PROFESSIONAL PRACTICE GUIDELINES

CIVIL AND TRANSPORTATION INFRASTRUCTURE

PERFORMANCE-BASED SEISMIC DESIGN OF BRIDGES IN BC

VERSION 1.0
PUBLISHED MARCH 23, 2018

ENGINEERS &
GEOSCIENTISTS
BRITISH COLUMBIA

VERSION 1.0, MARCH 23, 2018

PREFACE

The Professional Practice Guidelines – Performance Based Seismic Design of Bridges in British Columbia (BC) have been developed with the support of the BC Ministry of Transportation and Infrastructure (BC MoTI), the Canadian Association of Earthquake Engineering, and the Structural Engineers Association of BC. These guidelines will assist Engineering Professionals in carrying out performance-based seismic design of bridges in a consistent manner while incorporating best practices.

This document has been prepared for the information of Engineering Professionals, statutory decision-makers, regulators, the public at large, and a range of other stakeholders who might be involved in, or have an interest in, performance-based seismic design of bridges.

These guidelines provide a common level of expectation for the various stakeholders with respect to the level of effort, due diligence, and standard of practice to be followed when carrying out performance-based seismic design of bridges. This document should be read in conjunction with the CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014). The BC MoTI Supplement to the Code (Supplement) is also referenced in these guidelines (BC MoTI 2016).

It is important to note that these guidelines are not intended to replace any provisions of the Code and commentary but to provide guidance in applying them.

These guidelines outline the appropriate standard of practice to be followed at the time that they were prepared. However, this is a living document that is to be revised and updated, as required in the future, to reflect the developing state of practice.

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ABBREVIATIONS

ABBREVIATION	TERM
AHJ	authority having jurisdiction
BC	British Columbia
BC MoTI	BC Ministry of Transportation and Infrastructure
CHBDC	Canadian Highway Bridge Design Code
EDA	elastic dynamic analysis
ERS	earthquake-resisting system
ESA	elastic static analysis
FBD	force-based design
ISPA	inelastic static pushover analysis
NTHA	non-linear time history analysis
NRC	Natural Resources Canada
OBG	orthotropic box girder
P3	public-private partnership
PBD	performance-based design
RSA	response spectrum analysis
SPC	seismic performance category
SSI	soil structure interaction
TEBF	tubular eccentrically braced frame
UHS	uniform hazard spectra

DEFINED TERMS

The following definitions are specific to this guideline.

TERM	DEFINITION
Act	Engineers and Geoscientists Act [RSBC 1996] Chapter 116.
Association	The Association of Professional Engineers and Geoscientists of the Province of British Columbia, also operating as Engineers and Geoscientists BC.
Bylaws	The Bylaws of Engineers and Geoscientists BC made under the *Act*.
Code	CAN/CSA-S6-14 Canadian Highway Bridge Design Code.
Engineers and Geoscientists BC	The Association of Professional Engineers and Geoscientists of the Province of British Columbia, also operating as Engineers and Geoscientists BC.
Engineering Professional(s)	Professional engineers, including licensees, who are licensed to practice by Engineers and Geoscientists BC.
Engineer of Record	For the purposes of these guidelines, the Engineer of Record is an Engineering Professional with the appropriate education, training, and experience to provide professional services related to performance-based seismic design of bridges as described in these guidelines.
Supplement	BC Ministry of Transportation Supplement to CAN/CSA-S6-14.

1.0 INTRODUCTION

This document is intended to provide guidance to Engineering Professionals undertaking performance-based seismic design of bridges in British Columbia (BC).

Furthermore, the guidance in this document will support the consistent and appropriate application of the performance-based seismic bridge design requirements in the CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014).

Reference has also been made to the BC Ministry of Transportation and Infrastructure (BC MoTI) Supplement to CAN/CSA-S6-14 (the Supplement) (BC MoTI 2016) in this document, which reflects BC MoTI-specific requirements. It should be noted that other jurisdictions and/or owners are not obligated to follow the stipulations in the Supplement.

In many parts of the world, building regulations are developed from a desire to mitigate the potential for unacceptable losses of life, property, and economic stability due to fire and natural hazard events. The traditional prescriptive codes generally require less analysis and calculation, as they are based on distinct and discrete actions and a single level of seismic force. The prescribed materials and construction methods are based on past experience and common availability. Prescriptive, or force-based, seismic design codes require elastic analysis with estimated factors for ensuring no collapse at the designated seismic force. Design specifications are for individual components to achieve minimum strengths, omitting direct consideration of global structural interactions. Improved performance levels are achieved indirectly by applying larger importance factors to design forces, and it is often difficult to ascertain the actual level of performance delivered.

The Code is the first national bridge code to adopt performance-based design (PBD) methodology and to date has done so only in the seismic design section.

PBD codes focus more on the performance expected under varying seismic conditions and less on specific materials, mechanisms, and technologies. The intent is to limit damage, public vulnerability, emergency response, and post-earthquake repair, and to speed recovery. PBD codes direct design to meet each bridge's specific operational expectation and acceptable risk. PBD facilitates innovation in materials, technologies, and construction methodologies, and, while meeting the core performance criteria, allows modification of the design to reflect changes in environment, functionality, sustainability, and resilience expectations. PBD should assist in the clear communication of measurable criteria between design engineers, owners, emergency planners, and the public to provide a common understanding of the expected performance of the bridge.

1.1 PURPOSE OF THESE GUIDELINES

This document provides guidance for qualified Engineering Professionals who are involved in the performance-based seismic design of bridges in BC. These guidelines provide a common approach to be followed for carrying out a range of professional activities related to performance-based seismic design of bridges in BC.

Following are the specific objectives of these guidelines:

1. Describe the standard of practice Engineering Professionals should follow in providing professional services related to this professional activity.
2. Outline the tasks that should generally be performed by Engineering Professionals when carrying out this professional activity to fulfill the member's professional obligations under the *Act*. These obligations include the member's primary duty to protect the safety, health, and welfare of the public and the environment.
3. Outline the professional services that should generally be provided by Engineering Professionals conducting this type of work.
4. Describe the roles and responsibilities of the various participants/stakeholders involved in such work. The document will assist in delineating the roles and responsibilities of the various participants/stakeholders, which will include the Engineer of Record, owners, and authorities having jurisdiction (AHJ).
5. Describe the necessary skill sets, which are consistent with the training and experience required to carry out this professional activity.
6. Provide an assurance statement, which the Engineer of Record must seal, sign, and date (**Appendix A**). This assurance statement will confirm that, with respect to the specific professional activity carried out, the appropriate considerations have been addressed (both regulatory and technical).
7. Provide guidance as to how to meet the seven quality management (QM) requirements under the *Act* and Bylaws when carrying out the professional activities identified in these professional practice guidelines.
8. Provide case studies outlining examples of the application of the guidelines for chosen bridge materials, configurations, and components (**Appendix B**).

1.2 ROLE OF ENGINEERS AND GEOSCIENTISTS BC

These guidelines have been formally adopted by the Engineers and Geoscientists BC Council, and form part of the ongoing commitment to maintaining the quality of professional services that Engineering Professionals provide to their clients and the general public. Engineering Professionals are professionally accountable for their work under the *Act*, which is enforced by Engineers and Geoscientists BC.

An Engineering Professional must exercise professional judgment when providing professional services; as such, application of these guidelines will vary depending on the circumstances.

Engineers and Geoscientists BC supports the principle that appropriate financial, professional, and technical resources should be provided (for example, by the client and/or the employer) to support Engineering Professionals responsible for carrying out performance-based seismic design of bridges in BC. These guidelines may be used to assist in establishing the objectives, level of service, and terms of reference of an agreement between an Engineering Professional and a client.

By following these guidelines, an Engineering Professional will fulfill his/her professional obligations, especially with regards to the Code of Ethics Principle 1 (hold paramount the safety, health, and welfare of the public, protection of the environment, and promote health and safety in the workplace). Failure to meet the intent of these guidelines could be evidence of unprofessional conduct and lead to disciplinary proceedings by Engineers and Geoscientists BC.

1.3 INTRODUCTION OF TERMS

For the purposes of these guidelines, the Engineer of Record is an Engineering Professional with the appropriate education, training, and experience to provide professional services related to performance-based seismic design of bridges as described in these guidelines.

See the **Defined Terms** section at the front of the document for a full list of definitions specific to these guidelines.

1.4 SCOPE OF THESE GUIDELINES

These guidelines summarize various aspects of professional practice related to performance-based seismic design of bridges in BC.

This document provides a summary of different project delivery models, the roles and responsibilities of various entities within each model, and guidelines for best professional practice related to different design steps. In addition, there is discussion of related quality control and quality assurance issues.

Several case study examples have been provided in **Appendix B** for an assortment of bridge types encompassing different analysis techniques and demonstration of various performance criteria.

The guidance in this document can, in principle, be used towards both the design of new bridges and the retrofit of existing bridges in BC.

1.5 APPLICABILITY OF THESE GUIDELINES

These guidelines are not intended to provide complete step-by-step instructions for carrying out performance-based seismic design of bridges in BC. Rather, these guidelines outline considerations for this activity.

An Engineering Professional’s decision not to follow one or more aspects of these guidelines does not necessarily mean a failure to meet his or her professional obligations. Such judgments and decisions depend upon weighing facts and circumstances to determine whether another reasonable and prudent Engineering Professional in a similar situation would have conducted himself/herself similarly.

The Code provides rules for when the prescriptive force-based design (FBD) may be used based on the importance category and the site spectral acceleration values. PBD is required for all other categories but can be used for all categories and seismic events at the discretion of the owner. The importance category is defined by the owner as lifeline, major route, or other bridge. The site spectral acceleration values for the site are as determined by the Geological Survey of Canada for the relevant return period event unless site-specific analysis is carried out.

1.6 ACKNOWLEDGEMENTS

These guidelines were prepared on behalf of Engineers and Geoscientists BC by a committee of Engineering Professionals and was reviewed by a review task force and various Engineers and Geoscientists BC committees. The authors and the members of the review task force are listed in Appendix C.

The authors thank the reviewers for their constructive suggestions. A review of this document does not necessarily indicate that reviewers and/or their employer, agency, or affiliated association endorse everything in these guidelines.

2.0 ROLES AND RESPONSIBILITIES

This section summarizes the organization of various delivery models and delineates the roles and responsibilities for various parties involved in design and project delivery.

2.1 COMMON FORMS OF PROJECT ORGANIZATION

There are several types of project delivery models for bridge projects which can affect the design engineer/owner relationship and communication links. Each has its own organization, which impacts the roles of the Engineer of Record, the owner, and the owner's engineer. Broadly speaking, these can be divided into design-bid-build (traditional), design-build, and public-private-partnership (P3) delivery models.

Within the traditional design-bid-build delivery model, most of the project risks reside with the owner. In contrast, the design-build and P3 delivery models, through contract, generally distribute the technical, financial, and operational risks and responsibility among the owner, the design-build contractor, and, for P3 projects, the concessionaire, who takes on all or part of the financing, operation, and maintenance of the project through the term of the agreement (Auditor General of British Columbia, 2012).

Following are brief descriptions of each delivery model.

2.1.1 DESIGN-BID-BUILD (TRADITIONAL) DELIVERY

In the traditional design-bid-build model, the various project phases are procured and delivered under separate contracts. The different contract stages under this delivery model comprise design, construction and operation, and maintenance in a sequential fashion. The design engineer is contracted directly to the owner or the owner's representative.

Communications with respect to PBD are directly between the Engineer of Record and the owner.

2.1.2 DESIGN-BUILD AND P3 DELIVERY

In the design-build model, the Engineer of Record is contracted to the contractor instead of the owner. The contractor is responsible to the owner for both the design and construction. Once substantial completion of the project is reached, the owner takes over responsibility for operation and maintenance. The contractor subsequently maintains design and construction responsibility through a warranty period.

In the P3 delivery model, the public sector owner generally contracts with a concessionaire, who takes overall responsibility for design, construction, some or all of the financing, and, in some cases, operation and maintenance. The Engineer of Record is engaged by the contractor, who is responsible to the concessionaire through a warranty period. The concessionaire is responsible to the owner for the term of financing, typically 25 to 35 years. The facility is then turned over to the owner.

For these project delivery models, the Engineer of Record does not have direct communications with the owner, and decisions relating to PBD must be made by the owner and included in the design-build specifications. The owner generally engages an owner's engineer with appropriate technical expertise to advise, prepare the specifications, and review the design-build contractor's submittals for conformance with the specifications. Ongoing communication between the Engineer of Record and the owner's engineer can facilitate the various challenges in the interpretation of PBD criteria, which are by nature non-prescriptive.

2.2 RESPONSIBILITIES

Within the framework of PBD of bridges, various entities have different and overlapping responsibilities under different delivery models. The responsibilities may lie with the owner, the Engineer of Record, or the owner's engineer. The following discussion summarizes common roles and responsibilities for PBD in this context.

One of the goals of PBD is to provide owners and authorities with a clearer understanding of the structural performance and serviceability of a structure during and after a seismic event. A significant advantage of PBD lies in aligning the owner's and the Engineer of Record's requirements and expectations early in the design process. The Engineer of Record and owner's engineer should be familiar with the Code provisions, inform the owner of the different performance levels, and discuss the need and requirements for emergency response on the traffic route after a seismic event. The owner then decides which performance and design requirements to prescribe for the bridge.

In a traditional design-bid-build project delivery model, the responsibility of informing and educating the owner generally lies with the Engineer of Record. However, in a P3 model, this responsibility falls to the owner's engineer. During design, the Engineer of Record is responsible for interpreting the Code's performance criteria into design criteria. The Engineer of Record should communicate any issues that may present obstacles to achieving the desired performance.

2.2.1 OWNER

An owner has the following responsibilities:

- Determine the earthquake performance needed from the structure and establish the importance category of the bridge. Depending on the complexity and importance of the bridge, this may entail discussions with emergency planning stakeholders.
- Provide a clearly documented understanding of all seismic performance expectations.
- Discuss with the Engineer of Record (in the case of design-bid-build project delivery) or the owner's engineer (in the case of design-build and P3 project delivery) additional seismic design criteria that may be required for the project.
- Accept the design criteria, performance levels, and design seismic inputs to be used.
- Receive the assurance statement from the Engineer of Record upon completion of the design activities as outlined in these guidelines.

2.2.2 OWNER'S ENGINEER

In design-build or P3 models, the owner's engineer is responsible for advising the owner and developing project specifications. The owner's engineer may be tasked with developing additional specifications for competing proponents.

2.2.3 ENGINEER OF RECORD

The Engineer of Record is responsible for carrying out the performance-based seismic design of bridges in BC.

Prior to carrying out the design, the Engineer of Record should:

- confirm that he or she has appropriate training and experience and identify when additional expertise is required;
- identify important aspects of communication with the client regarding PBD;
- lead communication between the structural and geotechnical disciplines to achieve effective interaction in developing the PBD;
- review the design criteria provided by the owner or the owner's engineer for appropriateness and discuss with the client; and
- ensure that Engineers and Geoscientists BC requirements for documented independent review of structural design and for documented checks of engineering work are followed (Engineers and Geoscientists BC 2018a and 2018b).

Upon completion of the design, the Engineer of Record should complete the assurance statement found in Appendix A: Engineer of Record – Bridge Seismic Design Assurance Statement. The assurance statement, along with the appropriate design documentation, are to be provided to the owner/client.

Other responsibilities of the Engineer of Record do not differ from any other structural design.

2.2.4 OWNER OR CLIENT AND ENGINEERING PROFESSIONAL INTERACTION

The interactions between the owner (or the client, where this intermediary exists) and the various professional engineering disciplines form the backbone of the process and are critical to the PBD of bridges. These interactions are necessary to ensure the owner's or client's performance requirements are understood by the Engineering Professionals designing and delivering the project.

The owner or client can work with the Engineering Professionals to specify the bridge importance category, discuss the resultant seismic performance category (SPC), and determine the required level of analysis and the corresponding performance requirements. In addition, such interactions provide a platform for the Engineering Professionals to describe the design earthquake parameters, the intended earthquake load-resisting components, the required level and type of analysis, and how the performance objectives will be met and demonstrated.

Early agreement reached through such cooperation will save critical time and reduce the potential for conflict at later stages of the project by providing consistent expectations at the beginning. Interaction between the owner or client and the Engineering Professional is also critical to understanding the impact of any deviations and ensuring that the owner's expectations are met and the post-seismic bridge performance and functionality is not adversely impacted.

2.2.5 AUTHORITY HAVING JURISDICTION

The preamble in the Code states:

> "In Canada, the legal mandate for establishing design and construction requirements for highways, including highway bridges, lies with the provincial and territorial governments. All provinces and territories, with the exception of Manitoba, have mandated this Code for use under their jurisdictions."

Bridges on federal highways are therefore designed to the Code.

Each province has the authority to provide exceptions to the Code to reflect the specific needs and local conditions within the province. In BC, the BC MoTI has published the Supplement, which is to be used on bridges under its jurisdiction. Other changes to the Code or additional criteria may also be specified by the AHJ or the owner as part of the terms of reference or project agreement.

2.3 PROJECT COORDINATION

Within the context of PBD of bridges, coordination needs to be carried out amongst the different parties such as the owner, owner's engineer, Engineer of Record, and design-build contractor. Similarly, the various design disciplines comprising the Engineering Professionals need to coordinate closely. This is particularly important for the structural and geotechnical disciplines.

2.3.1 STRUCTURAL ENGINEER AND GEOTECHNICAL ENGINEER COORDINATION

PBD involves modelling and analysis of a structure (or its parts) and the soils supporting its foundations. The soil-structure interaction modelling aspects for bridge design can be much more complex in seismic zones with soils that may lose strength and/or undergo deformations during a seismic event. Modelling the soils involves much greater uncertainties in geometry and material properties than structural modelling, and requires more than one iteration in most cases.

Interaction between the geotechnical Engineering Professional and the structural Engineering Professional is required to discuss, understand, and document the following:

- Design and analysis methods used
- Geotechnical input required for the structural analysis and vice versa
- Assumptions made in the geotechnical modelling and interpretation of geotechnical data
- Results of structural analysis
- Possible failure mechanisms
- Sensitivity of the geotechnical design input to the anticipated structure response

2.3.2 COORDINATION WITH OTHER ENGINEERING DISCIPLINES

Coordination with other engineering disciplines, such as highways and utilities, is also required, as this can drive the structural solution and impact the resultant seismic behaviour of the structure.

2.4 PEER REVIEW OF PERFORMANCE-BASED DESIGN

The commentary to the Code notes that a peer review is in addition to the formal independent structural review of any structural system, which is required by the *Act* and Bylaws.

The independent structural review would ideally be performed in stages commensurate with the design development and bridge complexity. A peer review may also be performed in stages, and significant benefit is likely to be achieved when it is initiated early in the design development process when the earthquake-resisting system (ERS) is being established. If a peer review is initiated late in the process, significant analysis and design effort may already have been invested, and there may be a tendency to avoid design revisions or changes. Peer review of novel or unusual systems is encouraged to provide confidence in the seismic performance of the proposed system and to achieve the intended level of safety and post-seismic return to service.

The framework and principles for independent structural review mandated by the Association (Engineers and Geoscientists BC 2018a) also provide a useful framework for peer reviews of seismic systems. These include design criteria documents, calculations performed, documentation of the review questions, answers and dispensation, record keeping, timing, and other aspects.

Aspects that may be covered by the peer review may include the following:

- Clarity and appropriateness of the ERS for the bridge and route importance
- Performance objectives specified; confirmation that damage targets supplementary to the basic descriptions in the Code are identified and appropriate
- Any intended exceptions to the Code, if applicable and where acceptable
- Confirmation that the owner understands the implications of any exceptions
- General arrangement of the ERS for loading in all directions
- Nature of devices or elements to be used; properties and limitations identified
- Nature and importance of plan and vertical irregularities in mass or stiffness along the bridge
- Whether soil-structure interaction is expected to be important, and the approach to modelling and performance assessments
- Seismic hazard and derivation of seismic inputs (including record selection, scaling, site response, level or nature of seismic input to the soil-structure interaction (SSI), or structural model)
- Identifying quantitative and qualitative performance measures forming the basis of the PBD approach

2.5 SPECIALTY SERVICES AND PRODUCTS

PBD as defined in the Code provides a design framework for the use of specialized and potential proprietary products as essential components of lateral load-resisting systems. Previous editions of the Code recognized only elastic force-based design and ductile substructures as lateral load-resisting systems. One motivation for introducing PBD was to facilitate the use of a broader range of structural systems or components to help achieve seismically resilient bridges.

2.5.1 TYPES OF SPECIALIZED SEISMIC DEVICES AND SYSTEMS

The use of seismic isolation and other seismic control devices are increasing in application within both bridges and buildings, to reduce or eliminate damage and structural repairs post-earthquakes. Properly engineered seismic devices have the potential to enhance the post-seismic performance and accelerate the return to service of highway bridges in BC and, as such, their use in appropriate applications in a coordinated and integrated fashion is encouraged by the Code.

Chapter 4 of the Code allows other systems to be used where their reliability and performance can be demonstrated. Following are examples of potential systems, which require appropriate engineering by the Engineer of Record or other qualified Engineering Professionals:

- Base-isolation bearings, including lead-rubber bearings, laminated elastomeric bearings, friction-pendulum bearings, and sliding systems, or combinations of these components
- Dampers (shock absorbers)
- Lock-up devices or shock transmission units (for seismic force transfer rather than energy dissipation)
- Yielding components such as ductile fuses (yielding flexural or shear plates)
- Ductile end diaphragms
- Buckling-restrained braces or other ductile or semi-ductile braces
- Rocking or stepping foundations or piers
- Fibre-reinforced polymers (strength or ductility enhancement)
- Proprietary couplers or connectors with or without post-tensioning, to allow non-linear behaviour such as opening/closing joints in precast or prefabricated components
- High-performance materials in components or splices, such as ultra-high performance fibre-reinforced concrete, shape-memory alloys, or other specialized materials

2.5.2 DESIGN AND PROCUREMENT CONSIDERATIONS

The use of specialized or proprietary products as part of the structural system introduces a number of issues and requirements that must be addressed by the structural Engineering Professional to achieve the intended outcome. These include the following:

- Consideration of specialized products and structural systems is ideally done during the feasibility or conceptual design stage. Throughout this process, the global ERS should normally remain under the control of the Engineering Professional, with appropriate inputs and technical support from the supplier of a proprietary system. The Engineering Professional would normally perform the dynamic analysis and global design, assure load path completeness, and determine overall effectiveness and structural performance. The design should allow for an acceptable range of demands and properties for the device as part of sensitivity studies. The Engineering Professional should normally not expect or direct the supplier to perform or repeat these functions.
- Suppliers are unlikely to be compensated during the feasibility and concept design phases;

however, experienced suppliers are typically willing and able to provide inputs to the design development process. The Engineering Professional should maintain commercial confidentiality, avoid conflicts of interest, and avoid creating expectations for favourable treatment. The Engineering Professional should limit the engineering support to reasonable levels during this phase, and communicate clearly what engineering support is anticipated.

- The procurement of proprietary products may comprise a distinct design-build package within a broader contractual context. As such, interface and other issues will be created that must be coordinated and confirmed by the Engineering Professional. Interfaces can include the following:
  - Engineering responsibility interfaces as part of the analyses or design.
  - Structural design interfaces, including connections, load path continuity, and interface component capacities.
  - Contractual aspects, including pre-qualification, procurement, testing, property verification, submissions, and reviews.
  - Bid-stage evaluation of the products, which may not be possible or practical unless written into the contract clearly. Prequalification is one option to facilitate this process.
- An important aspect of the structure's seismic performance is effectively delegated to a third party. The Engineering Professional should define the performance and design requirements, design responsibilities and divisions, submission and review processes, and structural interfaces. The Engineering Professional must be clear what is being delegated, and whether it is only the supply of a product or includes engineering support for a customized solution. In the former case, it may not be necessary to have the engineering of the component itself certified by an Engineering Professional registered in BC. For example, a proprietary base-isolation bearing or a patented damper or shock absorber using proprietary materials or elements will have been designed, validated, and tested to established standards to the satisfaction of the Engineering Professional. This acceptance should be based on adequate knowledge, understanding, track record, prequalification if appropriate, and capacity for supplier follow-through during construction. The Engineering Professional may rely on the design of the component, while confirming that an appropriate design and quality assurance process has been achieved.
- While the Engineering Professional cannot warranty the behaviour or durability of such products, he or she must recognize the importance of these systems to seismic performance and the need for them to function reliably for many years. In both respects, an appropriate level of technical and contractual diligence, professional responsibility and continuity through construction, and quality assurance must be exercised.
- The Engineering Professional should recognize that limited engineering is likely to be performed by contractors or suppliers during either the bid phase or the supply phase, unless clearly identified otherwise. Engineering support should normally be limited to confirming that the load path through the device has been provided for and interface aspects have been addressed, and that engineering property requirements have or

can be met, depending on the stage considered. Submission requirements by the supplier should be limited to documentation of test results and important properties, design of interface elements, shop drawings, and other items included in the contract requirements.

- In some cases, the proprietary products are covered and specified in established standards or peer reviewed guide specifications that mandate prototype and production testing. The viability and reliability of these products, including their track record, availability of engineering support by the originator of the device, prototype testing, production testing, and quality assurance during fabrication, are important considerations for the Engineering Professional. Independent structural review of any structural system is mandated by Engineers and Geoscientists BC, and peer review of novel or unusual systems should be considered carefully to provide a high degree of confidence in the seismic performance of the proposed system.
- Special seismic components may be designed by the Engineering Professional and may not be based on the same level of validation or testing as established and proven proprietary products. This aspect should be considered carefully as part of the independent structural review (and peer review, if carried out), and should also be considered in the construction-stage testing and quality assurance process mandated contractually. This may include yielding diaphragms or braces, rocking foundations, or engineered bearing systems intended to achieve seismic isolation.
- Analysis and design parameters that are used during the design must ultimately be demonstrated as being achieved during procurement and construction. Sensitivity analyses or bounds should be run as part of the design process to aid in this process, but this step should be included in the Engineering Professional's role with the owner. If this is not the case, then the risks to the owner and others should be discussed by the Engineering Professional.
- The Engineering Professional should also describe the procurement, supply, contractual procedures, and quality assurance processes to be followed and met. Where applicable, this would include the submission, review, and approval of engineered shop drawings. Where specialized or proprietary products are important elements of the seismic lateral load-resisting system, then continuity of involvement of the Engineering Professional through construction is strongly recommended. Where the Engineering Professional's role is contractually limited during construction, then these processes and the seismic performance should be communicated clearly and in writing to the owner or their authorized delegate. In all instances, the requirements for engineering signoff must meet the requirements under the *Act* and the roles and responsibilities must be clearly defined.

3.0 PROFESSIONAL REGISTRATION & EDUCATION, TRAINING, AND EXPERIENCE

3.1 PROFESSIONAL REGISTRATION

It is the responsibility of Engineering Professionals to determine whether they are qualified by training and/or experience to undertake and accept responsibility for carrying out performance-based seismic design of bridges in BC (Code of Ethics, Principle 2).

3.2 EDUCATION, TRAINING, AND EXPERIENCE

Performance-based seismic design of bridges, as described in these guidelines, requires certain minimum levels of education, training, and experience in many overlapping areas of engineering.

The Engineering Professional taking responsibility must adhere to the Association's Code of Ethics (to undertake and accept responsibility for professional assignments only when qualified by training or experience) and, therefore, must evaluate his/her qualifications and must possess the appropriate education, training, and experience to provide the services.

The level of education, training, and experience required of the Engineering Professional should be commensurate with the complexity of the project.

The academic training can be acquired by taking formal university or college courses or through continuing professional development. There may be some overlap in courses, and specific courses may not correlate to specific skill sets. An Engineering Professional should also remain current with evolving topics, through continuing professional development. Continuing professional development can include taking formal courses; attending conferences, workshops, seminars, and technical talks; reading technical publications; doing web research; and participating in field trips.

4.0 QUALITY MANAGEMENT IN PROFESSIONAL PRACTICE

Engineering Professionals must adhere to the applicable quality management requirements during all phases of the work, in accordance with the Association's Bylaws. It is also important to be aware if additional quality management requirements exist from the AHJ or through service contracts.

4.1 QUALITY MANAGEMENT REQUIREMENTS

To meet the intent of the quality requirements, Engineering Professionals must establish and maintain documented quality management processes for the following activities:

- The application of relevant professional practice guidelines
- Authentication of professional documents by the application of the professional seal
- Direct supervision of delegated professional engineering activities
- Retention of complete project documentation
- Regular documented checks using a written quality control process
- Documented field reviews of engineering designs/recommendations during implementation or construction
- Where applicable, documented independent review of structural designs prior to construction

4.1.1 PROFESSIONAL PRACTICE GUIDELINES

Pursuant to the *Act*, s.4(1) and Bylaw 11(e)(4)(h), Engineering Professionals are required to comply with the intent of any applicable professional practice guidelines related to the engineering work they undertake. One of the three objectives of the Association, as stated in the *Act* is "to establish, maintain, and enforce standards for the qualifications and practice of its members and licensees." Professional practice guidelines are one means by which the Association fulfills this obligation.

4.1.2 USE OF SEAL

According to the *Act*, s.20(9), Engineering Professionals are required to seal all professional engineering documents they prepare or deliver in their professional capacity to others who will rely on the information contained in the documents. This applies to documents that Engineering Professionals have personally prepared and those that others have prepared under their direct supervision. Failure to seal these engineering documents is a breach of the *Act*.

For more information, refer to *Quality Management Guidelines – Use of Seal* (Engineers and Geoscientists BC 2017).

4.1.3 DIRECT SUPERVISION

According to the *Act*, s.1(1) and 20(9), Engineering Professionals are required to directly supervise any engineering work they delegate. When working under the direct supervision of an Engineering Professional, unlicensed persons or non-members may assist in performing engineering work, but they may not assume responsibility for it. Engineering Professionals who are limited licensees may only directly supervise work within the scope of their license.

With regard to direct supervision, the Engineering Professional having overall responsibility should consider:

- the complexity of the project and the nature of the risks;
- which aspects of the work should be delegated;
- the training and experience of individuals to whom work is delegated; and
- the amount of instruction, supervision, and review required.

Careful consideration must be given to delegating fieldwork. Due to the complex nature of fieldwork, direct supervision is difficult and care must be taken so delegated work meets the standard expected by the Engineering Professional with overall responsibility. Typically, such direct supervision could take the form of specific instructions on what to observe, check, confirm, record, and report to the supervising Engineering Professional. Engineering Professionals with overall responsibility should exercise judgment when relying on delegated field observations, and they should conduct a sufficient level of review to have confidence in the quality and accuracy of those field observations.

For more information, refer to *Quality Management Guidelines – Direct Supervision* (Engineers and Geoscientists BC 2018c).

4.1.4 RETENTION OF PROJECT DOCUMENTATION

Pursuant to Bylaw 14(b)(1), Engineering Professionals are required to establish and maintain documented quality management processes that include retaining complete project documentation for a minimum of ten (10) years after the completion of a project or ten (10) years after engineering documentation is no longer in use.

These obligations apply to Engineering Professionals in all sectors. Project documentation in this context includes documentation related to any ongoing engineering work, which may not have a discrete start and end, and may occur in any sector.

Many Engineering Professionals are employed by organizations, which ultimately own the project documentation. Engineering Professionals are considered compliant with this quality management requirement when a complete set of project documentation is retained by the organizations that employ them using means and methods that are consistent with the Association's Bylaws and guidelines.

For more information, refer to *Quality Management Guidelines – Retention of Project Documentation* (Engineers and Geoscientists BC 2018d).

4.1.5 DOCUMENTED CHECKS OF ENGINEERING WORK

In accordance with Bylaw 14(b)(2), Engineering Professionals are required to undergo documented quality checking and review of engineering work appropriate to the risk associated with that work.

Regardless of sector, Engineering Professionals must meet this quality management requirement. In this context, 'checking' means all professional deliverables must undergo a documented checking and review process before being finalized and delivered. This process would normally involve an internal review by another Engineering Professional within the same firm. Where an appropriate internal reviewer is not available, an external reviewer (i.e., one outside the organization) must be engaged. Where an internal or external review has been carried out, this must be documented.

Engineering Professionals are responsible for ensuring that checks being performed are appropriate to the level of risk. Considerations for the level of review should include the type of document and the complexity of the subject matter and the underlying conditions; quality and reliability of background information, field data, and elements at risk; and the Engineering Professional's training and experience.

For more information, refer to *Quality Management Guidelines – Documented Checks of Engineering and Geoscience Work* (Engineers and Geoscientists BC 2018b).

4.1.6 DOCUMENTED FIELD REVIEWS DURING IMPLEMENTATION OR CONSTRUCTION

In accordance with Bylaw 14(b)(3), field reviews are reviews conducted at the site of the construction or implementation of the engineering work. They are carried out by an Engineering Professional or a subordinate acting under the Engineering Professional's direct supervision. Field reviews enable the Engineering Professional to ascertain whether the construction or implementation of the work substantially complies in all material respects with the engineering concepts or intent reflected in the engineering documents prepared for the work.

Engineering Professionals are required to establish and maintain documented quality management processes, which include carrying out documented field reviews of their domestic projects or work during implementation or construction. Domestic works or projects include those located in Canada and for which an Engineering Professional meets the registration requirements for the engineering regulatory body that has jurisdiction.

For more information, refer to *Quality Management Guidelines – Documented Field Reviews during Implementation or Construction* (Engineers and Geoscientists BC 2018e).

4.1.7 DOCUMENTED INDEPENDENT REVIEW OF STRUCTURAL DESIGNS

Bylaw 14(b)(4) refers to an independent review in the context of structural engineering. An independent review is a documented evaluation of the structural design concept, details, and documentation based on a qualitative examination of the substantially complete structural design documents, which occurs before those documents are issued for construction. It is carried out by an experienced Engineering Professional qualified to practice structural engineering, who has not been involved in preparing the design.

For more information, refer to *Quality Management Guidelines – Documented Independent Review of Structural Designs* (Engineers and Geoscientists BC 2018a).

5.0 GUIDELINES FOR PROFESSIONAL PRACTICE

This section provides guidance for professional practice related to performance-based seismic design of bridges in BC. General underlying principles, importance categories, geotechnical and structural analysis, design requirements, assurance statement, and other topics are discussed.

In addition, **Appendix B: Case Studies** provides examples of the application of performance-based seismic design for structures such as a reinforced concrete column bridge, an extended concrete pile bent bridge, and a system comprising tubular eccentrically braced frames.

As noted previously, the guidance in this document will support the consistent and appropriate application of the performance-based seismic bridge design requirements in the CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014).

This document also references the BC Ministry of Transportation and Infrastructure (BC MoTI) Supplement to CAN/CSA-S6-14 (the Supplement) (BC MoTI 2016), which reflects BC MoTI-specific requirements.

It should be noted that other jurisdictions and/or owners are not obligated to follow the stipulations in the Supplement.

5.1 GENERAL PRINCIPLES

Gravity or vertical load design of a structure is primarily strength-based; that is, load (force) capacity must exceed demands. The demands are primarily based on linear analysis, while the capacity is based on material strains in the non-linear range. Gravity design generally does not account for cyclic loading effects (with the exception of fatigue), while load redistribution is often not relied upon. Over-prediction of design strengths for gravity load-resisting systems can result in catastrophic outcomes.

On the other hand, seismic loads are transient and primarily lateral. The effects of the cyclic nature of loading on strength, stiffness, and ductility have to be accounted for using best-estimate material properties and section capacities. For seismic design, under-prediction of design strength can lead to unintended brittle failures where ductile behaviour is required, and should therefore be avoided at all cost. This is further explained in **Section 5.8.5 Capacity-Protected Elements**.

Load redistribution is also relied upon in ductile systems, and accurate assessments of design strengths are important for capturing this phenomenon. In general, sound seismic design should incorporate best estimates of material properties, section capacities, and appropriate non-linear analysis techniques.

5.1.1 EARTHQUAKE RESISTANCE AND STRUCTURAL/GEOTECHNICAL FUSES

It can be uneconomical to design and build structures that resist the low probability, long return period seismic events elastically. The principles of ductility and capacity design are intended to make specified parts of the structure purposefully weaker and able to undergo post-yield displacements without excessive damage or collapse.

Structural or geotechnical fuses ensure the forces to be resisted by the earthquake-resisting system (ERS) elements are controlled and restricted to a pre-determined level. The fuse elements are detailed properly to ensure ductility and energy dissipation via stable hysteretic behaviour. The system therefore resists the seismic displacement demands in an inelastic manner, while operating at a known maximum force level. Following are a few examples:

- Plastic hinges in bridge bent columns
- Link beams in eccentrically braced frames
- Buckling resistant braces
- Base-isolation bearings
- Rocking foundations, soil yielding, and energy dissipation behind structures such as abutment walls

All elements except the fuses are required to resist maximum seismic forces, corresponding to fuse over-strength demands in an essentially elastic manner.

5.1.2 ALLOWABLE SEISMIC DESIGN APPROACHES

The Code allows performance-based design (PBD) for all bridge structures. According to the Code, force-based design (FBD) may still be used, depending on the structure's seismic performance category (SPC) and structural regularity.

Structures categorized as irregular major route bridges and as regular and irregular lifeline bridges for SPC 2 and SPC 3 must employ the PBD approach. Similarly, structures categorized as irregular other bridges in SPC 3 must also use the PBD approach. The AHJ may also mandate the use of a PBD approach for a structure categorized as a regular major route bridge in SPC 3. The remaining cases can be designed using the FBD approach.

It should be noted that in the Supplement, Table 4.10, row 2, the SPC for lifeline bridges has been changed from 2 to 1. However, lifeline bridges in SPC 1 have to adopt structural detailing of elements per SPC 2 as a minimum.

5.1.3 ELASTIC VERSUS INELASTIC DISPLACEMENTS FOR SEISMIC DESIGN

Structural displacements are of critical importance for the seismic design of structures. Both linear and non-linear analysis can be used for calculating design displacements.

A generally employed rule for calculating displacements for a structure responding in the non-linear range is the equal-displacement principle. The equal-displacement principle posits that the seismic displacement of a linear elastic system is equal to the seismic displacement of an inelastic system with the same initial, elastic period.

For the FBD methodology, this leads to the force-reduction factor, R, for flexural design, as shown in **Figure 1**.

For the FBD methodology,

- design is carried out for a lower force level, F_{y2}, instead of the elastic force demand, F_{y1};
- the designer details the system to respond in the non-linear range, achieved by creating 'ductility' within the lateral resisting members and joints;
- the provided ductility must exceed the force-reduction factor, R, which is provided for

implicitly by limiting the maximum value for R factors for various systems; and

- ductility is provided based on prescriptive, code-based detailing but not checked explicitly as part of the design process.

For the PBD methodology, the equal-displacement principle can provide the target displacement for damage quantification and service level verification. It is, however, recognized that the equal-displacement principle holds true for a restricted period range and linear analysis-based models underestimate displacements for short-period structures, as described in the ATC 32 report (Applied Technology Council 1996). Some codes, such as ATC 32, provide amplification factors to calculate inelastic displacement values from the linear elastic displacement values. It should be noted that no correction for elastic displacement values is needed for designs using the Code. Refined estimates of inelastic displacements can be obtained using non-linear time history analysis for PBD.

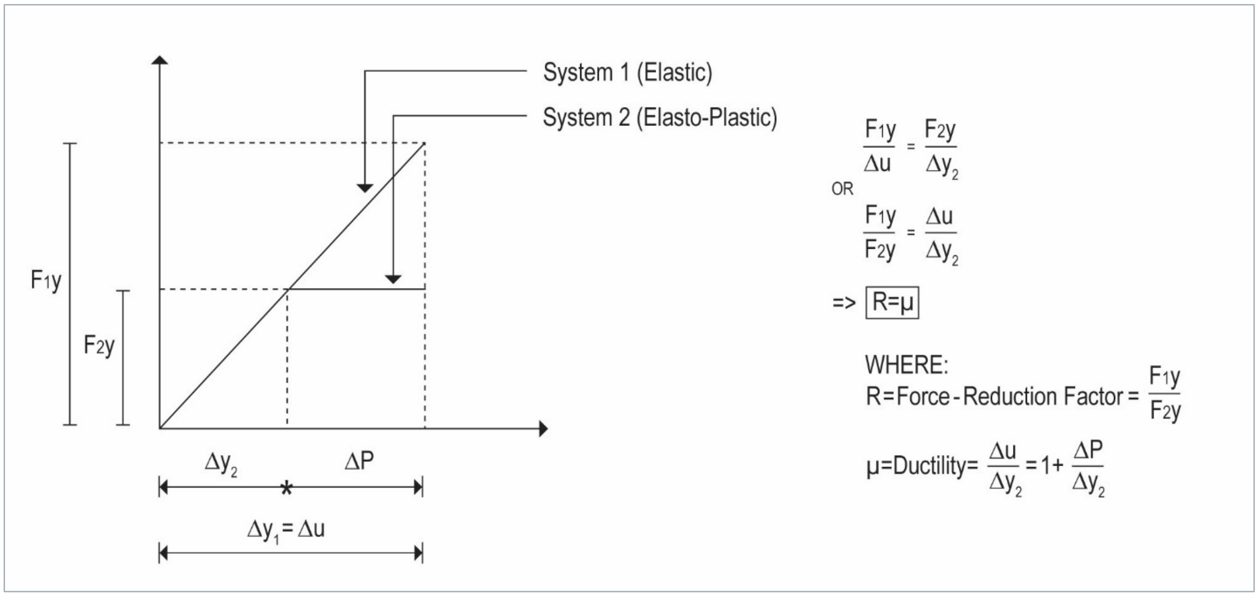


Figure 1: Equal displacement principle and force-reduction factor, R

5.1.4 PRINCIPAL LIMITATIONS OF THE FORCE-BASED DESIGN APPROACH

The FBD approach can be used for many bridges and, coupled with capacity design principles, yields appropriate design solutions. However, the Engineering Professional should be aware of its limitations, use it judiciously, and understand when a PBD approach will lead to a better design through a rational understanding of the structural behaviour.

Following are some limitations of the FBD approach:

- The FBD approach is simple and is consistent with the application of other vertical and lateral loads such as self-weight, live loads, wind, and braking. However, the earthquake load primarily imparts a lateral displacement demand on a structure. Complications and inconsistencies arise as a result of quantifying this phenomenon in force-based terms.

- FBD ignores the interdependency of strength and stiffness.
- Ductility demand and the Force Reduction Factor (R-factor): There is no consensus amongst codes internationally, due to variability in defining yield and ultimate displacements. Therefore, the resulting R factor values for similar ERSs can vary significantly from code to code.
- System ductility is not considered. For example, for an ERS comprising a bent with highly variable column/extended pile lengths, the use of a constant R-factor for the shortest and tallest column/extended pile is inaccurate. The ductility imposed on the various columns/extended piles in such a scenario will be highly variable; this is because the shorter members will start to behave plastically earlier and will need to resist larger ductility demands compared to the taller members.
- Post-earthquake performance cannot be reliably quantified based on the R-factor approach.
- FBD is applicable to strength-based and ductility-based design only. It has limited applicability to many other viable seismic load-resisting systems.

5.1.5 PERFORMANCE-BASED DESIGN METHODOLOGY ADVANTAGES

In the past, the main design goal for various codes has been life safety with emphasis on collapse prevention, while the design basis has predominantly been force and strength criteria. However, there has been a shift from 'strength' to 'performance' in design and a recognition that the two are not the same. There is now consensus that an increase in strength does not necessarily mean enhanced safety, nor does it imply less damage. In fact, a large increase in strength can be detrimental, and strength without ductility is futile in a seismic environment (Priestley et al. 1996).

Owners increasingly expect their structures to be serviceable after small and moderate earthquakes. In some instances, only repairable damage may be allowed, even in case of large earthquakes. In the case of bridges, a return to traffic may be an expectation and requirement, but this cannot be demonstrated using implicit FBD methods. In recent earthquakes (for example, Christchurch, New Zealand, 2011) there was a clear disconnect between the owner's and society's expectations and the seismic design assumptions used by designers. There has since been a push to better understand and demonstrate structural performance explicitly. PBD is the tool that allows us to articulate, understand, demonstrate, and incorporate such requirements into the seismic design of bridges.

5.2 PERFORMANCE-BASED SEISMIC DESIGN PROCESS

5.2.1 FRAMEWORK FOR THE PBD PROCESS

The general framework of the PBD process can be summarized as follows:

1. Define various performance levels and corresponding levels of seismicity (design loads).
2. Correlate performance to demand-capacity measures. Global displacement, hinge rotations, and material strains are examples of such measures.
3. Determine deformation and force capacities.
4. Determine deformation and force demands.
5. Ensure that capacity is greater than demand and various performance requirements have been met.

6. Carry out capacity design for all locations other than fuses, and for brittle failure modes.

The flow charts in **Figure 2** summarize the basic steps that may be used for PBD of a ductile ERS.

Examples of other performance criteria relate to items such as connections, restrainers, permanent offsets and foundation misalignments, pounding damage, bearings and joints, post-earthquake dead and live load capacity, and aftershock performance. The designer should consult the Code and/or project-specific criteria to ensure all performance requirements are adequately met.

5.2.2 ROADMAP FOR THE PBD PROCESS

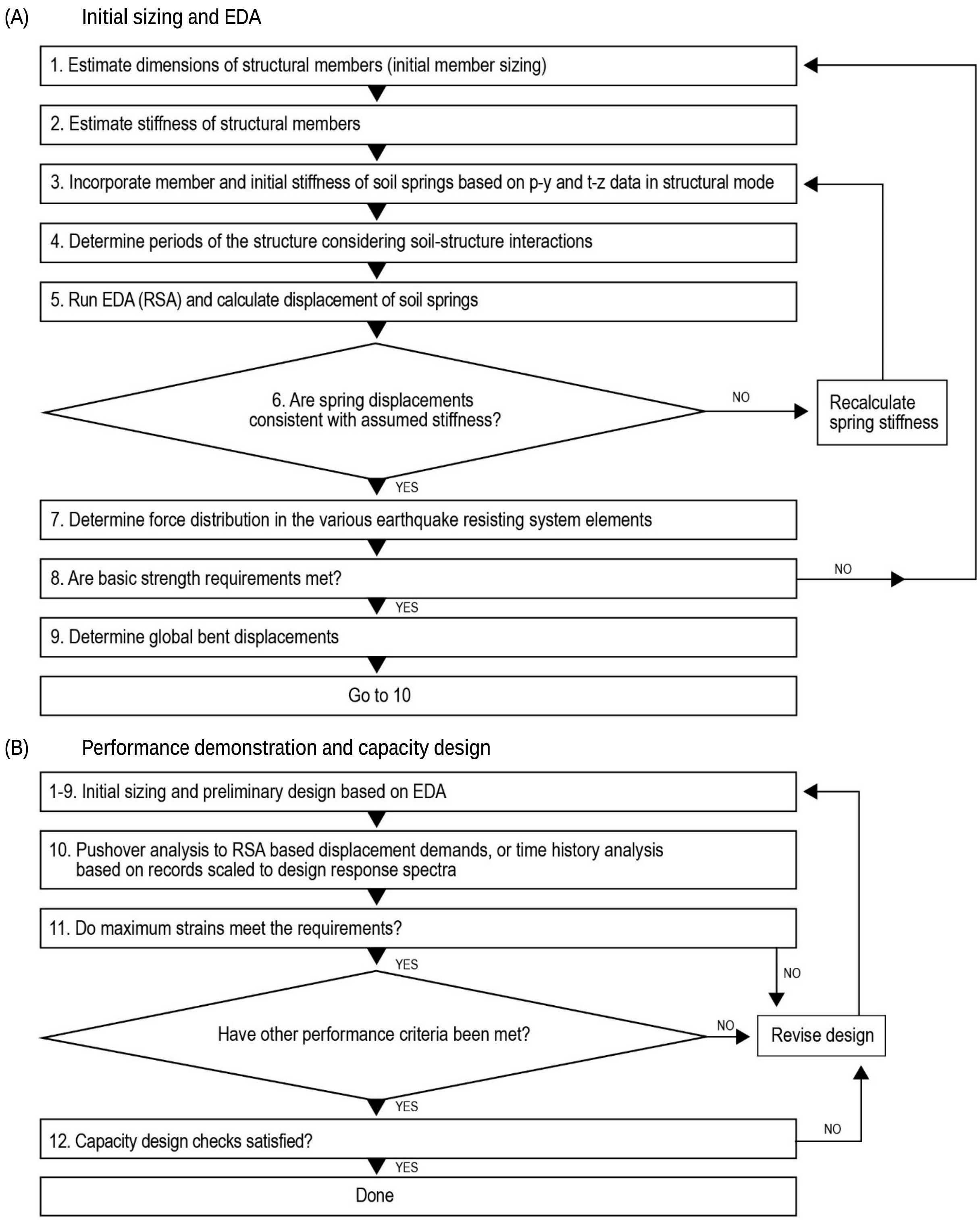
To gain the potential benefits of PBD, the process that owners and Engineering Professionals use to define objectives, requirements, and expected outcomes is an important contributor to success. Some owners are knowledgeable about or have guiding principles for the seismic design of bridges, while others need more support from Engineering Professionals.

One fundamental aspect of PBD is that performance objectives should be appropriate for the context of the bridge crossing and the owner's communication network and must be articulated clearly. Where guidance is needed, then informed discussions between the Engineering Professional and the owner should occur early enough in the process to factor requirements into the work plan and fee to ensure expectations can be met. The roadmap or framework adopted by a given owner will likely evolve as all parties gain experience from discussions and implementation in projects.

A framework for appropriate implementation of PBD in bridge designs within a given owner's network may include the following considerations:

- Understanding the role and importance of a given bridge within a municipal or regional transportation network. This includes identifying its intended function as part of a local or regional disaster-response and economic-recovery network. This allows an appropriate importance category to be assigned to each bridge, i.e., lifeline, major route, or other, as defined in the Code and the commentary Clauses 4.4.2 and C4.4.2, respectively. The bridge importance, combined with the seismic hazard at the site, defines an SPC of 1, 2, or 3, which affects the required design approach and seismic performance objectives for PBD.

  As such, the designation of the bridge and route will reflect the risk level acceptable to the owner, will have economic consequences for the construction or retrofit cost of the bridge, and can allow funding to be directed to well-considered priority crossings. This does not imply that PBD for important bridges will be more expensive than PBD for bridges of lesser seismic importance or for those designed with force-based approaches. It is simply more likely that the discussions and design approaches that are intrinsic to designing to performance will lead to overall cost savings for some bridges.



Note: EDA = elastic dynamic analysis; RSA = response spectrum analysis

Figure 2: Performance-based design process flowcharts for: (A) initial sizing and EDA; and (B) performance demonstration and capacity design

- Understanding the seismic performance objectives specified in the Code, and deciding whether minimum Code requirements are sufficient or appropriate for the crossing. This provides a useful check on the designated route and bridge classification. For example, if the specified damage and return-to-service objectives appear too high because of route redundancy or other reasons, then the bridge importance can be reviewed. Once the bridge importance and seismic hazard at the site are confirmed, the design requirements for new bridges can be set within the Code. For existing bridges, considerable latitude is provided to owners to consider the bridge's importance, role, age, and condition. Post-seismic performance expectations for bridges include considerations such as immediate or partial use, return-to-service times, repair expectations, risks of not meeting performance criteria, and aftershock performance expectations. These discussions may influence the bridge arrangement and seismic systems.
- As part of the above consideration, aspects of the discussion may include the following:
  - The importance of return-to-service and expected repair or replacement timelines.
  - Whether seismic isolation or other low-damage systems are desirable, and what risk, cost, and performance trade-offs the owner is prepared to make.
  - Whether any aspect of the design, while performing well seismically, results in compromised access for inspection or repair. For example, extended piles can be robust and perform well seismically but may result in in-ground plastic hinges, which may be difficult to inspect or repair.
  - The overall bridge form and its past track record of function, durability, and seismic performance. For example, integral or semi-integral abutment bridges or continuous superstructure bridges can be low-maintenance and perform well seismically. Some bridge attributes may be important for aspects other than seismicity but should also be considered for their seismic implications.
  - Importance attached to capacity design. The Code does not mandate capacity design; however, it can provide far more seismic resiliency than a nominally elastic system, which may fail at demands only slightly larger than those adopted for design.
  - Implications of soil conditions and requirements. For example, lateral spreading, cyclic softening, and pile/soil performance measures.
- The initial setting of the bridge arrangement and substructure proportioning. This is far less constrained in PBD than in FBD; more systems and proportioning methods are available.
- The Engineering Professional may use any proportioning method preferred by the design, including using R-factors for approximations.
- Modelling, analysis, design, and detailing should be discussed between the owner and the Engineering Professional.
- Some prescriptive detailing requirements remain for substructures; for example, whether they are expected to experience plastic behaviour or not.
- Some consideration should be given to return-to-service importance in an aftershock environment. The Code requires an assessment of aftershock capacity, but the state of practice for this task is still evolving. Some calibration studies for

bridges on firm ground show that well-designed bridges using ductility should have considerable resilience beyond minimum design targets, while some bridges located on soft soils may not. Base-isolated or other low-damage systems have intrinsic advantages for resisting aftershocks.

5.3 BRIDGE IMPORTANCE CATEGORIES AND PERFORMANCE LEVELS

The Code defines three different importance categories for bridges for defining different seismic performance levels: lifeline, major route, and other. The Code provides guidance about each importance category.

The service and damage levels associated with each category have been defined in the Code, and in some cases have been modified in the Supplement.

For owners, it is important to understand what the service levels mean. Following is a brief description and interpretation of the Code definitions of service levels:

- **Immediate** – The bridge shall be fully serviceable for normal traffic, and repair work does not cause any service disruption.
- **Limited** – The bridge shall be usable for emergency traffic and be repairable without requiring bridge closure. At least 50% of the lanes, but not less than one lane, shall remain operational. If damaged, normal service shall be restored within 1 month. The time limit is to be from the start of repair, not from the time of the event.
- **Service disruption** – The bridge shall be usable for restricted emergency traffic after inspection. The bridge shall be repairable. Repairs to restore the bridge to full service might require bridge closure. Such allowed restrictions may include specification of useable lanes, weight restrictions, vehicle clearances, rerouting around ramps, or speed restrictions.
- **Life safety** – The structure shall not collapse and it shall be possible to evacuate the bridge safely. While it may not be possible for users to drive off the structure, they must be able to walk off safely.

Damage levels provide more specific descriptions of the damage and permanent deformations corresponding to the service levels. These damage levels, which have been modified in the Supplement, provide guidance to designers and include strain limits for various materials/components. Currently, the Code damage requirement of 'none' carries no description in the document and may be intended to represent no damage in excess of normal operational effects consistent with the bridge age and usage. The lowest damage level included in the Supplement is 'minimal.'

In the early stages of project development, it is important for the owner and owner's engineer or Engineer of Record to discuss and understand the ramification of the importance category designation. In some cases, depending on seismic zone, soil conditions, and the size/height of the structure, there can be considerable additional costs if higher performance categories are chosen.

The category of lifeline bridge is intended for large and/or complex bridges such as the new Champlain Bridge in Montreal, Quebec and the Port Mann Bridge in Coquitlam and Surrey, BC. The owner may also designate a bridge as a lifeline bridge if it provides sole access to critical infrastructure. Most bridges will not fall into this category. The decision to define a bridge as a major route bridge or other bridge should take into account the regional emergency response plan. If the bridge is on a route that will be required as part of the disaster response plan, then consideration should be given to using the major route designation. In this category, the bridge should be able to carry restricted emergency traffic after a 2,475-year earthquake, although it may be significantly damaged.

If the bridge is defined as other, unless the owner defines the optional performance levels for 475-year and 975-year earthquakes, there is only one performance level defined corresponding to the 2,475-year earthquake. The bridge is not expected to be able to carry emergency vehicles after that event. The owner may wish to consider the optional performance levels.

5.4 SEISMIC HAZARD ASSESSMENTS

5.4.1 DETERMINING REGIONAL SEISMICITY

The objective of a seismic hazard assessment is to establish ground motion parameters applicable for seismic design. The design ground motions should represent the plate tectonic setup of the region, considering the regional faults identified in geologic and seismic hazard maps and evidence of potential fault movements within a radius of approximately 500 km from the bridge site.

Seismic hazard assessment in Canada continues to be developed in support of the National Building Code of Canada. Improvements to the seismic hazard assessment process incorporate ongoing refinements of our understanding of the seismic source zones, ground motion prediction equations, and modelling uncertainties.

Seismicity in southwestern BC results from the offshore subducting of the Juan de Fuca plate beneath the North American plate. This unique plate tectonic environment results in the following three different earthquake types for this region, each with its own characteristics such as intensity of ground shaking, magnitude, distance to fault rupture, and duration of shaking:

1. Shallow crustal earthquakes that occur in the North American plate.
2. Deep in-slab earthquakes that occur in the subducting Juan de Fuca plate.
3. Interface subduction earthquakes that occur at the interface of the North American and Juan de Fuca plates.

Seismicity in northwestern BC results from the strike-slip reverse faulting boundary between the Pacific and North American plates. The Queen Charlotte Fault marks the major transpressive boundary (strike-slip and reverse faulting) between the Pacific and North American plates from northern Vancouver Island to northern BC. The Queen Charlotte Fault extends more than 500 km from a southerly triple junction with the Explorer, North American, and Pacific plates, to the southern extent of the Denali and Fairweather faults of Alaska. In eastern BC, away from the offshore plate tectonic boundaries, the historical seismicity is low. In eastern and northwestern BC, shallow crustal earthquakes control site seismicity.

For a given site, the intensity and duration of shaking are dependent on the earthquake magnitude (which is a measure of how large the fault rupture is), the distance from the rupture zone to the site, and the fault rupture mechanism. The duration of strong shaking, which indirectly represents the number of cycles of loading, is correlated to the magnitude of the earthquake. The moment magnitude scale (denoted by M_w), which measures the total energy released by an earthquake, is commonly used for engineering applications. Incorporating the effects of both the intensity and duration of shaking is important when carrying out geotechnical analysis of foundation soils for PBD. Earthquakes of magnitude less than or equal to 5, regardless of the distance to the rupture zone, are not expected to cause damage in well-built structures.

The release of energy during a mainshock and the associated re-adjustment of the stress-fields usually trigger aftershocks near the mainshock, mostly within the same rupture area. Typically, the largest aftershock is one magnitude unit smaller than the mainshock. The expected aftershock patterns vary depending on the different types of earthquakes that occur in southwestern BC (interface subduction, shallow crustal, and deep in-slab). Historically, deep in-slab earthquakes (e.g., 1949 Olympia [M_w 7], 1965 Seattle [M_w 6.5], 2001 Nisqually [M_w 6.9]) have produced few or no aftershocks, and shallow crustal earthquakes (1918 Vancouver Island [M_w 7], 1997 Georgia Strait [M_w 4.7]) have produced dozens to hundreds of aftershocks. Large interface subduction earthquakes are expected to produce thousands of aftershocks continuing over a long time (months to years). Aftershocks, although smaller (by definition) than the mainshock, may generate stronger shaking in some locations if they are much closer to the site than the mainshock.

Predicting aftershocks is not currently possible. They can occur days, weeks, months, or years later. Recent examples include earthquakes in Sumatra (2004), Chile (2010), and New Zealand (2011). The probability of aftershock events may be predicted from the mainshock magnitude using regional statistical models that are adjusted as the aftershock sequence evolves. The intensity and duration of shaking of the aftershocks can be predicted from standard ground motion models, and the typical pattern is that aftershock magnitude and frequency decreases with time (J. Adam and J. Cassidy, email message to U. Atukorala, April 28, 2017; Seemann et al 2008).

5.4.2 EARTHQUAKE GROUND MOTION PARAMETERS

Natural Resources Canada (NRC) provides seismic hazard maps and an online seismic hazard calculator for computing ground motion parameters for a given site based on its latitude and longitude (NRC 2016). The online NRC hazard calculator provides ground motion parameters for firm ground, or a reference ground condition, for four different return periods varying from 100 years to 2,475 years. The ground motion parameters are provided in the form of uniform hazard response spectra for horizontal shaking. The de-aggregation of seismic hazard data that provide information such as contribution of the magnitude-distance pairs at varying periods can also be obtained upon request.

For sites located in southwestern BC, the seismic design should incorporate the effects of crustal, in-slab, and interface earthquakes. When considering scenario earthquakes, the Code commentary recommends a minimum of 5 records for each scenario or period range; the total number of records covering all scenarios or period ranges should be no less than 11. No specific breakdown of records to be used for crustal, inslab, and interface earthquakes is otherwise specified. Specific earthquake records to be used in the design will depend on the owner's requirements and the SPC of the bridge. Consideration should be given to developing scenario spectra that apply to each of these types of earthquakes, and selecting ground motion time-histories that closely resemble the spectral shapes of real earthquakes is important in PBD (rather than using synthetic or semi-synthetic records matched to uniform hazard response spectra at all periods).

Vertical ground motions are important for designing anchors or hold-down devices for a bridge. NRC's online hazard calculator does not provide vertical ground motion parameters. The vertical hazard spectra are generally established by the seismologists using the vertical to horizontal spectral acceleration ratios proposed by Gulerce and Abrahamson (2011) that vary with period, earthquake magnitude, and distance to fault rupture. Alternatively, the vertical spectral coordinates can be taken as two-thirds of the corresponding horizontal spectral coordinates when spectral analysis methods are used for design.

Non-linear analysis of bridge-foundation systems require ground motions as input. Suitable ground motions should be selected based on the tectonic regime, earthquake magnitudes, and rupture distances that control the seismic hazard, and on the local geotechnical conditions at the site. The mean response spectra of ground motions should closely represent the target uniform hazard response spectra over a period range of 0.15 to 2.0 times the first-mode period of the structural system designed.

Recorded ground motions are generally preferred. However, modified ground motions or synthetic ground motions may be used as an alternative if appropriate records are not available.

5.5 GEOTECHNICAL INVESTIGATION, SOIL LIQUEFACTION, AND MITIGATION OF LIQUEFACTION

5.5.1 GEOTECHNICAL INVESTIGATION

The objective of the geotechnical investigation is to collect subsurface data to develop a geotechnical model for the bridge site. Establishing the zones of potentially liquefiable soils and the likely lateral spreading and settlements, including load-carrying capacity of foundations both during and after earthquake shaking, are important in PBD of bridges.

The geotechnical investigation should be of sufficient lateral extent and depth to collect data in order to develop a geological/geotechnical model of the subsurface conditions underlying the bridge site to achieve the following, as a minimum:

- Determine the response of the foundation soils to design seismic loading
- Determine the response of both existing and new slopes and embankments to design seismic loading
- Complete the foundation design
- Define realistic baseline assumptions for construction

The data, as a minimum, should consist of in-situ measurements to define soil types, soil stratigraphy, in-situ relative density and consistency of soils, depth to the permanent water table, and site topography. The spatial variations in soil conditions should also be established to the extent practicable.

A site investigation is typically one of the first engineering activities that will be completed for the project. As a result, information such as the actual bridge alignment, type of bridge structure, configuration of foundations and embankments, and tolerable foundation settlements and displacements are commonly not available at the outset of the project. Consequently, it is often beneficial to carry out the geotechnical investigation in phases, starting with broader objectives and progressing through increasingly narrower and more focussed data collection phases.

5.5.1.1 Geology and Available Data

The geotechnical investigation should focus on both the bridge site and the surrounding region. Prior to executing the geotechnical investigation, desktop studies should be carried out to identify the anticipated type, depth, and consistency of subsurface soils at the site. This effort should include referring to available surficial geological maps, geotechnical reports from past investigations near the project site, LiDAR survey data, and aerial photographs for the area. Liquefaction hazard, flood hazard, and landslide hazard maps are available for some areas and can be useful sources of information.

The aim of the desktop studies should be to (1) identify areas that are underlain by recent sediments comprising coarse-grained soils (sand and gravel with cobbles and boulders), sand and silt, and normally to over-consolidated deposits of fine-grained soils; (2) locate the regional water table; and (3) ascertain areas of potential fill materials. This information is useful for planning the investigation, including the type of drilling equipment required, number and depth of test holes, and any specific conditions that require special attention such as artesian conditions.

5.5.1.2 Geotechnical Site Investigation Techniques

It is important to carry out the site investigation using established site investigation techniques and tools. This is required for the assessment of liquefaction of foundation soils using empirical liquefaction resistance charts developed from select site investigation techniques. Accepted practice is to use the standard penetration test (SPT), cone penetration test (CPT), Becker penetration test (BPT), or a combination of these test methods. Non-intrusive geophysical testing such as in-situ measurement of shear wave velocity of soils with depth can be used as a screening method to delineate soil deposits that are susceptible to liquefaction.

In the case of soft silty clays and low plastic silts, although these types of soils may not liquefy in the traditional sense, earthquake shaking can result in significant softening and deformations. The response of silty clays and low plastic silts to seismic loading is best evaluated using undisturbed sampling and laboratory testing supported by in-situ vane shear testing and/or the piezocone penetration test (CPTu).

5.5.2 SOIL LIQUEFACTION

Liquefaction is the process by which sediments located below the water table temporarily lose strength as a result of the application of earthquake-induced cyclic shear stresses and behave as a viscous liquid rather than a soil. The types of sediments that are most susceptible to liquefaction are mixtures of non-plastic silts, sands, and gravels.

The liquefaction phenomenon is complex, and laboratory testing and analytical modelling have not evolved sufficiently to an extent where they can be applied with confidence. As a result, current practice relies heavily on empirical procedures, which are based primarily on interpretation of case histories and past performance of constructed works (Finn et al. 2010).

5.5.2.1 Impact of Liquefaction on Foundations

Liquefaction of soils leads to loss of bearing resistance of foundations, slope and/or ground instability, lateral spreading of ground, settlement of ground, and increased lateral loads on abutments and retaining walls. These effects could result in loss of functionality of bridges due to foundation movements or failure.

When slope and/or ground instability at and in the vicinity of the bridge is predicted as a result of soil liquefaction, the effects on bridge foundations should be assessed to confirm that the bridge structure meets the minimum performance levels. An assessment of the impact of both inertial loading and kinematic loading on foundations due to lateral spreading should be completed. If the zone of instability is shallow and the bridge is supported on pile foundations that penetrate into deeper non-liquefiable soils for vertical and lateral support, the inertial and kinematic loading effects may be accommodated while meeting the minimum performance levels. If the minimum performance levels cannot be achieved, liquefaction remediation measures are required to improve the seismic stability and lateral spreading of site soils.

5.5.2.2 Characteristic Penetration Resistance Values for Performance-Based Seismic Design

Field measurement of penetration resistance vary both with depth and horizontal distance. Characteristic soil penetration values should be established for engineering analysis. Reasonable to conservative estimates of soil liquefaction and deterministic estimates of lateral spreading displacements may be established using the 33rd percentile penetration resistance profiles for the soil units (Montgomery and Boulanger 2017), when large variations in measurements occur. The design should be checked against the 50th percentile penetration resistance profiles for soil units as part of sensitivity studies.

When feasible, the variability of the characteristic penetration resistance values of soils along the bridge alignment should be included in the analysis of lateral displacements. These analyses are time-consuming and require the involvement of trained specialists and computer software that meets current practice requirements. An alternative approach may be to consider uniform variations in the characteristic penetration resistance values in between test holes and soil units.

5.5.2.3 Mitigation of Liquefaction

A number of procedures are available for mitigating soil liquefaction at bridge sites. The methods used for a particular bridge site depend on the types of soils underlying the site, the depth of treatment required, the proximity of the site to other structures, whether mitigation measures are being implemented for existing or new foundations, and cost considerations for the project.

In the design of mitigation measures, one method can have more than one function, and several methods can be combined. Mitigation should provide suitable protection against potential lateral spreading or flow failures, bearing capacity failure, and foundation settlement.

The different mitigation methods are broadly classified into the following different categories based on the function to be achieved (Task Force Report 2007):

- Densification
- Drainage
- Dewatering
- Mixing and solidification

- Reinforcement and containment
- Removal and replacement

Some commonly used techniques for densification include vibro-compaction, vibro-replacement stone columns, compaction grouting, and dynamic compaction. Commonly used techniques for mixing and solidification include jet grouting and deep soil mixing. Reinforcing soils by installing displacement piles, such as timber or concrete piles, is another technique often used by practitioners. Removing and replacing poorly performing soils to mitigate soil liquefaction is only practical when the depths of liquefiable layers are shallow.

Mitigation of soil liquefaction using in-situ treatment is a specialized area of expertise, and many factors (including those noted previously) should be considered before a particular technique, or a combination of techniques, is selected. Input from specialty contractors should be solicited to assess the advantages and disadvantages of different methods applicable to a particular bridge site, when a requirement for mitigation of soil liquefaction and its effects has been confirmed.

Implementing ground improvement measures can result in measureable lateral and vertical displacements over lateral distances of up to 30 m. The displacements should be estimated and any adverse effects on existing and nearby structures should be assessed as part of the work.

5.6 SOIL-STRUCTURE INTERACTION

The objective of a soil-structure interaction analysis is to incorporate the soil and foundation flexibility in seismic design. The soil-structure interaction response of a bridge pier can be assessed using either uncoupled or coupled analysis methods. In an uncoupled analysis, the soil, foundation, and superstructure are modelled separately. In a coupled analysis, the soil, foundation, and superstructure are modelled together.

Including soil-structure interaction in an analysis generally has the overall effect of increasing the fundamental period of vibration and allowing for effects of radiation and material damping that can lead to reduced seismic demand on structure elements when compared to a fixed-base system. However, in some cases, depending on the period shift and input energy, displacement demands on the structure can be increased.

Computational models incorporating a soil-structure system may be used for design in most cases. These models use a single or a range of soil stiffness and damping values for soils, foundation elements, and superstructure. The superstructure should be included when the inertial loads of the superstructure are significant.

It is common to use the Winkler spring computational model in the structural analysis, where the non-linear soil-foundation interface response is represented by linear or non-linear foundation compliance springs. The compliance springs should be derived using location-specific soil stratigraphy and properties; hence, they vary along the bridge.

Coupled analysis is complex and is not required in all cases. Because this type of analysis is time-consuming, requires engineering judgment, and depends on considerable interaction between the geotechnical and structural engineers, it should be pursued only by Engineering Professionals who are experienced in conducting these types of analyses.

5.6.1 ANALYSIS REQUIREMENTS

Geotechnical analyses should incorporate the non-linear and inelastic behavior of overburden soils for the three levels of ground shaking described in the Code. Soil behaviour plays an important role in determining both the seismic demand on bridge structures (as the seismic waves enter the structure through the foundations) and the seismic capacity of the foundations.

Geotechnical models developed for site response analysis, and the computer software used for analyses, should be capable of incorporating the non-linear soil effects associated with the intensity and duration of shaking applicable for the site, as well as pre- and post-earthquake stress-strain-strength characteristics of soils.

For sites where soil liquefaction is predicted to occur, the effects of kinematic loading from permanent ground deformations on the structure must be evaluated and combined with the effects of inertial loading. Soil liquefaction and the associated softening of soils generally reduce the inertial loads transmitted to the structure. In practice, incorporating these effects as accurately as possible is important when carrying out PBD.

Soil liquefaction requires the application of several cycles of loading. Prior to onset of liquefaction, soils are capable of transmitting ground motions associated with strong shaking. For soil profiles where soil liquefaction is predicted to occur after some cycles of loading, the inertial loads can be conservatively estimated based on spectra computed from site response analysis, without considering the effects of soil liquefaction.

5.6.2 DOCUMENTATION

Soil-structure interaction analyses involve idealizing the geometry, material properties, and loading on the structure and its foundations. The analyses depend on the characteristics of the input ground motions, geotechnical models, sensitivity of the geotechnical design input to the anticipated structure response, structural models, and computer programs used for dynamic analysis.

These details should be documented appropriately, according to best practices in engineering.

5.7 CONSEQUENCE LEVELS AND GEOTECHNICAL RESISTANCE FACTORS

The consequence levels and geotechnical resistance factors specified in the Code are used to size the foundations and establish embankment configurations that satisfy the ultimate and serviceability limit states when subjected to static loading.

The consequence levels (and the corresponding consequence factors) reflect the anticipated consequences associated with exceeding the limit states. Assigning a consequence level to a bridge is the responsibility of the owner of the bridge or the AHJ, not the Engineering Professionals.

The geotechnical resistance factors reflect uncertainties associated with the geotechnical parameters (including measurement error), the construction, and the prediction model, and collectively reflect the degree of site understanding. Selecting the geotechnical resistance factors applicable for a given limit state is the responsibility of the geotechnical Engineering Professional.

For PBD, the foundations and embankments should be configured to meet the damage and service levels applicable to the importance category of the bridge, regardless of the geotechnical resistance factors used for design for static-loading conditions.

5.8 DESIGN CONSIDERATIONS, DAMAGE LEVELS, ANALYSIS TOOLS, AND PERFORMANCE DEMONSTRATION

Successful implementation of PBD depends on several important considerations to ensure adequate structural performance corresponding to the various seismicity levels. This section summarizes preferred design strategies, various failure modes for checking, Code-prescribed damage levels, the different types of analyses, and how performance can be demonstrated explicitly using the available analysis tools.

5.8.1 DESIGN CONSIDERATIONS

Seismic bridge design requires clearly identifiable ERSs. An ERS must be able to provide a reliable and uninterrupted load path for transfer of seismic forces to the supporting soil. In addition, sufficient energy dissipation and/or restraint must be provided to control seismic displacements.

Examples of ERSs include the following:

- Ductile substructure with essentially elastic superstructure
- Essentially elastic substructure with ductile superstructure (only for steel superstructures with ductile end diaphragms)
- Elastic superstructure and substructure with a fusing and/or damping mechanism between the two (such as isolated bridges and bridges with dampers)

The objective of good seismic design practice is to provide a structure with balanced geometry and stiffness. This may not always be achievable; however, this should be the goal as much as project constraints allow. Interaction among the various teams, such as highways, utilities, and structures, may help achieve this to a large degree. It is worth noting that the PBD approach is particularly suited for designing irregular structures and demonstrating that the intended performance meets the design.

Various checks are essential to ensure required seismic performance, including checking basic plastic-hinge zone strength, rotations and strains for resisting higher levels of seismicity, capacity-protected element shear and flexure capacity, and effects of foundation strength under inertial loading. For isolation and damping design, isolation bearing and damping device performance must be determined. Capacity design principles still apply. In addition, foundation strength and plastic rotations and/or strains under kinematic or inertial plus kinematic loads may also need to be checked, where applicable.

Design revisions will sometimes be necessary when certain performance criteria cannot be met. Major design revisions, such as a change of the basic ERS, will warrant a re-check of all performance criteria.

5.8.2 DAMAGE LEVELS TO SATISFY PERFORMANCE

The Code provides different performance levels through various service and damage criteria that correspond to different levels of seismicity and importance categories. Table 4.15 of the Code should be consulted in this regard.

Following are brief summaries of each damage level, along with associated criteria related to substructure elements according to the Code and the BC MoTI modifications in the Supplement. It should be noted that other jurisdictions may provide a different set of modifications to the Code or no modifications at all.

5.8.2.1 Minimal Damage

- The extreme fibre concrete and reinforcement steel limiting strains are $\epsilon c \leq 0.004$ and $\epsilon s \leq \epsilon y$ (no yielding), respectively, for concrete structures. The Supplement allows strain limits of $\epsilon c \leq 0.006$ and $\epsilon s \leq 0.01$, respectively.
- Local or global buckling is not allowed in steel structures.

5.8.2.2 Repairable Damage

- Full dead plus live load carrying capability must be verified post-event. This requirement has been deleted in the Supplement.
- For concrete structures, $\epsilon s \leq 0.015$. This limit has been changed to $\epsilon s \leq 0.025$ in the Supplement.
- No buckling of primary steel members is allowed; buckling of secondary members is allowed if stability is ensured.
- Net area rupture of primary steel members at connections is not allowed.
- To ensure aftershock resilience, 90% seismic capacity has to be retained; full capacity has to be restored after repairs. This requirement has been deleted in the Supplement.

5.8.2.3 Extensive Damage

- Full dead load plus 50% live load carrying capability must be ensured post-event. This requirement has been deleted in the Supplement. The Supplement, however, requires that the members be able to support dead load plus one lane of live load in each direction (for emergency traffic), including P-delta effects.
- Extensive concrete spalling is allowed; however, the concrete core is not allowed to crush. The Supplement specifies that the confined core concrete strain cannot exceed 80% of its ultimate confined strain limit; $\epsilon s \leq 0.05$.
- Global buckling of gravity supporting elements is not allowed.
- To ensure aftershock resilience, 80% seismic capacity has to be retained; full capacity has to be restored after repairs. This requirement has been deleted in the Supplement.

5.8.2.4 Probable Replacement

- The bridge may be unusable and need replacement, but collapse must be prevented.
- The Code does not give concrete and steel reinforcement strains for this level. The Supplement specifies that the confined core concrete strain cannot exceed its ultimate confined strain limit; $\epsilon s \leq 0.075$, except for 35M and larger bars, where $\epsilon s \leq 0.06$.
- The bridge must be able to carry full dead load plus 30% live load without impact, including P-delta effects.

5.8.3 ANALYSIS TECHNIQUES

Various analysis techniques can be employed for PBD, depending on the complexity and performance of a structure. At minimum, an elastic static analysis (ESA) or an elastic dynamic analysis (EDA), coupled with an inelastic static pushover analysis (ISPA) is required for PBD. Following is a summary of considerations and the available analysis techniques.

5.8.3.1 Effective Member Stiffness

For a deformation-based design philosophy, the use of uncracked section properties for analysis is usually not conservative. The Code therefore addresses the issue of effective section properties of concrete ductile substructure components.

The effective flexural stiffness must be based on the slope of the moment-curvature diagram between the origin and the point representing first rebar yield, as shown in **Figure 3** below.

Hence, for modelling flexural stiffness, the following equation is used:

$$EI_{eff} = M_{iy} / \Phi_{iy}$$

For modelling shear stiffness, the following equation is used:

$$(GA)_{eff} = Gc\ Acv\ Ieff/Ig$$

where Gc and Acv are the concrete shear modulus and element shear area; this may be neglected when appropriate.

5.8.3.2 Elastic Static Analysis

The ESA comprising the uniform-load method and the single-mode method can only be used for bridges classified as regular bridges, which primarily respond in their first mode in each principle direction.

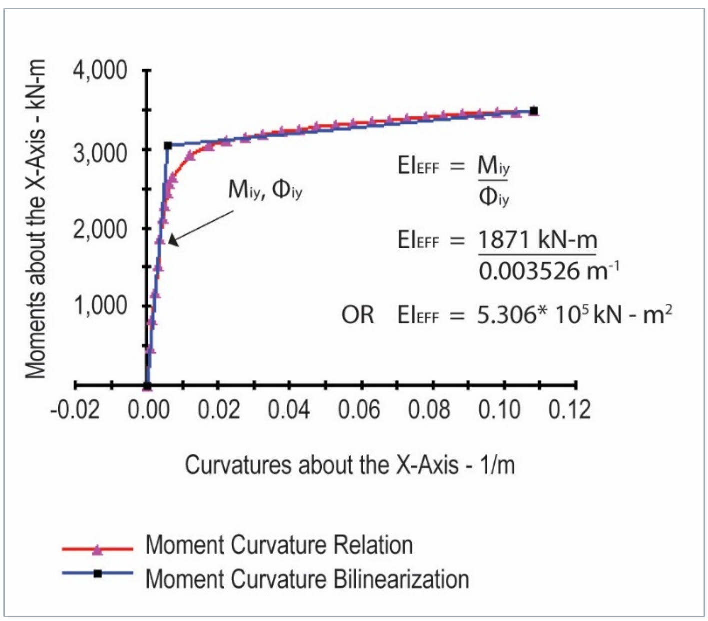


Figure 3: Cracked member stiffness determination using moment-curvature analysis

5.8.3.3 Elastic Dynamic Analysis – Response Spectrum Analysis

The EDA comprising the multi-mode response spectrum analysis (RSA) is required for structures whose behaviour can only be captured through several modes. The Code specifies accounting for enough modes such that 90% of the overall seismic mass of the structure is captured. Following are important points regarding the response spectrum based EDA:

- Use effective section stiffness values where applicable.
- Iterations for determining demand compatible secant soil spring stiffness are needed (**Figure 4**) in order to capture foundation flexibility and appropriate force and global displacement demands.
- For modal combinations, CQC (complete quadratic combination) works well for both closely spaced and well-separated frequencies. Use SRSS (square root of the sum of squares) for well-separated frequencies only.
- Use 100% of longitudinal demand from longitudinal analysis and add 30% of the longitudinal demand from the transverse analysis.
- Similarly, use 100% of transverse demand from transverse analysis and add 30% of the transverse demand from the longitudinal analysis.
- Vertical demands must be accounted for. This can be done either by applying the maximum/minimum dead load factors within the dead plus seismic load combination or through the explicit use of the vertical response spectrum.
- For straight bridges with little coupling in the two principal directions, the longitudinal demands from the transverse analysis and vice versa will be small.
- Advantages: Provides force demands for immediate service (minimal damage) as well as displacement demand targets for higher damage states.
- Limitations: Incapable of capturing highly non-linear behavior such as abutment yielding, joint opening and closing, etc. Force-demands for higher return period events causing inelastic behaviour will have significant error.

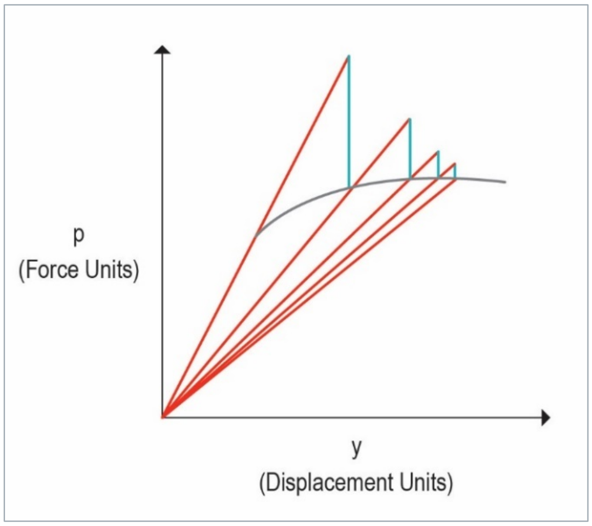


Figure 4: Soil p-y curve iterations

5.8.3.4 Moment-Curvature Analysis

The moment-curvature analysis is a strain compatibility based analysis used to quantify the moment and curvature behaviour and capacity of a section within an element. It helps determine the effective elastic stiffness of a section, as well as the effective yield and ultimate moments and the effective yield and ultimate curvatures.

Axial load-moment interaction can be easily captured in the moment-curvature analysis. Information based on a moment-curvature analysis, such as pre- and post-yield effective stiffness, effective yield moment, and effective yield curvature, is used as direct input into the inelastic static pushover and non-linear time history analysis.

Unconfined and confined concrete properties based on an appropriate model, such Mander's model, can be determined and incorporated (**Figure 5**). Coupled with inelastic static and dynamic analysis, the moment-curvature analysis can be used to determine various material strains, such as unconfined and confined concrete compressive strains along with rebar tensile strains at given curvatures corresponding to target displacement values. Such material strain quantification is required for damage and performance demonstration in a PBD context.

In accordance with the Code, the moment-curvature analysis must be carried out using either nominal or expected nominal material properties for the design of ductile substructure elements. This depends on the level of damage; nominal properties are required for minimal and repairable damage, while expected nominal properties are required for higher damage levels. For capacity-protected elements and brittle failure modes, probable material strengths are required for determining overstrength demands. Commercially available software is available for carrying out the moment-curvature analysis.

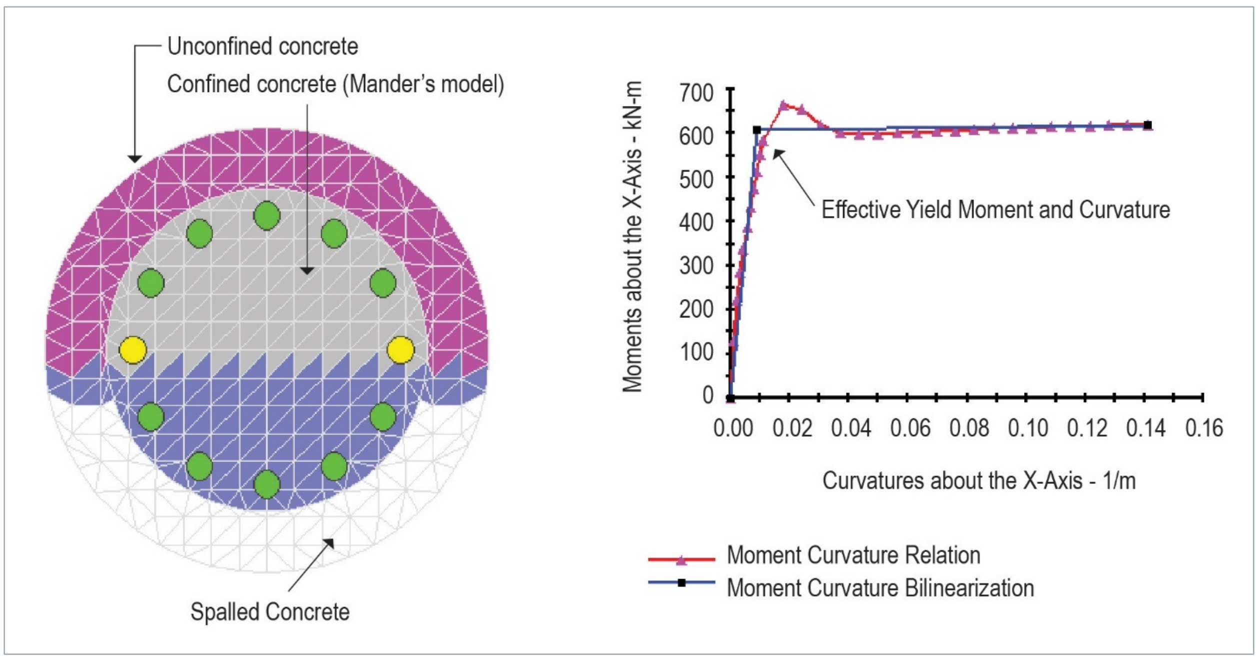


Figure 5: Moment-curvature analysis with unconfined and confined concrete properties

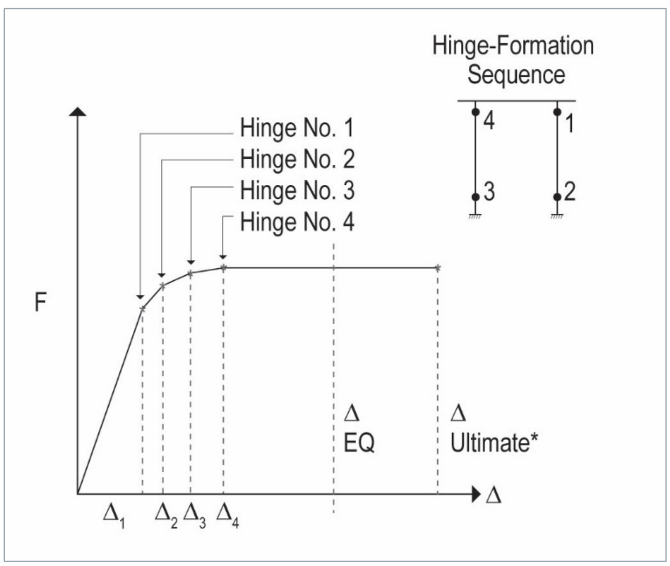
5.8.3.5 Inelastic Static Pushover Analysis

The ISPA, or simply the pushover analysis, is a non-linear analysis tool comprising a stepwise linear approach. As appropriate, a pushover analysis may be carried out by developing a local substructure model of a bridge bent, or a model of a complete bridge, and subjecting it to an increasing pattern of lateral loading.

When each major non-linear event occurs, such as the formation of a plastic hinge, the structural model is altered to incorporate the resultant stiffness change in the model. A further increment in lateral load/displacement is carried out for this updated static system. This process is repeated until mechanism formation (Figure 6). The analysis should always start from the stressed dead load state.

This method can be used for the following purposes:

- To account for the sequence of inelastic actions, such as plastic hinge formation, and identify the global collapse mechanism.
- To ascertain the intermediate damage states of a structure based on the local plastic rotations, curvatures, and material strains in the ductile substructure elements.
  - One critical consideration in this regard is the global displacement level for determining the corresponding element rotations and material strains. The approach in the Code is to push the structure to the global displacement demands determined using the ESA or EDA and incorporating cracked stiffness values.
- To determine the ultimate displacement capacity of bridge substructures and determine global reserve displacement capacities.



* Note: Corresponds to when the critical hinge reaches its plastic rotation limit

Figure 6: Typical transverse pushover curve for a two-column bent with plastic hinges at column ends

- To determine the degraded shear capacities due to increasing local ductility demands within the plastic hinge location.
- To determine overstrength force demands corresponding to non-ductile failure modes (such as shear) and for the design of capacity-protected elements.
  - Footings, beam-column and column-footing joints, and cap beams are examples of capacity-protected elements that are usually designed for overstrength demands arising in the plastic hinges.
  - It should be noted that although the pushover analysis can help produce

overstrength demands for capacity-protected elements and non-ductile failure modes, it will not pick up such failure modes on its own. The designer must be aware of such limitations and use the available analysis tools judiciously.

- To quantify reserve seismic capacity if strength and stiffness degradation are accounted for in the plastic hinge properties. It can also be carried out while incorporating P-delta effects.

In certain cases, global 3D models are required for pushover analysis. For example, for a bent that is monolithic with the superstructure, the longitudinal pushover should incorporate the deck to capture the reversed curvature behaviour of the column(s). Similarly, for a structure with highly variable column heights, a global model should be employed for a longitudinal pushover analysis to determine the appropriate hinge sequence and resulting ductility demands.

Current practice incorporates the first-mode lateral force pattern in each principle direction for carrying out the pushover analysis. While the multi-mode pushover analysis technique is sometimes used for multi-story buildings, it requires considerable post-processing and statistical combination of demands because the various peak modal demands occur at different times. As such, it loses the simplicity and lucidity of the first-mode based pushover analysis commonly carried out for bridge structures. Instead of utilizing the modal pushover technique, a non-linear time history analysis may be more appropriate for complex bridge structures to ascertain the structural demands and demonstrate performance appropriately.

5.8.3.6 Non-Linear Time History Analysis

The non-linear time history analysis (NTHA) combines the demand and capacity sides of the seismic response. Time history ground motion input and cyclic non-linear member characterization are incorporated into the analysis. This method consists of the step-by-step integration of the coupled equations of motions, and the analysis is started from the stressed dead load state. Global demands and corresponding plastic actions are obtained directly from the NTHA. Critical demands values are obtained concurrently, and statistical combinations are not required.

Damping modelling is a critical consideration for the NTHA. Mass and stiffness proportional Rayleigh damping is usually employed for this purpose, although other methods are acceptable and available. The damping is anchored to two modes with the largest mass participation (Figure 7).

Using Rayleigh damping for NTHA is a topic of current debate, but traditionally 2% and 5% damping values have been used for steel and concrete structures, respectively. While fitting the Rayleigh curve, care should be taken not to overdamp the system by using large values at other periods. A conservative approach would be to incorporate small damping values to ensure numerical stability in the solution, while modelling the hysteretic behaviour of the fuses appropriately to adequately capture the post-yield non-linear behaviour. Employing a group damping technique, where different Rayleigh curves are applied to various sets of elements, can help avoid overdamping the system.

Where appropriate, soil radiation damping may be relied upon and modelled, using dashpots along with soil springs in the structural model.

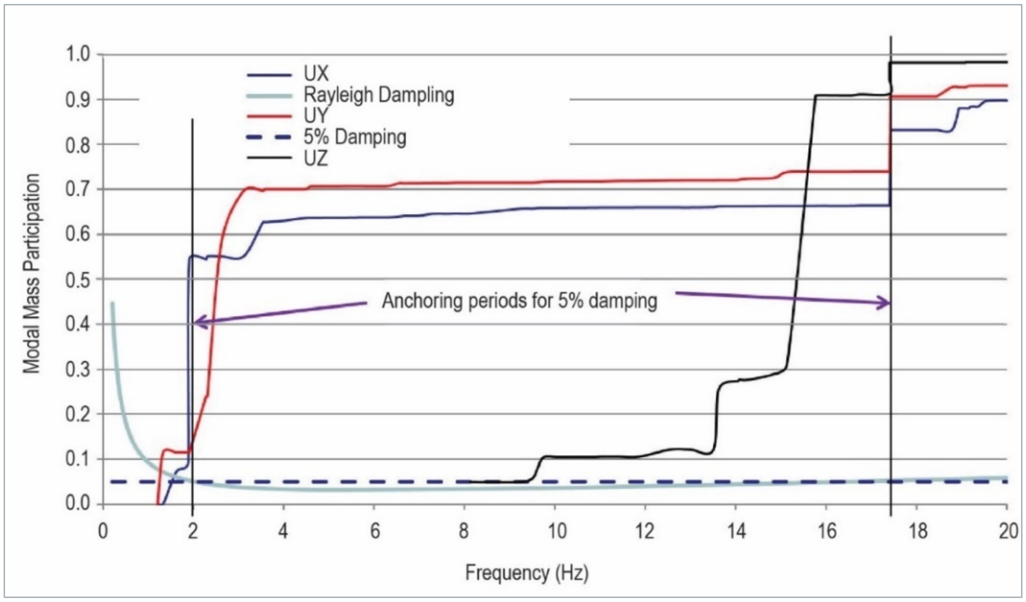


Figure 7: Mass and stiffness proportional Rayleigh damping example

Strength and stiffness degradation should be realistically accounted for in an NTHA. To accomplish this, a backbone curve (force-displacement capacity boundary) can be incorporated into the analysis (**Figure 8**). Structural response cannot cross the force-displacement capacity boundary. Material and detail appropriate hysteresis models must be employed to account for strength and stiffness degradation in NTHA.

The Code requires a minimum of 11 spectrally matched time histories, while the mean response quantity is required for design purposes.

It should be noted that the NTHA produces vast amounts of data that require experience and judgement for interpretation, so this method should be used with caution. The NTHA should not be treated as a design tool, rather as a design verification and performance demonstration tool. Before using the NTHA, the designer should employ simpler analyses, such as the RSA coupled with pushovers, to gain an understanding of the seismic load path and structural behaviour. Modelling damping for complex structures that derive contributions from several modes can be problematic; since damping can only be anchored to two modal periods using the Rayleigh approach, it can be under- or over-estimated for other modes with significant mass participation, especially if their periods/frequencies are significantly different.

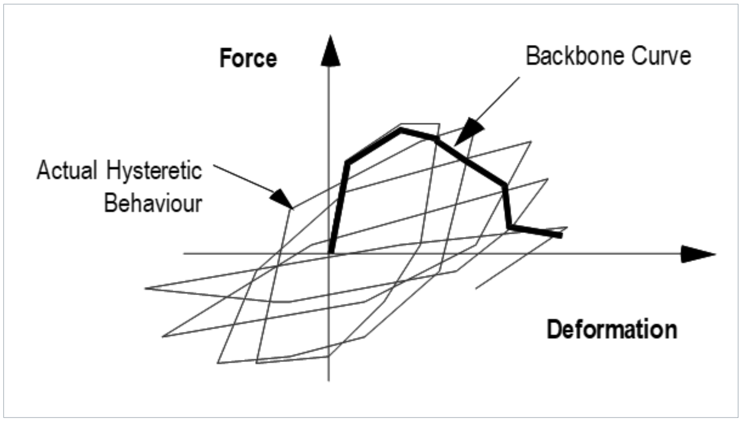


Figure 8: Concept of backbone curve (adapted from FEMA 440)

5.8.4 EXPLICIT PERFORMANCE DEMONSTRATION

For PBD of bridges, damage and service compliance is basically demonstrated through material strains and plastic rotations. ESA or EDA, coupled with ISPA, are required as a minimum level of analysis to show design compliance, unless the structures are designed elastically corresponding to the highest return-period event. The required displacement level for calculating items such as material strains and element rotations must be equal to that predicted by the ESA or EDA with cracked section properties.

Additional static or dynamic non-linear analysis is required to show that the structure can resist full dead load and a percentage of live load, including P delta effects for post-event service.

Aftershock compliance can be best demonstrated using NTHA. Aftershock capacity is more difficult to quantify due to modelling limitations related to cyclic strength and stiffness degradation. The material hysteresis must properly capture such behaviour, and the software must be able to incorporate it appropriately. An NTHA starting from the stressed state and accounting for previous damage due to the mainshock time history should be used for such an assessment. It should be noted that this is a greater concern for older bridges with inappropriate loading considerations, deficient seismic detailing, and lack of capacity protection. New bridges with appropriate detailing are not likely to experience such degradation and are expected to adequately resist aftershocks of equal or smaller magnitude than the mainshock.

It should be noted that often the Engineer of Record will be required to explain the performance of the structure in physical terms to the owner. The owner usually requires some assistance in interpreting the numbers, tables, and graphs used to demonstrate performance. As such, the Engineer of Record should be able to explain, in simple terms, the overall structural performance, post-event damage and service states, and load carrying capability of the bridge.

The following subsections provide brief summaries of performance demonstration of column and pile elements using non-linear static analysis. Instead of the following, more refined, non-linear time history analysis can also be used.

5.8.4.1 Column Performance Demonstration

An ESA or EDA must initially be carried out to establish global displacement demands. For quantification of column inelastic performance, use the following approaches:

- Carry out the ISPA with hinges modelled at all preselected locations.
- Corresponding to the global displacement level, output the plastic rotations and divide by analytical plastic hinge lengths (e.g., ATC 32 Eq. R8-19) to arrive at plastic curvature values ($\Phi p = \Theta p/Lp$).
- Using moment-curvature output corresponding to appropriate axial load, determine the total curvature by adding the plastic curvature to the equivalent yield curvature ($\Phi u = \Phi y + \Phi p$).
- Determine corresponding concrete and rebar strain values from the moment-curvature output and compare with the corresponding strain limits (ϵc, ϵs).
- Alternatively, hinge locations can be modelled using distributed plasticity, employing fibre models to calculate material strains directly. An NTHA can directly provide a plot of plastic hinge strain using such an approach (**Figure 9**).

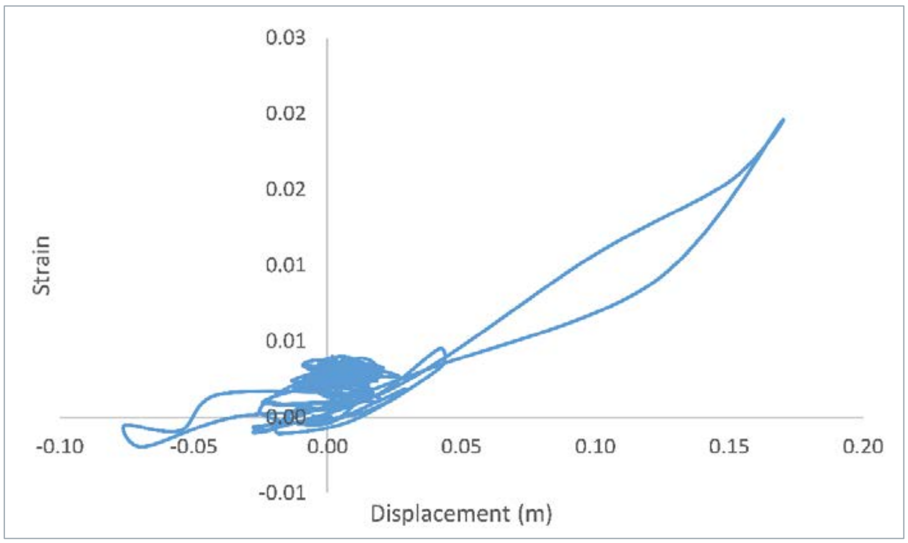


Figure 9: Steel reinforcement strain versus bent displacement (distributed plasticity model)

5.8.4.2 Pile Performance Demonstration

For RSA, the soil p-y behaviour is modelled with linear springs using values for effective stiffness, as explained earlier. For ISPA, non-linear springs incorporating full p-y behaviour should be modelled using non-linear springs.

The first analytical run can be carried out with hinges modelled at the pile cap locations only, but not in-ground. The second analytical run can then incorporate hinges both at the underside of the pile cap and the in-ground locations, where maximum flexural demands larger than elastic pile capacities occur (**Figure 10**). Computer software can report the plastic rotations in all applicable pile hinges directly. These can be compared with the allowable limits to show performance compliance.

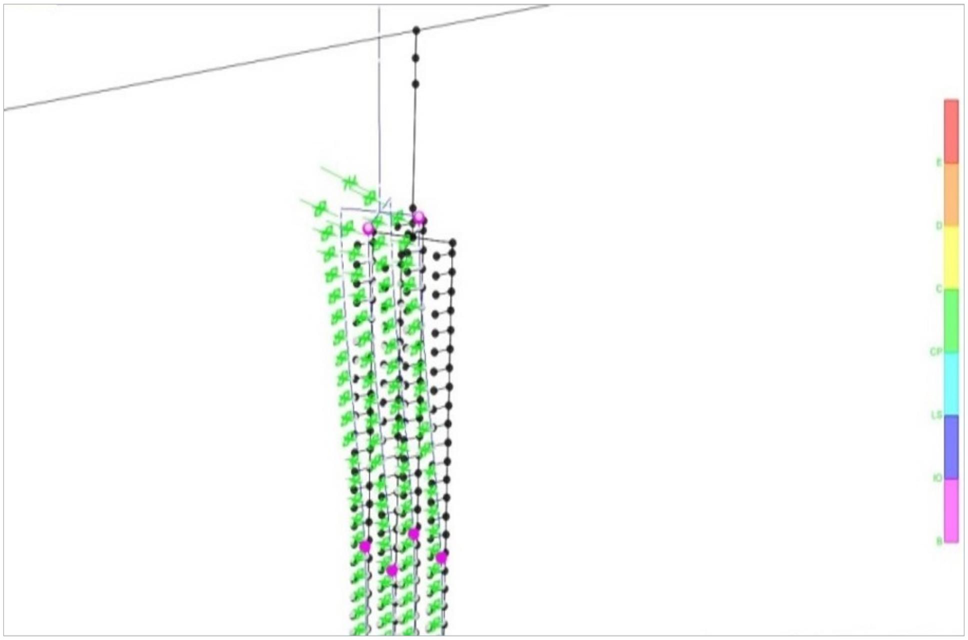


Figure 10: Liquefaction analysis with p-y curves incorporating pilecap bottom and in-ground hinges

For the liquefied case with kinematic demands, use the following approaches:

- Both inertial and kinematic effects should be captured using liquefied soil springs.
- Two-node non-linear soil springs with one end attached to the substructure element and the other fixed in space could be used.
- Lateral spread values can be applied to the fixed side of the springs, thus imposing ground deformation demands through the non-linear spring stiffness values.
- Lateral spreading analysis can be carried out using a global 3D model employing a non-linear static analysis approach; material non-linearity should be modelled.
- The Code does not address the kinematic plus inertial combination; however, the Supplement requires adding 50% inertial displacement demands to 100% kinematic demands and vice versa.
- It is more practical to superimpose inertial demands using individual bent models.
- Using a non-linear static analysis, the designer must accurately capture the force and displacement state at the end of kinematic loading before imposing inertial demands. A more complicated, coupled non-linear time history analysis may be used to account for the combined inertial and kinematic effects simultaneously.
- A simplified approach to account for the kinematic effects is for the geotechnical Engineering Professional to provide the structural Engineering Professional with appropriate liquefaction forces at different elevations of the pile. These may then be applied in the structural model, removing soil support in the liquefied zone while modelling the soil stiffness outside such a zone with soil springs.

5.8.5 CAPACITY-PROTECTED ELEMENTS

As described earlier, plastic hinge locations or other fuses are purposefully made weaker, corresponding to the elastic demand values arising from seismic events with large return periods. However, all elements except the fuses must be able to resist maximum seismic forces in an essentially elastic manner.

The fuses provide an upper bound on the force that needs to be resisted by the ERS. It should be noted that an overestimation of the flexural hinge capacity is not critical and simply implies a higher local ductility demand. However, if the overstrength of a plastic hinge is underestimated, it can give rise to brittle failure mechanisms (Priestly et al. 1996). Therefore, it is much more important not to under-predict the plastic hinge overstrength capacity. Capacity-protection thus endeavours to suppress brittle failure modes and make the structure perform as intended, while resisting higher than predicted seismic demands.

Element and joint shear are examples of non-ductile failure modes. Non-ductile elements requiring capacity-protection include cap beams, beam-column joints, footings, column-footing joints, and superstructure.

5.9 BASE ISOLATION AND ENERGY DISSIPATION

One of the most powerful tools available to designers in PBD of bridges is the use of base isolation and energy dissipation devices. This approach can be used to protect the structure from strong ground motions and limit deformations and damage to the structure.

5.9.1 BASE ISOLATION CONCEPTS

The commentary to Section 4.10 of the Code provides considerable information about base isolation of bridges and includes a thorough discussion of the Code requirements.

The fundamental concept of base isolation is to introduce flexible elements into the structure in order to shift the fundamental periods of vibration so the critical components above the isolators, and the ERS elements, are subjected to much lower accelerations. However, the introduced flexibility also results in much larger lateral displacements. Isolation systems control these displacements by introducing high damping, either by including energy dissipation characteristics in the bearing design, or by adding supplemental dampers. Shock transmission units, which allow slow movements to accommodate thermal movements but lock and transmit load under fast movements such as earthquake motions, can be used in some situations.

Generally, base isolation of bridges is achieved by using specially designed isolation bearings, which support the superstructure girders on the substructure and replace the normal bridge bearings. Supplemental dampers or shock transmission units can be connected horizontally between the substructures and superstructure.

One of the desired benefits of base isolation is to prevent damage to tall substructures by limiting the deflections imposed on piers due to inertial loading. This can be important in meeting damage limits for structures categorized as major route or lifeline bridges. Isolators can also be used in seismic retrofits of existing bridges to protect substructures, thereby limiting the retrofits required to the piers and abutments.

5.9.2 CODE REQUIREMENTS

The Code, Section 4.10 addresses the use of base isolation and energy dissipation devices. The performance criteria for bridges using these devices is the same as for other bridges; however, additional criteria for the isolator and damping units is provided in the Code, Section 4.10.4.3, Table 4.19.

A key challenge of PBD in the design of isolated bridges is damage limits at the interfaces of the isolated components, generally the joints at abutments. The bridge superstructure moves independently of the abutment, and the resulting joint damage can limit service on the bridge. The designer must pay special attention to meeting damage and service limits at the bridge joints and to any elements connecting to the isolated parts of the structure.

Analysis procedures are provided in the Code, Section 4.10.5. Elastic static or elastic dynamic methods may be used for simple bridges, within the significant limitations provided in the Code. In general practice, these methods can be used for preliminary design, but isolated bridges will require 3-D non-linear time history analysis to verify the design.

5.9.3 DEVICES AND SYSTEMS

A number of base isolation and energy dissipation systems for bridges are available. They are proprietary products that suppliers have developed with significant investments in research and development. There are advantages and disadvantages to the various types of systems with respect to vertical loads, deflection demands, environmental exposure, and seismic performance requirements.

The Code specifies extensive testing and quality control requirements for isolation devices in Sections 4.10.9 through 4.10.11. Base isolator properties used in the analysis must be verified through the testing requirements described in Section 4.10.9. Established suppliers may have pre-approved or certified test data from prototypes that can be used. Extrapolation of design properties from tests of isolator units of similar type and size is permitted. Testing requirements for supplemental dampers and shock transmission units are included in Sections 4.10.12 and 4.10.13.

The bridge’s integrity during an earthquake will depend on the base isolation system used to limit the loads and deflections imposed on the elements within the lateral load path, and are similar to capacity-protected elements. To provide a margin of protection for these elements, Section 4.10.6 of the Code requires that the isolators and structure be designed for 1.25 times the displacements from the analysis. The Code also includes requirements for ductile design of substructures and requirements for connection forces in Section 4.10.7.

5.10 PBD APPLICATION USING THE CODE

Engineering Professionals will likely face challenges while trying to demonstrate some performance requirements according to the Code. The Supplement provides additional guidance on some of the issues and aims to provide more consistent criteria. Following are brief summaries of such issues:

- **No damage versus minimal damage**: As described earlier, the Code does not provide a description for the ‘none’ damage state. It is impractical to design structures to have no further cracking beyond the normal service level cracks under seismic loads. The category of ‘none’ for damage in the Code is currently under review, while it has been deleted and replaced by the ‘minimal’ category in the Supplement.

- **Rebar strain for minimal damage**: The Code stipulates no rebar yielding for the minimal damage state. This requirement in the Code is currently under review. The unintended consequence for reinforced concrete substructures is that this requirement results in impractically high rebar ratios in plastic hinges, which directly impacts capacity design and introduces constructability issues. The Supplement has changed this requirement by stipulating a more practical rebar strain limit of 0.01.
- **Restricted emergency traffic**: For service disruption, the Code requires the bridge to be usable for restricted emergency vehicles after inspection but provides no guidance on the weight and type of such vehicles. Emergency traffic can vary significantly from jurisdiction to jurisdiction and should be agreed between the designer and the owner for post-event performance requirements.
- **Aftershock capacity demonstration**: The Code requires the bridge to retain a certain percentage of its capacity corresponding to given service and damage. A rigorous way to demonstrate required performance is to use NTHA incorporating strength and stiffness degradation, and to run mainshock-aftershock time history scenarios. A simplified approach would be to use an ISPA incorporating strength and stiffness degradation and, where appropriate, P-delta effects, to show that the base-shear degradation at the design displacement is less than 10% or 20%, as required (**Figure 11**).

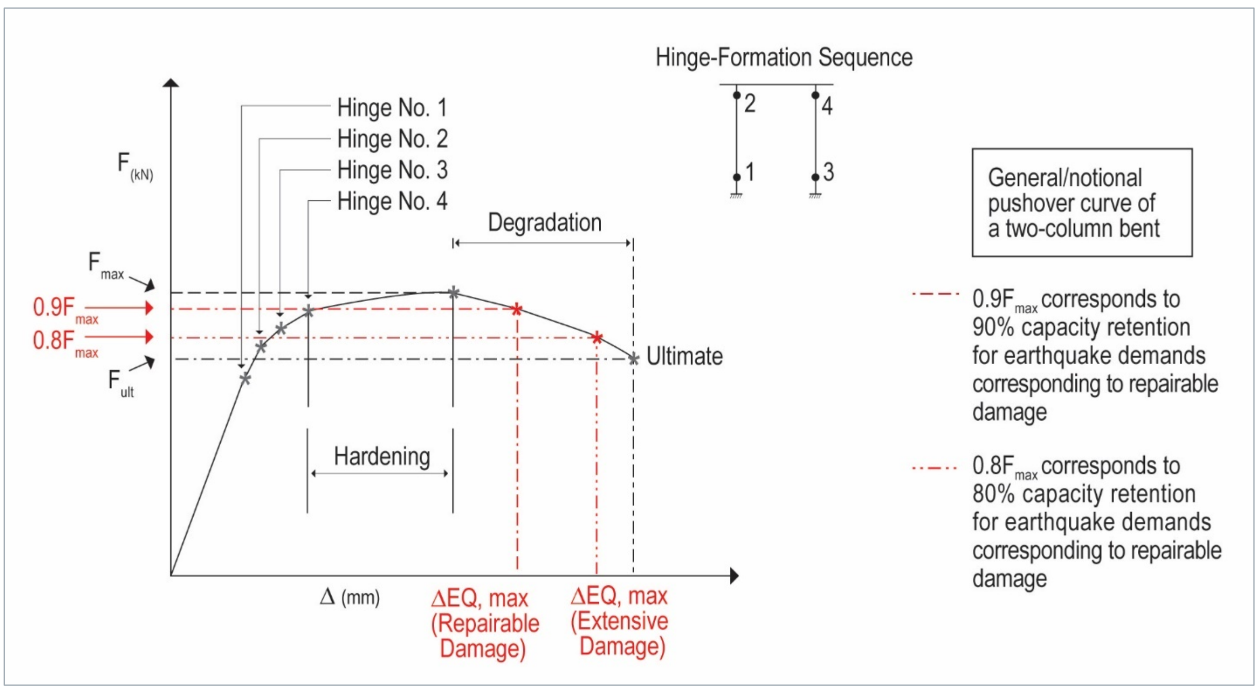


Figure 11: Aftershock capacity demonstration using inelastic static pushover analysis (ISPA)

- **Steel substructure performance criteria**: Steel bents are not regularly used as the ERS for bridges. Although the Code covers damage and performance criteria in general, it does not describe steel substructure element performance in detail. Various clauses address force capacities to some degree, but are largely silent on explicit steel strains or rotations for performance demonstration of various steel bent configurations, such as moment-resisting frames or ductile concentrically braced frames. Other clauses, such as for ductile eccentrically braced frames, suggest designing the bents using the R-factor (force-based) approach. Recent literature and research, as well as other building codes, provide relevant information (for example, on plastic rotation limits for eccentrically braced frame shear links); these sources may be relied upon to help demonstrate performance for such systems.
- **Shear capacity determination**: The shear capacity provided by concrete within a plastic hinge zone degrades as the hinge experiences large ductility demands due to a decrease in concrete aggregate interlock. The Code provides an expression for determining the reduced concrete capacity for such a case. However, the Code expression does not explicitly account for the level of ductility and therefore provides a lower bound shear capacity. The use of refined seismic shear design methodologies such as those provided by Priestly et al. (2007) may be considered.
- **Detailing for cracked joints**: It may be impractical to provide adequately large beam-column joints to prevent joint cracking under overstrength plastic hinge demands. If so, supplementary reinforcement must be provided to ensure capacity protection of such zones. For guidance on design of these elements, publications such as Caltrans Seismic Design Criteria (Caltrans 2013) and Priestley et al. (1996) can be consulted.

5.11 ASSURANCE STATEMENT

Refer to **Section 2.2.3 Engineer of Record** for the responsibilities of the Engineer of Record in completing the assurance statement included in **Appendix A: Engineer of Record – Bridge Seismic Design Assurance Statement**.

6.0 REFERENCES AND RELATED DOCUMENTS

Engineers and Geoscientists Act [RSBC 1996], Chapter 116.

Applied Technology Council. 1996. ATC-32 Improved Seismic Design Criteria for California Bridges: Provisional Recommendations. Redwood City, CA: Applied Technology Council. 215 pp.

Auditor General of British Columbia (BC). 2012. Understanding Public Private Partnerships. Victoria, BC: Auditor General of BC. [accessed 2018 Jan 17]. https://www.bcauditor.com/sites/default/files/publications/files/oagbc-understanding-p3-public-private-partnerships_0.pdf.

BC Ministry of Transportation and Infrastructure (BC MoTI). 2016. Bridge Standards and Procedures Manual. Supplement to CHBDC S6-14. (October 2016). Victoria, BC: BC MoTI.

California Department of Transportation (Caltrans). 2013. Seismic Design Criteria. April 2013. Version 1.7. Sacramento, CA: Caltrans.

Canadian Standards Association (CSA). 2014. Canadian Highway Bridge Design Code. CSA Standard S6-14. (December 2014). Toronto, ON: CSA.

Cassidy JF, Rogers GC, Lamontagne M, Halchuck S, Adams J. 2010. Canada's Earthquakes: 'The Good, the Bad, and the Ugly'. Geoscience Canada. 37(1). [accessed 2018 Jan 15]. https://journals.lib.unb.ca/index.php/GC/article/view/15300/16404.

Engineers and Geoscientists BC. 2017. Quality Management Guidelines: Use of Seal. Version 2.0. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 10 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Engineers and Geoscientists BC. 2018a. Quality Management Guidelines: Documented Independent Review of Structural Designs. Version 1.4. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 08 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Engineers and Geoscientists BC. 2018b. Quality Management Guidelines: Documented Checks of Engineering and Geoscience Work. Version 1.3. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 08 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Engineers and Geoscientists BC. 2018c. Quality Management Guidelines: Direct Supervision. Version 1.3. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 10 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Engineers and Geoscientists BC. 2018d. Quality Management Guidelines: Retention of Project Documentation. Version 1.3. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 10 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Engineers and Geoscientists BC. 2018e. Quality Management Guidelines: Documented Field Reviews During Implementation or Construction. Version 1.3. Burnaby, BC, Canada: Engineers and Geoscientists BC. [accessed 10 Jan 2018]. https://www.egbc.ca/Practice-Resources/Quality-Management-Guidelines.

Federal Emergency Management Agency (FEMA). 2005. Improvement of Nonlinear Static Seismic Analysis Procedures. FEMA 440. June 2005. Washington, DC: FEMA.

Finn WDL, Kramer SL, O'Rourke TD, Youd TL. 2010. Earthquake Engineering Research Institute (EERI) Ad Hoc Committee on Soil Liquefaction During Earthquakes. Final Report: Technical Issues In Dispute with EERI MNO-12, Soil Liquefaction During Earthquakes. 16 August 2010.

Gülerce Z, Abrahamson N., 2011. Site-Specific Design Spectra for Vertical Ground-Motion. Earthquake Spectra. 27(4): 1023-47.

Montgomery J, Boulanger RW. 2017. Effects of Spatial Variability on Liquefaction-Induced Settlement and Lateral Spreading. J Geotech Geoenviron Eng. 143(1). January 2017. doi: 10.1061/(ASCE)GT.1943-5606.0001584.

Natural Resources Canada (NRC). 2016. Seismic Design Tools for Engineers. [website]. Ottawa, ON: NRC. [date modified: 2016-02-10]. [accessed 2018 Jan 11]. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php.

Priestley MJN, Calvi MC, Kowalski MJ. 2007. Displacement-Based Seismic Design of Structures. Pavia: IUSS Press.

Priestley MJN, Seible F, Calvi GM. 1996. Seismic Design and Retrofit of Bridges. New York, NY: John Wiley and Sons.

Seemann MR, Onur T, Cassidy J. 2008. Seismic Hazard Resulting from Aftershock Activity Following a Cascadia Subduction Earthquake. Paper presented at: 14th World Conference on Earthquake Engineering; 2008 Oct 12-17, 2008; Beijing, China.

Task Force Report. 2007. Geotechnical Design Guidelines for Buildings on Liquefiable Sites in Accordance with NBC 2005 for Greater Vancouver Region. May 8, 2007. [accessed 12 Jan 2018]. https://www.researchgate.net/publication/297808652_Geotechnical_design_guidelines_for_buildings_on_liquefiable_sites_in_accordance_with_NBC_2005_for_Greater_Vancouver_Region.

7.0 APPENDICES

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- B1: Reinforced Concrete Bridge
- B2: Performance-Based Design of an Extended Pile Concrete Bent Highway Bridge
- B3: Tubular Eccentrically Braced Frames

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APPENDIX A: ENGINEER OF RECORD – BRIDGE SEISMIC DESIGN ASSURANCE STATEMENT

ENGINEER OF RECORD – BRIDGE SEISMIC DESIGN ASSURANCE STATEMENT

Note: This statement is to be read and completed in conjunction with the *Professional Practice Guidelines – Performance Based Seismic Design of Bridges in BC* (these guidelines).

[Print clearly and legibly]

TO: **OWNER** DATE: ____________________

____
Name

____
Address

____

FOR: **BRIDGE**

____
Northing and easting (location)

____
Located at (description)

____
Name of bridge or description

____
Importance classification

I am a qualified Engineers and Geoscientists BC-registered professional and the **Engineer of Record** for the bridge project identified above.

ENGINEER OF RECORD – BRIDGE SEISMIC DESIGN ASSURANCE STATEMENT

In preparing the bridge design, I have confirmed that the following activities have been completed:

COMPLETED BY THE ENGINEER OF RECORD (INITIALS)	ACTIVITY
	GENERAL – BRIDGE SEISMIC DESIGN
	Route importance, bridge importance category, seismic performance category, site classification, seismic design approach, peer review (if any), and seismic performance levels (if applicable) and criteria (note exceptions below) have been discussed with the owner and consented to by the owner.
	Bridge geotechnical investigations and related seismic design parameters, and soil-structure interaction effects, where applicable, have been incorporated into the bridge and seismic design in accordance with this guideline and/or as consented to by the owner.
	The material and modelling assumptions have been identified and appropriately included in the analysis.
	The type and arrangement of the earthquake-resisting system (ERS) for seismic loading in all directions has been identified and described.
	Bridge structural seismic design has been performed in accordance with these guidelines and the Canadian Highway Bridge Design Code (latest edition), including documented modifications or exceptions consented to by the owner.
	ADDITIONAL PERFORMANCE-BASED SEISMIC DESIGN REQUIREMENTS
	Bridge post-seismic performance requirements for return to traffic service levels, expected damage, and related repair have been specified by or consented to by the owner.
	The design inputs (seismic hazard, geotechnical, other), modelling (geotechnical, structural, and soil-structure interaction, where appropriate), analyses, and design are appropriate to demonstrate expected performance.
	The potential for soil liquefaction has been addressed and its effects (where applicable) have been accounted for in the seismic design.
	The bridge design report has been prepared in accordance with these guidelines, and owner exceptions or modifications have been identified, as noted above.
	DESIGN USING SPECIALTY SYSTEMS OR PRODUCTS
	Appropriate properties and design methods based on the Canadian Highway Bridge Design Code (latest edition) or owner-accepted requirements for specialty devices have been incorporated into the seismic analysis and design. The bridge design report has been prepared in accordance with these guidelines, and owner exceptions or modifications have been identified as noted above.
	Appropriate testing of specialty devices has been specified to allow the owner and designer to verify that the seismic performance required by the analysis and design can be met.
	Test results of specialty devices to confirm analysis and design assumptions have been specified and will need to be reviewed and confirmed at the construction stage (note exceptions below).

ENGINEER OF RECORD – BRIDGE SEISMIC DESIGN ASSURANCE STATEMENT

Exceptions or modifications to these guidelines or assurances in this assurance statement have been discussed with the owner and accepted into the design as follows.
[Print clearly and legibly]

I certify that I am an Engineering Professional as defined below.

DATE: ______________________

Name

Signature

Address

Telephone

Email

(Affix PROFESSIONAL SEAL here)

(If the Engineering Professional is a member of a firm, complete the following.)

I am a member of the firm ______________________ (Name of firm)
and I sign this letter on behalf of the firm.

APPENDIX B: CASE STUDIES

- B1: Reinforced Concrete Bridge
- B2: Performance-Based Design of an Extended Pile Concrete Bent Highway Bridge
- B3: Tubular Eccentrically Braced Frames

APPENDIX B1: REINFORCED CONCRETE BRIDGE

AUTHORS: S. ASHTARI, PH.D. CANDIDATE, DR. C. VENTURA, P.ENG., S. KHAN, P.ENG., DR. U. ATUKORALA, P.ENG.

B1.1 INTRODUCTION

This case study describes the step-by-step application of the provisions for performance-based design (PBD) in the CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014) to the design of a reinforced concrete bridge.

The performance assessment of the bridge described here uses two sets of performance criteria: the performance criteria of the Code for reinforced concrete bridges, and the criteria adopted in the British Columbia (BC) Ministry of Transportation and Infrastructure (MoTI) supplement to the Code (the Supplement) (BC MoTI 2016).

The PBD approach requires meeting certain performance criteria, described as tolerated levels of structural damage, and serviceability objectives at three hazard levels with 10%, 5%, and 2% probabilities of exceedance in 50 years. For brevity, in this case study these will be referred to as 10%/50, 5%/50, and 2%/50. Moreover, only the performance criteria relevant to the flexural response of ductile substructure elements will be discussed (in this case, columns). Additional performance checks must be performed for the full seismic design of the bridge.

B1.2 BRIDGE DESCRIPTION

The bridge in this case study is a major route bridge located in Victoria, BC, Canada. The assumed coordinates of the bridge site are 48.4284, -123.3656. It was designed as a two-span, single-bent, reinforced concrete bridge with steel girders. The initial member sizing of the bridge was achieved using force-based design principles and based on experience.

B1.2.1 BRIDGE STRUCTURE

Schematic elevation views of the entire bridge, as well as the bridge pier and deck, are shown in **Figures B1-1** and **B1-2**.

- The total length of the bridge is 125 m, with west and east spans of 60 m and 65 m, respectively.
- The deck is comprised of three steel girders topped with a 0.225 m concrete slab and a 0.09 m asphalt overlay.
- The section of the steel girders changes along each span, as shown in **Figure B1-1**, and the maximum depth of the girders is 2.9 m.
- The bridge bent includes two 8 m high circular reinforced concrete columns, connected at the top with a 2.1 x 1.8 m reinforced concrete capbeam.
- The columns are both 1.525 m in diameter with 36-35M longitudinal rebars, making up a 2% longitudinal reinforcement ratio. They are

laterally reinforced with 20M spirals at 0.07 m pitch in the plastic hinge region (1.5 m from the top and bottom of the columns) and 0.15 m pitch elsewhere.

- The thickness of the cover concrete for both columns is 0.075, and their axial force ratio ($P_a/f'_c A_g$) is 0.10.
- The concrete for all members has a minimum specified compressive strength of 35 MPa and a unit weight of 24 kN/m^3.
- The reinforcement steel grade is 400R, with minimum specified yield strength of 400 MPa and ultimate yield strength of 540 MPa.
- The unit weight of the steel is 77 kN/m^3.
- Each column has a 1.5 m deep, 6 x 6.5 m concrete spread footing.
- At the abutments the bridge has expansion bearings, and at the bent it has pinned bearings.

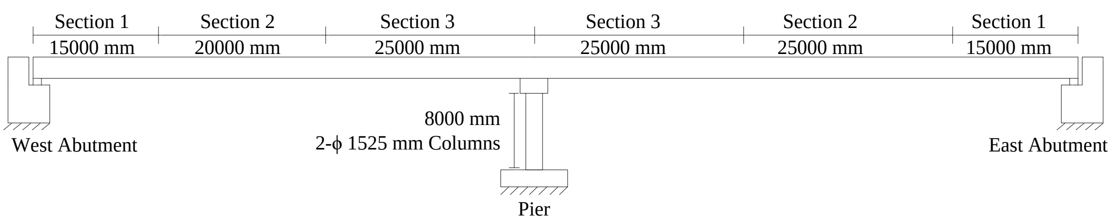

Figure B1 - 1: Schematic elevation view of the bridge

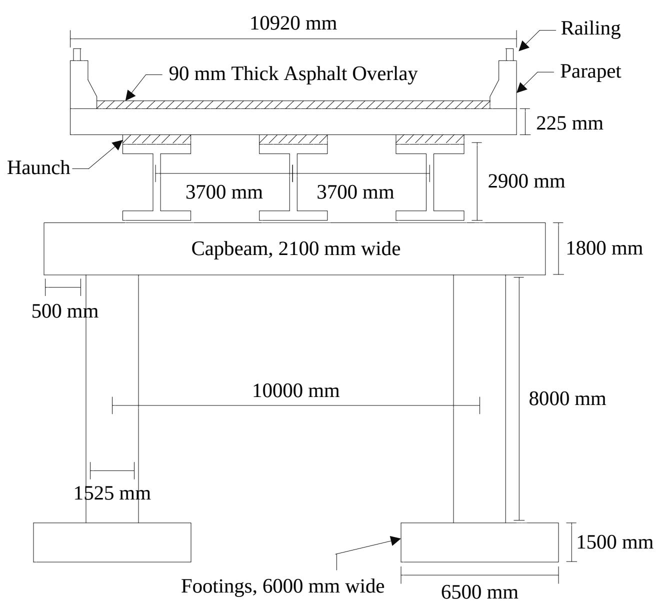

Figure B1 - 2: Schematic elevation view of the bridge bent and superstructure

B1.2.2 SITE PROPERTIES

The soil profile at the bridge site includes soft rock to very dense soil corresponding to the site class C in the Code. These conditions roughly correspond to a uniform sand layer with assumed shearwave velocity of 650 m/s, friction angle of 32 degrees, zero cohesion, Poisson's ratio of 0.3, and unit weight of 18 kN/m^3.

For this site condition, the effects of soil-structure interaction were ignored and a fixed-base model was used for analysis of the bridge.

B1.3 SEISMIC HAZARD

Three distinctive sources of earthquakes are active in the region: shallow crustal, deep subcrustal sources, and the Cascadia subduction zone. All three sources contribute to the hazard, depending on the fundamental period of the structure and the distance of the site to source.

The uniform hazard spectrum (UHS) values for Victoria were obtained using the 2015 National Building Code of Canada seismic hazard calculator, available online at the Natural Resources Canada (NRC) website (NRC 2015), for the 10%/50, 5%/50, and 2%/50 hazard levels. These hazard levels correspond to the 475-year, 975-year, and 2,475-year return periods, respectively. The design spectra was then calculated following Clause 4.4.3.4 of the Code, using the UHS values at each hazard level and the appropriate site coefficients from Clause 4.4.3.3. Since the abutments were not specifically designed for sustained soil mobilization, according to the Code, 5% damped spectral response acceleration values should be used (Clause 4.4.3.5).

The 5% damped design spectra of the bridge at the specified hazard levels are shown in **Figure B1-3**. These calculated spectra were used for the response spectrum analysis (RSA) of the bridge.

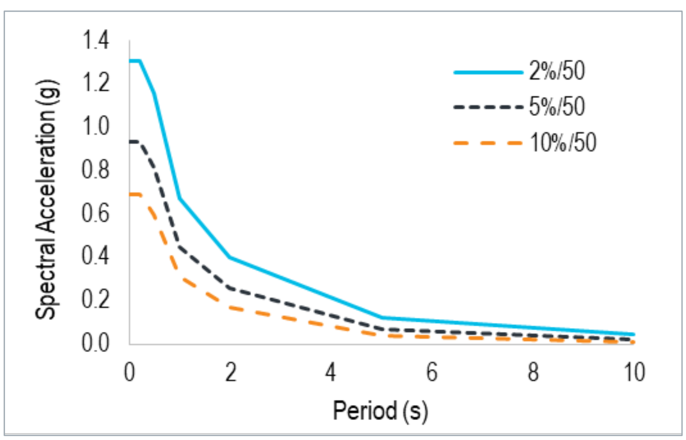

Figure B1 - 3: Design spectrum for the three hazard levels (2%/50, 5%/50, 10%/50)

B1.4 PERFORMANCE REQUIREMENTS

B1.4.1 SEISMIC PERFORMANCE CATEGORY

The fundamental period of the bridge in both of the longitudinal and lateral directions is greater than 0.5 s. The seismic performance category (SPC) of a major route bridge with $T \geq 0.5$ s and $S(1.0) \geq 0.3$ is SPC 3 (Code, Clause 4.4.4).

B1.4.2 REGULARITY AND MINIMUM ANALYSIS REQUIREMENTS

According to the definition in the Code, Clause 4.4.5.3.2, this case study bridge is classified as a regular bridge. The minimum analysis requirements of a regular major route bridge in SPC 3 is elastic dynamic analysis at 2%/50 and 5%/50 hazard levels, and elastic static analysis at 10%/50 hazard level (Code, Clause 4.4.5.3.1, Tables 4.12 and 4.13).

For this case study, RSA was performed to obtain the seismic demands on the bridge at the specified hazard levels. In addition, inelastic static pushover analysis was used to obtain the sequence of plastic hinge formation in the ductile members (that is, columns), and the drift capacities corresponding to the first occurrence of the considered performance criteria.

B1.4.3 MINIMUM PERFORMANCE LEVELS

The minimum performance levels for major route bridges in terms of tolerable structural damage are 'minimal' at 10%/50, 'repairable' at 5%/50, and 'extensive' at 2%/50 hazard level (Code, Clause 4.4.6.2). The minimum serviceability objectives for these performance levels are 'immediate,' 'service limited,' and 'service disruption,' respectively.

B1.4.4 POSSIBLE FAILURE MECHANISMS

Before setting the performance criteria, the possible local and global failure mechanisms should be determined. Following are four possible failure mechanisms:

1. Ductile failure of the columns in flexure (local failure).
2. Brittle failure of the columns in shear (local failure).
3. Unseating of the deck at the abutments in the longitudinal direction (global failure).
4. Pounding between the deck and the abutments (global failure).

Other failure mechanisms, such as foundation soil failure and abutment backfill soil failure, should also be considered; these are outside the objectives of this case study.

B1.4.5 PERFORMANCE CRITERIA

B1.4.5.1 Flexural Failure of the Columns

The performance requirements of the Code for the flexural response of ductile reinforced concrete members are stated in terms of reinforcement steel and concrete strain limits. Each strain limit represents the initiation of a damage state in ductile concrete members.

The relevant strain limits for each performance level are listed in **Tables B1-1** and **B1-2**. In the tables, ε_c and ε_s are concrete and reinforcement steel strains, respectively.

Table B1 - 1: Strain Limits Associated with the Performance Levels of a Major Route Bridge, According To the Code and Supplement

HAZARD	PERFORMANCE LEVEL	CODE[a]	SUPPLEMENT[b]
10%/50	Minimal damage	ε_c >-0.004, $\varepsilon_s < \varepsilon_y$	ε_c >-0.006, ε_s <0.010
5%/50	Repairable damage	ε_s <0.015	ε_s <0.025
2%/50	Extensive damage	ε_c >-0.0163, ε_s <0.050	ε_c >-0.0130, ε_s <0.050

Notes:

ε_c = concrete steel strains; ε_s = reinforcement steel strains

[a] CSA 2014

[b] BC MoTI 2016

Table B1 - 2: Strain Limits Associated with the Flexural Damage States of Reinforced Concrete Columns

DAMAGE STATE		STRAIN LIMIT (M/M)
(1)	Yielding	ε_s < 0.0024
(2)	Cover spalling 1	ε_c < -0.004
(3)	Cover spalling 2	ε_c < -0.006
(4)	Serviceability limit 1	ε_s < 0.01
(5)	Serviceability limit 2	ε_s < 0.015
(6)	Reduced buckling	ε_s < 0.025
(7)	80% Core crushing	ε_c < -0.0130
(8)	Core crushing	ε_c < -0.0163
(9)	Reduced fracture	ε_s < 0.05

Notes:

ε_c = concrete steel strains; ε_s = reinforcement steel strains

These damage states can be described as follows (numbers correspond to **Table B1-2**):

- (1) Yielding of the longitudinal rebars
- (2, 3) Spalling of the cover concrete
- (4) Longitudinal reinforcement strain that cause minimal damage
- (5) Serviceability limit state of the longitudinal rebars, which corresponds to residual crack width exceeding 1 mm (Kowalsky 2000)
- (6) Preventing buckling in the longitudinal rebars
- (7, 8) Crushing of the core concrete
- (9) Initiation of buckling in the longitudinal rebars (Goodnight et al 2013) and preventing the fracture of the previously buckled rebars

The ultimate strain capacity of confined concrete can be calculated using the formula in Priestley et al. (1996), as follows:

$$E_{cu}=0.004+1.4\frac{\rho_s f_{yh}\varepsilon_{fs}}{f'_{cc}} \tag{1}$$

In the above expression, ρ_s is the spiral reinforcement ratio, f_{yh} is the spiral yield strength, ε_{fs} is the spiral fracture strain, and f'_{cc} is the confined concrete compressive strength.

For ε_{fs}, a value of 0.09 can be used in the formula, following the recommendation in the Caltrans Seismic Design Criteria (SDC) (Caltrans 2013), for the reduced ultimate tensile strain of Grade 400 #10 (Metric #32) rebars or smaller. The value of f'_{cc} can be obtained using the Mander et al. (1988) constitutive model.

Some programs have a built-in module to calculate confinement factor from the inputs for a section. The confinement factor for the column cross-section in the plastic hinge region is 1.288, which, multiplied by the expected compressive strength of f'_{ce}=43.75 MPa, yields f'_{cc}=56.35 MPa. Substituting all values in the above expression gives an ultimate compressive strain capacity of -0.0163 for the plastic hinge region.

When using the tabulated strain limits of **Table B1-2** for performance assessment, it should be noted that these values are conservative. For instance, the ultimate compressive strain capacity of equation (1) is observed to be consistently conservative by about 50% (Kowalsky 2000).

B1.4.5.2 Shear Failure of the Columns

The brittle shear failure of the columns is checked by comparing the shear demand versus capacity of the columns. Clause 4.4.10.4.3 of the Code defines the shear demand as either the unreduced elastic design shear, or the shear corresponding to inelastic hinging of the columns, calculated by using probable flexural resistance of the member and its effective height (CSA 2014). However, this has been modified in the Supplement to exclude the former method (BC MoTI 2016).

The shear capacity of concrete can be calculated using either the simplified method with β=0.1 and θ=45° (Clause 4.7.5.2.4), or by using the general method, which modifies the shear capacity based on the member axial strain (Clause 8.9.3.7). The Supplement allows using more refined methods to calculate seismic shear capacity, which modify the shear capacity based on ductility demands.

B1.4.5.3 Unseating and Pounding of the Deck with the Abutments

To check the last two failure mechanisms, the longitudinal displacement at the deck level should meet the following two criteria:

$$\Delta_{deck} \leq L_{expansion} \tag{2}$$

$$\Delta_{deck} \leq N \tag{3}$$

In the above expressions, $L_{expansion}$ is the length of the longitudinal gap and N is the provided support length at the abutments.

B1.5 MODELLING

A 3D spine model of the bridge was generated in SAP2000. Expected material properties were used in the definition of steel and concrete materials.

The behaviour of the unconfined and confined concrete was modelled with the Mander et al. (1988) constitutive model. The program automatically calculates and applies the confinement factor to the confined concrete material from the input information of a section.

Two models were used for the bridge: an elastic model with effective material properties for RSA and modal analysis, and a non-linear fibre hinge model for the pushover analysis. The two models differ in how they represent the non-linear behaviour of the substructure ductile elements (that is, columns), but both use similar superstructure models and boundary conditions.

B1.5.1 ELASTIC CRACKED MODEL

For the elastic model, cracked section properties of the columns were calculated based on the moment-curvature analysis of the columns section, and the flexural and shear stiffness modifiers were applied to the column frame elements accordingly. The cap beam was modelled using elastic frame elements with cracked section properties.

Following Clause 4.4.5.3.3 of the Code, the effective flexural stiffness can be calculated from the moment-curvature response of the column section (**Figure B1-4**) as the slope of the line connecting the origin to the point of first yield in the longitudinal rebars.

This will give E_cI_{eff}=0.456 E_cI_g. A similar stiffness modifier was obtained for the effective shear stiffness of the columns. A property modifier of 0.2 was also applied to the torsional constant of the column, following Caltrans SDC recommendations (Caltrans 2013). The flexural stiffness of the cap beam was also modified by a 0.5 factor.

Since the superstructure steel girders were capacity-protected, it was assumed that they would remain essentially elastic under seismic loading. Therefore, the steel girders and the concrete deck slab were modelled using elastic frame elements with composite section properties, as calculated in **Table B1-3**. A nominal linear spring was assigned to the ends of the deck in the lateral direction, to mimic the restraining effect of shearkeys and remove the unrealistic modes of vibration in that direction. In the longitudinal direction, the deck is free to move, and simplified roller boundary conditions were employed to model the seat-type abutments. Fixed-boundary conditions at the column foundations were assumed, as mentioned above.

Table B1 - 3: Composite Section Properties of the Deck at Different Sections

	SECTION 1	SECTION 2	SECTION 3
Equivalent Steel Area (m^2)	1.61	1.65	1.82
Dead Load (kN/m)	124	127	140
$I_{vertical}$ (m^4)	0.82	0.91	0.96
$I_{transverse}$ (m^4)	8.20	8.50	9.00

B1.5.2 FIBRE HINGE MODEL

Unlike OpenSees© and SeismoStruct®, SAP2000 does not have the option of distributed plasticity models. Instead, non-linear behaviour can be modelled with concentrated plasticity models, assigning plastic hinges with a specified length to elastic frame elements.

Fibre hinges were employed here to model the non-linear response of the columns. This model is able to capture post-yield degradation and softening, but is unable to model pinching and bond slip effects. The shear and torsion behaviour of the cross-section are elastic. So the loss of shear stiffness should be captured by applying shear area modification factors to the elastic frame elements.

The plastic hinge length assigned to fibre hinges can be calculated using the following expression of Paulay and Priestley (1992), which is recommended in the Caltrans SDC (Caltrans 2013):

$$L_p = 0.08L + 0.022 f_{ye} d_b > 0.044 f_{ye} d_b \text{ (mm, MPa)} \quad (4)$$

In this formula, L is the member length from the point of maximum moment to the point of contra-flexure, f_{ye} is the expected yield strength of the longitudinal rebars, and d_b is the nominal diameter of the longitudinal rebars.

The fibre hinge can be assigned to the mid-height of the plastic hinge zone, assuming that the plastic curvature remains constant in the plastic hinge zone. Using the above formula, the plastic hinge length for the longitudinal direction with single curvature is 1,089 mm, and for the lateral direction with double curvature is 754 mm.

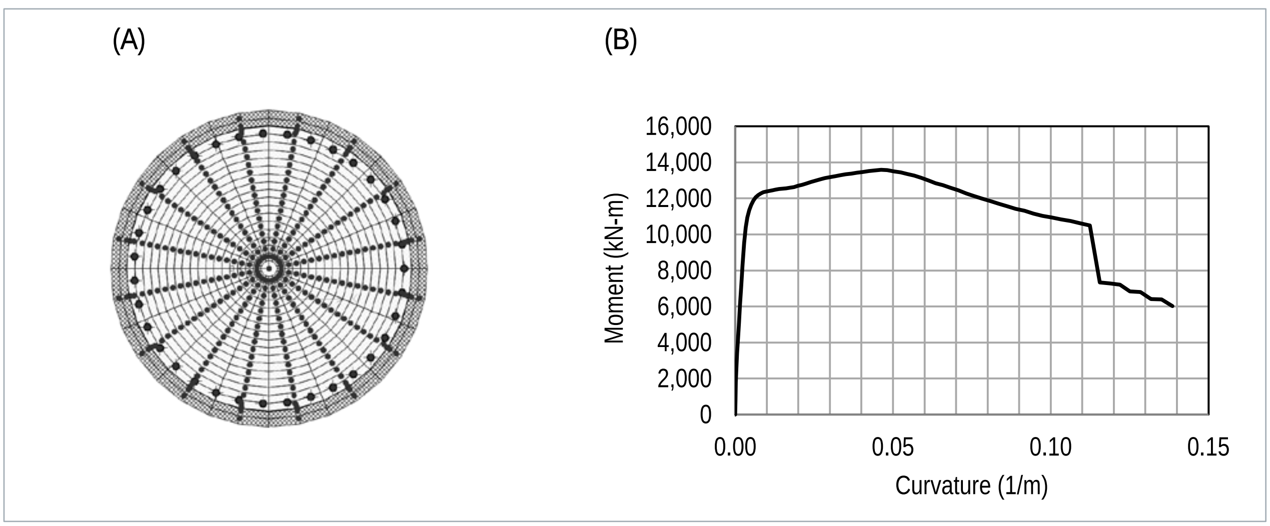

Figure B1 - 4: (A) Fibre cross-section of the columns in SAP2000, and (B) plastic hinge moment-curvature (response under dead load).

B1.6 ANALYSIS

B1.6.1 RESPONSE SPECTRUM ANALYSIS

The seismic demands on the bridge structure were obtained using RSA. The fundamental period of the bridge in the longitudinal and lateral directions were calculated as 1.58 s and 0.53 s, respectively.

At each hazard level, two load cases were considered, according to the following instructions in Clause 4.4.9.2 of the Code (CSA 2014):

> "The horizontal elastic seismic effects on each of the principal axes of a component resulting from analyses in the two perpendicular horizontal directions shall be combined within each direction from the absolute values to form two load cases as follows:
>
> a) 100% of the absolute value of the effects resulting from an analysis in one of the perpendicular directions combined with 30% of the absolute value of the force effects from the analysis in the second perpendicular direction.
>
> b) 100% of the absolute value of the effects from the analysis in the second perpendicular direction combined with 30% of the absolute value of the force effects resulting from the analysis in the first perpendicular direction."

Therefore, the seismic load combination included 125% to 80% dead load, 100% seismic load in one direction, and 30% seismic load in the orthogonal direction (see Clause 3.5.1 for load combinations). For modal combination of the seismic effects, the SRSS (square root of the sum of squares) rule was applied, since the contributing modes were well separated.

B1.6.2 PUSHOVER ANALYSIS

The extent of the flexural damage in the columns can be predicted by checking the maximum relative drift ratios of the columns from RSA against the relative drift ratios corresponding to the first occurrence of each of the damage states.

Separate pushover analyses were conducted on the bridge structure in the longitudinal and transverse directions. The structure was pushed to the point of failure, indicated by significant reduction in the strength capacity of the columns. The drift ratios corresponding to the first occurrence of each damage state in the columns were considered as the limiting drift ratios for those damage states. This can be obtained by checking the fibre hinge strains against the strain limits in **Table B1-2**. **Table B1-4** shows the obtained drift ratio capacity of the columns at each of the considered damage states.

Table B1 - 4: Column Drift Ratio Capacities Associated with the First Occurrence of the Damage States

PERFORMANCE CRITERIA	LONGITUDINAL DRIFT (%)	LATERAL DRIFT (%)
Yielding	0.82	0.51
Cover spalling 1	1.82	1.15
Serviceability limit 1	1.80	1.31
Cover spalling 2	2.40	1.68
Serviceability limit 2	2.23	1.83
Reduced buckling	3.26	2.88
80% Core crushing	5.17	4.01
Core crushing	6.24	4.93
Reduced fracture	5.82	5.59

B1.7 RESULTS AND DISCUSSION

The maximum drift ratios of the columns in the longitudinal and transverse directions from RSA, along with the predicted level of damage, are summarized in **Table B1-5**.

The results indicate that the bridge undergoes yielding in the lateral direction, while the endured level of damage in the longitudinal direction is much higher. This is due to the fact that in the lateral direction, the bridge benefits from the framing action and the restraining effect of the shear keys. The lower period of the bridge in this direction imposes lower displacement demands on the structure as well. On the contrary, in the longitudinal direction, the bridge primarily acts as a cantilever; therefore, the imposed displacement demand is considerably larger.

To verify the performance of the columns under the flexure failure mechanism, the ratio of drift demands to drift capacities for each of the performance criteria in **Table B1-2** were calculated (**Table B1-6**).

The drift demand-to-capacity ratios were obtained considering performance criteria from both the Code and the Supplement. The following can be concluded:

- Employing the Code criteria, the bridge meets the specified performance criteria at the 2%/50 and 5%/50 hazard levels with acceptable reserve capacity, while it fails to meet the yielding criteria at the 10%/50 hazard level.
- Employing the Supplement criteria, the bridge meets all the specified performance criteria at all hazard levels with reasonable reserve capacity.
- The controlling performance criteria using both the Code and the Supplement is at the 10%/50 hazard level.
- The calculated reserve capacities at different hazard levels are more uniform using the Supplement criteria compared to the Code criteria.

The maximum longitudinal displacements of the deck at the three hazard levels are listed in **Table B1-7**.

The provided support length and the longitudinal gap should be checked against these values. The large displacements at the 2%/50 and 5%/50 hazard levels indicate the possibility of pounding between the deck and the abutments. This can be rectified using one of the following three options:

1. Incorporating elastomeric bearings at the abutments to control the longitudinal displacements of the girders.
2. Redesigning the abutment to semi-integral.
3. Reducing the longitudinal drifts of the columns by increasing the column cross-sections, and therefore increasing the longitudinal stiffness.

The shear capacity of the columns was also checked against the shear demand and it passed the criteria. However, the details of the calculations are not presented here, as they were carried out using similar methods employed in the force-based design approach.

Table B1 - 5: Column Drift Demands from RSA in the Longitudinal (X) and Transverse (Y) Directions, and the Predicted Damage

HAZARD LEVEL	D_X (%)	DAMAGE	D_Y (%)	DAMAGE
2%/50	3.23	SL2	0.98	Y
5%/50	2.14	SP1	0.66	Y
10%/50	1.46	Y	0.51	Y

Notes:
SL2: serviceability limit 2; SP1: cover spalling 1; Y: yielding of longitudinal reinforcements; RSA = response spectrum analysis

Table B1 - 6: Ratio of the Drift Demand to Drift Capacity of the Columns in the Longitudinal (x) and Lateral (y) Directions, and the Reserve Drift Capacity for Each Hazard Level

	CODE[a]			SUPPLEMENT[b]		
HAZARD LEVEL	Δ_D/Δ_C (%)-X	Δ_D/Δ_C (%)-Y	RESERVE (%)	Δ_D/Δ_C (%)-X	Δ_D/Δ_C (%)-Y	RESERVE (%)
2%/50	55.5	19.9	44.5	62.5	24.5	37.5
5%/50	95.6	38.1	4.4	65.5	24.2	34.5
10%/50	179.9	100.0	-79.9	81.2	38.7	18.8

Notes:
[a] CSA 2014
[b] BC MoTI 2016

Table B1 - 7: Maximum Longitudinal and Lateral Displacement of the Deck

HAZARD LEVEL	Δ_{DECK-X} (M)	Δ_{DECK-Y} (M)
2%/50	0.317	0.063
5%/50	0.209	0.018
10%/50	0.143	0.043

B1.8 REFERENCES

Applied Technology Council. 1996. ATC-32 Improved Seismic Design Criteria for California Bridges: Provisional Recommendations. Redwood City, CA: ATC. 215 pp.

BC Ministry of Transportation and Infrastructure (BC MoTI). 2016. Bridge Standards and Procedures Manual. Supplement to CHBDC S6-14. (October 2016). Victoria, BC: BC MoTI.

California Department of Transportation (Caltrans). 2013. Seismic Design Criteria. April 2013. Version 1.7. Sacramento, CA: Caltrans.

Canadian Standards Association (CSA). 2014. Canadian Highway Bridge Design Code. CSA Standard S6-14. (December 2014). Toronto, ON: Canadian Standards Association.

Goodnight JC, Kowalsky MJ, Nau JM. 2013. Effect of Load History on Performance Limit States of Circular Bridge Columns. J Bridge Eng. 18(12):1383-1396.

Kowalsky MJ. 2000. Deformation Limit States for Circular Reinforced Concrete Bridge Columns. J Struct Eng. 126(8):869-878.

Mander JP, Priestley MJN, Park R. 1988. Theoretical Stress-Strain Model for Confined Concrete. J Struct Eng. 114(8):1804-1826.

Natural Resources Canada (NRC). 2015. 2015 National Building Code of Canada Seismic Hazard Calculator. [website]. Ottawa, ON: NRC. [date modified: 2016-02-10]. [accessed 2018 Jan 18]. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php.

Paulay T, Priestley MJN. 1992. Seismic Design of Reinforced Concrete and Masonry Buildings. New York, NY: Wiley.

Priestley MJN, Seible F, Calvi GM. 1996. Seismic Design and Retrofit of Bridges. New York, NY: John Wiley and Sons.

APPENDIX B2: PERFORMANCE-BASED DESIGN OF AN EXTENDED PILE CONCRETE BENT HIGHWAY BRIDGE

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B2.1 INTRODUCTION

The CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014) initiated performance-based design (PBD), which requires that engineers explicitly demonstrate structural performance. In the 2014 edition of the Code, the force-based design (FBD) method is only permitted for certain cases, whereas PBD may be used in all cases.

In the previous edition, CAN/CSA-S6-06 (CSA 2006), bridges were designed using the FBD method. FBD calculates the seismic force demands by either single-mode or multi-mode spectral method for most of the bridge categories. The base shear force is reduced to the design base shear level using a force reduction factor R. Then the structure is designed according to this reduced force.

However, the current FBD method has several short-comings (Priestley et al. 2007). The major limitation of the FBD method is that it cannot explicitly demonstrate the performance of the bridges. However, in PBD, designs are checked using non-linear analyses so structural performance can be explicitly demonstrated.

This case study compares FBD and PBD of an extended pile concrete bent highway bridge and shows the basic steps required to explicitly demonstrate some of the performance criteria. In the Code, performance criteria include concrete and steel strains, damage states of bearings and joints, and other structural elements. The criteria considered in this case study are mainly material strains.

B2.2 PERFORMANCE OBJECTIVES

PBD relates performance objectives to the design process. For the specified response parameter criteria, the Code uses material strains. The damage states from the Code are briefly described in **Table B2-1**.

After determining performance levels at the beginning of the design, the performance criteria are assigned to different levels of earthquake events for different bridge categories. The bridge category is usually defined based on the importance of the bridge. In the Code, there are three categories: lifeline bridges, major route bridges, and other bridges.

The bridge in this case study is a major route bridge. A major route bridge is described as one that is a crucial part of the regional transportation and is critical to post-disaster event and security. Based on the category of the bridge, performance levels are assigned to achieve the PBD goals.

Table B2 - 1: CAN/CSA-S6-14 Performance Criteria[a]

LEVEL	SERVICE	DAMAGE	CRITERIA
1	Immediate	Minimal damage	• Concrete compressive strains (ε_c) ≤ 0.004 • Steel strains (ε_{st}) ≤ yield strain (ε_y)
2	Limited	Repairable damage	• Steel strains (ε_{st}) ≤ 0.015
3	Service disruption	Extensive damage	• Confined core concrete strain (ε_{cc}) ≤ concrete crushing strain (ε_{cu}) • Steel strains ≤ 0.05
4	Life safety	Probable replacement	• Bridge spans shall remain in place but the bridge may be unusable and may have to be extensively repaired or replaced

Note:
[a] CSA 2014

B2.3 BRIDGE DESCRIPTION

The bridge is a multi-span concrete bridge with multi-column bents located in Burnaby, British Columbia (BC), Canada. The total span of the bridge is 100 m and the width of the bridge is 40 m. The bridge has three bents as piers and two bents providing support as abutments. Each bent has eight columns that are supported by individual piles. The clear height of each column is approximately 5 m and the length of each pile is approximately 20 m. The soil-structure interaction was considered in the bridge design and performance assessment.

B2.3.1 BRIDGE MODELLING

In the design phase, the bridge model was built in SAP2000® (version 8) (CSI 2010) and the soil-structure interaction was simulated by using a series of p-y springs.

The finite element model of the bridge is shown in **Figure B2-1**. The first and second mode shapes are shown in **Figures B2-2** and **B2-3**. Site-specific response spectra were used for the design; the spectral accelerations are shown in **Figure B2-4**.

In this case study, the shallow soil is not strong enough to resist loads from the bridge; hence, pile foundations are used. The soil-structure interaction is an important factor that affects the seismic performance of the bridge.

In p-y curves, p stands for lateral resistance force per unit pile length from the soil, and y stands for lateral displacement of piles. **Figure B2-5** shows a typical p-y curve where the soil loses its strength and stiffness with the increase of displacement.

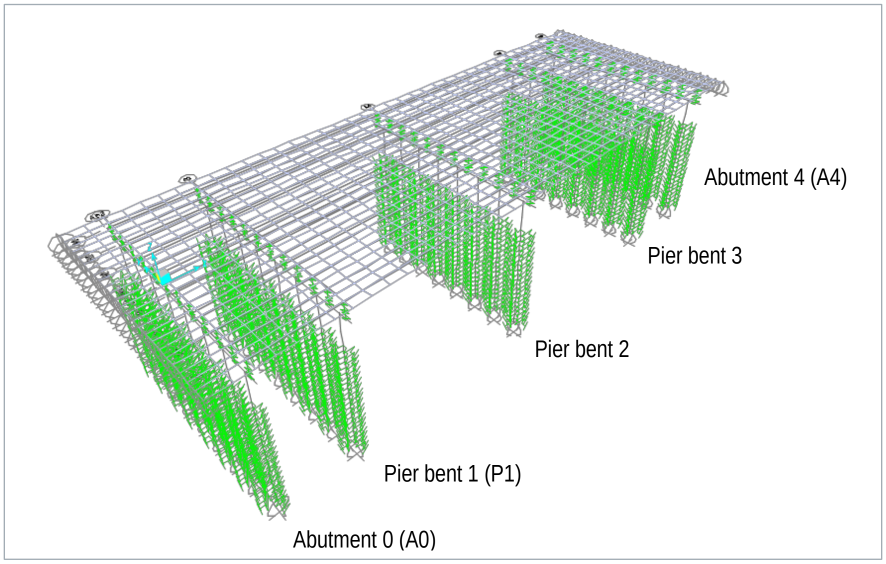


Figure B2 - 1: Finite element model in SAP 2000 (adapted from Zhang et al. 2016)

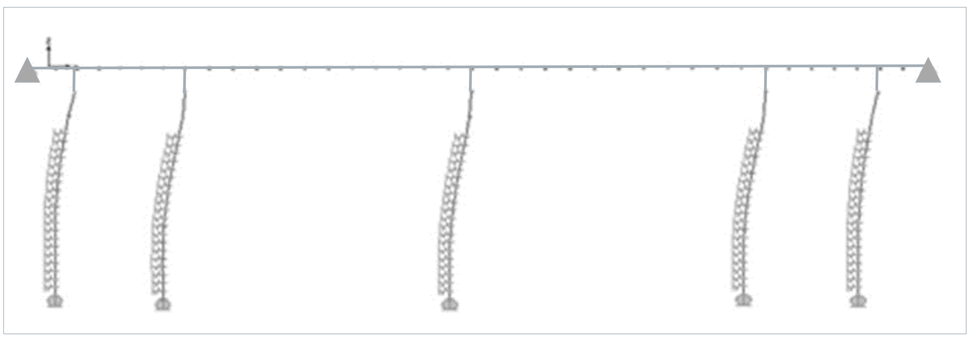


Figure B2 - 2: First mode shape (longitudinal direction)

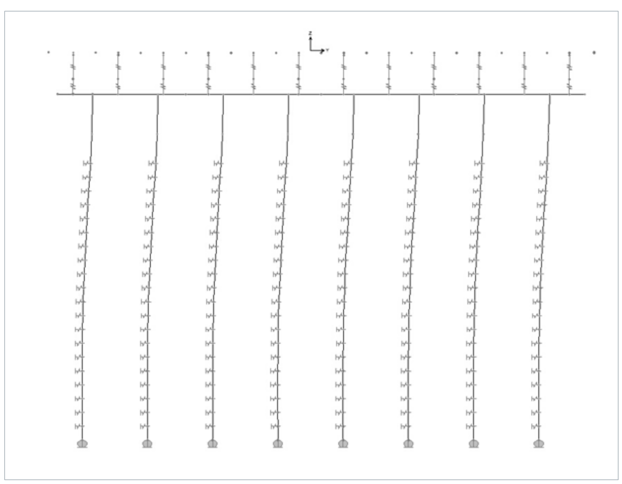


Figure B2 - 3: Second mode shape (transverse direction)

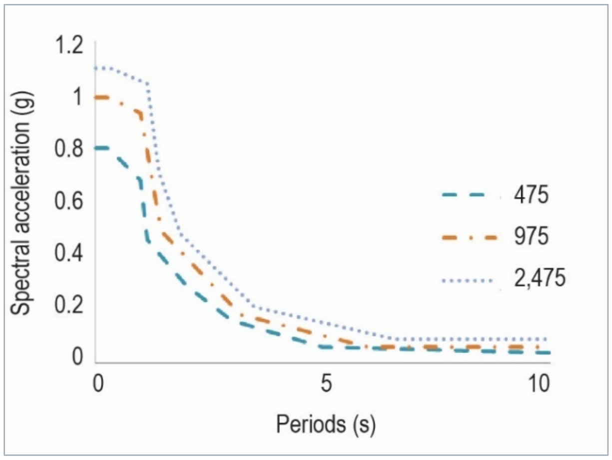


Figure B2 - 4: Response spectra (adapted from Zhang et al. 2016)

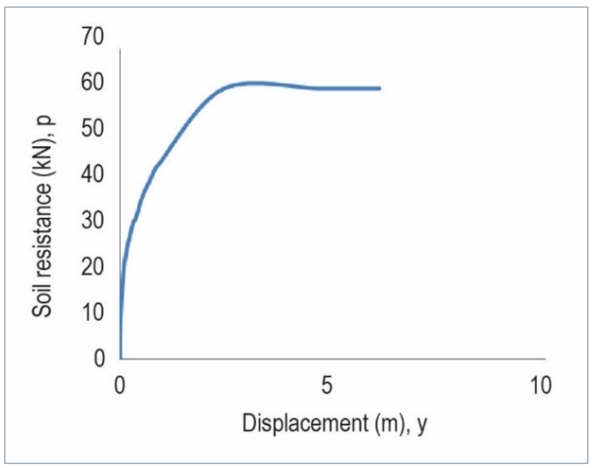


Figure B2 - 5: Typical p-y spring (adapted from Zhang et al. 2016)

B2.4 FORCE-BASED DESIGN AND PERFORMANCE-BASED DESIGN PROCESS

B2.4.1 FORCE-BASED DESIGN PROCESS

In FBD, forces are calculated based on cracked stiffness, which can be estimated at the beginning of the design. Then a force reduction factor is used to represent the ductility capacity. The reduced force is used for seismic design. Determining the final displacement of the soil springs is carried out iteratively to arrive at seismic demand compatible spring stiffnesses to appropriately determine the modal periods and the associated final bases shear values.

It should be noted that steps such as initial sizing, cracked stiffness determination for analysis, and soil spring iterations for RSA apply equally to FBD and PBD. The flowcharts of the steps are shown in the main body of these guidelines in **Figure 2**, found in **Section 5.2.1 Framework for the PBD Process**.

B2.4.1.1 Initial Sizing

First, the number and size of columns should be determined. A simple method to determine the size and number of the column is to maintain a 10% column axial load ratio. In this case study, it was determined that eight columns per bent and five bents in total would be appropriate for the bridge. Based on the 10% axial load ratio from top of the columns, the size of the column was assumed to be 0.914 m for a FBD. Initial sizing may also be governed by non-seismic load requirements.

B2.4.1.2 Cracked Stiffness

Then cracked stiffness is used to consider the reduction of stiffness. The stiffness is estimated based on axial load ratio and reinforcement ratio. The cracked stiffness can be initially found from the chart produced by Priestly et al. (1996) or, more precisely, from the moment-curvature analysis. **Figure B2-6** shows the chart adapted from Priestly et al. (1996).

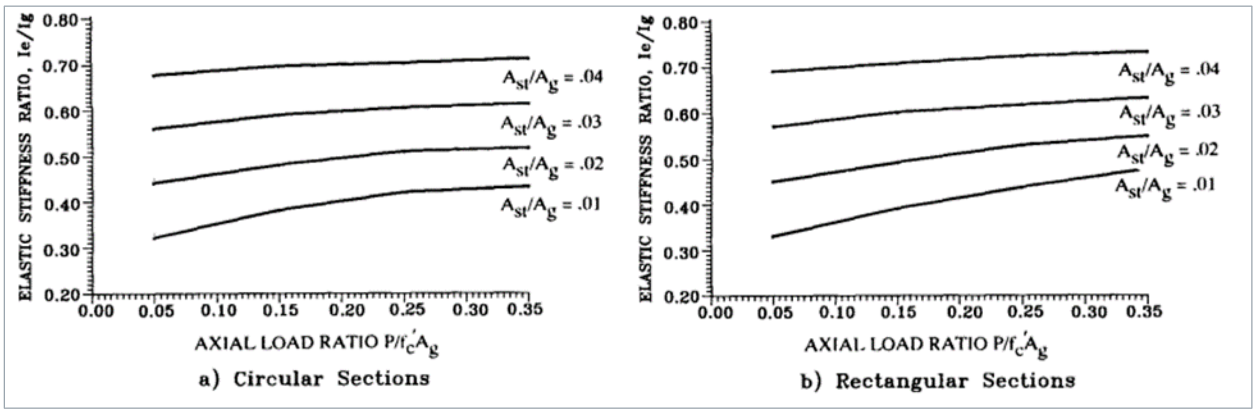


Figure B2 - 6: Column cracked stiffness (adapted from Priestly et al. 1996)

In the next step, the periods may be calculated from stiffness and mass of the structures using Equation 1. The equation is written as:

$$T=2\pi\sqrt{\frac{m}{K}} \quad (1)$$

where m is the effective mass and K is the stiffness. Because the soil spring stiffness changes with the change of lateral load, the bridge has different fundamental periods at different earthquake events.

B2.4.1.3 Modelling and Analysis

The bridge model was built in SAP2000 (CSI 2010) for modal analysis and RSA. RSA is a linear analysis, so only a linear soil spring can be used in the model.

Since the soil loses strength and stiffness with the increase of lateral load, effective spring stiffness is used for the design. The effective stiffness of springs can be determined by conducting modal analysis and RSA iteratively. At the beginning of the spring iterations, initial stiffness is defined and RSA is conducted. The displacement of springs can be calculated from spectrum analysis, and then another set of spring stiffness can be calculated based on the new displacement and the compatible spring force.

This process is repeated until spring force and displacement values change minimally between subsequent iterations. At different earthquake events, there should be different sets of soil springs that are iteratively determined for each event due to specific demands. Periods and elastic forces from acceleration spectrum were determined with the converged spring stiffness at the end.

B2.4.1.4 Force Reduction Factor

As the next step, the force reduction factor is defined by design codes and incorporated into the FBD. In this example, a factor of 5.0 is used. The elastic flexural demands for the columns are reduced by the force reduction factor and the columns are designed for these demands. The column shear demand can be determined by the lesser of elastic force or the actual force that causes the columns to form plastic hinges. The interaction between axial load, moment, and shear should be considered.

B2.4.2 PERFORMANCE-BASED DESIGN PROCESS

In the Code (CSA 2014), the PBD process requires explicit performance demonstration. The major consideration is that inelastic static pushover analysis or non-linear time history is required to assess and demonstrate structural performance in PBD.

Here, the material strain is one of the most important criteria in determining seismic performance. For major route bridges, the damage should be limited within the minimal, repairable, and extensive damage levels, corresponding to the 1 in 475, 1 in 975, and 1 in 2,475-year return period events. It should be noted that the preliminary member sizing may be based on any design methods including FBD.

At the beginning of PBD, to determine which material governs the 1 in 475-year return period design, a simple section analysis can be conducted. A simple example is shown here, assuming the diameter of column section is 914 mm and the height is 6 m. The concrete strength is 35 MPa and reinforcement yielding stress is 400 MPa. Concrete cover thickness is 70 mm, and the spiral is assumed to be 15M@75 mm, while the axial load ratio of the column is assumed at 10%.

From the section analysis, it was shown that when steel strain reaches 0.002, the concrete strain was lower than 0.004. Therefore, steel strain governs the design. **Table B2-2** shows corresponding concrete and steel strain values for columns with 1% and 2% rebar ratio. The calculation was performed using XTRACT (version 2004). **Table B2-1** shows that concrete strain does not generally govern the design. When concrete strain reaches 0.004, steel strain is around 0.01. When steel reaches yielding, the concrete strain is between 0.0012 and 0.0014.

There are different approaches to calculate the shear capacity of reinforced columns (CSA 2014; ATC 1996; Priestley et al. 1996). The shear capacity of concrete can be reduced by flexural ductility, and it can also be affected by axial load ratios.

The shear capacity calculated from the Code was 1,577 kN. **Tables B2-3** and **B2-4** compare the column shear capacity calculated based on different approaches.

Table B2 - 2: Corresponding Concrete and Steel Strains

1% REINFORCEMENT RATIO		2% REINFORCEMENT RATIO	
CONCRETE	STEEL	CONCRETE	STEEL
0.00127	0.002	0.0014	0.002
0.0026	0.005	0.0029	0.005
0.0039	0.01	0.0046	0.01
0.006	0.015	0.007	0.015
0.004	0.011	0.004	0.0085
0.005	0.013	0.005	0.011
0.007	0.025	0.0088	0.025
0.015	0.05	0.0179	0.05

Table B2 - 3: Priestley et al. (1996) Equation

EVENT	DUCTILITY	SHEAR CAPACITY (kN)
475-year event	2.7	3,165
975-year event	4	2,870
2,475-year event	10	2,668
Steel strain=0.015	6.3	2,823
Steel strain=0.05	18	2,420

Table B2 - 4: ATC-32 Equation (ATC 1996)

AXIAL LOAD RATIO	SHEAR CAPACITY (kN)
0.2	1,810
0.1	1,684
0	1,557

B2.4.2.1 Column Design

Two FBDs were conducted according to CAN/CSA-S6-06 (CSA 2006) and the Code (CSA 2014), which are denoted as case numbers D1 and D2, respectively. One PBD was conducted according to the Code, which is denoted as case number D3.

The Code requires that an importance factor of 1.5 be considered for major route bridges in FBD. The design for FBD has to correspond to the 1 in 2,475-year return period event. The three design results are shown in **Table B2-5** and **Figure B2-7**.

Comparing the two FBDs, case D2 has a higher reinforcement ratio due to the longer return period and the importance factor of 1.5. It can be seen that the Code results in higher reinforcement ratio compared with CAN/CSA-S6-06. This conclusion may also apply to other similar design cases.

However, for case D3, the designed longitudinal reinforcement is extremely high, although the diameter of the column was increased to 1.2 m to reduce displacement demands. This is mainly driven by the no rebar yield requirement in the Code, corresponding to the 1 in 475-year return period event.

Table B2 - 5: Design Cases

CASE NO.	DESIGN METHOD	DESIGN CODE CAN / CSA-S6	COLUMN DIAMETER (M)	PIER LONGITUDINAL REINFORCEMENT RATIO	RETURN PERIOD (YEARS)	LONGITUDINAL PERIOD (S)	TRANSVERSE PERIOD (S)
D1	FBD	2006	0.914	1.9%	475	1.984	1.787
D2	FBD	2014	0.914	2.7%	2,475	2.244	2.068
D3	PBD	2014	1.2	5.3%	475	1.598	1.362
					975	1.621	1.422
					2,475	1.700	1.474

Notes: FBD = force-based design; PBD = performance-based design

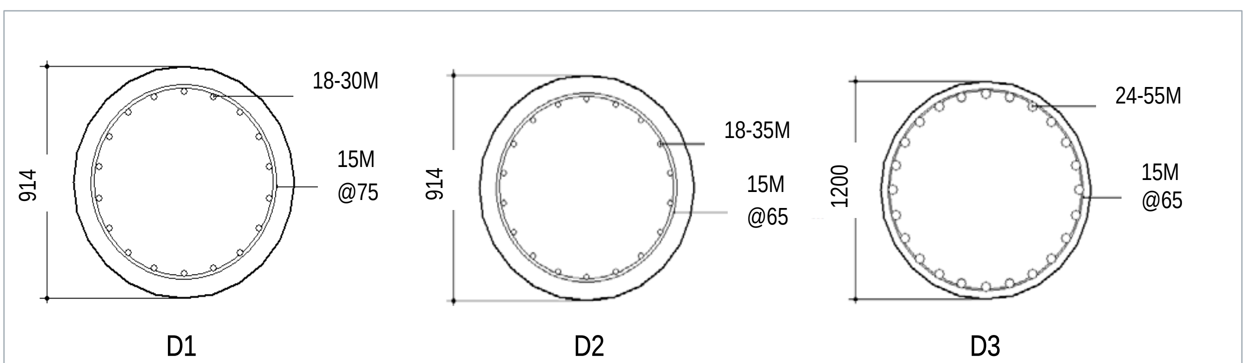


Figure B2 - 7: Column section (adapted from Zhang et al. 2016)

B2.5 PUSHOVER ANALYSIS

To assess the performance of the bridge, a pushover analysis was conducted in the transverse direction of each bent. Bents were pushed to the displacement demands calculated from the RSA. In the non-linear pushover analysis, the plasticity can be considered using distributed plasticity models or lumped plasticity models, which are incorporated into a number of programs. For example, SeismoStruct® uses distributed plasticity models, whereas SAP2000 uses lumped plasticity models. In this example, the pushover analysis was carried out using SeismoStruct (SeismoSoft 2010). However, SAP2000 is also briefly presented for comparison.

SeismoStruct is a fibre-based program capable of carrying out non-linear analysis. Performance criteria such as strains can be directly obtained from SeismoStruct. In SAP2000, the steel strain can be calculated from the plastic rotation, plastic hinge length, and moment-curvature analysis.

In pushover analysis with lumped plasticity, plastic hinges can be defined by designers. **Figure B2-8** shows an example moment-rotation curve in SAP2000. After running the pushover analysis, the hinge results will show the plastic rotations of the hinges. An example of this is shown in **Figure B2-9**.

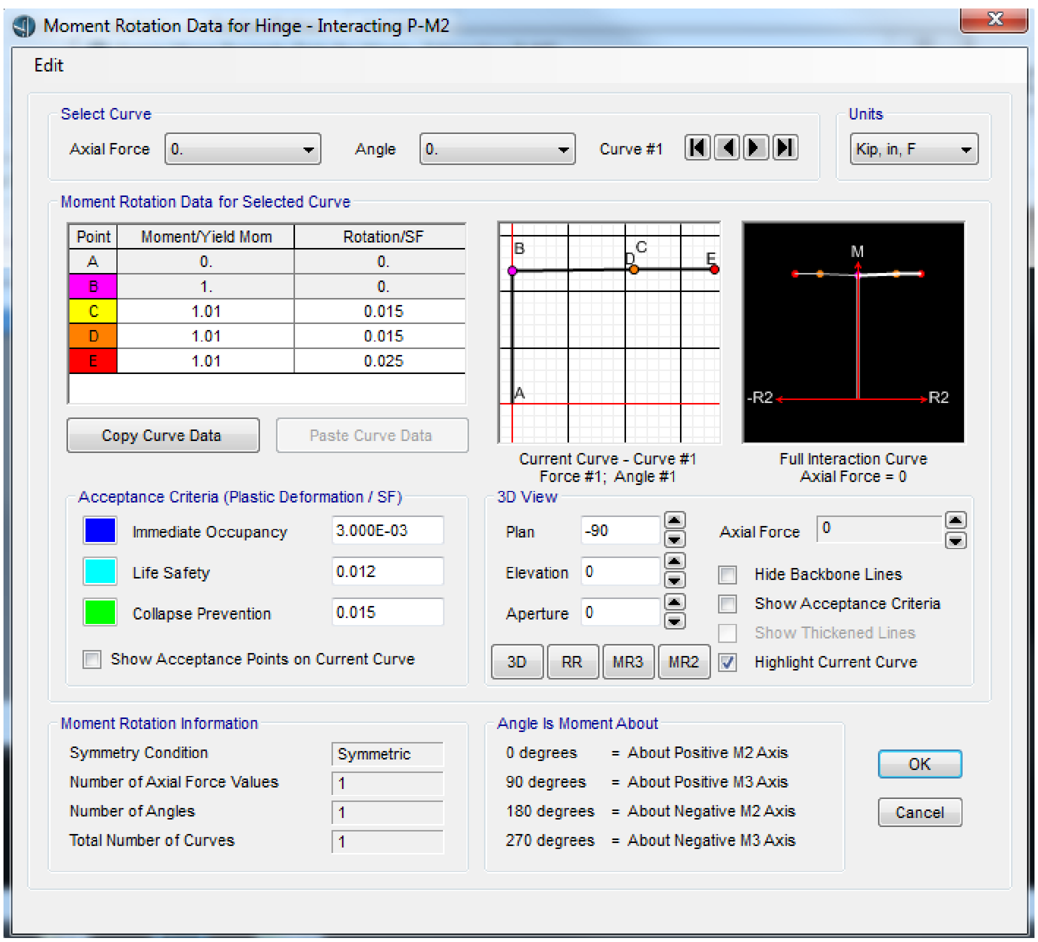


Figure B2 - 8: Moment-rotation curve in SAP2000

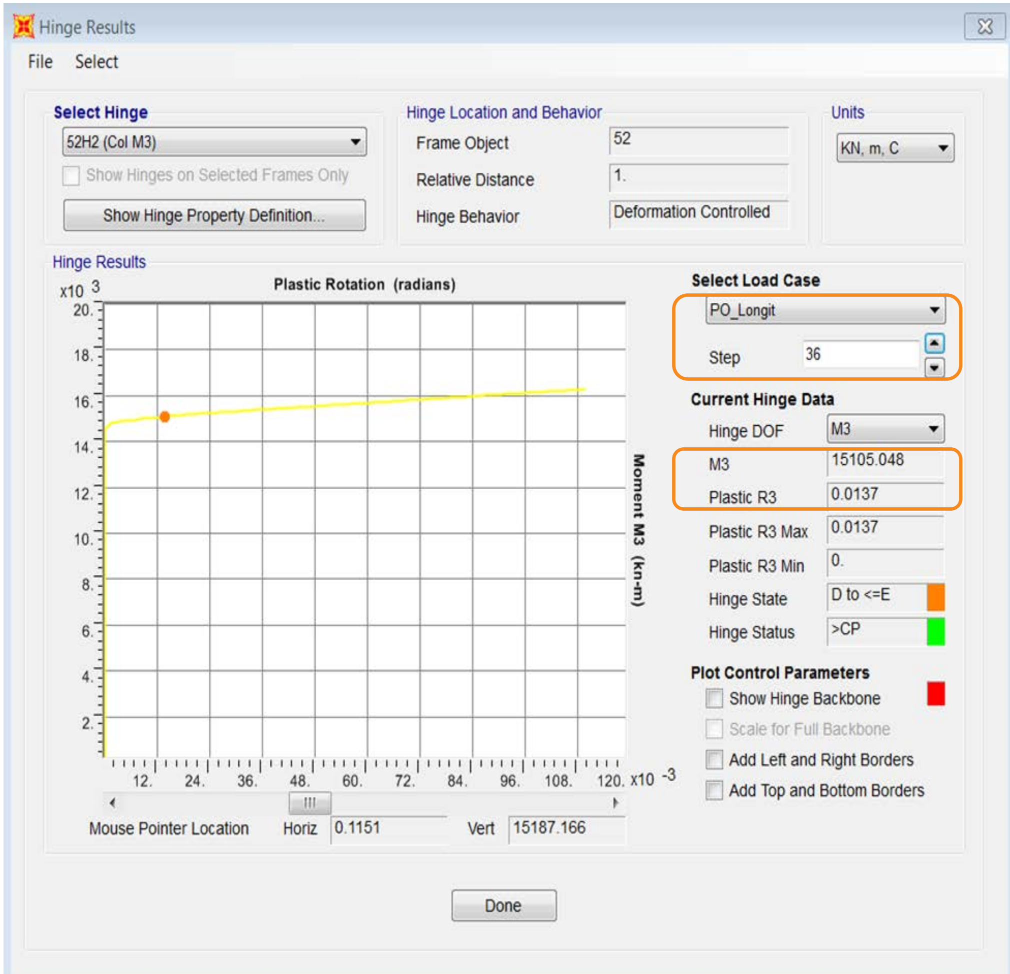


Figure B2 - 9: Hinge output in SAP2000

Equations proposed by Priestley et al (1996) can be used to calculate plastic hinge length (equation 2) for column on footings.

Plastic curvature can be calculated using equation 3 and equation 4. The equations are written as:

$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044\, f_{ye} d_{bl}$ (2)

$\Theta_p = (\varphi_u - \varphi_y)\, L_p = \varphi_p\, L_p$ (3)

$\Phi_p = \Theta_p / L_p$ (4)

where L is the distance from the critical section of the plastic hinge to the point of contraflexure and d_{bl} is the diameter of the longitudinal rebar. Θp is plastic rotation, φu is total curvature, φy is yielding curvature, and L_p is the plastic hinge length.

In this example, when using distributed plasticity model in SeismoStruct, the plastic hinge length does not need to be defined.

Case D1 was designed according to CAN/CSA-S6-06; its reinforcement ratio is 1.9%. The criteria from the Code were used to assess and demonstrate its seismic performance. Transverse pushover analysis was carried out for each bent incorporating non-linear p-y springs.

The plastic hinge sequence of bent 1 is shown in **Figure B2-10**. The pushover load direction was from left to right. The yielding sequence is marked in **Figure B2-10**.

Figures B2-11 to B2-16 show the pushover curves from SeismoStruct with displacement demands and strain limits. The displacement demands from different events are shown with dashed vertical lines. The displacement demands were calculated from spectral analysis. Strain criteria are marked on the curves. This is an important step for demonstrating performance compliance.

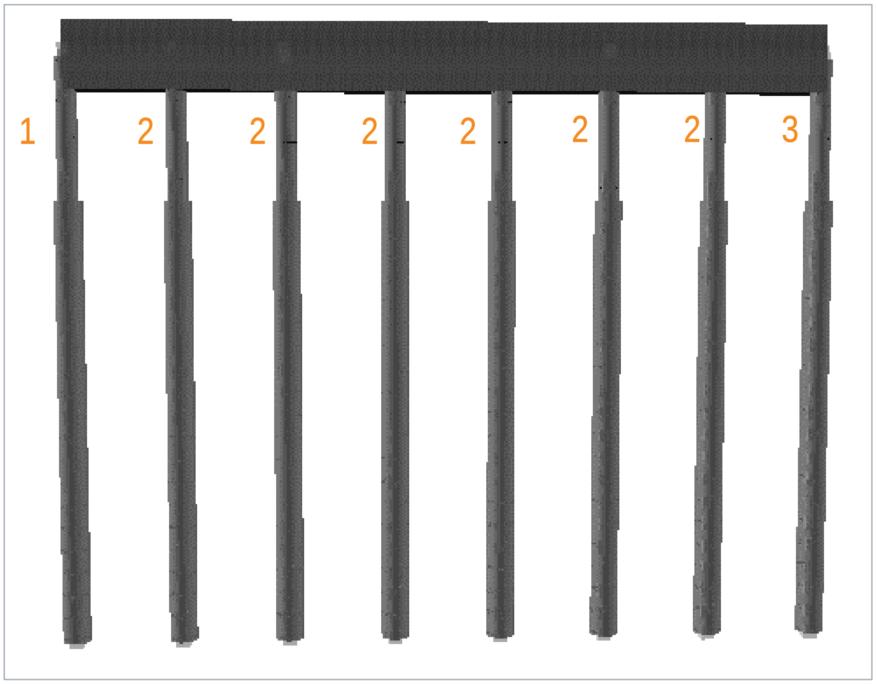


Figure B2 - 10: Plastic hinge sequence

As shown in **Figures B2-11 to B2-15**, all the bents reach yielding far before the 1 in 475-year return period event. Generally, the first yielding happens when bents reach half of the displacement demands for the 1 in 475-year return period event, which means that none of the bents meet the criteria from the Code for the 1 in 475-year return period event.

For the 1 in 975-year return period event, the Code requires that steel strains not exceed 0.015. Although not stipulated by the Code, the concrete strain of 0.006 was also checked as a criterion for repairable damage based on a project-specific criterion for this structure. It was observed that Abutment 4 barely meets this requirement, thus the bridge may reach extensive damage state for the 1 in 975-year return period event. Abutment 4 shows damage much earlier than the other bents. This is because the soil conditions of abutments and piers are different. Such differences in performance between different bridge supports are all but impossible to ascertain using FBD.

[continued]

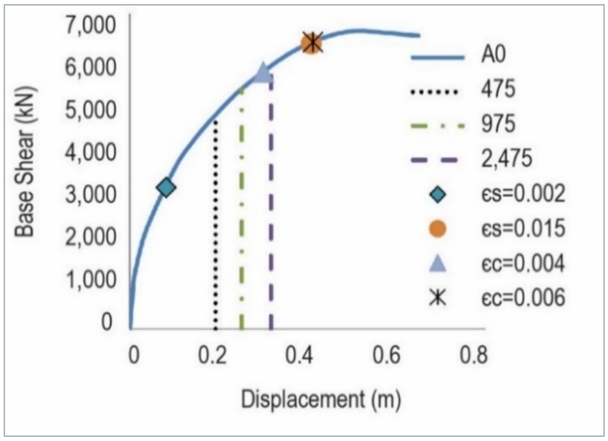


Figure B2 - 11: Abutment 0 pushover curve in case D1 (adapted from Zhang et al. 2016)

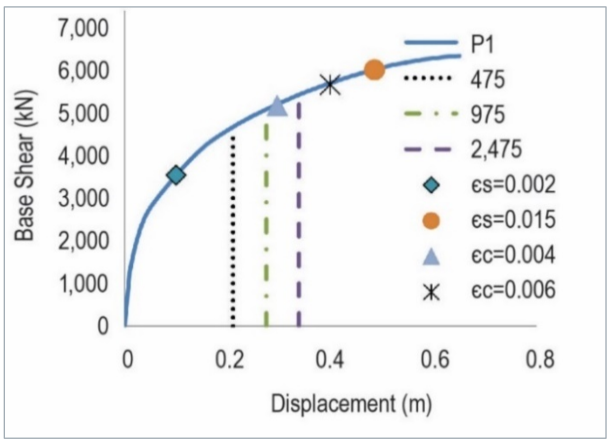


Figure B2 - 12: Bent 1 pushover curve in case D1 (adapted from Zhang et al. 2016)

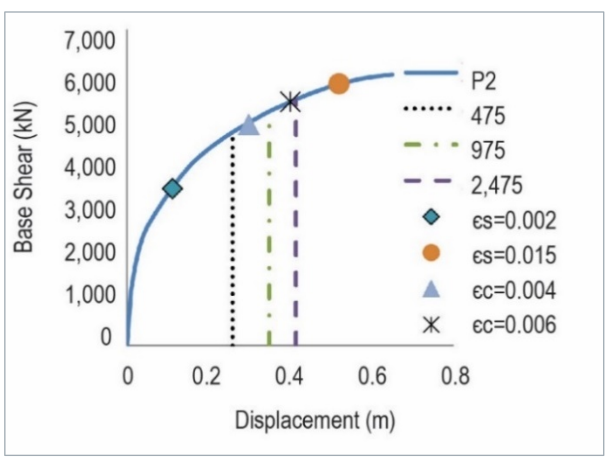


Figure B2 - 13: Bent 2 pushover curve in case D1 (adapted from Zhang et al. 2016)

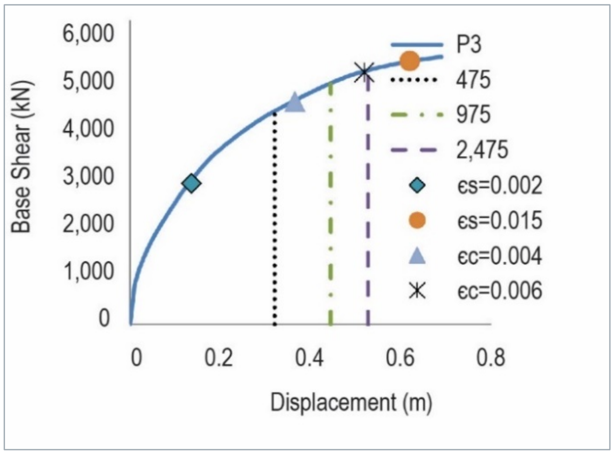


Figure B2 - 14: *Bent 3 pushover curve in case D1 (adapted from Zhang et al. 2016)*

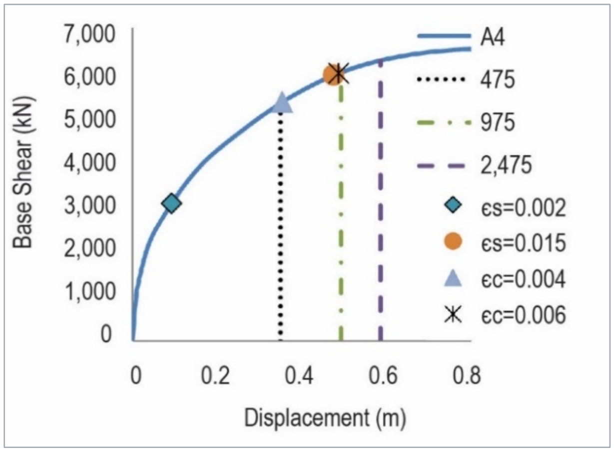


Figure B2 - 15: *Abutment 4 pushover curve in case D1 (adapted from Zhang et al. 2016)*

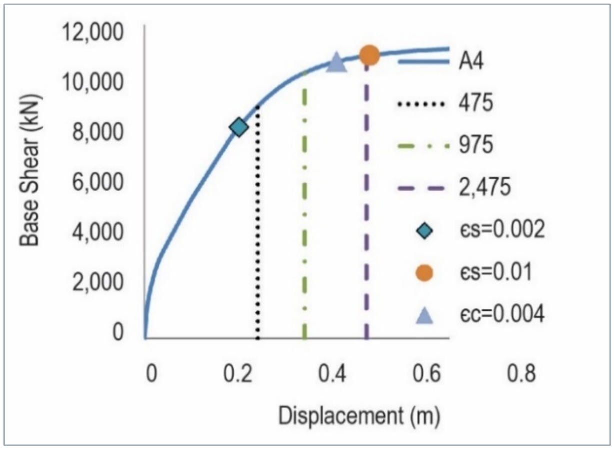


Figure B2 - 16: *Abutment 4 Pushover curve in case D3 (adapted from Zhang et al. 2016)*

Bent 3 and Abutment 4 are supported by the weakest soils among all the piers. The poor soil conditions at Abutment 4 lead to higher displacement demands and more damage. It was also found that all the bents can meet the criteria for 1 in 2,475-year return period event, since no significant strength degradation occurs and the steel stain of 0.05 was not reached.

To conduct PBD, non-linear pushover or time history analysis is required at the design phase. In the PBD of this case study, it was realized that Abutment 4 experiences the highest displacement demand and shows the most damages, so a pushover analysis was carried out only on Abutment 4 for case D3, which was the critical bent for the design. The pushover curve of Abutment 4 is shown in **Figure B2-16**.

B2.5.1 PERFORMANCE DISCUSSION

The Code requires that steel strains not exceed yield for the 1 in 475-year return period event. This requirement resulted in a very high longitudinal reinforcement ratio of 5.3% in piers. However, even with such a high reinforcement ratio, the first yielding of reinforcement still occurs slightly before the displacement demand. When the bent was pushed to the displacement demand, the maximum steel strain was 0.0024. Considering that displacement demands are calculated from effective soil stiffness, whereas non-linear analysis uses secant stiffness, the demands may be over-estimated. The strain of 0.0024 may be considered as meeting the requirement of the Code with acceptable tolerance. However, due to the high reinforcement ratio, the structure has a huge amount of capacity after the first yielding.

For the 1 in 975-year return period event, the concrete strain is even smaller than 0.004, which corresponds to the minimal damage level. The steel strain only increases to 0.01 for the 1 in 2,475-year return period event, while the concrete strain is smaller than 0.006. Based on the given criteria, it can be seen that once the requirements for the 1 in 475-year return period event are satisfied, the bridge does not even experience repairable damage corresponding to the 1 in 2,475-year return period event.

When comparing case D3 to case D1, case D3 exhibits much more conservative design but can be considered to be beyond the practical limits of constructability due to the high reinforcement ratio. Another challenge for such a design is the extremely high overstrength demands being generated through the plastic hinge/fuse elements. Capacity protection against such demands can be extremely challenging. The utility and of PBD primarily relying on displacements rather than forces and explicitly showing performance is clear from this exercise.

B2.6 PERFORMANCE ASSESSMENT BASED ON TIME HISTORY ANALYSIS

To conduct a rigorous assessment of the seismic performance of case D1, case D2, and case D3, time history analyses were carried out using SeismoStruct. In SeismoStruct, the non-linear hysteretic behaviour is included in the non-linear fibre models. Users do not have to define external damping; however, defining additional damping helps the analysis converge for inelastic dynamic analysis (SeismoSoft 2014). The performance criteria from the Code were used for the evaluation.

In the time history analysis, 7 earthquake records were selected from the Canadian Association for Earthquake Engineering (Naumoski et al. 1988) for demonstration purpose. A rigorous code-based design requires 11 time histories. Ground motions that represent the site and hazard will be determined.

Two sample original acceleration time histories are plotted in **Figure B2-17**.

The records were scaled based on site-specific response spectra. The scaled acceleration time histories are plotted in **Figure B2-18**. To better compare the scaled records with the original records, the original and matched response spectra are also plotted.

Figure B2-19 shows the unmatched accelerogram spectra with the target spectra. **Figure B2-20** shows the matched accelerogram spectra with the target spectra. It can be seen that the match spectra are scaled higher to the design level. Acceleration loads were applied in both horizontal directions.

Table B2-6 lists the records selected for time history analysis.

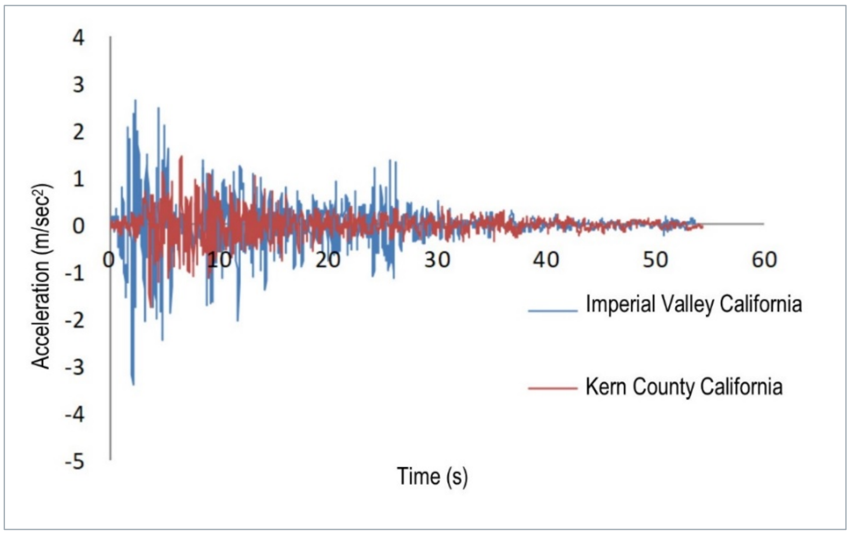


Figure B2 - 17: Original acceleration time histories (adapted from Zhang et al. 2016)

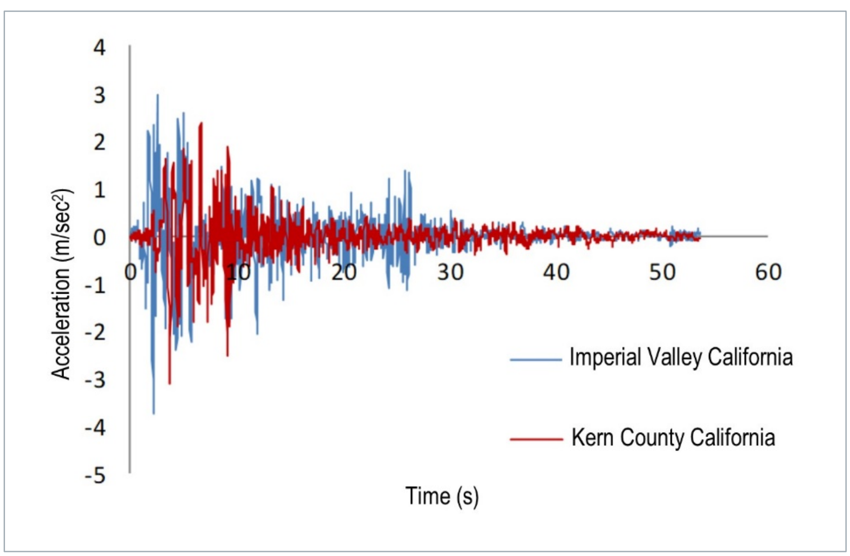


Figure B2 - 18: Scaled acceleration time histories (adapted from Zhang et al. 2016)

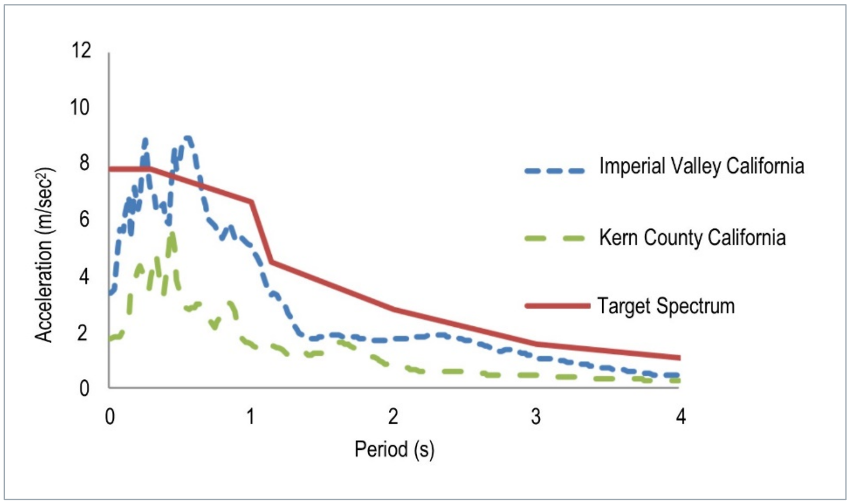


Figure B2 - 19: Target and original spectra (adapted from Zhang et al. 2016)

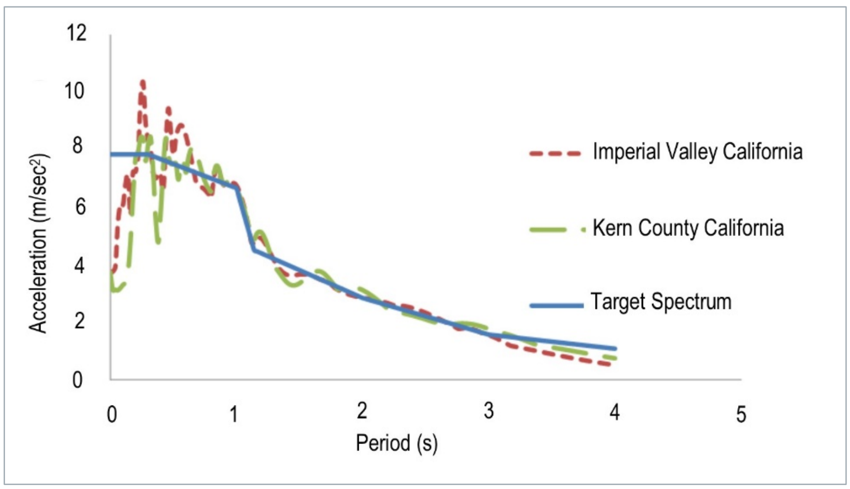


Figure B2 - 20: Target and matched spectra (adapted from Zhang et al. 2016)

Table B2 - 6: Earthquake Records (Naumoski et al. 1988)

RECORD NO.	EARTHQUAKE	DATE	MAGNITUDE	SITE	MAXIMUM ACCELERATION A (G)	MAXIMUM VELOCITY V (M/S)
1	Imperial Valley, California	1940 May 18	6.6	El Centro	0.348	0.334
2	Kern County, California	1952 Jul 21	7.6	Taft Lincoln School Tunnel	0.179	0.177
3	San Fernando, California	1971 Feb 9	6.4	Hollywood Storage P.E. Lot, Los Angeles	0.211	0.211
4	San Fernando, California	1971 Feb 9	6.4	Griffith Park Observatory, Los Angeles	0.18	0.205
5	San Fernando, California	1971 Feb 9	6.4	234 Figueroa St., Los Angeles	0.199	0.167
6	Near East Coast of Honshu, Japan	1971 Aug 2	7.0	Kushiro Central Wharf	0.078	0.068
7	Monte Negro, Yugoslavia	1979 Apr 15	7.0	Albatross Hotel, Ulcinj	0.171	0.194

Many useful structural responses can be generated by using time history analysis, such as displacement and strain. At the top of Bent 1, from time history analysis using Imperial Valley, California records, the maximum displacement demand was about 0.17 m, which is close to the displacement from RSA.

One example relation between strain and displacement is shown in **Figure B2-21**.

Displacement time history curves are shown in **Figures B2-22** and **B2-23** for Imperial Valley, California records and Kern County California records, respectively.

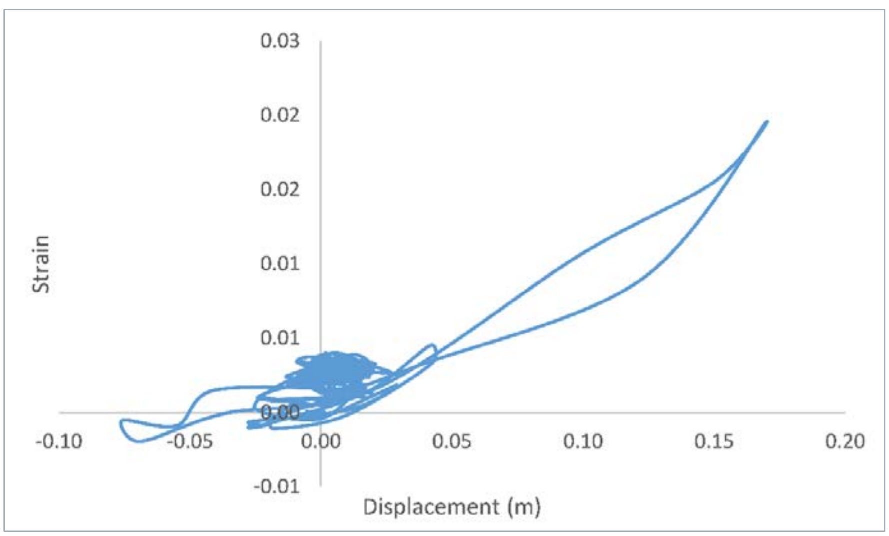


Figure B2 - 21: Steel reinforcement strain versus bent displacement (distributed plasticity model)

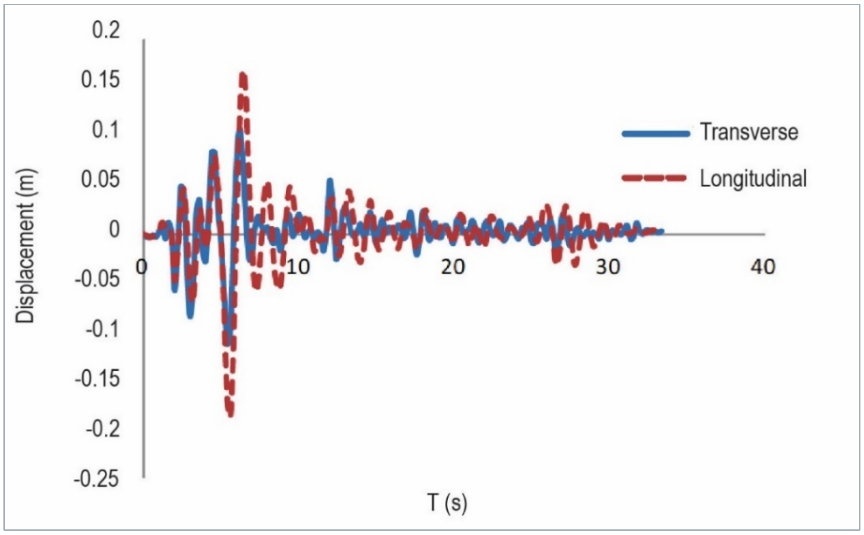


Figure B2 - 22: Bent displacement time history (Bent No.1, Imperial Valley, California records)

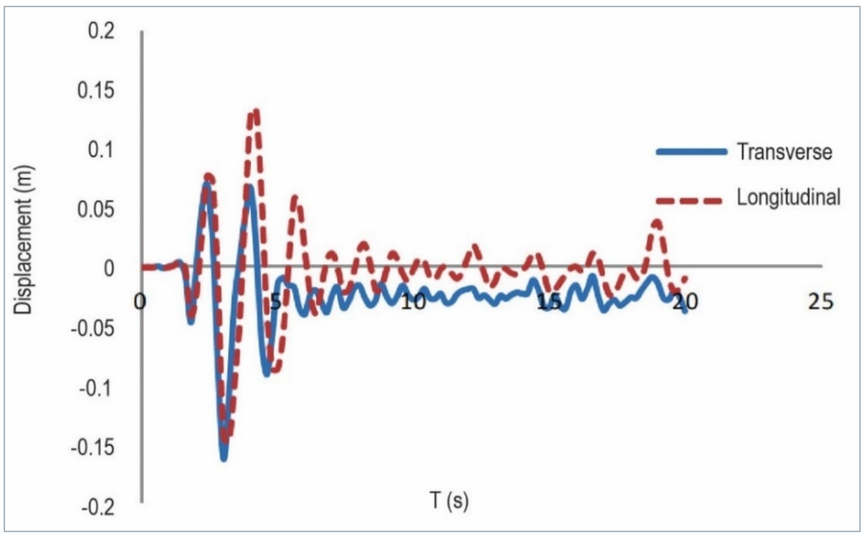


Figure B2 - 23: Bent displacement time history (Bent No.1, Kern County, California records)

Maximum strains from time history analyses are presented in Tables B2-7 to B2-9 for the three design cases (D1, D2, and D3). It should be noted that only the results from the first three records are shown because of limited space.

Table B2-10 shows the damage states of the three designs determined from average strains of time history analysis.

From the time history analysis, it was concluded that case D1 fails to meet the criteria for the 1 in 475-year return period event but meets the criteria for the 1 in 2,475-year return period event. This conclusion is the same with the findings from pushover analysis.

The steel strain reaches 0.002 before the 1 in 475-year return period event and remains smaller than 0.05 for the 1 in 2,475-year return period event. For the 1 in 975-year return period event, the maximum steel from pushover analysis is smaller than 0.015.

In the time history analysis, the steel strains are around 0.01. Case D2 also fails to meet the criteria for the 1 in 475-year return period event. D3 meets the criteria at all earthquake events and only reaches repairable damage states for the 1 in 2,475-year return period event.

It should be noted that although the maximum reinforcement ratio from the Code is 6%, which is higher than 5.3%, such a design would make concrete placement and proper vibration extremely difficult.

Based on the results below, it can be inferred that case D1 tends to induce a high degree of damage, although life safety is protected. This will result in a very high repair cost. Case D3 tends to be too conservative with a huge amount of residual capacity. Considering the reinforcement ratio, proper construction may be very difficult.

Table B2 - 7: Maximum Strains from Time History Analysis – Case D1

RETURN PERIOD (YEARS)	MATERIAL DAMAGE	EARTHQUAKE RECORD NUMBER		
		1	2	3
475	Concrete	0.003	0.003	0.003
	Steel	0.006	0.006	0.006
	Damage	Repairable	Repairable	Repairable
975	Concrete	0.004	0.005	0.006
	Steel	0.01	0.009	0.01
	Damage	Repairable	Repairable	Repairable
2,475	Concrete	0.015	0.006	0.015
	Steel	0.03	0.02	0.03
	Damage	Extensive	Extensive	Extensive

Note: $\varepsilon_y = 0.002$; $\varepsilon_{cu} = 0.019$

Table B2 - 8: Maximum Strains from Time History Analysis – Case D2

RETURN PERIOD (YEARS)	MATERIAL DAMAGE	EARTHQUAKE RECORD NUMBER		
		1	2	3
475	Concrete	0.003	0.003	0.003
	Steel	0.004	0.005	0.004
	Damage	Repairable	Repairable	Repairable
975	Concrete	0.004	0.004	0.005
	Steel	0.006	0.006	0.008
	Damage	Repairable	Repairable	Repairable
2,475	Concrete	0.007	0.006	0.007
	Steel	0.013	0.010	0.012
	Damage	Repairable	Repairable	Repairable

Note: ε_y = 0.002; ε_{cu} = 0.019

Table B2 - 9: Maximum Strains from Time History Analysis – Case D3

RETURN PERIOD (YEARS)	MATERIAL DAMAGE	EARTHQUAKE RECORD NUMBER		
		1	2	3
475	Concrete	0.001	0.001	0.001
	Steel	0.0015	0.002	0.0017
	Damage	Minimal	Minimal	Minimal
975	Concrete	0.001	0.001	0.002
	Steel	0.002	0.002	0.002
	Damage	Minimal	Minimal	Minimal
2,475	Concrete	0.003	0.001	0.002
	Steel	0.004	0.002	0.003
	Damage	Repairable	Minimal	Repairable

Note: ε_y = 0.002; ε_{cu} = 0.019

Table B2 - 10: Damage States – Cases D1, D2, and D3

CASE NUMBER	RETURN PERIOD (YEARS)		
	475	975	2,475
D1	Repairable	Repairable	Extensive
D2	Repairable	Repairable	Repairable
D3	Minimal	Minimal	Repairable

B2.7 SUMMARY AND CONCLUSIONS

Examples of typical highway bridge designs are presented in this case study. The bridge was designed using FBD according to CAN/CSA-S6-06 (denoted as case D1) and the Code (denoted as case D2), and was also designed with PBD according to the Code (denoted as case D3). Site-specific spectral accelerations and soil conditions were used in the design. The soil-structure interactions were considered by using a series of p-y curves.

Case D2 had a higher reinforcement ratio than case D1. This is reasonable because the Code is meant to improve structural safety. Case D3 had a much higher reinforcement ratio due to the strict requirements at 1 in 475-year return period event design. The 1 in 475-year return period event dominated the PBD.

After designing the bridge with three different approaches, pushover analysis and time history analysis were conducted to evaluate and explicitly demonstrate its seismic performance. The results from pushover and time history analyses were similar in terms of damage states. It was found that cases D1 and D2 fail to meet the criteria for the 1 in 475-year return period event. However, although cases D1 and D2 both met the criteria for the 1 in 975-year event and the 1 in 2,475-year return period event, case D2 showed much less damage than did case D1.

It should be noted that the treatment provided in this example is not exhaustive and may not satisfy all PBD criteria included in the Code. It only describes the relevant procedure for one set of criteria corresponding to plastic hinge material strains in columns. Other criteria such as bearing and joint damage, foundation performance, permanent offsets, and emergency vehicle access would also need to be satisfied and demonstrated by the designer.

B2.8 ACKNOWLEDGMENTS

The work presented here was carried out under the support from MMM Group Limited as part of the Natural Sciences and Engineering Research Council of Canada (NSERC) – Industrial Postgraduate Scholarships Program. The financial support provided by MMM Group Limited and NSERC is gratefully acknowledged.

B2.9 REFERENCES

Applied Technology Council (ATC). 1996. ATC-32 Improved Seismic Design Criteria for California Bridges: Provisional Recommendations. Redwood City, CA: ATC. 215 pp.

Canadian Standards Association (CSA). 2014. Canadian Highway Bridge Design Code. CSA Standard S6-14. (December 2014). Toronto, ON: Canadian Standards Association.

CSA. 2006. Canadian Highway Bridge Design Code. CSA Standard S6-06. Toronto, ON: Canadian Standards Association.

CSI. 2010. SAP2000, S.V. 8, 2002. Integrated Finite Element Analysis and Design of Structures Basic Analysis Reference Manual. Berkeley, CA: Computers and Structures, Inc. (CSI).

Naumoski N, Tso W, Heidebrecht A. 1988. A Selection of Representative Strong Motion Earthquake Records Having Different A/V Ratios. Earthquake Engineering Research Group (EERG) Report 88-01. Hamilton, ON: EERG, McMaster University.

SeismoSoft. 2010. A Computer Program (Seismostruct) for Static and Dynamic Nonlinear Analysis of Framed Structures.

Priestley MJN, Calvi GM, Kowalsky, M. 2007. Direct Displacement-Based Seismic Design of Structures. IUSS Press.

Priestley MJN, Seible F, Calvi GM. 1996. Seismic Design and Retrofit of Bridges. New York, NY: John Wiley and Sons.

Zhang Q, Alam MS, Khan S, Jiang J. 2016. Seismic Performance Comparison between Force-Based and Performance-Based Design as per Canadian Highway Bridge Design Code (CHBDC) 2014. Canadian Journal of Civil Engineering. 43(8): 741-48.

APPENDIX B3: TUBULAR ECCENTRICALLY BRACED FRAMES

AUTHOR: B. HAMERSLEY, P.ENG.

B3.1 BACKGROUND

This case study is based on a design for temporary works for the new San Francisco Oakland Bay Bridge (circa 2010). The design was developed to meet project-specific criteria.

The signature span for the new San Francisco Oakland Bay Bridge is the world's largest self-anchored suspension (SAS) bridge. During construction of the bridge, the deck had to be supported on temporary structures until the cable was installed and the deck weight transferred to the cable. The massive temporary works used to support and manoeuver the orthotropic box girder (OBG) deck segments included twin truss bridges supported on temporary steel towers.

The large mass of OBGs supported up to 55 m above the ocean gave rise to high seismic demands. To meet the stringent ductility requirements for the project, the six pairs of temporary towers supporting the truss were designed using tubular eccentrically braced frames (TEBFs) in the transverse direction to provide a ductile seismic load-resisting system. TEBFs are constructed from rectangular hollow sections that provide stability to the link so out-of-plane bracing is not required (as it is with I-sections). The TEBFs would allow the towers to undergo substantial deflections without compromising the lateral resistance.

TEBFs utilize yielding of short links in the bracing system, which act as ductile elements. Specific information such as limitations on the plastic rotations of the links, probable to nominal strength ratios, proportioning limits for the link elements, and stiffener requirements, are not covered by the codes but are instead found in the literature. Considerable research has been done on TEBFs at the Multi-disciplinary Center for Earthquake Engineering Research (MCEER) at the University at Buffalo, led by Professor Michel Bruneau.

The design example in this case study was developed here initially to meet the force-based design (FBD) requirements of the CAN/CSA-S6-06 Canadian Highway Bridge Design Code (CSA 2006), assuming the bridge is classified as an emergency route structure.

The design is then tested against the performance-based design (PBD) requirements of CAN/CSA-S6-14 Canadian Highway Bridge Design Code (the Code) (CSA 2014), assuming the bridge is classified as a major route structure (note the nomenclature change between codes), which has varying performance requirements for the 475-year, 975-year, and 2,475-year return period events.

It was found that although the design meets the performance requirements for the 975-year and 2,475-year events, it does not meet the performance requirements for the 475-year event. Yielding of the links is occurring, which is not allowed under the criteria for minimal damage. The brace sizes need to be increased to prevent yielding. A consequence of this design change is that strengthening the links to meet this requirement actually significantly reduces the ultimate displacement capacity of the structure, although it still exceeds the displacement demand for the 2,475-year event by a substantial amount.

B3.2 PROBLEM STATEMENT

The temporary Tower C supporting the east truss consists of approximately 36 m tall, 2 column steel bents founded on a dense, well-graded, sand and gravel fill. Tower C is fitted with TEBFs in the transverse direction and supports the weight of the east line truss and OBG. The tower columns and braces are made up of rectangular hollow structural sections (HSS). The Tower C geometry is provided in **Figure B3-1**.

For the purpose of this design example the bridge is irregular and is classified as a major route (emergency route) bridge.

B3.3 ASSUMPTIONS

Material properties for the steel are as follows:

- Elastic modulus, E_{steel} = 200,000,000 kN/m^2
- Shear modulus, G_{steel} = 77,000,000 kN/m^2
- Yield strength, $F_{y\ steel}$ = 345 MPa

Additional assumptions include the following:

- The weight supported by the tower is 22,000 kN acting 11 m above the top chord of the tower
- The tower and truss members have zero mass
- The design tower members for seismic loads are in the transverse direction only
- Consider only one load combination: 1.2DL + 1.0EQ
- Peak horizontal ground accelerations are for Vancouver, BC, Canada

B3.4 PART 1 – DESIGN ACCORDING TO CAN/CSA-S6-06

Design the tower links, columns, and brace members in accordance with CAN/CSA-S6-06 (CSA 2006). The shear links should be designed to yield in shear before flexure.

B3.4.1 STEP 1 – DEVELOP DESIGN RESPONSE SPECTRUM

Obtain the uniform hazard spectrum (UHS) values using the 2010 National Building Code of Canada seismic hazard calculator, available online at the Natural Resources Canada (NRC) website (NRC 2010) (**Figure B3-2**).

Design Data:

- Importance factor: I = 1.5 (emergency route bridge)
- Design earthquake: 10% probability of exceedance in 50 years, equivalent to an earthquake with return period of 475 years
- Peak ground acceleration: 0.23 g
- Zonal acceleration ratio: A = 0.3
- Seismic performance zone: 4
- Site coefficient: S = 1.0

Table B3 - 1: Spectral Ordinates

PERIOD (SECONDS)	C_{sm} (I = 1.0)	C_{sm} (I = 1.5)
0.01	0.75	1.13
0.25	0.75	1.13
1.00	0.36	0.54
2.00	0.23	0.34
3.00	0.17	0.26
4.00	0.14	0.21
5.00	0.14	0.21
6.00	0.14	0.21

Notes: I = importance factor

Panel width, W4 = 10.0m

Column length, L4 = =8.78m

Brace Angle, ϴ4 = 61.0°

Panel width, W3 = 11.2m

Column length, L3 = 8.78m

Brace Angle, ϴ3 = 58.4°

Panel width, W2 = 12.3m

Column length, L2 = 8.78m

Brace Angle, ϴ2 = 55.9°

Panel width, W1 = 13.5m

Column length, L1 = 8.78m

Brace Angle, ϴ1 = 53.5°

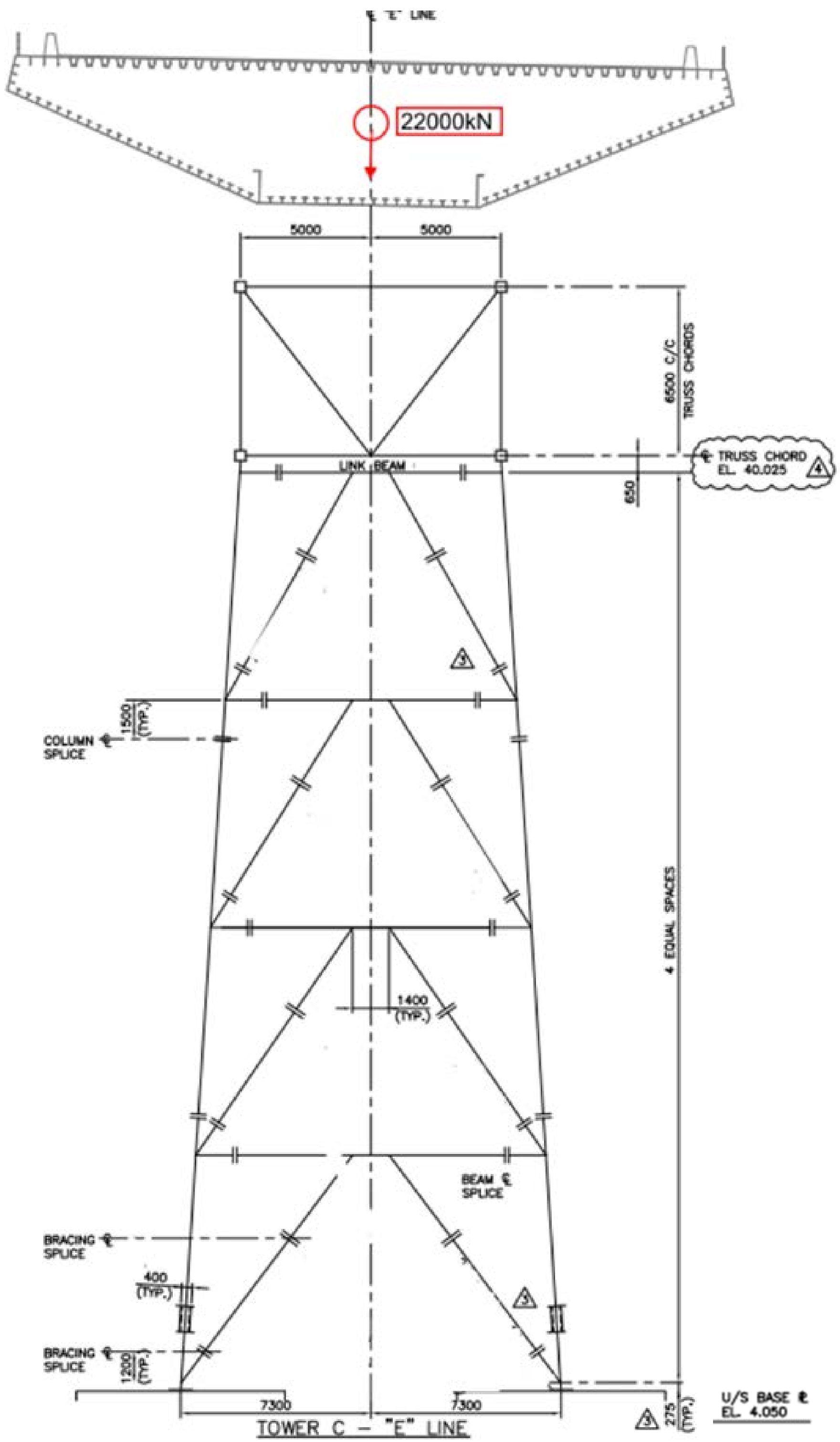
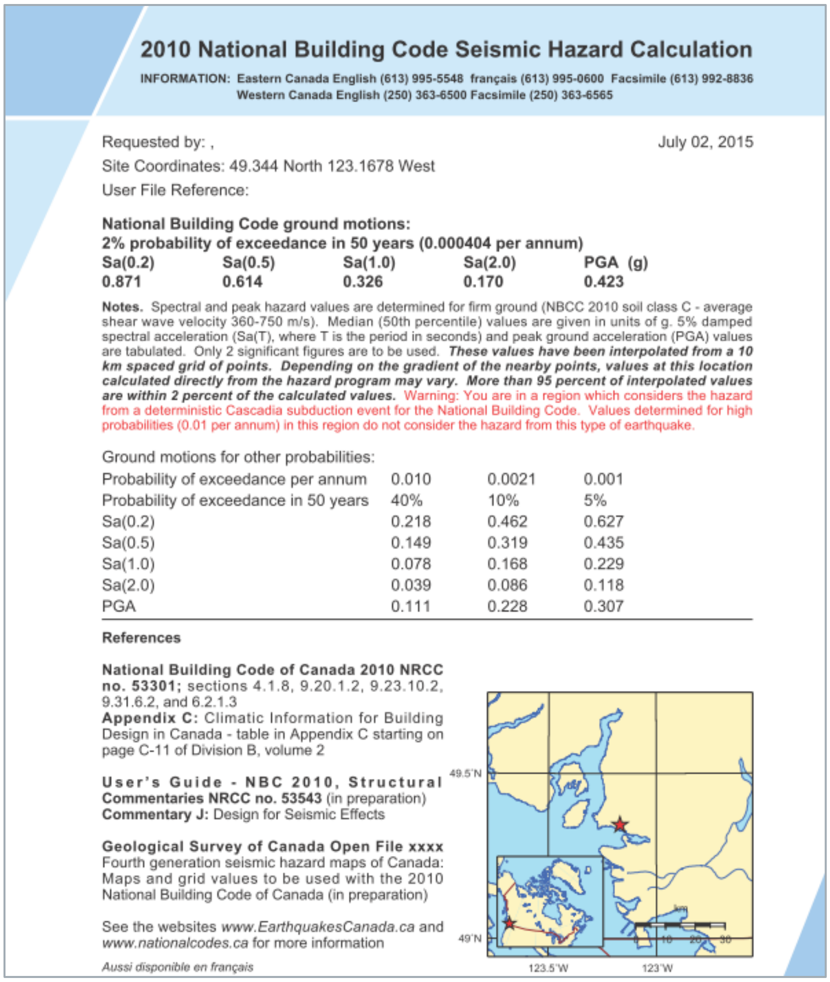


Figure B3 - 1: Tower C geometry



2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , July 02, 2015
Site Coordinates: 49.344 North 123.1678 West
User File Reference:

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.871	**0.614**	**0.326**	**0.170**	**0.423**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*** Warning: You are in a region which considers the hazard from a deterministic Cascadia subduction event for the National Building Code. Values determined for high probabilities (0.01 per annum) in this region do not consider the hazard from this type of earthquake.

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.218	0.462	0.627
Sa(0.5)	0.149	0.319	0.435
Sa(1.0)	0.078	0.168	0.229
Sa(2.0)	0.039	0.086	0.118
PGA	0.111	0.228	0.307

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français

Figure B3 - 2: 2010 National Building Code seismic hazard calculation

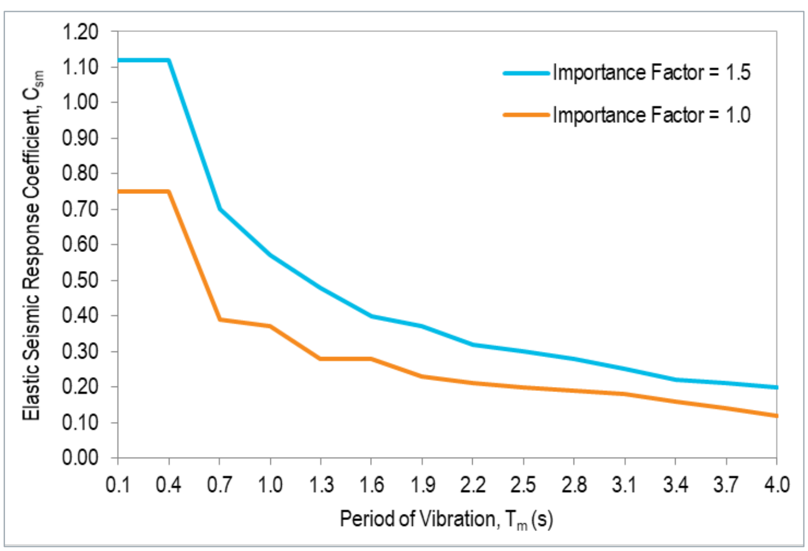


Figure B3 - 3: Design response spectrum – 1/475 year EQ

B3.4.2 STEP 2 – ESTIMATE COLUMN AND BRACE MEMBER SIZES

Select preliminary column and brace member sizes. Members must have sufficient capacity to support factored dead load demands, plus reserve for seismic demands.

Select 1150 x 400 x 32 column members:

- N_{rT} = 20,100 kN
- M_{rT} = 4,700 kN
- V_{rT} = 4,000 kN

Select 400 x 400 x 12 brace members:

- N_{rT} = 4,500
- M_{rT} = 840 kN
- V_{rT} = 1,700 kN

B3.4.3 STEP 3 – ESTIMATE LINK MEMBER SIZES

Select preliminary link member sizes. Shear links must be proportioned to yield in shear prior to flexural hinging at the link ends. Ductile EBFs are proportioned in accordance with CAN/CSA-S6-06, Clause 27.7 using R=5.

Select 100 x 400 x 25 x 10 link members.

[See Shear Link Properties below.]

Link Beam Capacities

Table B3 - 2: Link Beam Capacities

LEVEL	COMPRESSION (KN)	MOMENT (KNM)	SHEAR (KN)
4	930	240	190
3	760	240	190
2	640	240	190
1	540	240	190

Shear Link Properties

(Note: Red highlighting indicates Mathcad inputs.)

Input:

- section properties of a boxed section

width $b := 400mm$

height $h := 100mm$

flange thickness $t_f := 25mm$

web thickness $t_w := 10mm$

link length $e_o := 1400mm$

- material properties:

yield strength $F_y := 345MPa$

Elastic Modulus $E_s := 200000MPa$

shear yield strength $F_{yv} := \frac{1}{\sqrt{3}} \cdot F_y$ $F_{yv} = 199\ MPa$

Calculated section properties:

Gross Area	$A_g := b \cdot h - (b - 2t_w)\cdot(h - 2t_f)$	$A_g = 21000\,mm^2$ $\quad A_g = 0.021\,m^2$
Web Area	$A_w := 2\cdot(h - 2\cdot t_f)\cdot t_w$	$A_w = 1000\,mm^2$
yield shear	$V_p := F_{yv} \cdot A_w$	$\boxed{V_p = 199\,kN}$
Plastic Section Modulus	$Z := t_f\cdot(b - 2\cdot t_w)\cdot(h - t_f) + \frac{t_w \cdot h^2}{2}$	$Z = 7.63 \times 10^5\,mm^3$
plastic moment	$M_p := F_y \cdot Z$	$\boxed{M_p = 263\,kN \cdot m}$

$\frac{2\cdot M_p}{e_o} = 376\,kN \qquad \phi_v := 0.9$

$M_u := \phi_v \cdot M_p \qquad \boxed{M_u = 237\,kN \cdot m}$

nominal shear	$V_n := \mathrm{if}\left(V_p \le \frac{2\cdot M_p}{e_o}, V_p, \frac{2\cdot M_p}{e_o}\right)$	$\boxed{V_n = 199\,kN}$
	$V_u := \phi_v \cdot V_n$	$\boxed{V_u = 179\,kN}$
for short (shear) links	$e_v := 1.6 \cdot \frac{M_p}{V_p}$	$e_v = 2.113\,m$
for long (flexural) links	$e_f := 2.6 \cdot \frac{M_p}{V_p}$	$e_f = 3.43\,m$

$\mathrm{check} := \mathrm{if}\left(e_o \le e_v, \text{"shear link"}, \mathrm{if}\left(e_o \ge e_f, \text{"flexural link"}, \text{"intermidiate link"}\right)\right)$

$\boxed{\mathrm{check} = \text{"shear link"}}$

link rotation

$\gamma := \mathrm{if}\left[e_o \le e_v, 0.08, \mathrm{if}\left[e_o \ge e_f, 0.02, 0.02 + (e_f - e_o)\cdot\frac{0.08 - 0.02}{e_f - e_v}\right]\right]$

$\boxed{\gamma = 0.08}$

Compactness limits of flange

$\lambda_1 := \dfrac{b - 2\cdot t_w}{t_f}$ $\lambda_1 = 15.2$

$\lambda_f := 0.64\cdot\sqrt{\dfrac{E_s}{F_y}}$ $\lambda_f = 15.41$

$\text{check\_f} := \text{if}\left(\lambda_1 \le \lambda_f, \text{"OK"}, \text{"NG"}\right)$ $\boxed{\text{check\_f} = \text{"OK"}}$

Compactness limits of web

$\lambda_2 := \dfrac{h - 2\cdot t_f}{t_w}$ $\lambda_2 = 5$

$\lambda_{w1} := 1.67\cdot\sqrt{\dfrac{E_s}{F_y}}$ $\lambda_{w1} = 40.21$

$\lambda_{w2} := 0.64\cdot\sqrt{\dfrac{E_s}{F_y}}$ $\lambda_{w2} = 15.41$

$\text{check\_w} := \text{if}\left(e_o \le e_v, \text{if}\left(\lambda_2 \le \lambda_{w2}, \text{"no stiff."}, \text{if}\left(\lambda_2 \le \lambda_{w1}, \text{"stiff. req."}, \text{"NG"}\right)\right), \text{if}\left(\lambda_2 \le \lambda_{w2}, \text{"OK"}, \text{"NG"}\right.$

$\boxed{\text{check\_w} = \text{"no stiff."}}$

Stiffener spacing (if required)

for max. link rotation of 0.08 rad $C_{B1} := 20$ $a_1 := \left(C_{B1} - \dfrac{h}{8\cdot t_w}\right)\cdot t_w$ $a_1 = 0.19\,m$

for max. link rotation of 0.02 rad $C_{B2} := 37$ $a_2 := \left(C_{B2} - \dfrac{h}{8\cdot t_w}\right)\cdot t_w$ $a_2 = 0.36\,m$

stiffener spacing $a := a_1 + (0.08 - \gamma)\cdot\dfrac{a_2 - a_1}{0.08 - 0.02}$ $\boxed{a = 0.19\,m}$

Link Rotation

maximum shear link rotation $\gamma_{p\_max} := 0.08\text{rad}$ $\gamma_{p\_max} = 4.58\,\text{deg}$

frame length, height and number of stories with EBF $L := 14.6m$ $H := 9m$ $n := 4$

maximum frame rotation $\theta_{p\_max} := \gamma_{p\_max}\cdot\dfrac{e_o}{L}$ $\theta_{p\_max} = 0.44\,\text{deg}$

maximum story drift $\Delta_p := H\cdot\tan\left(\theta_{p\_max}\right)$ $\Delta_p = 69\,mm$

maximum frame drift $\Delta_{p\_f} := n\cdot\Delta_p$ $\boxed{\Delta_{p\_f} = 276\,mm}$

Note: This is a conservative assumption as it does not include elastic rotation.

B3.4.4 STEP 4 – PERFORM RESPONSE SPECTRUM ANALYSIS

Perform response spectrum analysis (RSA) using SAP2000 to determine member loads and tower deflections.

Response Spectrum Results

Using I=1.5, produces the following results.

Shear Links

- V_{DL} = 0kN
- V_{EQ} = 920kN

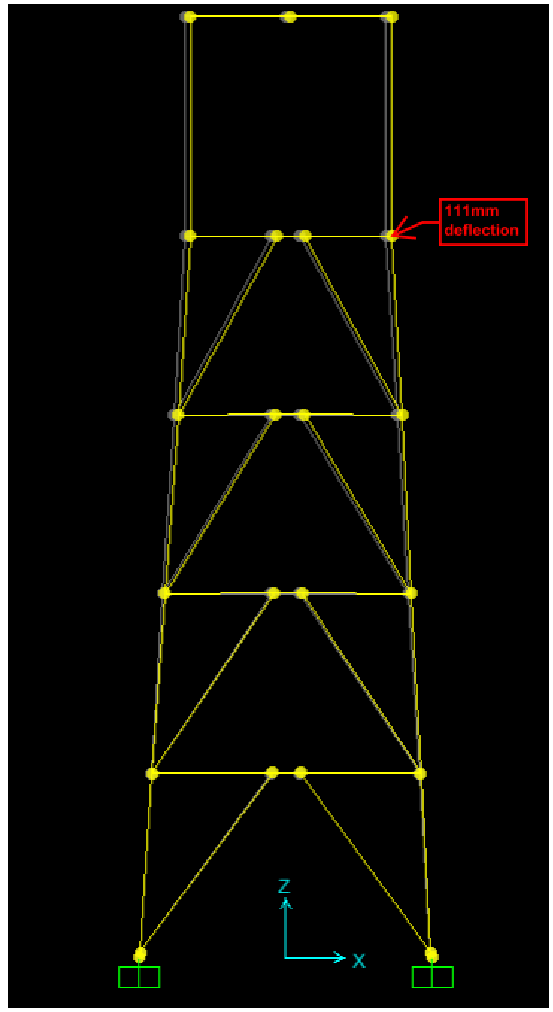


Figure B3 - 4: SAP2000 model

Columns

Table B3 - 3: Response Spectrum Analysis Results – Columns

LEVEL	DEAD LOADS			RESPONSE SPECTRUM RESULTS (Δ_{SD})		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	11,018	33	2	1,845	165	17
3	11,018	37	5	2,376	414	29
2	11,019	6	1	3,296	465	18
1	11,020	6	1	4,184	1,941	153

Braces

Table B3 - 4: Response Spectrum Analysis Results – Braces

LEVEL	DEAD LOADS			RESPONSE SPECTRUM RESULTS (Δ_{SD})		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	4	7	1	1,125	569	83
3	3	5	1	1,061	528	75
2	2	4	1	926	453	64
1	2	4	0	568	276	42

Link Beams

Table B3 - 5: Response Spectrum Analysis Results – Link Beams

LEVEL	DEAD LOADS			RESPONSE SPECTRUM RESULTS (Δ_{SD})		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	160	9	4	605	79	26
3	160	7	3	609	66	20
2	160	6	2	564	53	14
1	160	5	2	368	25	5

B3.4.5 STEP 5 – CHECK MEMBER CAPACITIES

Shear Links

- $1.2V_{DL} + 1.0V_{EQ}$ = 920kN
- V_{EQ} / V_U = 920kN / 179kN = 5.14

From CAN/CSA-S6-06, Clause 27.7:

- Probable resistance of shear links, V_P = 1.44 x Ry x V_n = 1.44 x 1.1 x 199kN = 316kN
- Column overstrength factor, R = V_P / V_{EQ} = 316kN / 920kN = 0.34

Columns

Response Spectrum Results

Table B3 - 6: Column Demand/Capacity Ratios

LEVEL	1.2DL + 1.0 EQ (V_U / V_{EQ})			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	13,581	72	6	0.66	0.02	0.0	D/C<1, OK
3	13,684	125	12	0.66	0.03	0.0	D/C<1, OK
2	13,864	98	5	0.67	0.02	0.0	D/C<1, OK
1	14,038	385	31	0.68	0.08	0.01	D/C<1, OK

Capacity-Protected Results

Design column members as capacity-protected elements with factored resistances equal to or greater than the maximum force effect that can be developed by the shear links attaining their probable resistance.

Note that the columns see a seismic load in addition to the shear link demands, due to the eccentricity of the mass above the tower, which is approximately 11 m above the top chord of the tower. This load is applied to the columns by taking the overstrength shear demand at the base, and reapplying it at the OBG level to produce a force couple into the columns from the eccentric load.

Following are the horizontal force calculations for tower level 4.

- Horizontal force from column dead load = N_{DL} x (7.3m-5m)/35.1m
- Horizontal force from column dead load = 11,000kN x (2.3m)/35.1m
- Horizontal force from column dead load = 721kN
- Horizontal force from brace seismic force = V_p / tan(Θ_4)
- Horizontal force from brace seismic force = 316kN / tan(61.0°)
- Horizontal force from brace seismic force = 176kN

The horizontal force from the column seismic force is an iterative calculation, assuming a seismic column axial load value and then confirming that it is correct.

- Horizontal force from column seismic force = seismic column F_{axial} x sin(tan(7.3m-5m)/35.1m)
- Horizontal force from column seismic force = 770kN x sin(tan(2.3m)/35.1m)
- Horizontal force from column seismic force = 50kN

Table B3 - 7: Columns

LEVEL	HORIZONTAL FORCE (KN)			
	FROM COLUMN DEAD LOAD FORCE N_{DL} x 2.3m/35.1m	FROM BRACE SEISMIC FORCE $V_p / \tan(\theta_N)$	FROM COLUMN SEISMIC FORCE N_{EQ} x 2.3m/35.1m	TOTAL BASE SHEAR (1.2DL + EQ)
4	721	176	50	–
3	721	196	71	–
2	721	217	92	–
1	722	238	113	–
West base shear	722	238	113	1,216
East base shear	-722	238	113	-515
Total base shear	–	–	–	Σ = 701

- Force couple = total base shear x (11m / 10m)
- Force couple = 701kN x (11m / 10m)
- Force couple = 770kN
- Link beam shear = V_p x (4-N)
- Link beam shear = V_p x (4-4)
- Link beam shear = 0kN (at top level)

Table B3 - 8: Column Load Combinations

LEVEL	LOAD CASE TRANSVERSE DIRECTION ($3\Delta_{SD}$) AXIAL FORCE (kN)			LOAD COMBINATION (1.2DL + 1.0EQ) TRANSVERSE DIRECTION ($3\Delta_{SD}$)		
	FORCE COUPLE V_{base} x 11m/10m	LINK BEAM SHEAR	TOTAL	AXIAL FORCE (kN) 1.2DL + EQ	MOMENT (kNm) M_{EQ} x R	SHEAR (kN) V_{EQ} x R
4	770	0	770	13,992	96	8
3	770	316	1,086	14,307	187	16
2	770	631	1,401	14,624	167	7
1	770	947	1,717	14,941	674	54
Base	770	1,263	2,033	15,257	–	–

Table B3 - 9: Column Demand/Capacity Ratios

LEVEL	D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	0.69	0.02	0.0	D/C<1, OK
3	0.70	0.04	0.0	D/C<1, OK
2	0.72	0.04	0.0	D/C<1, OK
1	0.73	0.14	0.01	D/C<1, OK

The 1,150 x 400 x 32 member size selected for the columns is therefore sufficient to carry the design loads.

Braces

Design brace members as capacity-protected elements with factored resistances equal to or greater than the maximum force effect that can be developed by the shear links attaining their probable resistance.

Table B3 - 10: Braces

LEVEL	AXIAL (kN)		MOMENT (kNm)		SHEAR (kN)	
	DL	EQ $V_p / \sin(\Theta_N)$	DL	EQ Δ_{SD} x R	DL	EQ Δ_{SD} x R
4	4	361	7	193	1	28
3	3	371	5	180	1	26
2	2	381	4	154	1	22
1	2	393	4	94	0	14

Table B3 - 11: Brace Demand/Capacity Ratios

LEVEL	1.2DL + 1.0EQ			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	366	204	30	0.08	0.24	0.03	D/C<1, OK
3	374	187	27	0.08	0.22	0.03	D/C<1, OK
2	384	160	23	0.09	0.19	0.03	D/C<1, OK
1	395	100	14	0.09	0.12	0.02	D/C<1, OK

The 400 x 400 x 12 member size selected for the braces is therefore sufficient to carry the design loads.

Link Beams

Design link beam members as capacity-protected elements with factored resistances equal to or greater than the maximum force effect that can be developed by the shear links attaining their probable resistance.

Table B3 - 12: Link Beams

LEVEL	AXIAL (kN)		MOMENT (kNm)		SHEAR (kN)	
	DL	EQ Δ_{SD} x R	DL	EQ Δ_{SD} x R	DL	EQ Δ_{SD} x R
4	160	206	9	27	4	9
3	160	207	7	22	3	7
2	160	192	6	18	2	5
1	160	125	5	9	2	2

Table B3 - 13: Link Beams Demand/Capacity Ratios

LEVEL	1.2DL + 1.0EQ			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	400	27	9	0.53	0.24	0.03	D/C<1, OK
3	401	23	7	0.52	0.22	0.03	D/C<1, OK
2	386	18	5	0.61	0.19	0.03	D/C<1, OK
1	318	9	2	0.59	0.12	0.02	D/C<1, OK

The 100 x 400 x 25 x 10 member size selected for the link beams is therefore sufficient to carry the design loads.

B3.4.6 STEP 6 – PERFORM NONLINEAR PUSHOVER ANALYSIS

Perform a non-linear pushover analysis using SAP2000 to verify the results. Shear link elements are modelled as non-linear link elements with the following properties.

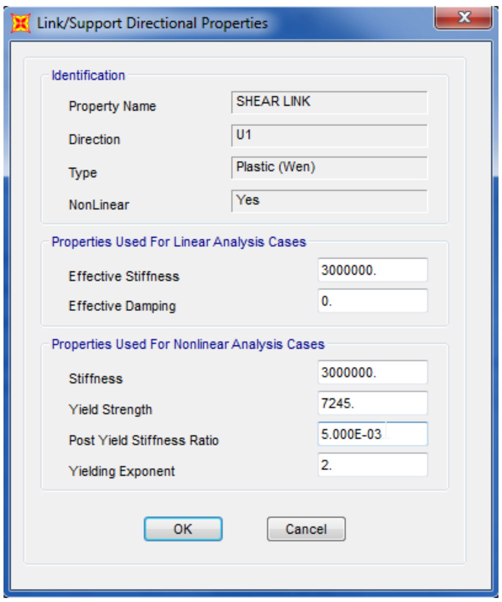


Figure B3 - 5: SAP2000 shear link properties

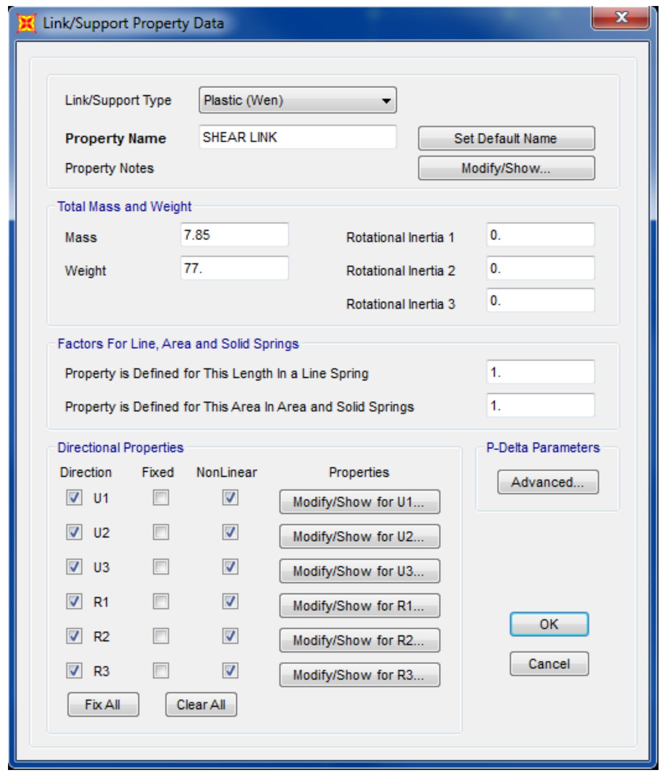


Figure B3 - 6: SAP2000 shear link properties

(Note: Yellow highlighting indicates Mathcad inputs.)

Input:

- section properties of boxed section $b := 400mm$ $t_f := 25mm$

$h := 100mm$ $t_w := 10mm$

$L := 1400mm$

shear deformation location $L_v := \frac{L}{2}$ $L_v = 0.7\,m$

- material properties: $F_y := 345MPa$ $F_u := 1.2 \cdot F_y$ $F_u = 414 \cdot MPa$

$F_{yv} := 0.577 \cdot F_y$ $F_{yv} = 199 \cdot MPa$

$E_s := 200000MPa$ $G_s := 77000MPa$

Calkulated section properties: $A_s := b \cdot h - (b - 2t_w) \cdot (h - 2t_f)$ $A_s = 21000 \cdot mm^2$

$$I_3 := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t_w) \cdot (h - 2 \cdot t_f)^3}{12} \qquad I_3 = 2.938 \times 10^7 \cdot mm^4$$

$$I_2 := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t_f) \cdot (b - 2 \cdot t_w)^3}{12} \qquad I_2 = 3.047 \times 10^8 \cdot mm^4$$

Nonlinear section properties (Wen)

Direction U1 - axial

- effective stiffness $K_{u1} := \frac{E_s \cdot A_s}{L}$ $\boxed{K_{u1} = 3 \times 10^6 \cdot \frac{kN}{m}}$

- yield strength $P_y := F_y \cdot A_s$ $\boxed{P_y = 7.245 \times 10^3 \cdot kN}$

- post yielding stiffness ratio $PYSR := 5\%$

- yielding exponent $EX_y := 2$

Direction U2 - shear

- moment of inertia $I := I_3$ $I = 2.938 \times 10^7 \cdot mm^4$

- effective stiffness $K_{u2} := \frac{12 \cdot E_s \cdot I}{L^3}$ $\boxed{K_{u2} = 2.569 \times 10^4 \cdot \frac{kN}{m}}$

- yield strength $A_w := 2 \cdot (h - 2 \cdot t_f) \cdot t_w$ $A_w = 1000 \cdot mm^2$

$V_y := F_{yv} \cdot A_w$ $\boxed{V_y = 199 \cdot kN}$

- post yielding stiffness ratio $PYSR := 0.5\%$

- yielding exponent $EX_w := 2$

Direction U3 - shear

- moment of inertia $I := I_2$ $I = 3.047 \times 10^{8} \cdot mm^{4}$

- effective stiffness $K_{u3} := \frac{12 \cdot E_s \cdot I}{L^3}$ $\boxed{K_{u3} = 2.665 \times 10^{5} \cdot \frac{kN}{m}}$

- yield strength $A_w := 2 \cdot (b - 2 \cdot t_w) \cdot t_f$ $A_w = 19000 \cdot mm^{2}$

$V_y := F_{yv} \cdot A_w$ $\boxed{V_y = 3782 \cdot kN}$

- post yielding stiffness ratio $PYSR := 0.5\%$

- yielding exponent $EX := 2$

Direction R1 - torsion

$A_m := (b - t_w) \cdot (h - t_f)$ $A_m = 2.925 \times 10^{4} \cdot mm^{2}$

$$J_s := \frac{4 \cdot A_m^{\ 2}}{\frac{2 \cdot (b - t_w)}{t_f} + \frac{2 \cdot (h - t_f)}{t_w}} \qquad J_s = 7.407 \times 10^{7} \cdot mm^{4}$$

- effective stiffness $K_{R1} := \frac{G_s \cdot J_s}{L}$ $\boxed{K_{R1} = 4.074 \times 10^{3} \cdot kN \cdot m}$

- yield strength $M_{Ty} := F_y \cdot (h - t_f) \cdot t_w \cdot (b - t_w) + F_y \cdot (b - t_w) \cdot t_f \cdot (h - t_f)$

$\boxed{M_{Ty} = 353.194 \cdot kN \cdot m}$

- post yielding stiffness ratio $PYSR := 0.5\%$

- yielding exponent $EX := 2$

Direction R2 - bending about local axis 2

- moment of inertia $I := I_2$ $I = 3.047 \times 10^{8} \cdot mm^{4}$

- effective stiffness $K_{R2} := \frac{E_s \cdot I}{L}$ $\boxed{K_{R2} = 4.353 \times 10^{4} \cdot kN \cdot m}$

- section class (AISC Table B4.1)

$B := h - 2 \cdot t_f$ $B = 0.05\,m$

$T_f := t_w$ $T_f = 0.01\,m$

$H := b - 2 \cdot t_w$ $B = 0.05\,m$

$T_w := t_f$ $T_w = 0.025\,m$

- section plastic modulus $Z := \frac{h \cdot b^2 - H \cdot B^2}{4}$ $Z = 3.763 \times 10^6 \cdot mm^3$

$M_p := Z \cdot F_y$ $M_p = 1.298 \times 10^3 \cdot kN \cdot m$

- section elastic modulus $S := \frac{h \cdot b^3 - H \cdot B^3}{6 \cdot b}$ $S = 2.647 \times 10^6 \cdot mm^3$

$M_y := S \cdot F_y$ $M_y = 913.172 \cdot kN \cdot m$

$$M := if\left(\frac{B}{T_f} \le 1.12 \cdot \sqrt{\frac{E_s}{F_y}}, if\left(\frac{H}{T_w} \le 2.42 \cdot \sqrt{\frac{E_s}{F_y}}, M_p, M_y\right), M_y\right)$$

- yield strength $\boxed{M = 1298 \cdot kN \cdot m}$
- post yielding stiffness ratio $PYSR := 0.5\%$
- yielding exponent $EX := 2$

Direction R3 - bending about local axis 3

- moment of inertia $I := I_3$ $I = 2.938 \times 10^7 \cdot mm^4$
- effective stiffness $K_{R3} := \frac{E_s \cdot I}{L}$ $\boxed{K_{R3} = 4.196 \times 10^3 \cdot kN \cdot m}$
- section class (AISC Table B4.1)

$H := h - 2 \cdot t_f$ $H = 0.05\, m$

$T_w := t_w$ $T_w = 0.01\, m$

$B := b - 2 \cdot t_w$ $B = 0.38\, m$

$T_f := t_f$ $T_f = 0.025\, m$

- section plastic modulus $Z := \frac{b \cdot h^2 - B \cdot H^2}{4}$ $Z = 7.625 \times 10^5 \cdot mm^3$

$M_p := Z \cdot F_y$ $M_p = 263.1 \cdot kN \cdot m$

- section elastic modulus $S := \frac{b \cdot h^3 - B \cdot H^3}{6 \cdot h}$ $S = 5.875 \times 10^5 \cdot mm^3$

$M_y := S \cdot F_y$ $M_y = 202.7 \cdot kN \cdot m$

$$M := if\left(\frac{B}{T_f} \le 1.12 \cdot \sqrt{\frac{E_s}{F_y}}, if\left(\frac{H}{T_w} \le 2.42 \cdot \sqrt{\frac{E_s}{F_y}}, M_p, M_y\right), M_y\right)$$

- yield strength $\boxed{M = 263 \cdot kN \cdot m}$
- post yielding stiffness ratio $PYSR := 0.5\%$
- yielding exponent $EX := 2$

B3.4.7 STEP 7 – DEVELOP PUSHOVER CURVE

Create a pushover curve by plotting base shear demands versus the displacement at the top pf the tower. Use a pushover curve to determine the base shear force and link beam rotation for Δ_y, Δ_{SD}, and 3 x Δ_{SD}.

Note that the base shears and other demands are generally lower than the demands found from the empirical calculations.

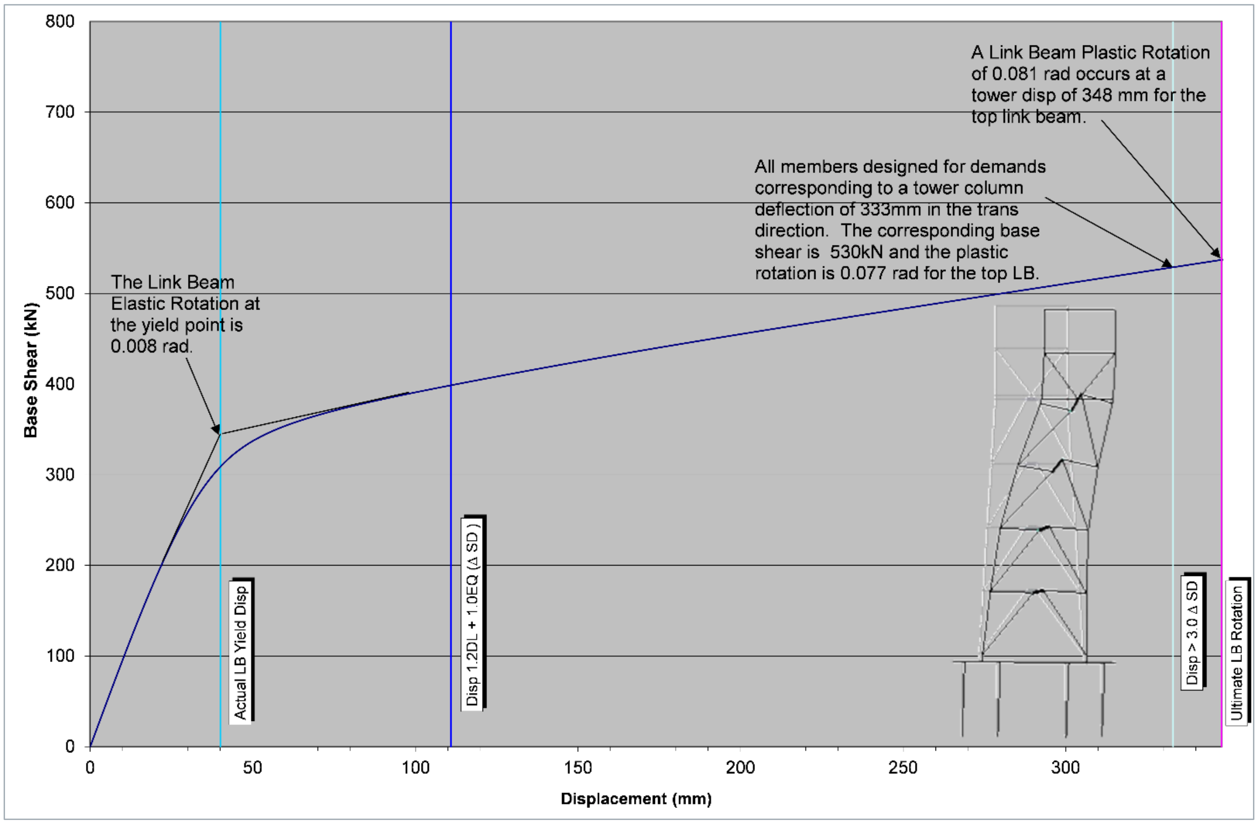


Figure B3 - 7: Tower C pushover results and demands per S6-06

B3.5 PART 2 – DESIGN ACCORDING TO CAN/CSA-S6-14

Design the tower links, columns, and brace members in accordance with CAN/CSA-S6-14 (the Code) (CSA 2014). The link beams should be designed to yield in shear before flexure.

B3.5.1 STEP 1 – DEVELOP DESIGN RESPONSE SPECTRUM

Design Data:

- Importance Factor: I_E = 1.5 (major route bridge)
- Design earthquakes for PBD: 475-year, 975-year, and 2,475-year
- Design earthquake for FBD: 2,475-year
- Site class: D (stiff soil)

Design spectral acceleration values determined from S(T) = F(T)Sa(T), with F(T) using the Code, Tables 4.2 to 4.7:

- 475-year: $S_a(0.2)/PGA = 0.462/0.228 = 2.02 > 2.0$
  - Use PGA to determine F(T)
- 975-year: $S_a(0.2)/PGA = 0.627/0.307 = 2.04 > 2.0$
  - Use PGA to determine F(T)
- 2,475-year: $S_a(0.2)/PGA = 0.871/0.423 = 2.06 > 2.0$
  - Use PGA to determine F(T)

Table B3 - 14: Spectral Ordinates

PERIOD (SECONDS)	S(T) 475-YEAR	S(T) 975-YEAR	S(T) 2,475-YEAR
0	0.50	0.63	0.82
0.2	0.50	0.63	0.82
0.5	0.41	0.53	0.70
1.0	0.23	0.30	0.41
2.0	0.13	0.16	0.22
5.0	0.13	0.16	0.22

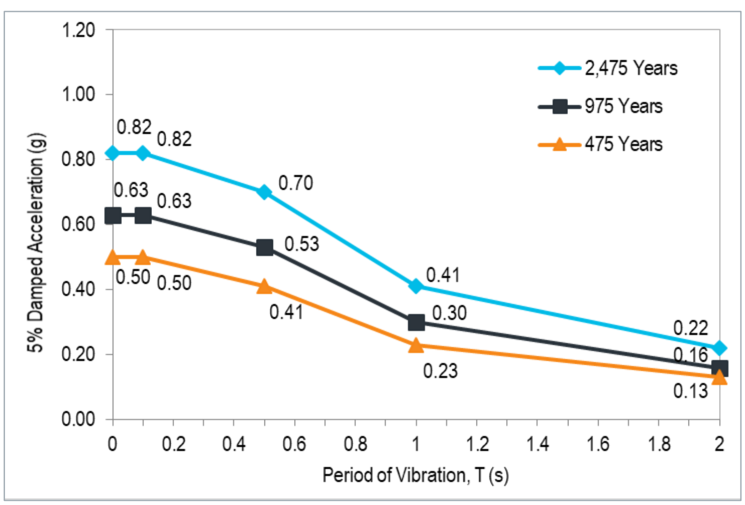


Figure B3 - 8: Design response spectra

B3.5.2 STEP 2 – PERFORM RESPONSE SPECTRUM ANALYSIS

Perform RSA using SAP2000 to determine tower deflections. Try using the same member sizes as for Part 1.

- 475-year deflection: 73 mm
- 975-year deflection: 94 mm
- 2,475-year deflection: 129 mm

B3.5.3 STEP 3 – CHECK EARTHQUAKE DISPLACEMENTS WITH PUSHOVER CURVE

Plot the 475-year, 975-year, and 2,475-year earthquake displacements on the pushover curve developed in Part 1 to compare to the yield displacement and maximum system displacement.

It can be seen from the pushover curve that the shear links yield during the 475-year seismic event does not meet the performance criteria outlined in the Code, Table 4.16.

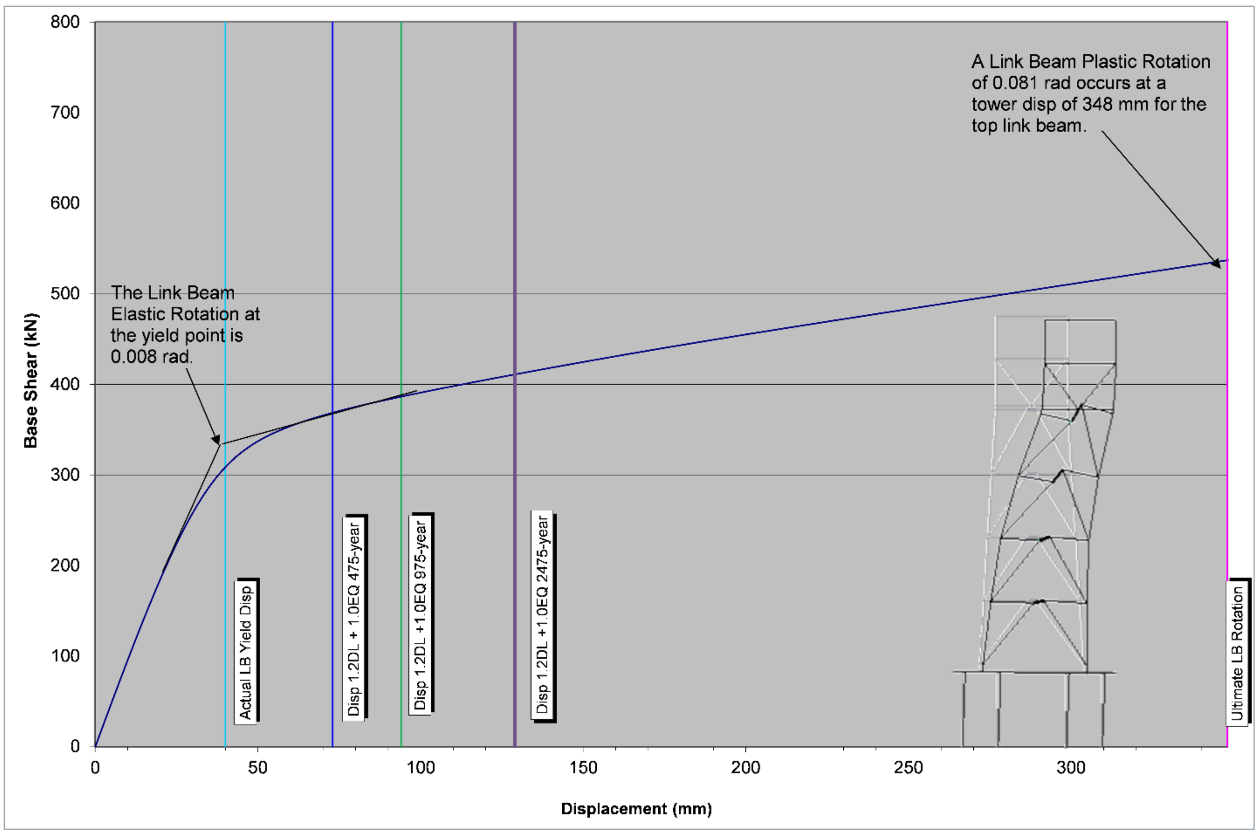


Figure B3 - 9: Tower C pushover results and performance per S6-14

B3.5.4 STEP 4 – ESTIMATE NEW COLUMN AND BRACE MEMBER SIZES

Select new column and brace member sizes. Members must have sufficient capacity to support the factored dead load demands, plus reserve for seismic demands.

Select 1150 x 400 x 40 column members:

- N_{rT} = 24,700 kN
- M_{rT} = 5,800 kN
- V_{rT} = 4,800 kN

Select 400 x 400 x 12 brace members:

- N_{rT} = 4,900
- M_{rT} = 840 kN
- V_{rT} = 1,700 kN

B3.5.5 STEP 5 – ESTIMATE NEW LINK MEMBER SIZES

Select link member sizes. Link members must be proportioned to yield in shear prior to flexural hinging at the link ends.

Select 200 x 400 x 25 x 10 link beam members:

[See calculations below.]

Link Beam Capacities

Table B3 - 15: Link Beam Capacities

LEVEL	COMPRESSION (kN)	MOMENT (kNm)	SHEAR (kN)
4	4,260	580	660
3	3,760	580	660
2	3,270	580	660
1	2,805	580	660

(Note: Red highlighting indicates Mathcad inputs.)

Input:

- section properties of a boxed section

width	$b := 400mm$
height	$h := 200mm$
flange thickness	$t_f := 25mm$
web thickness	$t_w := 10mm$
link length	$e_o := 1400mm$

- material properties:

yield strength	$F_y := 345MPa$	
Elastic Modulus	$E_s := 200000MPa$	
shear yield strength	$F_{yv} := \frac{1}{\sqrt{3}} \cdot F_y$	$F_{yv} = 199 \cdot MPa$

Calculated section properties:

Gross Area $A_g := b \cdot h - (b - 2t_w) \cdot (h - 2t_f)$ $A_g = 23000 \cdot mm^2$ $A_g = 0.023\,m^2$

Web Area $A_w := 2 \cdot (h - 2 \cdot t_f) \cdot t_w$ $A_w = 3000 \cdot mm^2$

yield shear $V_p := F_{yv} \cdot A_w$ $\boxed{V_p = 598 \cdot kN}$

Plastic Section Modulus $Z := t_f \cdot (b - 2 \cdot t_w) \cdot (h - t_f) + \dfrac{t_w \cdot h^2}{2}$ $Z = 1.86 \times 10^6 \cdot mm^3$

plastic moment $M_p := F_y \cdot Z$ $\boxed{M_p = 643 \cdot kN \cdot m}$

$\dfrac{2 \cdot M_p}{e_o} = 918 \cdot kN$ $\phi_v := 0.9$

$M_u := \phi_v \cdot M_p$ $\boxed{M_u = 578 \cdot kN \cdot m}$

nominal shear $V_n := \mathrm{if}\left(V_p \le \dfrac{2 \cdot M_p}{e_o}, V_p, \dfrac{2 \cdot M_p}{e_o}\right)$ $\boxed{V_n = 598 \cdot kN}$

$V_u := \phi_v \cdot V_n$ $\boxed{V_u = 538 \cdot kN}$

for short (shear) links $e_v := 1.6 \cdot \dfrac{M_p}{V_p}$ $e_v = 1.721 \cdot m$

for long (flexural) links $e_f := 2.6 \cdot \dfrac{M_p}{V_p}$ $e_f = 2.8\,m$

check := if$(e_o \le e_v$, "shear link" , if$(e_o \ge e_f$, "flexural link" , "intermidiate link" $))$

$\boxed{\text{check = "shear link"}}$

link rotation $\gamma := \mathrm{if}\left[e_o \le e_v, 0.08, \mathrm{if}\left[e_o \ge e_f, 0.02, 0.02 + (e_f - e_o) \cdot \dfrac{0.08 - 0.02}{e_f - e_v}\right]\right]$

$\boxed{\gamma = 0.08}$

Compactness limits of flange $\lambda_1 := \dfrac{b - 2 \cdot t_w}{t_f}$ $\lambda_1 = 15.2$

$\lambda_f := 0.64 \cdot \sqrt{\dfrac{E_s}{F_y}}$ $\lambda_f = 15.41$

check_f := if$(\lambda_1 \le \lambda_f$, "OK" , "NG"$)$ $\boxed{\text{check\_f = "OK"}}$

Compactness limits of web

$$\lambda_2 := \frac{h - 2 \cdot t_f}{t_w} \qquad \lambda_2 = 15$$

$$\lambda_{w1} := 1.67 \cdot \sqrt{\frac{E_s}{F_y}} \qquad \lambda_{w1} = 40.21$$

$$\lambda_{w2} := 0.64 \cdot \sqrt{\frac{E_s}{F_y}} \qquad \lambda_{w2} = 15.41$$

$$\text{check\_w} := \text{if}\left(e_o \le e_v, \text{if}\left(\lambda_2 \le \lambda_{w2}, \text{"no stiff."}, \text{if}\left(\lambda_2 \le \lambda_{w1}, \text{"stiff. req."}, \text{"NG"}\right)\right), \text{if}\left(\lambda_2 \le \lambda_{w2}, \text{"OK"}, \text{"NG"}\right)\right.$$

$$\boxed{\text{check\_w} = \text{"no stiff."}}$$

Stiffener spacing (if required)

for max. link rotation of 0.08 rad $\quad C_{B1} := 20 \quad a_1 := \left(C_{B1} - \frac{h}{8 \cdot t_w}\right) \cdot t_w \quad a_1 = 0.18\,m$

for max. link rotation of 0.02 rad $\quad C_{B2} := 37 \quad a_2 := \left(C_{B2} - \frac{h}{8 \cdot t_w}\right) \cdot t_w \quad a_2 = 0.35\,m$

stiffener spacing $\quad a := a_1 + (0.08 - \gamma) \cdot \frac{a_2 - a_1}{0.08 - 0.02} \quad \boxed{a = 0.18\,m}$

Link Rotation

maximum shear link rotation $\quad \gamma_{p\_max} := 0.08rad \quad \gamma_{p\_max} = 4.58 \cdot deg$

frame length, height and number of stories with EBF $\quad L := 14.6m \quad H := 9m \quad n := 4$

maximum frame rotation $\quad \theta_{p\_max} := \gamma_{p\_max} \cdot \frac{e_o}{L} \quad \theta_{p\_max} = 0.44 \cdot deg$

maximum story drift $\quad \Delta_p := H \cdot \tan\left(\theta_{p\_max}\right) \quad \Delta_p = 69 \cdot mm$

maximum frame drift $\quad \Delta_{p\_f} := n \cdot \Delta_p \quad \boxed{\Delta_{p\_f} = 276 \cdot mm}$

Note: This is a conservative assumption as it does not include elastic rotation.

B3.5.6 STEP 6 – REPEAT RESPONSE SPECTRA ANALYSIS

Perform RSA using SAP2000 with new member sizes to determine tower deflections:

- 475-year deflection at top of tower: 51 mm
- 975-year deflection at top of tower: 67 mm
- 2,475-year deflection at top of tower: 99 mm

B3.5.7 STEP 7 – REPEAT NONLINEAR PUSHOVER ANALYSIS

Perform a non-linear pushover analysis using SAP2000 with the new member properties, and develop a new pushover curve. The shear link elements are modelled as non-linear link elements, calculated in the same way as for Part 1.

From the pushover curve, it can be seen that the shear links do not yield at the 475-year earthquake, but yield at the 975-year and 2,475-year earthquakes; this meets the performance criteria outlined in the Code, Table 4.16.

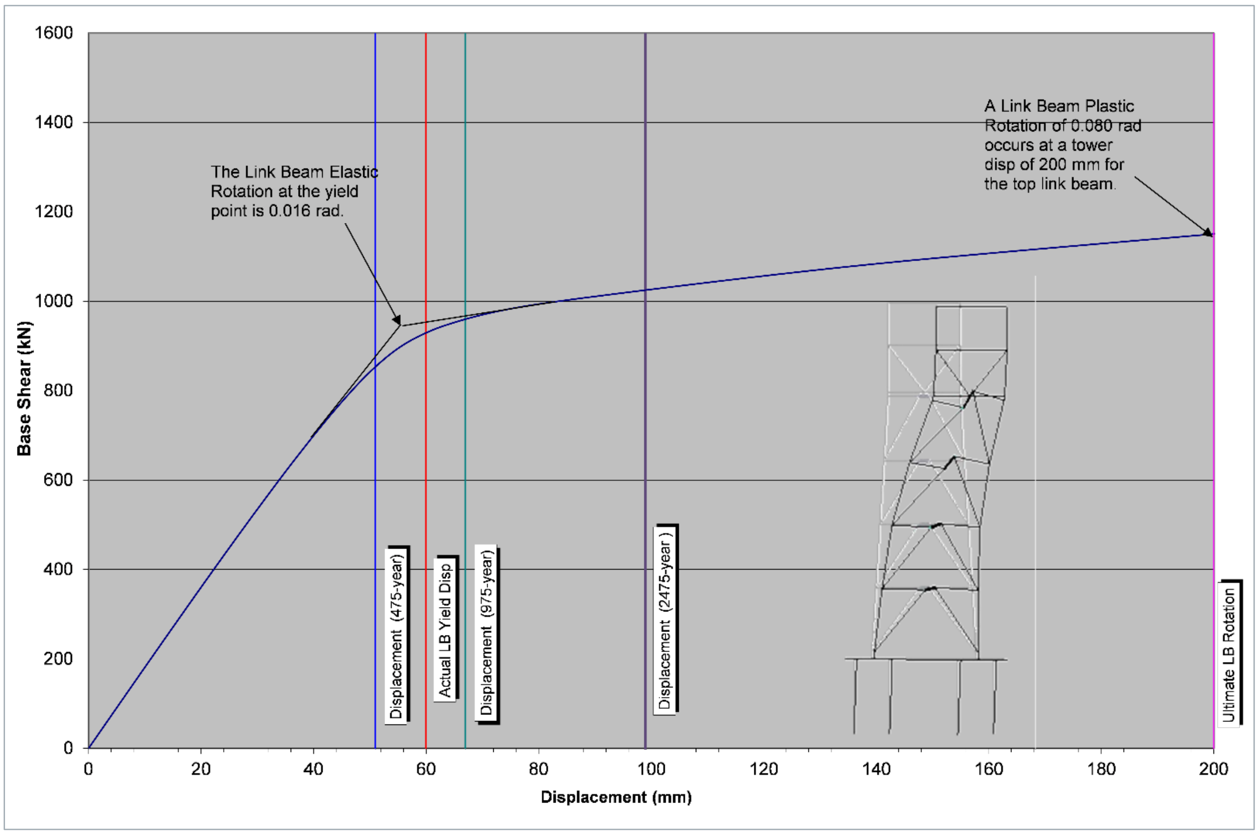


Figure B3 - 10: Tower C (E-Line) pushover results

B3.5.8 STEP 8 – CHECK MEMBER CAPACITIES

Columns

Table B3 - 16: Columns – DL and 475-year Demands

LEVEL	DEAD LOADS			PUSHOVER RESULTS (475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	11,018	33	2	886	37	8
3	11,018	37	5	1,461	32	5
2	11,019	6	1	1,955	139	8
1	11,020	6	1	2,374	650	53

Table B3 - 17: Columns – 975-year and 475-year Demands

LEVEL	PUSHOVER RESULTS (975-YEAR)			PUSHOVER RESULTS (2,475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	1,007	40	9	1,092	40	9
3	1,631	39	6	1,737	41	5
2	2,208	168	9	2,343	222	7
1	2,687	738	58	2,899	797	55

Table B3 - 18: Columns – 475-year D/C

LEVEL	1.2DL + 1.0EQ (475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	14,108	77	10	0.57	0.01	0.00	D/C<1, OK
3	14,683	76	11	0.59	0.01	0.00	D/C<1, OK
2	15,178	146	9	0.61	0.03	0.00	D/C<1, OK
1	15,598	657	54	0.63	0.11	0.01	D/C<1, OK

Table B3 - 19: Columns – 975-year D/C

LEVEL	1.2DL + 1.0EQ (975-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	14,229	80	11	0.58	0.01	0.00	D/C<1, OK
3	14,853	83	12	0.60	0.01	0.00	D/C<1, OK
2	15,431	175	10	0.62	0.03	0.00	D/C<1, OK
1	15,911	745	59	0.64	0.13	0.01	D/C<1, OK

Table B3 - 20: Columns – 2,475-year D/C

LEVEL	1.2DL + 1.0EQ (2,475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	14,314	80	11	0.58	0.01	0.00	D/C<1, OK
3	14,959	85	11	0.61	0.01	0.00	D/C<1, OK
2	15,566	229	8	0.63	0.04	0.00	D/C<1, OK
1	16,123	804	56	0.65	0.14	0.01	D/C<1, OK

Braces

Table B3 - 21: Braces – DL and 475-year Demands

LEVEL	DEAD LOADS			PUSHOVER RESULTS (475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	4	7	1	701	213	26
3	3	5	1	629	213	31
2	2	4	1	532	172	24
1	2	4	0	652	119	17

Table B3 - 22: Braces – 975-year and 475-year Demands

LEVEL	PUSHOVER RESULTS (975-YEAR)			PUSHOVER RESULTS (2,475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	764	234	29	785	243	31
3	733	251	37	777	278	43
2	609	198	27	706	230	32
1	401	135	20	440	149	22

Table B3 - 23: Braces – 475-year D/C

LEVEL	1.2DL + 1.0EQ (475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	706	221	27	0.14	0.26	0.02	D/C<1, OK
3	633	219	32	0.13	0.26	0.02	D/C<1, OK
2	534	177	25	0.11	0.21	0.01	D/C<1, OK
1	654	124	17	0.13	0.15	0.01	D/C<1, OK

Table B3 - 24: Braces – 975-year D/C

LEVEL	1.2DL + 1.0EQ (975-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	769	242	30	0.16	0.29	0.02	D/C<1, OK
3	737	257	38	0.15	0.31	0.02	D/C<1, OK
2	611	203	28	0.12	0.24	0.02	D/C<1, OK
1	403	140	20	0.08	0.17	0.01	D/C<1, OK

Table B3 - 25: Braces – 2,475-year D/C

LEVEL	1.2DL + 1.0EQ (2,475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	790	251	32	0.16	0.30	0.02	D/C<1, OK
3	781	284	44	0.16	0.34	0.03	D/C<1, OK
2	708	235	33	0.14	0.28	0.02	D/C<1, OK
1	442	154	22	0.09	0.18	0.01	D/C<1, OK

Link Beams

Table B3 - 26: Link Beams – DL and 475-year Demands

LEVEL	DEAD LOADS			PUSHOVER RESULTS (475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	160	9	4	372	167	58
3	160	7	3	319	130	30
2	160	6	2	325	108	28
1	160	5	2	228	63	14

Table B3 - 27: Link Beams – 975-year and 475-year Demands

LEVEL	PUSHOVER RESULTS (975-YEAR)			PUSHOVER RESULTS (2,475-YEAR)		
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)
4	416	181	51	442	185	52
3	348	149	34	359	149	32
2	379	123	32	404	142	37
1	263	72	16	321	79	18

Table B3 - 28: Link Beams – 475-year D/C

LEVEL	1.2DL + 1.0EQ (475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	564	178	63	0.13	0.31	0.10	D/C<1, OK
3	511	138	34	0.14	0.24	0.05	D/C<1, OK
2	517	115	30	0.16	0.20	0.05	D/C<1, OK
1	420	69	16	0.15	0.12	0.02	D/C<1, OK

Table B3 - 29: Link Beams – 975-year D/C

LEVEL	1.2DL + 1.0EQ (975-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	608	192	56	0.14	0.33	0.08	D/C<1, OK
3	540	157	38	0.14	0.27	0.06	D/C<1, OK
2	571	130	34	0.17	0.22	0.05	D/C<1, OK
1	455	78	18	0.16	0.13	0.03	D/C<1, OK

Table B3 - 30: Link Beams – 2,475-year D/C

LEVEL	1.2DL + 1.0EQ (2,475-YEAR)			D/C			
	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	AXIAL (kN)	MOMENT (kNm)	SHEAR (kN)	
4	634	196	57	0.15	0.34	0.09	D/C<1, OK
3	551	157	36	0.15	0.27	0.05	D/C<1, OK
2	596	149	39	0.18	0.26	0.06	D/C<1, OK
1	513	85	20	0.18	0.15	0.03	D/C<1, OK

Based on the results, the column, brace, and link beam members selected all have sufficient capacity to carry the design loads.

The revised structure meets the performance criteria in the Code, Table 4.16. It is noted that the ultimate displacement capacity of the structure is reduced due to the design change, although it is still well beyond the displacement demand of the 2,475-year event.

B3.6 REFERENCES

Canadian Standards Association (CSA). 2014. Canadian Highway Bridge Design Code. CSA Standard S6-14. (December 2014). Toronto, ON: Canadian Standards Association.

CSA. 2006. Canadian Highway Bridge Design Code. CSA Standard S6-06. Toronto, ON: Canadian Standards Association.

Natural Resources Canada (NRC). 2010. 2010 National Building Code of Canada Seismic Hazard Calculator. [website]. Ottawa, ON: NRC. [date modified: 2016-02-10]. [accessed 2018 Jan 23]. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2010-en.php.

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