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A Static Analysis-Based Seismic Design Method for Upper-Deck Steel Arch Bridges against Level 2 Ground Motions

Osman Tunc Cetinkaya

A dissertation submitted to Nagasaki University in partial fulfillment of the requirements for the degree of Doctor of Philosophy

Graduate School of Science and Technology
Nagasaki University

December 2007
A Static Analysis-Based Seismic Design Method for Upper-Deck Steel Arch Bridges against Level 2 Ground Motions

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Osman Tunc Cetinkaya
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ABSTRACT

This dissertation presents a simplified seismic design method for upper-deck steel arch bridges against level 2 earthquake ground motions. The method is based on the comparison of the seismic demand and capacity of the bridge components. Simplification is achieved by eliminating the dynamic response analysis from the seismic demand estimation and generation of ductility equations for the prediction of failure strain of thin-walled members of either pipe or box sections. The influence of axial force fluctuation on the ductility is also studied and incorporated in the design formulae since large axial force fluctuations are induced at the arch ribs of the steel arch bridges during severe ground motion excitations.

The estimation of seismic demand without dynamic response analysis is studied through the numerical analysis of parametric upper-deck steel arch bridge models. The equal-energy assumption is applied on the results of pushover analysis and response spectrum method to predict the inelastic response at the reference points where the maximum structural response is observed. Applicability of equal-energy assumption is investigated for the transverse and longitudinal Level 2 ground motion excitations and certain correction functions are proposed in order to improve the estimation accuracy. Having improved the estimates of the maximum structural response, the seismic demand of whole structural members can be obtained from the pushover analysis.

The design formulae to be used for the capacity evaluations are generated by studying the ductility of parametric short steel cylinders and short box columns under combined compression and bending. Influence of axial force fluctuation is assessed by comparing the bending behavior of constant and fluctuating axial force cases. It is found that ductile capacity corresponding to the post-peak region of bending behavior is significantly improved when axial force fluctuation is considered. Design formulae for failure strain taking into account this capacity improvement are proposed for different limit states. The validity of the proposed formulae is demonstrated through numerical analysis.
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NOTATIONS

\( k_{hc} \): Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion.

\( k_{hc0} \): Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion.

\( c_s \): Force reduction factor.

\( c_z \): Modification coefficient for earthquake zone.

\( \mu_a \): Allowable ductility factor for the structural system having a plastic force displacement relation.

\( S \): Acceleration response spectra for Level 1 Earthquake Ground Motion.

\( S_I \): Acceleration response spectra for Level 2 Type I Earthquake ground motion.

\( S_{II} \): Acceleration response spectra for Level 2 Type II Earthquake ground motion.

\( c_z \): Modification factor for zones.

\( h \): Modal damping ratio.

\( c_D \): Modification factor for damping ratio.

\( S_0 \): Standard acceleration response spectra for Level 1 Earthquake Ground Motion.

\( S_{I0} \): Standard acceleration response spectra for Level 2 Type I Earthquake Ground Motion.

\( S_{II0} \): Standard acceleration response spectra for Level 2 Type II Earthquake Ground Motion.

\( \sigma_y \): Yield displacement.

\( \varepsilon_y \): Yield strain.

\( E \): Young Modulus.

\( \nu \): Poisson’s ratio.

\( \delta_{SP} \): Estimated maximum non-linear dynamic response.

\( \delta_{DP} \): Maximum inelastic dynamic response.

\( \delta_{DE} \): Maximum elastic dynamic response.

\( \{H_i\} \): Lateral force matrix of pushover analysis.

\( \{\phi_i\} \): Eigenvector.

\( m_i \): Mass component of the structural mass matrix.
\( \phi \): Transverse component of the eigenvector.

\( a_i \): lateral acceleration at node i.

\( \delta_y \): Yield displacement.

\( \mu_E \): Estimated ductility factor.

\( \mu \): Ductility factor.

\( f(\mu_E) \): Correction function.

\( \mu' \): Corrected ductility factor.

\( \delta_{SP} \): Corrected estimated maximum inelastic response.

\( P_\delta \): Incremental displacement load.

\( R_t \): Radius-thickness ratio parameter of pipe sections.

\( D \): Diameter of cylinder.

\( t \): Plate thickness.

\( L_{cr} \): Critical cylinder length.

\( E' \): Assumed strain-hardening modulus for steel short cylinder and short box column models.

\( E_{st} \): Strain-hardening modulus parameter.

\( \eta \): Strain hardening-modulus parameter.

\( \xi \): Material Coefficient.

\( \varepsilon_{st} \): Strain at the onset of strain hardening.

\( w \): Outward displacement of initial deflection.

\( w_{max} \): Maximum outward displacement of initial deflection

\( P_y \): Squash load.

\( P_i \): Initial axial force.

\( P_f \): Final axial force.

\( \alpha \): Axial force fluctuation amount.

\( M \): Bending moment.

\( N \): Axial force.

\( e \): Eccentricity.

\( M_1/M_2 \): Moment gradient.

\( M_y \): Bending moment causing the yielding of the section.

\( \theta \): Rotation angle of the section during bending.

\( \theta_y \): Rotation angle of the section during bending at the yielding instant.

\( \varepsilon_u \): Failure strain.
$M_{95}$ : Limit state for failure strain defined as the strain corresponding to the 95% of the maximum moment after the peak.

$M_{90}$ : Limit state for failure strain defined as the strain corresponding to the 90% of the maximum moment after the peak.

$M_{80}$ : Limit state for failure strain defined as the strain corresponding to the 80% of the maximum moment after the peak.

$\varepsilon_u$ : Failure strain.

$\varepsilon_{N,\text{fixed}}$ : Failure strain for constant axial force case.

$\varepsilon_{N,\text{fluct}}$ : Failure strain for fluctuating axial force case.

$f(R_t,P_f/P_y,\alpha)$ : Correction function for pipe sections accounting for the influence of axial force fluctuation.

$f(R_t,P_f/P_y)$ : Constant axial force case formula for pipe sections.

$R_f$ : Flange width-thickness ratio of thin-walled box sections.

$b$ : Flange width.

$n$ : Number of subpanels separated by the stiffener plates.

$\bar{\lambda_s}$ : Stiffener plate slenderness ratio.

$\gamma$ : Stiffener plate relative flexural rigidity.

$\gamma^*$ : Optimum value of stiffener plate’s relative flexural rigidity.

$\gamma^*/\gamma^*$ : Ratio of stiffener plate’s relative flexural rigidity to its optimum value.

$r_s$ : Radius of gyration of the T-shaped cross section which consist of one longitudinal stiffener and adjacent subpanels.

$Q$ : Local buckling strength of the subpanel plate.

$I_1$ : Stiffener plate section inertia moment with respect to its end connected to the plate.

$b_s$ : Width of the stiffener plate.

$t_s$ : Thickness of the stiffener plate.

$\alpha_a$ : Aspect ratio.

$\alpha_0$ : Critical aspect ratio.

$L_b$ : Length of stiffened steel box short columns

$\delta$ : Total inward initial deflection of flange or web plates of stiffened steel box short columns.

$\delta_G$ : Global initial deflection of flange or web plates.

$\delta_L$ : Local initial deflection of flange or web plates.
$m$ : Number of half-waves of local initial deflections in the longitudinal direction.

$f(R_t, P_f/P_y)$ : Correction function for stiffened steel box sections accounting for the influence of axial force fluctuation.

$D_s$ : Failure criterion.

$L_e$ : Effective failure length of a steel bridge member.
CHAPTER 1.

INTRODUCTION
1.1 Background

The Hyogo-ken Nanbu earthquake of 17 January 1995, which was more severe earthquake than that considered in the design code for structures, caused destructive damage to many structures [1]. Steel bridges were no exception. The range of damage included the collapse of steel bridge piers, as well as local buckling of stiffened box and pipe sections [2]. After the extensive research efforts to understand the damage by this earthquake, Japanese Seismic Design Code for Highway Bridges (JRA code) [3-4] was revised. The latest code specifies a performance-based design to be conducted by comparing the demand and the capacity of bridge members for different performance levels against two-levels of ground motions which are moderate (called Level 1) and extreme (called Level 2) ground motions. Three performance levels are introduced to be verified depending on the importance of the bridge and the level of the considered ground motions. Against the Level 2 ground motions, a performance level allowing damages due to inelastic behavior is specified. The verification of this performance level creates the necessity to evaluate the inelastic seismic behavior for all structures.

For the estimation of the inelastic seismic demand, the powerful method of non-linear dynamic response (time-history) analysis is the most rigorous analysis method. However, implementation is time consuming, which hampers its wide application to everyday design. There is a desire for a method of seismic design that does not rely on dynamic response analysis. The JRA code specifies a simplified method called the Ductility Design Method, which is based on static analysis. This is a force-based design procedure utilizing elastic analysis in which a force reduction factor is adopted to account for inelastic behavior. The force reduction factor is calculated using the equal-energy assumption [5], which assumes the elastic energy stored in the elastic and inelastic systems is identical. However, the application of this method is limited only to simple structures, because the applicability of the equal-energy assumption is not clear in the case of structures with complex dynamic response characteristics. In the JRA code, simple dynamic behavior implies that the structure is a system with a predominant first vibration mode and the possible location of the primary plastic hinge can be easily foreseen. This confines use of the method to reinforced concrete piers and steel piers in-filled with concrete. For other structures, referred to as 'complicated structures' by the JRA code (including steel arch bridges), dynamic response analysis should be conducted for seismic performance verification.
Steel arch bridges were conventionally treated as structures for which earthquake loading is not predominant, as they are normally built in mountainous areas with little chance of major earthquakes, since ocean-type earthquakes are common in Japan. Moreover, even if experienced, earthquake excitation was not thought to be crucial, because arches are structures of relatively long natural period and are generally built on rock foundations. For this reason, conventional design took into consideration only moderate earthquakes, during which the structure should remain in the elastic range. However, the compulsory evaluation of inelastic seismic demand and the capacity against the Level 2 ground motions greatly complicates design process of arch bridges compared to the conventional practice.

The verification of the capacity of individual bridge members against the estimated seismic demand is crucial especially for the arch ribs and the side piers of arch bridges since they are the members subjected to most severe loading which may put them into critical condition at their support sections due to the occurrence of local buckling. According to the new provision, the capacity of the steel components has to be determined either by conducting cyclic loading tests using specimens or analysis capable of considering local buckling effects. In practice, the results of investigations that include loading tests or numerical evaluations of similar structure are used instead of time-consuming cyclic loading tests or elasto-plastic large-displacement analysis. But there is a great need for simplified calculation methods which can express the ultimate strength and deformation of structural members. In particular, ductile capacity is very important since deformation-based design is a more rational approach to seismic design for extreme ground motions. The ductility of the structural members needs to be assessed in accordance with the philosophy of performance-based design by incorporating their capacity in the inelastic region even after buckling. The real state behaviors which are likely to have influence on the ductility needs to be carefully measured in order to achieve a rational seismic design method.

1.2 Previous Works Done in the Field

Since the devastating Hyogo-ken Nanbu earthquake, many efforts to improve the seismic performance of steel structures have been made in Japan. These efforts began with the simplest and most common structures such as cantilevered steel piers and portal frame piers. The strength and ductility of these structures under cyclic loading
have been examined experimentally or numerically [6-12]. With time the trend has
shifted to clarify the inelastic seismic behavior of more rare but complex structures,
such as steel truss [13], arch [14-21] and elevated bridges [22-23]. Recently, also, more
interest is being given in the development and application of vibration control devices to
structures [24]. Some findings have been introduced into the revised version of the JRA
code.

The adoption of the Level 2 ground motion attracted the attention of many
researchers [14-21] to understand the inelastic characteristics of steel arch bridges since
severe earthquake loading could put them in a critical situation. Usami et al. [19]
investigated the inelastic seismic performance of a typical upper-deck steel arch bridge
subjected to major earthquakes. They found that seismic responses are small under
longitudinal ground motion input but severe plasticization and performance deficiencies
are observed under transverse excitation. This study proves that Level 2 ground motion
can be critical for upper-deck steel arch bridges.

Meanwhile, the elimination of dynamic response analysis from the seismic design
has always been a hot topic for various types of structures. As a powerful seismic
evaluation tool, the static non-linear pushover analysis has become popular due to its
simplicity compared with the conventional dynamic time-history analysis procedure
[20-21, 25-34] and recommended for seismic evaluation in some provisions [35-37].
Pushover analysis can provide an insight into structural aspects that control performance
during earthquakes. Lu et al. [20-21] utilized pushover analysis for the equivalent
single-degree-of-freedom (SDOF) system approximation of upper-deck steel arch
bridges and estimate the inelastic seismic demand through the dynamic response
analysis of the SDOF system. Although the method is very reliable, it is still necessary
to carry out dynamic response analysis.

As an alternative way to estimating the inelastic seismic demand, the equal-energy
assumption can be utilized. The inelastic demand can be computed by equating the
elastic and inelastic energy demands through the combination of the results of response
spectrum method and pushover analysis. However, the applicability of the equal-energy
assumption to complicated structures is questionable since it was originally proposed
for SDOF systems. There have been some previous reports on the applicability of the
equal-energy assumption to steel bridges. Usami et al. [38] examined the applicability
of both equal-energy and equal-displacement assumptions through pseudo-dynamic
tests of cantilevered columns in steel bridge piers. They found that a fairly good estimation of non-linear response was achieved by using the equal-energy assumption, while the response predicted by the equal-displacement assumption was much smaller than in the actual tests. Nakajima et al. [39] investigated the applicability of the equal-energy assumption to the seismic design of steel portal frames. The paper concludes that it gives a conservative estimation of the maximum non-linear response, but the estimated maximum displacement can be much larger than that given by elasto-plastic dynamic response analysis. Nakamura et al. [40] also investigated the applicability of the equal-energy assumption to steel portal frames. Their study showed that the equal-energy assumption results in a conservative prediction of maximum response, with the results being too conservative in many cases. They also suggested some correction functions that improve estimation accuracy. It can be considered that a similar approach is also applicable to the inelastic seismic demand estimation of steel arch bridges.

The capacity evaluation of the members in steel arch bridges is very much linked to the ultimate capacity of thin-walled steel sections since generally the cross sections of either pipe or box shapes are employed for the arch ribs and the side piers. There are some previous investigations which have involved the study of the ductility of cylinders or box columns subjected to pure compression, bending or combined compression and bending [41-52]. As for steel bridge piers subjected to combined compression and bending, a number of experiments and numerical analyses have shown that local buckling of thin-walled steel structures always happens in the compressive flange within the effective failure range [53-56] and that maximum structural ductility is governed by the capacity of this critical local part. For the pipe sections, the ductility of this part was investigated by Gao et al. [45] through numerical analysis of short cylinders subjected to monotonic loading. The results were compared with earlier loading test results [57] and empirical ductility equations for short steel cylinders, which are expected to simulate the behavior of the local buckling part, were presented. A similar work is conducted by Zheng et al [46] to generate ductility equations for the short steel box columns. The formulations of Gao et al. and Zheng et al. were modified by Ge et al. [58] to extend their applicability to a wider range of axial force magnitudes. The modified formulation is very reliable for use under constant axial force considerations. However, in many structures the axial force fluctuates considerably.
along with the bending moment during an earthquake. This fluctuation is significant and may have some influence on capacity, especially in portal frame bridge piers and arch bridges. Aoki and Susantha [12] conducted cyclic loading tests using a varying axial load on individual column specimens from portal frames. They found that ductility was slightly improved compared to the constant axial load case under a loading condition where a small amount of variation at a moderate axial force magnitude is considered. This finding suggests that different degrees of improvement may be obtained under different loading conditions. For this reason, it is necessary to evaluate the influence of axial force fluctuation on the ductility of steel members under a variety of structural and loading conditions and establish formulae that take this real state behavior into account. Such formulae would contribute to the rationalization of seismic design taking improved ductility into account.

1.3 Objectives

The main aim of the research introduced in this dissertation is to establish a simplified seismic design method for steel arch bridges by eliminating the difficulties of demand and capacity predictions in the design against the Level 2 ground motions. The primary focus is directed to the upper-deck type since they are more sensitive to severe ground motion excitations compared to the other types of arch bridges.

Elimination of the dynamic response analysis constitutes the main objective to simplify the seismic demand estimation of upper-deck steel arch bridges. Pushover analysis and response spectrum method are adopted together with the equal-energy assumption for the estimation of maximum dynamic response. Numerical investigations revealed that application of the equal-energy assumption results in too conservative estimates for both the out-of-plane and in-plane estimations. However certain correction functions are developed to improve the accuracy. Correction functions are used to combine the response spectrum method with the pushover analysis to constitute a method of inelastic seismic demand prediction which is based on static analysis.

Simplification of the capacity evaluations of the bridge components is carried out through the generation of ductility formulae which can also express the influence of axial force fluctuation. The formulae are proposed for short steel cylinders and short box columns respectively. These short members are studied under combined compression and bending and the bending behavior of various constant and fluctuating
axial force cases are compared to evaluate the effect of axial force fluctuation on ductility. It is found that consideration of axial force fluctuation has improving influence on the ductility at various limit states defined in the post-buckling region. By the consideration of this improving effect in the ductility equations, the capacity evaluations are aimed not only to be simplified but also to be made more rational.

1.4 Contents and Layout of the Dissertation

A simplified seismic design method for upper-deck steel arch bridges based on demand and capacity comparison is proposed in this dissertation. The dissertation is composed of 7 chapters as explained below and illustrated in Figure 1.1.

Chapter 1 gives the background and objectives of the research together with a list of major works conducted previously in the related field.

Chapter 2 briefly explains the main concepts of the current Japanese Seismic Design Code for Highway Bridges.

Chapter 3 deals with the maximum inelastic out-of-plane response estimation without the need of non-linear dynamic response analysis. Numerical analyses are carried out on 6 parametric upper-deck steel arch bridge models. Applicability of equal-energy assumption is investigated by comparing the response estimated by using the equal-energy assumption with the dynamic response analysis results. Although the estimates are found to be on the conservative side, the accuracy was too low in many cases. However, some solid tendencies are found that make it possible to generate certain correction functions for improving the estimation accuracy. The generated correction functions are combined with the pushover analysis and the response spectrum method to establish a static analysis-based method for the maximum out-of-plane response estimation.

In Chapter 4, the method proposed for the out-of-plane response is discussed for its applicability to the in-plane response estimation. The investigations are conducted numerically on the same bridge models of the previous chapter. First, the load pattern that should be used in the pushover analysis is examined. Then, applicability of equal-energy assumption is studied for the in-plane response estimation. Finally, it is found that the method proposed for the out-of-plane response estimation can be also used for the estimation of in-plane response only by modifying the pushover analysis procedure.
In Chapter 5, generation of ductility equations for the bridge members with pipe sections which can consider the influence of axial force fluctuation is discussed. Ductility is studied on parametric short steel cylinder models under combined compression and bending. Bending behavior of constant and fluctuating axial force cases are compared. It is found that ductility corresponding to the post-peak region of bending behavior is significantly improved when the axial force fluctuation is considered. Design formulae for failure strain taking into account this capacity improvement are proposed and their validity and efficiency are verified through numerical analysis.

The similar investigation is carried out on the bridge components with stiffened box section in Chapter 6. Parametric stiffened short steel box columns are generated and bending behavior of constant and fluctuating axial force cases are compared. Axial force fluctuation is again found to have an improving effect on the post-peak ductility. Design formulae for the failure strain are proposed and their validity and efficiency are demonstrated through numerical analysis.

In Chapter 7, how to carry out the seismic design of upper-deck steel arch bridges through the application of the proposed demand and capacity prediction methods is discussed. The main findings of each chapter are also summarized. Finally, the points that need to be solved in the future work are indicated.
CHAPTER 2
The Japanese Seismic Design Code
• Outline of the Japanese Seismic Design Code for Highway Bridges

PREDICTION OF SEISMIC DEMAND

CHAPTER 3
Out-of-plane Response Estimation
• Generation of parametric models
• Applicability of equal-energy assumption
• Generation of correction functions
• Method based on static analysis for the inelastic response estimation

CHAPTER 4
In-plane Response Estimation
• Load pattern for pushover analysis
• Applicability of equal-energy assumption
• Applicability of correction functions of Chapter 3.
• Application of the simplified method for the in-plane response estimation.

PREDICTION OF CAPACITY

CHAPTER 5
Ductility Formulae for Pipe Sections
• Numerical analysis of short steel cylinders
• Assessment of the influence of axial force fluctuation on ductile capacity.
• Generation of design formulae for the failure strain considering the influence of axial force fluctuation.

CHAPTER 6
Ductility Formulae for Box Sections
• Numerical analysis of stiffened short steel box columns
• Assessment of the influence of axial force fluctuation on ductile capacity.
• Generation of design formulae for the failure strain considering the influence of axial force fluctuation.

Chapter 7
Proposed Seismic Design Method and Concluding Remarks
• Basic steps involved in the application of the method.
• Summary of the findings of each chapter
• Future work

Figure 1.1. Layout of the dissertation
CHAPTER 2.

SEISMIC DESIGN PRACTICE IN JAPAN
This chapter introduces the outline of the seismic design of Highway Bridges specified by the JRA code including the new concepts adopted after the Hyogo-ken Nanbu earthquake.

2.1 Principles of Seismic Design

Two levels of design earthquake ground motions are specified for the seismic design of a bridge: The first level corresponds to an earthquake with high probability of occurrence during the bridge service life (called “Level 1 Earthquake Ground Motion”), and the second level corresponds to an earthquake with less probability of occurrence during the bridge service life but strong enough to cause critical damage (called “Level 2 Earthquake Ground Motion”). For the Level 2 Earthquake Ground Motion, two types of earthquake ground motions having different characteristics shall be taken into account, namely, Type I of a plate boundary earthquake with large magnitude like the great Kanto Earthquake and Type II of an inland direct strike type earthquake like the Hyogo-ken nanbu earthquake. Type I represents the one with large magnitude and longer duration, while Type II motion is the one with strong accelerations and shorter duration.

Depending on the social functions, roles for disaster reduction efforts after an earthquake, and influences of function losses, bridges are classified into two groups: bridges of standard importance (Class A), and bridges of high importance (Class B).

Seismic performances of bridges as a target of seismic design are classified into three levels in view of the seismic behavior of the bridge:

1) Seismic Performance Level 1 “Performance level of a bridge keeping its sound functions during an earthquake”: The structure should behave in an elastic manner without any essential damage. The bridge shall be protected safely from unseating, no emergency repair is needed to recover the functions soon after the earthquake, and also repair work which may take a long time can be easily conducted.

2) Seismic Performance Level 2 “Performance level of a bridge sustaining limited damages during an earthquake and capable of recovery within a short period”: This performance can ensure not only the safety of unseating prevention, but also capability of recovering the functions soon after the event as well as reparability by a comparatively easy long-term repair work.
3) Seismic Performance Level 3 “Performance Level of a bridge sustaining no critical damage during an earthquake”: The safety against unseating should be ensured, but does not cover the functions necessary for serviceability and reparability for seismic design.

**Table 2.1** summarizes items of Seismic Performances 1 to 3 in view of safety, serviceability and reparability for seismic design. Safety implies performance to avoid loss of life due to unseating of superstructure during an earthquake. Serviceability means that a bridge is capable of keeping its bridge functions such as fundamental transportation function, role of evacuation routes and emergency routes for rescue, first aid, medical services, firefighting and transportation of emergency goods to refugees. Reparability denotes capability of repairing seismic damages.

**Table 2.1. Seismic Performance of Bridges**

<table>
<thead>
<tr>
<th>Seismic Performance</th>
<th>Seismic Safety Design</th>
<th>Seismic Serviceability Design</th>
<th>Seismic Reparability Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Performance Level 1: Keeping the sound functions of bridges</td>
<td>To ensure the safety against girder unseating</td>
<td>To ensure the normal functions of the bridges</td>
<td>No repair work is needed to recover the functions</td>
</tr>
<tr>
<td>Seismic Performance Level 2: Limited damages and recovery</td>
<td>Same as above</td>
<td>Capable of recovering functions within a short period after the event</td>
<td>Capable of recovering functions by emergency repair works</td>
</tr>
<tr>
<td>Seismic Performance Level 3: No critical damages</td>
<td>Same as above</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

A performance based design approach is specified which targets one of the above seismic performance levels for the seismic behavior of the bridge depending on its importance and levels of design earthquake motions. According to this approach the seismic design should conform to the following.

1) Both Class A and Class B bridges shall be designed so that the Seismic Performance Level 1 is ensured to the Level 1 Earthquake Ground Motion.
2) To the Level 2 Earthquake Ground Motion, Class A bridges shall be designed so that the Seismic Performance Level 3 is ensured, while Class B bridges should be designed so that the Seismic Performance Level 2 is ensured.

These target performance levels for different bridge classes and ground motion levels are summarized in **Table 2.2**.

**Table 2.2. Design Earthquake Ground Motions and Seismic Performance of Bridges**

<table>
<thead>
<tr>
<th>Earthquake Ground Motions</th>
<th>Class A Bridges</th>
<th>Class B Bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1 Earthquake Ground Motion (highly probable during the bridge service life)</td>
<td>Keeping sound functions of bridges (Seismic Performance Level 1)</td>
<td></td>
</tr>
<tr>
<td>Level 2 Earthquake Ground Motion</td>
<td>Type I Earthquake Ground Motion (a plate boundary type earthquake with a large magnitude)</td>
<td>No critical damages (Seismic Performance Level 3)</td>
</tr>
<tr>
<td></td>
<td>Type II (an inland direct strike type earthquake like Hyogo-ken Nanbu Earthquake)</td>
<td>Limited seismic damages and capable of recovering bridge functions within a short period (Seismic Performance Level 2)</td>
</tr>
</tbody>
</table>

### 2.2 Verification of Seismic Performance

In verifying the seismic performance, the limit state of each structural member shall be appropriately determined in accordance with the target performance level of the bridge. Limit states for Seismic Performance Level 1 shall be properly established so that the mechanical properties of the bridge are maintained within the elastic ranges. For each structural member, the stress induced by an earthquake shall not exceed its allowable value. For the limit states of performance level 2 and performance Level 3 plastic behavior is also taken into account. The structural member, in which the generations of plastic behavior are allowed, deforms plastically within a range of easy functional recovery for performance Level 2. The limit states for performance Level 3 are generated in a way that the plastic behavior is allowed to take place within a range of the ductility limit of the member without the concern of functional recovery.

The verification shall be performed so that the state of each structural member of a bridge due to the design seismic force does not exceed its limit state. The general
verification procedure is illustrated in Figure 2.1. The verification is carried out first for Level 1 Earthquake Ground Motion and then for Level 2 Earthquake ground motion by employing either static analysis or dynamic response analysis. Static Analysis is applicable to bridges which have no complicated seismic behavior. For the bridges with complicated seismic behavior, dynamic analysis is required.
The bridge is seismically complex

Calculation of design horizontal seismic coefficients and inertia forces (Seismic Coefficient Method)

Calculation of sectional forces and displacements by static analysis.

Verification of seismic performance for Level 1 Earthquake Ground Motion

Calculation of Limit States for Performance Level 1 (Allowable stresses, etc)

Calculation of responses by dynamic analysis.

Verification for Level 1 Earthquake Ground Motion

Calculation of limit states for performance Level 2 or Level 3 depending on the bridge class (Ultimate horizontal strength, allowable displacements, etc)

Calculation of design horizontal seismic coefficients and inertia forces (Ductility Design Method)

Calculation of sectional forces and displacements by static analysis.

Verification of seismic performance for Level 2 Earthquake Ground Motion

Design for Unseating Prevention System

Figure 2.1. Seismic Design Flowchart
2.2.1 Verification of seismic performance based on static analysis

In static analysis, responses can be obtained by substituting static loads for the reactions induced in structures or in the ground due to effects of earthquake, so that seismic behavior could be comparatively simply estimated. The method is applicable only to bridges without complicated seismic behavior which means that the structure is a system with a predominant first vibration mode and clear location where primary plastic behavior generates in case of Level 2 earthquake motions is easy to predict.

Static analysis-based verification methods include two kinds of approaches, Seismic Performance Verification for Level 1 Earthquake Ground Motion and Seismic Performance Verification for Level 2 Earthquake Ground Motion called as "Seismic Coefficient Method" and "Ductility Design Method" in the previous editions of the JRA code, respectively. The former refers to the design method in which vibration characteristics of elastic range is considered while the latter is the method in which deformation property and dynamic strength of non-linear zone of a structure is taken into account. Both of the approaches employ design horizontal seismic coefficients that convert the dynamic forces into static ones. Static inertia forces obtained by multiplying these coefficients with the structural weight are applied to the structure in lateral direction in order to estimate the seismic response.

1) For Level 1 earthquake ground motion

In the verification for Level 1 Earthquake Ground Motion, the first mode of vibration in elastic range of the objective structure is taken into account and associated elastic responses can be estimated by substituting static forces for the seismic reactions. Stresses or displacements resulted from the responses is then confirmed to be less than each allowable value of the limit states for Seismic Performance Level 1.

The design horizontal seismic coefficient to be used for this method is defined by equation (2.1) in terms of the standard value of the design horizontal seismic coefficient presented in Table 2.3. However, if the value obtained from this equation is less than 0.1, the seismic coefficient is set to 0.1

\[ k_h = c_2 k_{h0} \]  

(2.1)

where,

\[ k_h \]:Design horizontal seismic coefficient.
$k_{h0}$: Standard value of the design horizontal seismic coefficient for Level 1 Earthquake Ground Motion shown in Table 2.3.

$c_Z$: Modification coefficient for zone, as shown in Figure 2.2.

**Table 2.3.** Standard Values of the Design Horizontal Seismic Coefficient for Level 1 Earthquake Ground Motion, $k_{h0}$

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>$k_{h0}$ value for natural period $T$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Group I (stiff)</strong></td>
<td></td>
</tr>
<tr>
<td>$T&lt;0.1$</td>
<td>$k_{h0}=0.431 T^{4/3}$</td>
</tr>
<tr>
<td>But $k_{h0} \geq 0.16$</td>
<td></td>
</tr>
<tr>
<td>$0.1 \leq T \leq 1.1$</td>
<td>$k_{h0}=0.2$</td>
</tr>
<tr>
<td>$1.1 &lt; T$</td>
<td>$k_{h0}=0.213 T^{2/3}$</td>
</tr>
<tr>
<td><strong>Group II (Moderate)</strong></td>
<td></td>
</tr>
<tr>
<td>$T&lt;0.1$</td>
<td>$k_{h0}=0.427 T^{4/3}$</td>
</tr>
<tr>
<td>But $k_{h0} \geq 0.20$</td>
<td></td>
</tr>
<tr>
<td>$0.2 \leq T \leq 1.3$</td>
<td>$k_{h0}=0.25$</td>
</tr>
<tr>
<td>$1.3 &lt; T$</td>
<td>$k_{h0}=0.298 T^{2/3}$</td>
</tr>
<tr>
<td><strong>Group III (soft)</strong></td>
<td></td>
</tr>
<tr>
<td>$T&lt;0.1$</td>
<td>$k_{h0}=0.430 T^{4/3}$</td>
</tr>
<tr>
<td>But $k_{h0} \geq 0.24$</td>
<td></td>
</tr>
<tr>
<td>$0.34 \leq T \leq 1.5$</td>
<td>$k_{h0}=0.3$</td>
</tr>
<tr>
<td>$1.5 &lt; T$</td>
<td>$k_{h0}=0.393 T^{2/3}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zoning</th>
<th>Correction factor $c_Z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0</td>
</tr>
<tr>
<td>B</td>
<td>0.85</td>
</tr>
<tr>
<td>C</td>
<td>0.7</td>
</tr>
</tbody>
</table>

**Figure 2.2.** Earthquake Zones
2) For Level 2 earthquake ground motion

In verification of Level 2 Earthquake Ground Motion by the static analysis, the plastic behavior is considered since the target seismic performance levels are Level 2 and Level 3 depending on the class of the bridge. The dynamic inelastic response of the bridge is estimated with the equal-energy assumption of one-degree-of-freedom system, and ductility or strength is taken into account within plastic ranges of the members by reducing the static inertia force applied to the structure.

The design horizontal seismic coefficient to be used for this method is calculated by Equation (2.2). For Ground Motion Type I, when the product of the standard value of the design horizontal seismic coefficient \( k_{hc0} \) and modification factor for zones \( c_Z \) is less than 0.3, design horizontal seismic coefficient shall be obtained by multiplying the force reduction factor \( c_S \) by 0.3. In addition, when the design horizontal seismic coefficient is less than 0.4 times the modification factor for zones \( c_Z \), the design horizontal seismic coefficient shall be equal to 0.4 times \( c_Z \).

\[
k_{hc} = c_S c_Z k_{hc0}
\]

(2.2)

where

\( k_{hc} \): Design horizontal seismic coefficient for Level 2 Earthquake Ground Motion.
\( k_{hc0} \): Standard value of the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion shown in Table 2.4.
\( c_S \): Force reduction factor as in equation (2.3)
\( c_Z \): Modification coefficient for zone.

For a structural system that can be modeled as a one degree-of-freedom vibration system having a plastic force-displacement relation, force reduction factor is calculated as Equation (2.3) based on the equal-energy assumption.

\[
c_S = \frac{1}{\sqrt{2\mu_a} - 1}
\]

(2.3)

where

\( \mu_a \): Allowable ductility factor for the structural system having a plastic force displacement relation.
Table 2.4. Standard Values of the Design Horizontal Seismic Coefficient for Level 2 Earthquake Ground Motion, $k_{hc0}$

(a) Type I Ground Motions

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>$k_{hc0}$ value for natural period $T$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I (stiff)</td>
<td>$k_{hc0}=0.7$ For $T \leq 1.4$</td>
</tr>
<tr>
<td>Type II (Moderate)</td>
<td>$k_{hc0}=1.51T^{4/3}$ For $T &lt; 0.18$</td>
</tr>
<tr>
<td>Type III (soft)</td>
<td>$k_{hc0}=1.51T^{4/3}$ For $T &lt; 0.29$</td>
</tr>
</tbody>
</table>

(b) Type II Ground Motions

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>$k_{hc0}$ value for natural period $T$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I (stiff)</td>
<td>$k_{hc0}=4.46T^{2/3}$ For $T \leq 0.3$</td>
</tr>
<tr>
<td>Type II (Moderate)</td>
<td>$k_{hc0}=3.22T^{2/3}$ For $T &lt; 0.4$</td>
</tr>
<tr>
<td>Type III (soft)</td>
<td>$K_{hc0}=2.38T^{2/3}$ For $T &lt; 0.5$</td>
</tr>
</tbody>
</table>

2.2.2 Verification methods of seismic performance based on dynamic analysis

In verification of seismic performance for bridges with complicated seismic behavior, a dynamic analysis shall be applied to obtain the seismic response. “Bridges with complicated seismic behavior” indicates bridges that the application of the static analysis is limited because of the reasons given below.

i) In case that vibration modes primarily affecting responses of the bridge defer considerably from ones assumed by the static analysis method.

ii) There are more than 2 types of vibration modes contributing to responses of the bridge.

iii) In verification of seismic performance for Level 2 Earthquake Ground Motion, plural plastic hinges are expected or locations of plastic hinges cannot be specified due to complicated structure.

iv) In case the application of equal-energy assumption is not clear for the verification of seismic performance for Level 2 Earthquake Ground Motion.

Depending on the above issues, bridges that should be verified with the dynamic analysis method are as follows.
1) Bridges with longer natural periods (generally more than 1.5s), or bridges with higher piers (generally more than 30m)
2) Bridges of horizontal force distributed structure with rubber bearings
3) Seismically-isolated bridges
4) Rigid-frame bridges
5) Bridges with steel piers in which plasticity are allowed
6) Bridges with cables such as cable-stayed bridges or suspension bridges
7) Upper-deck type or half-through type arch bridges
8) Curved bridges with a large angle between ends of superstructure at a small curvature.

During the verification of seismic performance by dynamic method, the maximum response values such as sectional force and displacement occurred in each structural member, which are obtained from dynamic response analysis results, shall be kept below the allowable values. The methods of dynamic response analysis include response spectrum method and time-history response analysis method. The verification of seismic performance for each level of ground motion should be conducted by using the average seismic response for at least three input ground motions.

The ground motions used in the dynamic response analysis are spectral fitted to the following response spectra for Level 1 and Level 2 ground motions, respectively;

\[
S = c_Z . c_D . S_0
\]
\[
S_I = c_Z . c_D . S_{I0}
\]
\[
S_{II} = c_Z . c_D . S_{II0}
\]

where

\( S \): Acceleration response spectra for Level 1 Earthquake Ground Motion
\( S_I \): Acceleration response spectra for Level 2 Type I Earthquake Ground Motion.
\( S_{II} \): Acceleration response spectra for Level 2 Type II Earthquake Ground Motion.
\( c_Z \): Modification factor for zones.
\( c_D \): Modification factor for damping ratio. It is calculated by Equation (2.7) in accordance with the damping ratio \( h \).
\( S_0 \): Standard acceleration response spectra (cm/sec^2) for Level 1 Earthquake Ground Motion given in Table 2.5 in accordance with fundamental period \( T \).
\( S_{i0} \): Standard acceleration response spectra (\( \text{cm/sec}^2 \)) for Level 2 Type I Earthquake Ground Motion given in **Table 2.6(a)** in accordance with fundamental period \( T \).

\( S_{ii0} \): Standard acceleration response spectra (\( \text{cm/sec}^2 \)) for Level 2 Type II Earthquake Ground Motion given in **Table 2.6(b)** in accordance with fundamental period \( T \).

The standard acceleration spectra are given for damping ratio \( h=0.05 \). When the considered modal damping ratio \( h_i \) of the structure is different from this value, the spectra is modified by \( c_D \) computed as:

\[
 c_D = \frac{1.5}{40h_i} + 0.5
\]  

(2.7)

The standard response spectra for the Level 2 ground motions are illustrated in **Figure 2.3** for the ground condition I and ground condition II.

**Table 2.5.** Standard Acceleration Response Spectra for Level 1 Earthquake Ground Motion \( (S_0) \)

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>Response Acceleration ( S_{i0} ) (( \text{cm/sec}^2 ))</th>
</tr>
</thead>
</table>
| **Group 1** (stiff) | \[
\begin{align*}
T<0.1 & \quad S_0=431T^{1/3} \\
& \quad \text{But } S_0 \geq 160 \\
0.1 \leq T \leq 1.1 & \quad S_0=200 \\
1.1<T & \quad S_0=220/T \\
\end{align*}
\] |
| **Group II** (Moderate) | \[
\begin{align*}
T<0.2 & \quad S_0=427T^{1/3} \\
& \quad \text{But } S_0 \geq 200 \\
0.2 \leq T \leq 1.3 & \quad S_0=250 \\
1.3<T & \quad S_0=325/T \\
\end{align*}
\] |
| **Group III** (soft) | \[
\begin{align*}
T<0.34 & \quad S_0=430T^{1/3} \\
& \quad \text{But } S_0 \geq 240 \\
0.34 \leq T \leq 1.5 & \quad S_0=300 \\
1.5<T & \quad S_0=450/T \\
\end{align*}
\] |
Table 2.6. Standard Acceleration Response Spectra for Level 2 Earthquake Ground Motion

(a) Type I Ground Motion

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>Response Acceleration $S_{i0}$ (cm/sec²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 (stiff)</td>
<td>$S_{i0}=700$ For $T_i \leq 1.4$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=980/T_i$ For $T_i &gt; 1.4$</td>
</tr>
<tr>
<td>Type II (Moderate)</td>
<td>$S_{i0}=1505T_i^{1/3}$ For $T_i &lt; 0.18$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=1511T_i^{1/3}$ For $T_i &lt; 0.29$</td>
</tr>
<tr>
<td>Type III (soft)</td>
<td>$S_{i0}=1000$ For $0.29 \leq T_i \leq 2.0$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=2000/T_i$ For $T_i &gt; 2.0$</td>
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</tbody>
</table>

(b) Type II Ground Motions

<table>
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<tr>
<th>Ground Condition</th>
<th>Response Acceleration $S_{i0}$ (cm/sec²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 (stiff)</td>
<td>$S_{i0}=4463T_i^{2/3}$ For $T_i \leq 0.3$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=2000$ For $0.3 \leq T_i \leq 0.7$</td>
</tr>
<tr>
<td>Type II (Moderate)</td>
<td>$S_{i0}=3224 T_i^{2/3}$ For $T_i &lt; 0.4$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=1750$ For $0.4 \leq T_i \leq 1.2$</td>
</tr>
<tr>
<td>Type III (soft)</td>
<td>$S_{i0}=2381T_i^{2/3}$ For $T_i &lt; 0.5$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=1500$ For $0.5 \leq T_i \leq 1.5$</td>
</tr>
<tr>
<td></td>
<td>$S_{i0}=2948/T_i^{5/3}$ For $T_i &gt; 1.5$</td>
</tr>
</tbody>
</table>

Figure 2.3. Standard acceleration response spectra for Level 2 ground motions $h=0.05$
CHAPTER 3.

PREDICTION OF MAXIMUM OUT-OF-PLANE RESPONSE [74-76]
3.1 Introduction

The generation procedure of the static analysis-based method for the prediction of maximum out-of-plane response is discussed in this chapter.

First, numerical analyses are carried out on the parametric upper-deck steel arch bridge models to examine the applicability of equal-energy assumption. Examinations are conducted by comparing estimation results obtained by the equal-energy assumption with the results of dynamic response analysis. Conservative estimates with low accuracy are obtained as a result.

Next, the factors influencing on the estimation accuracy of equal-energy assumption is discussed. Some solid tendencies are found that make it possible to improve the estimation accuracy by developing certain correction functions.

Finally, the correction functions are combined with the response spectrum method and pushover analysis to establish a method to predict the maximum inelastic seismic response without the need of dynamic response analysis.
3.2 Applicability of Equal-Energy Assumption

3.2.1 Analyzed models

The applicability of the equal-energy assumption is examined numerically by studying six upper-deck steel arch bridge models. The models differ in their arch rise to span ratio and arch rib spacing, as shown in Table 3.1. These two structural parameters are given variations over a wide coherent range in order to obtain a pattern representing the behavior of general upper-deck steel arch bridges and also to examine their influence on the applicability of the equal-energy assumption.

Table 3.1. Structural parameters of the analyzed models

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Span Length (m)</th>
<th>Arch Rise (m)</th>
<th>Arch Rise Span Length</th>
<th>Arch Rib Spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>114</td>
<td>16.87</td>
<td>0.15</td>
<td>6.0</td>
</tr>
<tr>
<td>Model 2</td>
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<td>6.0</td>
</tr>
<tr>
<td>Model 3</td>
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<td>6.0</td>
</tr>
<tr>
<td>Model 4</td>
<td>114</td>
<td>45.60</td>
<td>0.40</td>
<td>6.0</td>
</tr>
<tr>
<td>Model 5</td>
<td>114</td>
<td>16.87</td>
<td>0.15</td>
<td>9.5</td>
</tr>
<tr>
<td>Model 6</td>
<td>114</td>
<td>16.87</td>
<td>0.15</td>
<td>13.0</td>
</tr>
</tbody>
</table>

Model 1 shown in Figure 3.1 is used as the template from which the other five parametric models are generated (See appendix for the Models 2-6). This bridge was adopted by the JSSC committee as a representative model for investigations of non-linear behavior during major earthquakes [59]. The parametric models are generated by using the JSP-15 W preliminary design software for steel arch bridges [60]. Models 2-4 are generated from Model 1 by changing only the arch rise. Models 5 and 6 are generated from Model 1 by changing only the spacing between the two arch ribs. The generation process is carried out carefully, in order to ensure that the newly generated models remain within realistic limits. The selected arch rise to span ratios can be found in existing steel arch bridges. The template Model 1 and newly generated Models 2-4 carry two-lane traffic. The distance between the arch ribs is widened in order to carry a three-lane deck in Model 5, and a four-lane deck in Model 6. In this way, realistic steel arch bridge models are generated for numerical analysis. Models 1, 2, 3 and 4 constitute a pattern demonstrating the effect of arch rise to span ratio, whereas Models 1, 5 and 6 are a series demonstrating the effect of arch rib spacing on the applicability of the equal-energy assumption.
Figure 3.1 also gives the cross sections of the main structural elements of the template model. A box-type section is used for the arch rib and side column, whereas an I-section is adopted for the stiffening girder. The figure shows the cross section of the arch rib near the support and that of the stiffening girder in the span center. The side columns have a uniform box section. The other five generated models have cross sections of the same shape based on dimensions given by the preliminary design software.
The bridges are modeled and analyzed using the general purpose MARC non-linear finite element (FE) analysis software [61]. Three-dimensional beam elements of type 14 and 79, as provided in the MARC element library, are employed to model the structural members. Element 14 is adopted for the box sections. This is a closed-section straight-beam element with no warping of the section but including twist based on Euler-Bernoulli beam theory. There are two nodes per element. The degrees of freedom associated with each node are three global displacements and three global rotations. Element 79 is used for the I-shaped sections. It is an open-section straight-beam element that includes warping and twisting of the section. It is composed of two nodes with seven associated degrees of freedom, three for global displacements, three for global rotations and one for warping of the section.

Material non-linearity is taken into account by 3D fiber modeling. For the box sections of the arch ribs, 26 integration points are specified. There are 24 integration points for the side columns and 25 for the I-shaped sections. Geometrical non-linearity is also taken into account in the FE analysis. The updated Lagrangian Formulation is employed to consider the large displacement effect.

The boundary conditions and connection types of the bridge models are shown in Figure 3.2. Typical boundary conditions are used for all of the models. As for the abutments, roller bearings are assumed in the longitudinal direction. The side pier ends consist of pivot-type bearings and the arch rib ends are pinned bearings. All connections between the members are rigid.
A lumped mass approach is used to model the mass of the bridges. The masses of the stiffening girder, arch ribs and piers are lumped at their nodal points. Further, the masses of the transverse and diagonal members are also considered; these are lumped on the nodal points of the corresponding stiffening girder, arch rib or vertical member. No rotational inertia is associated with the nodal points.

The reinforced concrete bridge deck is not modeled, but its mass is considered and lumped at the nodal points of the stiffening girder. Simpler models in which the deck is not modeled can be used in studying the applicability of the equal-energy assumption because the individual effect of the reinforced concrete deck to the estimation accuracy is thought to be negligible; since the same no-deck model is used for both the estimation procedure and the time history analysis. As will be explained later in Section 3.2.2, the accuracy of this assumption is assessed by comparing the estimated inelastic response
using the equal-energy assumption with the calculated response obtained from inelastic dynamic response analysis.

A single type of steel, JIS-SMA490, is adopted for all of the bridge models (yield stress \( \sigma_y = 355 \) MPa; Young’s modulus, \( E = 206 \) GPa; Poisson’s ratio, \( \nu = 0.3 \)). A bilinear stress-strain relation with a strain hardening slope \( E’ = E/100 \) and a kinematic hardening rule is assumed, as seen in Figure 3.3.

![Figure 3.3. Material model for steel](image)

The natural frequencies, modal participations and mode definitions of the first 10 modes of all the analyzed models are listed in Table 3.2. Since dynamic response in the out-of-plane direction is the concern of this study, only the eigenmodes of the transverse direction are evaluated. The first and third out-of-plane modes make the greatest contribution as they have the largest effective mass ratios. These are selected as the predominant modes and their shapes are illustrated in Figure 3.4. This shows that they are symmetric out-of-plane modes and they exhibit similar shapes for the different models, despite differences in arch rise and deck width. Of these two modes, the contribution of the first one is much greater. When the effective mass ratios of this mode for Models 1, 2, 3 and 4 (which differ only in arch rise) are compared, it can be seen that the contribution increases as the arch rise to span ratio increases. The ratio is about 60% for Model 1, increasing to about 72% for Model 4. This suggests that a bridge will have a greater tendency to vibrate mainly in the first out-of-plane mode as the arch rise to span ratio increases. It can also be seen, by comparing the effective mass ratios of Model 1, 5 and 6, that the arch rib spacing does not significantly affect the modal contribution of the predominant modes.
Table 3.2. Eigenvalue analysis results

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode</th>
<th>Natural Frequency (Hz)</th>
<th>Effective Mass Ratio</th>
<th>Deflection Mode</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
<td></td>
<td>Longitudinal (%)</td>
<td>Transverse (%)</td>
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Table 3.2 (Continued)

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<th>Deflection Mode</th>
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</tr>
<tr>
<td>Model</td>
<td>1st out-of-plane mode</td>
<td>3rd out-of-plane mode</td>
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<td></td>
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<tr>
<td>--------</td>
<td>-----------------------</td>
<td>-----------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model 1</td>
<td><img src="image1" alt="Mode 1" /> $f=1.041$ Hz</td>
<td><img src="image2" alt="Mode 3" /> $f=2.590$ Hz</td>
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<td></td>
</tr>
<tr>
<td>Model 2</td>
<td><img src="image3" alt="Mode 2" /> $f=0.995$ Hz</td>
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<td></td>
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</tr>
<tr>
<td>Model 3</td>
<td><img src="image5" alt="Mode 5" /> $f=0.824$ Hz</td>
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</tr>
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<td></td>
<td></td>
</tr>
<tr>
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<td></td>
</tr>
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<td>Model 6</td>
<td><img src="image11" alt="Mode 11" /> $f=1.363$ Hz</td>
<td><img src="image12" alt="Mode 12" /> $f=2.323$ Hz</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 3.4.** Predominant eigenmodes
3.2.2 Examination procedure

The accuracy of estimations made by the equal-energy assumption is assessed by comparing the estimated inelastic maximum response with the value obtained using non-linear dynamic response analysis. The examination procedure is described below.

1) Free vibration analysis is carried out to obtain the principle natural frequencies and mode shapes.

2) Elasto-plastic finite displacement pushover analysis is performed in order to obtain the curve for the relationship between total out-of-plane base shear force and displacement for each model.

3) The maximum elastic response displacement is obtained by performing elastic dynamic response analysis. The corresponding total base shear force, which is the total out-of-plane reaction force summed over all supports at the maximum response displacement, is also calculated. Using these two values, the maximum strain energy stored in the structure is computed.

4) The maximum inelastic response displacement $\delta_{SP}$ is estimated by applying the equal-energy assumption to the curve of total out-of-plane base shear force versus displacement, as obtained in 2) above, and the maximum strain energy, obtained in 3) (See Figure 3.5).

5) Inelastic finite displacement dynamic response analysis is used to obtain the maximum inelastic response displacement $\delta_{DP}$.

6) The estimated maximum response displacement ($\delta_{SP}$) and the calculated value ($\delta_{DP}$) are compared in order to evaluate the accuracy of the assumption.

![Equal Energy Assumption](image)

**Figure 3.5.** Equal-energy assumption
For the pushover analysis, the modal force distribution from the single dominant mode in the transverse direction (the first symmetric out-of-plane mode) is adopted as the lateral load distribution pattern, expressed as:

\[
\{H_i\} = \{m_i\phi_i\} \tag{3.1}
\]

in which \(m_i\) is concentrated mass and \(\phi_i\) is the transverse component of the eigenvector \(\{\phi\}\) at node \(i\).

The modal force distribution is used here as it serves as a sufficiently accurate pattern in pushover analysis to approximate the inertia force distribution during earthquake excitations as shown by Lu et al. [20, 21]. In these references, the inertia force distribution, which is the lateral inertia force distribution at the moment of maximum displacement demand in an elastic Level 2 dynamic response analysis, is evaluated as an alternative lateral load distribution pattern, expressed as:

\[
\{H_i\} = \{-m_i a_i\} \tag{3.2}
\]

in which \(H_i\), \(m_i\) and \(a_i\) are the lateral force, concentrated mass and lateral acceleration at node \(i\), respectively. Although it is believed that this load distribution pattern represents the actual inertia force distribution under earthquake excitations well, the dynamic response analysis requirement that the acceleration at each node be obtained makes this option unsuitable in this study, remembering that the main goal is to eliminate dynamic response analysis from seismic design.

The mid point of the stiffening girder is adopted as the reference point for pushover analysis since the maximum transverse displacements for all models are observed at this point. Through the pushover analysis, the displacement of this reference point is plotted against total base shear force in the out-of-plane direction for all supports to obtain the inelastic behavior of the structure.

Six Level 2, Type II ground motions are used for the dynamic response analysis: three for ground condition I (stiff ground) and three for ground condition II (moderate ground). Their titles and maximum accelerations are summarized in Table 3.3. These ground motions are the standard ground motions specified in the JRA code [3, 4] for use in the seismic design of highway bridges in Japan. They were generated by modifying the near-fault strong ground motions recorded at various locations during the Hyogo-ken Nanbu earthquake to fit the specified response spectrum for Level 2 ground
motions (See Figure 2.3). The response spectrum specified for the damping ratio \( \xi = 0.05 \) can be modified for other damping ratios by multiplying the original spectrum by modification factor for damping constant \( c_D \). (See Equation 2.7)

**Table 3.3. Input ground motions for the dynamic response analysis**

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>Name</th>
<th>Duration (sec)</th>
<th>Maximum Acceleration (cm/sec^2)</th>
<th>Amplification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground I (Stiff)</td>
<td>1995 JMA Kobe OBS N-S (Le2.t211)</td>
<td>30</td>
<td>812</td>
<td>1.2, 1.5, 1.7, 2, 5</td>
</tr>
<tr>
<td></td>
<td>1995 JMA Kobe OBS E-W (Le2.t212)</td>
<td>30</td>
<td>766</td>
<td>1.5, 2, 5</td>
</tr>
<tr>
<td></td>
<td>1995 HEPC Inagawa N-S (Le2.t213)</td>
<td>30</td>
<td>780</td>
<td>1.5, 2, 5</td>
</tr>
<tr>
<td>Ground II (Moderate)</td>
<td>1995 JR Takatori Sta. N-S (Le2.t221)</td>
<td>40</td>
<td>687</td>
<td>1.5, 2</td>
</tr>
<tr>
<td></td>
<td>1995 JR Takatori Sta. E-W (Le2.t222)</td>
<td>40</td>
<td>673</td>
<td>1.5, 2</td>
</tr>
<tr>
<td></td>
<td>1995 OGAS Fukiai N27W (Le2.t223)</td>
<td>40</td>
<td>736</td>
<td>1.5, 2</td>
</tr>
</tbody>
</table>

The above-mentioned input ground motions are applied to the structure in the out-of-plane direction. Additionally, they are amplified by the coefficients shown in Table 3.3. to obtain sufficiently inelastic response. By this method, a pattern reflecting the effects of increasing ground motion intensity can be studied, for the evaluation of the applicability of the equal-energy assumption. These ground motions are applied to the supports of the structure uniformly in the same phase, although there are some studies pointing out that out-of-phase ground motion inputs may influence the response of arch bridges [62, 63]. The JRA code does not require consideration of the influence of out-of-phase ground excitations in the seismic design of bridges shorter than 200 m in span.

Newmark's \( \beta \) method [64] is employed to solve the equation of motion for both elastic and inelastic dynamic response analyses. The \( \beta \) value is taken as 1/4. Rayleigh damping [64] is assumed for all of the models by considering only the predominant eigenmodes and assuming modal damping ratios of 0.03.

The whole examination procedure is carried out for the reference point mentioned before. The equilibrium energy equation is drawn for this point on the curve of total out-of-plane base shear force versus displacement obtained in the pushover analysis in order to estimate the maximum inelastic response. The reference point is simply used to
obtain a control value of displacement, from which the deformed shape of the whole structure can be predicted. The total absorbed energy of the structure is already taken into account by employing a modal force distribution pattern as a substitute for the total dynamic response.

### 3.2.3 Validity of using pushover analysis to represent dynamic behavior

It is necessary to verify that dynamic behavior is sufficiently well represented by pushover analysis in which a modal force distribution is used as a lateral loading pattern. For this purpose, the displacement distribution obtained by pushover analysis is compared with that obtained from the non-linear dynamic response. This comparison is carried out for each model using the most severe dynamic excitation. The displacement distribution obtained in the dynamic response analysis at the time increment representing the maximum response at the reference point is compared with the distribution given by pushover analysis at the static force increment corresponding to the same reference point displacement. The comparisons are given in Figure 3.6 for the stiffening girder and the arch rib of each model, respectively. These comparisons demonstrate that the displacement distributions match each other quite closely (although there are some differences in the case of Model 3). It can be concluded that pushover analysis carried out using a modal force distribution based on the first out-of-plane vibration mode with an effective mass ratio of more than 60% suitably accounts for dynamic behavior, matching the findings of Lu et al. [21].
Figure 3.6. Displacement distributions for pushover and dynamic response analysis (Model 1,2,3)
Figure 3.6 (Continued). Displacement distributions for pushover and dynamic response analysis (Model 4, 5 and 6)
3.2.4 Accuracy of estimation and influencing parameters

The ratio of estimated maximum inelastic response ($\delta_{SP}$) to the actual dynamic response calculated by inelastic dynamic response analysis ($\delta_{DP}$) is used as an index of estimation accuracy. The natural frequency and the structural parameters such as Arch Rise/Span Length ratio and the spacing between the arch ribs can be considered to have an influence on the applicability of the equal-energy assumption. The relationship between these parameters and $\delta_{SP}/\delta_{DP}$ is examined.

**Figure 3.7** illustrates the relationship between $\delta_{SP}/\delta_{DP}$ and 1st symmetric side sway mode frequency for the ground motions of ground condition I and ground condition II. Any correlation between $\delta_{SP}/\delta_{DP}$ and natural frequencies can not be found, suggesting that the natural frequency of the structure has no apparent effect on the accuracy of the estimation. But it can be seen in the figure that all values of $\delta_{SP}/\delta_{DP}$ are greater than 1.0. This means that the equal-energy assumption results in safe side estimation. However, in many cases, the estimated results are much larger than the responses calculated by inelastic dynamic response analysis; that is, the accuracy of the estimation is quite low.

![Figure 3.7](image)

**Figure 3.7.** $\delta_{SP}/\delta_{DP}$—Natural frequency relationship

The influence of the considered structural parameters are studied by evaluating the relationship between this estimation accuracy index ($\delta_{SP}/\delta_{DP}$) and estimated ductility factor ($\mu_E$). The estimated ductility factor is expressed as

$$
\mu_E = \frac{\delta_{SP}}{\delta_y}
$$

(3.3)
in which $\delta_{SP}$ is the estimated maximum non-linear response and $\delta_y$ is the yield displacement obtained by pushover analysis. The ductility ratio $\mu$ ($\mu=\delta_{DP}/\delta_y$) can be used instead of $\mu_E$ to evaluate applicability once inelastic dynamic response analysis has
been carried out. However, such an approach would not appropriate for this study, whose objective is to find an alternative to dynamic response analysis for the prediction of inelastic maximum response.

The $\delta_{SP}/\delta_{DP}-\mu_E$ relationships for the different models are illustrated together for various input ground motions in Figure 3.8. The results on the right represent estimation accuracy for more ground motions with greater magnification. In these cases, the ductility factors may appear larger than is practical for any actual design procedure. But it should be noted that $\mu_E$ is not the same as the ductility ratio, $\mu$, obtained through dynamic response analysis. The estimated ductility factor $\mu_E$ includes estimation errors that may be more than 300% in some cases. In the case of Models 5 and 6, however, the ductility ratios are also too large for the ground condition 1 motions amplified by a factor of 5, especially with the Le2.t211 excitation. The values of ductility ratio, $\mu$, range from 5 to 6, which are impractical for the design procedure. These values can simply be excluded from consideration.

Observation of the $\delta_{SP}/\delta_{DP}-\mu_E$ relationships shown in Figure 3.8 clearly shows that, for all input ground motions, they exhibit a similar tendency to reduced estimation accuracy as the estimated ductility factor $\mu_E$ increases. The trend is almost the same for all models, despite their different structural parameters. This suggests that the structural parameters considered in this study, which are the ratio of arch rise to span and the arch rib spacing, have no significant influence on the applicability of the assumption.
Figure 3.8. $\delta_{SP}/\delta_{DP}$-μE relationships for individual ground motions
The JRA code recommends using at least three ground motions per dynamic analysis, taking an average of them to evaluate response for seismic design. This means it is necessary to calculate the average of the estimated responses for three ground motions for each ground condition, respectively. This also gives a better understanding of the influence of the structural parameters considered in this research. In Figure 3.9 the relationship between $\delta_{SP}/\delta_{DP}$ and $\mu_E$ for the average estimated response displacements is shown for both ground conditions. It is clear that there is no significant difference in estimation accuracy among the different models and that the relationship can be roughly represented by a linear function. Further, the overall tendency is similar for both ground conditions, suggesting that estimation accuracy is not significantly influenced by the structural parameters and input ground conditions.

![Figure 3.9. $\delta_{SP}/\delta_{DP}$-$\mu_E$ relationships for average response displacements](image)

**3.2.5 Approximation of relationship between $\delta_{SP}/\delta_{DP}$ and $\mu_E$**

Having shown that estimation accuracy does not depend on the model or the type of ground condition, it is possible to approximate the relationship between $\delta_{SP}/\delta_{DP}$ and $\mu_E$ using a single function that represents the overall trend for different ground motions and structural parameters. This approximation is achieved by taking into account only the average response displacement results, as recommended in the JRA code. The average and lower bound values of $\delta_{SP}/\delta_{DP}$ are marked in Figure 3.10 by lines. The average approximation is the optimum line through the $\delta_{SP}/\delta_{DP}$ values as calculated by the least squares method. On the other hand, the lower bound approximation is the
bottom boundary line of the $\delta_{SP}/\delta_{DP} - \mu_E$ relationship. By the help of these lines it is possible to predict the estimation accuracy for any given $\mu_E$ value.

![Graph](image)

**Figure 3.10.** Approximation of $\delta_{SP}/\delta_{DP} - \mu_E$ relationship

### 3.2.6 Correction functions for equal-energy assumption

Estimation accuracy can be improved by modifying the approximations in the relationship between $\delta_{SP}/\delta_{DP}$ and $\mu_E$. By this method, a correction function $f(\mu_E)$ is proposed for both the average estimation and the lower bound estimation. The average estimation correction function (3.4), which is obtained by modifying the average approximation, is proposed to give the optimum estimation results, whereas the lower bound estimation correction function (3.5) is obtained from the lower bound approximation and guarantees that the estimated maximum inelastic response is always equal to or greater than the actual inelastic response ($\delta_{DP}$). Estimation results are corrected by simply multiplying the estimated maximum inelastic response by the correction function of the desired type, as shown in equation (3.6).

**Average Estimation**

$$f(\mu_E) = \frac{1}{(0.1843\mu_E + 0.8159)}, \quad (0 < f(\mu_E) \leq 1)$$

**Lower Bound Estimation**

$$f(\mu_E) = \frac{1}{(0.1700\mu_E + 0.7050)}, \quad (0 < f(\mu_E) \leq 1)$$

$$\delta_{SP} = f(\mu_E) \times \delta_{SP}$$

$$\delta'_{SP} = f(\mu_E) \times \delta_{SP}$$
where, $f(\mu_E)$: correction function; $\mu_E$: estimated ductility factor; $\delta_{SP}$: corrected value of estimated maximum response.

A correction function should be used if its calculated value for a given $\mu_E$ is less than or equal to 1. Otherwise no correction is needed and the estimated value can be used as it is. This is generally encountered in the very small values of $\mu_E$ or when the response is completely elastic.

Corrected values of the estimated ductility factor ($\mu' = \mu_E f(\mu_E)$), as calculated from the average response displacements for both ground conditions, are plotted in Figure 3.11 against the ductility ratio ($\mu$), together with the values without correction. It can be seen that estimation accuracy is significantly improved as the corrected ductility factor becomes closer to the actual ductility ratio. The established correction functions are also applied to the results for individual ground motions, as shown in Figure 3.12. Although the correction functions are generated only by considering the average response displacements, it can be seen that estimation accuracy is also improved for the individual input ground motions. The lower bound estimation is not plotted since it is meaningful only for design procedure in which the average of three ground motion response displacements should be taken.

![Figure 3.11. Correction results for the average response displacements](image-url)
3.3 Proposed Prediction Method

3.3.1 Prediction procedure

In the preceding section, the applicability of the equal-energy assumption when used with the proposed correction functions is verified for the estimated maximum response. During the application procedure, the maximum elastic response, which is necessary for prediction of the maximum non-linear response, is obtained by performing linear dynamic response analysis in order to achieve the most accurate estimate possible on the basis of the assumption. However, it is desirable to carry out seismic design without dynamic response analysis for reasons of calculation time and cost. This is possible if the elastic response is estimated from the response spectrum.

The basic steps in the proposed method for predicting the maximum inelastic response are given in Figure 3.13 and explained below.

Step 1. Establish a FE model for the upper-deck steel arch bridge under investigation.

Step 2. Perform eigenvalue analysis to acquire the predominant free vibration mode. Based on this, calculate the force distribution to be used as the lateral force pattern in pushover analysis.

Step 3. Obtain the relationship between total out-of-plane base shear force and displacement as well as the yield displacement $\delta_y$ by performing pushover analysis using the modal force distribution obtained in Step 2.

Figure 3.12. Correction results for the individual ground motions
Step 4. Obtain the maximum response from the response spectrum specified in the JRA code for Level 2 ground motion depending on the corresponding ground condition and modal damping ratio. Calculate the corresponding elastic strain energy.

Step 5. Estimate the maximum inelastic response displacement $\delta_{SP}$ by applying the equal-energy assumption and calculate the estimated ductility factor $\mu_E$.

Step 6. Calculate the value of correction function $f(\mu_E)$ either for the average estimation or the lower bound estimation. If $f(\mu_E)$ is less than 1, multiply $\delta_{SP}$ by $f(\mu_E)$ to get the final value of maximum inelastic response. If $f(\mu_E)$ is greater than 1, no correction is necessary and $\delta_{SP}$ can be used directly.
3.3.2 Validity of the method

In order to illustrate the accuracy of the proposed method, the estimated maximum non-linear response $\delta_{SP}$ it yields is compared with the actual maximum dynamic response $\delta_{DP}$ calculated directly by non-linear dynamic response analysis. This comparison is shown for the average estimation in Figure 3.14 for ground conditions I and II, respectively. The estimation error range is around ±20% for the individual
ground motions and ±15% for the average response displacements. The lower bound estimation is studied only for average response displacements, as indicated before, and the error in this case is found to be less than 20% as shown in Figure 3.15. Judging from these figures, it is considered that the proposed method is applicable to the preliminary design of upper-deck steel arch bridges as a simple way of predicting their maximum response.
Figure 3.14. Average estimation results with the proposed method.
For further confirmation of its validity, the proposed method is applied to the same models using a different set of ground motions. These are ground motions not considered during the development of the correction functions. Type I ground motions for ground conditions I and II, amplified by factors of 1.5, 2 and 5, are employed as the input ground motions in this examination. The estimation obtained, $\delta_{SP'}$, are compared
with the actual dynamic response, $\delta_{DP}$, in Figure 3.16. The results for average response displacements are within the error range of $\pm 20\%$. It can be also seen that fairly good estimation results are obtained for individual ground motions. These findings verify the proposed method for Type I ground motions in addition to Type II. However, it should be noted that the lower bound estimation results, which are supposed to fall on the safe side, are given as less than the actual response in a few cases.

![Figure 3.16. Estimation accuracy for the Type I ground motions](image)

### 3.4 Summary

Static pushover analysis, linear and non-linear dynamic response analysis of six upper-deck steel arch bridges were carried out. On the basis of the results, the applicability of the equal-energy assumption for out-of-plane response of the structures was examined, and correction functions were developed to improve the estimation accuracy of the maximum response displacement. Based on these correction functions and the response spectrum, a prediction method for maximum inelastic out-of-plane seismic response of upper-deck steel arch bridges was proposed that does not rely on dynamic response analysis. The validity of the proposed method was evaluated through numerical examples. The main findings of this research are summarized below:
1) The predicted maximum inelastic response displacement based on the equal-energy assumption is conservative for upper-deck steel arch bridges. In many cases, though, the results may be too conservative.

2) The ground condition type and the structural parameters considered in this investigation (ratio of arch rise to span and arch rib spacing) have no significant influence on the applicability of the equal-energy assumption.

3) The prediction accuracy of the equal-energy assumption can be improved by using the proposed correction functions.

4) The proposed method of predicting maximum inelastic out-of-plane seismic response displacement can be successfully applied to upper-deck steel arch bridges as shown in the numerical examples. It is considered that this method will be useful as a simplified prediction method of maximum inelastic response for the preliminary seismic design of upper-deck steel arch bridges.
CHAPTER 4.

PREDICTION OF MAXIMUM IN-PLANE RESPONSE [77]
4.1 Introduction

Although seismic deficiencies under longitudinal excitations in steel arch bridges are minor [19], a simplified approach is also necessary for the in-plane response that can be an additional tool for the evaluation of the overall seismic performance. For this purpose the applicability of the method proposed in the previous chapter to maximum in-plane response estimation is discussed in this chapter.

First, the load pattern for the pushover analysis that can approximate the inertia force distribution by in-plane ground excitation is investigated. Then the applicability of the equal-energy assumption is examined and influence of the structural parameters on the estimation accuracy is evaluated. A reasonable error range is found which makes the correction functions generated for the out-of-plane response valid also for the in-plane response estimation.

Finally, the application of the method for the maximum in-plane response estimation is validated through the numerical analysis.
4.2 Numerical Analysis

4.2.1 Analyzed models

The numerical analysis are conducted on the six upper-deck steel arch bridge models which are the models originally generated to establish the method for the out-of-plane direction. The influence of the two considered parameters (Arch rise to Span ratio and arch rib spacing) on the applicability of the equal-energy assumption to the maximum in-plane response estimation is examined. The cross sections of the two central vertical members connecting the stiffening girder to the arch rib in Model 2, 5 and 6 are enlarged from the original models in Chapter 3 since yielding occurred in very small response displacements by longitudinal excitations.

The models are again analyzed by using Marc non-linear finite element (FE) analysis software [61].

The natural frequencies and modal participations of predominant eigenmodes in the longitudinal direction are listed in Table 4.1. The first and the third in-plane modes have the greatest contribution to the overall in-plane response as they have larger effective mass ratios. However, it should be noted that the contribution of these two modes is quite small compared to the case of the predominant modes in the out-of-plane direction because of the significant participation coming from the higher modes. The shapes of these two modes are illustrated in Figure 4.1. This shows that they are asymmetric in-plane modes. The contribution of the first in-plane mode is greater for Models 1, 2, 3 and 4 (which differ only in arch rise) and when the effective mass ratios of this mode are compared, it can be seen that the contribution increases as the arch rise to span ratio increases. The ratio is about 20% for Model 1, increasing to about 68% for Model 4. This suggests that a bridge will have greater tendency to vibrate in the first in-plane mode as the arch rise to span ratio increases. It can be also seen, by comparing the effective mass ratios of Model 1, 5, and 6 that the contribution of the third in-plane mode increases as the arch rib spacing increases, even exceeding the contribution of the first in-plane mode for Model 5 and 6. No significant influence can be observed on the contribution of the first in-plane mode when the arch rib spacing alters.
Table 4.1. Principle mode frequencies and contributions

<table>
<thead>
<tr>
<th>Model</th>
<th>1st In-plane Mode Frequency (Hz)</th>
<th>Longitudinal Direction Effective Mass Ratio (%)</th>
<th>3rd In-plane Mode Frequency (Hz)</th>
<th>Longitudinal Direction Effective Mass Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.788</td>
<td>20.03</td>
<td>2.960</td>
<td>6.96</td>
</tr>
<tr>
<td>2</td>
<td>0.751</td>
<td>33.15</td>
<td>2.844</td>
<td>7.92</td>
</tr>
<tr>
<td>3</td>
<td>0.785</td>
<td>55.94</td>
<td>2.690</td>
<td>1.14</td>
</tr>
<tr>
<td>4</td>
<td>0.580</td>
<td>67.68</td>
<td>1.952</td>
<td>2.26</td>
</tr>
<tr>
<td>5</td>
<td>0.822</td>
<td>19.47</td>
<td>2.721</td>
<td>19.97</td>
</tr>
<tr>
<td>6</td>
<td>0.788</td>
<td>19.51</td>
<td>2.575</td>
<td>24.15</td>
</tr>
</tbody>
</table>

1st In-plane mode

3rd In-plane mode

Figure 4.1. Predominant in-plane eigenmodes (Model 1)
4.2.2 Pushover analysis

The applicability of the method to the in-plane response estimation very much depends on selecting a correct load pattern for the pushover analysis that will deform the structure similar to maximum dynamic response. In the out-of-plane direction the load pattern is constituted proportional to the eigenvector of the dominant single mode and the distribution of the concentrated mass. A similar approach is applied also to the in-plane pushover analysis by adopting a modal force distribution from the single dominant mode in the longitudinal direction (1st in-plane mode). Vertical component of this mode is also taken into account as vertical displacement is significant in the longitudinal excitations. However, analysis revealed that the deformed shape of the pushover analysis is significantly different from the displacement distribution of the dynamic response when such a load pattern is employed. The reason for this is basically that the participation of this mode in the overall response is quite small compared to the case of the out-of-plane direction.

Although there are some improvements for pushover analysis proposed for better predictions such as considering more than one mode [30-32], they will result in more complicated procedures. Therefore, an alternative load pattern shown in Figure 4.2 is adopted for the pushover analysis to simulate dynamic response at its ultimate stage due to its simplicity. This is an incremental displacement load ($P_\delta$) applied at the mid point of the stiffening girder from the both sides, as shown in the figure. To check the validity of using this kind of loading pattern the displacement distribution obtained by pushover analysis is compared with that obtained from the dynamic response analysis. The comparison is carried out for each model using a severe dynamic excitation. A Level 2 Type II earthquake ground motion magnified with a factor of 5 is utilized in order to get enough plasticity in the members. The reference point is selected as the node at the 1/4 span on the stiffening girder since the maximum vertical displacement is observed at this node during dynamic response analysis. The displacement distribution obtained in the dynamic response analysis at the time increment representing the maximum value of the vertical displacement at the reference point is compared with the distribution given by pushover analysis at the static force increment corresponding to the same value at the reference point. The comparisons are given in Figure 4.3 for all of the models both for the stiffening girders and the arch ribs. These comparisons demonstrate that the displacement distributions agree well each other although there are some discrepancies
for Model 5. This indicates that the employed load pattern is sufficiently accurate to account for the in-plane dynamic behavior.

**Figure 4.2.** Reference point and the load pattern for the pushover analysis
Figure 4.3. Displacement distributions for pushover and dynamic response analysis (Mode 1, Model 2, Model 3)
Figure 4.3 (Continued). Displacement distributions for pushover and dynamic response analysis (Mode 4, Model 5, Model 6)
4.2.3 Applicability of equal-energy assumption to in-plane response estimation

1) Methodology

Estimation accuracy of the maximum inelastic response by the application of equal-energy assumption is evaluated numerically through the following steps.

1) Elastic dynamic response analyses of the models are conducted to acquire maximum elastic response and maximum strain energy stored in the system.

2) Equal-energy assumption is used on the force-displacement curve obtained by the above mentioned pushover analysis procedure. Inelastic maximum response is estimated (δSP) by equating the strain energy stored in the elastic system to the inelastic one (See Figure 3.5).

3) Dynamic response analysis is conducted to acquire the actual dynamic response (δDP).

4) δSP and δDP are compared for the assessment of the accuracy of the assumption.

It should be noted that the response spectrum method is not utilized for the calculation of the maximum elastic response in order to concentrate on the estimation accuracy of the equal-energy assumption by eliminating the further error that could be induced by adopting the response spectrum method.

For the elastic and inelastic dynamic response analysis six Level 2 Type II ground motions are utilized: three for ground condition I (stiff ground) and three for ground condition II (moderate ground). Their titles and maximum accelerations are summarized in Table 4.2. The ground motions are applied to the structure in the longitudinal direction. Additionally, they are amplified by the coefficients shown in the Table 4.2 to obtain sufficiently inelastic response. By this method, a pattern reflecting the effects of increasing ground motion intensity can be studied, for the evaluation of the estimation accuracy of the equal-energy assumption.
Table 4.2. Input ground motions for the dynamic response analysis

<table>
<thead>
<tr>
<th>Ground Condition</th>
<th>Name</th>
<th>Duration (s)</th>
<th>Maximum acceleration (cm/s²)</th>
<th>Amplification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground I (Stiff)</td>
<td>1995 JMA Kobe OBS N-S (Le2.t211)</td>
<td>30</td>
<td>812</td>
<td>1.5, 2, 5</td>
</tr>
<tr>
<td></td>
<td>1995 JMA Kobe OBS E-W (Le3.t212)</td>
<td>30</td>
<td>766</td>
<td>1.5, 2, 5</td>
</tr>
<tr>
<td></td>
<td>1995 HEPC Inagawa N-S (Le2.t213)</td>
<td>30</td>
<td>780</td>
<td>1.5, 2, 5</td>
</tr>
<tr>
<td>Ground II (Moderate)</td>
<td>1995 JR Takatori Sta. N-S (Le2.t221)</td>
<td>40</td>
<td>687</td>
<td>1.5, 2</td>
</tr>
<tr>
<td></td>
<td>1995 JR Takatori Sta. E-W (Le2.t222)</td>
<td>40</td>
<td>673</td>
<td>1.5, 2</td>
</tr>
<tr>
<td></td>
<td>1995 OGAS Fukiai N27W (Le2.t223)</td>
<td>40</td>
<td>736</td>
<td>1.5, 2</td>
</tr>
</tbody>
</table>

Newmark’s $\beta$ method [64] is employed to solve the equation of motion for both elastic and inelastic dynamic response analyses. The $\beta$ value is taken as 1/4. Rayleigh damping [64] is assumed for all of the models by considering only the predominant eigenmodes and assuming modal damping ratios of 0.03.

The whole examination procedure is carried out for the vertical displacement at the reference point mentioned before. The equilibrium energy equation is drawn on the curve obtained in the pushover analysis procedure which reflects the variation of the vertical displacement of this node with respect to the total in-plane base shear force. The reference point is used to obtain a control value of displacement, from which the deformed shape of the whole structure can be predicted. Seismic demand of a given earthquake ground motion can be obtained for the whole structure through pushover analysis by loading the system until the estimated maximum value of control displacement is reached.

2) Estimation accuracy and influencing parameters

Accuracy of the estimation is evaluated by plotting the accuracy index ($\delta_{SP}/\delta_{DP}$) against the estimated ductility factor ($\mu_{E} = \delta_{SP}/\delta$) as is done for the out-of-plane direction. The $\delta_{SP}/\delta_{DP}-\mu_{E}$ relationships of all models are illustrated for various input ground motions in Figure 4.4. The results on the right side represent estimation accuracy for the ground motions with greater magnification. It can be seen that the $\delta_{SP}/\delta_{DP}$ ratio is greater than 1.0 in all cases which can be interpreted as equal-energy
assumption leads to conservative estimations for the in-plane response. However, estimation accuracy decreases with the increase in ductility factor. The estimated response gets as much as 2 times of the actual response in some cases. When the variation of this error is observed it can be seen that the tendency is similar for all models for the respective ground motions (although the results for model 4 seem to be scattered from the general tendency for the ground condition II ground motions). This suggests that the structural parameters considered in this study, which are the ratio of the arch rise to span and the arch rib spacing, have no significant influence on the applicability of the equal-energy assumption to the in-plane response estimation, corresponding to the finding of the out-of-plane direction.
For a better understanding of the influence of the structural parameters, the average response displacements of the three ground motions for the both ground conditions are calculated. In Figure 4.5 $\delta_{SP}/\delta_{DP}$-$\mu_E$ relationship for the average response displacements are shown for the both ground conditions. The illustrations verify that the estimation accuracy is not significantly influenced by the model type for the both ground conditions.
3) Comparison of the estimation error with that in the out-of-plane direction

The comparison is conducted by plotting the $\delta_{SP}/\delta_{DP} - \mu_E$ relationship for the average response displacement together with the approximated tendency of the corresponding relationship of the out-of-plane direction obtained in the previous chapter. Figure 4.6 illustrates this comparison where the approximated tendency function is given respectively for the average approximation and lower bound approximation. The average approximation is the optimum line through the $\delta_{SP}/\delta_{DP}$ values as calculated by the least-squares method, whereas the lower bound approximation is the bottom boundary line of the $\delta_{SP}/\delta_{DP} - \mu_E$ relationship for the response in the out-of-plane direction. It can be recognized that the functions maintain their role for the in-plane response since the average approximation follows an almost optimum path through the analysis results and the lower bound approximation is situated at the bottom boundary line.
4) Correction of the estimation error

In the out-of-plane direction the estimation error of the equal-energy assumption is corrected through the correction functions generated by simply taking the reciprocal of these approximations as can be seen in the equation (3.4) and (3.5). Since the same approximation functions are verified to be valid also for the in-plane response, it is considered the correction functions, $f(\mu_E)$, can be used also for the correction of the estimation error of the equal-energy assumption for the in-plane response either for the average estimation or the lower bound estimation.

The application of the correction functions to the correction of the estimation result of the maximum in-plane response is demonstrated in Figure 4.7. The corrected values of estimated ductility factor ($\mu' = \mu_E \times f(\mu_E)$) are plotted together with the not corrected ones ($\mu_E$) versus the actual ductility ratio ($\mu = \delta_{DP}/\delta_y$) for the average and individual response displacements in Figure 4.7 (a) and (b) respectively. In the figures it can be seen that the estimation accuracy is significantly improved as the corrected ductility factor becomes closer to the actual ductility ratio. This is more apparent when the ductility ratio ($\mu$) is larger than 2. For the plastic response levels when the ductility ratio ($\mu$) is less than 2, equal-energy assumption yields quite accurate results where almost no correction is needed.
4.3 Application of the method to in-plane response

Having verified the validity of the pushover analysis and equal-energy assumption for the in-plane response it is considered that the previously proposed method can be also applied to the estimation of the maximum inelastic in-plane response through the same steps mentioned before. The validity of the method to the in-plane response is illustrated through the numerical examples by comparing the maximum non-linear response $\delta_{SP}$ estimated by the method with the actual dynamic response $\delta_{DP}$ calculated directly by non-linear dynamic response analysis. This comparison is shown for average estimation in Figure 4.8 for ground condition I and II, respectively. The estimation error
is around ±20% for the individual ground motions and ±15% for the average response displacements. The lower bound estimation is studied only for average response displacements, and the error in this case is found to be less than 20% as shown in Figure 4.9. When these results are compared with the yields of out-of-plane direction presented in the previous chapter, it can be seen that the method results in similar accuracy for the both directions. Within this error range it is considered that proposed method can be used for the preliminary design of upper-deck steel arch bridges as a simple way of predicting the maximum in-plane response as well.
Ground Condition I

Ground Condition II

Figure 4.8. Average estimation results with the proposed method
Figure 4.9. Lower bound estimation results with the proposed method

For further confirmation of its validity, the proposed method is applied to the same models using different set of ground motions. For this purpose, Type I ground motions for ground condition I and II, amplified by factors of 1.5, 2 and 5, are employed as the
input ground motions. The estimation obtained, $\delta'_{SP}$, are compared with the actual dynamic response, $\delta_{DP}$, as shown in Figure 4.10. Fairly good estimation results are obtained for average estimations with the estimation error less than ±20%. Lower bound estimation also leads to an error of less than 20%. However, it should be noted that some of the estimation results are less than the actual results, although the safe side estimation should be achieved with the lower bound estimation.

![Figure 4.10. Estimation accuracy for the Type I ground motions](image)

**4.4 Summary**

The applicability of the previously proposed method for the estimation of the maximum inelastic out-of-plane response of upper-deck steel arch bridges to the maximum in-plane response estimation is examined. Examinations are carried out numerically on six parametric upper-deck steel arch bridge models. The suitable load pattern for the pushover analysis and the estimation error of the equal-energy assumption is evaluated. It is found that the method can be applied to the estimation of maximum in-plane response by only changing the pushover analysis procedure. The main findings are summarized below;

(1) Equal-energy assumption results in conservative estimation of maximum in-plane
inelastic response. The estimation accuracy is quite low when the plasticity is high.

(2) The estimation accuracy of the equal-energy assumption is not influenced from arch rise to span ratio and arch rib spacing.

(3) The correction functions previously proposed to improve the estimation accuracy by the equal-energy assumption for the out-of-plane response is also valid for the in-plane response.

(4) The proposed method can be applied to the maximum inelastic response estimation for in-plane ground motion inputs as well as the out-of-plane ones in the preliminary seismic design considerations.
CHAPTER 5.

ULTIMATE STRAIN OF STEEL PIPE SECTIONS UNDER BENDING MOMENT AND AXIAL FORCE FLUCTUATIONS [78-79]
5.1 Introduction

In this chapter design formulae for the prediction of structural ductility of pipe sections are given which will simplify the capacity evaluations of the bridge members during the seismic design considerations. Influence of axial force fluctuation are also studied and incorporated in the design formulae since fluctuation of axial force together with the bending moment is a significant phenomenon not only in arch bridges but also at columns of the portal frames. The ductility is studied on steel short cylinders with the length equal to the effective buckling length. Because the occurrence of local buckling always take place along a local part in the shape of so-called ‘elephant foot’ buckling [2] regardless the total length of the structural component and the whole ductility is governed with the ductility of this short section [53-56].

First, elasto-plastic large-displacement analyses are conducted on parametric short steel cylinders models which are loaded under combined compression and bending. Axial force fluctuation is considered through a monotonic loading condition.

Then, bending behavior under constant and fluctuating axial force cases are compared, where the final value of considered axial force fluctuation is the same as the axial force magnitude in the constant axial force case. It is found that moment and ductile capacity corresponding to the post-peak region of bending behavior are significantly improved when axial force fluctuation is considered.

Finally, design formulae for failure strain taking into account the influence of the consideration of axial force fluctuation are generated for different limit states. Validity and the efficiency of the formulae are verified through numerical analysis.
5.2 Numerical Analysis Method

5.2.1 Analyzed models

The influence of axial force fluctuation on the bending behavior of short steel cylinders is examined numerically for nine models with the structural parameters listed in Table 5.1.

<table>
<thead>
<tr>
<th>Model</th>
<th>Diameter (D-mm)</th>
<th>Thickness (t-mm)</th>
<th>Length (L-mm)</th>
<th>D/t</th>
<th>R_t</th>
<th>L/D</th>
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<td>20</td>
<td>173.6</td>
<td>53.1</td>
<td>0.050</td>
<td>0.163</td>
</tr>
<tr>
<td>2</td>
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<td>66.4</td>
<td>0.063</td>
<td>0.150</td>
</tr>
<tr>
<td>3</td>
<td>1988</td>
<td>20</td>
<td>252.5</td>
<td>99.4</td>
<td>0.094</td>
<td>0.127</td>
</tr>
<tr>
<td>4</td>
<td>2656</td>
<td>20</td>
<td>294.5</td>
<td>132.8</td>
<td>0.125</td>
<td>0.111</td>
</tr>
<tr>
<td>5</td>
<td>3980</td>
<td>20</td>
<td>353.5</td>
<td>199.0</td>
<td>0.188</td>
<td>0.089</td>
</tr>
<tr>
<td>6</td>
<td>5308</td>
<td>20</td>
<td>390.6</td>
<td>265.4</td>
<td>0.250</td>
<td>0.074</td>
</tr>
<tr>
<td>7</td>
<td>6636</td>
<td>20</td>
<td>413.1</td>
<td>331.8</td>
<td>0.313</td>
<td>0.062</td>
</tr>
<tr>
<td>8</td>
<td>7962</td>
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<td>398.1</td>
<td>0.375</td>
<td>0.053</td>
</tr>
<tr>
<td>9</td>
<td>10616</td>
<td>20</td>
<td>407.2</td>
<td>530.8</td>
<td>0.500</td>
<td>0.038</td>
</tr>
</tbody>
</table>

The radius-thickness ratio parameter ($R_t$) is adopted as the main structural parameter. It is defined by

$$R_t = \sqrt{\frac{3(1-\nu)^2}{\pi}} \frac{\sigma_y D}{E 2t} \quad (5.1)$$

where, $E =$ Young’s modulus, $\nu =$ Poisson’s ratio, $\sigma_y =$ yield stress, $D =$ Diameter of the cylinder and $t =$ thickness of cylinder wall. Cylinders with $R_t$ values ranging from 0.05 to 0.5 are generated by changing the diameter of the cylinder only, keeping an identical thickness of 20mm.

For a thin-walled cylinder subjected to compressive loading or a bending moment, the occurrence of local buckling has great influence on the ultimate strength and ductility. The determination of critical state often related with local buckling behavior. The buckled shape is symmetrical with respect to the axis of the cylinder and is in the shape of several half sine waves. In this study short cylindrical segments equal in length to the critical wavelength, i.e. the length which provides the minimum ultimate strength, are adopted. In Timoshenko’s elastic shell theory [65] the critical wave length in which the shell buckles is given by the formula,
The formula results in similar critical lengths to Timoshenko’s formula, which is an indication that Timoshenko’s formula is also valid in the inelastic region. Although the two formulae lead to identical results, in the present study formula by Gao et al. [45] is preferred to set the lengths of the cylinders shown in Table 5.1 since the formula is based on the analysis of cylinders with similar \( R_t \)-values.

### 5.2.2 Finite element (FE) modeling

The short cylinders are modeled and analyzed using MARC [61] non-linear FE analysis software. Because of the symmetry about the midsurface in the longitudinal direction, only half of each cylinder are modeled. A type of four-node doubly curved shell element (No. 75) included in the MARC element library is adopted for cylinder modeling. This thick-shell element has global displacements and rotations as degrees of freedom and has five integration points along the shell thickness. The finite element mesh of a cylinder model is shown in Figure 5.1. The numbers of divisions in the circumferential and longitudinal directions are 90 and 10, respectively.

A type of steel stress-strain relation that includes a strain-hardening component, as proposed by Usami et al. [66], is utilized (Figure 5.2). Here \( \sigma_y \) and \( \varepsilon_y \) denote the yield stress and strain, respectively, \( \varepsilon_{st} \) is the strain at the onset of strain hardening, \( E_{st} \) is the initial strain-hardening modulus and \( E' \) is the strain-hardening modulus assumed as

\[
E' = E_{st} \exp \left( -\eta \frac{\varepsilon - \varepsilon_{st}}{\varepsilon_y} \right)
\] (5.4)

where \( \xi \) = material coefficient. In this study, mild steel SS400 is utilized with \( \sigma_y = 235 \) MPa, \( E = 206 \) GPa, \( \nu = 0.3 \), \( \varepsilon_{st} = 10\varepsilon_y \), \( E_{st} = E/40 \) and \( \eta = 0.06 \). The large deformation effect is considered by the updated Langrangian formulation. The non-linear equilibrium is solved by arc-length method.
The boundary condition of the models is selected to simulate the local buckling to take the form of the upper half part of a half sine wave. For this purpose a fixed boundary condition is assumed for the transverse rotations and the vertical transition of the bottom edge of the models. The upper edge is set free in all degrees of freedom under the constraint to keep a plane. Deformed configuration of an analyzed cylinder is shown in Figure 5.3. It can be observed that the outward displacement is in the form of a single half wave when the symmetrical displacement in the lower part of the cylinder is taken into account.
2) Initial imperfections

The initial geometrical deflection pattern in this study is set by evaluating several deflection modes in longitudinal direction and selecting the one that gives the worst moment capacity. The considered initial deflection modes and the corresponding bending behavior are shown in Figure 5.4. Circumferential direction deflection modes are not considered as they don’t have any influence on the bending behavior of short cylinders [45]. The considered initial deflection modes are in the shape of single half-sine waves defined by the following equation

$$w = w_{max} \sin \left( \frac{\pi}{L} \left( \frac{L}{2} - z \right) \right)$$

Here, $w$ is the outward displacement at coordinate $z$ starting from the bottom of the half cylinder and $w_{max}$ is the maximum outward displacement. Different maximum deflections in outward or inward direction are considered as shown in the figure. Here, 0.01$L$ and 0.005$L$ are arbitrary values selected to evaluate the influence of maximum deflections on the capacity and 0.0025$L$ is the average value of the measured maximum deflections of the steel short cylinder test specimens of a prior experimental study [57]. The comparison is illustrated for Model 4 ($R_t=0.125$) under the constant axial magnitude of $0.4P_y$ ($P_y=$ Squash load).
Figure 5.4. Comparison of the bending behavior for different patterns of initial deflections (Model4, $R_t=0.125$, $P/P_y=0.4$)

It can be seen that outward direction displacement leads to lower capacity and the capacity decreases with the increase in the maximum deflection. For the deflection pattern of this study, a half sine wave is selected with $0.0025L$ as the maximum deflection since it is a realistic value and a value that is considered to be large enough when compared to the allowable initial deflections of plates in compression specified by the Japanese Design Code for Highway bridges [69].

In addition, in order to verify that single half wave is the most critical initial deflection mode, we considered different number of half sine waves with the maximum deflections of $0.0025L$. This is verified by the results illustrated in Figure 5.5.
Based on these results, it is considered that an initial deflection pattern with similar shape to the buckled geometry is the most unfavorable case so that a single half sine wave in outward direction is employed as the initial deflection pattern in this study. It should be noted that the employed initial deflection pattern is the same with the pattern in Gao et al.’s [45] study which evaluate the ductility of short cylinders of similar dimensions under constant axial load.

In this analysis, residual stress due to welding is also considered. The stress distribution is idealized as illustrated in Figure 5.6. The welding point is arranged to be on the compression side so as to obtain a conservative estimate of capacity.

![Figure 5.5. Comparison for different number of half waves (Model4, \(R_t=0.125\), \(P/P_y=0.4\))](image1.png)

![Figure 5.6. Distribution of residual stress](image2.png)
3) Validity of modeling only the upper half of the cylinder

For the steel cylinders symmetry in the buckled shape generally disappears after the ultimate load reached due to the localization of the buckling phenomenon. In such cases the half modeling based on the symmetrical deformation pattern cannot be valid. However studies by Goto et al. indicate that localization phenomenon does not take place for the cylinders where the length/radius ratio is less than 1 [67-68] and in the present study its largest value is 0.33 among the analyzed short cylinder models. In order to verify that no localization of buckling occurs, we conducted analysis with the full length cylinder model and compared to the results of half cylinder model under various constant axial loads. As an example, the comparison conducted for Model 1 (has the largest length/radius ratio) is illustrated in Figure 5.7 for the bending behavior and buckled geometry, respectively. It can be seen that the results of half model overlaps the results of the full model since localization of the buckling do not take place. Comparisons are also conducted for the other models and similar results are obtained suggesting that modeling only half of the cylinder is a valid idealization to evaluate the bending behavior of the cylinders considered in this study.

![Figure 5.7. Comparison of half and full cylinder modeling](image)

a) Moment-Average Strain at the outmost edge on the compressive side.

b) Buckled geometry (side view of the outmost fiber on the compressive side).

5.2.3 Loading conditions

In many structures, axial force fluctuates significantly together with the bending moment during an earthquake. A typical fluctuation pattern is given in Figure 5.8, which
shows the relationships between axial force and bending moment for a portal frame bridge pier and for the arch rib of a upper-deck type steel arch bridge under in-plane excitations. It can be seen that both relations have a linear form, although the one for the arch bridge is more complicated because of the complexity of the structure.

**Figure 5.8.** Axial force-bending moment relationship (a) for a portal frame (b) for arch rib of an arch bridge

In this study, the monotonic loading shown in **Figure 5.9** is adopted as an idealization of this kind of cyclic fluctuation. During loading, the axial force and bending moment increase together in a linear manner. They reach their maximum values at the same instant and start decreasing together after that. Although the axial force and bending moment relationship tends to be more complicated in arch bridges due to the contribution of the higher modes to the overall response, the loading condition used here is considered to be sufficient, as it will lead to conservative capacity estimations.

**Figure 5.9.** Assumed monotonic loading condition

In order to simulate the axial force fluctuation, an eccentric displacement load ($P_\delta$) that results in linear axial force and bending moment increments at upper segment center of the cylinder is applied, as shown in **Figure 5.10**. The top of this upper segment is constrained as a rigid plane and linked to the center node to impose bending of the
cylinder. A load \((P_i)\) that accounts for the initial value of the axial force fluctuation is applied to this node and the final axial force \((P_f)\) is adjusted to the desired value by adjusting the eccentricity \((e)\). The results are compared with the constant axial force case, in which the final axial force of the fluctuating axial force case is applied to the center node as a fixed value. In the constant axial force case, bending behavior is obtained by applying rotation increments to the center node.

**Figure 5.10.** Loading method for axial force fluctuation

It should be noted that the moment is assumed to have a uniform distribution in both the fluctuating and constant axial force loadings, as shown in the sketch in **Figure 5-11a**, whereas seismic action would cause a moment gradient in steel bridge piers. This assumption is based on a study by Zheng at al. [46] who pointed out that although the bending moment capacity increases with a decrease in moment gradient (defined as \(M_1/M_2\) in **Figure 5-11b**), the ductility changes slightly. Furthermore, the cylinders in this study are quite short compared to their diameters, making the influence of the moment gradient quite negligible.

**Figure 5.11.** Sketch of the studied short cylinder and moment gradient

The main focus in this research is on evaluating the influence of the axial force fluctuation on ductility. This is represented by the failure strain, which is defined as the average strain at the outmost edge on the compressive side and is calculated by
\[ \varepsilon = \frac{2u}{L} \quad (5.6) \]

where \( u \) = longitudinal displacement of the upper or lower end of the compressive side (point A in Fig. 8a) and \( L \) = length of the short cylinder.

### 5.2.4 Axial force fluctuation parameters

Analyses with different axial force fluctuation patterns are carried out for three different final axial force levels, as illustrated in Table 5.2. The amount of axial force fluctuation (\( \alpha \)) is the ratio of final axial force (\( P_f \)) to initial axial force (\( P_i \)). Different \( \alpha \)-values are obtained by changing the initial axial force for a given final axial force level. Additionally, analysis with constant axial force, which is represented by \( \alpha = 1 \), is carried out for each final axial force case. The final axial force values are selected as 20%, 40% and 60% of the squash load (\( P_y \)) of the cylinders and for each case different fluctuation amounts of 1.5, 2 and 3 are employed. These axial force fluctuation parameters are considered to represent a wide range of realistic values of axial force fluctuations that can take place in the sections of steel bridges under severe earthquake excitations.

<table>
<thead>
<tr>
<th>( P_f )</th>
<th>( \alpha )</th>
<th>( P_i )</th>
</tr>
</thead>
<tbody>
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<td>( 0.2P_y )</td>
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<td>( 0.20P_y )</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>( 0.13P_y )</td>
</tr>
<tr>
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<td>( 0.10P_y )</td>
</tr>
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<td>( 0.07P_y )</td>
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<td>( 0.40P_y )</td>
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<td>3.0</td>
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</tr>
<tr>
<td>( 0.6P_y )</td>
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<td>( 0.60P_y )</td>
</tr>
<tr>
<td></td>
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<td>( 0.40P_y )</td>
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</tr>
<tr>
<td></td>
<td>3.0</td>
<td>( 0.20P_y )</td>
</tr>
</tbody>
</table>

### 5.3 Influence of Axial Force Fluctuations

#### 5.3.1 Verification of loading conditions

The various different axial force patterns are illustrated in Figure 5.12, together with the corresponding bending behavior for Model 4 (\( R_t = 0.125 \)) with respect to the rotation of the section when the final axial force is \( 0.6P_y \). All axes are normalized by
their values at the yield state. It can be seen that the maximums of axial force and bending moment take place at the same rotational instants and that variations in bending moment and axial force are similar to the assumed monotonic loading condition shown in Figure 5.9. This suggests that simulation by eccentric loading is an efficient analogy for considering axial force fluctuations in short cylinder models.

![Diagram of axial force-rotation relationship](image)

![Diagram of bending moment-rotation relationship](image)

**Figure 5.12.** Bending behavior for different \( \alpha \)-values (Model 4)

### 5.3.2 Moment-rotation relationship

In the bending behavior of Model 4 shown in Figure 5.12 it can be observed that the ultimate moment capacities of constant axial force and different fluctuating axial force cases are the same. However, there is significant difference after the maximum moment is reached. In the post-peak region, the moment for fluctuating axial force cases is observed to drop more slowly, resulting in higher ductility which is more significant for higher amounts of fluctuation. The axial force in the fluctuating axial force case
decreases after the peak value, whereas it is maintained even after the maximum load in the constant axial force case. This induces the difference in moment-rotation relationships in the post-peak region. Although the constant axial force case is a more severe loading condition resulting in conservative design, it is more realistic to take the axial force fluctuation and the corresponding reduction in the post-peak region into consideration, leading to more rational design of steel sections.

5.3.3 Ductility

There are some design codes which allow for post-peak behavior up to 95% [70], 90% [70] and even as far as 80% [71] of the moment capacity. In order to study the ductility improvement at different post-peak locations, limit states in the present study are selected by defining the failure strain as the strain corresponding to the 95%, 90% and 80% of maximum post-peak moment, namely $M_{95}$, $M_{90}$, $M_{80}$ (See Figure 5.13). The ductility in constant and fluctuating axial force cases at these limit states is compared for a given final axial force level in Table 5.2. The comparison is illustrated in Figure 5.14, where the ratio of failure strain for the axial force fluctuation case under consideration to that of the corresponding constant axial force case is plotted with respect to the $R_t$ parameter for all final axial force levels. It can be seen that the improvement obtained by considering the effect of axial force fluctuation in post-peak ductility is valid for all models. The improvement is directly proportional to the final axial force level and the amount of axial force fluctuation. These two tendencies are more obvious when further post-peak behavior is considered. It can be also seen that the results follow a similar path with respect to the $R_t$ value, although the ratio is larger when $R_t$ is between 0.06 and 0.1. Excluding these scattered values, the overall trend of the improvement ratio can be approximated by the curves shown in the figure.

![Figure 5.13. Definition of limit states](image-url)
Figure 5.14. Comparison of the post-peak ductility ($M_{95}$, $M_{90}$)
Figure 5.14 (continued). Comparison of the post-peak ductility ($M_{80}$)
5.3.4 Comparison with existing numerical results

To verify the validity of the obtained results, normalized failure strain ($\varepsilon_u/\varepsilon_y$) in the constant axial force case is compared with values computed using formula (5.7) given by Ge et al. [58] for 95% of the ultimate post-peak strength, treating the axial force as a constant value.

$$
\frac{\varepsilon_u}{\varepsilon_y} = \frac{0.14(1.1 - P/P_y)^{1.8}}{(R_t - 0.03)^{1.4}} + \frac{3}{(1 + P/P_y)^{0.7}} \leq 20 \quad (0 \leq P/P_y \leq 1.0, \ 0.03 \leq R_t \leq 0.5) \quad (5.7)
$$

Figure 5.15 illustrates this comparison for the constant axial forces of $0.1P_y$, $0.2P_y$ and $0.3P_y$. Additionally, the individual normalized failure strains obtained in the present study are compared with those given by Gao et al. [45] in Table 5.3. It is seen that the results show sufficiently good agreement.

![Figure 5.15. Comparison with existing equation](image-url)
Table 5.3. Comparison with the numerical results by Gao et al.

<table>
<thead>
<tr>
<th>$R_t$</th>
<th>Results from Gao et al. ($\varepsilon_u/\varepsilon_y$)</th>
<th>Current study ($\varepsilon_u/\varepsilon_y$)</th>
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<td>$P/P_y=0.2$</td>
</tr>
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<td>0.063</td>
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</tr>
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</tr>
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<td>0.094</td>
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Gao et al./Current study

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5.4 Design Formulae

5.4.1 Generation of formulae

Rather than generating a completely new formula, we propose accounting for the influence of axial force fluctuation in the design procedure by modifying the existing constant axial force formula with appropriate correction functions.

The correction functions are developed based on curves approximating the relationship shown in Figure 5.14. These curves are obtained by using the least squares method to approximate all analysis results for one $\alpha$-value as a power function of $R_t$. The individual curve functions are combined into a single function for each limit state and given as equations (5.8-5.10) using the influence of the amount of axial force fluctuation and the final axial force level to position the curves.
By using these proposed correction functions (5.8-5.10), it is possible to estimate the failure strain in consideration of the axial force fluctuation effect for a given \( R_t \) value and given axial force fluctuation parameters if the corresponding value for the constant axial force case is known. Equation (5.7) may be utilized as the formula for the constant axial force case, but it is available only for the limit state corresponding to \( M_{95} \) (See Figure 5.13). For the other two limit states, similar equations are generated as shown below:

\[ M_{90} \]

\[
f(R_t, P_f / P_y, \alpha) = \frac{(0.095\alpha + 0.024)P_f / P_y + 1.001}{R_t[(0.017\alpha + 0.007)P_f / P_y - 0.006\alpha - 0.003]} \geq 1,
\]

\[0.05 \leq R_t \leq 0.5 \quad (5.8)\]

\[ M_{80} \]

\[
f(R_t, P_f / P_y, \alpha) = \frac{(0.193\alpha + 0.05)P_f / P_y + 0.981}{R_t[(0.003\alpha + 0.007)P_f / P_y - 0.005\alpha - 0.003]} \geq 1,
\]

\[0.05 \leq R_t \leq 0.5 \quad (5.9)\]

\[ M_{80} \]

\[
f(R_t, P_f / P_y, \alpha) = \frac{(0.396\alpha + 0.028)P_f / P_y + 0.967}{R_t[(0.004\alpha - 0.043)P_f / P_y - 0.013\alpha + 0.018]} \geq 1,
\]

\[0.05 \leq R_t \leq 0.5 \quad (5.10)\]

Estimates of failure strain obtained with the newly developed constant axial force case formulae of \( M_{90} \) and \( M_{80} \) are compared with the results of analysis for various constant axial force magnitudes in Figure 5.16. Results for the pure bending case \((P/P_y = 0)\) are also plotted. The equations result in fairly accurate and conservative estimates for axial forces of moderate magnitude, whereas the formulae tend to give inaccurate
results for axial forces more than $0.6P_y$. It should be noted that such high axial force levels are not usually considered in the design procedure. The upper limit for the estimation of normalized failure strain is set at 20, a realistic value similar to equation (6) given by Ge et al., since higher failure strains may cause problems due to low cycle fatigue or brittle fracture although there is no risk of local buckling.
Figure 5.16. Comparison of analysis results and the proposed equation for constant axial force case
5.4.2 Estimation of ductility using proposed formulae

The main steps involved in estimating failure strain using the proposed formulae are summarized below and illustrated in Figure 5.17.

1) Calculate the seismic demand of the structure and get the initial ($P_i$) and maximum ($P_f$) values of axial force occurring in the pipe section under consideration.

2) With the assumption of monotonic loading, calculate the $\alpha$-value defined as $P_f/P_i$ using the values obtained in 1).

3) Estimate the failure strain using the constant axial force case formula (equations (5.7), (5.11), (5.12)) for the desired limit states by substituting the maximum axial force $P_f$ for $P$.

4) Calculate the value of the correction function (equations (5.8), (5.9), (5.10)) for the corresponding limit state using the axial force fluctuation parameter in 2). Then multiply the obtained value with the result of 3) to get the final estimates.

**Figure 5.17.** Outline of the estimation of ductility capacity
5.4.3 Application range of the estimation

The range of loading conditions for which estimations with the proposed formulae are applicable is investigated by evaluating the accuracy of the estimates for axial force fluctuation patterns not considered during the generation of the correction functions. A wide range of realistic cases are studied by adding different amounts of axial force fluctuation to the initial axial force levels of $0.1P_y$, $0.2P_y$, $0.3P_y$ and $0.4P_y$, as shown in Table 5.4. Within the range of $\alpha$-values from 1.25 to 4, most of the estimates are found to be on the conservative side, with an error of less than 20% if the final axial force does not exceed $0.6P_y$ (the upper applicable limit of the constant axial force case formulae). This region is specified as the applicable range of the estimation and is marked in Table 5.4. Outside this region, the accuracy of estimates is low and there are many cases for which estimates on the safe side cannot be achieved. In Figure 5.18, the accuracy of estimates within the applicable range is illustrated by plotting estimates against analysis results. It can be seen that conservative and fairly accurate estimations are achieved especially when the failure strain is between 5 and 20, which is the preferred ductility range for seismic design considerations.

Table 5.4. Applicable range of the proposed method

<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>$P_I=0.1P_y$</th>
<th>$P_I=0.2P_y$</th>
<th>$P_I=0.3P_y$</th>
<th>$P_I=0.4P_y$</th>
</tr>
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<tbody>
<tr>
<td>1.25</td>
<td>Applicable</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
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<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>Invalid</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>Invalid</td>
</tr>
<tr>
<td>6</td>
<td>Invalid</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Invalid</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Although the proposed formulae cover a wide range of $R_t$ values from 0.05 to 0.5, the requirement for high ductility in design limits the practical application range of the proposed formulae in terms of $R_t$. In practice, $R_t$ values between 0.05 and 0.11 find most common application for pipe sections in steel bridge piers. On the other hand, $R_t = 0.3$ can be set as a reliable upper limit since there has been a study [44] in which the validity of large-displacement finite element analysis for large-diameter steel cylinders with $R_t$ as much as 0.3 is verified through comparison with test results.

The estimation of the ductility with the proposed formulae is considered to be valid also for different steel types since the formulae are generated as a function of yield stress which normalizes the failure strain. However, the estimation accuracy will be different for steel types having different inelastic stress-strain relationship parameters such as the length of the yield plateau and strain hardening slope. In order to understand the influence of these parameters on ductility capacity numerical examinations are carried out. Three different values of yield plateau are studied ($\varepsilon_{st} = 12\varepsilon_y$, $\varepsilon_{st} = 7\varepsilon_y$, $\varepsilon_{st} = 5\varepsilon_y$) for a fixed strain hardening slope ($E_{st}=E/40$) and three differing strain hardening slope values are studied ($E_{st}=E/20$, $E_{st}=E/30$, $E_{st}=E/50$) for fixed yield plateau length ($\varepsilon_{st} = 10\varepsilon_y$) in order to evaluate the individual effects of these two parameters on ductility. The analyses are conducted for all models for a constant axial force magnitude of $0.3 P_y$. 

**Figure 5.18.** Estimation accuracy
The failure strains computed for $M_{05}$ limit state are illustrated in Figure 5.19 respectively for the two studied cases. The constant axial force case formulae results are also plotted on the graphs. It can be seen that the ultimate ductility capacity drops when the length of the strain plateau gets longer and when the slope of the strain hardening gets smaller. Proposed formulae will result in conservative side estimation for most of the steel types. Because the steel type (SS400) used in the numerical analysis has relatively long yield plateau ($\varepsilon_{st} = 10\varepsilon_y$) and a small strain hardening slope ($E/40$) compared to other types of steel used in Japan.

![Graph showing influence of material modeling on failure strain](image)

**Figure 5.19.** Influence of Material Modeling on Failure Strain ($M_{05}$)
5.4.4 Efficiency of the proposed formulae

The proposed formulae increase the efficiency of limit state design by magnifying the failure strain obtained with the constant axial force assumption using correction functions accounting for the influence of axial force fluctuations. This leads to design with a higher radius-thickness ratio for a given ductility demand. Table 5.5 contains limit values of $R_t$ for certain required ductility values for three limit states. For a given ductility demand, the limit values of $R_t$ can be obtained from this table. For example, if the normalized failure strain is required to be more than 8 under the conditions of $P_f/P_y=0.2$ and $\alpha=3$ for the $M_{95}$ limit state, the $R_t$ value should not exceed 0.097, which is a higher value than the $\alpha=1$ case. It is seen that the increase in the limit $R_t$ value becomes more noticeable when the amount of axial force fluctuation is larger and when higher axial force magnitudes are considered. The table illustrates the limit values only for moderate final axial force magnitudes, i.e. less than 0.3$P_y$. The efficiency improvement will be more obvious for higher axial force levels between $P_f/P_y = 0.3$ and $P_f/P_y = 0.6$.

Table 5.5. Limit values of $R_t$ for required ductility

a) $M_{95}$

<table>
<thead>
<tr>
<th>$P_f=0.1P_y$</th>
<th>$P_f=0.2P_y$</th>
<th>$P_f=0.3P_y$</th>
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</thead>
<tbody>
<tr>
<td>$\alpha=1$</td>
<td>$\alpha=1.5$</td>
<td>$\alpha=2$</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=5</td>
<td>0.170 0.171 0.171 0.171</td>
<td>0.146 0.150 0.152 0.154</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=6</td>
<td>0.137 0.137 0.137 0.137</td>
<td>0.120 0.123 0.124 0.125</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=8</td>
<td>0.106 0.106 0.106 0.106</td>
<td>0.095 0.096 0.097 0.097</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=10</td>
<td>0.090 0.090 0.090 0.090</td>
<td>0.082 0.083 0.083 0.083</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=20</td>
<td>0.062 0.062 0.062 0.062</td>
<td>0.058 0.058 0.058 0.059</td>
</tr>
</tbody>
</table>

b) $M_{90}$

<table>
<thead>
<tr>
<th>$P_f=0.1P_y$</th>
<th>$P_f=0.2P_y$</th>
<th>$P_f=0.3P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha=1$</td>
<td>$\alpha=1.5$</td>
<td>$\alpha=2$</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=5</td>
<td>0.269 0.270 0.273 0.279</td>
<td>0.213 0.224 0.229 0.239</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=6</td>
<td>0.206 0.206 0.207 0.211</td>
<td>0.169 0.176 0.179 0.185</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=8</td>
<td>0.148 0.148 0.149 0.150</td>
<td>0.127 0.131 0.132 0.136</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=10</td>
<td>0.121 0.121 0.121 0.122</td>
<td>0.106 0.108 0.109 0.111</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=20</td>
<td>0.075 0.075 0.075 0.075</td>
<td>0.068 0.069 0.070 0.071</td>
</tr>
</tbody>
</table>

c) $M_{80}$

<table>
<thead>
<tr>
<th>$P_f=0.1P_y$</th>
<th>$P_f=0.2P_y$</th>
<th>$P_f=0.3P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha=1$</td>
<td>$\alpha=1.5$</td>
<td>$\alpha=2$</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=5</td>
<td>– – – –</td>
<td>0.396 0.468 0.505 0.595</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=6</td>
<td>0.411 0.432 0.445 0.474</td>
<td>0.291 0.329 0.345 0.383</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=8</td>
<td>0.257 0.265 0.269 0.278</td>
<td>0.200 0.218 0.225 0.241</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=10</td>
<td>0.195 0.200 0.202 0.206</td>
<td>0.158 0.170 0.174 0.183</td>
</tr>
<tr>
<td>$\varepsilon_u/\varepsilon_y$=20</td>
<td>0.105 0.106 0.106 0.107</td>
<td>0.091 0.095 0.096 0.099</td>
</tr>
</tbody>
</table>
5.5 Summary

This study involved the elasto-plastic large-displacement analysis of short steel cylinders subjected to a bending moment together with axial force fluctuations. The bending behavior of the cylinders was compared with the conventional constant axial force case and the dominant factors in the observed difference were clarified. Based on the results of this examination, design formulae that take into account axial force fluctuations were developed for the estimation of steel pipe failure strain under three different limit states. The accuracy of the formulae was evaluated through additional numerical analysis. Limit $R_t$ values for a given ductility requirement were tabulated for moderate final axial force cases. The findings can be summarized as follows.

1) Ductility and strength corresponding to post-peak behavior are improved when axial force fluctuations are considered.

2) The improvement in ductility is greater for higher axial force magnitudes and for larger axial force fluctuations.

3) The proposed formulae can be used to determine the ductility capacity of pipe section columns in portal frames and arch ribs in arch bridges subjected to bending as well as axial force fluctuations during earthquakes.

4) The consideration of axial force fluctuations using the proposed formulae will result in the use of higher ductile capacities as design values compared with conventional practice, making the seismic design more rational.
CHAPTER 6.

ULTIMATE STRAIN OF STIFFENED STEEL BOX SECTIONS UNDER BENDING MOMENT AND AXIAL FORCE FLUCTUATIONS
6.1 Introduction

In this chapter, similar steps with the previous chapter are followed to generate ductility formulae for the stiffened steel box sections. Steel box sections have larger application than the pipe sections at the arch ribs and the side piers of the upper-deck steel arch bridges as well as at the cantilever and portal frame bridge piers in Japan. They suffered extensive damage in the form of local-buckling during the Hyogo-ken Nanbu Earthquake [2].

The ductility investigations are carried out by focusing on the critical local part where the local buckling occurs. Parametric stiffened short steel box column models with the effective failure length of the whole structural component are generated. Elasto-plastic large displacement analyses of the models are conducted under combined compression and bending.

Influence of axial force fluctuation is investigated for several loading patterns. The ductile capacity is observed to be improved corresponding to the post-peak region similar to the case of the columns with pipe sections.

Tendency of the improvement with respect to several fluctuation parameters is studied and design formulae that considers the influence of axial force fluctuation is proposed. Validity and efficiency of the proposed formulae are demonstrated through numerical analysis.
6.2 Numerical Analysis Method

6.2.1 Main parameters of stiffened box short columns

The behavior of the thin-walled box columns are greatly influenced by the flange width-thickness ratio $R_f$ defined as

$$R_f = \frac{b}{t} \sqrt{\frac{12(1-v^2)}{4n^2\pi^2}} \sqrt{\frac{\sigma_y}{E}}$$

(6.1)

where $E$=Young’s modulus, $v$=Poisson’s ratio, $\sigma_y$=yield stress, $b$=flange width, $t$=plate thickness, and $n$=number of subpanels separated by the stiffeners (See Figure 6.1)

In addition to the $R_f$ parameter, characteristics of the stiffener plates are also influential. In a previous study conducted by Usami et al. [51] on the ductility of isolated panels with stiffeners, it was found that the stiffener’s slenderness ratio $\bar{\lambda}_s$ is a very essential parameter to represent the characteristics of stiffeners and panels because of its inherent relation to $R_f$ and $\gamma / \gamma^*$ (the ratio of the stiffener’s relative flexural rigidity to its optimum value obtained from the elastic buckling theory). The definition of $\bar{\lambda}_s$ is as follows [51]:

$$\bar{\lambda}_s = \frac{1}{Q r_s \sqrt{\pi}} \sqrt{\frac{\sigma_y}{E}}$$

(6.2)

$$Q = \frac{1}{2R_f} \left[ \beta - \sqrt{\beta^2 - 4R_f} \right]$$

(6.3)

$$\beta = 1.33R_f + 0.868$$

(6.4)

in which $r_s$= radius of gyration of the T-shape cross section which consists of one longitudinal stiffener and adjacent subpanels; $Q$= local buckling strength of the subpanel plate [47]. The stiffener’s relative flexural rigidity $\gamma$ and optimum value $\gamma^*$ are defined in the Japanese Design Code for Highway Bridges [69] as

$$\gamma = \frac{I_1}{bt^3}$$

(6.5)

$$\gamma^* = 4\alpha_a^2 n \left(1 + n \frac{b_j t_s}{b t} \right) - (\alpha_a^2 + 1)^2 / n, \; \alpha_a \leq \alpha_0$$

(6.6)
where \( I_1 = \) stiffener’s section inertia moment with respect to its end connected to plate; \( b_s \) and \( t_s \) = width and thickness of the stiffener, respectively; \( \alpha_a \) is the aspect ratio and \( \alpha_0 \) is critical aspect ratio of stiffened panel defined by the following equation.

\[
\alpha_0 = \sqrt[\frac{2}{3}]{1 + 4\gamma} \tag{6.7}
\]

### 6.2.2 Analyzed models

The \( R_f \) is selected as the main parameter and 6 models are generated with square cross sections by giving variation to it from 0.40 to 0.65. During the generation procedure, only the width of the cross section is changed. From a practical point of view the thickness is kept identical as 20 mm. Rectangular cross sections are not generated since failure strain discrepancy due to the depth-to-breadth ratio \((d/b)\) of cross section is found to be negligible by the study of Zheng et al. [46] within the range of 0.67 to 1.33 which is the most widely employed depth-to-breadth ratio range for rectangular sections of steel bridges in practical design [72]. The length of the short columns is taken as \( L_B = 0.7b \) which is the effective failure length of members with box-sections [56]. An identical cross section shape is employed for all models as illustrated in Figure 6.1.

In order to study the influence of the slenderness of the stiffener plates, \( \gamma/\gamma^* \) is selected as another parameter. Two sets of stiffener plate dimensions are generated for each \( R_f \) value by setting \( \gamma/\gamma^* \) either to 1 or 3. The structural parameters of the 12 models generated in this way are summarized in Table 6.1.

![Figure 6.1. Cross section of analyzed steel box short columns](image)
### Table 6.1. Structural parameters of the analyzed models

<table>
<thead>
<tr>
<th>Model</th>
<th>1</th>
<th>3</th>
<th>5</th>
<th>7</th>
<th>9</th>
<th>11</th>
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<td>$\gamma'/\gamma^*$</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
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<tr>
<td>$R_f$</td>
<td>0.40</td>
<td>0.45</td>
<td>0.50</td>
<td>0.55</td>
<td>0.60</td>
<td>0.65</td>
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<tr>
<td>$b$(mm)</td>
<td>1351.1</td>
<td>1520.0</td>
<td>1688.8</td>
<td>1857.7</td>
<td>2026.6</td>
<td>2195.5</td>
</tr>
<tr>
<td>$t$(mm)</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>$h_s$(mm)</td>
<td>98.2</td>
<td>101.6</td>
<td>104.7</td>
<td>107.7</td>
<td>110.5</td>
<td>113.1</td>
</tr>
<tr>
<td>$t_s$(mm)</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>$n$</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>$L_B$(mm)</td>
<td>945.8</td>
<td>1064.0</td>
<td>1182.2</td>
<td>1300.4</td>
<td>1418.6</td>
<td>1536.8</td>
</tr>
<tr>
<td>$\lambda_s$</td>
<td>0.389</td>
<td>0.444</td>
<td>0.501</td>
<td>0.560</td>
<td>0.622</td>
<td>0.687</td>
</tr>
<tr>
<td>$Q$</td>
<td>1.000</td>
<td>0.972</td>
<td>0.941</td>
<td>0.910</td>
<td>0.870</td>
<td>0.845</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.40</td>
<td>1.47</td>
<td>1.53</td>
<td>1.60</td>
<td>1.67</td>
<td>1.73</td>
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<table>
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<tr>
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</thead>
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<td>3.0</td>
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<td>3.0</td>
<td>3.0</td>
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<tr>
<td>$R_f$</td>
<td>0.40</td>
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<td>$b$</td>
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<tr>
<td>$h_s$</td>
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<td>$n$</td>
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<td>3</td>
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</tr>
<tr>
<td>$L_B$</td>
<td>945.8</td>
<td>1064.0</td>
<td>1182.2</td>
<td>1300.4</td>
<td>1418.6</td>
<td>1536.8</td>
</tr>
<tr>
<td>$\lambda_s$</td>
<td>0.245</td>
<td>0.279</td>
<td>0.314</td>
<td>0.350</td>
<td>0.388</td>
<td>0.427</td>
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<tr>
<td>$Q$</td>
<td>1.000</td>
<td>0.972</td>
<td>0.941</td>
<td>0.910</td>
<td>0.870</td>
<td>0.845</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.40</td>
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<td>1.53</td>
<td>1.60</td>
<td>1.67</td>
<td>1.73</td>
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</tbody>
</table>

### 6.2.3 Finite element (FE) modeling

The short box columns are modeled and analyzed using general purpose MARC non-linear FE analysis software. Because of the symmetry of the geometry, only the upper half of a model is analyzed. Element 75 given in the MARC element library is used. The finite element mesh of the modeled portion of a short box column is illustrated in Figure 6.2. There are 10 divisions in the vertical direction. In the horizontal direction, 3 divisions are made for the stiffener plate and 12 divisions are made for the each subpanel between the stiffener plates.
Mild steel SS400 is utilized similar to analysis in Chapter 5 together with the same stress-strain relationship including a strain hardening part which was proposed by Usami et al. [66] (See Figure 5.2). The large deformation effect is considered by the updated Langrangian formulation. The non-linear equilibrium equation is again solved by arc-length method.

1) Boundary conditions

A simply supported boundary condition is assumed along the column end boundaries to simulate the local buckling mode of a long column, which will deform into several waves along the length. Since only upper half of the short column is modeled, a symmetry boundary condition is utilized for the lower edge boundary nodes of the modeled portion that can enable the modeled portion to deform in the form of the upper half of the short column. The transverse rotations and vertical transitions of those nodes are fixed in order to achieve such deformation. The deformed shape of an analyzed model is shown in Figure 6.3. It can be seen that the buckled geometry is in
several half waves in column length and width direction when the symmetric deformation of the not modeled bottom portion is taken into account.

\[ \text{Figure 6.3. Buckling mode of stiffened short column (Model 4, } R_f=0.45, \gamma/\gamma^*=3) \]

2) Initial imperfections

Two kinds of initial imperfections are considered in the analysis, i.e. initial geometric deflection and residual stress due to manufacturing and welding.

For the initial deflections, the same pattern employed in the study of Zheng et al. [46] is adopted since similar short steel box models are used in this study. Global and local initial deflections are taken into account for the stiffened panels. The directions of the initial deflections are assumed inward for flange plates and outward for web plates. The inward initial deflections of the flange plates shown in Figure 6.4 are defined by the following equations

\[ \delta = \delta_G + \delta_L \]  \hspace{1cm} (6.8)

where

\[ \delta_G = \frac{L_B}{1000} \sin \left( \frac{\pi}{L_B} y \right) \cos \left( \frac{\pi}{b} y \right) \]  \hspace{1cm} (6.9)

\[ \delta_L = \frac{b_p}{150} \sin \left( \frac{\pi}{L_B / m} y \right) \cos \left( \frac{\pi}{b_p} y \right) \]  \hspace{1cm} (6.10)
in which $\delta_G$ and $\delta_L$ are the global and local initial deflections, respectively, $b_p$ is the width of the subpanels divided by stiffeners and $m$ is the number of half-waves of local initial deflections in the longitudinal direction. It is taken as 3 in the current study since lowest failure strain is obtained when the local initial deflection mode has three half-waves [46]. The equations lead to the maximum inward deflections of $L_B/1000$ along the column height and $b_p/150$ in the subpanels in correspondence with the maximum allowable initial deflections of plates under compression specified by the Japanese Design Code for Highway Bridges [69]. The initial deflections in the web plates are calculated by replacing $y$ in Equations (6.9) and (6.10) with $x$ and applying the deflection to the outward direction. It can be observed in the buckled geometry given in Figure 6.3 that the buckling modes of the flange and the web plates are in the similar shape with the initial local deflection modes along the length and the width direction. This is thought to be a confirmation that the assumed initial deflections will result in an unfavorable situation in terms of failure strain regarding to the finding of the previous chapter where ductile capacity corresponding to the initial deflection taken in the same shape as the buckled geometry was found to be lowest.

![Diagram](image)

**Figure 6.4. Initial deflections of flange plate**

As for the residual stress, a rectangular uniform stress distribution [73] is adopted for the web and flange plates, which is illustrated in Figure 6.5. The initial stress in
stiffener plates is neglected in the analysis.

**Figure 6.5.** Initial stress distributions in web and flange plates

### 6.2.4 Loading conditions

The linear cycling fluctuation pattern seen in portal frames and arch ribs of arch bridges mentioned in Chapter 5 (See Figure 5.8) is again assumed in a monotonic linear increasing pattern of axial force and bending moments to simulate axial force fluctuation on the short steel box columns. (See Figure 5.9)

The simulation of the monotonic loading is conducted through the displacement control eccentric loading procedure as shown in Figure 6.6, which will cause linear axial force and bending moment increments at the upper segment center of the short steel box columns. The top segment nodes of the short columns are linked to the center node with a constraint condition that makes the segment to keep plane during the induced bending. The initial axial load \( P_i \) is applied to the center node and the final value of the axial force fluctuation \( P_f \) is adjusted by modifying the eccentricity \( e \) of the displacement load \( P_\delta \) in the same way carried out for the short cylinder models. The results are compared with constant axial force case in which the final axial force of the fluctuating axial force case is applied to the short columns as a fixed value as in the conventional design practice. Linearly increasing rotation increments are applied to
upper segment center in the constant axial force case to obtain the bending behavior.

Figure 6.6. Loading method for axial force fluctuation

The ductility of the short columns is evaluated by using failure strain ($\varepsilon_u/\varepsilon_y$), where $\varepsilon_u$ is the average strain on the flange of the short column and $\varepsilon_y$ is the yield strain. It is calculated by the following equation

$$\varepsilon_u = \frac{2u}{L} \quad (6.11)$$

Here, $u=$longitudinal displacement of the upper end of the compressive flange, $L=$length of the short column.

6.2.5 Axial force fluctuation parameters

A parametric study is conducted to study the influence of different axial force fluctuation patterns on the failure strain. The fluctuation patterns used in Chapter 5 are adopted where 3 final axial force levels ($P_f=0.6P_y, P_f=0.4P_y, P_f=0.2P_y; P_y=$Squash load of short box columns) are studied respectively for three different axial force fluctuation amounts ($\alpha=3, \alpha=2, \alpha=1.5; \alpha=P_f/P_y$). The constant axial force case is conducted ($\alpha=1$) for each final axial force level.

6.3 Influence of Axial Force Fluctuation

6.3.1 Verification of loading condition

The validity of using the eccentric loading for the short steel box columns to simulate the idealized monotonic loading is studied by plotting the moment-rotation and axial load-rotation relationships of different axial force fluctuation patterns together on the same graph. This comparison is shown in Figure 6.7 for Model 4 ($R_f=0.45, \gamma_1\gamma_2=3$) when the maximum axial force is 0.6$P_y$. All axes are normalized by their values at yield state. It can be seen that axial force and bending moment increase together and reach their maximum values at the same instant as assumed by the monotonic loading condition.
6.3.2 Moment-rotation relationship

In the moment-rotation relationship shown in Figure 6.7, it can be observed that the ultimate moment is completely the same for all cases which suggests that consideration of axial force fluctuation has no influence on the moment capacity. However, exactly agreeing with the findings of the short cylinders, the ductility in the post-peak region is significantly different from the constant axial load case when the fluctuation of the axial force is considered. In the fluctuating axial force case, the decrease of the axial force after the ultimate moment leads to higher ductility in the post-peak region. It is considered that more rational design can be achieved by taking this conventionally neglected real state behavior into account.

Figure 6.7. Bending behavior for different $\alpha$-values (Model 4)
6.3.3 Ductility

The improvement in the ductility is studied by comparing the failure strains of constant and fluctuating axial force cases for a given final axial force magnitude. In order to study the degree of improvement at different level of post-peak states, the failure strains are selected based on two different limit state definitions; the strain levels corresponding to 95% and 90% of the ultimate moment after the peak ($M_{95}$ and $M_{90}$ as in Chapter 5, Figure 5.13).

The comparison is illustrated in Figure 6.8, where the ratios of failure strain for an axial force fluctuation case to that of the corresponding constant axial force case are plotted with respect to the $R_f$ parameter for all of the final axial force magnitudes. The results are given for the two limit states ($M_{95}$ and $M_{90}$) and $\gamma/\gamma^*$-values ($\gamma/\gamma^*=1$ and $\gamma/\gamma^*=3$). It can be observed that the improvement in ductility is valid for all models being as high as 40% in some cases. The improvement is larger for higher final axial force magnitudes and for larger axial force fluctuation amounts. When the level of improvement of the ductility of the two limit states is compared, it can be seen that improvement increases as further post-peak behavior is considered. Noticing that the similar results have also been obtained for the short steel pipes, it can be said that consideration of axial force fluctuation results in improvement of post-peak ductility regardless the cross section type of the bridge member.

When the relationship of the improvement in ductility with the $R_f$ is observed, it is seen that the improvement ratio follows an almost horizontal path although the ratios for the small $R_f$ values ($R_f=0.40$ and $R_f=0.45$) show some discrepancy from the general tendency. The tendency of the improvement ratios are also similar for $\gamma/\gamma^*=1$ and $\gamma/\gamma^*=3$ cases suggesting that not only the slenderness of the main cross section composed of the stiffened panels but also the stiffener plates’ stiffness have no significant influence on the improvement ratio of ductility.
Figure 6.8. Comparison of the post-peak ductility for $M_{95}$ and $M_{90}$ ($\gamma / \gamma ^* = 1$)
Figure 6.8 (continued). Comparison of the post-peak ductility for $M_{95}$ and $M_{90}$ ($\gamma^*/\gamma = 3$)
6.3.4 Comparison with existing numerical results

In order to verify the validity of the analysis, normalized failure strain values of the constant axial force cases are compared with the computed values by an existing constant axial force case formula. The formula (6.12) proposed by Ge et al. [58] is employed for this purpose. This formula yields the failure strain for the 95% of the ultimate moment after the peak. It should be noticed that the formula is given in terms of $R_f \overline{\lambda_s}^{0.18}$, which is an index that represents the stiffness of the whole cross section composed of stiffened panels and stiffeners. The study of Ge et al. expresses that no straightforward relation of failure strain with respect to the independent values of $R_f$ or $\overline{\lambda_s}$ is observed whereas a consistent inverse relationship between failure strain and $R_f \overline{\lambda_s}^{0.18}$ is found.

\[
\frac{\varepsilon_u}{\varepsilon_y} = \frac{0.7}{(R_f \overline{\lambda_s}^{0.18} - 0.18)^{1.3}} \left(1 + \frac{P}{P_y}\right)^{2.2} + \frac{3.2}{(1 + \frac{P}{P_y})} \leq 20.0,
\]

\[
0.3 \leq R_f \leq 0.5, \gamma^* \geq 1.0, 0.0 \leq P/P_y \leq 1.0 \quad (6.12)
\]

The comparison is illustrated in Figure 6.9. It is seen that the results of the current study are on the conservative side and sufficiently agree with the previously proposed equation, which is an indication of the validity of the conducted analysis.

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Equation by Ge et al.</th>
<th>$P/P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>▲</td>
<td>---</td>
<td>0.2</td>
</tr>
<tr>
<td>★</td>
<td>---</td>
<td>0.4</td>
</tr>
<tr>
<td>△</td>
<td>---</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Figure 6.9. Comparison with the existing equation
6.4 Design Formulae

6.4.1 Generation of formulae

Similar approach with the short steel cylinders is taken to establish formulae that can consider the influence of axial force fluctuation. Therefore, correction functions are generated that will modify the estimates by the constant axial case formulae. The presence of correction functions together with the constant axial case formulae will give the flexibility to the designer whether to consider the influence of axial force fluctuation into account or not.

The correction functions are generated by approximating the tendency of the relationship of failure strain improvement ratio with respect to the final axial force magnitude \((P_f/P_y)\) and axial force fluctuation amount \((\alpha)\). The model type parameters are not taken into account since they are found to be ineffective on the improvement ratio. Regression analyses are carried out to fit the relationship, and the lines expressed with the following functions (6.13-6.14) given for the two limit states of \(M_{95}\) and \(M_{90}\), respectively, are obtained.

\[ M_{95} \]

\[
F(\alpha, P_f / P_y) = (0.139\alpha + 0.066)P_f / P_y - 0.011\alpha + 0.985 \\
0.40 \leq R_f \leq 0.65, \gamma / \gamma^* \geq 1.0, 0.2 \leq P_f / P_y \leq 0.6, 1.5 \leq \alpha \leq 3
\] (6.13)

\[ M_{90} \]

\[
F(\alpha, P_f / P_y) = (0.167\alpha + 0.183)P_f / P_y + 0.004\alpha + 0.964 \\
0.40 \leq R_f \leq 0.65, \gamma / \gamma^* \geq 1.0, 0.2 \leq P_f / P_y \leq 0.6, 1.5 \leq \alpha \leq 3
\] (6.14)

Figure 6.10 illustrates the correction functions together with the improvement ratio obtained from the analysis results for all of the considered axial force fluctuation patterns. It can be seen that the lines expressed by the correction functions fit the analysis results in a reasonable accuracy. In most cases, the lines also follow lower boundary of the analysis results which will make it possible to achieve conservative estimations of the improvement ratio.
Figure 6.10. Approximation of the improvement ratio with the correction functions
By using the correction functions, it is possible to estimate the improvement ratio of the failure strain for a given axial force fluctuation pattern. The failure strain can be obtained by multiplying this value with the estimate of the corresponding constant axial case formulae. Equation (6.12) proposed by Ge et al. can be utilized for the $M_{95}$ limit state. However, there is no constant axial case formula for the $M_{90}$ limit state. A similar formula is generated for $M_{90}$ limit state by using the analysis results of the constant axial force cases of $0.2P_y$, $0.4P_y$, $0.6P_y$ and shown below by equation (6.15). The formula is expressed as a function of $R_f \lambda_S^{0.18}$ since the failure strain is found to be sensitive to this index also for the $M_{90}$ limit state.

$$\frac{\varepsilon_u}{\varepsilon_y} = \frac{1}{(1.24P/P_y + 0.08)R_f \lambda_S^{0.18} - 0.21P/P_y + 0.02} \leq 20$$

$$0.40 \leq R_f \leq 0.65, \frac{\gamma}{\gamma^*} \geq 1.0, 0.2 \leq P/P_y \leq 0.6$$

The analysis results are plotted together with the calculated values by the formula in Figure 6.11. It is seen that fairly accurate and conservative estimates are obtained for the considered axial force magnitudes.

**Figure 6.11.** Comparison of the analysis results with the proposed equation of constant axial force case ($M_{90}$)
6.4.2 Estimation of ductility using proposed formulae

The failure strain of box sections of the arch bridges or portal frame columns can be estimated by using the above proposed formulae. Estimation is conducted by following completely the same steps explained for the short cylinders in Chapter 5. (See Figure 5.17)

6.4.3 Application range of the estimation

The range of the loading conditions for which the proposed formulae are applicable is limited between the minimum and the maximum values of the studied axial force fluctuation parameters \(0.2 \leq P/P_y \leq 0.6, 1.5 \leq \alpha \leq 3\). The accuracy of the estimation within this range is illustrated in Figure 6.12 by plotting the estimates against the analysis results for the two limit states. Most of the analysis results are on the conservative side with an error of less than 30% indicating that a safety margin would still remain even when the axial force fluctuation is considered.

Similar to the formulae proposed in Chapter 5, the formulae for the box sections are considered to be valid also for the other types of steel since the formulae are generated as a function of yield strain which normalizes the failure strain. Also the proposed formulae will result in conservative side estimation for most types of steel used in Japan because of the characteristics of the stress-stain relationship employed for the generation of the formulae.
6.4.4 Efficiency of the proposed formulae

The proposed formulae will result in design with a smaller section for a given ductility demand. Table 6.2 contains limit values of $R_f \overline{\lambda_s}^{0.18}$ index for certain required ductility for the two limit states. For a given ductility demand, the limit values of $R_f \overline{\lambda_s}^{0.18}$ can be obtained from this table which makes it possible for the designer to find the most appropriate combination of the stiffened panel and the stiffener plate stiffness. For example, when $\varepsilon_u / \varepsilon_y$ is required to be more than 10, for the $M_{95}$ limit value $R_f \overline{\lambda_s}^{0.18}$ is 0.283 for $\alpha = 3$ and $P_f = 0.2P_y$ which is a larger value compared to $\alpha = 1$ case. It is seen that the increase is larger for the $M_{90}$ limit state and becomes more significant for both the larger axial force fluctuation amounts and higher final axial force magnitudes.
**Table 6.2. Limit values of $R_y \lambda_s^{0.18}$ for required ductility**

<table>
<thead>
<tr>
<th></th>
<th>$P_f=0.1P_y$</th>
<th>$P_f=0.2P_y$</th>
<th>$P_f=0.3P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha=1$</td>
<td>$\epsilon_u/\epsilon_y$</td>
<td>$\alpha=1.5$</td>
<td>$\alpha=2$</td>
</tr>
<tr>
<td>$\epsilon_u/\epsilon_y=5$</td>
<td>0.327 0.354 0.362 0.379</td>
<td>0.380 0.403 0.410 0.425</td>
<td>0.471 0.482 0.487 0.495</td>
</tr>
<tr>
<td>$\epsilon_u/\epsilon_y=6$</td>
<td>0.298 0.317 0.323 0.334</td>
<td>0.337 0.353 0.357 0.367</td>
<td>0.401 0.408 0.411 0.416</td>
</tr>
<tr>
<td>$\epsilon_u/\epsilon_y=8$</td>
<td>0.266 0.279 0.282 0.289</td>
<td>0.293 0.302 0.305 0.311</td>
<td>0.334 0.338 0.340 0.343</td>
</tr>
<tr>
<td>$\epsilon_u/\epsilon_y=10$</td>
<td>0.249 0.258 0.261 0.266</td>
<td>0.269 0.276 0.278 0.283</td>
<td>0.301 0.304 0.305 0.307</td>
</tr>
<tr>
<td>$\epsilon_u/\epsilon_y=20$</td>
<td>0.217 0.221 0.223 0.225</td>
<td>0.227 0.230 0.231 0.233</td>
<td>0.242 0.244 0.244 0.245</td>
</tr>
</tbody>
</table>

**6.5 Summary**

The influence of axial force fluctuation on the bending behavior of short steel boxes is studied in this chapter. Design formulae for the estimation of failure strain are proposed and their validity is verified through numerical analysis. The limit values of design parameters for certain required ductility are listed. The main findings of this chapter are summarized below:

1) Axial force fluctuation has an increasing effect on the post-peak ductility similar to the short steel cylinders.

2) The consideration of axial force fluctuation has no influence on the moment capacity.

3) The failure strain of box sections of arch ribs in arch bridges and portal frame columns can be estimated with the proposed formulae.

4) The consideration of axial force fluctuation through the proposed formulae will contribute to the implementation of a rational seismic design.
CHAPTER 7.

PROPOSED SEISMIC DESING METHOD AND CONCLUDING REMARKS
The basic steps, how to conduct the seismic design of upper-deck steel arch bridges through the application of the proposed demand and capacity prediction methods are given in this chapter together with the summary of the main findings of each chapter.

7.1 Outline of the proposed methodology

The proposed method is a ductility based procedure in which the strain demand and failure strain of individual bridge components are compared in order to carry out the seismic design. This method can be implemented for both design of new upper-deck steel arch bridges and retrofitting of existing ones composed of thin-walled members. The basic steps of the method are illustrated in Figure 7.1 and explained below.

1) Establish the analytical model of the upper-deck steel arch bridge based on the general layout and loading condition of the structure by using beam elements, which facilitates the FE modeling by considering the material non-linearity but does not account for local buckling.

2) Estimate the maximum inelastic response against a given Level 2 ground motion by the methods given in Chapter 3 and Chapter 4 for transverse and longitudinal excitations, respectively, which utilize pushover analysis, response spectrum method and the equal-energy assumption with the proposed correction functions. It should be noted that the suitable load pattern has to be determined depending on the direction of the considered ground motion excitation. Then, obtain the structural demand from the pushover analysis result corresponding to the maximum response estimated.

3) Estimate the ductile capacity of the individual thin-walled bridge component with either box or pipe cross sections by using empirical ductility equations given in Chapter 5 and Chapter 6. The parameters related to axial force in the constant axial force case formulae and in the correction functions accounting for the influence of axial force fluctuation should be gathered from the pushover analysis conducted for the estimation of demand in step 2).

4) Verify the seismic design on individual bridge components through the failure criterion defined as

\[ D_s = \frac{\varepsilon_u}{\varepsilon_a} \]  

When \( D_s \) is more than or equal to 1.0, the structure is considered to reach its ultimate limit state. Here, \( \varepsilon_a \) represents the strain demand obtained from the pushover analysis at
Step-2 which is the average strain of the meridional fiber with the maximum compression deformation (for pipe sections) or the outmost edge of the compressive flange (for box sections) over a certain effective failure length. The effective length is given by,

\[ L_e = \left[ \frac{0.585}{R_t^{0.08}} - 0.580 \right] \times D \quad (7.2) \]

for pipe sections \((R_t = \text{Radius thickness ratio}, D = \text{diameter of the section})\) in accordance with the capacity evaluations of short pipes studied in Chapter 5 and,

\[ L_e = 0.7 \times B \quad (7.3) \]

for box sections \((B = \text{flange width})\) in accordance with the capacity evaluations of short box columns in Chapter 6.

\(\varepsilon_u\) is the failure strain obtained in step 3.
Figure 7.1. Proposed seismic design method
7.2 Concluding Remarks

A static analysis-based seismic design method which is based on demand and capacity comparison is proposed in this dissertation for the simplification of the seismic design of upper-deck steel arch bridges. In the method, seismic demand is estimated without the need of dynamic response analysis, and ductile capacity of the bridge components with thin-walled sections are predicted by using practical design formulae which can also consider the influence of axial force fluctuation. Because of the error range of the demand estimation method and the current Japanese Seismic Design Code for Highway Bridges making the dynamic response analysis mandatory, it is considered that the proposed seismic design method can serve as a useful design method in the preliminary design considerations of the upper-deck steel arch bridges.

The method of estimation for the maximum seismic response is established based on the numerical analysis results of parametric upper-deck steel arch bridge models. The equal-energy assumption is applied on the results of pushover analysis and response spectrum method to predict the inelastic response at the reference points where the maximum structural response is observed in the case of transverse and longitudinal Level 2 ground motion excitations. Certain correction functions are proposed in order to improve the estimation accuracy of the equal-energy assumption. Having improved the estimates of the maximum structural response, seismic demand of the whole system can be obtained from the pushover analysis results corresponding to the estimated maximum response at the reference point.

The design formulae to be used for the capacity evaluations are generated by studying the ductility of parametric short steel cylinders and stiffened short box columns under combined compression and bending. Influence of axial force fluctuation is assessed by comparing the bending behavior of constant and fluctuating axial force cases. Design formulae to obtain the failure strain for different limit states are proposed that can consider the influence of axial force fluctuation.

The main findings obtained during the generation of the demand and capacity estimation procedures are summarized as follows.

Conclusions corresponding to demand estimation:

1) The equal-energy assumption results in conservative estimates both for out-of-plane and in-plane response estimations. However, the results may be too
conservative in many cases

2) The ground condition type and the structural parameters (ratio of arch rise to span and arch rib spacing) have no significant influence on the applicability of the equal-energy assumption regardless the direction of the ground motion excitation.

3) The prediction accuracy of the equal-energy assumption can be improved by using the proposed correction functions. The correction functions are applicable for both out-of-plane and in-plane response estimations.

4) The proposed method of predicting maximum inelastic response can be successfully applied to upper-deck steel arch bridges. Only the force pattern of the pushover analysis needs to be modified to adjust the method to the direction of the input ground motion. A modal force pattern should be used for the out-of-plane response estimations and a longitudinal displacement force, placed in the mid-point of stiffening girder should be used for in-plane response estimations.

Conclusions corresponding to capacity estimation

1) Ductility and strength corresponding to post-peak behavior are improved when axial force fluctuation is considered. The improvement is valid both for pipe and box sections of thin-walled bridge members.

2) The improvement in ductility is greater for higher axial force magnitudes and for larger axial force fluctuations.

3) The proposed formulae can be used to determine the ductile capacity of pipe and box sections of the arch ribs in arch bridges as well as the columns in portal frames which are subjected to bending as well as axial force fluctuation during earthquakes.

4) The consideration of the axial force fluctuations using the proposed formulae will result in the use of higher ductile capacities as design values compared with the conventional practice, making the seismic design more rational.

The investigations in this study rely on numerical analysis both for the demand and capacity estimations. Although exact applicable limit of the capacity estimations are indicated, the applicable range of the seismic demand estimation method is not defined clearly. For the estimation of seismic demand, numerical studies of six bridges are conducted which are representative in terms of the structural composition covering the realistic application range of upper-deck steel arch bridges. It is considered that the
demand estimation method will give satisfactory results if the dynamic characteristics of
the bridge being designed are similar to the ones studied in this dissertation. That is, if
the natural frequency is within the studied range and the modal composition is similar in
that it has one dominant out-of-plane vibration mode. Although natural frequency may
be considered as an index of general structural stiffness, differences in the stiffness
configuration of local members as well as different boundary conditions may result in
different plasticization behavior of the structure, causing the accuracy of the proposed
method to be reduced. Extra work is necessary to elucidate the specific limits of
applicable range of the demand estimation method. However, it is considered that the
proposed method may prove useful as a preliminary design method for a certain range
of upper-deck steel arch bridges.

In the future, the proposed seismic design method will be applied to existing
upper-deck steel arch bridges and will be evaluated for its accuracy and efficiency
compared to the conventional seismic design method. Also further parametric studies
will be conducted with the short box columns to investigate if the proposed ductility
formulae can be valid for a broader range of loading conditions.
REFERENCES


[71] California Department of Transportation (Caltrans). San Francisco-Oakland Bay Bridge west spans seismic retrofit design criteria. Sacramento (CA); 1997.


Figure A.1. Model 2
Figure A.2. Model 3
Three-Dimensional View

Figure A.3. Model 4

Box Section for Arch Ribs

Box-Section for Side Piers

I-section for Stiffening Girders

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Figure A.4. Model 5

Elevation

Plan

Three-Dimensional View

Box Section for Arch Ribs

Box-Section for Side piers

I-Section for Stiffening Girders
Figure A.5. Model 6