic hinges lead to higher system reserve ratios than those for the two-column bents, in most cases. Although it varies widely, on average, the strength reserve ratio for the four-column bent is 0.14 higher than that for the two-column bent. A comparison of Figures 2-12 and 2-33 reveals that differences between the two types of structures is the largest when the column width varies. For the case "narrow column on pile/normal foundation," the system reserve ratio is 1.39 for the two-column bent. For the four-column bent, the system reserve ratio is only 1.16. For the case "wide column on pile/soft foundation," the results are 1.20 for the two-column bent and 1.74 for the four-column bent.

The maximum concrete compressive strain at the extreme fiber required for the mechanism limit state to take place is shown in Figure 2-34. The strains observed at the points where mechanisms form in four-column bents are generally similar to those obtained for the two-column bents. A notable difference is that the four-column bents show significantly more cases (14) of extreme strains exceeding 5 percent than the two-column bent (5 cases). This indicates that, for four-column bents, part of the higher system reserve that develops *is associated with much larger local deformation demand* (higher strains) than those observed for two-column bents.

The system reserve ratios, R_u, and the maximum extreme-fiber strain, $\varepsilon_{c,max}$, are summarized in Table 2-6 for all the cases considered in the parametric studies and for both two-column bents (Figure 2-31) and four-column bents (Figure 2-34). These results are used in the reliability calculation. This comparison shows the more redundant nature of four-column bents relative to two-column bents. Further, it demonstrates that the higher redundancy requires much greater local deformation, and the higher redundancy can be realized only if the material and design details lead to sufficiently ductile columns.

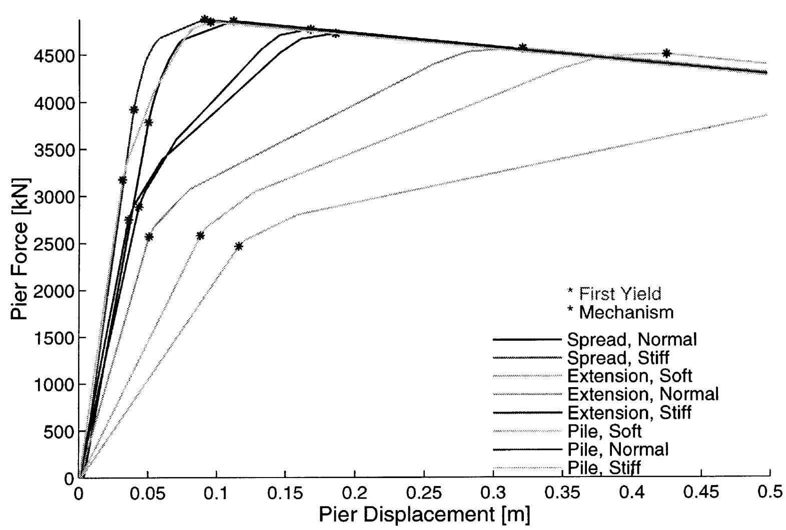


Figure 2-32. Four-column bent, force-displacement relation, average properties.

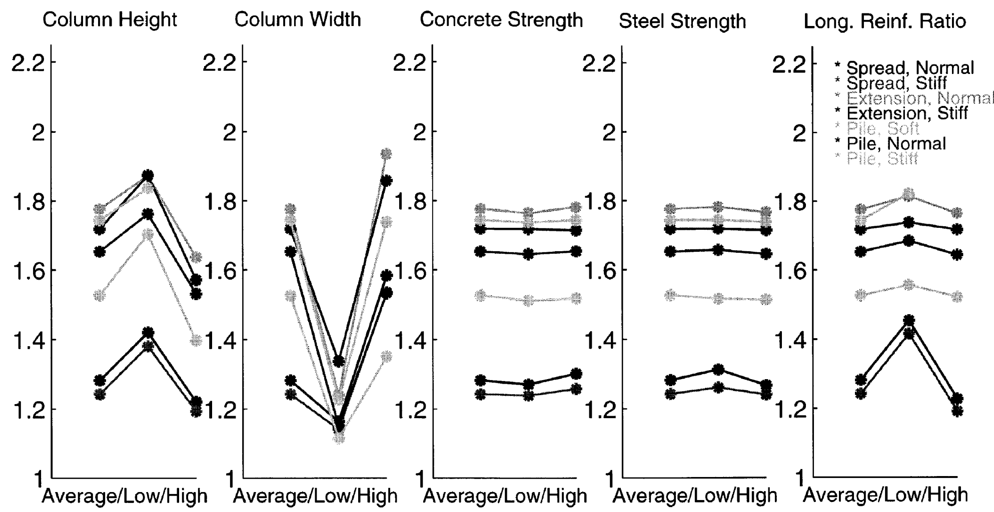


Figure 2-33. Four-column bent, reserve strength ratio.

2.8.4.1 Redundancy of Damaged Structures

A bridge may be partially damaged because of foundation scour or collision by a vessel, truck, debris, and so on. It is therefore necessary to examine whether the multicolumn bent would still remain redundant under such damage conditions. For this study, a four-column bent with average structural parameters (Case 1) founded on a spread foundation and normal soil condition is considered. For the intact structure, the first component damage occurs at a lateral load of 3787 kN (the ultimate lateral strength for confined concrete is controlled by the local deformation limit (i.e., exceeding the maximum strain of 1.5 percent at extreme fiber). The ultimate lateral load is 4801 kN, and the system reserve ratio is 1.27.

When the damaged column is completely removed from the model, the ultimate lateral strength is determined. For the case with a damaged interior column, the ultimate strength is 3,213 kN resulting in a system reserve ratio for damaged conditions R_d of 0.85 (=3,213kN/3,787kN); and for the case of a damaged exterior column, the ultimate strength is 3,324 kN resulting in a R_d of 0.88. These values correspond to the formation of a collapse mechanism. A careful review of the strain in the concrete, however, reveals that concrete crushing would occur at a load equal to 2,187 kN (if unconfined) when the exterior column is removed resulting in a damaged system reserve ratio for damaged conditions equal to 0.58 (2,187kN/3,787kN). Although all these values are lower than 1.0, they illustrate the fact that damaged bridges can still support a substantial lateral load before collapsing. Thus, some reserve strength is still available in the columns to carry a portion of the maximum expected loads. Such situations are highly desirable to ensure some traffic safety after a damage situation occurs.

However, for the columns of a damaged substructure to sustain such loads, it is critical that the column cap be able to redistribute the load that was originally in the damaged column to the remaining (surviving) columns. It is found that this is not always possible when the exterior column is the one that is damaged. In fact, removal of one column would produce a premature shearing failure in the bent cap. The cap beam typically does not have the additional strength. Figures 2-35 and 2-36 show moment diagrams for the two damaged conditions involving exterior and interior columns, respectively. It is shown that the cap beam may not survive the applied vertical loads.

Partial Damage—For a less severely damaged condition, the column may loose its flexural stiffness and strength but may still retain the axial load capacity. This is simulated by replacing the damaged column by an axial truss element. The ultimate limit state is now controlled by the limiting local deformation. For confined concrete, the ultimate strength is 3,453 kN for the case with damaged interior column and 3,479 kN for the case with damaged exterior column. The damaged condition system reserve ratios become, respectively, 0.91 and 0.92. This damage scenario obviously produces higher system reserve ratios.

In subsequent parts of this report, a damage scenario consisting of a complete removal of one column is chosen as the base scenario for damaged bridge substructures. This scenario was selected in order to be compatible with the damaged scenario used in *NCHRP Project 12-36* to study superstructure redundancy. Criteria specifying the system reserve ratios that damaged substructures should be able to sustain are proposed in Chapter 3, based on the results of the investigation carried out in this section. However, the criteria require that column caps be able to transfer the load to the remaining portion of a damaged bent. As seen in this example, this is by no means ensured for most "typical" bridge configurations.

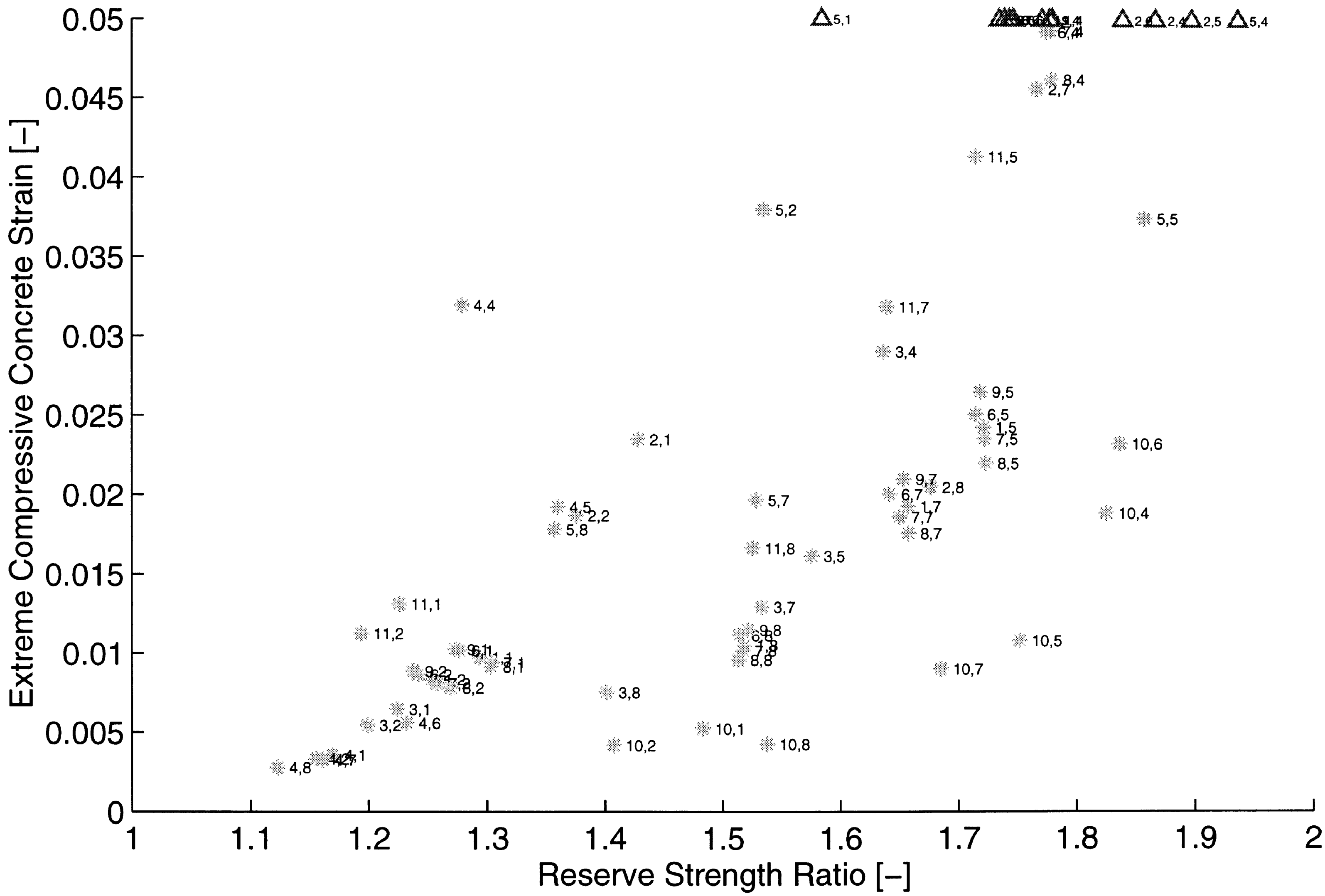


Figure 2-34. Four-column bent, extreme concrete compressive strain versus strength reserve ratio.

TABLE 2-6 Strength reserve ratio, R_u, and extreme fiber strain, ε_{max} (%), for two-column bents and four-column bents

Columns:

1. case number
2. case descriptor
3. strength reserve ratio for two-column bent (mechanism limit state only)
4. extreme concrete compressive strain [%] at mechanism limit state for two-column bent
5. strength reserve ratio for four-column bent (mechanism limit state only)
6. extreme concrete compressive strain [%] at mechanism limit state for four-column bent

NaN: strain larger than 5%

~~0.00~~: limit state not reached within the imposed displacement.

1	2	3	4	5	6
		Two-Col. Bent*		Four-Col. Bent	
Case	Description	R_u	ε_{max} (%)	R_u	ε_{max} (%)
1	all_average\spread\normal\	1.20	0.74	1.29	0.97
2	all_average\spread\stiff\	1.20	0.68	1.25	0.83
3	all_average\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
4	all_average\extension\normal\	1.62	3.29	1.78	NaN
5	all_average\extension\stiff\	1.57	1.78	1.72	2.43
6	all_average\pile\soft\	1.51	1.70	1.75	NaN
7	all_average\pile\normal\	1.35	0.70	1.66	1.93
8	all_average\pile\stiff\	1.24	0.49	1.52	1.05
9	col_height\low\spread\normal\	1.65	2.16	1.43	2.35
10	col_height\low\spread\stiff\	1.59	1.91	1.38	1.87
11	col_height\low\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
12	col_height\low\extension\normal\	1.72	NaN	1.87	NaN
13	col_height\low\extension\stiff\	1.71	NaN	1.90	NaN
14	col_height\low\pile\soft\	1.60	NaN	1.84	NaN
15	col_height\low\pile\normal\	1.42	1.61	1.77	4.56
16	col_height\low\pile\stiff\	1.26	1.27	1.68	2.05
17	col_height\high\spread\normal\	1.13	0.46	1.22	0.65
18	col_height\high\spread\stiff\	1.13	0.43	1.20	0.54
19	col_height\high\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
20	col_height\high\extension\normal\	1.40	1.87	1.64	2.90
21	col_height\high\extension\stiff\	1.38	1.13	1.58	1.61
22	col_height\high\pile\soft\	1.36	1.08	~~0.00~~	~~0.00~~
23	col_height\high\pile\normal\	1.24	0.45	1.53	1.29
24	col_height\high\pile\stiff\	1.18	0.33	1.40	0.75
25	col_width\low\spread\normal\	1.13	0.39	1.17	0.36
26	col_width\low\spread\stiff\	1.15	0.39	1.16	0.33
27	col_width\low\extension\soft\	1.04	NaN	~~0.00~~	~~0.00~~
28	col_width\low\extension\normal\	1.37	2.77	1.28	3.19
29	col_width\low\extension\stiff\	1.44	1.70	1.36	1.92
30	col_width\low\pile\soft\	1.42	1.89	1.23	0.56
31	col_width\low\pile\normal\	1.39	0.86	1.16	0.33
32	col_width\low\pile\stiff\	1.34	0.55	1.12	0.28
33	col_width\high\spread\normal\	1.42	1.56	1.58	NaN
34	col_width\high\spread\stiff\	1.39	1.38	1.53	3.80
35	col_width\high\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
36	col_width\high\extension\normal\	1.51	3.33	1.94	NaN
37	col_width\high\extension\stiff\	1.38	1.61	1.86	3.75
38	col_width\high\pile\soft\	1.20	0.78	1.73	NaN
39	col_width\high\pile\normal\	1.14	0.64	1.53	1.96
40	col_width\high\pile\stiff\	1.19	0.82	1.36	1.78
41	f'c\low\spread\normal\	1.22	0.83	1.28	1.02
42	f'c\low\spread\stiff\	1.22	0.76	1.24	0.86
43	f'c\low\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
44	f'c\low\extension\normal\	1.66	3.40	1.77	4.92
45	f'c\low\extension\stiff\	1.61	1.91	1.71	2.51

(*continued on next page*)

TABLE 2-6 *(Continued)*

1	2	3	4	5	6
		Two-Col. Bent*		Four-Col. Bent	
Case	Description	R_u	ε_{max} (%)	R_u	ε_{max} (%)
46	f'c\low\pile\soft\	1.54	1.82	1.74	NaN
47	f'c\low\pile\normal\	1.38	0.77	1.64	2.00
48	f'c\low\pile\stiff\	1.27	0.60	1.51	1.11
49	f'c\high\spread\normal\	1.18	0.67	1.30	0.94
50	f'c\high\spread\stiff\	1.19	0.60	1.26	0.81
51	f'c\high\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
52	f'c\high\extension\normal\	1.59	3.04	1.78	4.93
53	f'c\high\extension\stiff\	1.54	1.67	1.72	2.35
54	f'c\high\pile\soft\	1.48	1.60	1.75	NaN
55	f'c\high\pile\normal\	1.32	0.63	1.65	1.86
56	f'c\high\pile\stiff\	1.22	0.40	1.52	1.02
57	fy\low\spread\normal\	1.20	0.68	1.30	0.91
58	fy\low\spread\stiff\	1.20	0.61	1.27	0.78
59	fy\low\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
60	fy\low\extension\normal\	1.63	2.89	1.78	4.62
61	fy\low\extension\stiff\	1.59	1.62	1.72	2.20
62	fy\low\pile\soft\	1.52	1.55	1.74	NaN
63	fy\low\pile\normal\	1.36	0.64	1.66	1.76
64	fy\low\pile\stiff\	1.26	0.45	1.51	0.96
65	fy\high\spread\normal\	1.20	0.80	1.27	1.03
66	fy\high\spread\stiff\	1.20	0.74	1.24	0.89
67	fy\high\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
68	fy\high\extension\normal\	1.61	3.69	1.78	NaN
69	fy\high\extension\stiff\	1.56	1.96	1.72	2.65
70	fy\high\pile\soft\	1.50	1.88	1.74	NaN
71	fy\high\pile\normal\	1.33	0.75	1.65	2.10
72	fy\high\pile\stiff\	1.23	0.52	1.52	1.15
73	percent_long\low\spread\normal\	1.22	0.45	1.48	0.52
74	percent_long\low\spread\stiff\	1.23	0.41	1.41	0.42
75	percent_long\low\extension\soft\	1.56	4.28	1.80	NaN
76	percent_long\low\extension\normal\	1.76	1.74	1.82	1.89
77	percent_long\low\extension\stiff\	1.76	1.05	1.75	1.08
78	percent_long\low\pile\soft\	1.65	1.01	1.84	2.32
79	percent_long\low\pile\normal\	1.52	0.43	1.68	0.90
80	percent_long\low\pile\stiff\	1.42	0.38	1.54	0.43
81	percent_long\high\spread\normal\	1.19	0.98	1.23	1.31
82	percent_long\high\spread\stiff\	1.20	0.90	1.19	1.13
83	percent_long\high\extension\soft\	~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~
84	percent_long\high\extension\normal\	1.57	NaN	1.77	NaN
85	percent_long\high\extension\stiff\	1.51	2.64	1.71	4.13
86	percent_long\high\pile\soft\	1.47	2.52	~~0.00~~	~~0.00~~
87	percent_long\high\pile\normal\	1.29	0.91	1.64	3.18
88	percent_long\high\pile\stiff\	1.18	0.57	1.53	1.66

Hence, typical bridge configurations are deemed to be nonredundant for damaged conditions unless they are specifically designed for such an eventuality.

2.9 SUMMARY

Results of extensive parametric studies are provided in Appendix B for two-column bents and four-column bents of typical configurations and material properties as collected from the survey of U.S. state DOTs. The four-column bents are intended to model the behavior of all multicolumn bents. Single-column bents and pier walls are found to be nonredundant. The effect of the superstructure on bridge substructure redundancy is found to be negligible for most typical constructions. The emphasis in this study is on failures due to bending of the columns as shearing failures are nonredundant.

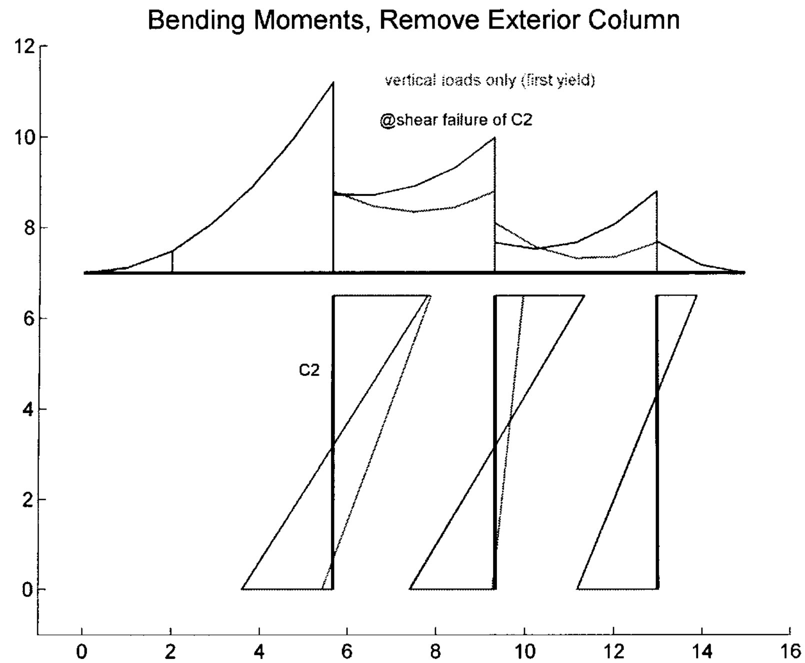


Figure 2-35. Moment diagram for damaged condition—exterior column damaged.

This chapter interprets the behavior observed during the analysis process. The interpretation of the results focused on the mechanism limit state for the sake of studying the effect of various parameters on one particular failure criterion, although the crushing of the concrete in confined and unconfined columns was also analyzed. This chapter illustrates that the most critical factor for predicting bridge substructure redundancy is the flexibility of the foundation/soil system relative to the structure. As shown in Figure 2-13, this relative flexibility controls the shape of the moment diagram when the *first component damage* is reached. This moment diagram greatly influences the system reserve ratio. The P-Δ is found to be less critical because its effect becomes significant only when very large displacement levels are observed

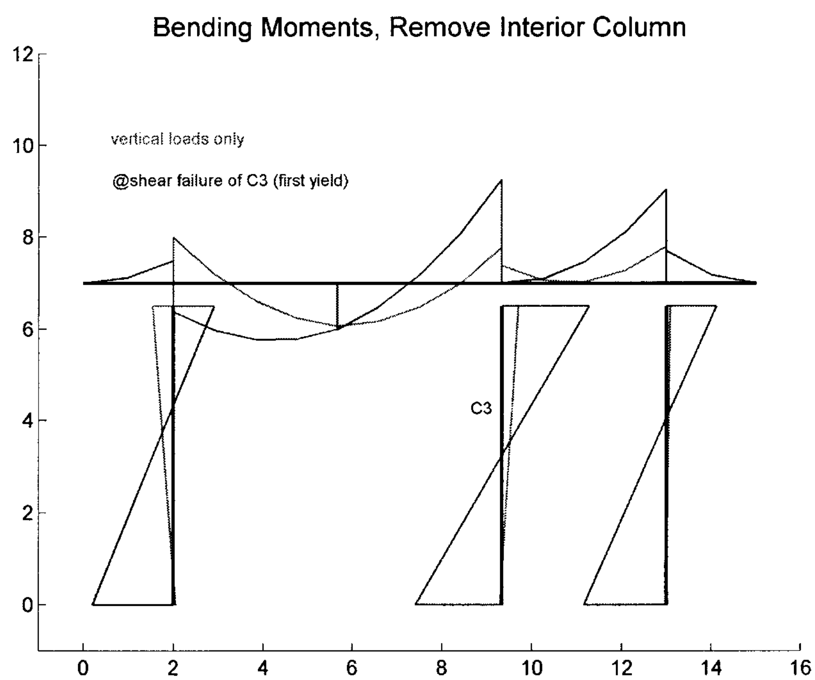


Figure 2-36. Moment diagram for damaged condition—interior column damaged.

or when concrete crushing strain limits are permitted that are higher than those obtained in typical constructions.

For the ultimate damage states, both the cases of system mechanism and local deformation limit (concrete crushing) are considered. High system reserve ratios are only possible if all critical sections, including the cap beam, are sufficiently ductile to prevent any premature rupture.

For the functionality limit states, several displacement limits are compared. Three of these correspond to different levels of total cap displacement. The fourth limit consists of comparing the displacement of the top of the cap relative to the base of the bent. The criterion selected corresponds to a total lateral displacement equal to column height/50. This criterion is selected because, for reasonably stiff foundation levels, it gives results of the same order of magnitude as those of the loads that cause the crushing of concrete in confined columns. The purpose of the functionality criterion is to control excessive total displacements, including those due to the foundation/soil system, so that bridges may still be usable even if they are subjected to high lateral loads.

All the results of the analyses performed in this study are listed in Appendix B. These results are used in the calibration of the system factors and the criteria used to develop them as presented in Chapter 3.

CHAPTER 3

RELIABILITY CALIBRATION OF SUBSTRUCTURE REDUNDANCY

3.1 INTRODUCTION

In this study, following the work of NCHRP Project 12-36 as described in *NCHRP Report 406* (Ghosn and Moses, 1998), redundancy is defined as the capability of a bridge substructure to continue to carry loads after the failure of one of its members. This means that the substructure may have additional reserve strength such that the failure of one member does not result in the failure of the complete substructure system. The bridge substructure, in this context, includes columns, piles or footings as well as the supporting soil. Member failure could be either brittle or ductile. Member failure could be caused by the application of large loads or the sudden failure of one element due to fatigue, brittle fracture, or an accident such as a collision by a truck, ship, or debris. The failure may also be due to scour (for soils).

Following the procedure described in *NCHRP Report 406*, one convenient method to represent the redundancy of bridge substructure systems would consist of developing a set of system factors that can be included as specifications in bridge design and evaluation codes. Alternatively, a direct analysis approach would evaluate redundancy using a structural model and a finite element analysis program that can perform a pushover nonlinear analysis accounting for the elastic and inelastic behavior of the substructure-foundation system.

This chapter explains how the system factors are calibrated and codified to account for the redundancy of bridge substructures. The outlined calibration process first performs a direct analysis on typical substructure configurations. The level of redundancy inherent in each configuration is then evaluated and quantified. System factors are chosen so that bridge substructure configurations that do not satisfy a minimum level of redundancy are penalized by requiring that their primary members (columns) be designed to higher safety standards than the members of substructure configurations that satisfy the minimum level of redundancy. For systems that exceed the minimum level of redundancy, the system factors actually reduce the member capacity requirements. The system factors are incorporated in the LRFD checking equation as modifiers to the resistance factors.

3.2 GENERAL APPROACH

As previously defined, redundancy is the capability of the structure to continue to carry loads after the failure of one main member. Thus, a comparison between the ultimate load capacity and the capacity of the system to resist the first member failure provides a direct measure of the level of bridge redundancy. Based on the assumptions described in Chapter 2, the parametric studies are focused on the failure analysis of substructure columns. Hence, the subsequent reliability analysis addresses only column failures. The effect of soil flexibility is addressed by monitoring the *total* bent displacement that includes the displacement of foundation and soil and the displacement due to the bending of the columns. Shearing failures are brittle and do not provide any reserve strength and thus substructure systems are considered nonredundant for shear.

Ultimate Limit State—The traditional analysis of bridge substructures consists of applying the gravity (dead and vertical live) loads and then applying a lateral load and verifying that the substructure capacity is higher than the applied load effects. The reserve capacity is measured by increasing the lateral load until a failure occurs. As defined in Chapter 2, the load and the load factor leading to the first component failure are F_1 and LF_1; where $F_1 = LF_1 \cdot W_n$, W_n being the applied design (nominal) lateral load. The load and load factor that cause the complete substructure system failure are F_u and LF_u ($F_u = LF_u W_n$). The system reserve ratio for the ultimate limit state, R_u, is defined as:

$$R_u \equiv \frac{F_u}{F_1} = \frac{LF_u}{LF_1} \tag{3.1}$$

According to this definition, *the system reserve ratio* R_u *is a nominal (deterministic) measure of bridge redundancy*. For example, when the ratio R_u is equal to 1.0, the ultimate capacity of the substructure system is equal to the capacity of the substructure to resist first component failure. Such a bridge is nonredundant. As R_u increases, the level of bridge redundancy increases.

LF_u and LF_1 can be calculated by performing the static nonlinear analysis of bridge substructure systems using the program PIERPUSH described in Chapter 2. Alternatively, other nonlinear analysis programs, such as FLORIDA-PIER and any other commercial or specialized finite element analysis program with nonlinear analysis capability may be used to perform the pushover analysis.

During the pushover analysis, the nominal (or design) lateral and gravity (dead plus live) loads are applied on the substructure first, and the lateral load is increased beyond the linear-elastic limit until the first member (column) reaches its limit strength capacity. The analysis procedure accounts for the P-Δ moments produced from the vertical loads. The loading is further increased beyond the first member failure until the ultimate capacity of the system is reached. This ultimate capacity may, for example, correspond to the formation of a plastic collapse mechanism in the bent. Or, if any column's ductility is exhausted before the formation of the mechanism, the column will crush and immediate unloading of the system may ensue. In this study, the ultimate capacity is defined as the load that would cause the formation of a mechanism or that will cause the crushing of any column in the system, whichever limit is reached first. The system reserve ratio, R_u, as defined in Equation 3.1, reflects the level of the reserve strength provided by the system.

Functionality Limit State—In certain cases, the lateral load applied on a bridge substructure may induce large total lateral displacements rendering the bridge unfit for traffic passage even before a collapse mechanism or concrete crushing occurs. Thus, the bridge becomes "nonfunctional" even if the ultimate strength limit state is not reached. Such situations may occur when the soil fails or when the soil/foundation stiffness is small. Note that the total displacement criterion includes the displacements in the soil and foundation as well as the column deformation itself. In this study, the functionality limit state is defined as the load at which the total lateral displacement reaches a value equal to H/50, where H is the clear column height. This lateral displacement limit was observed to occur after columns of typical bridge configurations have exceeded their elastic limits. The lateral load corresponding to this limit state criterion is denoted by F_f and can be obtained by multiplying the original nominal load (W_n) by a load factor, LF_f, that is, $F_f = LF_f \cdot W_n$. Using a definition similar to that provided in Equation 3.1, a system reserve ratio for the functionality limit state is defined as

$$R_u \equiv \frac{F_u}{F_1} = \frac{LF_u}{LF_1} \qquad (3.2)$$

A system failure may occur because of a variety of modes. These failure modes include the formation of a collapse mechanism, the crushing of the concrete, the failure of the soil-foundation system, a large lateral displacement rendering the bridge nonfunctional, a brittle failure in a column due to shear, and so on. In this study, each of these failure modes is analyzed separately. Based on the results described in Chapter 2 and Appendix B, the collapse mechanism and the crushing of concrete in the column are treated together such that the limit state reached first is defined as the ultimate limit state. The functionality limit state, as defined above, is treated separately in order to allow the evaluating engineer the option of ignoring it if, in certain situations, the bridge substructure is allowed to exhibit high levels of lateral displacements. Failure of the soil-foundation system is not directly addressed, though this failure is implicitly considered in the functionality limit state that accounts for the soil's flexibility. Shear failures being brittle provide no levels of redundancy producing a system reserve ratio $R_u = 1.0$ in all the cases where shear controls. Similarly, pullout failures and connection failures are considered brittle and are also associated with a system reserve ratio $R_u = 1.0$.

Ultimate Limit State for Damaged Condition—In addition to checking the reserve strength ratio of intact substructures, a check of the redundancy of bridge substructures, after the loss of one of their columns (e.g., the washing away of one column's supporting foundation due to scour, or the damage of a concrete column due to an accident) may be performed to check the "robustness" of substructures. This scenario is defined as the damaged condition. The redundancy of the damaged substructure can be performed using the same analysis procedure outlined above. The analysis would consist of finding the ultimate lateral load, F_d that will produce a collapse mechanism in the remaining portion (after the removal of the damaged column) of the substructure or the crushing of the concrete in one of the remaining columns. This lateral load may be obtained by the load factor LF_d, such that $F_d = LF_d \cdot W_n$. The system reserve ratio for the damaged condition is defined as

$$R_u = \frac{LF_u}{LF_1} \qquad (3.3)$$

where LF_1 is the same lateral load factor used in Equations 1 and 2, which is the load factor that causes the failure of the first column in the intact structure.

Chapter 2 demonstrated that the brittle failure of one column for substructures, designed according to current practice, would result in the collapse of the complete substructure system since the column caps are normally unable to transfer the vertical loads to the remaining (surviving) columns. Hence, the damage scenario defined in *NCHRP Report 406* as the brittle failure of one member (one column in this context) is not directly addressed in this study that is concerned with studying the redundancy of "typical designs." The direct analysis method can be used so that under appropriate conditions an engineer will be able to determine whether a particular substructure would provide sufficient levels of redundancy after the brittle failure of one of its supporting columns.

Calibration for Codified Implementation—In a reliability-based approach such as the LRFD method, the calibration of the system factors should account for the system reserve ratio of bridge substructures expressed by R_u and R_f, as well as the uncertainties associated with determining member and system capacities, material properties, and the maximum expected loads.

Structural reliability methods have been developed to account for load and resistance uncertainties but must be simplified for practical implementation on a regular basis. To facilitate the implementation of reliability methods, code-writing groups have bridged the gap between reliability theory and the deterministic approach by calibrating design and evaluation codes that provide uniform levels of reliability. This technique, known as Level I reliability analysis, was used in the development of the AASHTO LRFD Specifications. In Level I methods, the reliability model is transparent to the end user of the code. That is, while the load and resistance factors are calibrated based on reliability models, the end user of the code performs a deterministic check of the member safety using these load and resistance factors without referring to reliability theory. A similar approach was used in *NCHRP Report 406* to calibrate system factors to account for the redundancy of different superstructure configurations. The system factors are then incorporated as part of the member resistance factors in the standard LRFD checking procedure.

This study uses the same approach described in *NCHRP Report 406* to obtain reliability-based measures of bridge substructure redundancy. These measures are then used to calibrate a deterministic (Level I) format that implicitly accounts for the resistance and load uncertainties.

3.3 RELIABILITY-BASED MEASURES OF REDUNDANCY

The measure of safety used in the development of the AASHTO LRFD Specifications is the reliability index, β. The reliability index can be used as a measure of the reliability of structural members as well as structural systems. The reliability index accounts for both the margin of safety implied by the design procedure, and the uncertainties in estimating member strengths and applied loads.

Typically in LRFD specifications, the resistance and load factors are calibrated to satisfy a target reliability index, $\beta = 3.5$ for individual members. This calibration would produce a probability of member failures of 2.33×10^{-3}. Actually, the presence of redundancy would lead to higher system reliability levels. For superstructures, for example, the target system reliability level was 0.85 higher than the member reliability, that is, $\beta = 4.35$ was required corresponding to a much lower system probability of failure, 6.81×10^{-6}, i.e., three order of magnitude lower than the member failure probability.

Component Reliability—Assume that the capacity of the substructure to resist the first member failure (represented by the load factor LF_1) and the applied maximum lifetime lateral load (represented by the factor LW) are random variables that follow lognormal distributions. Then, the reliability index, β_{member}, for the failure of the first member can be expressed using a lognormal format as follows:

$$\beta_{\text{member}} = \frac{\ln \dfrac{\overline{LF}_1}{\overline{LL}}}{\sqrt{V_{LF}^2 + V_{LW}^2}} \tag{3.4}$$

where $\overline{LF}_1$ is the mean value of the lateral load factor that will cause the first member failure in the substructure. $\overline{LW}$ is the mean value of the lateral load bias factor (i.e., it is the factor by which the nominal lateral load is multiplied to obtain the mean value of the expected maximum lateral load). V_{LF} is the coefficient of variation (COV) (defined as the ratio of the standard deviation to the mean value) of the lateral load factor LF_1. It reflects the level of uncertainty associated with estimating the demand associated with first member failure, LF_1. V_{LW} is the COV of the maximum expected lateral load factor. It reflects the level of uncertainty associated with determining the value of LW.

Equation 3.4 gives an approximate value for the reliability index when both LF_1 and LW follow lognormal distributions. The approximation is valid for values of V_{LF} and V_{LW} on the order of 0.20 to 0.25. An exact expression for the lognormal reliability index is available in reference books on reliability theory (e.g., Baker and Thoft-Christensen, 1982 or Melchers, 1999). On the other hand, the reliability index may be calculated for a variety of probability distribution types using a First Order Second Moment Reliability Method (FOSM/FORM) algorithm. Equation 3.4 is provided only for illustration purposes. The actual calculations performed in this study were executed using a program based on the FORM algorithm.

Under the effect of a lateral load applied on the pier substructure, failure of the first column occurs when the lateral load is multiplied by a factor LF_1. LF_1 is a function of the strength properties of the substructure (including column strength, and soil/foundation stiffness) and the magnitude of the gravity loads (dead loads and live loads) that are present when the failure of the first column occurs. The total effect of the gravity load, Q_n, is the summation of the nominal live load effect, L_n, and the nominal dead load effect, D_n.

$$Q_n = D_n + L_n \tag{3.5}$$

In the calculations performed, the nominal (design) values are used for D_n and L_n, where L_n as provided in the AASHTO LRFD Specifications, is equivalent to the expected 75-year maximum truck load (where the 75-year period corresponds to the design life of the bridge). Using the nominal live load would provide a conservative approximation to the load capacity of the substructure because the probability of having the 75-year live load simultaneously with the 75-year lateral load is very small. Also, the live load, on the average, constitutes only about 20 percent of the total vertical load applied on the substructure. Furthermore, the effect of the lateral load is the primary contributor to the bending stresses that will cause substructure failures. Thus, using the nominal

vertical live load during the analysis gives results that are slightly on the conservative side.

As the moment capacity, R, of the column increases, the load factor, LF_1, that causes the column to fail also increases. On the other hand, as the magnitude of the applied vertical loads Q increases, LF_1 is expected to decrease. Thus, the load factor LF_1 is a function of the load margin R – Q that may be represented as

$$F_1 \equiv LF_1 \times W_n = f_1\,(R - Q) \tag{3.6}$$

where W_n is the nominal lateral load used as the base case for the design and analysis of the substructure. The right hand side of Equation 3.6, f_1 (R – Q), represents a complex function of many random variables: the moment due to the applied vertical dead and live loads; the stiffness of the substructure system; the soil/foundation system; as well as the moment capacity of the columns. The vertical load effects include the moments at the base and top of the first column to fail, as well as the axial compressive load in that column. The interaction between the moment and the axial force determines the strength capacity of the column, R. The distribution of the moments and axial forces to each column of the pier system is a function of the stiffness of the soil/foundation system and that of the bent cap. In addition, the superstructure would provide additional lateral stiffness depending on the bridge type and geometry including the superstructure/substructure connection and attachment type, that is, whether the columns are built monolithically with the superstructure or whether the load from the superstructure is transferred to the columns through bearing supports.

The functional relationship for the strength limit state expressed in Equation 3.6 is difficult to obtain in closed form. However, structural analysis programs, such as PIERPUSH and FLORIDA-PIER, can be used to analyze individual structures and obtain the corresponding values of LF_1. Using a *perturbation technique* on the input and the results from the structural analysis, a functional relationship can be approximated. This process, often known as the *response surface approach*, will be described further below.

The mean value of the lateral load factor, LW, is related to the mean value of the maximum lifetime lateral load such that

$$\overline{LW} \times W_n = \overline{W_{\max}} \tag{3.7}$$

where $\overline{W_{max}}$ is the mean (expected value) of the maximum lateral load that will be applied on the substructure within its design life. W_n is the nominal design (code specified) value of the applied lateral load. The lateral load may be due to wind, seismic activity, or collision forces.

The denominator in Equation 3.4, being a function of the COVs V_{LF} and V_{LW}, gives an overall measure of the uncertainty in estimating the resistance, the vertical and the lateral loads applied on the pier column. The assumption is that the factors LF_1 and LW are random variables that follow lognormal distributions.

System Reliability—In a similar manner, assuming that the load factor LF_u and the lateral load factor LW follow lognormal distributions, the reliability index of the *substructure system* for the ultimate limit state can be defined as

$$\beta_{ult.} = \frac{\ln \dfrac{\overline{LF_u}}{\overline{LW}}}{\sqrt{V_{LF_u}^2 + V_{LW}^2}} \tag{3.8}$$

where $\overline{LF_u}$ is the mean value of the load factor corresponding to the ultimate limit state. LF_u depends on the strength capacity of the complete system and the applied permanent load. $\overline{LW}$ and V_{LW} are the same values used to calculate β_{member}, because the magnitude of the expected maximum lateral load that is applied on the substructure is independent of whether one is checking the failure of the first member or the failure of the complete substructure. V_{LF_u} is the COV of the ultimate capacity. In general, V_{LF_u} may be different than V_{LF} used in Equation 3.4. However, as demonstrated below for the average substructure configuration, the difference observed between V_{LF_u} and V_{LF} is negligible. Also, the denominator of Equations 3.4 and 3.8 is usually dominated by the high value of V_{LW} rendering the small differences between V_{LF_u} and V_{LF} insignificant.

LF_u can be represented in terms of the member resistances, R, the magnitude of the applied vertical loads, Q_n, and other material properties using a function, f_u, that is different from the function, f_1, used in Equation 3.6.

$$F_u \equiv LF_u \times W_n = f_u(R,Q) \tag{3.9}$$

The function f_u also represents a complex relationship between the individual column resistances, effects of the applied vertical loads, the soil/foundation stiffnesses, and the column cap stiffness. It also includes all other factors that affect the ductility of the column and the overall stability of the pier system. As mentioned above for the f_1 function, the functional relationship as expressed in Equation 3.9 *cannot* be obtained in closed-form. However, the response surface approximation can be obtained from a perturbation on the input values. The increase in $\beta_{ult.}$ over β_{member} is due to the increase in the system capacity compared to the member capacity. *This increase is thus related to the system reserve ratio, R_u, defined in Equation 3.1.*

Following the same logic outlined above and assuming a lognormal reliability model, the system reliability for the functionality limit state, β_{funct}, is expressed as

$$\beta_{funct} = \frac{\ln \dfrac{\overline{LF_f}}{\overline{LW}}}{\sqrt{v_{LF_f}^2 + v_{LW}^2}} \tag{3.10}$$

where $\overline{LF_f}$ is the mean value of the load factor corresponding to the functionality limit state. LF_f depends on the strength capacity of the complete system and the applied

permanent load. $\overline{LW}$ and V_{LW} are the same values used to calculate β_{member} and $\beta_{ult.}$ because the magnitude of the expected maximum lateral load that is applied on the substructure is independent of whether one is checking the failure of the first member or the failure of the complete substructure. V_{LF_f} is the coefficient of variation of LF_f. In general, V_{LF_f} may be different from V_{LF} and V_{LF_u}. However, the effects of these differences are negligible.

Finally, the system reliability for the *damaged condition*, $\beta_{damaged}$, is expressed as

$$\beta_{damaged} = \frac{\ln \dfrac{\overline{LF}_d}{\overline{LW}_2}}{\sqrt{V_{LF_d}^2 + V_{LW_2}^2}} \tag{3.11}$$

where $\overline{LF}_d$ is the mean value of the load factor corresponding to the damaged condition. The load factor, $\overline{LW}_2$, and the COV, V_{LW_2}, are different than the values $\overline{LW}$ and V_{LW} used to calculate β_{member}, β_{ult}, and β_{funct} reflecting the fact that a damaged substructure is not expected to withstand the maximum design life load but rather the load over a shorter exposure period lasting between the occurrence of the damage and the execution of the repairs. Normally, $\overline{LW}_2$ is lower than $\overline{LW}$ and V_{LW_2} is higher than V_{LW}. Furthermore, the final probability of failure for a system subjected to a "damage" event is equal to the probability of occurrence of the damage (e.g., probability of collision or occurrence of scour, etc.) times the probability of system failure given that the event has occurred. Thus, β_{damage} that is related to the conditional probability of failure given that damage has occurred need not be as high as β_{member} or $\beta_{ult.}$. Following the method described in *NCHRP Report 406* it is proposed to use a *2-year exposure period* for damaged substructures. *The 2-year period is chosen to coincide with the biannual inspection implying that the maximum period that a damage may remain undetected (and unrepaired) is 2 years.* V_{LF_d} is the COV of the damaged ultimate capacity. In general, V_{LF_d} may be different than V_{LF}, V_{LF_u}, and V_{LF_d}. However, as demonstrated below, the differences between these values are negligible.

Reliability-Based Measure of Redundancy—Redundancy is defined as the capability of a substructure system to continue to carry load after the failure of its most critical member (the first member to fail). Hence, to study the redundancy of a system, it is useful to examine the difference between the reliability indexes of the system expressed in terms of $\beta_{ult.}$, β_{funct}, and $\beta_{damaged}$ and the reliability index of the most critical member of the intact structure expressed in terms of β_{member}. The relative reliability indices are defined as

$$\Delta\beta_u = \beta_{ult.} - \beta_{member} \tag{3.12}$$

$$\Delta\beta_f = \beta_{funct} - \beta_{member} \tag{3.13}$$

$$\Delta\beta_d = \beta_{damaged} - \beta_{member} \tag{3.14}$$

These relative reliability indices give measures of the additional safety provided by the substructure system compared to the nominal safety against first member failure. It is proposed to use the relative reliability indices to provide a reliability-based measure of redundancy as described in *NCHRP Report 406*. Thus, a substructure system will provide adequate levels of system redundancy if the relative reliability indices are adequate. The $\Delta\beta$ are functions of the type of loading (through V_{LW}). Thus, they will not lead to the same values for all types of lateral loads (e.g., wind or earthquakes). However, they will provide consistent values of target system reserve ratio and system factors as will be seen in Section 3.5.

Substituting Equation 3.4 into Equations 3.12 through 3.14, Equation 3.8 into 3.12, Equation 3.10 into 3.13, and Equation 3.11 into 3.14, the relative reliability indices can be calculated as a function of the expected values of the substructure system capacity, the capacity of the first member to fail, the maximum lateral load, as well as the COVs of each of these random variables.

In this study, a reliability-based calibration is performed to determine the minimum value of system reserve ratio, R_u, (i.e., the ratio of system capacity with respect to member capacity LF_u/LF_1) that is required to ensure an adequate level of bridge redundancy. Target values for $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ are obtained by reviewing the performance of typical substructure configurations for the pertinent limit states. These include the crushing of one column of the system, the formation of a structure collapse mechanism, the loss of functionality, and the remaining capacity of the system after the brittle failure of one column (a discussion of these limit states will be presented in Section 3.6).

3.4 DEFINITION OF SYSTEM FACTORS

This study develops tables of system factors, ϕ_s, applicable to common bridge substructure configurations. The system factors are intended to be used in the design checking equation of substructure members such that:

$$\phi_s \phi R' = \gamma_d D_n + \gamma_L L_n + \gamma_W W_n \tag{3.15}$$

where ϕ_s is the system factor defined as a statistically based multiplier relating to the safety, redundancy, and ductility of the substructure system. ϕ is the member resistance factor; R' is the required nominal resistance capacity of the member accounting for the redundancy of the system; γ_d is the dead load factor and D_n is the nominal dead load effect; γ_L is the vehicular live load factor and L_n is the nominal (code specified) vehicular live load; and γ_W is the lateral load factor and W_n is the nominal effect of the lateral load applied on the substructure (e.g., wind load, earthquake load, etc.). The system factor is applied to the factored nominal member resistance. The system factor proposed herein replaces the two components η_D and η_R of the load modifier, η, used in Section 1.3.2 of the 1998 LRFD Specifications. These η_D and η_R factors

relate to the ductility and redundancy of the member and system. (A third component, η_I, included in η relates to the "operational importance.") The factor, ϕ_s, is placed on the left side of the equation because the system factor is related to the capacity of the system and as such should be placed on the resistance side of the equation, as is the norm in reliability-based calibration. When ϕ_s is equal to 1.0, Equation 3.15 becomes the same as the current design equation. If ϕ_s is greater than 1.0, it indicates that the system's configuration provides a sufficient level of redundancy. When it is less than 1.0, then the level of redundancy is not sufficient and Equation 3.15 requires that the members be more conservatively designed to improve the overall performance of the system. *Notice that applying a system factor of less than 1.0 on a nonredundant system will not render the substructure system redundant, but will only improve its overall safety to an acceptable level.*

The approach used to develop the system factor tables for bridge substructures is similar to the approach used in *NCHRP Report 406* to provide consistent levels of redundancy for bridge superstructures. The system factor is calibrated such that a value of 1.0 indicates the bridge substructure under consideration will have relative reliability indices $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ (or the reserve ratios R_u, R_f, and R_d) equal to appropriate target values, which are determined from the review of "acceptable" substructure configurations. Acceptable configurations are those that have sufficient levels of redundancy based on current practice and engineering judgment. The next section illustrates the calibration procedure using a substructure example analyzed with the program PIERPUSH. Subsequently, a representative number of substructure types are analyzed to develop target redundancy levels and to specify tables of system factors.

3.5 ILLUSTRATION OF CALIBRATION PROCEDURE

3.5.1 Description of Substructure Model

To illustrate the methodology followed during the calibration of the system factors, two representative examples are discussed in this section. These examples are for a two-column bent and a four-column bent. The two substructures have columns that are 11 m and 6.5 m high, respectively. The geometrical and material properties are shown in Tables 3-1 and 3-2. These properties were obtained from the survey of state DOTs conducted during the course of this study and represent the average values expected for typical two-column and four-column bents resting on *spread footings* set on a soil of *average* properties.

The analysis evaluates the redundancy of the two substructure systems under the effect of lateral loads. Lateral loads would model the effects of seismic, wind, and collision forces. The gravity loads applied on the substructure include both the dead load and the vehicular live load. The analysis process will increment the lateral load until system failure

TABLE 3-1 Input data for analysis of two-column bridge example

Categories	Variable	Symbol	Nominal Value	Distribution type	COV	Bias	Source
Concrete properties	Strength	f'_c	27 Mpa	Normal	15%	0.80	Ellingwood
	Modulus of elasticity	E_c	24900 Mpa	Varies in function of f'c			
Steel properties	Yielding	f_y	450 Mpa	Beta	10%	1.13	Ellingwood
	Modulus of elasticity	E_s	200,000 Mpa	Normal (a)	3.3%	1.02	Melchers
Foundation properties	Stiffness	K_v: vertical K_h:horizontal Kr: rotational	97200 kN/m 72900 kN/m 365000 kN.m	Normal (a)	30%	1.0	Beckcer
Loads	Dead loads	D_1	5560 kN	Normal	10%	1.05	Ellingwood
	Dead loads	D_2	Selfweight	Normal	8%	1.05	Ellingwood
	Live loads	L	1387 kN	Lognormal	20%	1.00	Nowak
Failure parameter	Maximum concrete strain	ε_c ε_u	Confined = 0.015 Unconfined = 0.004	Normal (a)	40% 40%	1.20 1.00	Ghosn
Geometry	Steel reinf.	As	44 bars x753 mm^2	Deterministic			
	Cover	Cs	75 mm				
	Height	Hc	11m				
	Column spacing	Sp.	10m				
	Column section	B_cxW_c	1.2mx1.2m				
	Cantilever		2 m				
	Cap size		1.2mx1.2m				

(a) = Distribution type assumed by the authors.

TABLE 3-2 Input data for analysis of four-column bridge example

Categories	Variable	Symbol	Nominal Value	Distribution type	COV	Bias	Source
Concrete properties	Strength	f'_c	27 Mpa	Normal	15%	0.80	Ellingwood
	Modulus of elasticity	E_c	24900 Mpa	Varies in function of f'c			
Steel properties	Yielding	f_y	450 Mpa	Beta	10%	1.13	Ellingwood
	Modulus of elasticity	E_s	200,000 Mpa	Normal (a)	3.3%	1.02	Melchers
Foundation properties	Stiffness	K_v: vertical K_h: horizontal Kr: rotational	77800 kN/m 58300 kN/m 1870000 kN.m	Normal (a)	30%	1.0	Beckcer
Loads	Dead loads	D_1	5985 kN	Normal	10%	1.05	Ellingwood
	Dead loads	D_2	Selfweight	Normal	8%	1.05	Ellingwood
	Live loads	L	1387 kN	Lognormal	20%	1.00	Nowak
Failure parameter	Maximum concrete strain	ε_c ε_u	Confined = 0.015 Unconfined = 0.004	Normal (a)	40% 40%	1.20 1.00	Ghosn
Geometry	Steel reinf.	As	44 bars x420.5 mm^2	Deterministic			
	Cover	Cs	75 mm				
	Height	Hc	6.5m				
	Column spacing	Sp.	3.67m				
	Column section	B_cxW_c	1.0mx1.0m				
	Cantilever		2 m				
	Cap size		1.0mx1.0m				

(a) = Distribution type assumed by the authors.

occurs. In this analysis example, it is assumed that the vertical loads (dead load + vehicular live load) are set at their mean lifetime maximum values. This approach is conservative as it is unlikely that the vehicular live load will be at its expected maximum lifetime value when the maximum lateral (wind or seismic) load is applied on the structure. The pier configurations used in this analysis are illustrated in Figures 3-1 and 3-2. The input values used during this analysis are given in Tables 3-1 and 3-2.

Tables 3-1 and 3-2 give the nominal (design) values for the input variables as well as the biases for the material properties and the vertical loads. The mean (or expected) values are calculated by multiplying the nominal values by their respective biases. In addition, Tables 3-1 and 3-2 give the COVs associated with the input variables and the bias. The COV gives a measure of the uncertainty associated with determining the random variables used in the analysis. The biases and COV values of Tables 3-1 and 3-2 are similar to the values used during the calibration of the AASHTO LRFD Specifications.

The material properties (concrete strength, yielding stress of steel, etc.) and geometric properties (section size and amount and location of reinforcement) combine to produce the

Figure 3-1. Configuration of four-column bent.

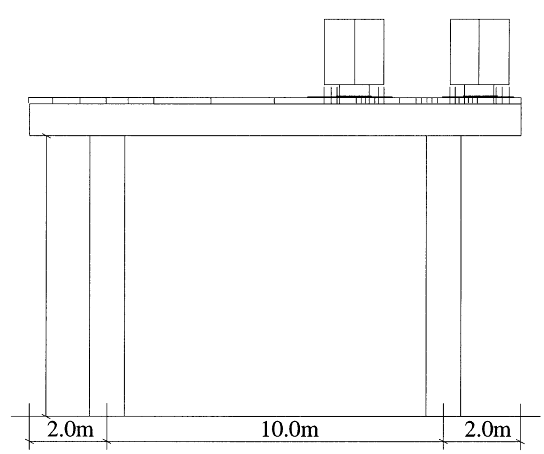


Figure 3-2. Configuration of two-column bent.

moment capacity of the column section. Two limiting values for the strain that produce concrete crushing are given. The first value assumes that the columns are *unconfined* (labeled ε_u). The second value is for the *confined* columns (labeled ε_c). The model used for the analysis accounts for the P-Δ effect produced when large values of lateral displacement interact with gravity loads to increase the moments in the columns.

The analysis performed in this section uses the mean values for all the variables. For the live loads, the mean values are assumed to be the same as the nominal HL-93 vehicular live loads recommended by Nowak (1994). All the live load variables (lane, truck, and impact) are assumed to be correlated, namely, a percentage change in any of these variables will automatically produce the same percentage change in the other variables. Similarly, the soil-foundation stiffnesses are assumed correlated in such a way that a percentage change in the rotational spring stiffness will produce the same percentage change in the horizontal and vertical spring stiffnesses. The variables listed in the tables without COVs are assumed to be deterministic. Particularly, the geometric properties are all assumed to be well defined such that the variability in their values is minimal and does not produce any noticeable effect on the reliability of the substructure.

3.5.2 PIERPUSH Results

In the first step, the incremental pushover analysis is performed, using PIERPUSH, by increasing the lateral load gradually until a large lateral displacement is observed (0.2 m for the four-column bent and 0.4 m for the two-column bent). During the incremental loading process, the vertical loads (dead load and live load) are kept fixed at their mean values. All the material variables are also set at their mean values without load or resistance factors. Figures 3-3 and 3-4 give the plots showing the lateral deflection of the pier versus the applied lateral load for each of the two bents. The calculations flag the lateral load where various critical events and limit states are reached. These are as follows:

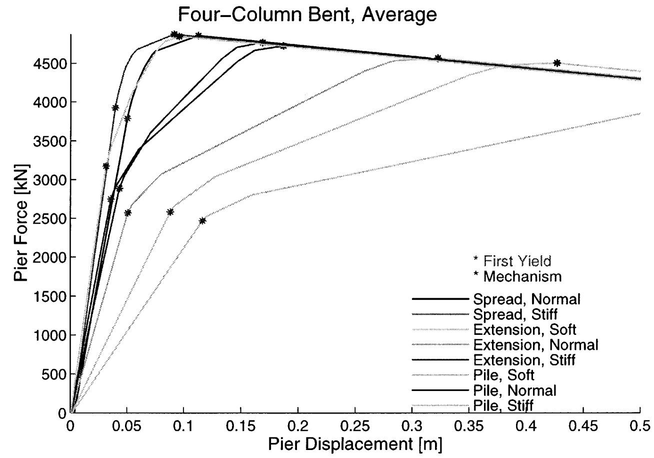


Figure 3-4. Four-column bent, force-displacement relation, average.

1. The load at which the first column reaches its ultimate bending strength, P_1^*,
2. The load at which a mechanism is formed in the system, P_m^*,
3. The load at which one of the columns reaches its crushing strain (ductility exhausted) assuming all the columns are *unconfined* P_u^*,
4. The load at which one of the columns reaches its crushing strain assuming all the columns are *confined* P_c^*,
5. The load that causes a lateral deflection equal to 2.5 percent of column height, P_f^*.

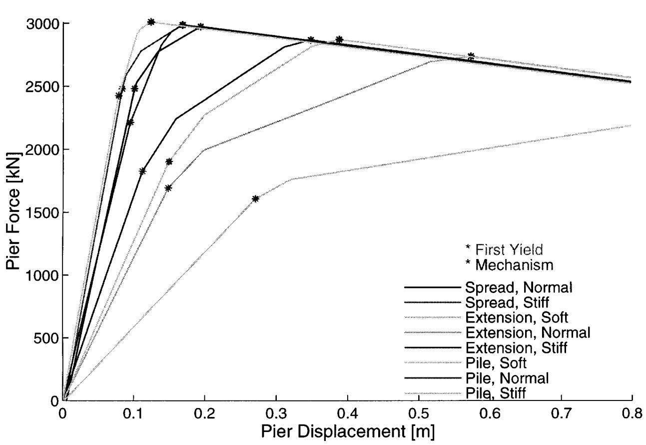


Figure 3-3. Two-column bent, force-displacement relation, average properties.

The P* values give a representation of the capacity of the system to resist failure in a given limit state (failure mode). Failure occurs in a given mode when the applied lateral load P is higher than the P* corresponding to the limit state being considered. The results of the limit states considered are summarized in Table 3-3.

Figures 3-3 and 3-4 show a softening in the lateral load for increasing lateral deflection. This softening is caused by the inclusion of the P-Δ effects whereby the moments at the bases of the columns are amplified due to the effects of the vertical loads subjected to a lateral displacement. The plots show that the crushing of unconfined columns will generally occur before the mechanism is formed. The cases analyzed in this section, however, show that a collapse mechanism occurs before confined columns reach their crushing strain. The 2.5 percent drift occurs at loads close to those that cause the crushing of confined columns. These situations are specific to the material and geometric properties used in these examples and may not be representative of all substructure geometries and foundation types. Results for both lateral forces and lateral displacements are included in Appendix B for all limit states.

TABLE 3-3 Lateral load capacities for two-column and four-column piers

(All input variables are set at their mean values)

Limit state	Symbol	Four-column bent	Two-column bent
First member failure	P_1^*	4002 kN (5)	2522 kN (10)
Mechanism	P_m*	5052 kN (11.3)	3077 kN (19.3)
Crushing of unconfined member	P_u^*	4731 kN (7.4)	2847 kN (14.8)
Crushing of confined member	P_c^*	4988 kN (15.1)	3005 kN (28.9)
Functionality, Δ=2.5%H	P_f^*	4948 kN (13)	3009 kN (22)

Note: Numbers in parenthesis are the corresponding displacement in cm.

3.5.3 Response Surface Analysis

The reliability analysis of substructures requires the knowledge of the mean and standard deviation (or the COV) of the capacity of the structure to resist the first member failure; the ultimate capacity of the structure; the load at which the functionality limit state is reached; as well as those of the expected loads. Although information is available on the statistics of the capacity of individual members and of the applied bridge loads (e.g., dead loads, traffic loads, wind loads, and earthquake loads), very little information is available on the uncertainties associated with determining the ultimate lateral capacity of bridge substructures.

Tables 3-1 and 3-2 summarize the most important random variables that affect the determination of the bridge substructure capacity. If an *explicit* closed-form expression describing the relationship between these individual random variables and the ultimate bridge substructure capacity is available, then the reliability calculations can be easily performed. This calculation would lead to the statistical data on the ultimate capacity, the probability of failure, and the reliability index, β. Unfortunately such closed-form expressions are not available and one has to rely on numerical deterministic analyses, such as those performed by a nonlinear program (e.g., PIERPUSH, FLORIDA-PIER, etc.). An efficient numerical technique that can be used to calculate the reliability of bridge systems when the failure equations cannot be explicitly formulated is the *response surface method* [Ghosn et al. (1994) and Augusti et al. (1984)]. The method uses the deterministic results from a structural analysis program to determine the reliability of the system. The approach is further described in the following paragraphs.

The program, PIERPUSH, can be used to obtain the capacity of the bridge system for predetermined values of structural member and soil properties. For the two substructures analyzed in the previous section, this means that for a given set of values for the column properties (f'_c, E_c, f_y, E_s, A_s, C_s, B_c, W_c, H_c, S_p, and ε_u or ε_c, etc.), the foundation stiffnesses, K_v, K_h, K_r as well as the vertical dead loads, D_1 and D_2, and the vertical live load, L (representing the summation of truck loads and lane loads including impact factor, I), a value of the horizontal load P* that will produce the collapse of the system can be obtained. As mentioned above, P* is a representation of the capacity of the system to carry the lateral load. The applied lateral load P may be smaller or larger than the capacity P*. If P is larger than P*, the system collapses. If P is smaller than P*, the system is safe.

The variables f'_c, f_y, E_s, K_v, D_1, D_2, L, and ε_u (or ε_c) are random having the biases and the COVs listed in Tables 3-1 and 3-2. These values have been collected from the data provided by Nowak (1994), Becker (1996 a,b), Ellingwood et al. (1980), and Ghosn and Moses (1998). All other geometric and material parameters are assumed to be deterministic.

Several deterministic analyses are performed using PIERPUSH for different fixed values of the random variables. For each combination of values, the capacity P* is found for each of the limit states listed in Table 3-3. A sensitivity analysis is performed by perturbing each variable from its initial value. Thus, several sets of data and corresponding P* values are obtained. The first set assumes that all the random variables are fixed at their mean values as described in the previous paragraph. Then, the variables are changed one at a time to (1) values equal to the mean value minus one standard deviation and, (2) to values equal to the mean plus one standard deviation. Hence, for the 9 random variables, a total of 18 additional deterministic analyses (for a total of 19 analyses) are performed. For each of the 18 additional analyses, one of the variables is perturbed from its original value. As an example, the values used for each of the random variables of the two-column bent are given in Table 3-4.

TABLE 3-4 Values of random variables used in perturbation analysis of two-column bent

	f'_c [MPa]	F_y [MPa]	E_s [GPa]	K_v [kN/m]	D_1 [kN]	D_2 [kN]	L [kN]	ε_c	ε_u
Mean	21.6	508.5	204	97200	5838	782	1387	0.0180	0.004
Mean –Standard deviation	18.36	457.7	197.3	68040	5254	719	1110	0.0108	0.0024
Mean +Standard deviation	24.84	559.4	210.7	126360	6422	844	1664	0.0252	0.0056

As mentioned above, all the foundation stiffnesses are changed simultaneously because these variables are assumed to be fully correlated. Similarly all the live loads are combined together to form one random variable.

For each analysis, the value of the ultimate capacity of the substructure system P* is calculated for each of the limit states. Tables 3-5 and 3-6 illustrate the results obtained for P_1^*, P_m^*, P_u^*, P_c^*, and P_f^* as defined above. The results of the perturbation analysis are provided in Table 3-5 for the two-column bent and Table 3-6 for the four-column bent.

The results of the deterministic analyses are then used to obtain functional relationships between each of P_1^*, P_m^*, P_u^*, P_c^*, P_f^* and the random variables f'_c, f_y, E_s, K_v, D_1, D_2, L, and ε_u (or ε_c). For each of the limit states, the functional relationship is obtained by a multivariable regression analysis. This functional relationship is often known as the response surface (or the response function). The response surface will thus give a relationship between the capacity of the bridge substructure and the random variables that affect the capacity of the bridge substructure system to carry the load. Once this response surface is found, it can be used to obtain the reliability index of the system and to calibrate the appropriate system factor. The results of the regression fit are shown below for each of the five limit states of the two and four-column bents.

The regression analysis of the results for the two-column bent produced the following functional relationships:

$$P_1^* = 129.82 + 32.25 f'_c + 2.88 f_y + 1.72 \times 10^{-3} E_s + 2.57 \times 10^{-4} K_v - 8.56 \times 10^{-3} D_1 - 6.93 \times 10^{-5} D_2 - 6.86\ 10^{-2} L$$

$$P_m^* = 160.98 + 29.47 f'_c + 3.53 f_y + 2.01 \times 10^{-3} E_s + 4.80 \times 10^{-4} K_v + 5.14 \times 10^{-3} D_1 + 7.87 \times 10^{-3} D_2 - 7.22 \times 10^{-3} L$$

$$P_u^* = -125.74 + 37.35 f'_c + 2.75 f_y + 2.46 \times 10^{-3} E_s + 1.41 \times 10^{-3} K_v - 1.97 \times 10^{-2} D_1 + 1.46 \times 10^{-2} D_2 - 6.32 \times 10^{-2} L + 76875\, \varepsilon_u \tag{3.16}$$

$$P_c^* = 195.15 + 26.08 f'_c + 3.72 f_y + 1.94 \times 10^{-3} E_s + 2.06 \times 10^{-4} K_v + 1.28 \times 10^{-2} D_1 + 1.62 \times 10^{-2} D_2 + 3.61 \times 10^{-3} L - 8403\, \varepsilon_c$$

$$P_f^* = 67.54 + 29.94 f'_c + 3.68 f_y + 2.09 \times 10^{-3} E_s - 1.71 \times 10^{-5} K_v + 2.57 \times 10^{-3} D_1 - 3.62 \times 10^{-5} D_2 - 7.22 \times 10^{-3} L$$

The regression analysis of the results for the four-column bent produced the following functions:

$$P_1^* = -405.80 + 9.78 f'_c + 5.78 f_y + 2.39 \times 10^{-3} E_s + 4.39 \times 10^{-3} K_v + 5.31 \times 10^{-2} D_1 + 7.94 \times 10^{-3} D_2 - 2.17 \times 10^{-1} L$$

$$P_m^* = 47.14 + 46.60 f'_c + 5.93 f_y + 2.91 \times 10^{-3} E_s + 7.50 \times 10^{-4} K_v + 4.97 \times 10^{-2} D_1 + 2.36 \times 10^{-2} D_2 + 1.44 \times 10^{-2} L$$

$$P_u^* = -635 + 55.71 f'_c + 4.75 f_y + 3.28 \times 10^{-3} E_s + 5.66 \times 10^{-3} K_v + 2.65 \times 10^{-2} D_1 + 1.39 \times 10^{-2} D_2 - 1.46 \times 10^{-1} L + 164375\, \varepsilon_u \tag{3.17}$$

$$P_c^* = 132.77 + 45.37 f'_c + 6.04 f_y + 2.91 \times 10^{-3} E_s + 2.36 \times 10^{-4} K_v + 4.88 \times 10^{-2} D_1 + 2.36 \times 10^{-2} D_2 + 2.53 \times 10^{-2} L - 8194\, \varepsilon_c$$

$$P_f^* = -27.10 + 48.61 f'_c + 6.00 f_y + 3.06 \times 10^{-3} E_s - 8.57 \times 10^{-5} K_v + 3.77 \times 10^{-2} D_1 + 1.56 \times 10^{-2} D_2 + 1.62 \times 10^{-2} L$$

TABLE 3-5 Results of sensitivity analysis for two-column bent

Case #			P_1*	P_m*	P_u*	P_c*	P_f*
1	All Mean		2522	3077	2847	3005	3009
2	F'c,	Low	2417	2978	2718	2918	2908
3	F'c,	High	2626	3169	2960	3087	3102
4	fy,	Low	2375	2894	2693	2812	2818
5	fy,	High	2668	3253	2973	3190	3191
6	E_s,	Low	2520	3070	2837	2998	3001
7	E_S,	High	2543	3097	2870	3024	3029
8	K_v	Low	2508	3059	2785	2997	3010
9	K_v,	High	2523	3087	2867	3009	3009
10	D_1,	Low	2527	3073	2856	2999	3007
11	D_1,	High	2517	3079	2833	3014	3010
12	D_2,	Low	2522	3077	2846	3004	3009
13	D_2,	High	2522	3078	2848	3006	3009
14	L,	Low	2534	3079	2863	3005	3011
15	L,	High	2496	3075	2828	3007	3007
16	ε_c,	Low	2522	3077	2847	3071	3009
17	ε_c,	High	2522	3077	2847	2950	3009
18	ε_u,	Low	2522	3077	2663	3005	3009
19	ε_u,	High	2522	3077	2909	3005	3009

Units of P* are kN.

TABLE 3-6 Results of sensitivity analysis for four-column bent

Case #			P_l*	P_m*	P_u*	P_c*	P_f*
1	All Mean		4022	5052	4731	4988	4948
2	F'c,	Low	3925	4890	4543	4830	4778
3	F'c,	High	4118	5192	4904	5124	5093
4	fy,	Low	3727	4738	4481	4667	4629
5	fy,	High	4315	5341	4964	5281	5239
6	E_s,	Low	4020	5033	4709	4969	4927
7	E_S,	High	4052	5072	4753	5008	4968
8	K_v	Low	3907	5030	4554	4981	4950
9	K_v,	High	4112	5065	4818	4992	4946
10	D_1,	Low	3991	5023	4720	4959	4925
11	D_1,	High	4053	5081	4751	5016	4969
12	D_2,	Low	4022	5051	4730	4987	4947
13	D_2,	High	4023	5054	4732	4990	4949
14	L,	Low	4082	5048	4776	4981	4943
15	L,	High	3962	5056	4695	4995	4952
16	ε_c,	Low	4022	5052	4731	5051	4948
17	ε_c,	High	4022	5052	4731	4933	4948
18	ε_u,	Low	4022	5052	4361	4988	4948
19	ε_u,	High	4022	5052	4887	4988	4948

Units of P* are kN.

where the variables f'_c, f_y, E_s, K_v, D_1, D_2, L are expressed in kN and m. K_v is expressed in terms of the vertical stiffness, although, as mentioned above, the horizontal and rotational stiffnesses are fully correlated to the vertical stiffness.

Notice that a negative coefficient associated with any of the random variables in Equations 3.16 and 3.17 indicates that the ultimate capacity decreases when the value of the variable is increased. The regression coefficients for the fit of Equations 3.16 and 3.17 shown above give values of R^2 greater than 0.994 indicating an excellent fit for P^*_1, P^*_m, P^*_c, and P^*_f. The lowest regression coefficient was associated with the unconfined crushing limit state P^*_u that produced an R^2-value on the order of 0.94 that is still acceptably high.

The analysis performed considered each limit state separately in order to study how each is affected by the input parameters, although, in reality, the system's ultimate capacity is reached at either the formation of a collapse mechanism or at concrete crushing whichever limit state occurs first. Equations 3.16 and 3.17 show some unexpected relationships between the system capacities expressed as P* and the various random variables. For example, it is observed that an increase in the crushing strain, ε_u, of the unconfined columns increases P^*_u while an increase in the crushing strain of the confined columns, ε_c, reduces P^*_c. This is because, for the cases cited here, the crushing of the confined column occurs in the descending portion of the load versus deformation curve while the crushing of unconfined columns occurs in the ascending portion of the curve. Similarly, the researchers observe that the dead load and the live load may help increase the system's capacity while at other times they may decrease it. This phenomenon is attributed to the P-Δ effects as well as the effects of the column interaction (P-M) curve whereby, in certain loading combinations, the column stresses are below the balanced point of the column interaction curve and for other combinations the column stresses may lie above the balanced point. The increase in the foundation stiffness increases the strength capacity of the substructure while the loads for the functionality limit state remains unchanged.

The mean values for all the P* can be calculated by substituting the mean values of f'_c, f_y, E_s, K_f, D_1, D_2, and L into the functional Equations 3.16 and 3.17 given above. For the two-column example with the data given in Table 3-5, this will produce a mean of $P^*_1 = 2521$ kN compared to a value of 2522 kN when the mean values are used directly in the analysis. In the example studied here, the mean of the function can be approximated by the function of the means, and it confirms that the linearization process at points around the mean values operates well for this example.

The standard deviation, σ_p, for each limit state capacity, P*, σ_p can be obtained using the expression:

$$\sigma_p^2 = (b_1\,\sigma_{fc})^2 + (b_2\,\sigma_{fy})^2 + (b_3\,\sigma_{Ec})^2 + (b_4\,\sigma_{Kv})^2 + (b_5\,\sigma_{D1})^2 + (b_6\,\sigma_{D2})^2 + (b_7\,\sigma_L)^2 + (b_8\,\sigma_\varepsilon)^2 \quad (3.18)$$

where σ_{fc}, σ_{fy}, σ_{Es}, σ_{Kv}, σ_{D1}, σ_{D2}, σ_L, and σ_ε are the standard deviations of the random variables f'_c, f_y, E_s, K_v, D_1, D_2, L, and ε_u (ε_c), respectively, and the b_i gives the coefficients of each of these random variables in the order that they appear in the functional relationships shown in Equations 3.16 and 3.17. Equation 3.18 assumes independence among all the random variables listed.

Using the data of Tables 3-1 and 3-2, the standard deviation for each of the limit states analyzed above can be calculated. For example, the calculations produce a standard deviation for P^*_1 equal to 181 kN for the two-column bent producing a COV of 7.2 percent. It is also observed that the COV obtained from all limit states presented above vary between 6.64 percent and 9.00 percent. It should be noted, however, that these values of the COV do not account for the uncertainties in the finite element analysis modeling associated with the program PIERPUSH and do not account for the uncertainties associated with the use of the response surface method. Notice that the COV associated with the evaluation of concrete beams in bending is given as 13 percent by Nowak (1994). Therefore, it would be reasonable to assume that the modeling uncertainties would increase the COV of the system to at least 13 percent.

The results of the COVs obtained as explained above do not show any consistent trends or variations from one limit state to the other. Therefore, in this study it is assumed that the COV of 13 percent is valid for all the limit states considered. The next section will show that the calibration procedure followed in this study produces ϕ_s factors that are not sensitive to variations in the COVs of the limit states.

The means and standard deviations of P* can be used in Equations 3.4, 3.8, 3.10, and 3.11 to find the reliability indices of the substructure system and the reliability index of the first member to fail.

The regression analysis gives parameters for the means and the standard deviations that may be sensitive to the points around which the perturbation is performed. An iterative process can be used to improve the accuracy of the results. The iterative process consists of first performing a regression around the mean values of the random variables and then repeating the expansion at points close to the expected failure point once the expected failure point is identified from the reliability calculations. Ghosn et al. (1994) have shown that in general, such iterations do not produce significant changes in the final calculations of the safety indices.

The response surface method, shown here to be reasonably accurate and efficient for the reliability analysis of bridge systems, will require several nonlinear analyses for each bridge configuration. Because the project studies hundreds of configurations, it will be impossible to perform such an involved analysis for all bridges that are considered. Hence, the results of the two bridge configurations analyzed above are assumed to be representative and are projected to the other configurations.

3.5.4 Reliability Calibration of System Factors

Assume that predicting the capacity of the bridge system subjected to the applied loading conditions is uncertain with a COV equal to 13 percent (i.e., V_{LF} in Equations 3.4, 3.8, 3.10, and 3.11 are the same and set at 0.13). The analysis performed herein assumes that the lateral load is due to wind. This section demonstrates, however, that the final system factors are independent of the load type although the values of the reliability indices will be different. The expected maximum 50-year wind load is associated with a COV equal to 37 percent with a bias of 0.78 (i.e., the mean value of maximum expected wind load is 0.78 times the value used in design) (Ellingwood et al., 1980). Projecting these results for a 75-year period and assuming independence between the effects of windstorms, produces a bias equal to 0.87 and a COV equal to 33 percent. A 75-year return period was chosen to match the design service life used in the AASHTO LRFD Specifications. These bias and COV are typical for wind loads and are used to give a reference value for the reliability indices β_{member} and β_{ult}. The value used for bias has no effect on the relative reliability index $\Delta\beta$.

The structural analysis performed in the previous paragraph determined that the first member of the four-column bent system fails when the applied lateral load is equal to $F_1 = P_1^* =$ 4022 kN. *Keeping in mind that $F_1 = LF_1 W_n$ (Equation 3.6) and $W_{max} = LWW_n$* (Equation 3.7), the reliability index for the most critical member is calculated from Equation 3.4:

$$\beta_{member} = \frac{\ln \frac{\overline{LF}_1}{\overline{LW}}}{\sqrt{V_{LF}^2 + V_{LW}^2}} = \frac{\ln \frac{\overline{LF}_1}{\overline{LW}} \frac{W_n}{W_n}}{\sqrt{0.13^2 + 0.33^2}} = \frac{\ln \frac{4022}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} \tag{3.19}$$

where W_n is the nominal (code specified) 50-year wind load effect. Notice that the denominator of the reliability index, β, is dominated by $V_{LW} = 0.33$ such that the square root of the sum of 0.13^2 and 0.33^2 is equal to 0.35. Hence, variations in V_{LF} do not significantly affect the final value of β.

The calculation of the reliability index for ultimate limit state is performed using Equation 3.8. The results from PIERPUSH for the unconfined limit state show that the system will be able to resist a lateral force of 4731 kN before crushing of a column occurs. The reliability index for the ultimate system capacity assuming *unconfined* columns is obtained as

$$\beta_{ult.} = \frac{\ln \frac{\overline{LF}_u}{\overline{LW}}}{\sqrt{V_{LF_u}^2 + V_{LW}^2}} = \frac{\ln \frac{4731}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} \tag{3.20}$$

The difference between the system and member reliability indices for the four-column bent is

$$\Delta\beta_u = \beta_{ult.} - \beta_{member} = \frac{\ln \frac{4731}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} - \frac{\ln \frac{4022}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} = \frac{\ln \frac{4731}{4022}}{\sqrt{0.13^2 + 0.33^2}} = 0.46 \tag{3.21}$$

Notice that $\Delta\beta_u$ is neither a function of W_n nor of the bias as the subtraction of the logarithmic terms eliminates 0.87 W_n from the $\Delta\beta_u$ equation.

Repeating the same calculations for the two-column bent with a member capacity equal to 2522 kN and an unconfined column system capacity equal to 2847, a $\Delta\beta_u = 0.34$ is obtained as shown:

$$\Delta\beta_u = \beta_{ult.} - \beta_{member} = \frac{\ln \frac{2847}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} - \frac{\ln \frac{2522}{0.87 W_n}}{\sqrt{0.13^2 + 0.33^2}} = \frac{\ln \frac{2847}{2522}}{\sqrt{0.13^2 + 0.33^2}} = 0.34 \tag{3.22}$$

The 0.34 value for the two-column bent is lower than that observed for the four-column bent (0.46) indicating that the redundancy level of the two-column bent is lower than that of the four-column bent. The two-column bent's safety should be increased to obtain a system that provides a similar safety level as that of the four-column bent. The increase in the two-column bent safety may be achieved by applying a system

factor, ϕ_s, during the design of the members of the two-column substructure. The value of the system factor that should be used must reflect the additional level of safety that is required.

For illustration, let us assume that, for a bent to be considered adequately redundant, its system reliability index, $\beta_{ult.}$, must be higher than its member reliability index by at least 0.46. The four-column bent satisfies this requirement but the two-column bent does not. The fact that the two-column bridge analyzed has a relative reliability index (0.34) lower than the required (0.46) indicates that the two-column bridge's redundancy level is not adequate. For the two-column bent to be adequately redundant, its system reliability index, $\beta_{ult.}$, should have been higher than its current value by 0.12 (= 0.46 − 0.34). To obtain a higher $\beta_{ult.}$ under the expected loading condition, the value of LF_u in Equation 3.8 should be increased such that the new value, call it LF'_u, should produce a reliability index $\beta_{ult.}$ = 0.46 higher than β_{member}, while the current LF_u produces a reliability index $\beta_{ult.}$ = 0.34 higher than β_{member}. This means that LF'_u should produce a safety index higher than that of LF_u by 0.12 (= 0.46 – 0.34). This is expressed as

$$\frac{\ln \dfrac{LF'_u W_n}{0.87W_n}}{\sqrt{0.13^2+0.33^2}} - \frac{\ln \dfrac{2847}{0.87W_n}}{\sqrt{0.13^2+0.33^2}} = \frac{\ln \dfrac{LF'_u W}{2847}}{\sqrt{0.13^2+0.33^2}} = 0.12 \quad (3.23)$$

or,

$$\frac{LF'_u W_n}{2847} = \exp\left(0.12 * \sqrt{0.13^2+0.33^2}\right) = 1.04 \quad (3.24)$$

Thus, the fact that updated system capacity, LF'_u, should be higher than its current value LF_u by a factor of 1.04. LF'_u can also be calculated using a slightly different approach as follows. If the objective is to reach a target $\Delta\beta_u$ value = 0.46, then a new design should be such that

$$\frac{\ln \dfrac{LF'_u W_n}{0.87W_n}}{\sqrt{0.13^2+0.33^2}} - \frac{\ln \dfrac{2522}{0.87W_n}}{\sqrt{0.13^2+0.33^2}} = \frac{\ln \dfrac{LF'_u W}{2522}}{\sqrt{0.13^2+0.33^2}} = 0.46$$

This leads to a required value of LF'_u

$$LF'_u W_n = 2522e^{0.46\sqrt{0.13^2+0.33^2}} = 2969$$

Because the current system ultimate capacity is $F_u = LF_uW_n$ = 2847, then the updated system ultimate capacity should be higher than the current capacity, and consequently the ultimate load factor LF'_u higher than LF_u, by a factor = 1.04 (= 2969/2847).

Several methods could be devised to increase the system capacity of the substructure. For example, one could add columns or change the overall geometry. The simplest method would increase the capacity of each column. The primary effect produced in the columns of the bent due to a lateral load is the flexural bending of the columns. If the moments produced by the dead and vertical loads are relatively small compared to the moment caused by the lateral load, then increasing the capacity of the complete system to resist lateral loads by a 4 percent would require an approximate increase of the moment capacity of each column by 4 percent. Thus, one way to increase LF_u by 4 percent is to increase LF_1 by the same percentage, that is, an additional safety factor equal to 1.04 should be added to the safety factors used to design the columns of the two-column bent. Using an LRFD format, a safety factor of 1.04 is reflected by a resistance factor of 0.96 (1/1.04). This additional resistance factor is defined as the system ϕ_s shown in the left-hand side of Equation 3.15. More accuracy can be achieved if an iterative process is used whereby after a change of the member capacities, the analysis is repeated to verify that the target increase in system capacity is actually reached. This iteration may be worthwhile to undertake when the system factor ϕ_s is greater than 1.0 that required a reduction in member capacities so as to ensure that the reduction produced its intended safety target and not less.

In summary, the process outlined herein is based on two key assumptions:

1. To increase the ultimate system capacity expressed by LF_u by a certain factor, it is sufficient to increase the capacity of the system to resist first member failure represented by LF_1 by the same factor.
2. To increase the lateral capacity of the system to resist first member failure represented by LF_1 by a certain factor, it is sufficient to increase the moment capacity of the column section by the same factor.

As an example, Assumption 1 would be exactly satisfied if the moments due to the dead and live loads developed in all the columns of one bent are equal and if the formation of a collapse mechanism is used for the system limit state. A collapse mechanism occurs when hinges form on the tops and bottoms of all the columns of a bent. Assumption 2 implies that the (moment) effect of the gravity loads on the individual columns in the substructure system is relatively small compared with the (moment) effect due to the lateral load. Both these assumptions are reasonable for bents with stiff column caps with a reasonable level of column ductility as demonstrated in Chapter 2. Assumption 2 is only used to develop the system factor tables that are provided for "typical" substructure configurations as will be discussed below. Assumption 2 does not need to be satisfied if the direct redundancy analysis procedure described in the subsequent sections is used. The direct analysis procedure can also be adjusted as discussed in subsequent sections to take into consideration cases that do not satisfy Assumption 1.

To verify the above-stated assumptions, an example four-column bent with columns designed to produce a moment capacity equal to 4000 kN-m was loaded with its dead load and the live load from two lanes of traffic and analyzed for an increasing lateral load using the program PIERPUSH. The

lateral load that causes the crushing of the concrete of one column for confined concrete is 5922 kN. When the moment capacity of the columns was decreased by 13 percent down to 3540 kN-m, the lateral load that causes the crushing of the confined concrete becomes 5274 kN. The ratio of the system capacities 5922 kN/5274 kN = 1.12 is very close to the 1.13 decrease in the individual member capacities. When the moment capacity was increased by 13 percent up to 4520 kN-m, the lateral load that causes the crushing of the confined concrete becomes 6463 kN. The ratio of the system capacities 6463 kN/5922 kN = 1.09 is still acceptably close to the original 1.13 change in the moment capacity of the individual columns. The differences between the 1.09, 1.12, and 1.13 values are due to the moment effects of the vertical loads and the effects of the columns' moment-axial force interaction curves. This example demonstrates that the key assumptions used in the calibration procedure are reasonable for the purpose of providing system factors that reflect the level of redundancy available in typical bridge substructure systems. As mentioned above, more accuracy can be achieved by repeating the process until the exact target safety is reached.

3.5.5 Relationship Between System Factor ϕ_s, the Reliability Measure of Redundancy $\Delta\beta_u$, and the System Reserve Ratio R_u

This project, following the procedure outlined in *NCHRP Report 406*, has introduced three distinct measures of substructure redundancy: (1) the system factor, ϕ_s, used during the design process; (2) the reliability measure of redundancy, $\Delta\beta_u$, used for the calibration of the system factors; and (3) the system reserve ratio, $R_u = LF_u/LF_1$, obtained from the deterministic analysis of bridge substructures. This section demonstrates that these three measures as defined in this report are closely related to each other.

The objective of the calibration of the system factors as outlined in this study is to ensure that a bridge substructure will provide an adequate level of system safety. A bridge substructure configuration has a system capacity expressed by a lateral load factor LF_u and the substructure's capacity to resist the failure of the first column is represented by LF_1. If the redundancy of this *substructure system is not* adequate, the objective of the calibration process is to raise the value of LF_u to a new value LF'_u so that the system capacity becomes adequate. This objective would require that $\Delta\beta_u$ of the upgraded system should satisfy a target value, $\Delta\beta_{target}$, when illustrated in the following equation:

$$\Delta\beta_u = \beta_{ult.} - \beta_{member} = \frac{\ln\frac{\overline{LF'_u}}{\overline{LW}}}{\sqrt{v_{LF}^2 + v_{LW}^2}} - \frac{\ln\frac{\overline{LF_1}}{\overline{LW}}}{\sqrt{v_{LF}^2 + v_{LW}^2}} = \frac{\ln\frac{\overline{LF'_u}}{\overline{LF_1}}}{\sqrt{v_{LF}^2 + v_{LW}^2}} = \Delta\beta_{target} \tag{3.25}$$

The target, $\Delta\beta_{target}$, is obtained as the average value from a sample of substructures that are "known" to have a satisfactory level of redundancy. Then, this target could be expressed as

$$\Delta\beta_{target} = \text{average}\left(\frac{\ln\frac{\overline{LF_u}}{\overline{LF_1}}}{\sqrt{v_{LF}^2 + v_{LW}^2}}\right) \tag{3.26}$$

The target $\Delta\beta_{target}$ may be expressed as

$$\Delta\beta_{target} = \frac{\ln\left(\text{average}\left(\frac{\overline{LF_u}}{\overline{LF_1}}\right)\right)}{\sqrt{v_{LF}^2 + v_{LW}^2}} \tag{3.27}$$

Substituting Equation 3.27 into Equation 3.25

$$\frac{\ln\frac{\overline{LF'_u}}{\overline{LF_1}}}{\sqrt{v_{LF}^2 + v_{LW}^2}} = \frac{\ln\left(\text{average}\left(\frac{\overline{LF_u}}{\overline{LF_1}}\right)\right)}{\sqrt{v_{LF}^2 + v_{LW}^2}} \tag{3.28}$$

or

$$\frac{\overline{LF_u}}{\overline{LF_1}} = \text{average}\left(\frac{\overline{LF_u}}{\overline{LF_1}}\right) = \text{target}\left(\frac{\overline{LF_u}}{\overline{LF_1}}\right) \equiv R_{u,\,target} \tag{3.29}$$

Given that the current substructure has a system capacity LF_u, then LF'_u can be defined as

$$LF'_u = LF_u/\phi_s \tag{3.30}$$

Substituting Equation 3.30 into Equation 3.29, the system factor can be calculated from the target LF_u/LF_1 and the current system reserve ratio as

$$\phi_s = \frac{\frac{\overline{LF_u}}{\overline{LF_1}}}{\text{target}\left(\frac{\overline{LF_u}}{\overline{LF_1}}\right)} = \frac{\frac{LF_u}{LF_1}}{\text{target}\left(\frac{LF_u}{LF_1}\right)} \tag{3.31}$$

In addition to assuming a lognormal model for the reliability calculations, the assumptions used throughout this section are that the bias of the load factors, LF, and the COV of the load factors, V_{LF}, are the same for all the substructure configurations.

As an example, examine the two- and four-column bents studied above. The first member failure of the two-column bent occurred at a lateral load 2522 kN. The system failure occurred at a lateral load of 2847 kN. This produces a system reserve ratio $R_u = LF_u/LF_1 = 1.13$ (= 2847/2522). The system reserve ratio for the four-column bent is $R_u = LF_u/LF_1 = 1.17$ (= 4731/4022). Let us assume that the goal is to design two-column bents with the system safety levels as the four-column bent. Hence, for this example, the target value of LF_u/LF_1 that

any bridge substructure should satisfy is 1.17. Since the two-column bent provides a reserve ratio of 1.13, then, the safety of its members should be increased by applying a system factor ϕ_s during the design process. For this particular two-column bent, the system factor should be set equal to 1.13/1.17 = 0.96. In other words, the new system capacity should be higher than the current values by 1.04 (= 1/0.96), which is the same value calculated above from the reliability-based calibration.

This example describes the close relationship available among the system factor ϕ_s, the reliability measure of redundancy $\Delta\beta_u$, and the system reserve ratio R_u. The example also demonstrates that the reliability-based calibration method proposed in *NCHRP Report 406* and adapted in this study, produces robust system factors ϕ_s that are valid for either wind or earthquake loads. The actual values of the reliability indices are different for different load types. The difference in the reliability indices of a substructure subjected to wind versus the same substructure subjected to earthquakes is due to the difference in the values of V_{LW} for wind and earthquakes. However, as seen in Equation 3.31, the system factor, ϕ_s, is independent of V_{LW} and thus the same system factor is valid for all load types as long as the same target system reserve ratio $R_{u\ target}$ (Equation 3.29) is specified.

3.5.6 Summary

The calculations performed in this section are provided to illustrate the procedure used during the course of this study. The target reliability indices and the calibration of the system factors are performed based on the results of several hundred bent configurations described and analyzed in Chapter 2. The subsequent sections of this chapter will provide the final results of the calibration procedure.

3.6 ANALYSIS OF TYPICAL BRIDGE SUBSTRUCTURE CONFIGURATIONS

The analysis of typical bridge substructure configurations is performed using the program PIERPUSH as described in Chapter 2. For two- and four-column bents, the analyses are performed based on eight types of soil/foundation systems and a variety of geometric and material properties. The following variations in the pertinent parameters are considered:

- The eight foundation systems are spread footings on normal and stiff soils; extension piles on soft, normal, and stiff soils; and multiple-pile systems on soft, normal, and stiff soils.
- The analysis is also performed for column bents with a variety of column heights. The heights considered are 4 m, 11 m, and 18 m for the two-column bents and 3.5 m, 6.5 m, and 9.5 m for four-column bents.
- Column widths are 0.8 m, 1.2 m, and 1.6 m for the two-column bents and 0.5 m, 1.0 m, and 1.5 m for four-column bents.
- Different longitudinal steel reinforcement ratios are also considered. These are 1.1 percent, 2.3 percent, and 3.5 percent for the two-column bents and 0.60 percent, 1.85 percent, and 3.10 percent for the four-column bents.
- The material properties used range from 400 Mpa, 450 MPa, and 500 MPa for the yielding stress of reinforcing steel and 22, 27, and 32 MPa for the concrete strength. These ranges are obtained from the survey of state DOTs as reported in Chapter 2 and Appendix C.

The base cases for the two-column bents and all eight-foundation systems are 11-m columns with 1.2-m widths 2.3 percent longitudinal reinforcing steel, 27 MPa for concrete strength, and 450 MPa for steel yielding stress. The base cases for the four-column bents and all eight-foundation systems are 6.5-m columns with 1.0-m widths 1.85 percent longitudinal reinforcing steel, 27 MPa for concrete strength, and 450 MPa for steel yielding stress. The dimensions and material properties associated with the base cases are those of the average column bents as reported from the survey of the state DOTs. To study the effect of variations from the base case, geometric and material properties and the dimensions of the columns are varied one at a time to cover the ranges mentioned above. The results of the analyses subjected to the dead load and vehicular live load and an increasing lateral load are tabulated for the two-column bents as shown in Tables D-1 through D-4 in Appendix D (not published herein). Similarly, Tables D-5 through D-8 give the loads for the four-column bents. The results are given for four limit states as follows:

1. The lateral load that causes one column to reach its moment capacity (first member failure) (Tables D-1 and D-5).
2. The lateral load that causes the crushing of one column assuming all columns are unconfined (Tables D-2 and D-6). This value is compared to the load that causes a collapse mechanism to form (as calculated in the Tables of Appendix C). If the mechanism forms before crushing occurs, the load that causes the mechanism is used in Tables D-2 and D-6 instead of the crushing load.
3. The lateral load that causes the crushing of one column assuming that all the columns are confined (Tables D-3 and D-7). This value is also compared to the load that causes a collapse mechanism to form as calculated in the Tables of Appendix C. If the mechanism forms before crushing occurs, the load that causes the mechanism is used instead.
4. Functionality limit state. This corresponds to the load that causes a maximum lateral displacement equal to clear height of column/50 (H/50) (Tables D-4 and D-8).

The loads causing the above-listed limit states are obtained using the analysis procedure described in Chapter 2. The nominal vertical dead and live loads are applied on the substructure system and are kept constant at their nominal (design values). During the analysis performed in this section to execute the calibration of the system factors, no load factors are applied in order to study the behavior of substructures under expected loading conditions. (Load factors will be used when engineers will implement the results of this study during the design process.) A lateral load is applied at the level of the column caps and is continuously incremented past the yielding point and into the nonlinear range. The load versus lateral deformation relationship is determined. During the analysis process, different critical loads and deformations are flagged. Particularly, the lateral loads corresponding to the four limit states cited above are recorded. The first limit state corresponds to the current member-based approach to the design and analysis of bridge substructures. The unconfined limit state governs the system capacity when the columns have low ductility capacity, as is the case when they are not provided with confining lateral reinforcement, resulting in the early crushing of column section as the substructure undergoes nonlinear deformations. Concrete crushing of unconfined members occurs when the strain in the concrete reaches a value equal to 0.004. The confined limit state governs when sufficient lateral reinforcement is provided to improve column ductility by raising the concrete crushing strain to 0.015 in./in. For both unconfined and confined limit states, a check is made to verify whether a system collapse mechanism occurs before any one column in the system crushes. If the mechanism occurs first, then the lateral load that causes the formation of the collapse mechanism controls is recorded.

The functionality limit state chosen for bridge substructures corresponds to a maximum total lateral displacement equal to the H/50. This limit state accounts for the displacements at the base of the pier due to soil and foundation flexibility as well as the bending of the columns due to the lateral load and the bending caused by the vertical loads due to P-delta effects. This functionality limit state, which is not necessarily a structural limit state, implies that bridge substructures may become "unsafe" for traffic passage because of large displacements even before a structural failure occurs. Besides the clear height/50 limit, several other possible displacement limits were investigated. Such limits include height/100, height/200, and 0.25 percent column drift. The height/50 limit state has been selected for the functionality limit state for the following reasons:

1. The height/100 and height/200 displacements often occur when the substructure is still in the linear elastic range and before the first column reaches its limit capacity. Because the focus of this study is on the behavior of bridge substructures after the failure of one element, the height/100 and height/200 are deemed too strict and not appropriate for use.
2. The 0.25 percent column drift considers the drift between the top and the bottom of the column only. This ignores the possibly large deformations in the soil and the foundations that may significantly contribute to the total lateral displacements. These soil/foundations displacements could be very high producing dangerous traffic conditions.

Additional limit states, other than those discussed above, were also considered, but are not used. These include the load that would produce a moment in any column from the vertical loads (from P-delta effects) equal to 30 percent of the total moment. This limit state relating to the extent of lateral deformation in the bent has not been selected since the effect of lateral displacement is already covered in the functionality limit state. Also, the P-delta effects are included in the bending moments that contribute to the formation of a collapse mechanism and the crushing of the concrete. The PIERPUSH program also flags the load at which the shear capacity of any column is exhausted. This load is not listed because shearing failures are always brittle and bridge substructures do not have any redundancy after the failure of a column due to shear. Thus, if the AASHTO LRFD has been calibrated to provide the same reliability index $\beta_{member} = 3.5$ for both shear and bending, then the system safety of bents that may fail in shear will not provide sufficient levels of system safety.

Bar pullout and failure of the joints are not addressed in this study. These failures are considered to be brittle and no system redundancy will exist when they occur. Thus, this project is concentrating on studying the behavior of bridge substructure due to flexural bending as the other failure types have no ductility and thus no redundancy.

The first rows of Tables D-1 through D-8 give the results obtained from PIERPUSH for the base cases. These correspond to the average height (11 m for the two-column bent and 6.5 m for the four-column bent); average width (1.2 m and 1.0 m, respectively); average yielding stress of reinforcing steel f_y (450 MPa); average concrete strength f'_c (27 MPa); and average longitudinal steel reinforcement ratio (2.3 percent for the two-column bents and 1.85 percent for the four-column bents). The base cases consider eight different soil/foundation systems (Columns 1 through 8 of Tables D-1 through D-8). The soil foundation systems are (1) spread footings on normal soils, (2) spread footings on stiff soils, (3) extension piles on soft soils, (4) extension piles on normal soils, (5) extension piles on stiff soils, (6) multiple piles on soft soils, (7) multiple piles on normal soils, and (8) multiple piles on stiff soils. Soft soils are defined as soils that produce a N = 30 blow counts or higher.

The second rows in Tables D-1 through D-8 give the results for the short columns (3.5 m for the two-column bents, 4 m for four-column bents) when all the other properties are kept at their average values. The third rows are for the cases with high columns (18 m for two-column bents and

9.5 m for the four-column bents). The fourth row is for the cases with small widths (0.8 m for two-column bents and 0.5 m for the four-columns). The fifth row is for columns with large widths (1.6 m for two-column bents and 1.5 m for the four-columns). The sixth row is for columns with low concrete strength ($f'_c = 22$ MPa). The seventh row is for the cases with high concrete strength ($f'_c = 32$ MPa). The eighth row is for columns with low steel-yielding stress (f_y = 400 MPa). The ninth row is for the cases with high-yielding stress (f_y = 500 MPa). The tenth row is for the cases where the longitudinal steel-reinforcing ratio is low (1.1 percent for two-column bents, 0.6 percent for the four-columns). The eleventh row is for the cases where the longitudinal steel-reinforcing ratio is high (3.5 percent for two-column bents, 3.10 percent for the four-columns). All results are given in kN. The interpretation of the results and a discussion of the trends in the load obtained are discussed in detail in Chapter 2.

3.7 SYSTEM RESERVE RATIOS OF TYPICAL SUBSTRUCTURE CONFIGURATIONS

As mentioned in Section 3.5, the redundancy of the substructures analyzed is closely related to the system reserve ratios, R_u (unconf.), R_u (conf.), and R_f, which are defined as ratios of the loads producing the system limit states analyzed above to the load producing the first member failure. Specifically, R_u (unconf.) is the system reserve ratio for the ultimate state of unconfined columns; R_u (conf.) is the system reserve ratio for the ultimate limit state of confined columns; and R_f is the system reserve ratio for the functionality limit state. These system reserve ratios are provided in Appendix D, Tables D-9, D-10, and D-11 for the two-column bents and in Tables D-12, D-13, and D-14 for the four-column bents.

The results reflect a wide range of reserve ratios for the different substructure geometries, material, and foundation types considered. The ratio varies from a low of 0.17 for the functionality limit state of short two-column bents with extension piles on soft soil to a high value of 1.80 for the system limit state of confined four-column bents on extension piles in normal soils.

In general, the four-column bents show higher reserve ratios than two-column bents although a direct comparison is difficult to make because of the different column heights and widths used in the analysis. However, in comparing the low height columns that have heights of the same order of magnitude (i.e., 3.5 m for the four-column bents and 4 m for the two-column bents), the researchers notice that the reserve ratios are generally only slightly higher for the four-column bents. For the unconfined columns, the difference between the reserve ratio of the two-column bents and four-column bents vary from 0.02 to a maximum of 0.09. For the confined columns, the two-column bents give higher reserve ratios for the spread footings than those of the four-column bents (1.50 compared to 1.36). For other foundation types, the four-column bents produce higher reserve ratios with a difference up to 0.39 for the multiple piles on stiff soils. One reason for this inconsistent trend in the results is the different foundation stiffnesses used for the two-column and four-column bents. The foundation stiffnesses affect the distribution of the load to the individual columns differently depending on the stiffnesses of the columns and those of the column caps. The list of foundation stiffnesses used in the analysis were presented in Chapter 2.

The results indicate that the effect of changes in material properties is generally the least significant. For example, looking at confined two-column bents supported by extension piles on stiff soils, the range of the reserve ratio for the average cases and the cases with different concrete strength and steel-yielding stress is from 1.44 to 1.55 with an average value of 1.50. For the four-column bents this ranges is from 1.54 to 1.64 with an average value of 1.58. Hence, changing the material properties of the reinforcing steel and the strength of concrete from the average values of 450 Mpa and 27 Mpa, respectively, will change the reserve ratio by a maximum of value of 0.06. Such a narrow range in reserve ratios indicates that structural redundancy is insensitive to these strength parameters as compared with others.

It is noted that providing lateral reinforcement to confine the concrete columns will generally increase the ductility of the system producing higher system reserve ratios. Confining the columns may not improve the system reserve ratio if a mechanism forms before any one-column crushes. In other instances, confining the columns means that large lateral displacements take place before the crushing of a column occurs; hence, the bridge, although structurally safe, may be unfit for use, and the functionality limit state would govern. For example, this situation is observed for the four-column bent of low height (3.5 m) supported on pile extensions in soft soils. In this case, the reserve ratio for the functionality limit is only 0.45 while the system reserve ratio for the confined columns is 1.16 and for unconfined columns is 1.05. The flexibility of the foundation/soil system for this case renders the bridge susceptible to very high deformations before any permanent structural damage would occur. The reserve ratio of 0.45 for the functionality limit state indicates that the lateral load producing a total lateral displacement of H/50 is lower than that producing the failure of the first member. The functionality limit state is thereby considered to be of utmost importance for structures founded on soft soil conditions, and susceptible to foundation vulnerability.

3.7.1 Determination of Target Reserve Ratio

Current design practice for bridge substructures assumes that four-column bents are adequately redundant. Also, most bridge substructures (except for those in earthquake-prone regions) are designed with *unconfined* columns. Hence, it is herein proposed to consider the average system reserve ratio, R_u, from "*average four-column bents with unconfined*

columns" as the target ratio that all properly designed substructures should satisfy. According to the first row of Table D-12, the average four-column bent reserve ratio for average column height, column width, and material properties produces an average value equal to 1.20 (average of all foundation types). This average value is designated as the target value that any bridge substructure should satisfy so that it is considered "adequately redundant." This target reserve ratio of $R_{u\ target} = 1.20$ implies that bridge substructures should be able to withstand 20 percent more lateral load than the load that causes the first member failure before they actually collapse (due to either a mechanism or the crushing of the concrete in a column).

The target value of 1.20 for substructure system redundancy is lower than the 1.30 target value proposed in *NCHRP Report 406* for bridge superstructures. The difference reflects current design standards. The justification behind using the 1.20 value is that the industry is "on the average" satisfied with the safety of bridge substructures designed to satisfy current standards. Thus, the system factors should reflect that level of satisfaction by using the average value of 1.20 as target. The difference between the proposed 1.20 for substructures and 1.30 for superstructures may appear to reflect that current substructure designs are less redundant than superstructure designs. However, a direct comparison is not obvious because the actual reliability levels implied by the target system reserve ratios depend on both the reserve ratio and the COVs. Also, the different target reliability levels associated with the 1.20 reserve ratio for substructures and the 1.30 reserve ratio for superstructures would reflect the relative cost margins associated with increasing the reliability of the superstructure as compared to the costs associated with increasing the reliability of the substructure and foundations. The issue of including the cost margins in the determination of the target reliability levels is beyond the scope of this study and is being addressed in NCHRP Project 12-48.

Assuming a lognormal reliability model and using the relationship developed in Section 3.5 (Equation 3.27), a system reserve ratio of 1.20 corresponds to a $\Delta\beta_u$ of 0.52, which is rounded down to 0.50. The calculations of β were also performed using a FORM with an Extreme Type I Gumbel distribution for the applied load and a lognormal distribution for the system capacity of unconfined columns (represented by the LF factors and using the same mean and COVs provided in Section 3.5). The FORM algorithm produced a $\Delta\beta_u$ of 0.49 for the average four-column bent. As a result, a *target relative reliability index* $\Delta\beta_{u\ target}$ of 0.50 is specified in this project.

The functionality limit state is proposed to ensure that large levels of deflections that make the bridge lose its functionality occur only at sufficiently high levels of loads. In this case, it is decided to also use a target value $\Delta\beta_{f\ target}$ of 0.50 ($R_{f\ target} = 1.20$) for the functionality limit state. The same value is used for the functionality limit state because the tables of the system reserve ratios given in Appendix D show that the functionality limit state and the ultimate limit state occur at similar load levels for the four-column bent with confined concrete for the base case with 6.5-m columns and most common foundation/soil types. Thus, the same reserve ratio of 1.20 (coinciding with a $\Delta\beta_{target}$ of 0.50) is used as the required target for both the functionality and ultimate limit states such that the bents that produce higher system reserve ratios (whether by confining the concrete or changing the geometric configuration) are considered to be adequately designed for redundancy consideration. Notice that the functionality limit is not intended to eliminate designs that can safely exhibit large deformations but, on the contrary, is intended to ensure that the deformations occur at sufficiently high load levels. Such high deflections after first yielding reflect large levels of ductility, which is encouraged. Three factors may make a bridge substructure not meet the functionality criterion: (a) the system collapses early at a system reserve ratio less than $R_{f\ target}$ before it undergoes a sufficient level of ductility, (b) the system undergoes large levels of total deformations at an early load level before $R_f = 1.20$ is reached, and (c) the system's P-Δ effects contribute to causing system unloading such that R_f reduces to less than 1.20 when H/50 is reached. Notice that most unconfined columns reach their ultimate limit state before the functionality limit state. Thus, most unconfined columns will not satisfy the functionality limit state. Factor (a) is automatically met when the system reserve ratio R_u for the ultimate limit state is checked because, in this report, $R_{f\ target}$ and $R_{u\ target}$ are both equal to 1.20. Factor (b) is the most relevant situation for substructures subjected to the effect of increasing lateral forces. Factor (c) is relevant in the cases where displacement controlled loading governs (such as for substructures under earthquake loads). In the Specifications proposed in Appendix A, it is recommended that bridges that are classified as noncritical and not essential need not satisfy this functionality criterion. Imposing the functionality criterion for nonessential bridges might require a large program of rehabilitation and a heavy financial burden that would be beyond the means of bridge authorities.

3.7.2 Damaged Bridge Substructures

To study the behavior of the multiple column bents for the damage scenario, the four-column bent with all average geometric configuration and material properties (6.5-m high and 1.0-m × 1.0-m-wide columns, $f'_c = 27$ MPa, $f_y = 450$ Mpa, longitudinal reinforcing ratio = 1.85 percent) supported by spread footings on normal soil is analyzed assuming that its exterior column is totally damaged and unable to carry bending moment or axial load. The damaged bent was analyzed using the program PIERPUSH under the effect of the nominal (design) vertical dead and live loads and for an increasing lateral load. The analysis shows that, at a lateral load equal to 967 kN, one of the remaining columns will reach its maximum moment capacity. The first column crushes

when the applied lateral load reaches a value equal to 2187 kN if the column is unconfined. If the columns are confined, then the first column crushes at a lateral load equal to 3817 kN. The collapse mechanism is reached when the lateral load is 3821 kN. These values from the damaged substructure are compared with the following values for the intact structure: a first member failure at a lateral load equal to 3787 kN; a first crushing for unconfined columns when the lateral load is equal to 4659 kN; and a first crushing at a load equal to 4801 kN if the columns are confined; a collapse mechanism would form at a lateral load equal to 4856 kN. A careful review of the bending moments in the cap of the damaged structure under the effect of the applied gravity (vertical) load and the applied lateral load, when the limit states are reached, revealed that when the column caps are designed to current standards for the intact system, they are unable to resist the applied gravity load moments in the damaged condition, as shown in Chapter 2. Hence, current design procedures would not produce adequate reserve ratio for damaged multicolumn bents. However, if the column caps are strengthened, then the columns will show a reserve ratio for the damaged condition, R_d, equal to 0.58 (3821 kN/3787kN) for damaged bents formed by four unconfined columns. This ratio is higher than the $R_d = 0.50$ used in NCHRP Project 12-36 for damaged superstructures. Since the 1.20 ratio used for intact substructures is less than the 1.30 used for intact superstructures, it is suggested that the ratio for the damaged substructures should be less than or, at most, equal to that of damaged superstructures.

Therefore, it is proposed to use a minimum required system reserve ratio for damaged substructure, $R_d = 0.50$. If a damaged substructure system has a R_d value greater than the required 0.50 value, the substructure is defined as adequately redundant for the damaged scenario. The typical bridge substructures analyzed in this project are deemed *not* to have enough redundancy to sustain damage to an exterior column and still be able to carry some of the applied loads until repairs are completed. To achieve the required system reserve ratio, $R_d = 0.50$, the column caps must be designed to withstand a column failure. It is observed that using $R_d = 0.50$ in Equation 3.27, the target system safety index obtained would be equal to $\Delta\beta_d = -1.96$. These calculations assume that the COV for the capacity $V_{LF} = 13$ percent and the COV for the applied lateral load is $V_{LW} = 33$ percent corresponding to the uncertainty associated with a 75-year design life. This calculation could be rounded off to target value $\Delta\beta_{d\ target} = -2.00$. This assumption implies that if bridge columns are designed to satisfy a member reliability index $\beta_{member} = 3.5$, as specified in the AASHTO LRFD, then the substructure will be considered to be adequately redundant if after the brittle failure of any one column, the substructure system will still be able to provide a system reliability index for the damaged substructure, $\beta_{damaged} = 1.50$ (= 3.50 − 2.00). A reliability index of 1.50 corresponds to a probability of failure equal to 6.7 percent under the combined effects of vertical loads and the 75-year maximum lateral load. It should be noted that the total probability of failure for a damage state is equal to the probability of occurrence of the damage initiating event times the probability of system failure given that the damage has occurred. Thus, the reliability index of 1.50 given the occurrence of a damage-causing event (collision, scour, major corrosion) should be adequate. If the probability that a damaging event has occurred is less than or equal to 0.00348 (0.348 percent), then, the unconditional probability of failure becomes less than or equal to 0.2326×10^{-3} (= 0.00348×0.067) corresponding to an unconditional reliability index β greater than 3.5. Or, for a probability of the occurrence of a damaging event less than 4.72×10^{-4}, the unconditional reliability index is greater than 4.0. Furthermore, conditional probability of failures lower than 6.7 percent will be obtained for winds with lower return periods. For example, if one assumes that any damage to a substructure would at a minimum be detected during the biennial inspection, then the maximum possible load that a damaged substructure will be subjected to would be, at most, equal to the load observed over a 2-year return period (i.e., the time between two inspections.) According to Ellingwood et al. (1980), the maximum 1-year wind load is associated with a COV = 59 percent. The corresponding 2-year COV, V_{LW_2} would then be equal to 53 percent. The wind load bias for a 2-year period is found to be equal to 0.37. Plugging these values into Equation 3.22 with a target $R_d = 0.5$, the corresponding value of $\Delta\beta_{d\ target}$ becomes −1.24 for a 2-year return period. The reliability index, β_{member} = 3.5 for a 75-year return period would produce a reliability index $\beta_{member} = 3.84$ for a 2-year period, thus a $\Delta\beta_{d\ target}$ of −1.24 implies a reliability index for the damaged system, $\beta_{damaged} = 2.60$ for a 2-year period or a probability of failure given that a damage has occurred $P_f = 0.466 \times 10^{-2}$. Such levels of reliability indices and probability of failure are quite reasonable.

3.8 SYSTEM FACTORS FOR TYPICAL BRIDGE SUBSTRUCTURE CONFIGURATIONS

As observed in Section 3.7, the analysis of the average four-column bents for the unconfined columns produced an average system reserve ratio, $R_u = 1.20$. Because four column bents are normally considered to provide adequate levels of redundancy and most bridges outside of earthquake prone regions are designed with unconfined columns, it is recommended *to use a $R_u = 1.20$ as the target system reserve ratio that an adequately designed substructure system should achieve*. Bridge configurations that do not satisfy this requirement will have to be designed for higher member safety (i.e., they must be associated with lower values of system factors, ϕ_s). On the other hand, bridge configurations that provide higher R_u values may be designed to have lower member safety levels (i.e.,they are associated with higher values of system factors, ϕ_s) with the understanding that the system as a whole will still provide adequate safety against collapse.

Using a target $R_{u\ target} = 1.20$, the system factor ϕ_s is calculated for each bridge configuration studied in this project using the approach described in Section 3.5 above (Equation 3.31). The results are provided in Tables D-15 through D-20 of Appendix D for different geometric, material, and soil/foundations types of the two-and four-column bents with unconfined and confined columns. This target $R_{u\ target} = 1.20$ was found to correspond to a system reliability margin $\Delta\beta_u$ of 0.50. Note that if the bridge members are designed for a reliability index $\beta_{member} = 3.5$, as is specified in AASHTO LRFD, it will indicate that the probability of first member failure is equal to 0.023 percent. Requiring a $\Delta\beta_u = 0.50$ results in a system reliability index $\beta_{ult.} = 4.0$. Thus, an increment of the reliability index of 0.50 for system failure means that the probability of system collapse must be lower than 0.0032 percent.

The calibration of the system factors are done for all limit states assuming that all the member capacities, system capacities, and load variables follow lognormal distributions as shown in Section 3.5. The calibration results are found not to be significantly affected by the type of probability distribution chosen during the analysis process. The maximum difference observed in the system factors is less than 0.01 if the Extreme I (Gumbel) distribution is used for the applied lateral load.

The following observations are made from the results presented in Appendix D:

1. The range of the system factors is quite wide with a low value of 0.37 to a maximum of 1.50 for the four-column bents and a minimum of 0.14 and a maximum of 1.46 for the two-column bents. This range demonstrates a wide spread in the levels of redundancy of the bents analyzed.
2. The variation between the system factors of the bents analyzed is highly dependent on the foundation type and the overall flexibility of the system. The relative flexibility of the foundation/columns affects the moment distribution of the columns and thus causes large differences in the lateral load that causes the first member failure. In addition, flexible foundations and columns produce higher lateral displacements that induce higher moments due to the P-Δ effects.
3. The effect of column size is difficult to predict but seems significant. This difficulty is due to the effect of changes in column heights and column widths on the design of the foundations, which is reflected by different foundation stiffnesses.
4. The four-column bents provide higher system factors than the equivalent two-column bents. This observation is shown in Table D-21 where the results for a 4-m-high, 1.2-m-wide four-column bent are obtained by interpolation for the results shown in Tables D-18, D-19, and D-20. The increase in system factors may be as high as 0.29 as seen for the functionality limit state on multiple piles in stiff soils. The results show three exceptions: for the confined columns on spread footings in normal and stiff soils and for the functionality limit state on spread footings in stiff soils. These exceptions may be due to the effects of the differences in the foundation stiffnesses.
5. As expected, confined columns with higher ductility capacity produce higher system factors than unconfined columns. The highest difference is 0.38 for the four-column bent of 6.5-m-high column with a 1.5-m width on extension piles on stiff soils. For the two-column bents the largest difference between confined and unconfined column bents is 0.28 for the 4-m-high column with a 1.2-m width on spread footings with normal soils. In a limited number of instances, the P-delta effects and the resulting softening may cause the confined columns to produce lower system reserve ratios than unconfined columns.
6. Changes in material strength have *no* significant effect on the system factors. This is due to the fact that changes in material strengths while keeping all the other variables constant would change the bending capacity of the individual columns but, as seen in Section 3.5, the change in the bending capacity of the columns will generally change the system capacity by about the same factor keeping the system reserve ratio (and thus the system factor) practically unaffected.
7. Higher column longitudinal reinforcements reduce column ductility and thus decrease the system factors. Lower column longitudinal reinforcements increase column ductility and increase the system factors. The increase and decrease in system factor results in an average change of 0.10 in the system factors from the all-average cases.

The system factor tables developed in Appendix D could be used during the design of the columns of bridge substructures that have similar geometric and material properties as those used during the derivation of Tables D-15 to D-20. This approach will be explained in Section 3.10. For bridge configurations that are different than those provided in the tables, a direct check of the redundancy can be carried out as explained in Section 3.9.

3.9 DIRECT REDUNDANCY CHECK

The direct redundancy check procedure proposed in this section is based on satisfying minimum values of the system reserve ratio, R_u, and relative reliability index $\Delta\beta_u$.

The conclusions reached in the previous section revealed that bridge substructures with typical four-column configurations and average column dimensions designed without additional confining lateral reinforcement that have adequate levels of redundancy produced a system reserve ratio

for the ultimate limit states $R_u = 1.20$ or higher. This is found to be equivalent to a relative system reliability index, $\Delta\beta_u$, for the ultimate limit state equal to 0.50 or higher for lateral wind loads.

Based on these results, it is recommended that a bridge substructure system be defined as adequately redundant if the analysis of the substructure produces a system reserve ratio for the ultimate capacity, R_u, greater than or equal to 1.20. The required $R_{u\ req.}$ value of 1.20 means that the lateral load producing the collapse of the bridge substructure should be 20 percent higher than the lateral load that will cause the first member to reach its nominal moment capacity.

In addition, this study proposes a functionality limit state that is defined as the lateral load that will produce a total lateral displacement equal to H/50. A substructure is defined to be adequate for the functionality limit state if the load that causes a total lateral displacement of H/50 (where H is the clear column height) produces a system reserve ratio of $R_f = 1.20$. Notice that the functionality limit state is not a restriction on the level of deflection but it requires that these deflections occur at high loads ensuring an acceptable level of reserve strength before the system loses functionality (in this case a system reserve ratio of 1.20 is set as the criterion). Large levels of deflections after the system goes into the nonlinear range are an indication of system ductility.

Finally, bridge substructures are defined to be adequately redundant following the brittle loss of one column if the substructure will still be able to carry 50 percent of the lateral load that causes the failure of the first member in the intact structure. This is equivalent to a system reserve ratio for damaged substructures $R_d = 0.50$. These criteria are for bridge substructures with members designed to exactly satisfy the current AASHTO member design criteria. If bridge members are overdesigned, the criteria are adjusted to account for the additional safety as will be further explained in this section.

The $R_u = 1.20$ value proposed herein for the ultimate limit state of bridge substructures is less conservative than the 1.30 value used in *NCHRP Report 406* for the bridge superstructures. The 1.20 value is accepted in order to remain consistent with current methods for designing redundant bridge substructure systems. The $R_f = 1.20$ value proposed for the functionality limit state of substructures is slightly higher than the 1.10 used in *NCHRP Report 406* for superstructures. Note that the definition of the functionality limit state in *NCHRP Report 406* is different than that used in this study because of the differences in the structural system and behavior. The superstructure limit state was defined in *NCHRP Report 406* as the load producing a vertical displacement equal to span length/100. The $R_d = 0.50$ used in this report for the damaged limit state is the same as that used in *NCHRP Report 406* for the superstructure limit state.

It should be noted that the check of R_u, R_f, and R_d is a check on the redundancy of the system. *Bridges that are not redundant may still provide high levels of system safety if their members are overdesigned*. Therefore, the redundancy check should always be performed in conjunction with a member safety check. This check is achieved by comparing the actual capacity of the bridge members with the capacity required by the specifications. In this case, $R_{req.}$is defined as the member capacity required to satisfy AASHTO Specifications. Any acceptable member design criteria can be used. For example, the required nominal member capacity R_{req} is calculated for the most critical member using AASHTO's LRFD design and evaluation equations as

$$\phi R_{\mathrm{req}} = \gamma_d D_n + \gamma_l L_n + \gamma_w W_n \tag{3.32}$$

where ϕ is the resistance factor, γ_d is the dead load factor, γ_l is the live load factor, γ_w is the lateral load factor, D_n is the nominal or design dead load, L_n is the nominal or design live load including impact, and W_n is the lateral load (e.g., wind). Equation 3.32 has a general format that can be used for any AASHTO criteria. In LRFD, the ϕ factor depends on the type of material, g_d depends on the type of dead load (e.g., $\gamma_d = 1.25$ is used for component dead load). γ_l depends on the load combination used. For example, when wind load is applied on the structure, γ_l is either 1.35 or 0 combined with a load factor for wind of 0.40 or 1.40, respectively. Notice that during the actual implementation of the system redundancy procedure, factored loads are being used in order to remain consistent with the AASHTO LRFD methodology although the derivation of the system factors was based on the unfactored loads (that had been determined to be similar to the expected loads).

The required lateral load factor for one member, $LF_{1\ req.}$ is defined as

$$LF_{1\ \mathrm{req.}} = \frac{\phi R_{\mathrm{req.}} - \gamma_d D_n - \gamma_l L_n}{W_n} = \gamma_w \tag{3.33}$$

where $R_{req.}$ is the required member capacity obtained from Equation 3.32; D_n is the nominal dead load effect on the most critically loaded member; L_n is the nominal live load effect on the most critical column; W_n is the effect of the lateral load on the most critical member; ϕ, γ_d, γ_l, and γ_w are the member resistance and load factors.

Equation 3.33 indicates that if the columns of a substructure are designed to exactly satisfy current design standards, and if the factored dead load and live loads are applied on the structure, the lateral load factor that will cause the failure of the first member, $LF_{1\ req.}$ must be equal to the design load factor of the lateral load, γ_w. If the bridge columns are overdesigned, then the load factor LF_1 needed to cause the first member failure will be higher than that obtained from Equation 3.33. Conversely, if the bridge columns are underdesigned, then the load factor LF_1 will be lower than that obtained from Equation 3.33. The load factor LF_1 corresponding to the actual (provided) member capacity can be represented as

$$LF_1 = \frac{\phi R_{\mathrm{provided}} - \gamma_d D_n - \gamma_l L_n}{W_n} \tag{3.34}$$

To provide a measure of the adequacy of the actual member capacity represented by LF_1 to that required by the AASHTO specifications, the member reserve ratio r_1 is defined as follows:

$$r_1 = \frac{LF_1}{LF_{1\,\text{req.}}} = \frac{\dfrac{\phi R_{\text{provided}} - \gamma_n D_n - \gamma_n L_n}{W_n}}{\dfrac{\phi R_{\text{required}} - \gamma_n D_n - \gamma_n L_n}{W_n}} = \frac{\phi R_{\text{provided}} - \gamma_n D_n - \gamma_n L_n}{\gamma_w W_n} \tag{3.35}$$

The member reserve ratio, r_1, defined in Equation 3.35, is equivalent to the rating factor but applied to the lateral load (e.g., wind load) instead of the vehicular live load. Bridge columns that are designed to exactly match the AASHTO specifications will produce a member reserve ratio of 1.0, while members that are overdesigned will produce r_1 values higher than 1.0. It should be noted that this member evaluation procedure can be used with any bridge design criteria including WSD, LFD, and LRFD or bridge evaluation procedures including operating and inventory rating levels using HS20, HL-93 live loads or any other appropriate loading.

The member reserve ratio, r_1, is used in conjunction with a check of the system reserve ratio, R_u, to recommend system factors using the redundancy check concept as outlined in the direct redundancy analysis procedure described below.

3.9.1 Direct Redundancy Analysis Procedure

This section presents a direct method to determine the redundancy level of a bridge substructure using a detailed push-over nonlinear finite element analysis. The procedure can be used in conjunction with any member checking criteria including AASHTO's WSD, LFD, or LRFD for either inventory or operating ratings. The steps involved in the analysis of the redundancy of a given bridge substructure are:

Step 1. Use the AASHTO specifications to find the required column bending capacity, $R_{\text{req.}}$, for the bridge columns using Equation 3.32. Determine $LF_{1\,\text{req.}}$ from Equation 3.33.

Step 2. Develop a structural model of the bridge to conduct the static nonlinear pushover analysis of the substructure accounting for P-delta effects. In the model, use the nominal material properties of the columns and the best estimate for the foundation stiffnesses. Apply the factored dead and live loads as specified in the AASHTO manual for the case being analyzed.

Step 3. Use the applicable AASHTO Specification to find the magnitude of the lateral load (e.g., 75-year wind load) that should be applied on the substructure, that is, the nominal lateral load, W_n.

Step 4. Apply the nominal lateral load, W_n, on the substructure and keep increasing the load until the first column reaches its strength capacity. The factor by which the original W_n is multiplied for the first failure to occur is defined as LF_1. Take the ratio of LF_1 to $LF_{1\text{req}}$ to calculate the member reserve ratio, r_1, from Equation 3.35.

If the column is designed to exactly satisfy the AASHTO specifications, then $r_1 = 1.0$. Overdesigned members will have values of r_1 greater than 1.0.

Step 5. Continue the pushover analysis beyond the failure of the first member and keep incrementing the applied nominal lateral load until one of the following nonlinear events is met:

a. One of the columns reaches its compressive crushing strain, or
b. A collapse mechanism is formed.

Note the load factor LF_u by which the original lateral load is scaled to achieve either of these two limit states. Calculate the ratio $R_u = LF_u/LF_1$. If the ratio is higher than 1.20, then the bridge has a sufficient level of redundancy to satisfy the required redundancy criteria. Calculate the redundancy ratio for ultimate limit state r_u

$$r_u = \frac{R_u}{1.20} \tag{3.36}$$

Step 6. Continue the incremental loading (if necessary) and record the load factor LF_f at which a total lateral displacement equal to H/50 is reached (H being the clear column height). Calculate the ratio $R_f = LF_f/LF_1$. If the ratio is greater than 1.20, then the bridge substructure has enough ductility to satisfy the functionality limit state. Calculate the redundancy ratio for the functionality limit state r_f

$$r_f = \frac{R_f}{1.20} \tag{3.37}$$

Step 7. Identify substructure members whose failure might be critical to the structural integrity of the substructure system. These damage scenarios should be specified in consultation with the bridge owner. Such members could be (a) columns that can be damaged by an accidental collision by a vehicle, ship, or debris; (b) foundations that may be washed out because of scour; or (c) members (e.g., columns, steel, or prestressed concrete piles and extension piles) that are prone to corrosion damage or fatigue fracture.

Step 8. Remove one of the members identified in Step 7 from the structural model and repeat the pushover analysis. Determine the load factor of the damaged bridge LF_d that will cause either the crushing of one of the remaining columns or the formation of a collapse mechanism in the remaining portion of the substructure. Total displacement is not checked for the damaged limit state because a damaged bridge has already lost its functionality. The damage check is to ensure that the bridge will still be able to carry some loads until appropriate repairs are effected. Take the ratio $R_d = LF_d/LF_1$, where LF_1 is the load at the first member failure of the intact structure. If R_d is higher than 0.50, then the bridge has a sufficient level of redundancy to satisfy the required redundancy

criteria. Calculate the redundancy ratio for the damaged limit state r_d

$$r_f = \frac{R_f}{1.10} \tag{3.38}$$

Step 9. Place the member removed in Step 7 back into the model and remove another critical member. Repeat Step 8 until all the critical members identified in Step 7 are checked. Take the lowest value of r_d as the final redundancy ratio for the damaged limit state.

Step 10. If all the redundancy ratios, r_u, r_f, and r_d obtained from the pushover analyses are larger than 1.0, the bridge substructure has a sufficient level of redundancy. If one redundancy ratio is smaller than 1.0, the bridge substructure does not have a sufficient level of redundancy and corrective measures may need to be undertaken to improve substructure safety unless the bridge is sufficiently overdesigned (see further below). Corrective measures may include the strengthening of bridge columns, changing the bridge topology, or decreasing the rating of the bridge.

To improve the redundancy of a bridge substructure, the geometric configuration may be changed by adding columns. If this cannot be achieved, nonredundant substructures must be penalized by requiring their columns to provide higher safety levels than those of similar substructures with redundant configurations. By strengthening the columns, an overall satisfaction of the system reliability target is achieved. The member strengths of nonredundant substructures should be improved by increasing the column strength $R_{provided}$ by an additional safety factor such that the final resistance R_{final} is calculated as

$$R_{\text{final}} = R_{\text{provided}} / \min\,(r_1 r_u,\ r_1 r_f,\ r_1 r_d) \tag{3.39}$$

where r_1 is the member reserve ratio defined in Equation 3.35; r_u is the redundancy ratio for the ultimate limit state defined in Equation 3.36; r_f is the redundancy ratio for the functionality limit state defined in Equation 3.37; r_d is the redundancy ratio for the damaged limit state defined in Equation 3.38.

Using Equation 3.39 is equivalent to specifying that the required column capacity be obtained from a modified AASHTO-specified checking equation by adding a system factor such that the member capacity should satisfy the equation

$$\phi_s\, \phi\, R_{\text{req.}} = \gamma_d D_n + \gamma_l L_n + \gamma_w W_n \tag{3.40}$$

where ϕ_s is the system factor that is defined as a multiplier relating to the safety, redundancy, and ductility of the bridge substructure system; ϕ is the resistance factor; γ_d is the dead load factor; γ_l is the live load factor; γ_w is the lateral load factor; D_n is the nominal or design dead load; L_n is the nominal or design live load including impact; and W_n is the lateral load (e.g., wind).

The system factor ϕ_s is calculated as

$$\phi_s = \min\,(r_u,\ r_f,\ r_d) \tag{3.41}$$

If ϕ_s is less than 1.0, it indicates that the substructure under consideration has an inadequate level of system redundancy. A system factor greater than 1.0 indicates that the level of substructure system safety is adequate. The system factor ϕ_s is a penalty-reward factor whereby bridges with nonredundant configurations would be required to have higher column capacities than similar substructures with redundant configurations. On the other hand, redundant configurations will be rewarded by allowing their columns to have lower capacities. Notice that Equation 3.39 is not exactly equal to Equation 3.40 when Equation 3.41 is substituted. However, the sensitivity analysis performed in Section 3.5.4 shows that the approximation is reasonably close for practical situations when the lateral load is dominant.

To ensure that a minimum level of column safety is maintained, it is recommended that a maximum ϕ_s value of 1.20 be used as an upper limit. The 1.20 limit is based on the maximum load modifier factor proposed in the AASHTO LRFD Specifications. In fact, the LRFD Specifications propose a minimum load modifier of $0.95 \times 0.95 \times 0.95$. This product produces a minimum value of 0.86, which is equivalent to a maximum redundancy factor of 1.17. On the other hand, the minimum value of ϕ_s of 0.80 is proposed herein. That is, the maximum penalty that a nonredundant substructure is assigned is 20 percent while the maximum reward is also 20 percent. The 40 percent range is also the same range proposed in *NCHRP Report 406* for members of bridge superstructures. A minimum value is proposed so that future designs will not be drastically different than current designs. A 20 percent difference in the required member capacity is considered substantial and higher differences might meet resistance from the industry.

The same system factor, ϕ_s, may be applied to all the members of the bridge substructures. In reality, if the substructure is formed by columns of unequal lengths and capacities some columns may contribute less than the others toward the overall substructure system capacity. Using the same ϕ_s factor for all the columns may be inefficient in such cases. To be more efficient, the system factor ϕ_s may be applied to the most critical column(s) only and the full analysis described above repeated until the system redundancy requirement is satisfied.

It should be noted that applying a redundancy factor ϕ_s less than 1.0 will improve the substructure's column strengths represented by LF_1 and will also improve the substructure system strength expressed in terms of LF_u. Thus, the system reserve ratios, R_u, will remain unchanged and a nonredundant bridge will remain nonredundant. However, by applying a redundancy factor ϕ_s of less than 1.0, the reliability index for one column as well as the system reliability indices will be increased. Thus, nonredundant designs are penalized by

requiring higher component (column) safety levels than similar bridges with redundant configurations.

The procedure proposed in this section to perform the redundancy evaluation of bridge substructures is compatible with the method proposed in *NCHRP Report 406* for the analysis of superstructure redundancy. The procedure assumes that by changing the member capacity by the ϕ_s factor, both the LF_u and LF_1 values are changed by the same factor. An iterative process can be used by repeating Steps 2 through 10 to verify this assumption.

The procedure is applicable for all substructure configurations including two-column, and multicolumn substructures. It requires a validated pushover analysis program that would account for nonlinear behavior of columns including P-delta effects and foundation flexibility and that is capable of determining the lateral loads that will cause the crushing and the collapse mechanism of substructure systems.

3.10 SYSTEM FACTORS FOR COMMON TYPE BRIDGE SUBSTRUCTURES

The previous section provided a direct analysis procedure to evaluate the redundancy of bridge substructures. The direct analysis requires the engineer to use a program capable of performing a pushover nonlinear analysis of bridge substructures. The direct analysis evaluates substructure redundancy leading to system factors, ϕ_s, that provide measures of redundancy. Section 3.8 and Appendix D developed sets of system factors ϕ_s applicable to common-type two-column and multicolumn substructures supported by different soil-foundation systems. The system factors are calibrated for use in the design check equation of bridge columns such that

$$\phi_s\, \phi\, R' = \gamma_d\, D_n + \gamma_l\, L_n + \gamma_w W_n \qquad (3.42)$$

where the system factor, ϕ_s, is defined as a multiplier relating to the safety, redundancy, and ductility of the substructure. The system factors presented in Tables D-15 to D-21 are applied to the factored *nominal beam column bending capacity*. The proposed system factors replace the load modifier, η, used in Section 1.3.2 of the LRFD Specifications. The factor ϕ_s is placed on the left side of the equation because the system factor is related to the capacity of the system and as such should be placed on the resistance side of the design equation as is the norm in LRFD methods. In Equation 3.42, ϕ is the member resistance factor; R' is the required resistance capacity of the column accounting for the redundancy of the system; γ_d is the dead load factor; D_n is the dead load effect; γ_l is the live load factor; L_n is the live load effect on an individual member including the dynamic impact factor; γ_w is the live load factor; and W_n is the lateral load effect. When ϕ_s is equal to 1.0, Equation 3.42 becomes the same as the current design checking equation. If ϕ_s is greater than 1.0, it indicates that the system's configuration provides sufficient level of redundancy. When it is less than 1.0, the level of redundancy is not sufficient. The factor ϕ_s is calibrated to lead to consistent system reliability levels for all configurations.

The approach used to develop the system factor tables is similar to the approach used in Section 3.9. The system factor results developed in Section 3.8 are such that a system factor equal to 1.0 indicates that, for the ultimate limit state, the substructure configurations with adequate system redundancy produce an average reserve ratio R_u, equal to 1.20. The 1.20 system reserve ratio leads to a relative reliability index $\Delta\beta_u$ equal to 0.50 for wind loading. This means that the system capacity should be generally 20 percent higher than the capacity of the first member to fail leading to a system reliability index that is higher than the reliability index of the most critical (first to fail) column by 0.50.

The system factors were computed for a representative sample of substructure configurations taken from the survey of state DOTs. System factors for each bridge substructure configuration considered are given in Tables 3.7 through 3.10. These are adapted from Tables D-15 through D-20 of Appendix D.

For each configuration, three system factors are obtained for three system limit states (crushing of unconfined columns, crushing of confined columns, and the functionality limit state corresponding to a maximum lateral displacement equal to clear column height/50–H/50). The system factor that should be used is the minimum value that applies for the substructure under consideration. In addition, as recommended for the direct redundancy analysis check, it is recommended that the system factor be limited to a maximum value of 1.20 and a minimum value of 0.80. These maximum and minimum values are recommended in order to keep the changes in member capacities for new designs reasonably similar (within ±20 percent) to those obtained when using current specifications. These limits are set at this point until more experience

TABLE 3-7 System factors for unconfined 2-column piers

Foundation		Spread		Extension			Piles		
	Soil type	Normal	Stiff	Soft	Normal	Stiff	Soft	Normal	Stiff
Height	Width								
11m	1.2m	0.95	0.96	0.91	0.96	1.00	0.98	1.03	1.02
4m	1.2m	0.97	0.99	0.85	0.87	0.89	0.88	0.92	0.94
18m	1.2m	0.93	0.93	0.93	0.97	1.00	0.99	1.03	0.98
11m	0.8m	0.94	0.95	0.91	0.97	1.02	1.00	1.06	1.09
11m	1.6m	0.94	0.95	0.89	0.92	0.93	0.94	0.93	0.95

TABLE 3-8 System factors for unconfined 4-column piers

Foundation		Spread		Extension			Piles		
Soil type		Normal	Stiff	Soft	Normal	Stiff	Soft	Normal	Stiff
Height	Width								
6.5m	1.0m	1.02	1.00	0.91	0.98	1.02	0.94	1.03	1.07
3.5m	1.0m	0.98	1.01	0.88	0.90	0.95	0.88	0.94	1.01
9.5m	1.0m	1.00	0.98	0.95	1.00	1.03	0.98	1.04	1.07
6.5m	0.5m	0.97	0.95	0.91	0.94	0.96	1.01	0.96	0.93
6.5m	1.5m	0.99	1.01	0.89	0.95	0.99	0.95	1.02	1.04

is gained from using the system factors in actual practice. The 40 percent range has been chosen in this study to remain consistent with the 40 percent range used for superstructures (see *NCHRP Report 406*), which was chosen because it is on the same order of magnitude as the difference between inventory and operating rating stress levels.

The results in Tables 3.7 to 3.10 already account for the possibility of the formation of a collapse mechanism since the load that produces the mechanism was used whenever it occurs prior to the load that causes concrete crushing. System factors are not provided for the damaged scenario because the caps in typical substructure designs do not have the capacity to transfer the vertical live and dead loads to the remaining columns if an exterior column is damaged because of a collision, foundation failure, or general deterioration. Thus, bridges should be protected against such eventualities by checking the designs with the direct analysis procedure and giving special attention to the beam cap. The direct analysis procedure was described in Section 3.9.

The system factors given in Tables 3.7 through 3.10 are developed for two-column and four-column bents with evenly spaced columns of the same height and same capacity. Separate tables are provided for bents with two and four concrete columns of different heights and widths. The tables cover substructures whose columns are confined or unconfined with lateral reinforcements supported by different types of soil/foundation systems. For configurations that might be slightly different than those considered, it is necessary to extract the system factors from the tables by interpolation (or extrapolation) as described in the example provided in Section 3.10.6. For substructures that do not fit these categories, the direct analysis procedure outlined in Section 3.9 should be used.

The system factor tables are applicable for substructures that satisfy any acceptable AASHTO design criteria including the LFD, WSD, and LRFD criteria. The tables are applicable for bridge substructures whose design is dominated by the lateral load (i.e., for cases when the moment from the vertical loads constitute less than 25 percent of the moment capacity). For bridge columns where the moment from the vertical load is higher than 25 percent, the step-by-step analysis procedure described in Section 3.9 should be used.

3.10.1 System Factor Tables

The system factors are provided for different types of soil/foundation systems and different column heights and column widths. For each bent and soil condition, the foundations were designed to accommodate the geometrical and weight properties of the bents and the live loads. The results also account for the P-delta moments produced by the vertical loads.

The distribution of the loads to individual columns is a function of the foundation and column stiffnesses. In the instances where bents have more uniform distribution of moments in the elastic range, the bents will exhibit less system redundancy. The provided tables or a direct analysis makes it possible to predict how increases in column widths and heights would influence the redundancy of the substructure system and the system factor.

TABLE 3-9 System factors for confined 2-column piers

Foundation			Spread		Extension			Piles		
Soil type			Normal	Stiff	Soft	Normal	Stiff	Soft	Normal	Stiff
Height	Width	Limit state								
11m	1.2m	Ult,	1.00	1.00	0.98	1.13	1.25	1.21	1.12	1.03
		Func.	0.99	0.99	0.68	1.01	1.13	1.03	1.11	1.01
4m	1.2m	Ult,	1.25	1.26	0.89	0.96	1.05	1.02	1.16	1.05
		Func.	1.12	1.28	0.14	0.42	0.86	0.41	1.07	1.04
18m	1.2m	Ult,	0.94	0.94	0.97	1.13	1.15	1.13	1.04	0.98
		Func.	0.93	0.92	0.85	0.97	1.03	1.00	1.02	0.95
11m	0.8m	Ult,	0.94	0.95	0.90	1.11	1.20	1.19	1.16	1.11
		Func.	0.90	0.90	0.84	0.99	1.08	1.05	1.14	1.05
11m	1.6m	Ult,	1.18	1.15	0.99	1.12	1.15	1.00	0.95	1.00
		Func.	1.07	1.12	0.47	0.84	0.97	0.84	0.95	0.99

TABLE 3.10 System factors for confined 4-column piers

		Foundation	Spread		Extension			Piles		
		Soil type	Normal	Stiff	Soft	Normal	Stiff	Soft	Normal	Stiff
Height	Width	Limit state								
6.5m	1.0m	Ult,	1.07	1.04	1.02	1.17	1.32	1.12	1.35	1.27
		Func.	1.06	1.02	0.88	1.12	1.31	0.99	1.31	1.26
3.5m	1.0m	Ult,	1.15	1.14	0.97	1.06	1.19	1.01	1.19	1.38
		Func.	1.13	1.15	0.37	1.01	1.33	0.53	1.18	1.42
9.5m	1.0m	Ult,	1.02	0.99	1.04	1.22	1.31	1.17	1.28	1.16
		Func.	1.00	0.97	0.96	1.09	1.21	1.02	1.24	1.15
6.5m	0.5m	Ult,	0.97	0.95	0.84	1.03	1.11	1.02	0.96	0.93
		Func.	0.88	0.86	0.90	0.97	1.06	0.95	0.85	0.82
6.5m	1.5m	Ult,	1.26	1.23	1.03	1.21	1.37	1.26	1.27	1.13
		Func.	1.19	1.22	0.58	1.08	1.36	0.84	1.25	1.13

The base cases are for a typical column height of 11 m for the two-column bents and 6.5 m for the four-column bents. Average column widths are 1.2 × 1.2 m and 1.0 × 1.0 m, respectively, for the two-column and four-column bents. These values correspond to the most typical configurations encountered in existing substructures as reported in the survey of state DOTs and are used as the base cases for the analysis. Other cases provided in the tables are for 4-m and 18-m-high columns, and 0.8-m × 0.8-m and 1.6-m × 1.6-m wide columns for the two-column bents and 3.5-m and 9.5-m high columns and 0.5-m × 0.5-m and 1.5-m × 1.5-m-wide columns for the four-column bents. Interpolation should be used for other column sizes and heights.

The tables provided assume a typical steel reinforcement ratio of 2.3 percent for the two-column bents and 1.85 percent for the four-column bents (representing the average reinforcement ratios from typical existing bents). If this ratio is decreased by 1.2 percent (down to 1.1 percent for two-column bents and 0.65 percent for four-column bents), the system factors shown in the tables should be increased by 0.05. The increase is due to the increased level of ductility associated with the decrease in reinforcement ratio. If the reinforcement ratio is increased by 1.2 percent (up to 3.5 percent for two-column and 3.05 percent for four-column bents), then, the system factors shown should be decreased by 0.10. Use interpolation to find the appropriate change in the system factor for other reinforcement ratios.

The sensitivity analysis performed as part of this study has demonstrated that the concrete strength, f'_c, and the steel yielding stress, f_y, do not significantly affect the results. Thus, the tables are valid for columns designed for usual concrete strengths and steel grades.

3.10.2 Single-Column Bents and Pier Walls

In Section 2.7.1, it was shown that single-column bents are nonredundant, and the system reserve ratios are small: 1.02 for unconfined columns, and 1.04 for confined columns. As the target system reserve ratio was set at 1.20, the corresponding system factor ϕ_s can be approximately set as

$$\phi_s = \frac{R_u}{R_{u\ \text{target}}} = 0.85 \sim 0.87 \tag{3.42}$$

Therefore, it is recommended that for single-column bents and nonredundant pier walls, a system factor ϕ_s of 0.80 be used.

3.10.3 Two-Column and Four-Column Bents with Unconfined Concrete

Tables 3.7 and 3.8 give the system factors for two-column and four-column bents with unconfined concrete. For unconfined columns, the most critical column usually reaches its crushing strain before the P-delta moment becomes significant. In addition, the crushing strain is generally reached before the functionality limit state is reached. Tables 3.7 and 3.8 were derived assuming that the crushing occurs when the strain at any point of the concrete column reaches a value equal to 0.004.

3.10.4 Two-Column and Four-Column Bents with Confined Concrete

Tables 3.9 and 3.10 list the system factors for two-column and four-column bents with confined concrete. For confined columns, the most critical column often reaches its crushing strain after relatively high lateral displacements are produced. Thus, the P-delta moment may become significant.

In addition, the crushing strain is often reached after the serviceability limit state (total displacement equal to clear column H/50) is reached. Thus, both limit states should be checked and the lower system factor should be used. The two factors are kept separate to give the engineer the flexibility of choosing the most appropriate limit state for different situations and also to allow one to determine which limit state governs in case corrective actions need to be taken. For example, the engineer may choose to use a different foundation type to improve the system factor for the functionality limit state. It is also for these same reasons that the system factors provided in the tables are not pretruncated or rounded up to the proposed

limiting values of 0.80 and 1.20. For example, if the system factor for the ultimate limit state of 6.5-m-high and 1.0-m-wide column on extension piles in stiff soils is 1.32 and that for the functionality limit state is 1.31, the engineer should use the value of 1.20. If the system factor for the 11-m-high and 1.6-m-wide column on extension piles in soft soils is 1.03 for the ultimate limit state and 0.58 for the functionality limit state, take 0.8 as the final value to use in Equation 3.42.

Confined columns are defined as columns whose concrete crushes at a strain value equal to 0.015 (for unconfined concrete columns the strain at crushing is 0.004). Confined columns require heavy lateral reinforcement that is normally provided only when designing columns in earthquake-prone regions.

Tables 3.9 and 3.10 give two values of system factors for each column and foundation type. The top value is for the ultimate capacity limit state (concrete crushing or collapse mechanism) and the bottom value is for the functionality limit state. The lower of the two ϕ_s values must generally be used unless specific situations require the checking of only one of the limit states. In all cases, the lowest value that should be used should not be lower than 0.80. The highest value should not exceed 1.20. The results show that for short columns on soft soils, the functionality system factor is very low indicating large levels of lateral displacements rendering the bridge unfit for traffic crossing before crushing occurs.

3.10.5 General Comments

Tables 3.7 through 3.10 can be used during the routine design and evaluation of common type bridges by including the appropriate system factor ϕ_s in Equation 3.42. For bridges whose configurations are not covered in the tables, the design and/or evaluation engineer should use the direct analysis procedure outlined in Section 3.9.

3.10.6 Illustration of Interpolation Procedure

As an illustration, assume that the system factor for a four-column substructure with 8-m-high and 1.0 × 1.0-m-wide columns is required. The columns are supported on pile extensions in normal soils. The longitudinal reinforcement ratio is chosen at 2.5 percent of the column gross area.

Table 3.10 shows that for pile extensions on normal soils, the functionality limit state governs both the 6.5-m and the 9.5-m columns. The system factor for the 6.5-m column is 1.12, and the system factor for the 9.5-m column is 1.09. Thus, by interpolation the system factor for the 8-m column should be 1.105. But the tables are given for a longitudinal reinforcement ratio equal to 1.85 percent. Since the researchers are using a 2.5 percent ratio (a 0.65 percent increase in the reinforcement), then the system factor should be decreased. The decrease in the system factor should be equal to 0.10 for a 1.2 percent increase in the ratio. Thus, for a 0.65 percent increase in the ratio, the change in the system factor should be equal to 0.054. The system factor to be used becomes $1.105 - 0.054 = 1.05$. This system factor is less than 1.20 and greater than 0.80 and hence a final value $\phi_s = 1.05$ should be used in Equation 3.42. This system factor implies that one can reduce the safety factor of the columns of this substructure by 5 percent over current methods and still obtain a substructure system that would provide adequate levels of system safety.

TABLE 3.11 System factors for bridge substructures

	Foundation	Spread footing	Drilled shaft			Piles		
	Soil type	All soil types	Soft	Normal	Stiff	Soft	Normal	Stiff
Walls, abutments and one column bents in bending	ϕ_{su}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sc}							
	ϕ_{sf}							
2-column bents in bending	ϕ_{su}	0.95	0.85	0.90	0.95	0.90	0.95	1.00
	ϕ_{sc}	1.00	0.95	1.05	1.15	1.05	1.05	1.05
	ϕ_{sf}	1.00	0.80	0.85	1.00	0.95	0.95	0.95
multi-column bents in bending	ϕ_{su}	1.00	0.90	0.95	1.00	0.95	1.00	1.05
	ϕ_{sc}	1.05	1.00	1.15	1.20	1.10	1.10	1.10
	ϕ_{sf}	1.05	0.80	1.05	1.20	1.05	1.05	1.05
All members in shear	ϕ_{su}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sc}							
	ϕ_{sf}							
Bar pullout and other connection failures	ϕ_{su}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sc}							
	ϕ_{sf}							

3.10.7 Failure of Joints and Columns in Shear

The analyses performed in Chapter 2 concentrated on the nonlinear behavior of two-and four-column bents under lateral load. The models assume that bents will be able to continue to carry load after a member reaches its ultimate moment capacity. However because shearing failures and failures of joints (including bar pullout) are brittle, the substructure will generally unload when such failures occur. Thus, all systems are considered to be nonredundant for column shearing failures and joint failures, and a maximum penalty (low system factor ϕ_s) is recommended. The system factor $\phi_s = 0.80$ is recommended for shearing failures and joint failures of all bents to match the 0.80 factor recommended for single-column bents in bending.

3.10.8 Geotechnical Failure

Although the model used in Chapter 2 accounted for soil and foundation flexibility, this study did not consider failure of the soils or foundation. Thus, the system factors recommended are only applicable to the structural components of the substructures (i.e., the piers, walls, and abutments).

3.10.9 Simplified System Factor Table for Bridge Specifications

Tables 3.7 through 3.10 present a summary of the system factors calculated for all the substructures studied in Chapter 2. This set of factors gives an accurate quantification of substructure redundancy for the typical configurations analyzed in this project. A simplified set that is applicable for use in bridge specifications (e.g., the AASHTO LRFD) is given in Table 3.11. The system factors provided in Table 3.11 are obtained by averaging the system factors calculated for different column heights and widths. The objective is to provide a simplified method for considering substructure redundancy on a routine basis during the design and safety evaluation of bridges. A set of specifications compatible with AASHTO LRFD Specifications is provided in Appendix A of this report.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

This report presented the analytical formulation, modeling assumptions, reliability analysis, and calibration of system redundancy factors for implementation in codified design practice. Extensive parametric studies were performed to cover the possible variability of structural parameters and foundation types and soil conditions that may be encountered in the design and evaluation of bridge substructures.

4.1 CONCLUSIONS

The objective of this investigation was to develop a rational basis for considering substructure redundancy during the design and evaluation of highway bridge substructures, and to develop the necessary data for possible implementation into the AASHTO LRFD Bridge Design Specifications. These project objectives have been accomplished by (1) developing the analytical procedure to quantitatively determine the redundancy of bridge substructures and (2) providing a set of system redundancy factors applicable for typical substructure configurations. These system factors were calibrated using a rational approach that is consistent with the method used to calibrate the load and resistance factors of the AASHTO LRFD Specifications so that the reliability formulation is transparent to the designers. The system factors are also applicable to any other standards and specifications. In addition to the system factors provided for typical substructure configurations, a direct analysis approach was proposed to evaluate substructure redundancy for unusual structures or circumstances.

To improve the safety of a *nonredundant substructure*, two approaches can be followed: (1) modify the design, for example, by adding more columns or using a different foundation type to produce a more redundant structural system, and (2) alternatively, use the same structural configuration but design the members to higher capacity levels. This second approach will not make a *nonredundant* structure *redundant*; however, by requiring higher component design capacity, the overall system safety for the nonredundant structure is enhanced. Therefore, the proposed system redundancy factors proposed in this study are, in essence, penalty-reward factors. A system factor less than 1.0 results in higher capacity levels for component designs thus penalizing a nonredundant structural system. On the other hand, a redundant system will be rewarded by allowing less conservative component design that is justified because of the presence of high levels of system reserve.

The system factors were developed for two-column and four-column bents, representing the behavior of typical multi-column bents. As a first step in the implementation of system factors in design practice and until more experience is gained with their use, it is proposed to limit the range of the system factors (ϕ_s) for all parameter variations and foundation/soil conditions to a minimum value of 0.8 and a maximum value equal to 1.20.

Single-column bents are considered to be nonredundant because their system reserve ratios ($R_u = 1.02$) are less than the target system reserve of 1.20 for both confined and unconfined concrete. Therefore, the lower limit of system factor ($\phi_s = 0.8$) is recommended. Most pier walls can be considered to behave as single-column bents. For the cases where pier walls are designed in such a way as to provide some level of system reserve, the direct analysis approach can be used to determine the system factors.

Shear failures are brittle. Thus, when a substructure system failure is governed by shear, it is considered to be nonredundant and a ϕ_s of 0.8 is recommended.

Similarly, joint failures including bar pullout are considered brittle and substructure failures initiated by joint failures have no reserve strength and a ϕ_s equal to 0.8 is recommended. The bridge substructures analyzed in this study were connected to the superstructures through bearing supports. If integral connections are provided the system redundancy is expected to vastly improve.

This study did not consider the possibility of soil failures, although the effect of foundation/soil flexibility on the behavior of substructure systems has been included. In fact, the magnitude of soil/foundation flexibility relative to the structural flexibility was found to be the most important factor that affects the *redundancy of substructure systems*. This relative flexibility is related to the combined effects of the soil/foundation behavior, bent height, and column dimension (width) as described in Chapter 2. System redundancy is also affected by member ductility, which is controlled by lateral confinement, longitudinal reinforcement, and the concrete crushing strain of the concrete columns.

The proposed system redundancy factors were calibrated using reliability methods to quantify the additional safety margin provided by the system beyond first component failure.

Based on past experience and engineering judgment, four-column bents with unconfined concrete, which are typical for most design environments, represent an *adequately redundant* structural system. Therefore, the redundancy level provided by four-column substructures is used as the target that all bridge substructures should meet. Accounting for the uncertainties associated with determining substructure member and system capacities and expected design life loads, a target relative reliability index ($\Delta\beta$) of 0.5 was established as the criterion for the calibration of system factors subjected to wind loading. This $\Delta\beta = 0.5$ value corresponds to a system reserve ratio (R_u) of 1.20. Although $\Delta\beta$ is set based on lateral load due to winds, the final recommendation of $R_u = 1.20$ and the corresponding system factors are applicable for all load types.

Unlike the current load factor modifier in Equation 1.3.2.1-1 of the AASHTO LRFD Specifications, the proposed system factor consists of one term only tabulated for different substructure configurations. Differences in member *ductility* levels are considered by using different tabulated factors for confined and unconfined column systems. Instead of explicitly using *operational importance* factors, different limit states resulting in different system factors are considered giving the engineer and the bridge owner the option of choosing the appropriate limit state depending on the characteristics and the location of the bridge as well as its operational importance. This approach is consistent with current trends to develop *performance-based design methods* in bridge engineering.

For substructures not covered by the tabulated system factors, a step-by-step procedure should be used for the direct evaluation of substructure redundancy. This involves the use of a nonlinear analysis program such as the PIERPUSH program used in this investigation to conduct nonlinear analysis under incrementally applied lateral load and to monitor the development of nonlinear behavior in the structure. Several limit states are defined and must be checked. These include global system collapse mechanism, local component rupture, and large displacement due to both structural drift and foundation deformation. The column P-Δ effect under axial load must be taken into account during the analysis process, particularly for confined members. Once the lateral load levels producing the pertinent limit states are determined, the system reserve ratio and the system redundancy factor can be established. If it is determined that the system is nonredundant, the system factor can be used to determine the level of strengthening required. In applying the direct analysis procedure to the evaluation of existing bridges, it is important to account for the highly possible over-design that exists. Recognizing that foundation flexibility significantly affects system reserve, it is important not to ignore this effect. (In traditional bridge design, foundations are frequently considered as fixed.).

Unlike the superstructure redundancy, most bridge substructures subjected to damage to one column are not redundant. This is because the cap member is typically not designed sufficiently strong to transfer the loads from the damaged column to the surviving columns. For individual cases of damaged conditions, the direct analysis procedure can be applied to examine the remaining load-carrying capacity and the robustness of such systems or their ability to carry some load until the damage is detected and repairs are performed. The analysis of damaged scenarios is recommended for bridges that may be classified as critical and are vulnerable to collisions from ships, vehicles, debris carried by flooded streams, and so on. A system redundancy ratio $R_d = 0.5$ is recommended for damaged scenarios of critical bridges. This means that a damaged substructure of a critical bridge should be able to carry more than 50 percent of the load that would normally cause the first member of an intact structure to reach its limiting capacity.

4.2 FUTURE RESEARCH

A theoretical framework and analytical procedure were developed to evaluate the redundancy of bridge substructures. In this investigation, the focus of the substructure failure analysis was on the behavior of the columns. As described in Chapter 2, it was assumed that cap beams and, more importantly, the beam-column joints and column-footing connections are stronger than the members and are assumed to remain elastic. Despite the AASHTO LRFD Specifications joint design requirements, joints may not be designed to resist significant lateral loads, particularly for regions outside earthquake prone areas. To a lesser extent, the behavior of the cap beam may not be sufficiently strong even for the intact condition. Joints were found to be vulnerable to cyclic lateral loads such as those observed during an earthquake. Under other monotonic, nonseismic lateral loads, the vulnerability of the joints is less obvious. Because there are many existing bridges nationwide that don’t have good reinforcement details in the joint region, the cost to upgrade all these structures can be extremely high. (In earthquake-prone areas, this is a minimum requirement and is usually the first retrofit recommendation.) Hence, it would be advantageous to evaluate performance of typical joint details under *nonseismic lateral loads* and assess their impact on bridge substructure redundancy.

The analysis of substructure pier wall is another subject that requires further examination. Recent research has indicated that the strength and ductility of bearing walls under combined vertical and lateral loads may not be as brittle as previously thought. Thus, a more reliable analytical model based on recent findings should be developed to investigate both cantilever and squat walls. The model must be accurate and yet simple enough for practical usage. Guidelines describing how to classify and evaluate the redundancy of various types of pier walls should be developed based on the results of the analytical model.

The analysis of concrete crushing has used two values for the maximum strain that concrete can withstand: 0.040 for

unconfined concrete and 0.15 for confined concrete. In reality, the maximum concrete strain capacity is a function of many parameters including concrete strength and confinement ratio. Therefore, the maximum values used in this study should be re-examined to better account for the actual level of confinement in future revisions of the recommended system factors.

Possible failure of footings, piles, and pile extensions below the pile caps were not explicitly included in the analytical models used in this study. Only the overall flexibility of the foundation was included through the use of appropriate foundation stiffness coefficients. In many cases, the pile group is composed of vertical piles and battered piles. Under lateral loads, the battered piles carry a significantly high proportion of the total applied lateral load, and are vulnerable to structural failure. For steel H-piles, structural damage associated with local flange buckling and global member buckling must be carefully considered in the analytical model. The redundancy of such pile group foundations should be examined. Any upgrading of these foundations can be very expensive, as was found out during recent seismic retrofit of seven major toll bridges in California (Liu and Neuenhoffer, 1998.) It would be worthwhile to further refine the analytical model to account for such situations.

The models used throughout the course of the study assumed that the soil/foundation system remain linear throughout the loading process and no geotechnical or soil failures would occur. Although this is generally accepted for design purposes, the application of high levels of lateral loading at or near the ultimate structural capacity of the structural system may cause the soil to exhibit nonlinear behavior and even to fail. These effects must be included in future studies on substructure redundancy to further refine the proposed system factors.

The analyses performed in this study focused on the bridge substructure ignoring the contributions of the superstructure to the redundancy of the substructure. This approach was deemed reasonable for the cases when the superstructure is connected to the substructure through bearing-type supports. This observation should be further verified by extensive studies on complete bridge systems. Furthermore, the use of integral-type connections between the two subsystems, as is common on the West Coast, may produce a strong interaction between substructures and superstructures that would require special consideration.

Finally, the system factor tables should be expanded to account for major bridges with steel-braced frame substructures and the applicability of the direct analysis procedure for these substructure types must be verified. In the survey of DOTs, these substructures were not considered important. However, in major metropolitan areas, there are many river crossings founded on these types of systems. These structures are typically very important for operational reasons. A standard procedure should be developed for these systems.

BIBLIOGRAPHY

AASHTO, *Standard Specifications for Highway Bridges*, American Association of Highway and Transportation Officials, Washington, DC (1996).

AASHTO, *AASHTO Load and Resistance Factor Bridge Design Specifications*, American Association of Highway and Transportation Officials, Washington, DC (1994).

ATC-32, "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations," Applied Technology Council, Redwood City, CA (1996).

Agusti, G., Baratta, A., and Casciati, F., *Probabilistic Methods in Structural Engineering*, Chapman Hall, New York, NY (1984).

Becker, D.E., 18th Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part 1. An Overview of the Foundation Design Process, *Can. Geotech*. *Journal*, 33: 956-983 (1996a).

Becker, D.E., 18th Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part II. Development for the National Building Code of Canada, *Can. Geotech. Journal,* 33: 984-1007 (1996b).

Ellingwood B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A., "Development of a Probability Based Load criterion for American National Standard A58," National Bureau of Standards, Washington, DC (1980).

Ghosn, M. and Moses, F., *NCHRP Report 406*, "Redundancy in Highway Bridge Superstructures," Transportation Research Board, Washington, DC (1998).

Ghosn, M., Moses F., and Khedekar, N., "Response Functions and System Reliability of Bridges," IUTAM Symposium, Probabilistic Structural Mechanics: Advances in Structural Reliability Methods, Ed. Spanos, P.D. and Wu Y.T., Springer-Verlag, Heidelberg (1994) pp. 220-236.

Imbsen, R.A. and Penzien, J., *Evaluation of Energy Absorption Characteristics of Bridges under Seismic Conditions*, Earthquake Engineering Research Center, Report No. 84/17, University of California, Berkeley (1984).

Lam, P. and Martin, G., *Design of Highway Bridge Foundations to Resist Earthquake Loads*, FHWA Report RD86/101 (1986).

Liu, W. D., Chang, K.K., Chen, X., Dhillion, S.D., and Imbsen, R.A., "Nonlinear Seismic Soil-Pile Interaction of Major Bridges," *Proceedings 6th U.S. National Conference on Earthquake Engineering*, Seattle, WA (May 1998).

Liu, W. D., Chen, X., Chang, K.K., and Imbsen, R.A., "Performance-Based Seismic Evaluation of Bridge Foundations," ASCE Specialty Conference on Geotechnical Earthquake Engineering and Soil Dynamics, Seattle, WA (August 1998).

Liu, W. D. and Neuenhoffer, A., *Seismic Safety Margin Evaluation of As-Built and Retrofitted Piers*, Richmond-San Rafael Bridge Seismic Retrofit, Caltrans Contract No. 59X475, Report by Imbsen & Associates, Inc. (February 1998c).

Liu, W. D., Chang, K.K., and Imbsen, R.A., "Nonlinear Seismic Evaluation for Retrofit Design of Major Bridges Including Soil-Foundation-Structure-Interaction (SFSI) Effects," *Proceedings 2nd U.S. National Seismic Conference on Bridges and Highways*, Sacramento, CA (July 1997a).

Liu, W. D., Nobari, F.S., Schamber, R.A., and Imbsen, R.A., "Performance-Based Seismic Retrofit Design of Benicia-Martinez Bridges," *Proceedings 2nd U.S. National Seismic Conference on Bridges and Highways*, Sacramento, CA (July 1997b).

Liu, W. D., Ricles, J. M., Imbsen, R.A., Priestly, M.J.N., Seible, F., Nobari, F.S., and Yang, R., "Response of a Major Freeway Bridges During the Wittier Narrow Earthquake," *Proceedings 4th U.S. National Conference on Earthquake Engineering*, Palm Spring, CA (May 1990) pp. 997.

Nowak, A.S., "Calibration of LRFD Bridge Design Code", NCHRP Project 12-33 Final Report (December 1994).

Penzien, J., Imbsen, R. A., and Liu, W. D., *NEABS—Nonlinear Earthquake Analysis of Bridge Systems*, Earthquake Engineering Research Center, University of California, Berkeley (1981).

Throft-Christensen, P. and Baker, M. J., *Structural Reliability Theory and Its Applications*, Springer-Verlag, New York, NY (1982).

Tseng, W. S. and Penzien, J., *Analytical Investigations of the Seismic Response of Long Multiple-Span Highway Bridges*, Earthquake Engineering Research Center, Report No. 73-12, University of California, Berkeley (1993).

Wallace, J.W., *BIAX—Revision 1, A Computer Program for the Analysis of Reinforced Concrete and Reinforced Masonry Sections*, Report No. CU/CEE-92/4, Department of Civil Engineering, Clarkson University, Potsdam, NY (February 1992).

APPENDIX A

PROPOSED SPECIFICATIONS OUTLINE

A.1 Version I

A.2 Version II

Appendix A

PROPOSED SPECIFICATIONS OUTLINE

In developing the Specification outline, it was found difficult to produce the specifications without causing confusion. This was pointed out in the panel's comment as well as our review by Mr. Robert C. Cassano. For the purpose of this study, two alternative versions are developed:

Version I: The recommended revisions are limited to Chapter 11 of the AASHTO LRFD Specifications. This is consistent with the original scope. However, there exist obvious confusions.

Version II: An alternative version is provided. The recommended modifications are made to Article 1.3 of the AASHTO Specifications.

Appendix A.1

SPECIFICATIONS—Version I

The attached document provides recommended modifications to article 11.5.3 in Section 11 "Abutments, Piers and Walls" of the AASHTO LRFD Bridge Design Specifications. The modifications provide a method to include system factors that account for substructure ductility and redundancy during the design and safety evaluation of the structural components of a substructure system.

11.5.3 Strength Limit State

11.5.3.1 Geotechnical Design

Design of abutments, piers and walls shall be investigated at the strength limit state using Equation 1.3.2.1-1 for:

- bearing resistance failure
- lateral sliding
- excessive loss of base contact
- overall instability
- pull out failure of anchors or soil reinforcement

Resistance requirements shall satisfy article 11.5.4.

11.5.3.2 Structural Design

Design of abutments, piers and walls shall be investigated for structural safety at the strength limit state and extreme event limit state for each component and connection using Equation 11.5.3-1:

$$\phi_s \phi R_n = \sum \gamma_i Q_i \tag{11.5.3-1}$$

where:

ϕ_s = system factor relating to ductility and redundancy
ϕ = resistance factor
R_n = nominal resistance
γ_i = load factor
Q_i = force effect

The nominal resistance R_n and the resistance factor ϕ shall comply with the provisions of Sections 5, 6, 7 and 8. The system factor ϕ_s shall comply with the provisions of Article 11.5.3.2.1.

11.5.3.2.1 System Factor ϕ_s

The system factor is a multiplier applied to the nominal resistances of the structural components of a bridge's substructure to reflect the level of ductility and redundancy.

For bridges classified to be critical or susceptible to damage:
ϕ_s shall be calculated using the direct analysis approach using the provisions of Article 11.5.3.2.2

For bridges classified to be essential:

$\phi_s = \min(\phi_{sc}, \phi_{sf})$ for substructures with confined members

$\phi_s = \min(\phi_{su}, \phi_{sf})$ for substructures with unconfined members

For all other bridges:

$\phi_s = \phi_{sc}$ for substructures with confined members

$\phi_s = \phi_{su}$ for substructures with unconfined members

where:

ϕ_{su} = system factor for ultimate capacity of substructures with unconfined members

ϕ_{sc} = system factor for ultimate capacity of substructures with confined members

ϕ_{sf} = system factor for bridge substructure functionality

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

Confined concrete columns shall satisfy the provision of Article 5.10.11.4.1.d. Columns that do not satisfy Article 5.10.11.4.1.d shall be considered unconfined.

Recommended values for ϕ_{su}, ϕ_{sc} and ϕ_{sf} for piers, walls and abutments founded on spread footings, drilled shafts or piles are specified in Table 11.5.3.2.1-1 for soft, normal and stiff soils. For bridges with nontypical substructure configurations, the direct analysis approach of Article 1.5.3.2.2 shall be used.

Bridges susceptible to damage include those with members that are exposed to collisions from ships, vehicles and debris carried by swelling streams and rivers.

Table 11.5.3.2.1-1 – System factors for bridge substructures

	Foundation	Spread footing	Drilled shaft			Piles		
	Soil type	All soil types	Soft	Normal	Stiff	Soft	Normal	Stiff
Walls, abutments and one column bents in bending	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							
2-column bents in bending	ϕ_{su}	0.95	0.85	0.90	0.95	0.90	0.95	1.00
	ϕ_{sc}	1.00	0.95	1.05	1.15	1.05	1.05	1.05
	ϕ_{sf}	1.00	0.80	0.85	1.00	0.95	0.95	0.95
multi-column bents in bending	ϕ_{su}	1.00	0.90	0.95	1.00	0.95	1.00	1.05
	ϕ_{sc}	1.05	1.00	1.15	1.20	1.10	1.10	1.10
	ϕ_{sf}	1.05	0.80	1.05	1.20	1.05	1.05	1.05
All members in shear	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							
Bar pullout and other connection failures	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							

11.5.3.2.2 Direct Redundancy Analysis

For bridges classified to be critical and for bridges not covered in Table 11.5.3.2.1-1, the system factor of Article 11.5.3.1 shall be calculated from the results of a nonlinear pushover analysis using equation 11.5.3.2.2.1.

$$\phi_s = \min\left(\frac{R_u}{1.20}, \frac{R_f}{1.20}, \frac{R_d}{0.50}\right) \qquad 11.5.3.2.2.1$$

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

R_u is the system reserve ratio for the ultimate limit state, $R_u = \frac{LF_u}{LF_1}$

R_f is the system reserve ratio for the functionality limit state, $R_f = \frac{LF_f}{LF_1}$

R_d is the system reserve ratio for the damage condition, $R_d = \frac{LF_d}{LF_1}$

Where:

LF_u = the lateral load factor that causes the failure of the substructure
LF_f = the lateral load factor that causes the total lateral deflection of the substructure to reach a value equal to average clear column height/50
LF_d = the lateral load factor that causes the failure of a damaged substructure
LF_1 = the lateral load factor that causes the first member of the intact substructure to reach its limit capacity

COMMENTARY

The attached document provides commentary on the recommended modifications for article 11.5.3 in Section 11 "Abutments, Piers and Walls" of the AASHTO LRFD Bridge Design Specifications. The commentary provides information on the background and applicability of the proposed revisions.

C 11.5.3.2 Structural Design

The interaction of structural members of a bridge substructure should be considered in assessing the behavior of the substructure system. Bridge substructure redundancy is the capability of a substructure system to carry loads after damage to or the failure of one or more of its members. For substructures, the most common failures are the result of lateral overloads from earthquakes, wind, ice, stream flow or accidental collisions of vehicles or vessels. Consequently this provision focuses on lateral overload from any of the above sources.

System factors are used in this article to maintain an adequate level of substructure system safety. Less redundant systems are penalized by requiring their structural members to provide higher safety levels than those of similar substructures with redundant configurations. The aim of ϕ_s is to add member and system capacity to less redundant systems such that the overall system reliability is increased. When adequate redundancy is present, a system factor, ϕ_s, greater than 1.0 may be used.

Two methods are provided to determine appropriate values for ϕ_s:

a) Tables of ϕ_s provided for common substructure configurations (Table 11.5.3.2.1-1).
b) A direct analysis approach is recommended for substructures with nontypical configurations and members (Article 1.5.3.2.2).

C 11.5.3.2.1 System Factor, ϕ_s

The use of more stringent criteria for critical and essential bridges is consistent with current trends to use performance based design in bridge engineering practice. The classification of a bridge should be based on social/survival and/or security/defense requirements. The commentary of Article 3.10.3 provides some guidance on selecting importance categories as they relate to design for earthquakes. This information can be generalized for other situations.

The system factors provided in Table 11.5.3.2.1-1 are calibrated to satisfy Equation C 11.5.3.2.1-1 for a set of typical bridge substructure configurations. This set includes bridges with concrete columns varying in height between 3.5m to 18m and a vertical rebar reinforcement ratio of 1.85 to 2.3%. The nonlinear pushover analysis should be used for substructures with other configurations.

Soft soils are defined as soils that produce a blow count N=5. Normal soils are those with N=15. Stiff soils are those with N=30 or higher. SPT= Standard Penetration Test blow count (number of blows per one foot=0.035 m penetration into the soil). Use the nearest tabulated SPT for values of N not provided.

Members in shear as well as all joints and connections are assigned a system factor ϕ_s=0.80. This assumes that the resistance factor ϕ was calibrated to satisfy a target member reliability index β_{member}=3.5. Since shear failures and connection failures are brittle causing the failure of the complete system, the application of a system factor ϕ_s=0.80 will increase the reliability index of the member and also that of the system so that $\beta_{member}=\beta_{system} \approx 4.5$.

C 11.5.3.2.2 Direct Redundancy Analysis

The nonlinear pushover analysis of critical bridges shall be performed as described in NCHRP report 12/47 and NCHRP report 406 accounting for the nonlinear behavior of the structural elements of the substructure and considering soil/foundation flexibility. The analysis requires the availability of a nonlinear analysis program that provides a lateral load versus lateral deflection curve and that adequately models the nonlinear behavior of the substructure components up to crushing of concrete, rupture of steel, or the formation of a collapse mechanism. The program should also be able to model the stiffness and the soil/foundation system by either the use of equivalent springs or actual modeling of the nonlinear foundation. Programs such as FLPIER or PIERPUSH can be used for such purpose.

The ratio of the lateral force causing the failure of the bent system to the force causing the failure of one structural member is defined as the system reserve ratio for the ultimate capacity R_u.

The ratio of the lateral force causing a lateral deflection equal to clear column height/50 to the force causing the failure of one member is defined as the system reserve ratio for the functionality limit R_f.

The ratio of the lateral force causing the failure of a damaged bridge substructure to the force causing the failure of one member of the undamaged bridge is defined as the system reserve ratio for the damaged condition R_d.

Possible damage scenarios include the loss of a single column or a connection and should be considered in consultation with the bridge owner.

The nonlinear pushover analysis is effected by applying the factored loads (lateral and vertical live and dead loads) on a structural model of the substructure and incrementing the lateral loads until the failure of the first member. The factor by which the original lateral load is multiplied to cause the failure of the first member is defined as LF_1. The nonlinear analysis is then continued beyond the failure of the first member until the lateral displacement at the top of the bent reaches a value equal to average clear column height/50. This displacement limit is defined as the functionality limit state. The factor by which the original load is multiplied to reach this functionality limit is defined as LF_f. The analysis is further continued beyond this point until one member reaches its maximum strain and concrete crushing ensues, a steel bar ruptures, or until a hinge collapse mechanism occurs. Unconfined concrete members crush at a strain of 0.003. Confined members crush at a strain of 0.015. Steel bars in tension rupture at a percent elongation ductility of about 20%. These cases define the ultimate capacity limit state. The load factor by which the original lateral load is multiplied to reach the ultimate capacity is defined as LF_u.

When a bridge is classified as critical or susceptible to brittle damage, the same process outlined above is repeated for a model of the damaged bridge. The damage scenario must be realistic and must be chosen in consultation with the bridge owner. Damage scenarios may include the complete loss of a column that may be subjected to risk of brittle failure from collisions by ships, vehicles, or flooding debris, etc. The analysis of the damaged bridge is effected in the same manner outlined above for the intact structure. But only the ultimate capacity limit state of the damaged bridge needs to be checked. The load factor by which the original lateral load is multiplied to reach the ultimate capacity of the damaged structure is defined as LF_d.

Bridge substructures that produce redundancy ratios R_u=1.2, R_f=1.2 and R_d=0.5 or higher are classified as adequately redundant. Those that do not satisfy these criteria will have system factors ϕ_s less than 1.0 and require higher component safety levels. If bridge redundancy is sufficiently high a system factor greater than 1.0 may be used.

Satisfying the criteria for R_u, R_f and R_d is recommended for bridges classified to be critical. Satisfying the criteria for only R_u and R_f is recommended for essential bridges. Satisfying the criteria for only R_u is sufficient for all other bridges.

The check of R_u verifies that a bridge's ultimate system capacity is at least 20% higher than the load level that will cause the failure of one member.

The check of R_f verifies that a bridge's lateral deflection during the application of high loads is still acceptable allowing the bridge to remain functional for emergency situations.

The check of R_d verifies that a damaged bridge is still capable of carrying 50% of the load that a nondamaged bridge can carry before one member fails.

Appendix A.2

SPECIFICATIONS—Version II

*The attached document provides recommended modifications to **Article 1.3** in Section 1 "Introduction" of the AASHTO LRFD Bridge Design Specifications. The modifications provide a method to include system factors that account for system ductility and redundancy during the design and safety evaluation of highway bridges.*

1.3 DESIGN PHILOSOPHY

1.3.1 General

Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety and serviceability, with due regard to issues of inspectability, economy and aesthetics, as specified in Article 2.5.

Regardless of the type of analysis used, Equation 1.3.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

1.3.2 **Limit States**

1.3.2.1 General

Each component and connection shall satisfy Equation 1.3.2.1-1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0. All limit states shall be considered of equal importance.

$$\phi_s \phi R_n = \phi_s R_r \geq \sum \gamma_i Q_i \qquad (1.3.2.1\text{-}1)$$

where:

ϕ_s = system factor relating to ductility and redundancy as specified in Article 1.3.4 for the design of structural components for strength and extreme event limit states.

For all other limit states, the system factors shall be taken as 1.0.

ϕ = resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections, 5, 6, 7, 8, 10, 11 and 12
R_n = nominal resistance
R_r = factored resistance: ϕR_n
ϕ_i = load factor: a statistically based multiplier applied to force effects as specified in Article 3.4
Q_i = force effect as specified in Section 3

1.3.2.2 Service Limit State

The service limit state shall be taken as restrictions on stress, deformation and crack width under regular service conditions.

For service limit states, system factors and resistance factors shall be taken as 1.0.

1.3.2.3 Fatigue and Fracture Limit State

The fatigue limit state shall be taken as a set of restrictions on stress range due to a single fatigue truck occurring at the number of expected stress range cycles. The fracture limit state shall be taken as a set of material toughness requirements of the AASHTO Material Specifications.

For fatigue and fracture limit states, system factors shall be taken as 1.0.

1.3.2.4 Strength Limit State

Strength limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

For the design of structural components at the strength limit state, system factors shall be taken as specified in Article 1.3.4.

1.3.2.5 Extreme Event Limit States

The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle or ice flow possibly under scoured conditions.

For the design of structural components at the extreme event limit states, system factors shall be taken as specified in Article 1.3.4.

1.3.3 Importance Categories

The owner or those having jurisdiction shall classify bridges into one of three importance categories as follows:

- Critical bridges,
- Essential bridges, or
- Other bridges

The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, consideration should be given to possible future changes in conditions and requirements and the consequences of bridge collapse.

1.3.4 System Factor, ϕ_s

The structural system of a bridge shall be configured, proportioned and detailed to: (a) ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states prior to failure; and (b) the redistribution of load in the event of the brittle failure of a member. This implies the presence of sufficient member ductility and system redundancy through multiple-load-paths.

The system factor, ϕ_s, is a multiplier applied to the nominal resistances of the structural components of a bridge system or subsystem to reflect the level of ductility and redundancy.

For bridge superstructures, ϕ_s, shall be taken as specified in Article 1.3.4.1.

For bridge substructures, ϕ_s, shall be taken as specified in Article 1.3.4.2.

1.3.4.1 System Factors for Bridge Superstructures

Design of components, connections and joints of bridge superstructures shall be investigated at the strength and extreme event limit states using Equation 1.3.2.1-1.

The nominal resistance R_n and the resistance factor ϕ shall comply with the provisions of Sections 5, 6, 7 and 8. The loads and load combinations shall comply with the provisions of Section 3.

For the superstructures of bridges classified to be critical or that are susceptible to brittle damage:

ϕ_s shall be calculated using the direct analysis approach following the provisions of Article 1.3.4.1.1 for trusses and arch bridges

ϕ_s =min (ϕ_{su} ,ϕ_{sd1}) for multigirder systems with diaphragms spaced at not more than 7600 mm

ϕ_s =min (ϕ_{su} ,ϕ_{sd2}) for all other multigirder systems

For the superstructures of all other bridges:

$\phi_s = \phi_{su}$

where:

ϕ_{su} = system factor for superstructure ultimate capacity
ϕ_{sd1} = system factor for damage of superstructures with regularly spaced diaphragms
ϕ_{sd2} = system factor for damage of superstructures with no diaphragms

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

Recommended values for ϕ_{su}, ϕ_{sd1} and ϕ_{sd2} for typical superstructures are specified in Table 1.3.4.1-1 for different number of beams and beam spacing. For bridges with nontypical configurations, the direct analysis approach of Article 1.3.4.1.1 shall be used.

Bridges susceptible to brittle damage include bridges with fatigue-prone details, and those with members that are exposed to collisions from ships, vehicles, and debris carried by swelling streams and rivers.

Table 1.3.4.1-1 System factors for superstructures

Loading type	System/member type	ϕ_{su}	ϕ_{sd1}	ϕ_{sd2}
Bending of members of multi-girder systems	Two-girder bridges	0.85	0.80	0.80
	Three-girder bridges with spacing ≤1.8m	0.85	1.20	1.10
	Four-girder bridges with spacing ≤1.2m	0.85	1.20	1.10
	Other girder bridges with spacing ≤1.2m	1.00	1.20	1.15
	All girder bridges with spacing ≤1.8m	1.00	1.20	1.15
	All girder bridges with spacing ≤2.4m	1.00	1.15	1.05
	All girder bridges with spacing ≤3.0m	0.95	1.00	0.90
	All girder bridges with spacing ≤3.6m	0.90	0.80	0.80
	All girder bridges with spacing >3.6m	0.85	0.80	0.80
Slab bridges	For bending	1.00		
Truss/arch bridges	Members under axial forces	0.80		
All bridge types	Members in shear	0.80		
All joints & connections		0.80		

1.3.4.1.1 Direct Redundancy Analysis for Bridge Superstructures

For bridges classified to be critical, and for bridges not covered in Table 1.3.4.1-1, the system factor of Equation 1.3.2.1-1 for the structural components of a superstructure shall be calculated from the results of an incremental analysis using Equation 1.3.4.1.1-1:

$$\phi_s = \min\left(\frac{R_u}{1.30}, \frac{R_f}{1.10}, \frac{R_d}{0.50}\right) \qquad 1.3.4.1.1\text{-}1$$

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

Where:

R_u = system reserve ratio for the ultimate limit state, $R_u = \frac{LF_u}{LF_1}$

R_f = system reserve ratio for the functionality limit state, $R_f = \frac{LF_f}{LF_1}$

R_d = system reserve ratio for the damage condition, $R_d = \frac{LF_d}{LF_1}$

LF_u, LF_f, LF_d and LF_1 are obtained from the incremental analysis

where:

LF_u = the vertical load factor that causes the failure of the superstructure
LF_f = the vertical load factor that causes the maximum vertical deflection of the superstructure to reach a value equal to span length/100.
LF_d = the vertical load factor that causes the failure of a damaged superstructure
LF_1 = the vertical load factor that causes the first member of the intact superstructure to reach its limit capacity

1.3.4.2 Bridge Substructures

Geotechnical Design

Design of abutments, piers and walls shall be investigated at the strength and extreme event limit states using Equation 1.3.2.1-1 for:

- bearing resistance failure
- lateral sliding
- excessive loss of base contact
- overall instability
- pull out failure of anchors or soil reinforcement

The nominal resistance R_n and the resistance factor ϕ shall comply with the provisions of Sections 5, 6, 7, 8, 10 and 11. The system factor ϕ_s shall be taken as 1.0.

Structural Components

Design of abutments, piers and walls shall be investigated for structural safety at the strength and extreme event limit states for each structural component and joint using Equation 1.3.2.1-1.

The nominal resistance R_n and the resistance factor ϕ shall comply with the provisions of Sections 5, 6, 7 and 8. The loads and load combinations shall comply with the provisions of Section 3.

For the substructures of bridges classified to be critical or susceptible to brittle damage:

ϕ_s shall be calculated using the direct analysis approach following the provisions of Article 1.3.4.2.1

For the substructures of bridges classified to be essential:

$\phi_s = \min(\phi_{sc},\phi_{sf})$ for substructures with confined members

$\phi_s = \min(\phi_{su},\phi_{sf})$ for substructures with unconfined members

For the substructures of all other bridges:

$\phi_s = \phi_{sc}$ for substructures with confined members

$\phi_s = \phi_{su}$ for substructures with unconfined members

where:

ϕ_{su} = system factor for ultimate capacity of substructures with unconfined concrete members

ϕ_{sc} = system factor for ultimate capacity of substructures with confined concrete members

ϕ_{sf} = system factor for bridge substructure functionality

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

Confined concrete columns shall satisfy the provisions of Article 5.10.11.4.1d. Columns that do not satisfy Article 5.10.11.4.1d shall be considered unconfined.

Recommended values for ϕ_{su},ϕ_{sc} and ϕ_{sf} for typical substructures with columns, piers, walls and abutments founded on spread footings, drilled shafts or piles are specified in Table 1.4.1.2-1 for soft, normal and stiff soils. For bridges with nontypical configurations, the direct analysis approach of Article 1.3.4.2.1 shall be used.

Bridges susceptible to brittle damage include substructures with members that are exposed to collisions from ships, vehicles, and debris carried by swelling streams and rivers.

Table 1.3.4.2-1—System factors for bridge substructures

Substructure and member Types	Foundation	Spread footing	Drilled shaft			Piles		
	Soil type	All soil	Soft	Normal	Stiff	Soft	Normal	Stiff
Walls, abutments and one column bents in bending	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							
2-column bents in bending	ϕ_{su}	0.95	0.85	0.90	0.95	0.90	0.95	1.00
	ϕ_{sc}	1.00	0.95	1.05	1.15	1.05	1.05	1.05
	ϕ_{sf}	1.00	0.80	0.85	1.00	0.95	0.95	0.95
multi-column bents in bending	ϕ_{su}	1.00	0.90	0.95	1.00	0.95	1.00	1.05
	ϕ_{sc}	1.05	1.00	1.15	1.20	1.10	1.10	1.10
	ϕ_{sf}	1.05	0.80	1.05	1.20	1.05	1.05	1.05
All members in shear	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							
Bar pullout and other joint failures	ϕ_{su}							
	ϕ_{sc}	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	ϕ_{sf}							

1.3.4.2.1 Direct Redundancy Analysis for Substructures

For bridges classified to be critical, and for bridges not covered in Table 1.3.3.2-1, the system factor of Equation 1.3.2.1-1 for the structural components of a substructure system shall be calculated from the results of a nonlinear pushover analysis using Equation 1.3.3.2.1-1:

$$\phi_s = \min\left(\frac{R_u}{1.20}, \frac{R_f}{1.20}, \frac{R_d}{0.50}\right) \qquad 1.3.4.2.1\text{-}1$$

A minimum value of ϕ_s=0.80 is recommended but in no instance should ϕ_s be taken as greater than 1.20.

Where:

R_u = system reserve ratio for the ultimate limit state, $R_u = \frac{LF_u}{LF_1}$

R_f = system reserve ratio for the functionality limit state, $R_f = \frac{LF_f}{LF_1}$

R_d = system reserve ratio for the damage condition, $R_d = \frac{LF_d}{LF_1}$

LF_u, LF_f, LF_d and LF_1 are obtained from the nonlinear pushover analysis

where:

LF_u = the lateral load factor that causes the failure of the substructure
LF_f = the lateral load factor that causes the total lateral deflection of the substructure to reach a value equal to average clear column height/50
LF_d = the lateral load factor that causes the failure of a damaged substructure
LF_1 = the lateral load factor that causes the first member of the intact substructure to reach its limit capacity

COMMENTARY

The attached document provides a commentary on the recommended modifications to article 1.3 in Section 1 "Introduction" of the AASHTO LRFD Bridge Design Specifications. The modifications provide a method to include system factors that account for system ductility and redundancy during the design and safety evaluation of highway bridges.

C 1.3 DESIGN PHILOSOPHY

C1.3.1 General

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications due to incomplete knowledge of inelastic structural action. The use of system factors in the design equation is meant to account for the inelastic behavior and the presence of system reserve.

C.1.3.2 Limit States

C 1.3.2.1 General

Equation 1 is the basis of LRFD methodology. Assigning resistance factor ϕ = 1.0 to all nonstrength limit states is a temporary measure: development work is in progress.

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Ductility, redundancy, and operational importance are significant aspects affecting the margin of safety of bridge structural systems and the presence of system reserve strength. While the first two directly relate to the physical strength, the last concerns the consequences of the bridge being out of service.

The system factor, ϕ_s of Equation 1, provides a measure of the system reserve strength as it relates to ductility, redundancy, and operational importance, and their interaction and system synergy. These system factors are calibrated based on reliability techniques to provide bridges with adequate levels of overall safety and system reliability. Non-redundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. The aim of ϕ_s is to add reserve capacity for non-redundant systems as the overall system reliability is increased. If adequate redundancy levels are present, a system factor ϕ_s=1.0 is used. In the instances where the level of redundancy is high, a value of ϕ_s greater than 1.0 may be used. Upper and lower limits of 1.20 and 0.80 are proposed for ϕ_s until more experience is gained in the application of these factors in actual design situations.

Earlier editions of this document accounted for the system effects by applying load multipliers on the right-hand side of the equation. Starting with this edition, the system factor is applied on the left-hand side of the equation because redundancy relates to the capacity of the system and thus should be applied on the resistance side of the equation as is traditionally done in LRFD methods.

Unlike the load modifiers of previous editions, the system factor consists of one term only, tabulated (or calculated) for different superstructure and substructure configurations. Differences in member ductility and redundancy levels are considered by using different tables for different substructure and superstructure configurations and using different factors when providing members with additional ductility (e.g. when using confined versus unconfined concrete columns) or when the system is capable of better redistributing the load (e.g., when diaphragms

are provided). Instead of using a multiplier to account for operational importance, different system limit states are provided leaving the engineer in consultation with the Owner the option of choosing the appropriate limit states depending on the bridge's operational importance. This approach is consistent with current trends to develop performance-based design methods in bridge engineering.

C1.3.2.2 Service Limit State

The service limit state provides certain experience-related provisions that cannot always be derived solely from strength or statistical considerations.

C1.3.2.3 Fatigue and Fracture Limit State

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

C 1.3.2.4 Strength Limit State

Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained.

C1.3.2.5 Extreme Event Limit State

The extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge.

C 1.3.3 Importance Categories

Such classification should be based on social/survival and/or security/defense requirements. The commentary to Article 3.10.1 provides some guidance on selecting importance categories as they relate to design for earthquakes. This information can be generalized for other situations.

Also, the Owner should identify bridges that are susceptible to brittle damage. These include bridges with fatigue-prone details or bridges that may be subject to collision with ships, trucks, or debris and ice carried by overflowing streams.

C1.3.4 System Factor, ϕ_s

Separate tables of system factors and direct analysis methods are provided for bridge substructures and superstructures systems assuming nonmonolithic constructions. For monolithic constructions, the direct analysis approach should be used for vertical loads using 1.3.4.1 and for lateral loads using 1.3.4.2.

Ductility

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load carrying capacity immediately when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformations before any loss of load carrying capacity occurs. Ductile behavior provides warning of structural failure by large inelastic deformations. Under repeated seismic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structural survival.

If by means of confinement or other measures, a structural component or connection made of brittle materials can sustain inelastic deformations without significant loss of load carrying capacity, this component can be considered ductile. Such ductile performance shall be verified by testing.

In order to achieve adequate inelastic behavior the system should have a sufficient number of ductile members and either:

- joints and connections that are also ductile and can provide energy dissipation without loss of capacity, or,
- joints and connections that have sufficient excess strength so as to ensure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.

Statically ductile but dynamically non-ductile response characteristics should be avoided. Examples of this behavior are shear and bond failures in concrete members and loss of composite action in flexural components.

Past experience indicates that typical components designed in accordance with these provisions generally exhibit adequate ductility. Connection and joints require special attention to detailing and the provision of load paths.

The Owner may specify a minimum ductility factor as an assurance that ductile failure modes will be obtained. The factor may be defined as:

$$\mu = \frac{\Delta_u}{\Delta_y} \qquad \text{(C 1.3.4 .1)}$$

where:

Δ_u = deformation at ultimate
Δ_y = deformation at the elastic limit

The ductility capacity of structural components or connections may either be established by full or large scale testing, or with analytical models that are based on documented material behavior. The ductility capacity for a structural system may be determined by integrating local deformations over the entire structural system.

The special requirements for energy dissipating devices are imposed because of the rigorous demands placed on these components.

Redundancy

Bridge redundancy is the capability of a bridge structural system to carry loads after damage or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a system as non-redundant. System redundancy is related to a system’s configuration as well as the ductility of its members.

C1.3.4.1 Bridge Superstructures

The interaction of structural members of a bridge superstructure should be considered in assessing the behavior of the system. Bridge superstructure redundancy is the capability of a superstructure system to carry loads after damage to or the failure of one or more of its members. This provision focuses on vertical overloads because they cause most superstructure failures.

System factors are used in this article to maintain an adequate level of superstructure system safety. Less redundant systems are penalized by requiring their structural members to provide higher safety levels than those of similar superstructures with redundant configurations. The aim of ϕ_s is to add member and system capacity to less redundant systems such that the overall system reliability is increased. When adequate redundancy is present, a system factor, ϕ_s, greater than 1.0 may be used.

Two methods are provided to determine appropriate values for ϕ_s:

a) Tables of ϕ_s are provided for common superstructure configurations. (Table 1.3.4.1-1)
b) A direct analysis approach is recommended for superstructures with nontypical configurations and members. (Article 1.3.4.2.1).

The use of more stringent criteria for critical bridges is consistent with current trends to use performance-based design in bridge engineering practice. In addition to Article 1.3.3, the commentary of Article 3.10.3 provides some guidance on selecting importance categories as they relate to design for earthquakes. This information can be generalized for other situations.

The system factors provided in Table 1.3.4.1-1 are calibrated in NCHRP 406 to satisfy Equation 1.3.4.1.1-1 for a set of typical multi-girder bridge superstructure configurations. This set includes prestressed concrete and multi-girder composite steel simple span and continuous bridges with span lengths varying between 9m and 45m.

For the purpose of determining system factors, each web of a box girder may be considered as an I-girder.

Subsystems that are redundant should not be penalized if the overall system is non-redundant. Thus, closely spaced parallel stringers would be redundant even in a two-girder-floor-beam main system.

The values provided in Table 1.3.4.1-1 for truss and arch bridges are adopted from NCHRP project 12/46 for welded members. During the evaluation of truss and arch bridges with riveted members or eyebars, Table C.1.3.4.1-1 also adapted from NCHRP 12/46 should be used.

Table C.1.3.4.1-1 System Factors for Trusses and Arch Bridges

Structure	Member type	System factor ϕ_s
Truss/ Arch Bridges	Welded members	0.80
	Riveted members	0.90
	Multiple eye bar members	0.90

Members in shear as well as all joints and connections are assigned a system factor ϕ_s=0.80. This assumes that the resistance factor ϕ was calibrated to satisfy a target member reliability index β_{member}=3.5. Since shear failures and connection failures are brittle causing the failure of the complete system, the application of a system factor ϕ_s=0.80 will increase the reliability index of the member and also that of the system so that $\beta_{member}=\beta_{system} \approx 4.5$.

The incremental analysis described in Section 1.3.4.1.1 “Direct Analysis of Bridge Superstructures” should be used for superstructures with configurations not covered in Table 1.3.4.1-1 or to obtain more precise values of ϕ_s for all configurations.

C 1.3.4.1.1 Direct Redundancy Analysis for Superstructures

A nonlinear incremental analysis of critical bridges and bridges susceptible to brittle damage shall be performed as described in NCHRP report 406 accounting for the nonlinear behavior of the structural components. The analysis requires the availability of a nonlinear incremental analysis program that provides a vertical load versus vertical deflection curve and that adequately models the nonlinear behavior of the components up to crushing of concrete, rupture of steel, or the formation of a collapse mechanism. Programs such as NONBAN or commercially available finite element packages can be used for such purpose.

The ratio of the vertical force causing the failure of the superstructure system to the force causing the failure of one structural member is defined as the system reserve ratio for the ultimate capacity R_u.

The ratio of the vertical force causing a vertical deflection equal to span length/100 to the force causing the failure of one member is defined as the system reserve ratio for the functionality limit R_f.

The ratio of the vertical force causing the failure of a damaged bridge superstructure to the force causing the failure of one member of the undamaged bridge is defined as the system reserve ratio for the damaged condition R_d.

Possible damage scenarios include the loss of a single beam or a portion of beam or a connection and should be considered in consultation with the bridge Owner.

The nonlinear incremental load analysis is effected by applying the factored loads (vertical live and dead loads) on a structural model of the superstructure and incrementing the live loads until the failure of the first member. The factor by which the original vertical live load is multiplied to cause the failure of the first member is defined as LF_1. The nonlinear analysis is then continued beyond the failure of the first member until the maximum vertical displacement reaches a value equal to span length/100. This displacement limit is defined as the functionality limit state. The factor by which the original load is multiplied to reach this functionality limit is defined as LF_f. The analysis is further continued beyond this point until one member reaches its maximum strain and concrete crushing or steel rupture ensue or until a hinge collapse mechanism occurs. Concrete members crush at a strain of 0.003. Steel I-girder members in bending rupture at hinge rotation equal to 65 mrad. Structural steel ruptures when the ductility in percent elongation is around 20%. Compression members fail when their critical buckling loads are reached. These cases define the ultimate capacity limit state. The load factor by which the original vertical live load is multiplied to reach the ultimate capacity is defined as LF_u.

The same process is repeated for a model of a damaged bridge whenever consideration of the survival of a damaged bridge is required by the Owner (e.g., when the bridge is classified as critical or when a damage situation is considered likely). The damage scenario must be realistic and must be chosen in consultation with the bridge Owner. Damage scenarios may include the loss of a member that may be subjected to risk of fatigue fracture or brittle failure from collisions by ships, vehicles, or flooding debris, etc. The analysis of the damaged bridge is effected in the same manner outlined above for the intact structure. But only the ultimate capacity limit state of the damaged bridge needs to be checked. The load factor by which the original vertical live load is multiplied to reach the ultimate capacity of the damaged structure is defined as LF_d.

Bridge superstructures that produce redundancy ratios R_u=1.3, R_f=1.1, and R_d=0.5 or higher are classified as adequately redundant. Those that do not satisfy these criteria will have system factors ϕ_s less than 1.0 and require higher component safety levels. If bridge superstructure redundancy is sufficiently high, a system factor greater than 1.0 may be used.

Satisfying the criteria for R_u, R_f, and R_d is recommended for bridges classified to be critical or those that are susceptible to brittle damage. Satisfying the criteria for only R_u and R_f is recommended for essential bridges. Satisfying the criteria for only R_u is sufficient for all other bridges.

The check of R_u verifies that a bridge's ultimate system capacity is at least 30% higher than the load level that will cause the failure of one member.

The check of R_f verifies that a bridge's vertical deflection during the application of high loads is still acceptable allowing the bridge to remain functional for emergency situations.

The check of R_d verifies that a damaged bridge is still capable of carrying 50% of the load that a nondamaged bridge can carry before one member fails.

Table 1.3.4.1-1 does not provide values for the functionality limit state because for the typical cases tabulated, the results from the functionality limit state are similar to those for the ultimate capacity. This observation is not necessarily true for other bridge configurations and especially for truss and arch bridges. Hence, a check on the functionality limit state must be undertaken for all essential and critical bridges.

C 1.3.4.2 Bridge Substructures

The interaction of structural members of a bridge substructure should be considered in assessing the behavior of the substructure system. Bridge substructure redundancy is the capability of a substructure system to carry loads after damage or to the failure of one or more of its members. For substructures, the most common failures are the result of lateral overloads from earthquakes, wind, ice, stream flow or accidental collisions of vehicles or vessels. Consequently, this article focuses on lateral overload from any of the above sources.

System factors are used in this article to maintain an adequate level of substructure system safety. Less redundant systems are penalized by requiring their structural members to provide higher safety levels than those of similar substructures with redundant configurations. The aim of ϕ_s is to add member and system capacity to less redundant systems such that the overall system reliability is increased. When adequate redundancy is present, a system factor, ϕ_s, greater than 1.0 may be used.

Two methods are provided to determine appropriate values for ϕ_s:

a) Tables of ϕ_s are provided for common substructure configurations. (Table 1.3.4.2-1)
b) A direct analysis approach is recommended for substructures with nontypical configurations and members. (Article 1.3.4.2.1).

The use of more stringent criteria for critical and essential bridges is consistent with current trends to use performance-based design in bridge engineering practice. In addition to Article 1.3.3, the commentary of Article 3.10.3 provides some guidance on selecting importance categories as they relate to design for earthquakes. This information can be generalized for other situations.

The system factors provided in Table 1.3.4.2-1 are calibrated in NCHRP Report 12-47 to satisfy Equation1.3.4.2.1-1 for a set of typical bridge substructure configurations. This set includes bridges with concrete columns varying in height between 3.5m to 18m and a vertical rebar reinforcement ratio of 1.85 to 2.3%. The nonlinear pushover analysis described in Section 1.3.4.2.1 "Direct Analysis of Bridge Substructures" should be used for substructures with other configurations.

Soft soils are defined as soils that produce an SPT blow count N=5. Normal soils are those with N=15. Stiff soils are those with N=30 or higher. SPT blow count = Standard Penetration Test blow count (number of blows per 1 foot=0.035m penetration into the soil). Use the nearest tabulated SPT for values of N not provided in Table 1.3.4.2.1-1.

Members in shear as well as all joints and connections are assigned a system factor ϕ_s=0.80. This assumes that the resistance factor ϕ was calibrated to satisfy a target member reliability index β_{member}=3.5. Since shear failures and connection failures are brittle causing the failure of the complete system, the application of a system factor ϕ_s=0.80 will increase the reliability index of the member and also that of the system so that $\beta_{member}=\beta_{system} \approx 4.5$.

C 1.3.4.2.1 Direct Redundancy Analysis for Substructures

The nonlinear pushover analysis of critical bridges shall be performed as described in NCHRP report 12/47 accounting for the nonlinear behavior of the structural elements of the substructure and considering soil/foundation flexibility. The analysis requires the availability of a nonlinear analysis program that provides a lateral load versus lateral deflection curve and that adequately models the nonlinear behavior of the substructure components up to crushing of concrete, rupture of steel, or the formation of a collapse mechanism. The program should also be able to model the stiffness and the soil/foundation system by either the use of equivalent springs or actual modeling of the nonlinear foundation. Programs such as FLPIER or PIERPUSH can be used for such purpose.

The ratio of the lateral force causing the failure of the bent system to the force causing the failure of one structural member is defined as the system reserve ratio for the ultimate capacity R_u.

The ratio of the lateral force causing a lateral deflection equal to clear column height/50 to the force causing the failure of one member is defined as the system reserve ratio for the functionality limit R_f.

The ratio of the lateral force causing the failure of a damaged bridge substructure to the force causing the failure of one member of the undamaged bridge is defined as the system reserve ratio for the damaged condition R_d.

Possible damage scenarios include the loss of a single column or a connection and should be considered in consultation with the bridge Owner.

The nonlinear pushover analysis is effected by applying the factored loads (lateral and vertical live and dead loads) on a structural model of the substructure and incrementing the lateral loads until the failure of the first member. The factor by which the original lateral load is multiplied to cause the failure of the first member is defined as LF_1. The nonlinear analysis is then continued beyond the failure of the first member until the lateral displacement at the top of the bent reaches a value equal to average clear column height/50. This displacement limit is defined as the functionality limit state. The factor by which the original load is multiplied to reach this functionality limit is defined as LF_f. The analysis is further continued beyond this point until one member reaches its maximum strain and concrete crushing ensues or until a hinge collapse mechanism occurs. Unconfined concrete members crush at a strain of 0.003. Confined members crush at a strain of 0.015. These cases define the ultimate capacity limit state. The load factor by which the original lateral load is multiplied to reach the ultimate capacity is defined as LF_u.

The same process is repeated for a model of a damaged bridge whenever consideration of the survival of a damaged bridge is required by the Owner (e.g., when the bridge is classified as critical or when a damage situation is considered likely). The damage scenario must be realistic and must be chosen in consultation with the bridge Owner. Damage scenarios may

include the complete loss of a column that may be subjected to risk of brittle failure from collisions by ships, vehicles, or flooding debris, etc. The analysis of the damaged bridge is effected in the same manner outlined above for the intact structure. But only the ultimate capacity limit state of the damaged bridge needs to be checked. The load factor by which the original lateral load is multiplied to reach the ultimate capacity of the damaged structure is defined as LF_d.

Bridge substructures that produce redundancy ratios R_u=1.2, R_f=1.2, and R_d=0.5 or higher are classified as adequately redundant. Those that do not satisfy these criteria will have system factors ϕ_s less than 1.0 and require higher component safety levels. If bridge redundancy is sufficiently high, a system factor greater than 1.0 may be used.

Satisfying the criteria for R_u, R_f, and R_d is recommended for bridges classified to be critical. Satisfying the criteria for only R_u and R_f is recommended for essential bridges. Satisfying the criteria for only R_u is sufficient for all other bridges.

The check of R_u verifies that a bridge's ultimate system capacity is at least 20% higher than the load level that will cause the failure of one member.

The check of R_f verifies that a bridge's lateral deflection during the application of high loads is still acceptable allowing the bridge to remain functional for emergency situations.

The check of R_d verifies that a damaged bridge is still capable of carrying 50% of the load that a nondamaged bridge can carry before one member fails.

REFERENCES

Lichtenstein, A.G. & Associates, 2000, “Manual For Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges”, Final Draft, National Cooperative Highway Research Program, NCHRP 12-46, Transportation Research Board, Washington DC.

David Liu, Michel Ghosn, Fred Moses and Ansgar Neuenhoffer, 2000, “Redundancy in Highway Bridge Substructures”, National Cooperative Highway Research Program, NCHRP project 12-47, Transportation Research Board, National Research Council, Washington DC.

Michel Ghosn and Fred Moses, 1998, “Redundancy in Highway Bridge Superstructures”, National Cooperative Highway Research Program, NCHRP Report 406, Transportation Research Board, National Research Council, Washington DC.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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